Appendix IS-2

Geotechnical Engineering Investigations

Appendix IS-2.1

Geotechnical Engineering Investigation—North Block

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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

November 17, 2022
Revised November 2, 2023
File Number 22307

Bardas Investment Group
1015 N. Fairfax Avenue
West Hollywood, California 90046

Attention: Alex King

Subject: Geotechnical Engineering Investigation
Proposed 6311 Romaine Street Project – North Block
6400 – 6416 Santa Monica Boulevard, 1015 – 1045 N. Cahuenga Boulevard,
1006 – 1048 N. Cole Avenue, and 6311 W. Romaine Street,
Los Angeles, California

Dear Mr. King:

This letter transmits the Geotechnical Engineering Investigation for the property known as the "North Block", 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.

STANLEY SIU-KEUNG TANG
R.C.E. 56178
EXP. 12/31/24

STANLEY S. TANG
R.C.E. 56178

SST:km

Email to: [aking@bardasig.com]

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED 6311 ROMAINE STREET PROJECT - NORTH BLOCK
6400 – 6416 SANTA MONICA BOULEVARD,
1015 – 1045 N. CAHUENGA BOULEVARD,
1006 – 1048 N. COLE AVENUE, AND 6311 ROMAINE STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed for the property, known as the "North Block." The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the North Block, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included excavation of seven exploratory borings for the North Block, supplemented by two prior geotechnical borings previously performed for the North Block by The J. Byer Group (JBG), as discussed under the Research section of this report. Additionally, this investigation included collection of representative soil samples from the recent exploratory borings, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan in Appendix I. The exploration logs are presented in Appendix II, and the laboratory testing are presented in Appendix III of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. On the North Block, the Project proposes to renovate six existing structures and to construct a 6-story commercial building. Vehicular parking spaces would be provided on-site in a one-level subterranean garage below the new 6-story commercial building. The majority of the proposed subterranean parking garage will extend on the order of 13 to 15 feet below ground surface (bgs), and miscellaneous sump pits and elevator pits may extend between 18 and 20 feet bgs on the North Block.

Column loads are estimated to be between 600 and 1,000 kips. Wall loads are estimated to be between 6 and 12 kips per lineal foot. Grading will consist of excavations up to approximately 25 feet in depth bgs for the subterranean structure and foundation elements, and up to approximately 5 feet bgs for new conventional foundations may be anticipated as part of the proposed building renovations.

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

SITE CONDITIONS

The North Block is located at 6400 – 6416 Santa Monica Boulevard, 1015 – 1045 N. Cahuenga Boulevard, 1006 – 1048 N. Cole Avenue, and 6311 W. Romaine Street, in the City of Los Angeles, California. The site is bounded by Santa Monica Boulevard to the north, by North Cahuenga Boulevard to the east, by Romaine Street to the south, and by Cole Avenue to the west. The North Block Site occupies approximately the entire city block, with the exception of the parcel at the northwest corner of the city block, which is not part of the property. The site is currently developed with multiple 1 to 4-story commercial structures and at-grade parking lots.

The site slopes downward gently to the south. According to the topographic survey prepared by JRN Civil Engineers, dated March 4, 2021, the high site elevation is at 302.94 feet AMSL located at the northeast corner of the site, and the low site elevation is at 292.76 feet AMSL located at the southwest corner of the site. This corresponds to an approximate 10 feet of elevation change across the site. Drainage across the site is by sheetflow to the city streets. The vegetation on the site consists of isolated trees and shrubs within planter areas. The neighboring development consists primarily of commercial development.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

Seven exploratory borings were excavated as part of the geotechnical investigation by this firm, between September 19, 2022, and October 1, 2022, for the North Block. The explorations were supplemented by two prior geotechnical borings (Boring Number 8 and 12) previously performed for the North Block by The J. Byer Group (JBG), as discussed under the Research section of this report.

The exploratory borings performed by this firm varied between 30 and 90 feet in depth bgs. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan (Plate VI in Appendix I), and the geologic materials encountered are logged on Plates A-1 through A-7, presented in Appendix II. The prior boring logs relevant to the North Block by JBG are also presented in Appendix II for reference.

The locations of the elevation of the top of the exploratory borings were determined based on interpolation from the Topographic Survey prepared by JRN Civil Engineers. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials underlying the subject site consist of silty sands, sandy silts, and silty clays, which are dark brown to black in color, moist, medium dense to stiff, fine grained. Fill thickness on the order of 3 to 5 feet was encountered in the explorations.

Native soils consist of alluvial deposits consisting primarily of silty clays, sandy to clayey silts, silty to clayey sands, and sands. The native soils are yellowish brown to reddish brown, dark brown, grayish brown, gray to dark gray, and black in color, moist to wet, stiff to very stiff, medium dense to very dense, fine to coarse grained, with variable amounts of gravel. The native soils consist predominantly of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from individual boring logs.

Groundwater

Groundwater was encountered between 12.3 and 22 feet below the existing ground surface, corresponding to elevations 285.5 to 277.0 feet above mean sea level (AMSL). Due to the stratified layers of sands and clays underlying the site, it is likely that the encountered groundwater in the upper zone consists of a confined, perched groundwater layer.

Based on review of the Historically Highest Groundwater Levels Map presented in the Seismic Hazards Zones Report (CGS SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface.[a] A copy of the Historically Highest Groundwater Levels Map is presented on Plate IV in Appendix I of this report. However, since the result of site explorations indicates that the encountered groundwater

---

[a] CGS SHZR 026:
https://planning.lacity.org/eir/conventioncntr/DEIR/files/references/California%20Division%20of%20Mines%20and%20Geology,%20%20Hollywood%20Quadrangle,%201998.pdf

level is higher, it is recommended that the high groundwater elevation of 285.5 feet AMSL encountered during exploration be utilized for the project design purposes.

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

Research

Available geotechnical reports for the site were reviewed during the preparation of this investigation. Specifically reviewed is the following report prepared by The J. Byer Group, Inc. (JBG). A copy of this report is presented in Appendix IV for reference.

- Geotechnical Exploration, Proposed Parking Structure, Commercial Buildings, and Additions, 6311 Romaine Street, Hollywood, California, Project No. JB 18051-B, dated May 13, 1999.

A total of twelve exploratory borings were excavated by JBG, extending to depths between 15 to 50 feet below the site grade for both the North Block and the South Block. Two of the JBG borings (Boring Number 8 and 12) are located in the North Block. Between 1½ and 2 feet of existing fill materials were encountered at the North Block by JBG during exploration. The fill is underlain by firm natural alluvial deposits. Groundwater was not encountered in Boring Number 8 and 12, since these two borings were excavated only to a depth of 15 feet bgs.

The exploratory borings by JBG (Boring Number 8 and 12), which are relevant to the North Block, are plotted on the enclosed Plot Plan presented in Appendix I. The boring logs of these relevant boreholes are also presented in Appendix II of this report for reference.

This firm has reviewed the referenced document by JBG, and concurs with the findings provided therein. All JBG borings extended into the underlying native soils similar to those encountered during explorations performed by this firm. The recommendations contained herein shall supersede those previously provided by JBG for the planned development. This firm accepts the prior findings by JBG and the professional responsibility for the project as the geotechnical engineer of record.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The property 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.[b]

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,700 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.[c]

---

[b] CGS Note 36: https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-36.pdf

[c] CGS Special Publication 42: https://www.conservation.ca.gov/cgs/documents/publications/special-publications/SP_042.pdf

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.[d]

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines “active” and “potentially active” faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 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.[e]

---

[d] CGS Note 31: https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-31.pdf
[e] CGS Special Publication 42: https://www.conservation.ca.gov/cgs/documents/publications/special-publications/SP_042.pdf

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

Ground 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 (Zimas and NavigateLA).[g] 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 Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially "Liquefiable" area (Plate III of Appendix I). This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.[h]

---

[f] Ibid
[g] Zimas website: https://zimas.lacity.org/ and NavigateLA website: https://navigatela.lacity.org/navigatela/
[h] CGS SHZR 026:
https://planning.lacity.org/eir/conventioncntr/DEIR/files/references/California%20Division%20of%20Mines%20and%20Geology,%20%20Hollywood%20Quadrangle,%201998.pdf

A site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered between 12.3 and 22 feet below the existing ground surface, corresponding to 285.5 to 277.0 feet above mean sea level (AMSL). Based on review of the Seismic Hazards Zones Report (CGS SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface.[i] However, since the result of site exploration indicates that the encountered groundwater level is higher than the historically highest groundwater level, a high groundwater level of 12.3 feet below ground surface was conservatively utilized for the enclosed liquefaction analysis.

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A Site Class "D" (Stiff Soil Profile) and a published shear wave velocity of 259 meters per second were utilized for Vs30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.77 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.984g was obtained using the ASCE Hazard Tool website (https://asce7hazardtool.online/). These parameters are used in the enclosed liquefaction analyses (Appendix III).

The liquefaction analysis, entitled "Empirical Estimation of Liquefaction Potential," is based on Boring 3. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing

---

[i] Ibid

a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory boring are presented on the enclosed E and F Plates (Appendix III). 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. Furthermore, cohesive soils with PI between 7 and 12 and moisture content greater than 85 percent of the liquid limit are also susceptible to liquefaction.[j]

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

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

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

---

[j] CGS Special Publication 117A: https://www.conservation.ca.gov/cgs/Documents/Publications/Special-Publications/SP_117a.pdf

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.[k]

The enclosed liquefaction analysis included in the Appendix III of this report, indicates that site soils would not be prone to liquefaction during 2,475-year return period ground motion. Therefore, lateral spreading is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. The North Block is located approximately 11 miles from the Pacific Ocean. 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.[l]

---

[k] Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE Journal of Geotechnical Engineering, Vol. 128, No. 12, December: https://ascelibrary.org/doi/abs/10.1061/%28ASCE%291090-0241%282002%29128%3A12%281007%29

[l] County of Los Angeles General Plan Plates 1-8: https://planning.lacounty.gov/assets/upl/project/gp_web80-tech-plates-01-to-08.pdf

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately adjacent to the property. The Hollywood Reservoir/Mulholland Dam is located approximately 2 miles north of the North Block.

According to the County of Los Angeles General Plan, the site is within the potential inundation boundary of Hollywood Reservoir/Mulholland Dam.[m] The Hollywood Reservoir/Mulholland Dam as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake for the site as well as other conditions that could undermine the integrity of the dam. Pursuant to these regulations, the Hollywood Reservoir/Mulholland Dam is regularly inspected and meets current safety regulations. In addition, the LADWP has emergency response plans to address any potential impacts to its dams. Therefore, the risk of flooding from a seismically-induced seiche or dam failure is considered to be remote.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

---

[m] Ibid

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

Approximately 1½ to 5 feet of existing fill was encountered in the explorations by this firm and by JBG at the North Block. The site is underlain by alluvial deposits consisting primarily of silty clays, sandy to clayey silts, silty to clayey sands, and sands.

Groundwater was encountered between 12.3 and 22 feet below the exiting ground surface, corresponding to 285.5 to 277.0 feet above mean sea level (AMSL). Due to the stratified layers of sands and clays underlying the site, it is likely that the encountered groundwater in the upper zone consists of a confined, perched groundwater layer. According to the Seismic Hazards Zones Report (CGS, SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface. However, since the result of site explorations indicates that the encountered groundwater level is higher, it is recommended that the high groundwater elevation of 285.5 feet AMSL encountered during exploration be utilized for the project design purposes.

On the North Block, the Project proposes to renovate six existing structures and to construct a 6-story commercial building. Vehicular parking spaces would be provided on-site in a one-level subterranean garage below the new 6-story commercial building. The majority of the proposed subterranean parking garage will extend on the order of 13 to 15 feet bgs, and miscellaneous sump pits and elevator pits may extend between 18 and 20 feet bgs on the North Block. It is anticipated that excavation up to 25 feet bgs will be required for the proposed subterranean level and foundation elements.

Excavation for the proposed structure serviced by subterranean level will remove fill materials and expose the underlying firm native soils. Due to the high groundwater level, it is recommended that the subterranean structure be designed for hydrostatic pressure and a mat foundation should be utilized for support of the proposed structure.

The basement walls shall be designed for the soil and hydrostatic pressures based on the existing ground surface. In addition, the proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater elevation of 285.5 feet AMSL. The proposed uplift pressure to be used in the foundation design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the historically highest groundwater level.

Excavation of the proposed subterranean level will require shoring and temporary dewatering measures to provide a stable and dry excavation due to the depth of excavation, the presence of groundwater, and the proximity of public right of ways. Pumping of the high-moisture content soils at the bottom of the excavation is anticipated to occur during operation of heavy equipment. Recommendations for stabilizing the wet subgrade is provided in the Wet Soils section of this report.

The existing fill materials are not suitable for support of new foundations, floor slabs or additional fill. Where new foundations are required as part of the existing building renovations, new conventional foundations may bear in the underlying native soils and/or properly compacted fill. All existing fill materials and upper native soils shall be completely removed and recompacted to a minimum depth of 5 feet below the proposed grade, or 3 feet below the bottom of the proposed foundations, whichever is greater. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundation, whichever is greater.

Where the horizontal overexcavation cannot be achieved, the proposed footings may be deepened to bear into the underlying native soils, encountered at or below a depth of 5 feet. The deepened portion of the foundation may be backfilled with "hard rock" concrete having the same strength as the proposed structural footings. The concrete is denser than the surrounding soils and will transfer the structural loading into the underlying native soils.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which will not be tied-in to the proposed structures and are to be constructed immediately adjacent to property lines or adjacent structures, such that the recommended horizontal overexcavation and recompaction cannot be achieved, should be deepened to bear in the dense native soils.

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

SEISMIC DESIGN CONSIDERATIONS

2022 CBC / 2023 LABC Seismic Parameters

Based on information derived from the subsurface investigation, the North Block site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the ASCE 7 Hazard Tool website (https://asce7hazardtool.online/) to calculate ground motion parameters for the site, in accordance with the 2022 California Building Code (CBC) and 2023 Los Angeles Building Code (LABC).

2022 CBC / 2023 LABC SEISMIC PARAMETERS	
California Building Code	2022
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.088g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.088g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.392g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.746g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.268g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.845g*

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

FILL SOILS

Approximately 1½ to 5 feet of existing fill was encountered in the explorations by this firm and by JBG at the North Block. This material and any fill generated during demolition within the area of the proposed structure serviced by subterranean level, should be removed during excavation of the subterranean level and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the low to high expansion range. The Expansion Index was found to be 20, 102 and 110 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs-On-Grade" sections of this report.

SUBSIDENCE

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. No large-scale extraction of gas, oil, or geothermal energy currently occurs or is planned at the South Block. Additionally, the proposed structure will be designed to resist hydrostatic pressure, and therefore, no permanent dewatering will be required for the Project.

Temporary construction dewatering will be necessary in order to construct the proposed subterranean structure. The underlying native soils consisting of alluvial deposits comprising primarily of firm to very stiff clays with occasional layers of dense to very dense silty and clayey sands were encountered during explorations. These native soils are typical to this area of Los Angeles County. Additional field explorations and pump tests will be performed at the Project Site to evaluate the groundwater conditions, the proposed temporary dewatering approaches and methods, and subsidence impact (if any) due to construction dewatering. The final dewatering system methods and shoring design, which are subject to regulatory control for safety and subsidence, will be submitted to LADBS for review and approval as part of the building permit processes prior to construction.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomena in which the underlying soils collapse when wetted. Hydroconsolidation could potentially result in significant foundation movements, over a long period of time of wetting.

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

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed by HDR, Inc. indicate that the electrical resistivities of the soil was in the mildly corrosive and moderately corrosive categories with as-received moisture, and in the moderately corrosive and corrosive categories when saturated. Soil pH values varied from 6.3 to 7.1, indicating a slightly acidic to neutral condition and do not particularly increase soil corrosivity. The soluble salt content was low. The nitrate and sulfate concentrations were low. Ammonium was not detected in the soil samples.

In summary, the soil is classified as corrosive to ferrous metals. Sulfate exposure is considered to be negligible, and therefore, there are no restrictions on the type of cement to be utilized for concrete in contact with the underlying soils. Detailed results, discussion of results and recommended mitigating measures are provided within the report by HDR, Inc. presented herein.

METHANE ZONES

Based on review of the Navigate LA (http://navigatela.lacity.org/NavigateLA/) website, maintained by the City of Los Angeles, the North Block is not located within a Methane Buffer Zone as designated by the City.[n] A copy of the portion of the Methane Zone Risk map covering the North Block is presented on Plate V in Appendix I of this report.

GRADING GUIDELINES

The following grading guidelines may be followed for the preparation of the compacted fill pad recommended for support of new foundation which may be required as part of the building renovations, and for any other miscellaneous compaction that may be required, such as retaining wall or trench backfill, or subgrade preparation.

Site Preparation

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

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

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

---

[n] NavigateLA website: https://navigatela.lacity.org/navigatela/

- 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 and Blending

In the area of the proposed new footings as part of the building renovations, all existing fill materials and upper native soils shall be completely removed and recompacted to a minimum depth of 5 feet below the proposed grade, or 3 feet below the bottom of the proposed foundations, whichever is greater. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundation, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate, and the grading operation proceeds efficiently.

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

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum 90 percent of the maximum density, except for cohesionless soils having less than 15 percent finer than 0.005

millimeters, which shall be compacted to a minimum 95 percent of the maximum density in accordance with the most recent revision of the Los Angeles Building Code.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

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

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 90. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Wet Soils

At the time of exploration, the soils which will be exposed at the bottom of the excavation were above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane will require significant drying and aeration prior to recompaction.

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

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

The proposed structure located at the north end of the site, which will be constructed over 1 subterranean level extending below the historically highest groundwater level, may be supported on a mat foundation system bearing in the underlying native soils. In addition to the structural loading, the mat foundation shall be designed for hydrostatic uplift pressure acting on the base of the foundation, based on the recommended historically highest groundwater level of 285.5 feet AMSL.

Where new foundations are required as part of the existing building renovations, new conventional foundations may bear in the underlying native soils and/or properly compacted fill.

Conventional Foundations

Where new foundations are required as part of the existing building renovations, new conventional foundations may bear in the underlying native soils and/or properly compacted fill. Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 350 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

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

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

Conventional Foundation Settlement

Settlement of the conventional foundation system is expected to occur on initial application of loading. The maximum settlement is not expected to exceed 1-inch and would occur below the heaviest loaded elements. Differential settlement is not expected to exceed ½-inch in 30 feet.

Mat Foundation

It is recommended that the proposed structure at the north end of the site be supported on a mat foundation bearing in the underlying native soils. Excavations up to approximately 25 feet are anticipated for the subterranean parking level and foundation elements. Preliminarily, it is anticipated that the average bearing pressure will be on the order of 4,000 to 5,000 pounds per square foot. Foundation bearing pressure will vary across the mat footing, with higher concentrated loads up to 7,500 pounds per square foot, located below the central cores of the building.

Given the size of the proposed mat foundation, the average bearing pressure of 5,000 pounds per square foot is well below the allowable bearing pressures, with a factor of safety well exceeding 3. For design purposes, an average bearing pressure of 5,000 pounds per square foot, with locally higher pressures up to 7,500 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 150 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

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

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

Hydrostatic Considerations for Mat Foundations

The proposed subterranean structure will extend below the groundwater level, and therefore, the mat foundations shall be waterproofed and be designed to withstand the hydrostatic uplift pressure based on the high groundwater level.

As discussed in the "Groundwater" section of this report, the proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater elevation of 285.5 feet AMSL. The proposed uplift pressure to be used in the foundation design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the historically highest groundwater level. Where necessary, micropiles (tiedown anchors) may be utilized to provide uplift resistance in conjunction with the proposed mat foundation.

Mat Foundation Settlement

The majority of the mat foundation settlement is expected to occur on initial application of loading. It is anticipated that total settlement between 1.5 and 2 inches will occur below the more heavily loaded central core portions of the mat foundation beneath the building. Settlement on the edges of the mat foundation is expected to be 1 inch. Differential settlement is anticipated to be less than ¾-inch within 30 feet.

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 0.24 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Micropiles for Hydrostatic Uplift

The proposed subterranean structure will extend below the groundwater level, and therefore, the mat foundations shall be waterproofed and be designed to withstand the hydrostatic uplift pressure based on the high groundwater level.

Where necessary, an anchoring system consisting of micropiles may be designed to provide resistance against the anticipated hydrostatic uplift pressures acting at the bottom of the mat foundation. The proposed micropiles shall derive support from the underlying native alluvial soils, expected at the subterranean subgrade. It is recommended that a post-grouted micropile system be utilized for support of the potential hydrostatic tension loads. The micropiles shall be a minimum of 10 inches in diameter and shall have a minimum of 30 feet (bonded length) embedded into the underlying native alluvial soils. The reinforcing steel shall be corrosion protected. The micropiles shall only be utilized for tension support and shall not be utilized for support of any lateral loads.

An allowable upward frictional capacity of 7 kips per lineal foot for the bonded length may be utilized in the design of a 10-inch diameter post-grouted micropile. An allowable upward frictional capacity of 8 kips per lineal foot for the bonded length may be utilized in the design of a 12-inch diameter post-grouted micropile. A safety factor of 2 has been applied in determining the allowable frictional capacity. These allowable micropile design capacities shall be considered preliminary and are subject to verification or modification based on a Verification Test Pile Program as discussed below.

A 1/3 increase may be utilized for temporary loads, such as wind and seismic forces. Micropiles should be spaced at a minimum of 3 diameters or 30 inches on centers, whichever is greater. If so spaced, there will be no reduction in the downward capacity of the micropiles due to group action.

The City of Los Angeles requires a steel casing having a minimum thickness of 3/8-inch be installed for the top section of the micropile (the “unbonded zone”) to a depth of 120 percent of the point of zero curvature. The cased section of the micropile shall be considered as the unbounded zone and shall not be considered as contributing to friction.

Based on the enclosed RSPile Analysis (RocScience), depth to zero curvature for a 10-inch diameter micropile is approximately 12 feet for the free-head condition and approximately 14 feet for a fixed-head condition. Therefore, it is recommended that a steel casing be provided for a 10-inch diameter micropile for the upper 14½ and 17 feet for the free-head and fixed head conditions, respectively.

Similarly, depth to zero curvature for a 12-inch diameter micropile is approximately 14 feet for the free-head condition and approximately 16 feet for a fixed-head condition. Therefore, it is recommended that a steel casing be provided for a 12-inch diameter micropile for the upper 17 and 19 feet for the free-head and fixed head conditions, respectively.

Verification Micropile Test Program

A verification test pile program shall be performed in order to verify the design capacities, prior to installation of the production micropiles. Both compression and tension load tests shall be performed during verification test pile program. The verification test piles shall be sacrificial and shall not be utilized as part of the production piles. The number of verification test piles shall be a minimum of 2 test piles, or equivalent to a minimum of 1 percent of the production piles, whichever is greater.

The verification micropiles shall be tested to a minimum of 200 percent of the design load capacity. The load tests shall be performed in accordance with the latest version of ASTM D 3689/3689M, with at least one maintained load test. The testing reaction frame shall be sufficiently rigid such that excessive deformation of the testing equipment will not occur. The hydraulic jack, pressure gauges, and dial gauges shall be calibrated prior to performance of the load test. A copy of the calibration certifications shall be provided by the contractor to this firm prior to performance of the load test.

Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment. Pile top movement shall be recorded at the beginning and at the end of each test period.

The total vertical pile top movement during the verification test shall not exceed 1 inch at the design load (DL), and 2 inches at the maximum test load of 200 percent (2*DL). At the completion of the verification test, the test pile may be cut off at a minimum depth of 1 foot below the finished subgrade and abandoned in place.

If a verification tested micropile fails to meet the acceptance criteria, the contractor shall modify the design and/or the construction procedure. All modifications and changes shall be submitted to the Structural Engineer and the Geotechnical Engineer for review and approval.

Proof Load Tests

A minimum of 5 percent of the production piles shall be proof tested to a minimum test load of 160 percent of the design load. The proof test shall be made by incrementally loading the micropile in accordance with the ASTM D 3689/3689M. Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment. Pile top movement shall be recorded at the beginning and at the end of each test period. The total vertical pile top movement during the proof load test shall not exceed 1 inch at the design load.

Pile Integrity Testing

Pile integrity tests shall be performed for all verification test micropiles and reaction piles, as required by LADBS. Due to the slenderness and the anticipated lengths of the micropiles, it is recommended that Thermal Integrity Profiling (TIP) method be utilized for the pile integrity tests. TIP uses the heat generated during the concrete curing process in the foundation pile element, to evaluate the consistency of the concrete and the regularity of its shape. TIP could be used for evaluating the cross-sectional areas and the length of the pile. Due to the slenderness of the micropiles, it is recommended that TIP be performed using embedded thermal sensors, in accordance with Method B of the latest version of ASTM D7949.

Typically, LADBS requires pile integrity tests be performed on all test piles and reaction piles, and a minimum of 5 percent of the production micropiles. In addition, one of the test micropiles shall be exhumed for measurement of the pile diameter and physical examination of the pile integrity. However, in order to minimize disturbance of the underlying soils which will provide support of the proposed mat foundation below the subterranean structure, it is recommended that the requirement of exhumation of a test pile be eliminated, and the non-destructive pile integrity tests be performed on a minimum of 10 percent of the production micropiles.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structures may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing capacity of 1,500 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

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

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

Foundation Observations

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

RETAINING WALL DESIGN

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Due to the historically highest groundwater level for the North Block, it is recommended that the proposed subterranean walls be designed for full hydrostatic pressure based on the existing ground surface, and the code required wall subdrain system may be eliminated. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	**Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure with Hydrostatic Pressure (pcf)**	**Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure with Hydrostatic Pressure (pcf)**
20 feet	85 pcf	100 pcf

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures. For traffic surcharge, 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.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 28 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

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 2020-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

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

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone. As an alternative, the surcharge calculation method provided in the Naval Facilities Design Manual (NAVFAV 7.02) may be followed.

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtained by the most recent revision of ASTM D 1557. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 5 feet in vertical height will be required for the recommended recompaction and/or new footings as part of the building renovations, and up to 25 feet for the proposed subterranean level and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 25 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Temporary Dewatering

Groundwater was encountered at depths between 12.3 and 22 feet below the existing ground surface, corresponding to 285.5 to 277.0 feet above mean sea level (AMSL). Since the proposed subterranean level and foundation elements will extend below the current groundwater level, it is recommended that a qualified dewatering consultant should be retained during the design phase of the Project. The expected number and depth of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water. However, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected within the excavation through the use of French drains and sump pumps. The collected water should be pumped to an acceptable disposal area. The exposed subgrade is anticipated to be wet and pumping. Subgrade stabilization and wet soil treatment are provided in the "Wet Soils" section of this report.

Once the temporary dewatering system is discontinued, the groundwater level will likely return to the pre-development level. It is critical that the termination of temporary construction dewatering be coordinated with the project structural engineer to confirm that there is sufficient weight of the structure to resist the high groundwater level prior to discontinuation of dewatering.

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

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

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 0.25 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

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

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

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

Lagging

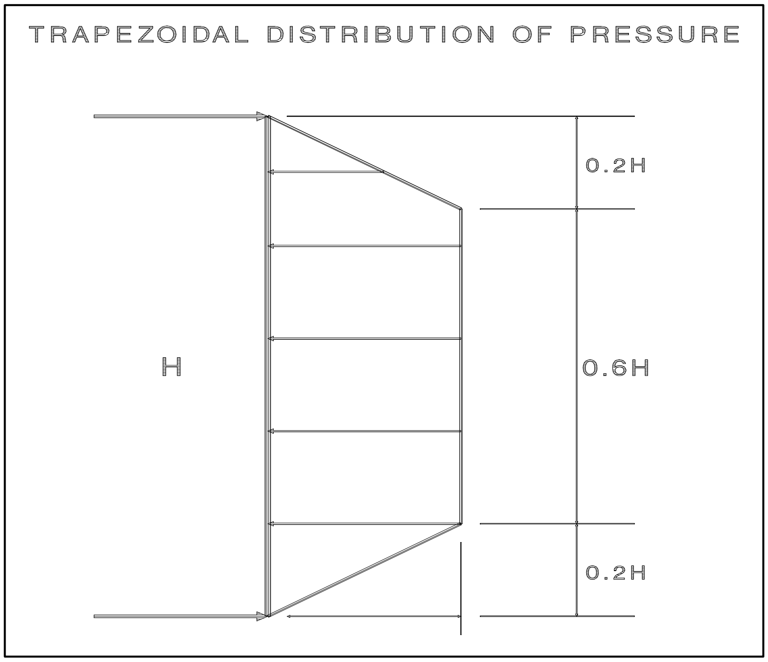
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but may be limited to a maximum of 400 pounds per square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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	**Cantilevered Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure**	**Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure**
25 feet	40 pcf	25H psf

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



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

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

Tied-Back Anchors

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

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

It is recommended that at least 3 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.

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 ensure 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

Where applicable, concrete slabs-on-grade shall be a minimum of 5 inches in thickness and shall be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness and shall be reinforced with a minimum of #3 steel bars on 12-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, where necessary, it is recommended that a qualified consultant should be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor on various components of the structure. Since the lowest subterranean level will extend below the historically highest groundwater level, the proposed subterranean structure and foundation shall be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for the proposed subterranean structure.

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

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

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	6	9

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness and shall be underlain by 6 inches of aggregate base. For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Groundwater was encountered between 12.3 and 22 feet below the existing ground surface. Due to the high groundwater level encountered at the North Block and the depth of the proposed subterranean level, stormwater infiltration will not be feasible for the project.

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 engineering purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

The 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. 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 general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. 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 D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10-pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Atterberg Limits

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

REFERENCES

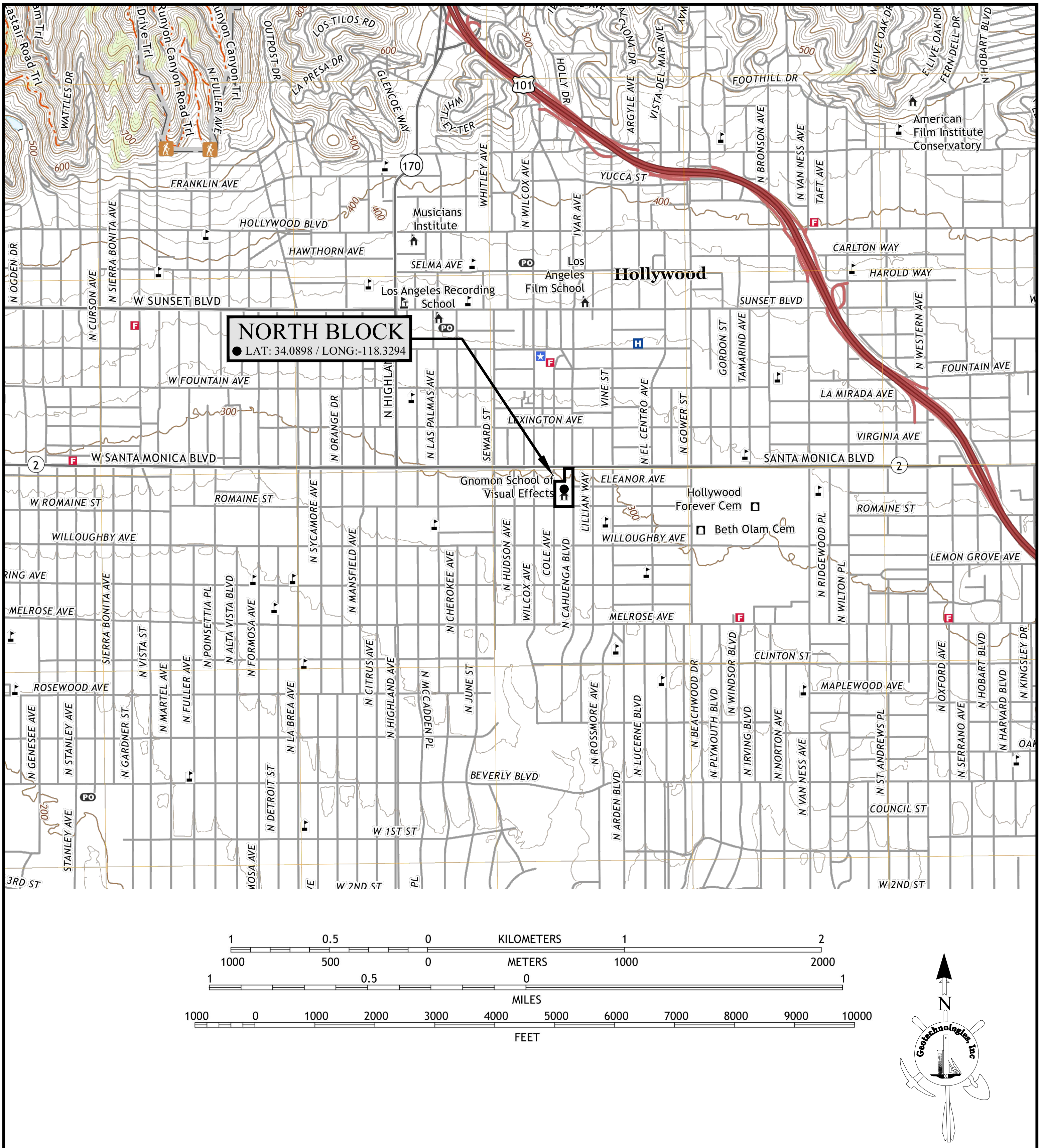
1. American Society of Civil Engineers, 1994, “Settlement Analysis,” Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 9.
2. Bartlett, S.F. and Youd, T.L., 1992, “Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads,” Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, SUNY-Buffalo, Buffalo, NY.
3. Bartlett, S.F. and Youd, T.L., 1995, “Empirical Prediction of Liquefaction-Induced lateral Spread,” Journal of Geotechnical Engineering, Vol. 121, No.4, April.
4. Bowles, Joseph E., 1977, “Foundation Analysis and Design,” 2nd Edition, McGraw-Hill, New York.
5. California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
6. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.
7. California Geological Survey, 2008, “Guidelines for Evaluation and Mitigation of Seismic Hazards in California,” CDMG Special Publication 117A.
8. City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
9. Crook, R., Jr., Proctor, R.J., 1992, The Hollywood and Santa Monica Fault and the Southern Boundary of the Transverse Ranges Province: in Pipkin, B., and Proctor, R.J. (eds.) Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.
10. Department of the Navy, NAVFAC Design Manual 7.1, 1982, “Soil Mechanics,” Naval Facilities Engineering Command, May.
11. Department of the Navy, NAVFAC Design Manual 7.02, 1986, “Foundations and Earth Structures,” Naval Facilities Engineering Command, September.
12. Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) 7.5-Minute Quadrangles, Map No DF-30, map scale 1: 24,000.
13. Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.
14. Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

REFERENCES - continued

15. Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.
16. Southern California Earthquake Center, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California," March.
17. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.
18. Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.
19. Tokimatsu, K. and Seed, H. B., 1987, "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August.
20. Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical Engineering, Vol. 128, No. 12, December.

APPENDIX I

MAPS AND PLOT PLAN



REFERENCE: USGS TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE 2022

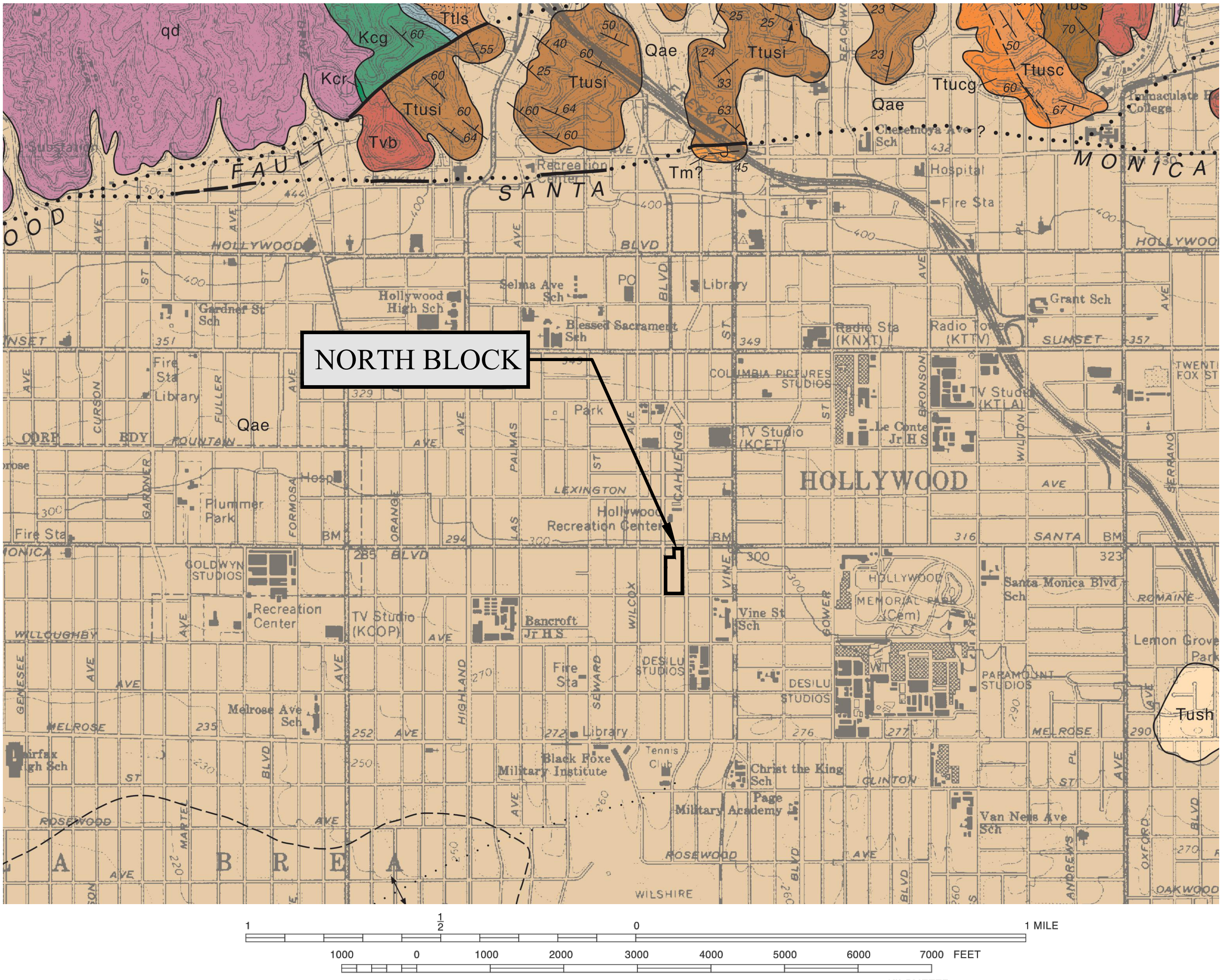
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

FILE NO: 22307

PLATE: I



LEGEND

Qae *Similar to* **Qa***, but slightly elevated and dissected; includes alluvial fan sediments*

Tush *Gray to light brown, thin-bedded silty clay shale, soft and crumbly; locally contains scattered hard calcareous nodules; in places contains laminae of fine grained soft sandstone*

Tm *White-weathering, thin bedded, platy siliceous shale, hard to semi-chalky; at Griffith Park directly overlies granodiorite basement rocks if not in fault (?) contact*

Ttusi *Mostly gray micaceous clay shale or claystone, crumbly where weathered, and thin interbeds of gray to tan semi-friable sandstone*

Ttusc *Light gray massive sandstone, with pebble-cobble conglomerate of detritus as in*

Ttucg *Cahuenga Conglomerate Member (of Dibblee 1989; includes "Cahuenga" and "Griffith" Beds of Neuerburg 1953); light to medium gray, crudely bedded; ranges from coarse pebbly sandstone to cobble-boulder conglomerate composed mostly of granitic detritus (granite to quartz diorite) and some of metavolcanic rocks, quartzite, gneiss, and basalt, in coarse weakly coherent sandstone matrix; grades and intertongues westward and southward into* **Ttusc** *and*

Tvb *Basaltic volcanic rocks: dark gray to black, fine grained, massive to locally vesicular and/or pillowed; composed of mafic minerals (augite and olivine) and plagioclase feldspar;*

Ttls *Tan, moderately hard, thick-bedded arkosic sandstone*

Tsl *Simi Conglomerate Member: gray, vaguely bedded, cobble conglomerate of smooth*

Kcg *Gray to brown, crudely bedded conglomerate of cobbles and pebbles of metavolcanic and granitic rocks and quartzite in brown sandy matrix*

Kcr *"Trabuco" Formation (of Durrell 1954; Colburn, in Fritsche 1973): rusty-brown conglomerate similar to Kcg but locally includes reddish sandstone and claystone; probably nonmarine; base sheared locally; possibly in fault contact with basement rocks (Durrell 1954; Denison and Aguilar 1991)*

qd *Quartz diorite (Lar and Vermont biotite quartz diorite of Neuerburg 1953, in Griffith Park area), medium to light gray, massive to vaguely gneissoid; composed mostly of plagioclase feldspar, and moderate amounts of quartz, biotite, and hornblende; moderately hard to somewhat incoherent where weathered*

REFERENCE: T.W. DIBBLEE (EDITED 2010) GEOLOGIC MAP OF THE HOLLYWOOD & SOUTH HALF BURBANK QUADRANGLES (#DF-30)

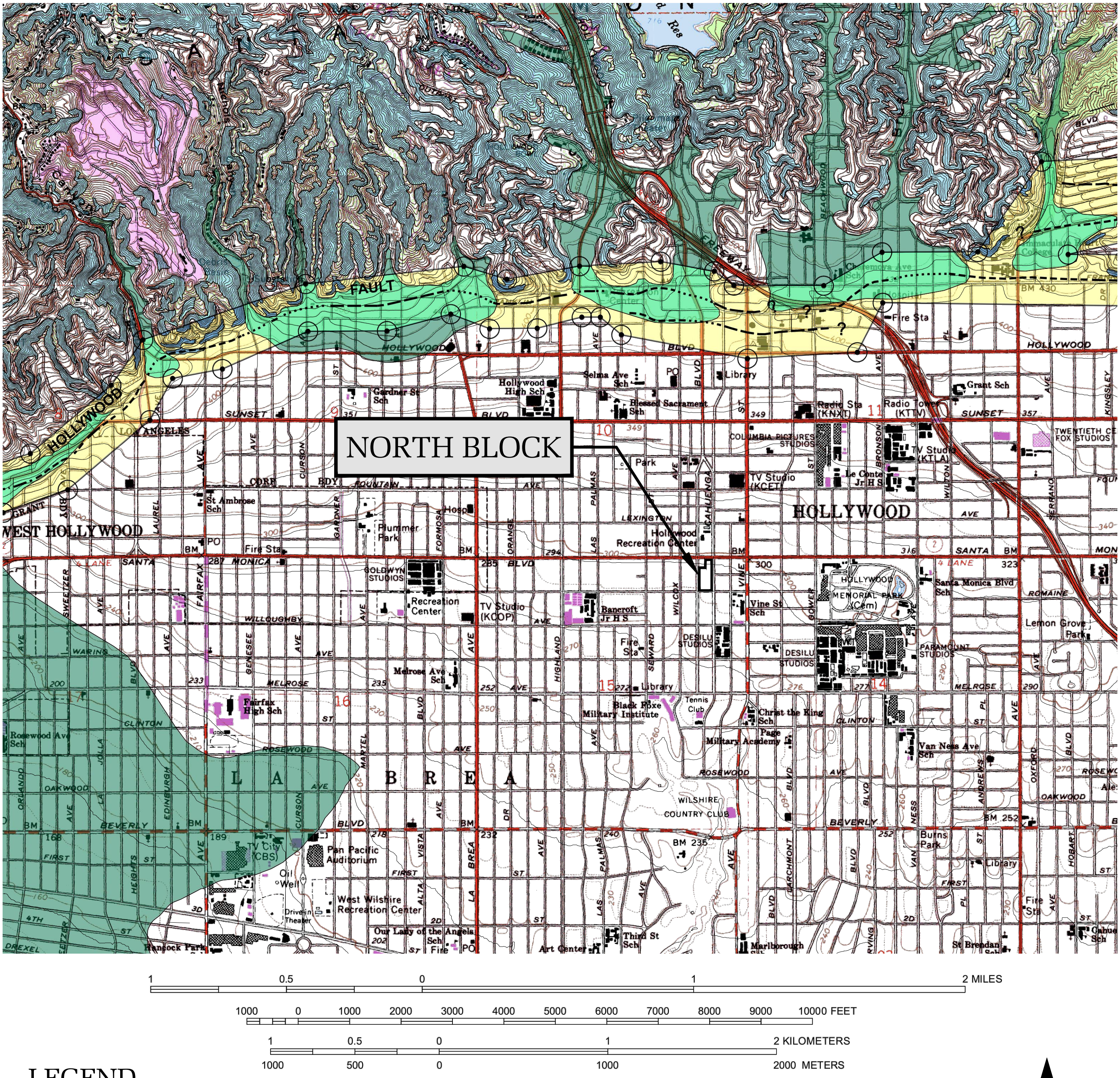
LOCAL GEOLOGIC MAP - DIBBLEE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

FILE NO: 22307

PLATE: II



LEGEND

🟨 EARTHQUAKE FAULT ZONES

🟩 LIQUEFACTION ZONES

🟦 EARTHQUAKE-INDUCED LANDSLIDE ZONES

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

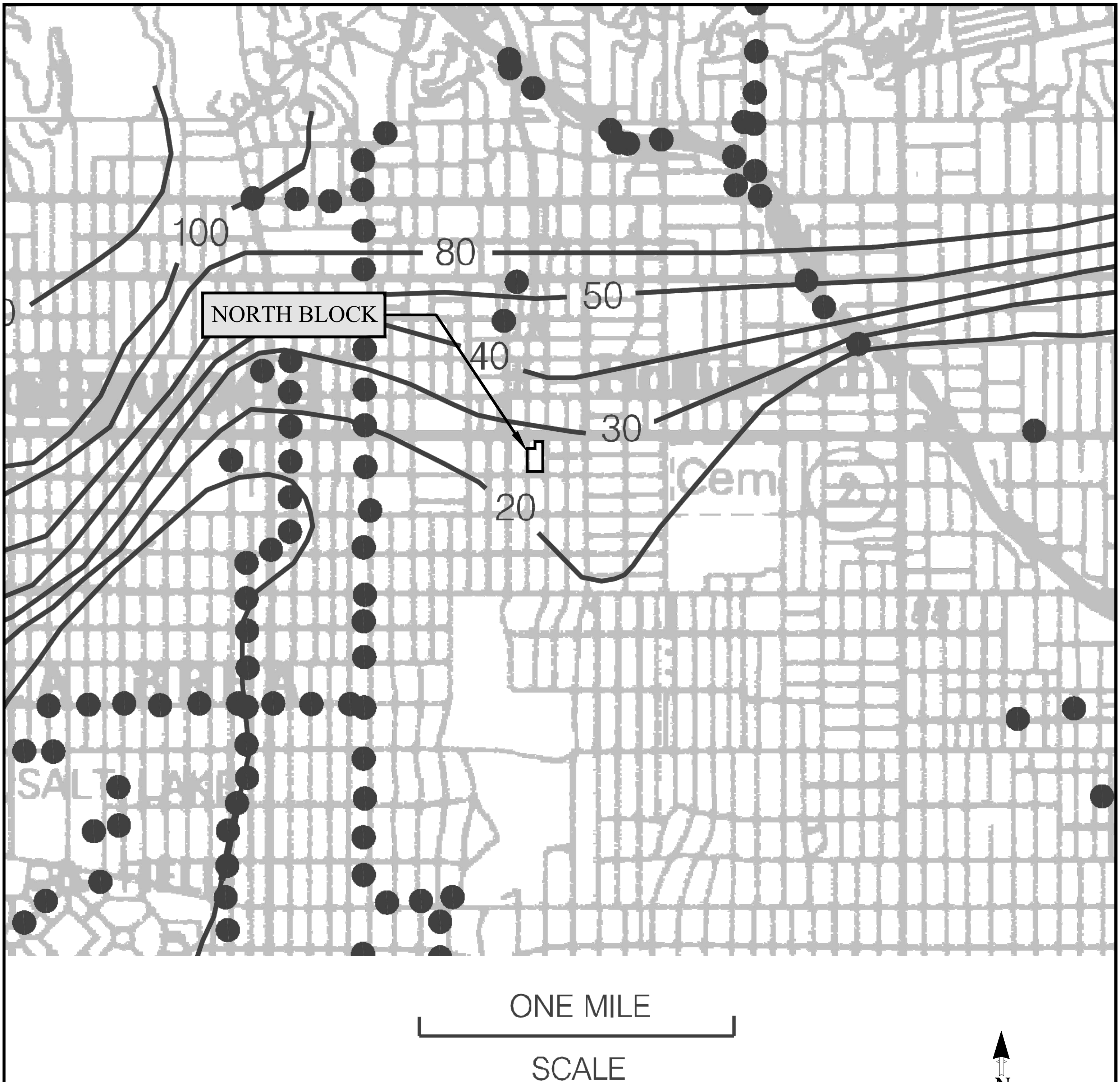
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

FILE NO: 22307

PLATE: III



LEGEND

● Borehole Site

30 Depth to ground water in feet

REFERENCE: SEISMIC HAZARD ZONE REPORT, 026 HOLLYWOOD, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998

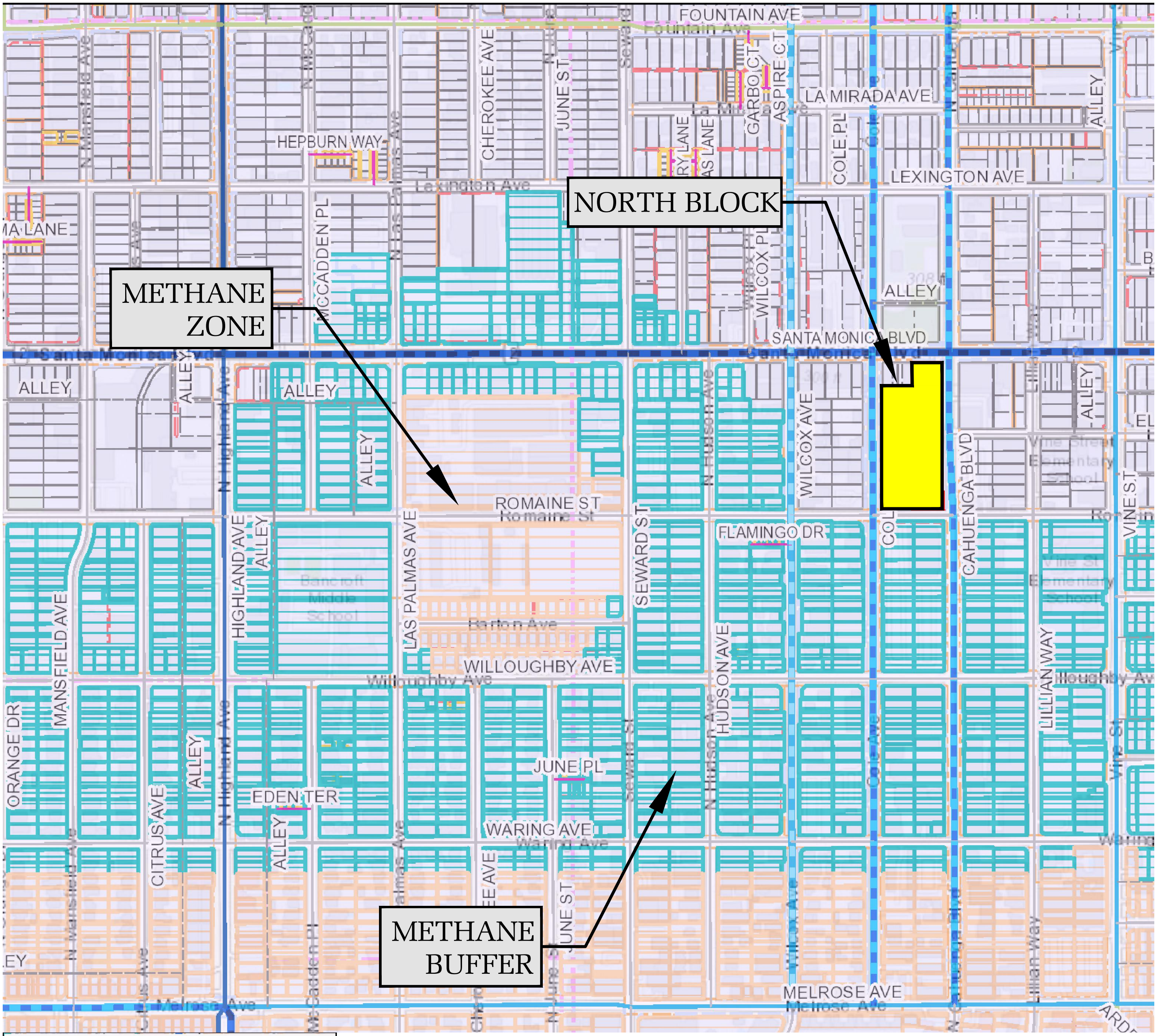
HISTORICALLY HIGHEST GROUND WATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

FILE NO: 22307

PLATE: IV



NORTH BLOCK

METHANE ZONE

METHANE BUFFER

L E G E N D

Methane Zone / Buffer Zone

- ▢ Methane Zone
- ▢ Methane Buffer Zone

500 250 0 500
Feet

This map is a user generated static output from an Intranet mapping site and is for reference only. Data layers that appear on this map may or may not be accurate,current, or otherwise reliable.

1 : 6,000

Eric Garcetti
Mayor

REFERENCE: https://navigatela.lacity.org/

METHANE ZONE RISK MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

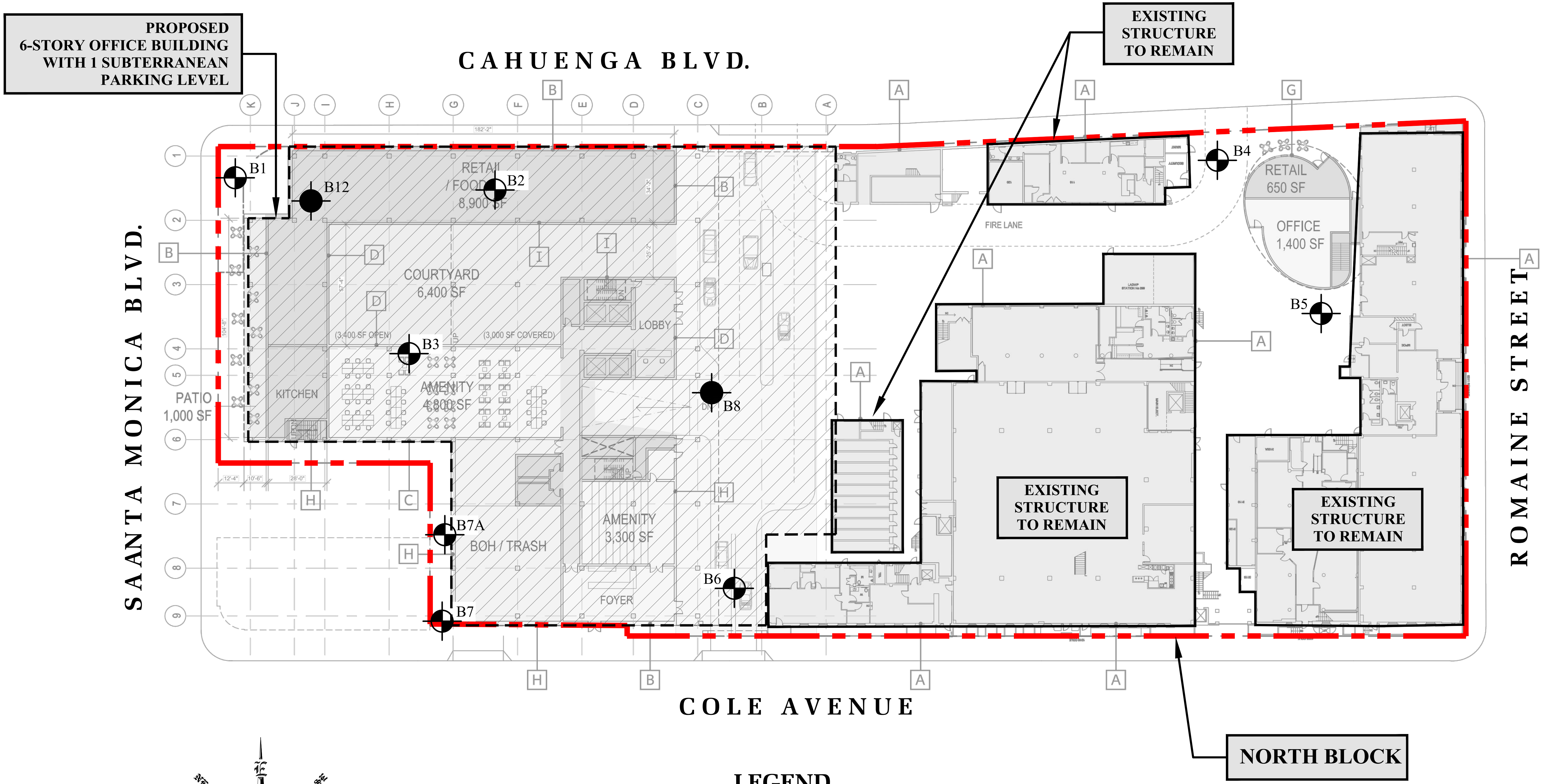
BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

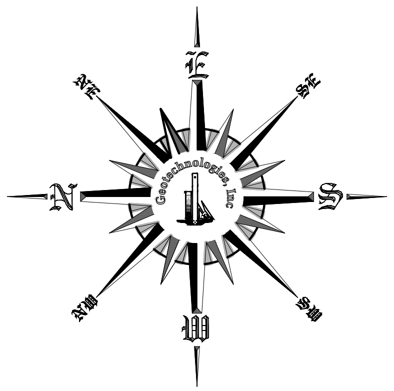
FILE NO: 22307

PLATE: V

Plot Date: Jun. 28, 2023

z:\shared\design cad_old\autocad\2023 projects\22307\22307 pp.dwg





SCALE IN FEET

0 25 50 100

1" = 50'

LEGEND

B7A ⊕ LOCATION & NUMBER OF BORING

B12 ● LOCATION & NUMBER OF BORING
(INVESTIGATION BY THE J. BYER GROUP, INC.
DATED: MAY 13, 1999)

REFERENCE: GROUND LEVEL PLAN BY RIOS
DATED: 06/10/2022

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC NORTH BLOCK

Drawn by: JD	File No.: 22307
Date: June 2023	Plate: VI

APPENDIX II

BORING LOGS

BORING LOG NUMBER 1

Bardas Investment Group **Date: 9/20/22** **Elevation: 302.5'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km ***Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 5-inch Concrete
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff, fine grained
				2 --		
2.5	22	19.8	103.2	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark gray, moist, stiff
				4 --		
				-		
5	25	16.5	114.6	5 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	30	16.3	115.8	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and yellowish brown, moist, dense to stiff, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	30	17.1	110.5	15 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark and yellowish brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	23	21.3	104.1	20 --		
				-	CL	Silty Clay, dark brown, moist, stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	73	23.3	105.4	25 --		
				-		

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22307

ln/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	68	19.1	109.0	30 --		
				-	SM	Silty Sand, dark grayish brown, wet, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	56	21.0	109.1	35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	69	24.1	105.2	40 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, wet, very dense to very stiff, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	75	22.2	110.1	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
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-	SM	Silty Sand, dark grayish brown, wet, very dense, fine grained
50	83	16.0	114.6	50 --		
				-		Total Depth: 50 feet Water at 17.3 feet Fill to 3 feet

BORING LOG NUMBER 2

Bardas Investment Group **Date: 9/20/22** **Elevation: 300.5'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		5-inch Asphalt over 3-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				2 --		
2.5	28	14.6	111.5	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	45	16.1	111.3	5 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	44	11.1	116.2	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	49	16.4	112.8	15 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	57	17.1	114.1	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	45	17.3	112.2	25 --		
				-		

BORING LOG NUMBER 2

Bardas Investment Group

File No. 22307

ln/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	40	14.4	115.9	30 --		
				-	SM	Sand to Silty Clay, dark brown, moist, dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	52	14.2	122.9	35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	49	17.9	113.0	40 --		
				-	SM/SP	Silty Sand to Sand, dark grayish brown, wet, dense, fine grained
				41 --		
				-		
				42 --		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
				-		
				43 --		
				-		
				44 --		
				-		
45	83	14.1	120.4	45 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark brown, moist, very stiff
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		Silty Sand to Sand, dark grayish brown, wet, very dense, fine grained
				-	SM/SP	
50	71	20.5	106.1	50 --		
				-		Total Depth: 50 feet Water at 18.3 feet Fill to 3 feet

BORING LOG NUMBER 3

Bardas Investment Group **Date: 9/19/22** **Elevation: 301'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		8-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt, dark brown, moist, firm, fine grained
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	34	11.6	SPT	5 --		
				-	CL	ALLUVIUM: Sandy Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	52	18.4	109.8	-		
				8 --		
				-		
				9 --		
				-		
10	19	23.6	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	44	19.8	100.5	-		
				13 --	ML/CL	Clayey Silt to Sandy Clay, dark brown, moist, stiff
				-		
				14 --		
				-		
15	24	13.9	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	65	20.8	109.2	-		
				18 --	CL	Sandy Clay, dark brown and gray, moist, very stiff
				-		
				19 --		
				-		
20	23	21.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	58	17.5	111.6	-		
				23 --		
				-		
				24 --		
				-		
25	25	16.6	SPT	25 --		
				-	SM/ML	Sandy Silt to Silty Sand, dark brown, moist, stiff to dense, fine grained

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22307

ln/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	56	17.6	114.2	-		
				28 --		
				-		
				29 --		
				-		
30	20	20.2	SPT	30 --		
				-	SP/ML	Sand to Sandy Silt, dark brown, very moist, dense to stiff, fine grained
				31 --		
				-		
				32 --		
32.5	48	26.6	103.3	-		
				33 --	SM/ML	Silty Sand to Sandy Silt, dark brown, very moist, dense to stiff, fine grained
				-		
				34 --		
				-		
35	21	23.9	SPT	35 --		
				-	ML	Sandy Silt, dark and grayish brown, moist, stiff, fine grained
				36 --		
				-		
				37 --		
37.5	60	22.6	101.5	-		
				38 --	SM	Silty Sand, dark brown, wet, dense, fine grained
				-		
				39 --		
				-		
40	26	19.1	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	65	15.7	117.0	-		
				43 --		
				-		
				44 --		
				-		
45	63	17.4	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	70	12.3	122.2	-		
				48 --	SM/SP	Silty Sand to Sand, dark brown, wet, dense to very dense, fine grained
				-		
				49 --		
				-		
50	44	13.6	SPT	50 --		
				-	SP	Sand, dark brown, wet, dense, fine to medium grained

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22307

ln/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	74	29.1	95.6	-		
				53 --	ML	Sandy to Clayey Silt, dark grayish brown, very moist, very stiff
				-		
				54 --		
				-		
55	49	23.6	SPT	55 --		
				-	SP	Sand, dark brown, wet, dense, fine to medium grained
				56 --		
				-		
				57 --		
57.5	72	12.8	123.6	-		
				58 --	SM/SP	Silty Sand to Sand, dark brown, wet, dense to very dense, fine to medium grained, minor gravel
				-		
				59 --		
				-		
60	52	26.6	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	61	18.6	111.0	-		
				63 --	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very dense to very stiff, fine grained
				-		
				64 --		
				-		
65	40	17.8	SPT	65 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark grayish brown, moist, very stiff
				66 --		
				-		
				67 --		
67.5	69	17.4	112.9	-		
				68 --		
				-		
				69 --		
				-		
70	36	15.6	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	72	14.4	119.9	-		
				73 --	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff
				-		
				74 --		
				-		
75	45	19.2	SPT	75 --		
				-		

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22307

ln/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	85	24.1	101.3	-		
				78 --	ML/CL	Clayey Silt to Silty Clay, dark grayish brown, moist, very stiff
				-		
				79 --		
				-		
80	73	15.3	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	38	12.8	122.0	-		
	50/4"			83 --	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff
				-		
				84 --		
				-		
85	80	16.9	SPT	85 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, very stiff
				86 --		
				-		
				87 --		
87.5	40	15.1	118.8	-		
	50/4"			88 --		
				-		
				89 --		
				-		
90	78	13.8	SPT	90 --		
				-		Total Depth: 90 feet
				91 --		Water at 19.3 feet
				-		Fill to 5 feet
				92 --		
				-		NOTE: The stratification lines represent the approximate
				93 --		boundary between earth types; the transition may be gradual.
				-		
				94 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				95 --		Modified California Sampler used unless otherwise noted
				-		
				96 --		SPT=Standard Penetration Test
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

BORING LOG NUMBER 4

Bardas Investment Group **Date: 9/24/22** **Elevation: 296.5'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Driveway
				0 --		4.5 Inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				-		
				2 --		
2.5	29	17.9	107.7	-		
				3 --		
				-	ML/CL	ALLUVIUM: Clayey Silt to Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	48	20.0	109.5	5 --		
				-	CL	Silty Clay, dark brown, moist, very stiff, fine grained
				6 --		
				-		
				7 --		
7.5	51	16.8	112.2	-		
				8 --		
				-		
				9 --		
				-		
10	27	14.1	105.4	10 --		
				-	SM/SP	Silty Sand to Sand, dark yellowish brown, moist, dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	14	47.9	75.4	15 --		
				-	CH	Silty Clay, dark and yellowish brown, wet, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	35	16.3	113.6	20 --		
				-	SM	Silty Sand, dark grayish brown, moist to very moist, dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	40	11.2	123.6	25 --		
	50/5"			-	SP	Sand, yellowish brown, wet, very dense, fine to medium grained, minor gravel

BORING LOG NUMBER 4

Bardas Investment Group

File No. 22307

ln/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, dense to stiff, fine grained
				-		
30	47	17.7	110.4	30 --		
				-		Total Depth: 30 feet Water at 12.3 feet Fill to 3 feet
				31 --		
				-		
				32 --		
				-		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				33 --		
				-		
				34 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 5

Bardas Investment Group **Date: 9/24/22** **Elevation: 296'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		4½-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				-		
				2 --		
2.5	22	20.1	104.2	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	42	20.8	107.0	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	26	20.0	105.2	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	22	47.3	74.0	15 --		
				-	CH	Silty Clay, dark brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	77	13.6	114.7	20 --		
				-	SP	Sand, dark yellowish brown, wet, very dense, fine grained, minor gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	80	13.0	120.8	25 --		
				-		

BORING LOG NUMBER 5

Bardas Investment Group

File No. 22307

ln/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-	CL	Silty Clay, yellow and grayish brown, moist, stiff
30	23	23.0	99.7	30 --		
				-		Total Depth: 30 feet
				31 --		Water at 16 feet
				-		Fill to 3 feet
				32 --		
				-		NOTE: The stratification lines represent the approximate
				33 --		boundary between earth types; the transition may be gradual.
				-		
				34 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				35 --		Modified California Sampler used unless otherwise noted
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 6

Bardas Investment **Date: 10/1/22** **Elevation: 299'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Driveway
				0 --		4-inch Concrete over 8-inch Base
				-		
				1 --		
				-		FILL: Silty Clay, black, moist, medium dense
				2 --		
2.5	24	19.1	107.2	-		
				3 --		
				-		
				4 --		
				-		
5	49	16.1	117.2	5 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	32	16.7	110.7	-		
				8 --		
				-		
				9 --		
				-		
10	22	9.5	115.9	10 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	39	18.2	113.7	15 --		
				-	CL	Silty Clay, yellowish to reddish brown, moist, very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	54	18.0	112.1	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	26	24.4	103.8	25 --		
				-	SM/SC	Silty Sand to Clayey Sand, reddish brown, very moist to wet, dense, fine grained

BORING LOG NUMBER 6

Bardas Investment

File No. 22307

ln/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	22	22.6	101.9	30 --		
				-		Silty Sand to Clayey Sand, reddish brown, wet, dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	33	No Recovery		35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	31	18.8	107.4	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	59	15.7	116.7	45 --		
				-	SM/ML	Silty Sand to Sandy Silt with Gravel, reddish brown, very moist, dense to very dense to stiff, fine and coarse grained
				46 --		
				-		
				47 --		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
				-		
				48 --		
				-		
				49 --		
				-		
50	61 50/6"	No Recovery		50 --		
				-		Total Depth: 50 feet Water at 22 feet Fill to 5 feet

BORING LOG NUMBER 7

Bardas Investment **Date: 10/1/22** **Elevation: 300'***

File No. 22307 **Method: 8-inch diameter Hollow Stem Auger**

ln/km

*Based on Survey Provided by JRN Civil Engineers, dated March 4, 2021

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Driveway
				0 --		4-inch Concrete over 2-inch Base
				-		
				1 --		FILL: Silty Clay, black, moist, stiff
				-		
				2 --		
2.5	24	21.0	105.0	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, black, moist, very stiff
				4 --		
				-		
5	45	16.8	111.8	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	23	4.3	115.2	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark reddish brown, moist, dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	35	9.2	117.1	15 --		
				-	SM	Silty Sand, reddish brown, moist, dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	31	11.9	118.3	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	43	20.7	107.3	25 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark reddish brown, very moist, very dense, very stiff, fine grained

BORING LOG NUMBER 7

Bardas Investment

File No. 22307

ln/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	13	26.9	97.8	30 --		
				-	CL	Silty Clay, brown to reddish brown, moist, stiff
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	37	23.8	111.9	35 --		
				-	SM	Silty Sand, dark reddish brown, wet, dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	36	16.8	112.6	40 --		
				-	CL	Silty Clay, brown to reddish brown, moist, very stiff
				41 --		
				-		NOTE: The stratification lines represent the approximate
				42 --		boundary between earth types; the transition may be gradual.
				-		
				43 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				44 --		Modified California Sampler used unless otherwise noted
				-		
45	60	17.5	114.4	45 --		
				-	SM	Silty Sand, dark brown to reddish brown, wet, dense to
				46 --		very dense, fine grained
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	9	16.2	105.3	50 --		
	50/6"			-		Total Depth: 50 feet
						Water at 20 feet
						Fill to 3 feet

Project No: JB 18051-B

Log of Boring 8

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Gravelly Sand, dark gray brown, slightly moist dense	▨							
-1	1		▨	SW						
		ALLUVIUM: Sandy Clay, black, moist, stiff, slightly porous	▤							
-2	2		▤	—	R	16	20.3	105.2		
-3	3		▤	CL						
-4	4	dark reddish brown, some gravel	▤							
-5	5		▤	----	R	21	14.0	113.9		
-6	6	Silty Sand, reddish brown, moist, dense to very dense, some clay binder, some gravel	░							
-7	7		░	SM	R	22	17.4	110.5		
-8	8		░							
-9	9	dense, slightly porous	░							
-10	10		░	----	R	15	13.0	109.8		
-11	11		░							
-12	12		░							
-13	13		░							
-14	14	Silty Sand, dark gray to brownish gray, moist, dense, slightly porous End at 15 Feet; No Water; Fill to 1½ Feet.	░							
-15	15			----	R	17	17.2	113.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Concrete Driveway

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 12

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL** Clayey Sand, brown to dark brown, moist, dense								
-1	1			SC						
-2	2	**ALLUVIUM:** Clay, black, moist, stiff		----	R	20	27.1	97.9		
-3	3			CL						
-4	4	Sandy Clay, brown, moist, very firm, slightly porous								
-5	5			----	R	19	19.2	106.8		
-6	6									
-7	7						20.8	109.4		
-8	8			----	R	21				
-9	9	Sandy Silt, brown to light brown, moist, firm								
-10	10			ML	R	12	20.0	100.5		
-11	11									
-12	12	Clayey Silt, dark gray brown to brown, moist, firm								
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet		----	R	14	28.6	92.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

APPENDIX III

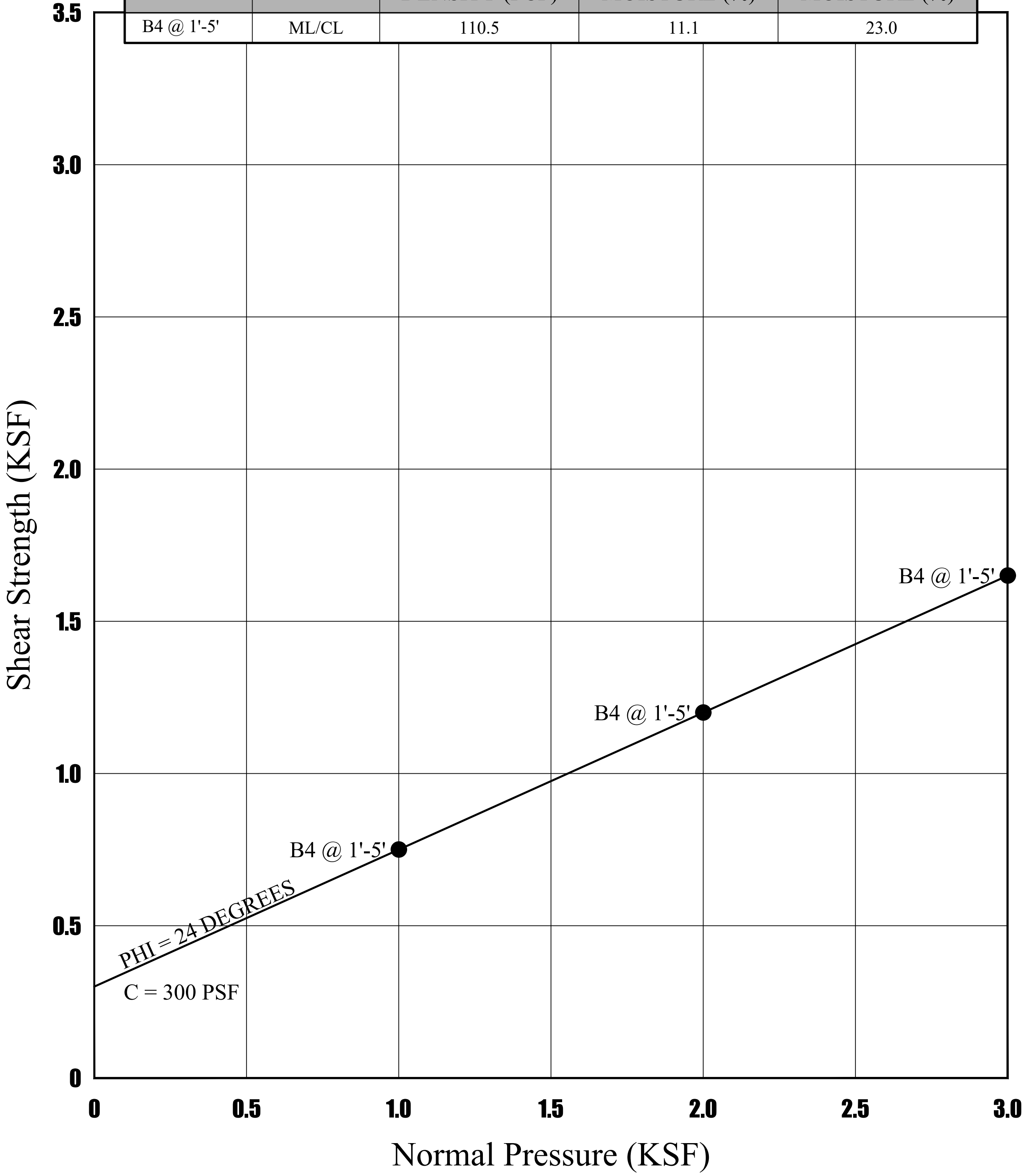
LABORATORY TESTING

SOIL CORROSIVITY REPORT

AND ANALYSIS

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4 @ 1'-5'	ML/CL	110.5	11.1	23.0



● Direct Shear, Saturated

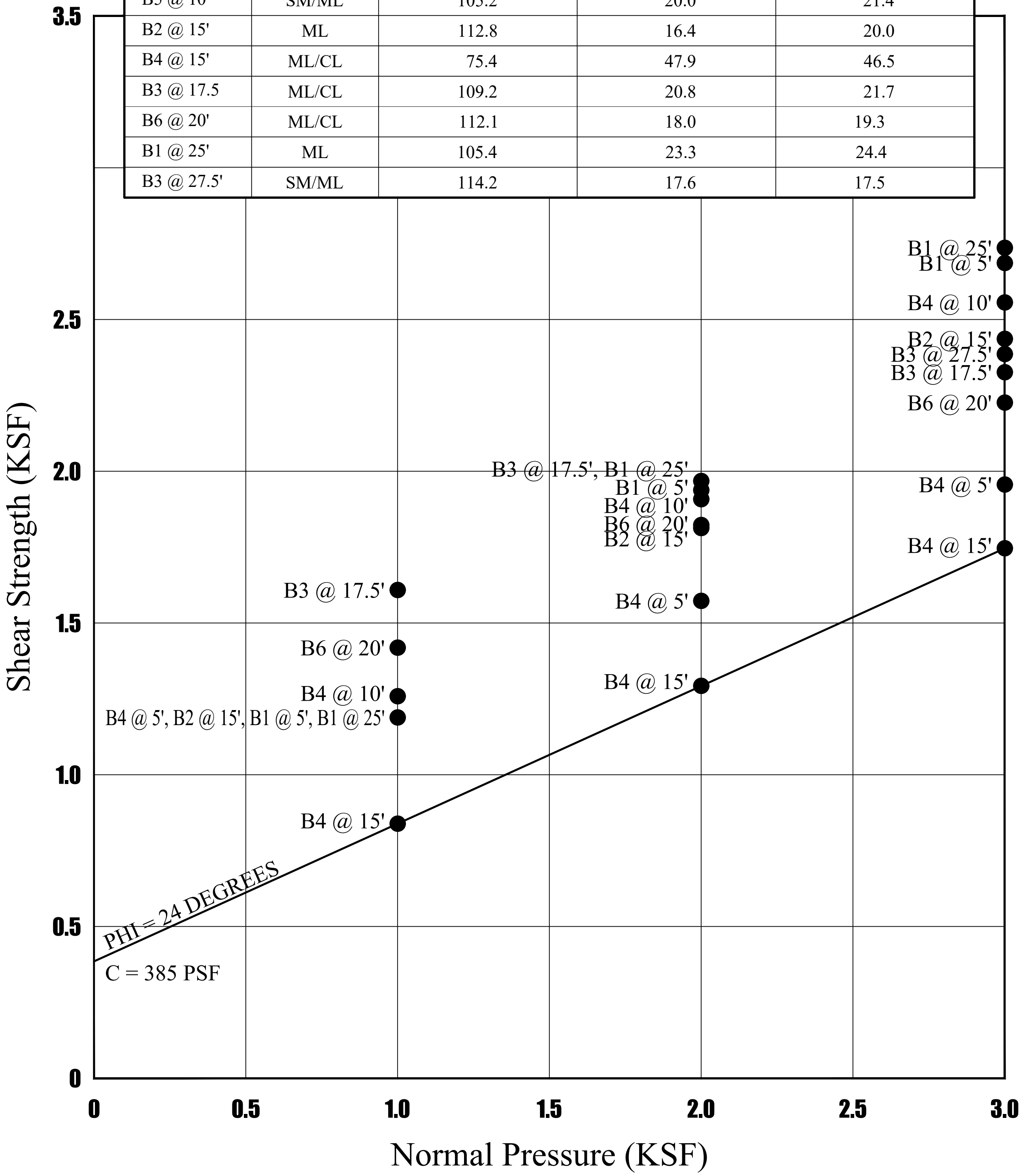
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
6311 Romaine Street, Los Angeles

FILE NO: 22307
PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	ML/CL	114.6	16.5	19.2
B4 @ 5'	SM/ML	109.5	20.0	24.4
B5 @ 10'	SM/ML	105.2	20.0	21.4
B2 @ 15'	ML	112.8	16.4	20.0
B4 @ 15'	ML/CL	75.4	47.9	46.5
B3 @ 17.5	ML/CL	109.2	20.8	21.7
B6 @ 20'	ML/CL	112.1	18.0	19.3
B1 @ 25'	ML	105.4	23.3	24.4
B3 @ 27.5'	SM/ML	114.2	17.6	17.5



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

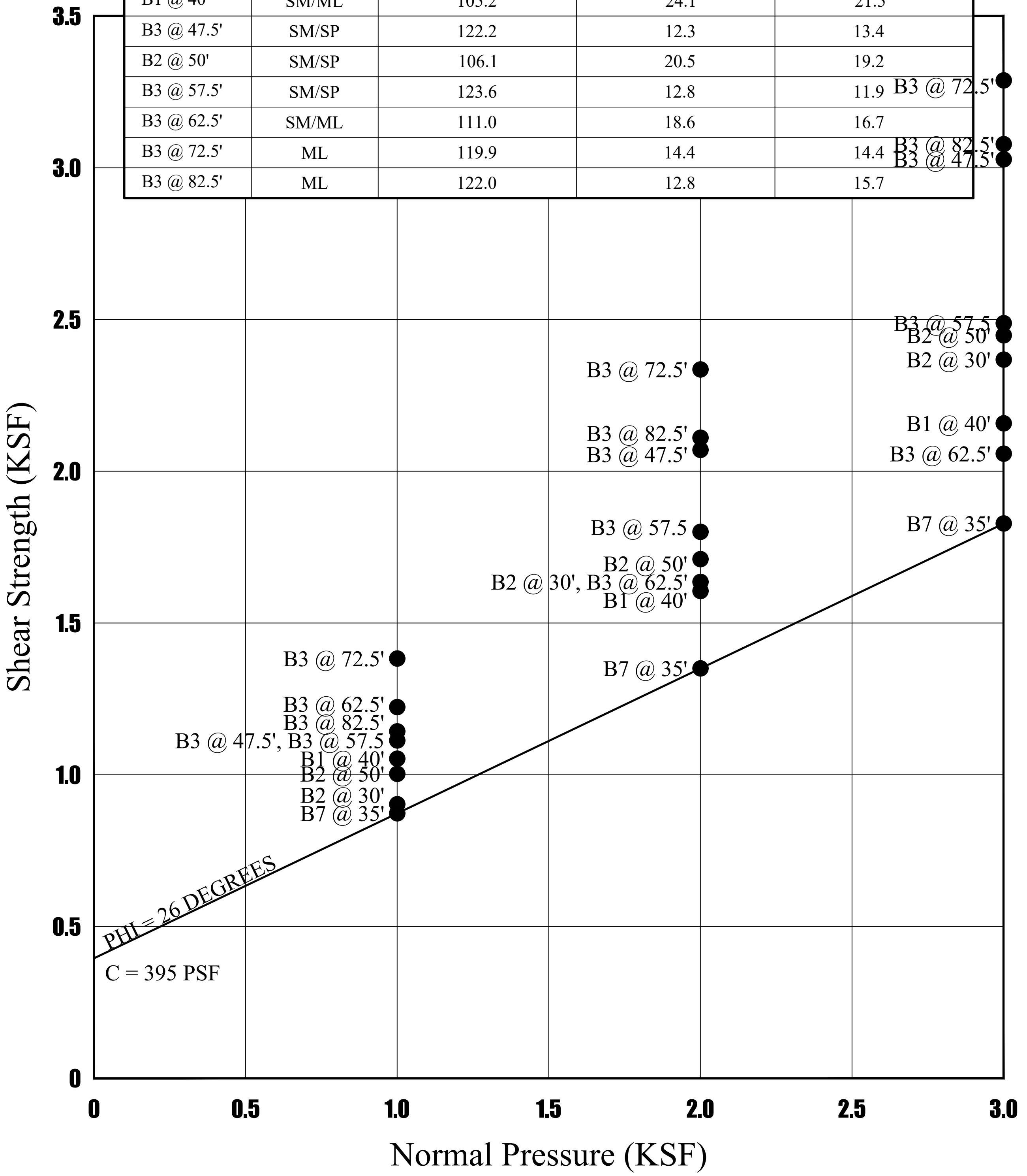
Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
6311 Romaine Street, Los Angeles

FILE NO: 22307

PLATE: B-2

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 30'	SM/ML	115.9	14.4	25.1
B7 @ 35'	SM	111.9	23.8	22.6
B1 @ 40'	SM/ML	105.2	24.1	21.5
B3 @ 47.5'	SM/SP	122.2	12.3	13.4
B2 @ 50'	SM/SP	106.1	20.5	19.2
B3 @ 57.5'	SM/SP	123.6	12.8	11.9
B3 @ 62.5'	SM/ML	111.0	18.6	16.7
B3 @ 72.5'	ML	119.9	14.4	14.4
B3 @ 82.5'	ML	122.0	12.8	15.7



● Direct Shear, Saturated

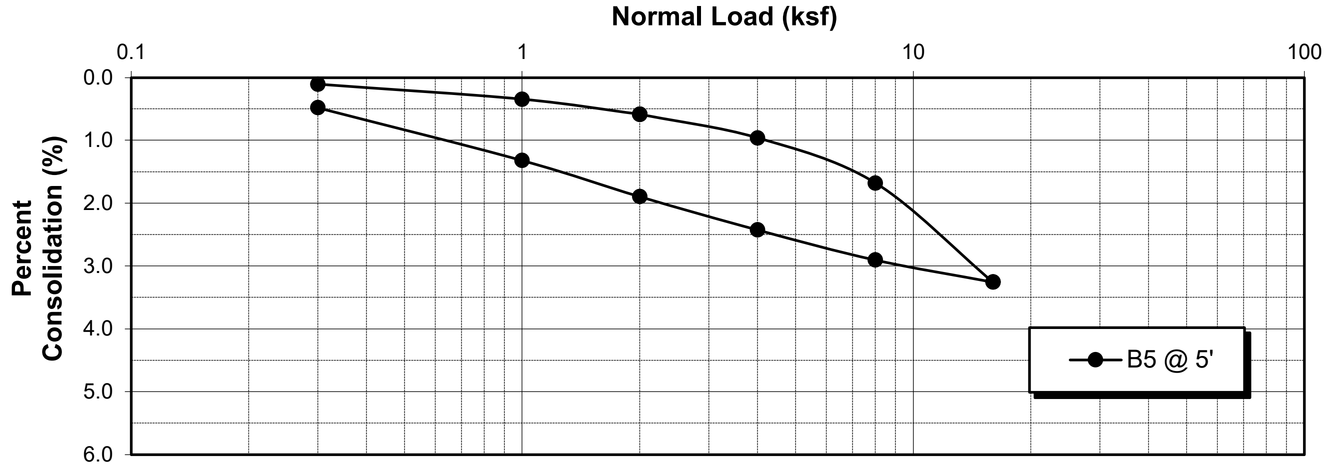
SHEAR TEST DIAGRAM

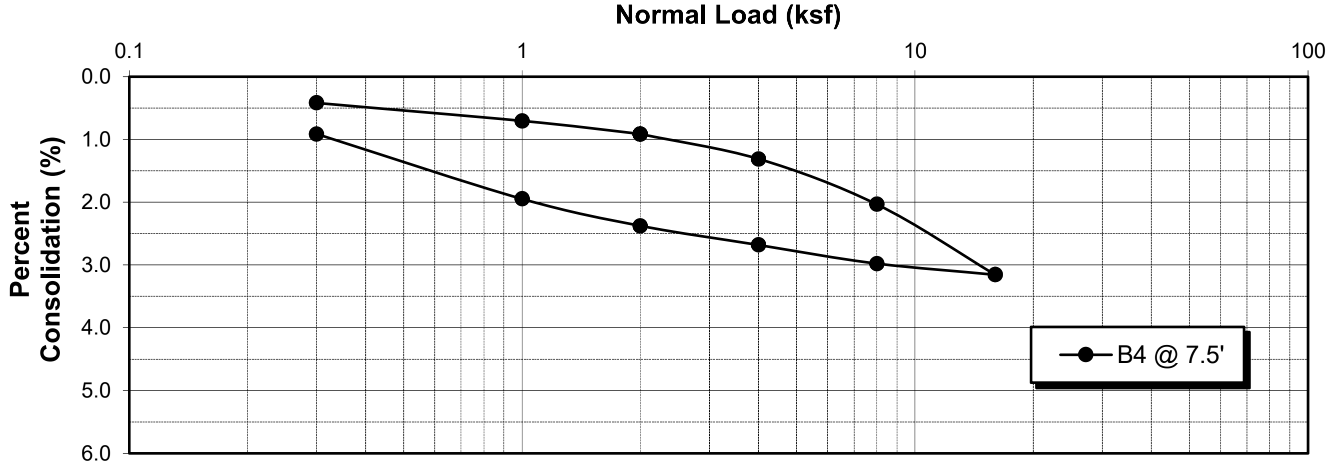
Geotechnologies, Inc.
Consulting Geotechnical Engineers

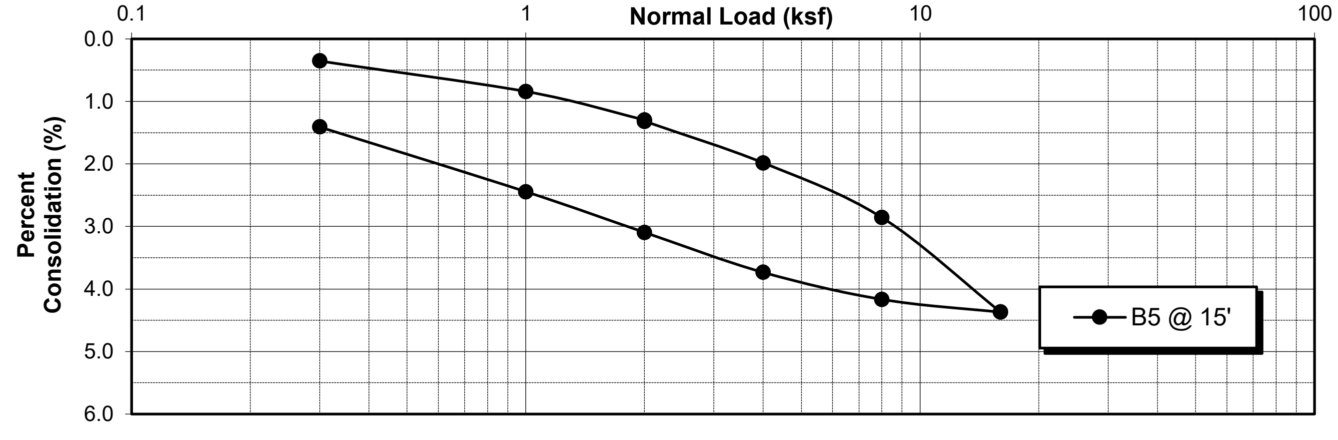
BARDAS INVESTMENT GROUP
6311 Romaine Street, Los Angeles

FILE NO: 22307

PLATE: B-3







Water added at 2 KSF

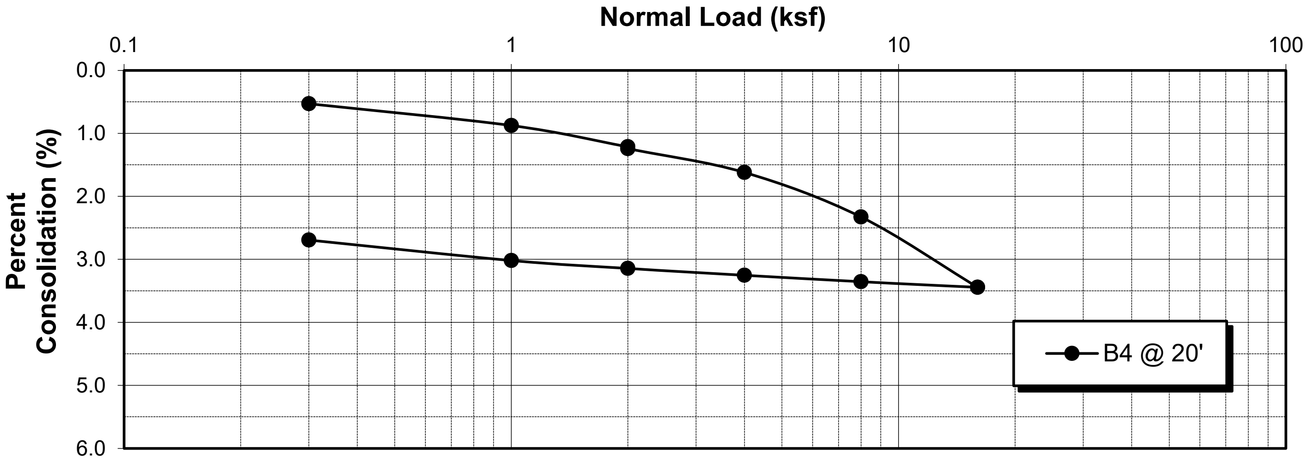
CONSOLIDATION

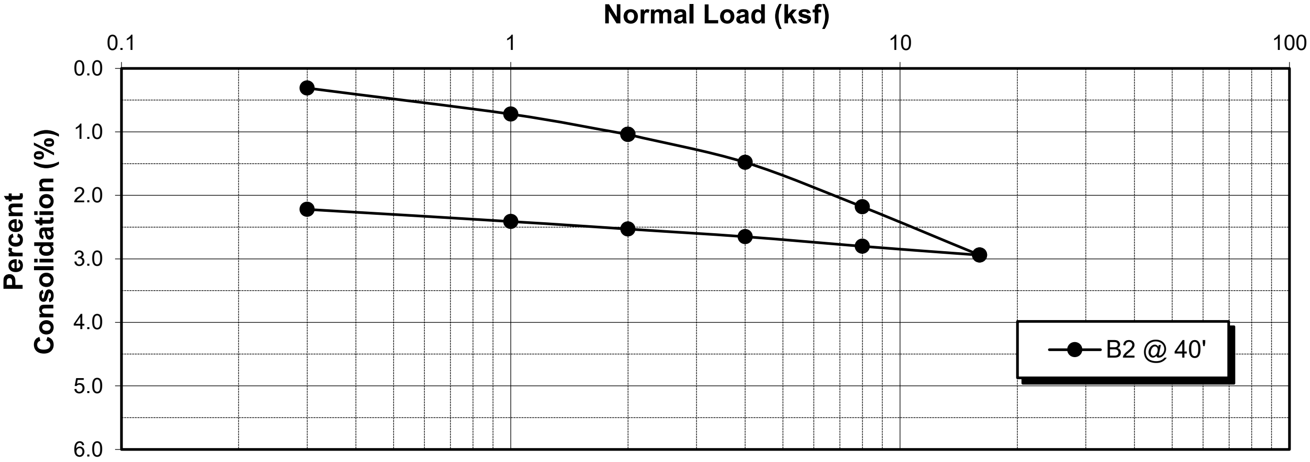
Geotechnologies, Inc.
Consulting Geotechnical Engineers

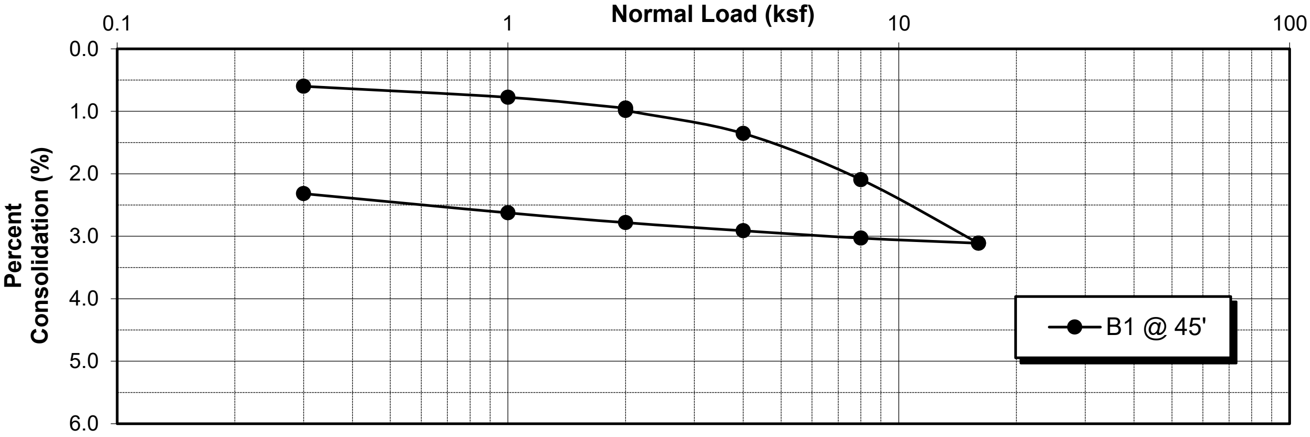
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

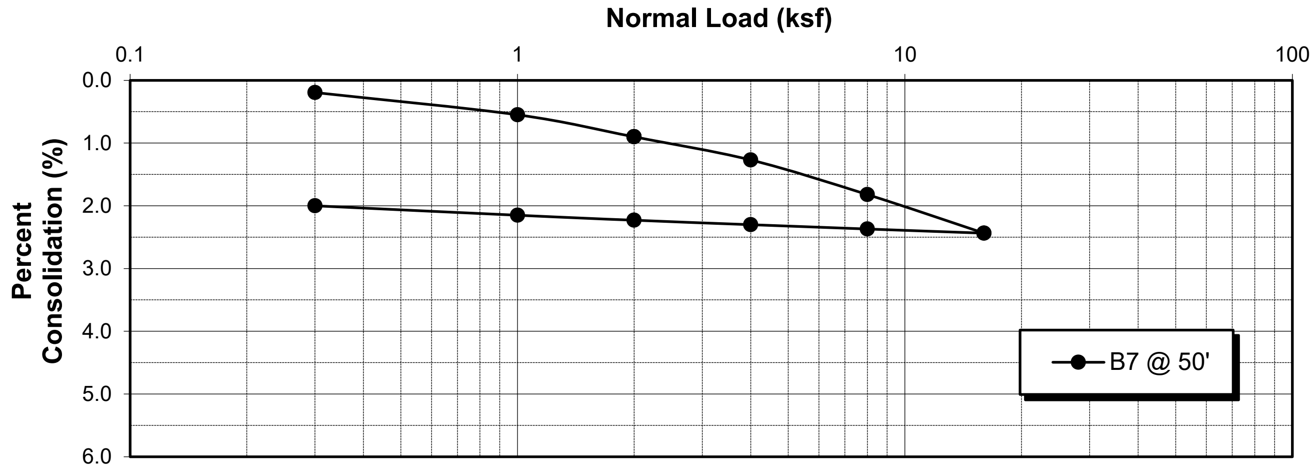
Geotechnologies, Inc.

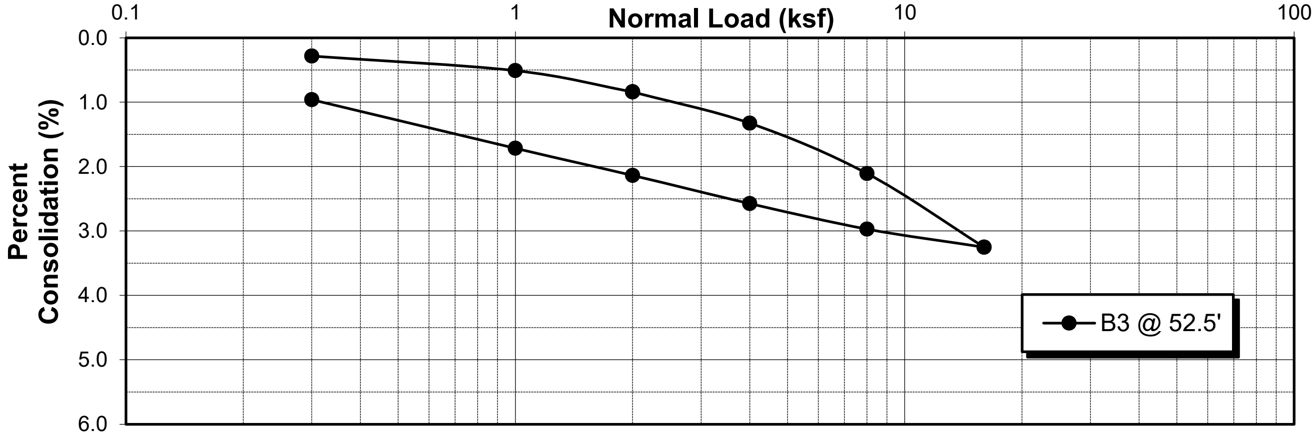
Consulting Geotechnical Engineers

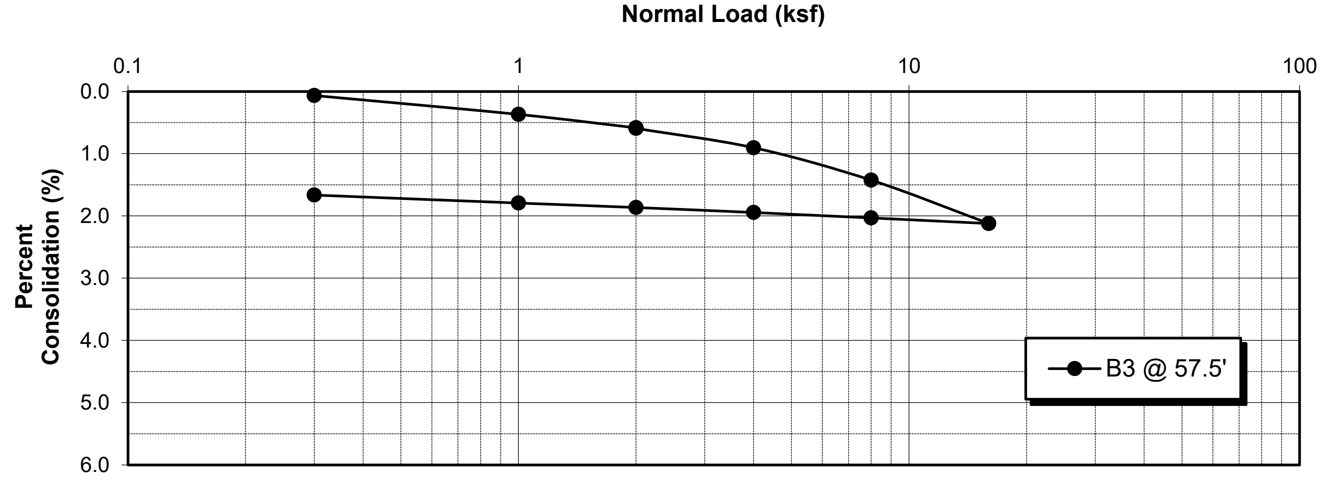
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: C-2







Water added at 2 KSF

CONSOLIDATION

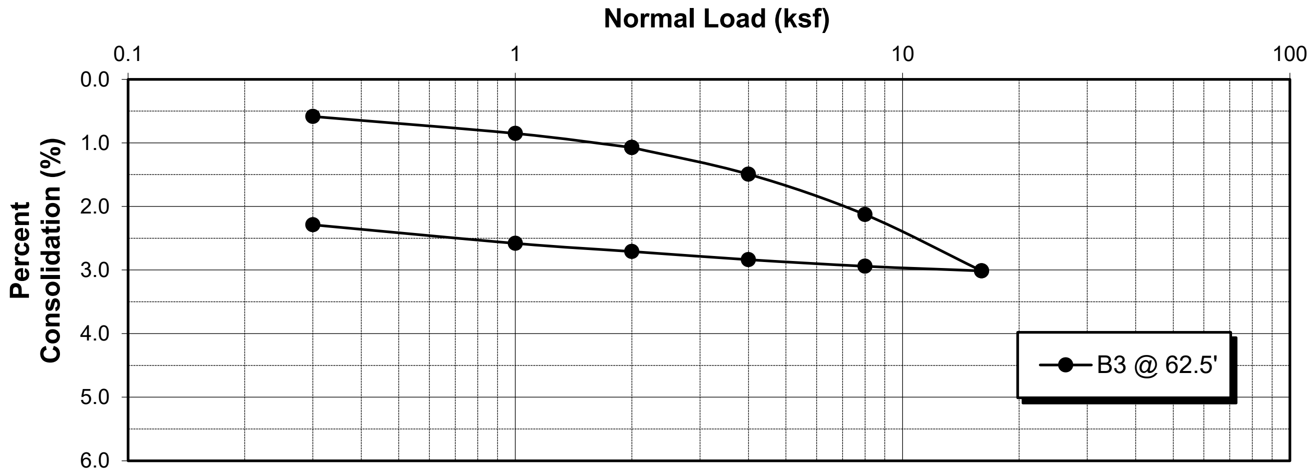
Geotechnologies, Inc.

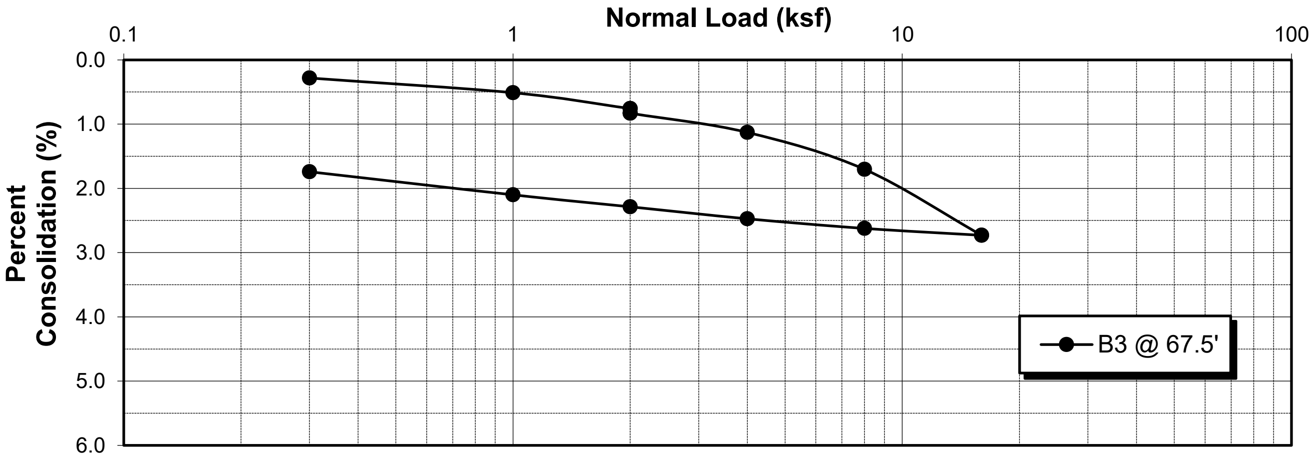
Consulting Geotechnical Engineers

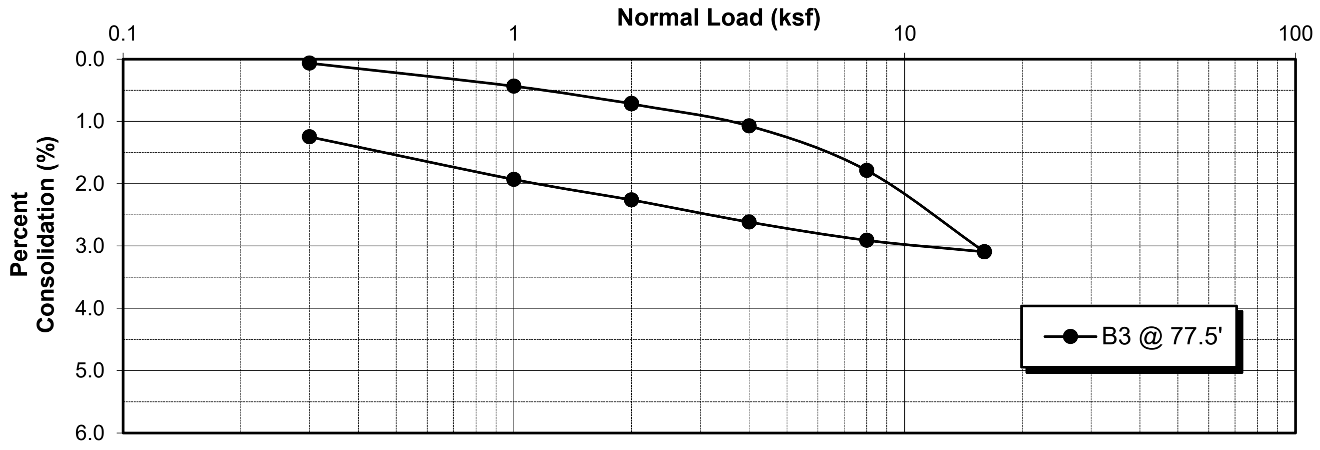
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: C-3







Water added at 2 KSF

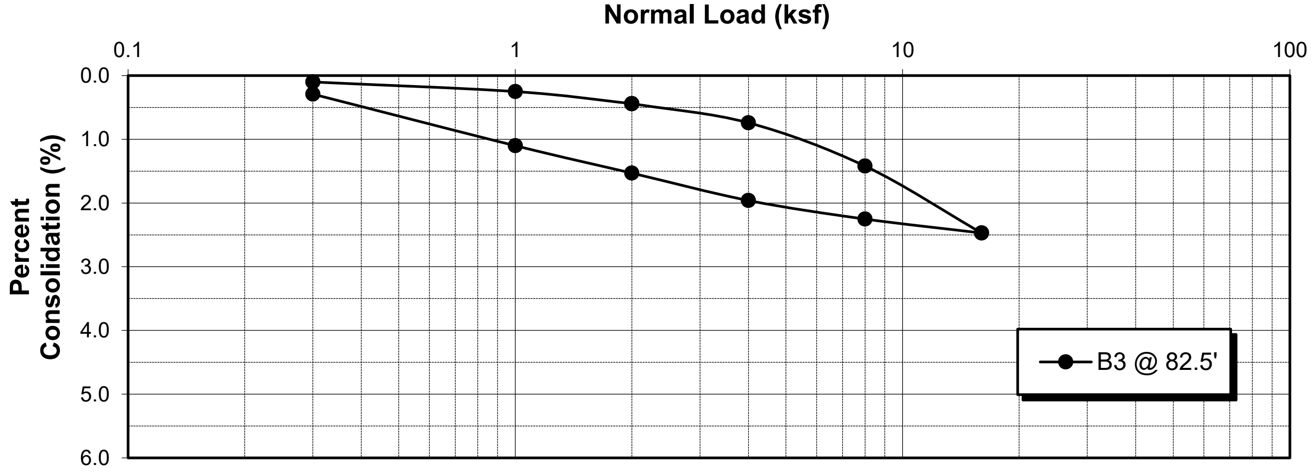
CONSOLIDATION

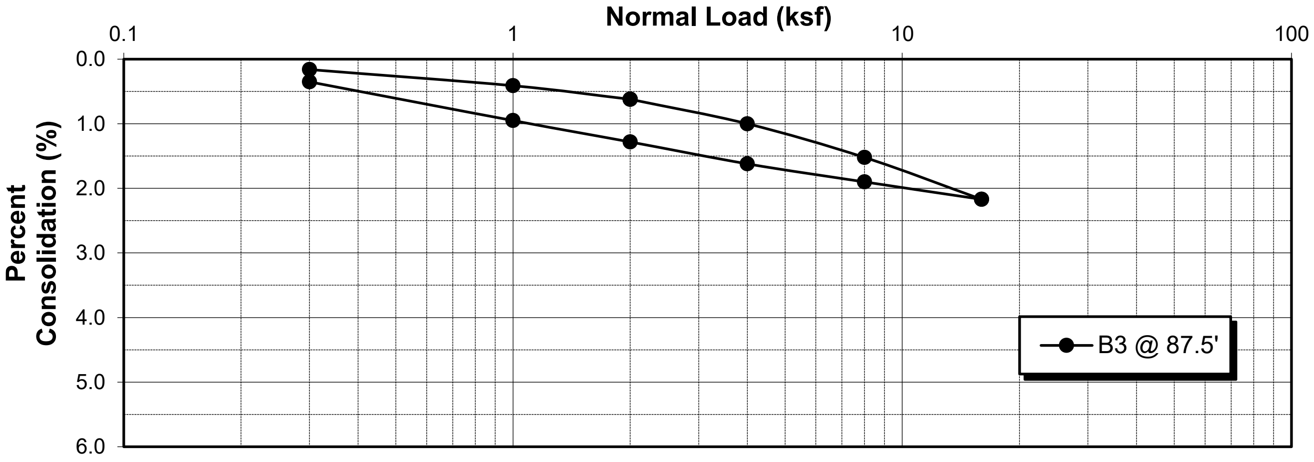
Geotechnologies, Inc.
Consulting Geotechnical Engineers

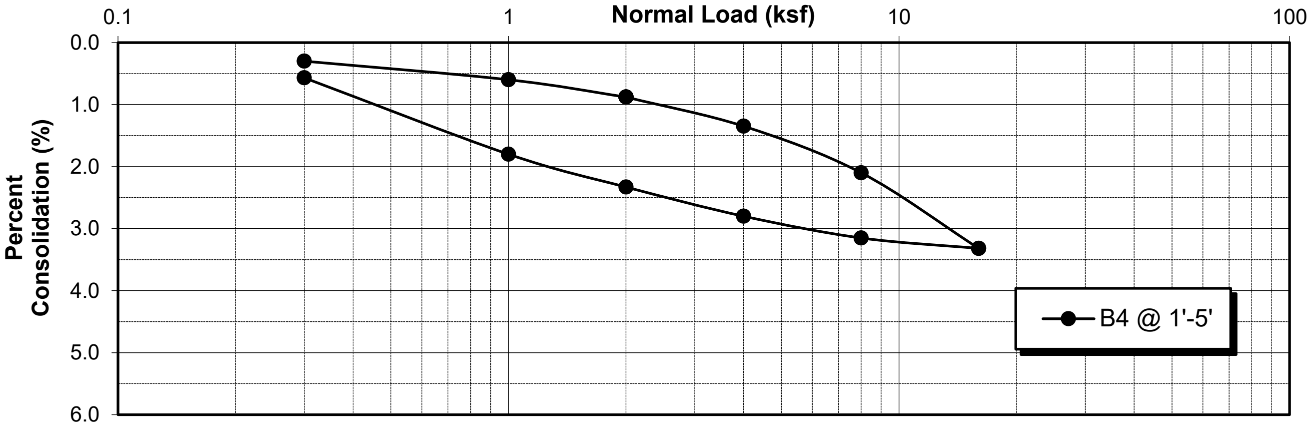
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: C-4







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

Consulting Geotechnical Engineers

PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: C-5

ASTM D-1557

SAMPLE	B1 @ 1'-5'	B3 @ 1'-5'	B4 @ 1'-5'
SOIL TYPE	SM	SM/ML	ML/CL
MAXIMUM DENSITY PCF.	121.8	120.1	121.1
OPTIMUM MOISTURE %	11.1	12.6	11.7

ASTM D 4829

SAMPLE	B1 @ 1'-5'	B3 @ 1'-5'	B4 @ 1'-5'
SOIL TYPE	SM	SM/ML	ML/CL
EXPANSION INDEX UBC STANDARD 18-2	20	102	110
EXPANSION CHARACTER	LOW	HIGH	HIGH

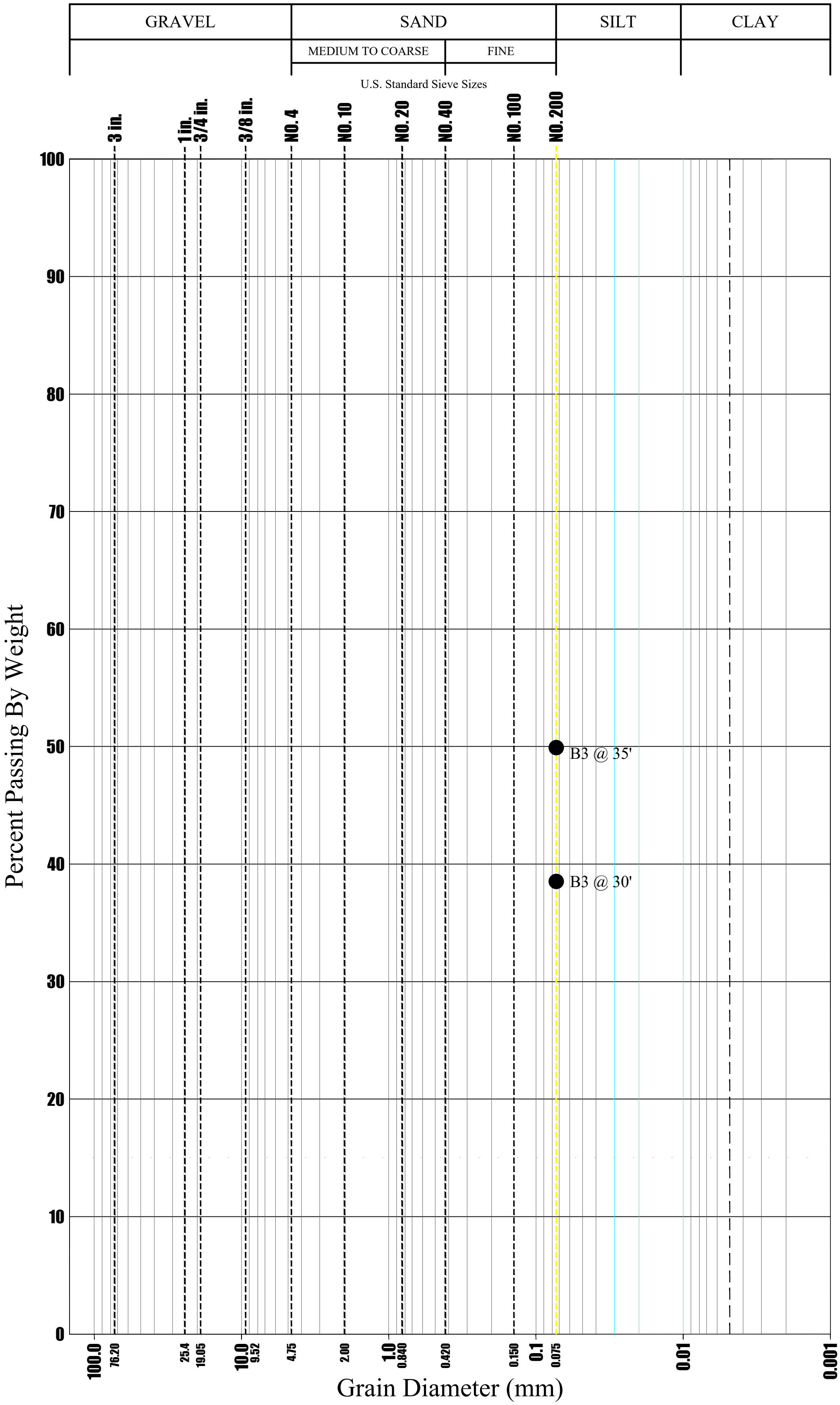
COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
6311 Romaine Street, Los Angeles

FILE NO: 22307

PLATE: D



GRAIN SIZE DISTRIBUTION

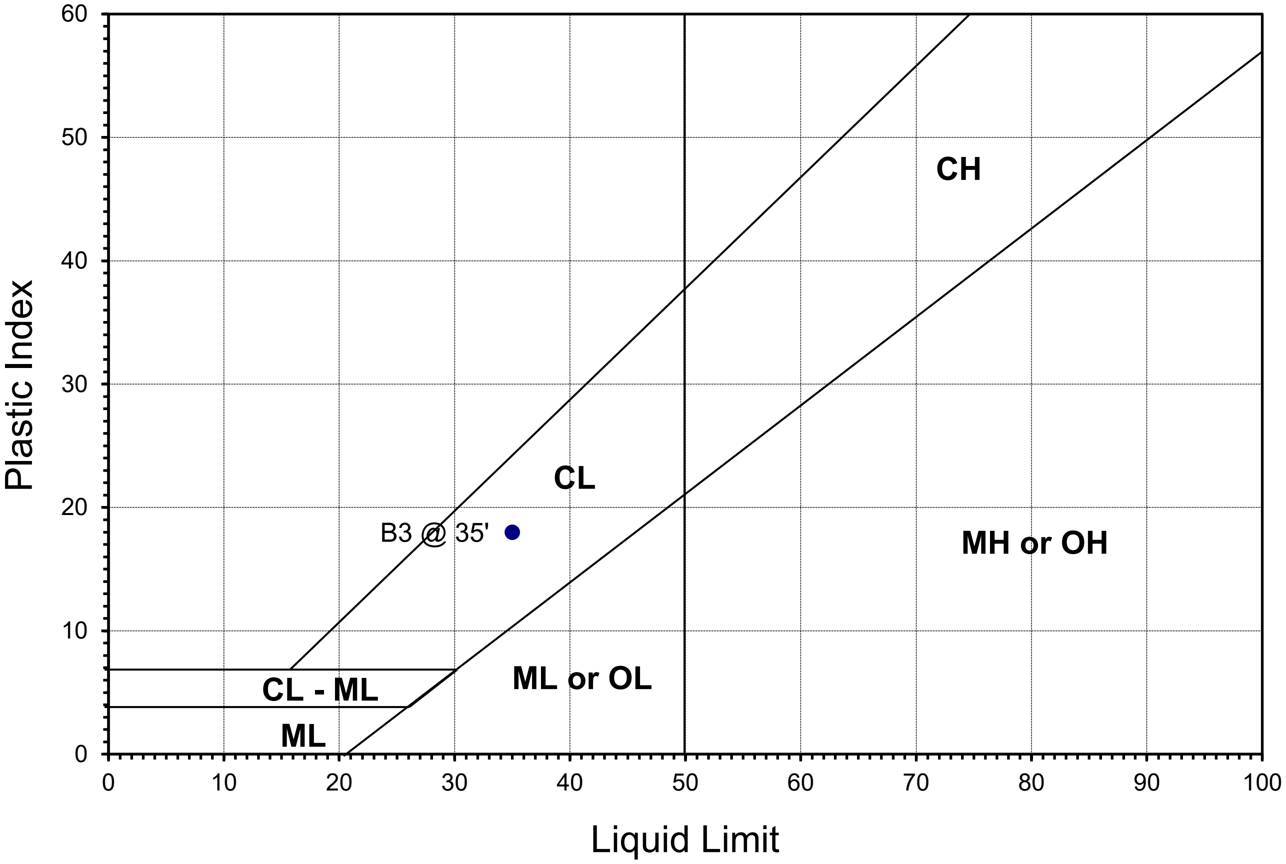
Geotechnologies, Inc.

Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
6311 Romaine Street, Los Angeles

FILE NO: 22307

PLATE: E



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B3 @ 35'	CL	52.2	35.0	17.0	18.0

ATTERBERG LIMITS

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22307

PLATE: F-1

November 18, 2022 via email: alozano@geoteq.com

Geotechnologies, Inc.
439 Western Ave.
Glendale, CA, 91201

Attention: Mr. Andres Lozano

Re: Soil Corrosivity Study
Bardas Investment Group
Los Angeles, CA
HDR #22-1017SCS, Geotechnologies #22307

Introduction

Laboratory tests have been completed on three soil samples provided to HDR for the Bardas Investment Group project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR assumes that the provided samples are representative of the most corrosive soils at the site.

The proposed commercial structures have one to four stories and four subterranean levels. The site is located at 6311 Romaine Street in Los Angeles, California, and the water table is reportedly 12 feet deep.

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

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per *ASTM International (ASTM)* G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and *American Water Works Association (AWWA)* Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory number 22-1017SCS. The full set of test results are shown in the attached Table A1.

Discussion

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil. A correlation between electrical resistivity and corrosivity toward ferrous metals is shown in Table 1.[1]

Table 1: Soil Corrosivity Categories

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

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

Electrical resistivities were in the mildly corrosive and moderately corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately corrosive and corrosive categories.

Soil pH values varied from 6.3 to 7.1. This range is slightly acidic to neutral.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low.

Per ACI-318, the soil is classified as S0 with respect to sulfate concentration.[3]

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

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

In conclusion, this soil is classified as corrosive to ferrous metals and negligible (S0) for sulfate attack on concrete.

---

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*
[3] *American Concrete Institute (ACI) 318-19 Table 19.3.1.1.*

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion. The following recommendations are based on the evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

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

    a. Dissimilar metals.

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

    c. Above ground steel pipe.

    d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:
      i. Polyurethane per AWWA C222 *or*
      ii. Extruded polyethylene per AWWA C215 *or*
      iii. A tape coating system per AWWA C214 *or*
      iv. Hot applied coal tar enamel per AWWA C203 *or*
      v. Fusion bonded epoxy per AWWA C213.
   b. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   As an alternative to the coating systems described in Option 1 and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

   a. At each end of the pipeline.
   b. At each end of any casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

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

      i. Polyethylene encasement per AWWA C105; *or*

      ii. Epoxy coating; *or*

      iii. Polyurethane; *or*

      iv. Wax tape.

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

   b. Apply cathodic protection to ductile iron piping as per NACE SP0169.

   OPTION 2

   As an alternative to the coating systems described in Option 1 and cathodic protection, encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe. Use the following parameters for clean sand backfill:

   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

   b. pH between 6.0 and 8.0.

   c. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]

2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286. Sleeve copper pipe through footings and foundations to prevent pH concentration cells and prevent leaks caused by settlement.

3. Electrically insulate cold water piping from hot water piping systems.

4. Protect cold water pipe using all of the following measures:

    a. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves. Ensure that sleeves are intact and free of cuts, tears, punctures, or other damage.

    b. Remove any construction debris, rocks, wood, or organic matter from the trench prior to backfill.

    c. Bed and backfill with at least 2 inches of clean sand all around the tubing, including the bedding. Use the following parameters for clean sand backfill:

        i. Minimum saturated resistivity of no less than 3,000 ohm-cm; and

        ii. pH between 6.0 and 8.0.

        iii. All backfill testing should be performed by a corrosion engineering laboratory.

    d. Copper tubing for cold water can also be treated the same as for hot water.

5. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing using one of the following measures:

    a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends using ammonia- and methanol-free caulk, or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; *or*

    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing with a suitable dielectric coating that is compatible with cathodic protection, such as Polyken 930.

---

[4] *2019 California Plumbing Code (CPC) Section 604.3.*

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately designed cathodic protection system per NACE SP0169.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

3. Due to the high ground water table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and ground water should be prevented as follows:

    a. For structures that extend below the water table, contact can be prevented with an impermeable waterproofing system. Options include a membrane such as Grace PrePrufe® products, a liquid applied barrier coating, or a waterproofing admixture such as Xypex® Admix. Visqueen, similar rolled barriers, or bentonite-based membranes are not viable waterproofing systems for corrosion protection.

    b. For structures above the water table, contact can be prevented with a gravel capillary break under the concrete and a vapor retarding membrane. Note that per ASTM E1643, "vapor retarders are not intended to provide a waterproofing function."[9] Alternatively, an impermeable waterproofing system may be used.

Hydraulic Elevators

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

    OPTION 1

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

---

[5] *2021 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[6] *2021 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[7] *2019 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[8] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*
[9] *ASTM E1643-18a : Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. ASTM International, 2018.*

    i. Polyurethane per AWWA C222 or
    ii. Extruded polyethylene per AWWA C215 or
    iii. A tape coating system per AWWA C214 or
    iv. Hot applied coal tar enamel per AWWA C203 or
    v. Fusion bonded epoxy per AWWA C213.

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

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

OPTION 2

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

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

OPTION 1

a. Provide a bonded dielectric coating,

b. Electrically isolate the pipeline, and

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

OPTION 2

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

Closure

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

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

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

Teddie Algorri

REGISTERED PROFESSIONAL ENGINEER
BRADLEY M. STUART
No. 7036
CHEMICAL
STATE OF CALIFORNIA

Teddie Algorri
Laboratory Coordinator

Bradley M. Stuart, PE
Corrosion Engineer

Enc: Table A1

22-1017SCS SCS TA-BS-TA.docx

Table A1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Bardas Investment Group
Your #22307, HDR Lab #22-1017SCS
2-Nov-22

Sample ID			B2 @ 45'	B3 @ 40'	B4 @ 1'-5'
Resistivity		**Units**			
as-received		ohm-cm	10,800	6,000	25,200
saturated		ohm-cm	1,200	3,360	1,600
pH			6.3	7.1	7.0
Electrical Conductivity		mS/cm	0.09	0.08	0.19
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	54	50	72
magnesium	Mg^{2+}	mg/kg	21	22	34
sodium	Na^{1+}	mg/kg	48	47	147
potassium	K^{1+}	mg/kg	24	23	28
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	110	85	195
fluoride	F^{1-}	mg/kg	16	12	16
chloride	Cl^{1-}	mg/kg	22	26	16
sulfate	SO_4^{2-}	mg/kg	69	76	184
nitrate	NO_3^{1-}	mg/kg	9.3	14	18
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND
Other Tests					
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Geotechnologies, Inc.

Project: Bardas Investment Group
File No.: 22307
Description: Liquefaction Analysis
Boring Numbe 3

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.98
Calculated Mag.Wtg.Factor:	1.212

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	19.3
Historically Highest Groundwater Level* (ft):	12.3
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

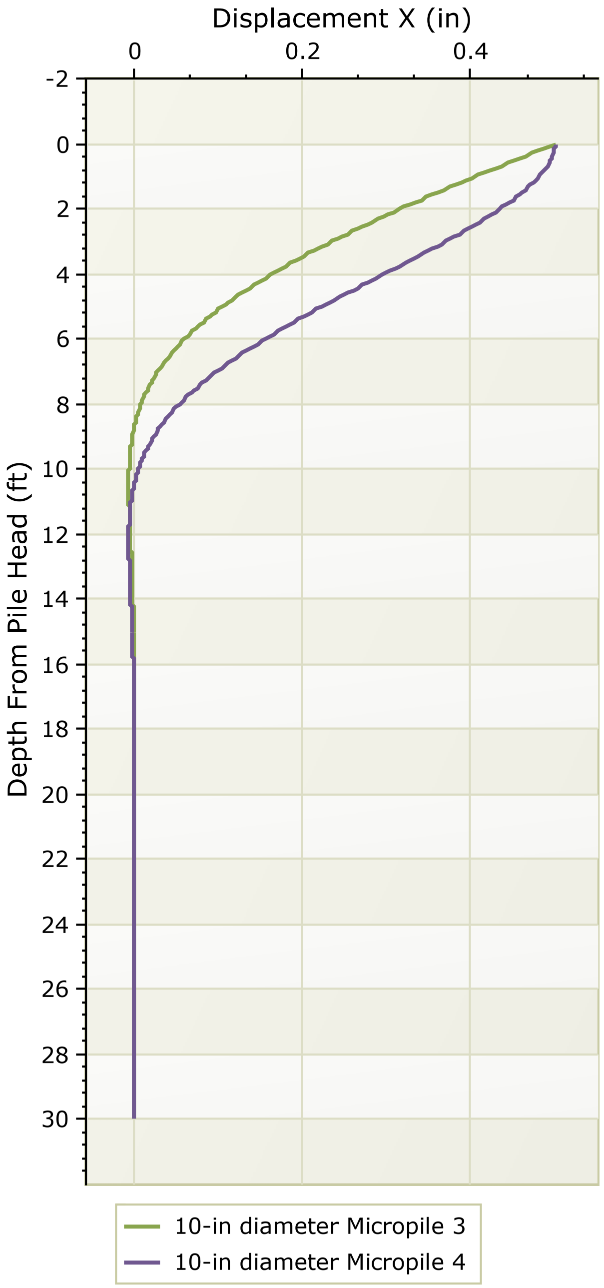
Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

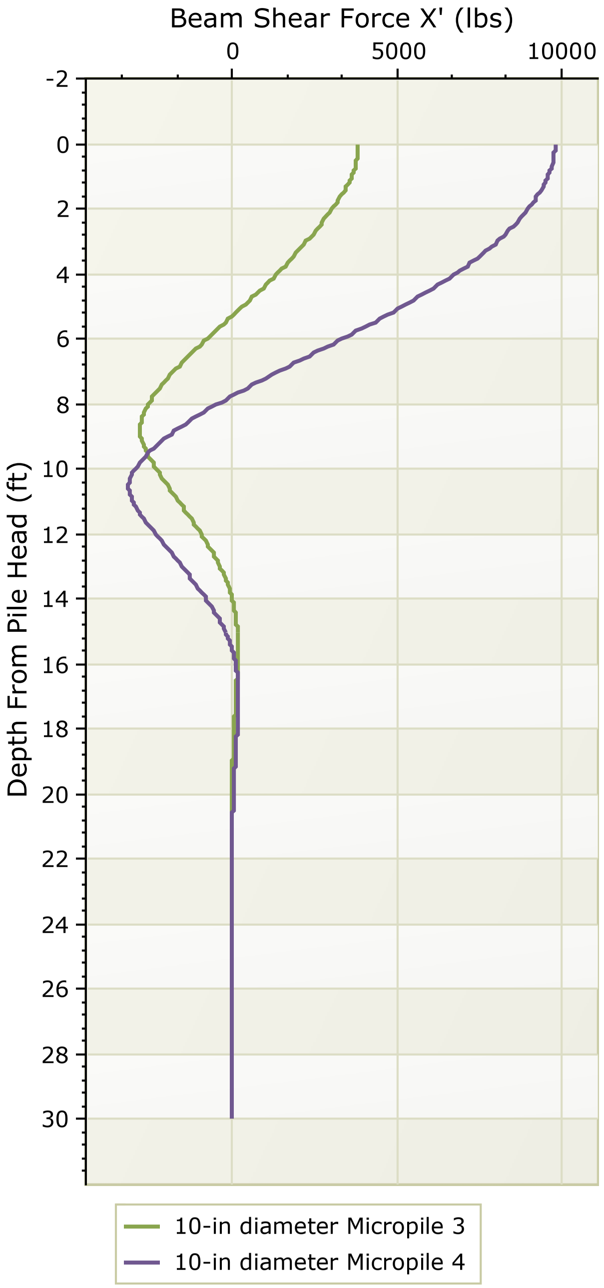
LIQUEFACTION BOUNDARY:

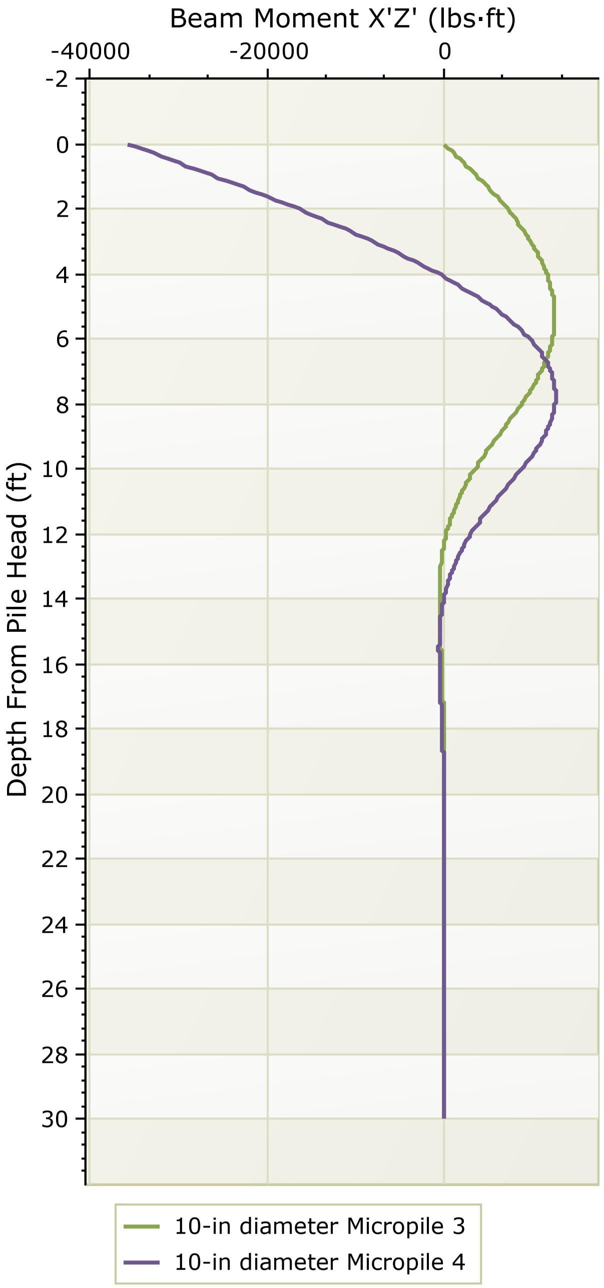
Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	130.0	Unsaturated	Unsaturated	34	5	0.0	0	130.0	130.0	81.0	1.00	0.642	2.000	Non-Liq.	0.00
2	130.0	Unsaturated	Unsaturated	34	5	0.0	0	260.0	260.0	81.0	1.00	0.640	2.000	Non-Liq.	0.00
3	130.0	Unsaturated	Unsaturated	34	5	0.0	0	390.0	390.0	74.3	1.00	0.638	2.000	Non-Liq.	0.00
4	130.0	Unsaturated	Unsaturated	34	5	0.0	0	520.0	520.0	68.9	0.99	0.636	2.000	Non-Liq.	0.00
5	130.0	Unsaturated	Unsaturated	34	5	0.0	0	650.0	650.0	69.3	0.99	0.634	2.000	Non-Liq.	0.00
6	130.0	Unsaturated	Unsaturated	34	5	0.0	0	780.0	780.0	66.0	0.99	0.631	2.000	Non-Liq.	0.00
7	130.0	Unsaturated	Unsaturated	34	5	0.0	0	910.0	910.0	63.4	0.98	0.629	2.000	Non-Liq.	0.00
8	130.0	Unsaturated	Unsaturated	19	10	0.0	0	1040.0	1040.0	37.0	0.98	0.626	2.000	Non-Liq.	0.00
9	130.0	Unsaturated	Unsaturated	19	10	0.0	0	1170.0	1170.0	37.4	0.97	0.623	2.000	Non-Liq.	0.00
10	130.0	Unsaturated	Unsaturated	19	10	0.0	0	1300.0	1300.0	35.7	0.97	0.621	1.709	Non-Liq.	0.00
11	130.0	Unsaturated	Unsaturated	19	10	0.0	0	1430.0	1430.0	34.2	0.97	0.618	1.247	Non-Liq.	0.00
12	130.0	Unsaturated	Unsaturated	19	10	0.0	0	1560.0	1560.0	32.8	0.96	0.615	0.957	Non-Liq.	0.00
13	120.3	Unsaturated	Saturated	24	15	0.0	0	1680.3	1636.6	41.2	0.96	0.628	2.000	3.2	0.00
14	120.3	Unsaturated	Saturated	24	15	0.0	0	1800.6	1694.5	40.8	0.95	0.647	2.000	3.1	0.00
15	120.3	Unsaturated	Saturated	24	15	0.0	0	1920.9	1752.4	44.9	0.95	0.664	2.000	3.0	0.00
16	120.3	Unsaturated	Saturated	24	15	0.0	0	2041.2	1810.3	44.5	0.94	0.679	2.000	2.9	0.00
17	120.3	Unsaturated	Saturated	24	15	0.0	0	2161.5	1868.2	44.1	0.94	0.693	2.000	2.9	0.00
18	131.9	Unsaturated	Saturated	23	20	0.0	0	2293.4	1937.7	41.9	0.93	0.705	2.000	2.8	0.00
19	131.9	Unsaturated	Saturated	23	20	0.0	0	2425.3	2007.2	41.4	0.93	0.716	2.000	2.8	0.00
20	131.9	Saturated	Saturated	23	20	0.0	0	2557.2	2076.7	41.0	0.92	0.725	2.000	2.8	0.00
21	131.9	Saturated	Saturated	23	20	0.0	0	2689.1	2146.2	40.6	0.92	0.734	2.000	2.7	0.00
22	131.9	Saturated	Saturated	23	20	0.0	0	2821.0	2215.7	40.2	0.91	0.741	2.000	2.7	0.00
23	131.1	Saturated	Saturated	25	25	0.0	0	2952.1	2284.4	43.4	0.90	0.748	2.000	2.7	0.00
24	131.1	Saturated	Saturated	25	25	0.0	0	3083.2	2353.1	43.1	0.90	0.753	2.000	2.7	0.00
25	131.1	Saturated	Saturated	25	25	0.0	0	3214.3	2421.8	42.7	0.89	0.758	2.000	2.6	0.00
26	131.1	Saturated	Saturated	25	25	0.0	0	3345.4	2490.5	42.4	0.89	0.763	2.000	2.6	0.00
27	131.1	Saturated	Saturated	25	25	0.0	0	3476.5	2559.2	42.1	0.88	0.766	2.000	2.6	0.00
28	134.4	Saturated	Saturated	20	30	38.5	0	3610.9	2631.2	39.3	0.88	0.769	2.000	2.6	0.00
29	134.4	Saturated	Saturated	20	30	38.5	0	3745.3	2703.2	38.9	0.87	0.771	2.000	2.6	0.00
30	134.4	Saturated	Saturated	20	30	38.5	0	3879.7	2775.2	38.6	0.86	0.773	2.000	2.6	0.00
31	134.4	Saturated	Saturated	20	30	38.5	0	4014.1	2847.2	38.2	0.86	0.774	2.000	2.6	0.00
32	134.4	Saturated	Saturated	20	30	38.5	0	4148.5	2919.2	37.9	0.85	0.775	2.000	2.6	0.00
33	130.9	Saturated	Saturated	21	35	52.2	18	4279.4	2987.7	39.7	0.85	0.775	2.000	2.6	0.00
34	130.9	Saturated	Saturated	21	35	52.2	18	4410.3	3056.2	39.4	0.84	0.776	2.000	2.6	0.00
35	130.9	Saturated	Saturated	21	35	52.2	18	4541.2	3124.7	39.1	0.83	0.776	2.000	2.6	0.00
36	130.9	Saturated	Saturated	21	35	52.2	18	4672.1	3193.2	38.8	0.83	0.775	2.000	2.6	0.00
37	130.9	Saturated	Saturated	21	35	52.2	18	4803.0	3261.7	38.5	0.82	0.774	2.000	2.6	0.00
38	124.4	Saturated	Saturated	26	40	0.0	0	4927.4	3323.7	43.1	0.82	0.774	2.000	2.6	0.00
39	124.4	Saturated	Saturated	26	40	0.0	0	5051.8	3385.7	42.9	0.81	0.773	2.000	2.6	0.00
40	124.4	Saturated	Saturated	26	40	0.0	0	5176.2	3447.7	42.7	0.80	0.772	2.000	2.6	0.00
41	124.4	Saturated	Saturated	26	40	0.0	0	5300.6	3509.7	42.5	0.80	0.771	2.000	2.6	0.00
42	124.4	Saturated	Saturated	26	40	0.0	0	5425.0	3571.7	42.3	0.79	0.769	2.000	2.6	0.00
43	135.3	Saturated	Saturated	63	45	0.0	0	5560.3	3644.6	102.0	0.79	0.767	2.000	2.6	0.00
44	135.3	Saturated	Saturated	63	45	0.0	0	5695.6	3717.5	101.4	0.78	0.764	2.000	2.6	0.00
45	135.3	Saturated	Saturated	63	45	0.0	0	5830.9	3790.4	100.9	0.77	0.762	2.000	2.6	0.00
46	135.3	Saturated	Saturated	63	45	0.0	0	5966.2	3863.3	100.4	0.77	0.759	1.991	2.6	0.00
47	135.3	Saturated	Saturated	63	45	0.0	0	6101.5	3936.2	99.9	0.76	0.756	1.978	2.6	0.00
48	137.3	Saturated	Saturated	44	50	0.0	0	6238.8	4011.1	69.4	0.76	0.752	1.964	2.6	0.00
49	137.3	Saturated	Saturated	44	50	0.0	0	6376.1	4086.0	69.1	0.75	0.749	1.951	2.6	0.00
50	137.3	Saturated	Saturated	44	50	0.0	0	6513.4	4160.9	68.8	0.74	0.745	1.938	2.6	0.00
51	137.3	Saturated	Saturated	44	50	0.0	0	6650.7	4235.8	68.4	0.74	0.742	1.925	2.6	0.00
52	137.3	Saturated	Saturated	44	50	0.0	0	6788.0	4310.7	68.1	0.73	0.738	1.913	2.6	0.00
53	123.5	Saturated	Saturated	39	55	0.0	0	6911.5	4371.8	60.2	0.73	0.735	1.903	2.6	0.00
54	123.5	Saturated	Saturated	39	55	0.0	0	7035.0	4432.9	59.9	0.72	0.732	1.893	2.6	0.00
55	123.5	Saturated	Saturated	39	55	0.0	0	7158.5	4494.0	59.7	0.72	0.729	1.883	2.6	0.00
56	123.5	Saturated	Saturated	39	55	0.0	0	7282.0	4555.1	59.5	0.71	0.726	1.873	2.6	0.00
57	123.5	Saturated	Saturated	39	55	0.0	0	7405.5	4616.2	59.3	0.70	0.723	1.864	2.6	0.00
58	139.4	Saturated	Saturated	52	60	0.0	0	7544.9	4693.2	78.7	0.70	0.719	1.852	2.6	0.00
59	139.4	Saturated	Saturated	52	60	0.0	0	7684.3	4770.2	78.4	0.69	0.715	1.840	2.6	0.00
60	139.4	Saturated	Saturated	52	60	0.0	0	7823.7	4847.2	78.1	0.69	0.711	1.829	2.6	0.00
61	139.4	Saturated	Saturated	52	60	0.0	0	7963.1	4924.2	77.7	0.68	0.707	1.818	2.6	0.00
62	139.4	Saturated	Saturated	52	60	0.0	0	8102.5	5001.2	77.4	0.68	0.703	1.807	2.6	0.00
63	131.7	Saturated	Saturated	40	65	0.0	0	8234.2	5070.5	59.3	0.67	0.699	1.797	2.6	0.00
64	131.7	Saturated	Saturated	40	65	0.0	0	8365.9	5139.8	59.1	0.67	0.695	1.787	2.6	0.00
65	131.7	Saturated	Saturated	40	65	0.0	0	8497.6	5209.1	58.9	0.66	0.692	1.777	2.6	0.00
66	131.7	Saturated	Saturated	40	65	0.0	0	8629.3	5278.4	58.7	0.66	0.688	1.768	2.6	0.00
67	131.7	Saturated	Saturated	40	65	0.0	0	8761.0	5347.7	58.5	0.65	0.684	1.759	2.6	0.00
68	132.6	Saturated	Saturated	36	70	0.0	0	8893.6	5417.9	52.5	0.65	0.681	1.749	2.6	0.00
69	132.6	Saturated	Saturated	36	70	0.0	0	9026.2	5488.1	52.3	0.64	0.677	1.740	2.6	0.00
70	132.6	Saturated	Saturated	36	70	0.0	0	9158.8	5558.3	52.1	0.64	0.673	1.731	2.6	0.00
71	132.6	Saturated	Saturated	36	70	0.0	0	9291.4	5628.5	52.0	0.63	0.670	1.722	2.6	0.00
72	132.6	Saturated	Saturated	36	70	0.0	0	9424.0	5698.7	51.8	0.63	0.666	1.713	2.6	0.00
73	137.2	Saturated	Saturated	45	75	0.0	0	9561.2	5773.5	64.5	0.63	0.663	1.704	2.6	0.00
74	137.2	Saturated	Saturated	45	75	0.0	0	9698.4	5848.3	64.3	0.62	0.659	1.695	2.6	0.00
75	137.2	Saturated	Saturated	45	75	0.0	0	9835.6	5923.1	64.1	0.62	0.655	1.686	2.6	0.00
76	137.2	Saturated	Saturated	45	75	0.0	0	9972.8	5997.9	63.9	0.61	0.652	1.677	2.6	0.00
77	137.2	Saturated	Saturated	45	75	0.0	0	10110.0	6072.7	63.7	0.61	0.648	1.668	2.6	0.00
78	125.7	Saturated	Saturated	73	80	0.0	0	10235.7	6136.0	103.0	0.61	0.646	1.660	2.6	0.00
79	125.7	Saturated	Saturated	73	80	0.0	0	10361.4	6199.3	102.7	0.60	0.643	1.653	2.6	0.00
80	125.7	Saturated	Saturated	73	80	0.0	0	10487.1	6262.6	102.5	0.60	0.640	1.646	2.6	0.00
81	125.7	Saturated	Saturated	73	80	0.0	0	10612.8	6325.9	102.2	0.59	0.637	1.638	2.6	0.00
82	125.7	Saturated	Saturated	73	80	0.0	0	10738.5	6389.2	101.9	0.59	0.634	1.631	2.6	0.00
83	137.6	Saturated	Saturated	50	85	0.0	0	10876.1	6464.4	69.6	0.59	0.631	1.623	2.6	0.00
84	137.6	Saturated	Saturated	50	85	0.0	0	11013.7	6539.6	69.4	0.58	0.628	1.615	2.6	0.00
85	137.6	Saturated	Saturated	50	85	0.0	0	11151.3	6614.8	69.2	0.58	0.625	1.607	2.6	0.00
86	137.6	Saturated	Saturated	50	85	0.0	0	11288.9	6690.0	69.0	0.58	0.622	1.598	2.6	0.00
87	137.6	Saturated	Saturated	50	85	0.0	0	11426.5	6765.2	68.8	0.57	0.620	1.590	2.6	0.00
88	136.8	Saturated	Saturated	50	90	0.0	0	11563.3	6839.6	68.6	0.57	0.617	1.583	2.6	0.00
89	136.8	Saturated	Saturated	50	90	0.0	0	11700.1	6914.0	68.4	0.57	0.614	1.575	2.6	0.00
90	136.8	Saturated	Saturated	50	90	0.0	0	11836.9	6988.4	68.2	0.56	0.612	1.567	2.6	0.00

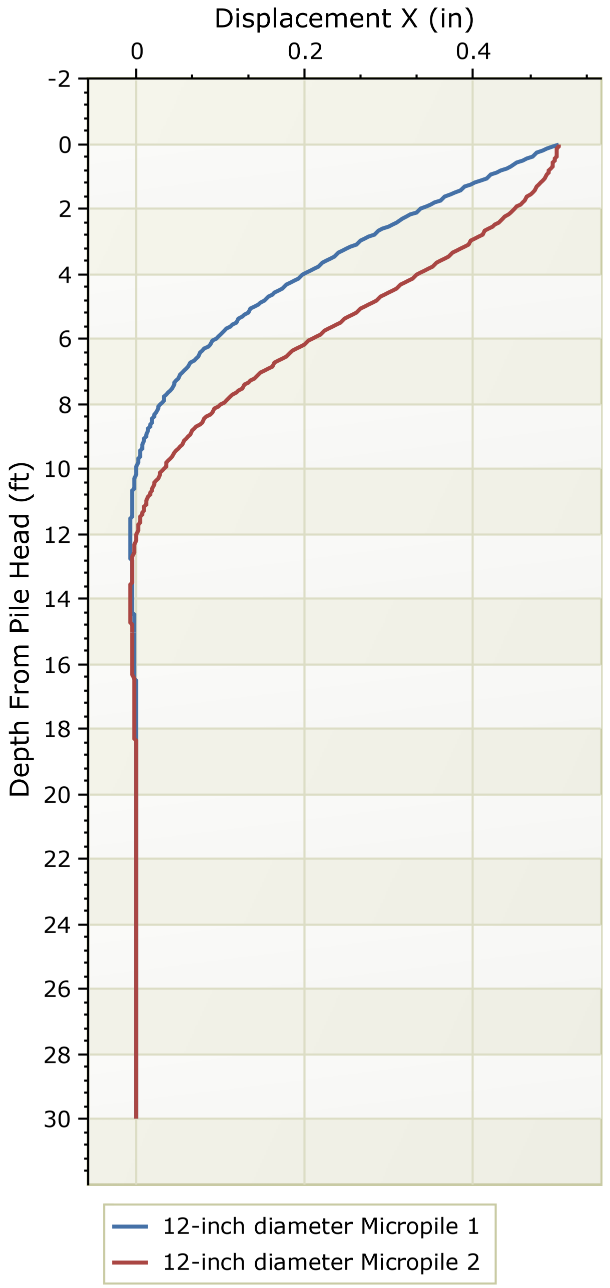
Total Liquefaction Settlement, S = 0.00 inches

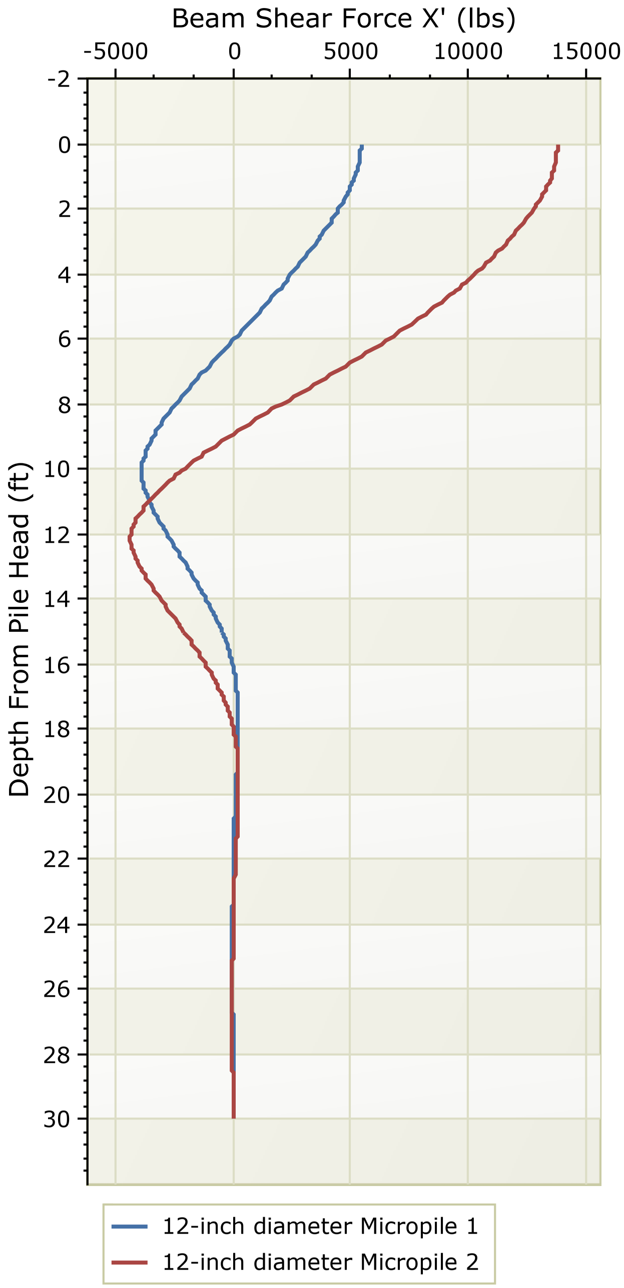


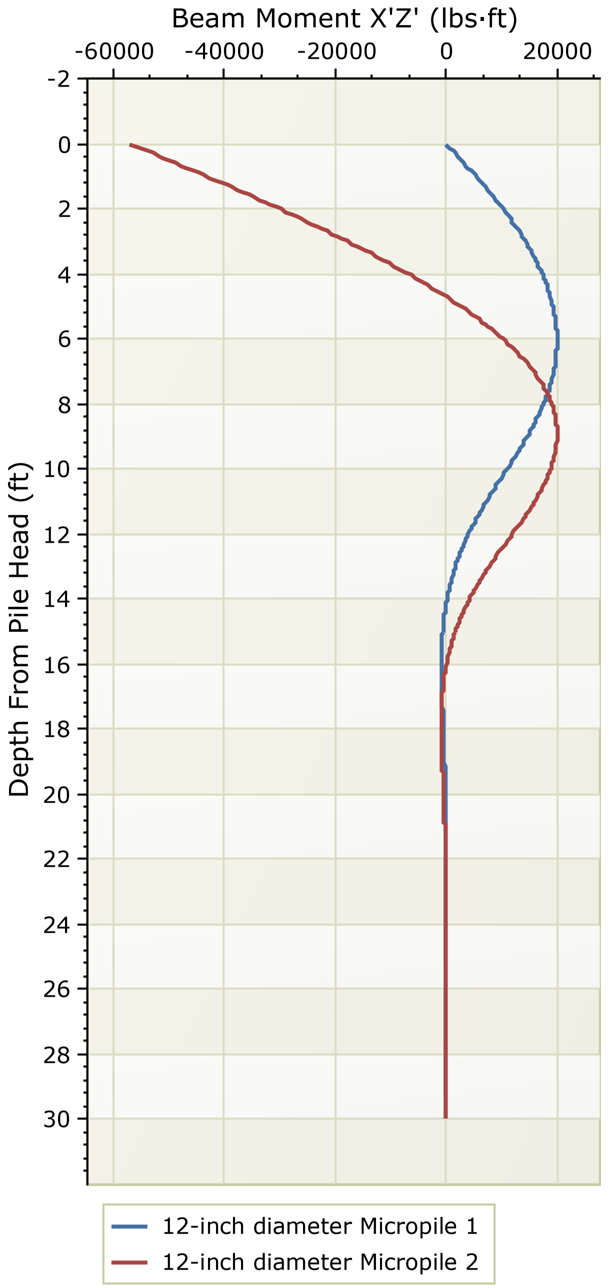




rocscience	*Project*	File No. 22307 - BARDAS TVC North	
	Analysis Description	Lateral Pile Analysis - Micropiles	
	Drawn By SST	*Company*	Geotechnologies, Inc.
RSPILE 3.014	*Date* 11/16/202	*File Name*	22307 Midropile - Lateral Pile Capacity-2.rspile2







rocscience

RSPILE 3.014

Project	File No. 22307 - BARDAS TVC North		
Analysis Description	Lateral Pile Analysis - Micropiles		
Drawn By	SST	*Company*	Geotechnologies, Inc.
Date	11/16/202	*File Name*	22307 Midropile - Lateral Pile Capacity-2.rspile2

APPENDIX IV

PRIOR REPORT

[redacted]

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVE. # 201, GLENDALE, CA 9[illegible]6
818•549•9959 TEL 818•543•3747 FAX

TRANSMITTAL

May 13, 1999
JB 18051-B

Television Center, Inc.
6311 Romaine Street
Hollywood, California 90038

Attention: Ana Ramirez

Subject

Geotechnical Exploration
Proposed Parking Structure, Commercial Buildings, and Additions
6311 Romaine Street
Hollywood, California

Gentlepersons:

Transmitted herewith is our geotechnical engineering report with respect to construction of the parking structure, commercial buildings, and additions at Television Center in Hollywood. Our tests borings indicate that the site is underlain by a surface layer of fill placed during past development of the site. The fill is on the order of two to five feet thick. Below the fill are natural alluvial soils that are strong and capable of supporting the proposed development. Conventional spread footings are recommended for support of the structures. The property is not subject to liquefaction and is not within any special studies zone for active earthquake faults.

The reviewing agency for this document is the City of Los Angeles Building Department. They require three copies of the report along with an application form and a filing fee. City review is expected to take four weeks. Copies of the report have been distributed as follows:

(1) Television Center, Inc., Attention: Ana Ramirez
(6) Turkel Architecture Group, Attention: Steven Turkel

It is our understanding that your architect will file the report with the City of Los Angeles. It is suggested that you read the report carefully prior to submittal to any reviewing agency. Any questions concerning the data or interpretation of the report should directed to the undersigned. The J. Byer Group appreciates the opportunity to offer our consultation and advice on this project. An invoice for this work has been included with the report copy sent to Television Center, Inc.

Very Truly Yours,
THE J. BYER GROUP, INC.

John W. Byer
President

xc: (1) Addressee
(1) Steve Turkel

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVE. # 201, GLENDALE, CA 91206
818•549•9959 TEL 818•543•3747 FAX

GEOTECHNICAL EXPLORATION
PROPOSED PARKING GARAGE, COMMERCIAL STRUCTURES AND ADDITIONS
EXISTING TELEVISION CENTER
6311 ROMAINE STREET
HOLLYWOOD, CALIFORNIA
FOR TELEVISION CENTER, INC.
THE J. BYER GROUP, INC. PROJECT NUMBER JB 18051-B
MAY 13, 1999

GEOTECHNICAL EXPLORATION
PROPOSED PARKING GARAGE, COMMERCIAL STRUCTURES AND ADDITIONS
EXISTING TELEVISION CENTER
6311 ROMAINE STREET
HOLLYWOOD, CALIFORNIA
FOR TELEVISION CENTER, INC.
THE J. BYER GROUP, INC. PROJECT NUMBER JB 18051-B
MAY 13, 1999

INTRODUCTION

Per your authorization, The J. Byer Group has performed geotechnical exploration at the property. The following report summarizes findings of the exploration and provides recommendations for development of the site. The purpose of this study is to evaluate the nature, distribution, and engineering properties of the earth materials underlying the site with respect to construction of a six story parking garage, two story commercial buildings, and additions to the existing facility.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

EXPLORATION

The scope of the field exploration was determined following our initial site visit and consultation with Mr. Steve Turkel, Project Architect. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Site Plan. Conditions affecting portions of the property outside the area explored, are beyond the scope of this report.

Exploration was conducted on April 27 and 28, 1999 with the aid of a hollow stem auger drill rig. It included advancing 12 borings to depths ranging from 15 to 50 feet below the ground surface. Samples of the earth materials were obtained at frequent intervals and returned to our soils engineering laboratory for testing and analysis.

Office tasks included laboratory testing of selected soil samples, review of our files with respect to geotechnical explorations in the area, review of State of California Seismic Hazard Maps, engineering analysis, preparation of the enclosed Site Plan, and the preparation of this report. The earth materials encountered in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project and the locations of the borings are shown on the Site Plan.

PROPOSED DEVELOPMENT

Information concerning the proposed project was provided by Mr. Steve Turkel, Architect. It is proposed to construct two small additions to the existing facility. One at the corner of Cahuenga Boulevard and Santa Monica Boulevard, and one in the interior area. These additions are planned to be one or two story wood frame structures. In the southern portion of the property, it is proposed to construct a six story parking structure. To the north of that structure will be two story wood frame structures. Foundation loads for the parking structure are expected to be relatively high. Foundation loads for the small commercial buildings and additions are expected to be low. The proposed project is considered preliminary at this time and formal plans have not been prepared.

SITE DESCRIPTION

The subject property is located in the Hollywood area of the City of Los Angeles. It fronts on Santa Monica Boulevard to the north, Cahuenga Boulevard on the east, Willoughby Avenue to the south and Cole Avenue on the west. The property consists of two square blocks divided by Romaine Street. The northern block is developed with numerous buildings, including the original Technicolor reinforced concrete vault building. Other additions are one and two story, small production studios. To the south is a surface level asphalt parking lot, which covers the entire block. The parking lot is essentially level but is elevated two to three feet above the surrounding streets. The surface of the parking lot is in moderate to poor condition. Drainage on the property is generally by sheetflow runoff down the natural slope of the land to the fronting streets.

GROUNDWATER

Groundwater was encountered in Borings 5, 6, and 9. In Boring 5, groundwater was seeping into the boring a depth of 15 feet. This boring is adjacent to a broken water main in Cole Avenue, which was under repair at the time of this exploration. It is probable that leakage from the water line is the source of the water. A confined layer of groundwater was found in Boring 6 at a depth of 26½ feet below grade. The water rose to 21½ feet below grade one hour after completion of the boring. A similar confined layer was encountered in Boring 9 at a depth of 24 feet below grade. The water rose to 16 feet below grade one hour after completion of the boring. The groundwater levels discussed are confined to sandy layers within the natural soil and do not represent a permanent water table. As indicated by the borings, fluctuations in groundwater levels may also occur due to conditions not evident at the time of exploration. Fluctuations may also occur across the site.

EARTH MATERIALS

Fill

A thin layer of fill, associated with previous site development and demolition, covers the entire property. Two feet of fill was encountered in Borings 8 and 12 where the proposed additions are planned. Two to five feet of fill was found below the existing asphalt parking lot in the southern portion of the property. The fill consists of various blends of silty sands, sandy gravel, and clay and contains construction and demolition debris, including concrete and brick fragments.

Alluvium

Natural alluvial soils derived as outwash from the south flank of the eastern Santa Monica Mountains underlies the entire property. The alluvium consists of an upper layer of dark brown to black sandy clay that is moist and firm to stiff. The upper clay rich layer is two to three feet thick. Below the clay, the alluvium becomes sandier and grades to silty sand and clayey sand that are brown to reddish brown, moist and dense. A persistent silt layer is found between 10 and 15 feet below the surface grade. The silt is light gray brown, moist to very moist, and firm to stiff. The silt bed rests on coarser sands, silty sands, and gravelly sands beginning at a depth of 15 feet. These sandy layers then grade into clayey sand and silty sand with depth.

Bedrock

Bedrock was not encountered in the borings to the depths explored. Sedimentary bedrock typical of the eastern Santa Monica Mountains is exposed approximately one mile north of the property.

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Division of Mines and Geology, private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies are shifting their focus to earthquake resistant structures as opposed to prediction. The purpose of the code seismic design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Within the past 25 years, southern California and vicinity have experienced an increase in seismic activity beginning with the San Fernando earthquake in 1971. In 1987, a moderate earthquake struck the Whittier area and was located on a previously unknown fault. Ground shaking from this event caused substantial damage to the City of Whittier, and surrounding cities.

The January 17, 1994, Northridge earthquake was initiated along a previously unrecognized fault below the San Fernando Valley. The energy released by the earthquake propagated to the southeast, northwest, and northeast in the form of shear and compression waves, which caused the strong ground shaking in portions of the San Fernando Valley, Simi Valley, City of Santa Clarita, and City of Santa Monica.

Southern California faults are classified as: active, potentially active, or inactive. Faults from past geologic periods of mountain building, but do not display any evidence of recent offset, are considered "potentially active". Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults". There are no known active faults within close vicinity of the subject property.

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including strapping water heaters and securing furniture. It is likely that the subject property will be shaken by future earthquakes produced in southern California. However, secondary effects such as surface rupture, lurching, liquefaction, consolidation, ridge shattering, and landsliding should not occur at the subject property.

Liquefaction

The subject property is not located within a liquefaction zone as indicated on the Hollywood Quadrangle (Official Map) of the State of California Seismic Hazard Zones, released March 25, 1999. Liquefaction is a condition where soil experiences deformation at constant low residual stress or with a low residual resistance due to build up and maintenance of high pore water pressures. Liquefaction can occur in either a static or dynamic condition and the possibility of occurrence depends upon the void ratio, relative density, and confining pressure of the soil. There are four general conditions necessary for liquefaction to occur. These include a high groundwater table, fine grained cohesionless soils, a low relative density, and strong ground shaking. The borings indicate that the subject property is underlain by dense alluvial soils that are not subject to liquefaction. The groundwater levels encountered are perched and confined layers that are locally discontinuous. It is the opinion of The J. Byer Group that the subject property is not subject to liquefaction.

SITE SEISMIC CONSIDERATIONS

Southern California is placed in Seismic Zone IV per the Uniform Building Code. The nearest mapped fault is the Hollywood, located approximately one mile north of the project. The Hollywood fault is not currently zoned as an active fault as determined by the Alquist-Priolo

Special Studies Zone Maps. The Hollywood fault is currently being studied by local universities and recent data may indicate the fault is active. The Hollywood fault has no history of seismic activity within the past 200 years. The nearest capable fault is considered the Newport-Inglewood fault, which crosses through the Baldwin Hills approximately five miles south of the property. The Newport-Inglewood fault can produce earthquakes in the moment magnitude range of 6.0 to 7.0.

The alluvial soils on the site are characterized as S_2 stiff soil per Table 16-J of 1994 Uniform Building Code. The alluvial soils are considered very dense soil S_c per Table 16-J of the 1997 Uniform Building Code. The nearest source factors N_a and N_v are 1.3 and 1.6, respectively for a type "B" fault (Hollywood) less than two kilometers north of the site.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon 12 borings, research of records, analysis of laboratory test data, consultation with your architect, and over 30 years experience providing geotechnical explorations on similar properties on similar settings. It is the finding of The J. Byer Group, Inc. that construction of the proposed project is feasible from a geotechnical engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is the natural alluvial soil found at depths ranging between two and five feet below the existing surface grade. Removal and recompaction of the surface fill may be required in the area of the additions depending upon the type of construction. For slab-on-grade, the fill inside of the building line should be removed to the natural grade and replaced as approved compacted fill. The proposed parking garage area may also require remedial grading for slab support depending upon the final floor elevation. The proposed lease areas may require removal and recompaction of all of the fill in order to provide foundation support. The following section provides general grading recommendations with respect to site preparation.

SITE PREPARATION

Surficial materials consisting of fill are present on the site. Remedial grading may be necessary to improve site conditions for foundation and floor slab support.

General Grading Specifications

The following guidelines may be used in preparation of the grading plan and job specifications. The J. Byer Group would appreciate the opportunity of reviewing the plans to insure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The site to receive compacted fill should be prepared by removing all existing fill. The exposed excavated area should be observed by the soils engineer prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum density.

B. If the compacted fill is to be utilized for foundation support the proposed building site shall be excavated to a minimum depth of three feet below the bottom of all footings or the depth of fill, whichever is deeper. The excavation should extend five feet beyond the building footprint as shown on the Site Plan. The excavated areas should be observed by the soils engineer prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts and compacted in six inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. The maximum density shall be determined by ASTM D 1557-91 or equivalent.

E. Field observation and testing shall be performed by the soils 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 90 percent compaction is obtained. One compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

F. It is estimated that the fill, when removed and replaced as compacted fill, will shrink in volume approximately five to ten percent. Imported soil may be necessary to complete the grading and achieve the finished grades.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads.

Spread Footings

Continuous and pad footings may be used to support the proposed additions, commercial buildings, and parking structure provided they are founded in the natural alluvial soil or approved compacted fill. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24 inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Alluvium	18	2,500	0.3	250	5,000
Future Compacted Fill	18	2,000	0.3	200	4,000

Increases in the bearing value are allowable at a rate of 20 percent for each additional foot of footing width or depth to a maximum of 4,000 pounds per square foot for compacted fill and 5,000 pounds per square foot for natural alluvium. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing values shown 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. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

All continuous footings should be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geotechnical engineer prior to placing forms, steel or concrete.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated. Differential settlement should not exceed ¼ inch.

RETAINING WALLS

General Design

Currently, retaining walls are not proposed for the project. However, should retaining walls be used the following design recommendations should be incorporated into the plans. Retaining walls up to 10 feet high, and with a level backslope may be designed for a minimum equivalent fluid pressure of 43 pounds per cubic foot. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾ inch crushed gravel.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾ inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted fill blanket to the surface. Retaining wall backfill should be capped with a paved surface drain.

Foundation Design

Retaining wall footings may be sized per the"Spread Footings" section of this report.

FLOOR SLABS

For slab-on-grade construction, the existing fill should be removed to expose the natural alluvium, and replaced as approved compacted fill. In the City of Los Angeles, slab-on-grade construction supported by approved compacted fill requires a minimum reinforcement of #4 steel bars at 16 inches on center, each way. Slabs which will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be covered with a thin layer of sand, about one inch, to prevent punctures and aid in the concrete cure.

PAVING

Prior to placing paving, the existing fill should be removed to the natural alluvial soil, and replaced as approved compacted fill, in accordance with the Site Preparation section of this report. Any trench backfill below paving, should be compacted to 90 percent of the laboratory maximum density. Irrigation water should be prevented from migrating below paving. For light passenger cars the recommend paving section is three inches of asphalt over four inches of compacted base. For heavy use, including truck lanes and storage, the recommended paving section is four inches of asphalt over six inches of compacted base.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised floor type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾ inch crushed gravel to help the collection of water. Yard areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

Construction of raised floor buildings where the grade under the floor has been lowered for joist clearance can also lead to moisture problems. Surface moisture can seep through the footing and pond in the underfloor area. Positive drainage away from the footings, waterproofing the footings, compaction of trench backfill and subdrains can help to reduce moisture intrusion.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by The J. Byer Group. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The Building Department requires that the geotechnical company provide site observations during construction. The observations include bottoms prior to placing fill, compaction of fill, and foundation excavations. All fill that is placed should be tested for compaction and approved by the soils engineer prior to use for support of engineered structures. The City of Los Angeles requires that all retaining wall subdrains be observed by a representative of the geotechnical company and the City Inspector.

Please advise The J. Byer Group, Inc. at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the jobsite and available to our representative. The project consultant will perform the observation and post a notice at the jobsite of his visit and findings. This notice should be given to the agency inspector.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions and excavation characteristics described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications or recommendations during construction requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

The J. Byer Group appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
THE J. BYER GROUP, INC.

John W. Byer
President

Robert I. Zweigler
G.E. 2120

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 6-30-02
GEOTECHNICAL
STATE OF CALIFORNIA

JWB:RIZ:flh
G:\FINAL\REPORTS\18051-B.RPT

Enc: Appendix I - Laboratory Testing
Shear Test Diagrams (2)
Consolidation Curves (12)
Vicinity Map
Log of Borings (15 Pages)

In Pocket: Site Plan

xc: (1) Addressee
(6) Steve Turkel, Architect

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill and alluvium were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring lined barrel sampler conforming to ASTM D-3550 with successive drops of the hammer. Experience has shown that sampling causes some disturbance of the sample, however the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The central portions of the samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D-2937. The moisture content of the samples was determined using the procedures outlined in ASTM D-2216. The results are shown on the Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill was determined by remolding a bulk sample of the existing fill using the procedures outlined in ASTM D 1557, a five-layer standard. Remolded samples were prepared at 90 percent of the maximum dry density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
1	2	Brown Sandy Clay	129.0	14.8	48

Expansion Test

To find the expansiveness of the future compacted fill, an expansion index test was performed. Based upon the testing, the future compacted fill will be slightly to moderately expansive.

Shear-Tests

Shear tests were performed on samples of the remolded fill and natural alluvium using the procedures outlined in ASTM D-3080 and a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the "Shear Test Diagrams".

Consolidation

Consolidation tests were performed on insitu samples of the alluvium. Results are graphed on the "Consolidation Curves".

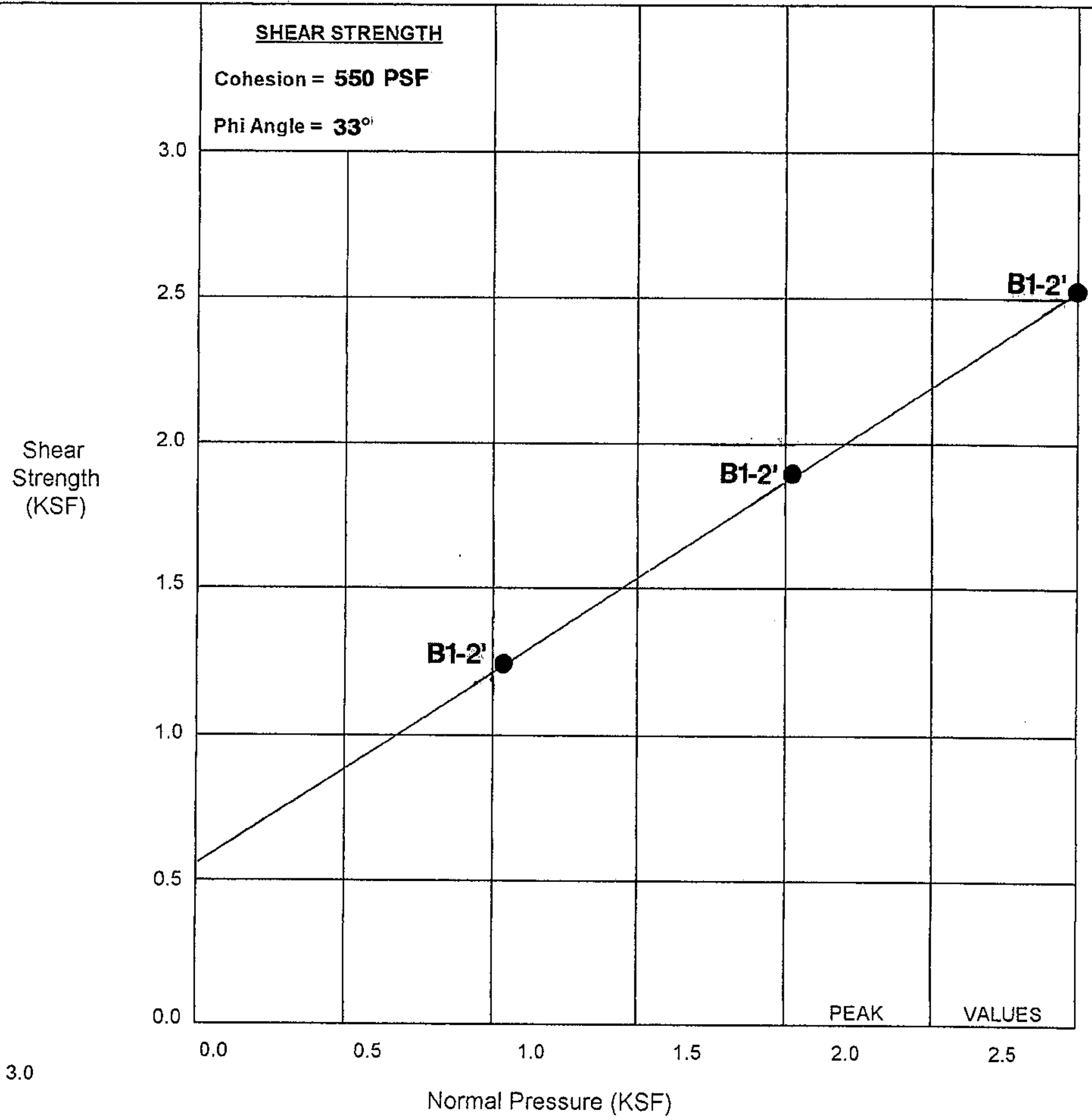
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

SHEAR TEST DIAGRAM

JB: 18051-B Television Center

SAMPLE: Future Compacted Fill

Samples remolded to 90% of maximum density



SHEAR STRENGTH

Cohesion = 550 PSF

Phi Angle = 33°

○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **19.5**

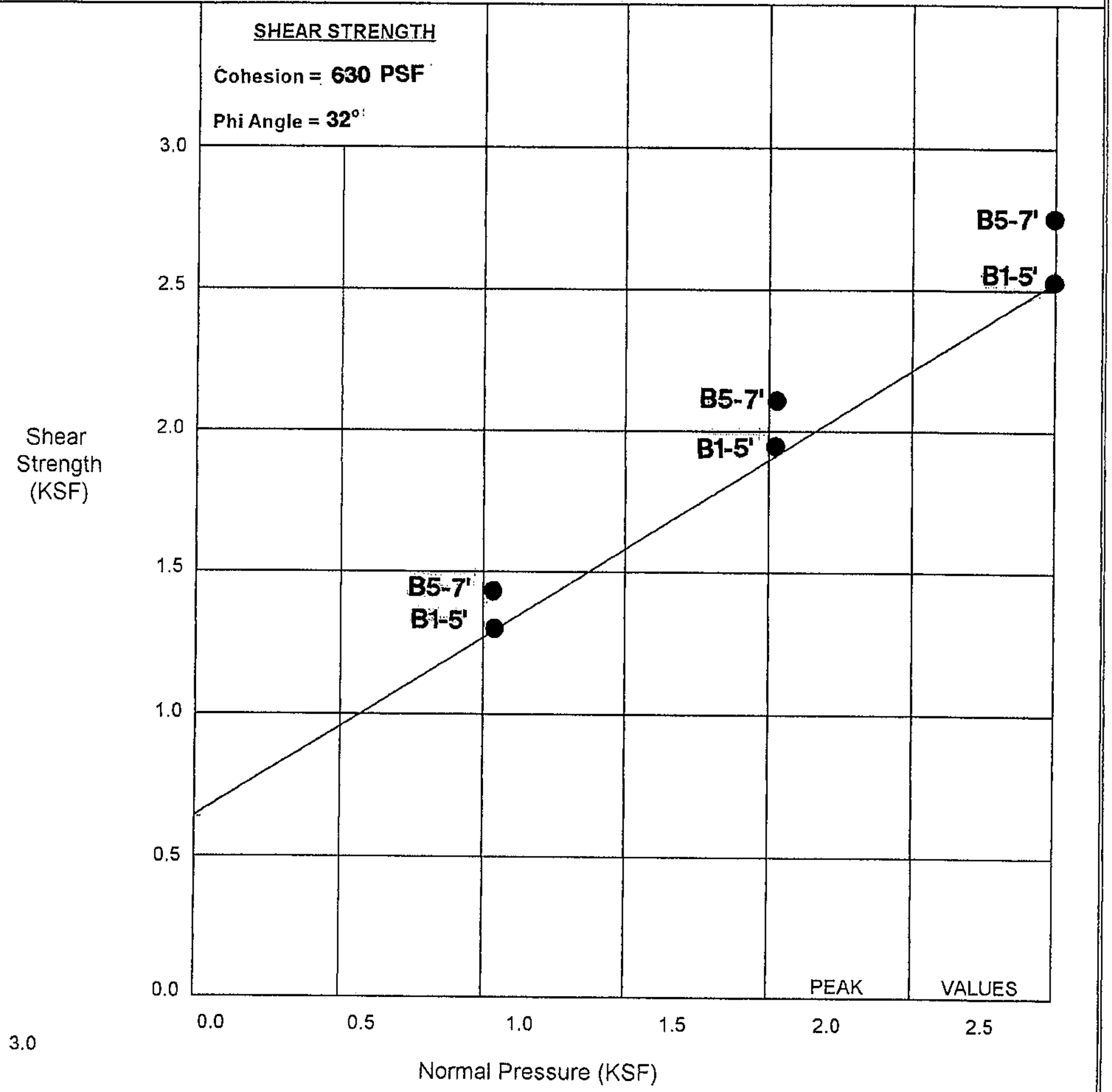
Dry Density (pcf) = **118.0**

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

SHEAR TEST DIAGRAM

JB: 18051-B Television Center

SAMPLE: Alluvium


SHEAR STRENGTH

Cohesion = **630 PSF**

Phi Angle = **32°**

○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **18.6**

Dry Density (pcf) = **120.6**

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

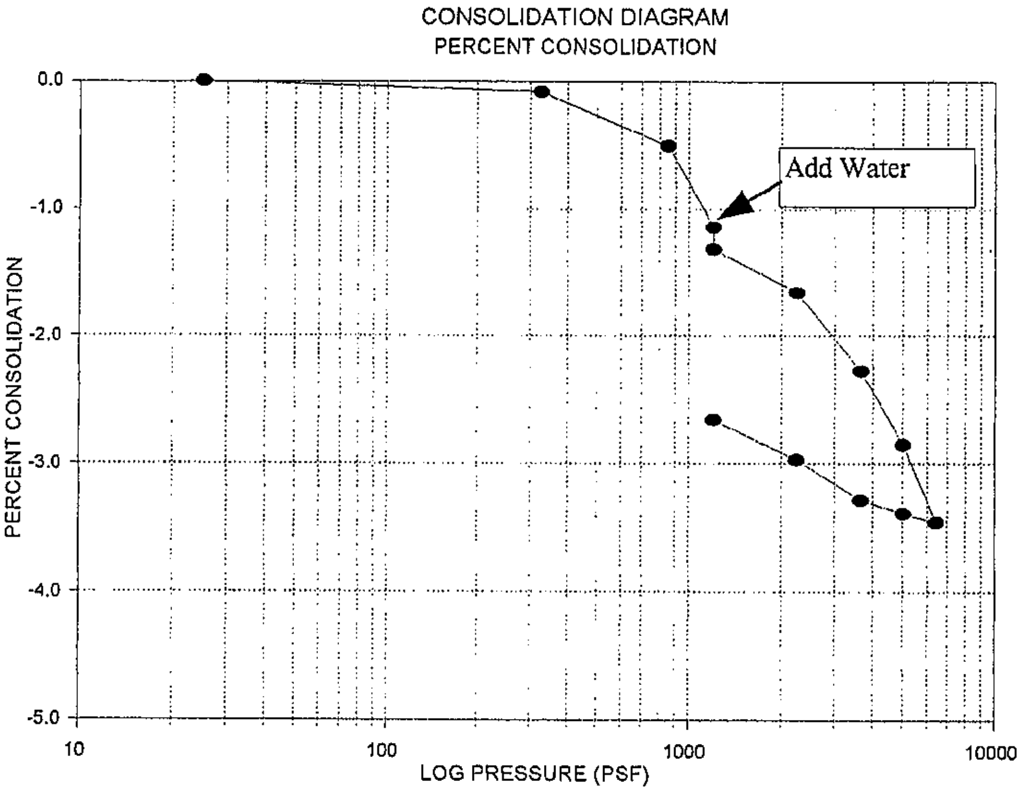
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B2-5'



Dry Density	97.4 pcf	Specific Gravity	2.65
Initial Moisture	10.2%	Initial Void Ratio	0.70
Initial % Saturation	38.7%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

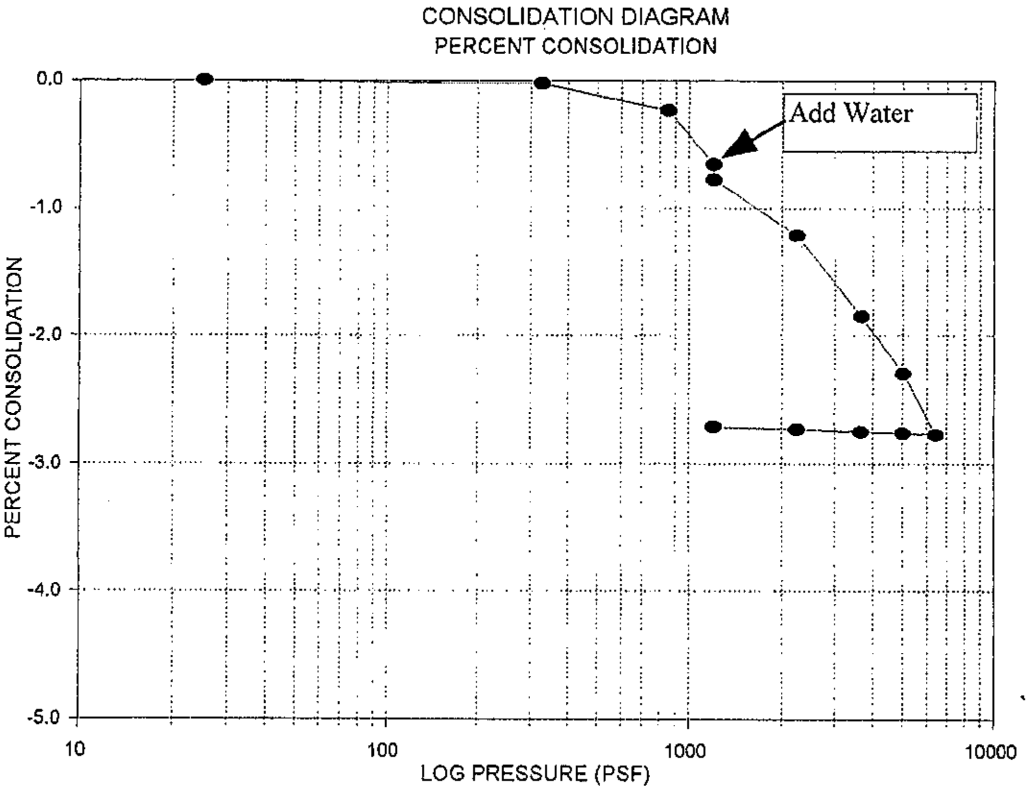
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B1-3'



Dry Density	115.4 pcf	Specific Gravity	2.65
Initial Moisture	15.7%	Initial Void Ratio	0.43
Initial % Saturation	96.1%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

CONSOLIDATION DIAGRAM

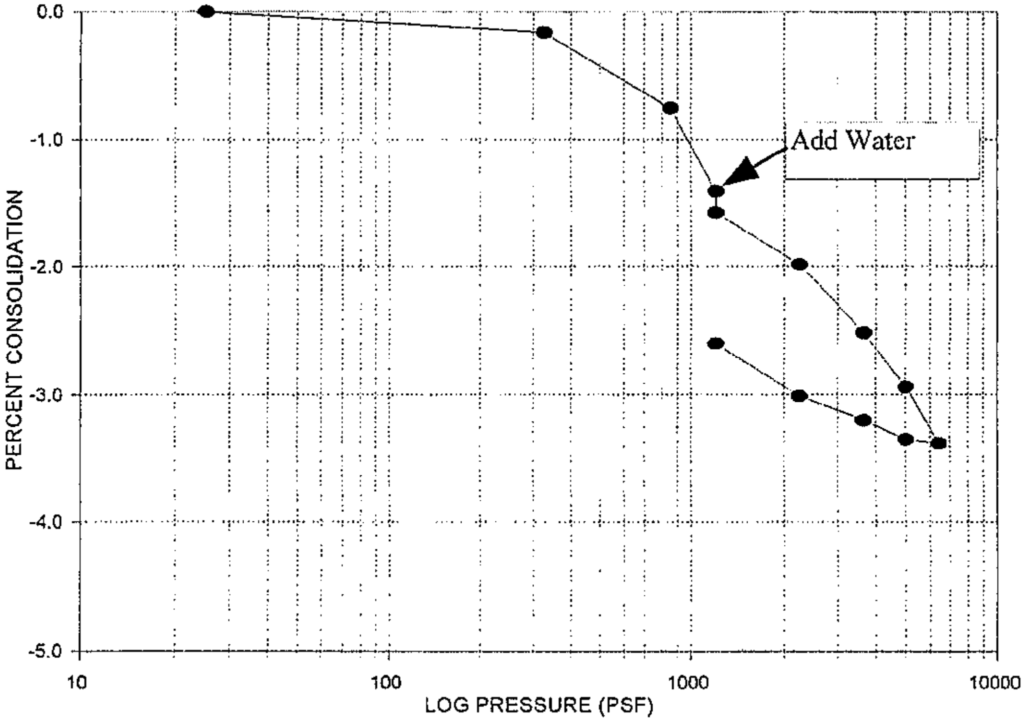
JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B4-7'

CONSOLIDATION DIAGRAM
PERCENT CONSOLIDATION



Dry Density	104.0 pcf	Specific Gravity	2.65
Initial Moisture	21.2%	Initial Void Ratio	0.59
Initial % Saturation	95.2%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

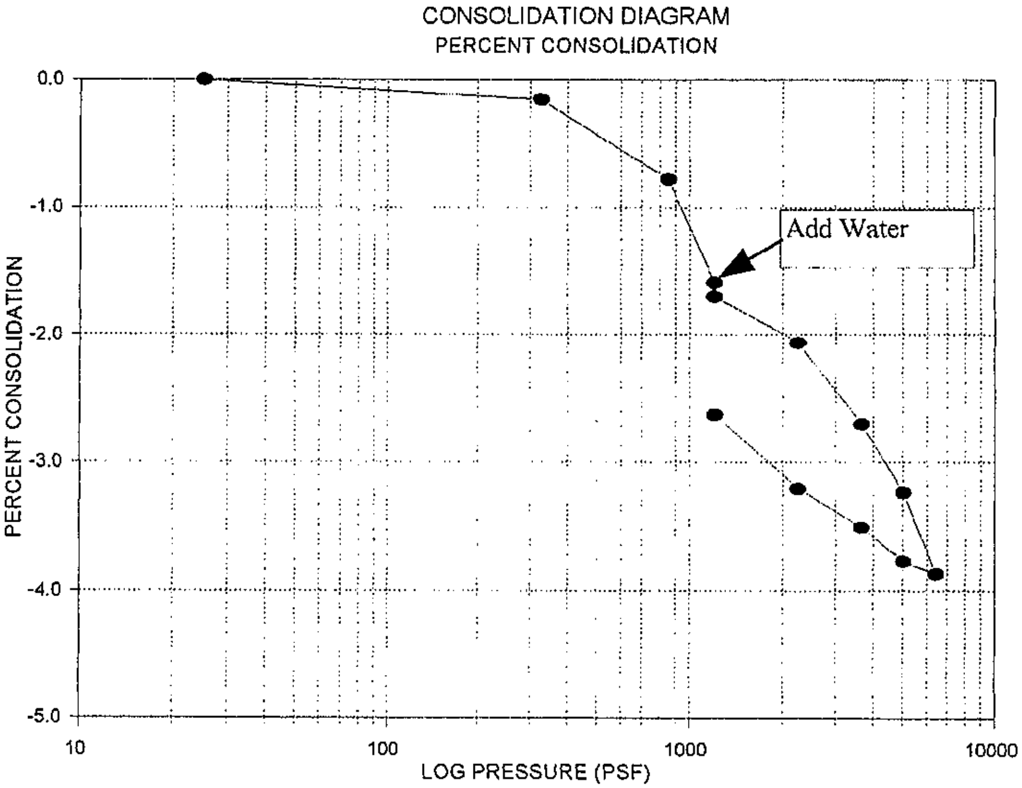
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B4-10'



Dry Density	97.4 pcf	Specific Gravity	2.65
Initial Moisture	27.0%	Initial Void Ratio	0.70
Initial % Saturation	102.5%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

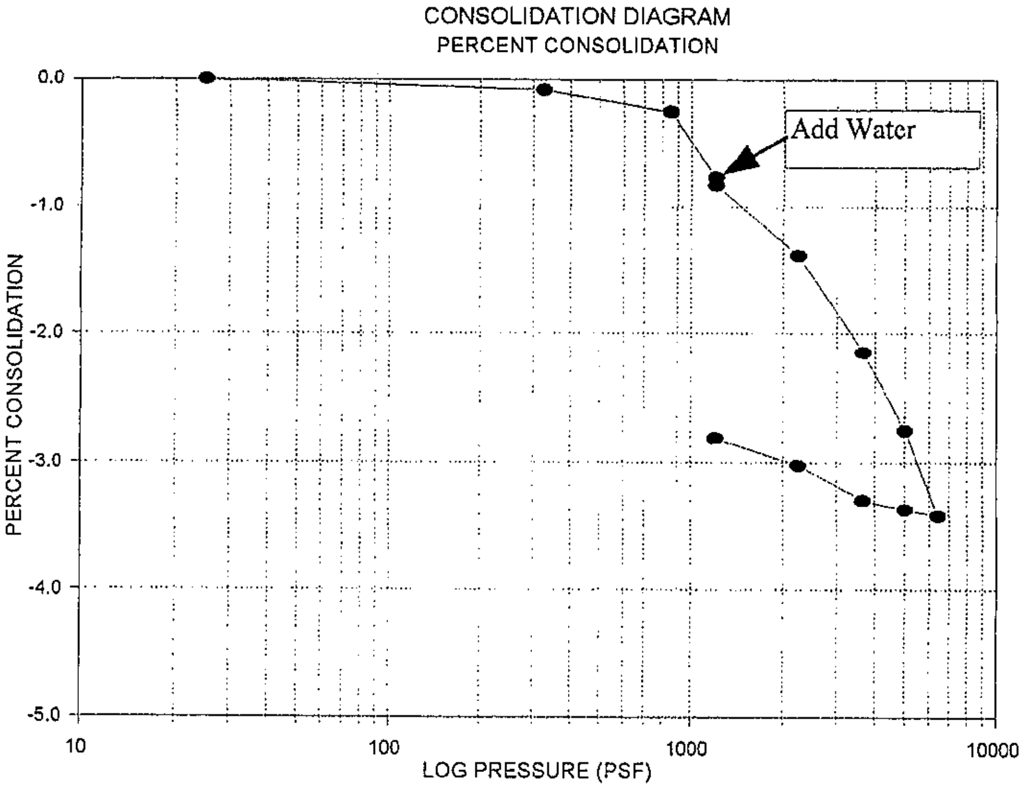
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B5-5'



Dry Density	112.4 pcf	Specific Gravity	2.65
Initial Moisture	14.9%	Initial Void Ratio	0.47
Initial % Saturation	83.8%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

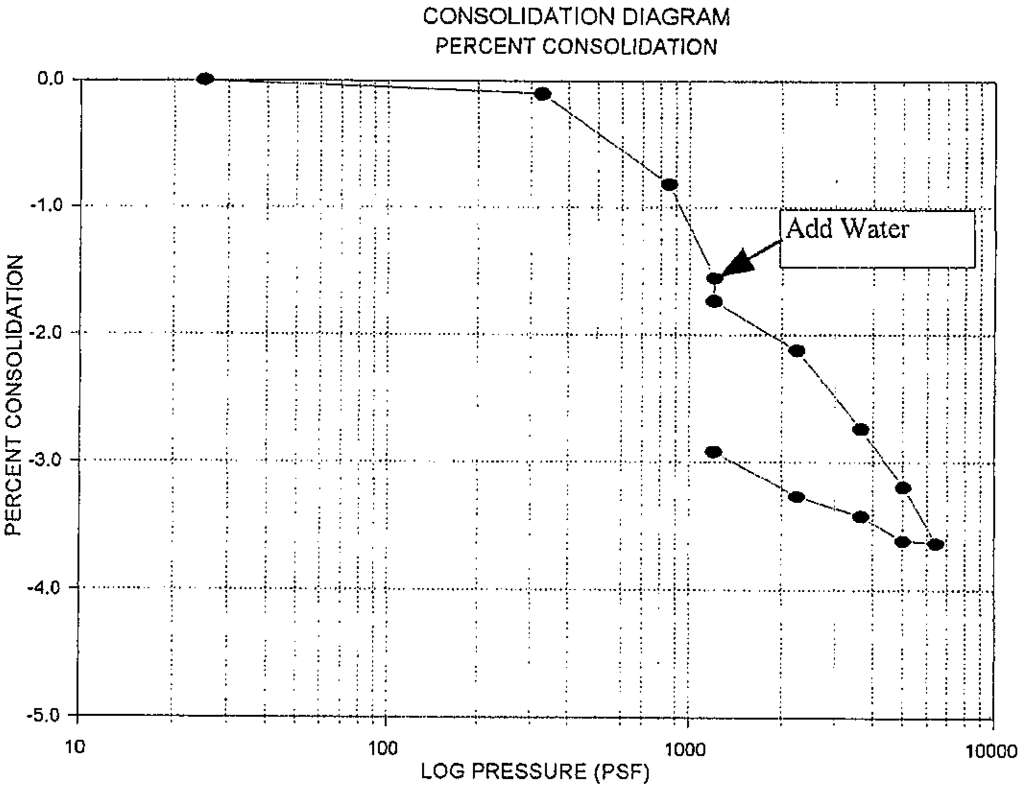
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B6-6'



Dry Density	109.2 pcf	Specific Gravity	2.65
Initial Moisture	19.4%	Initial Void Ratio	0.51
Initial % Saturation	100.0%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

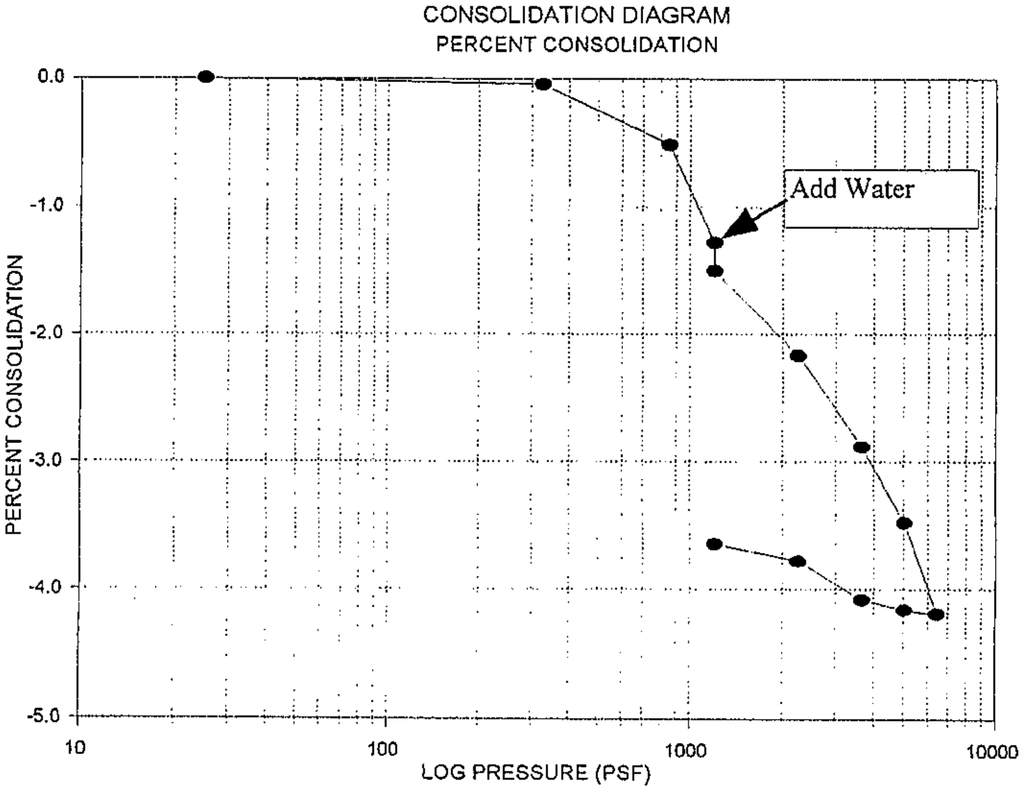
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B8-2'



Dry Density	105.2 pcf	Specific Gravity	2.65
Initial Moisture	20.3%	Initial Void Ratio	0.57
Initial % Saturation	94.1%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

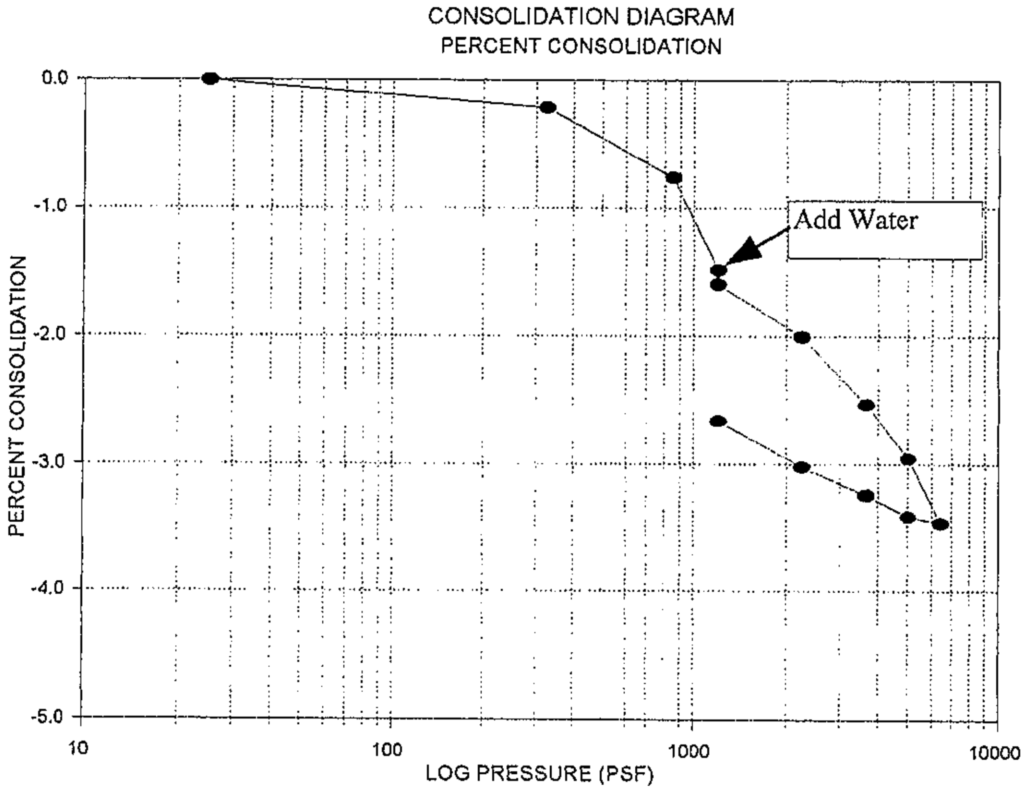
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B9-6'



Dry Density	104.7 pcf	Specific Gravity	2.65
Initial Moisture	21.1%	Initial Void Ratio	0.58
Initial % Saturation	96.5%	C'c	0.03

THE J. BYER GROUP, INC.

A GEOTECHNICAL CONSULTING FIRM

512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

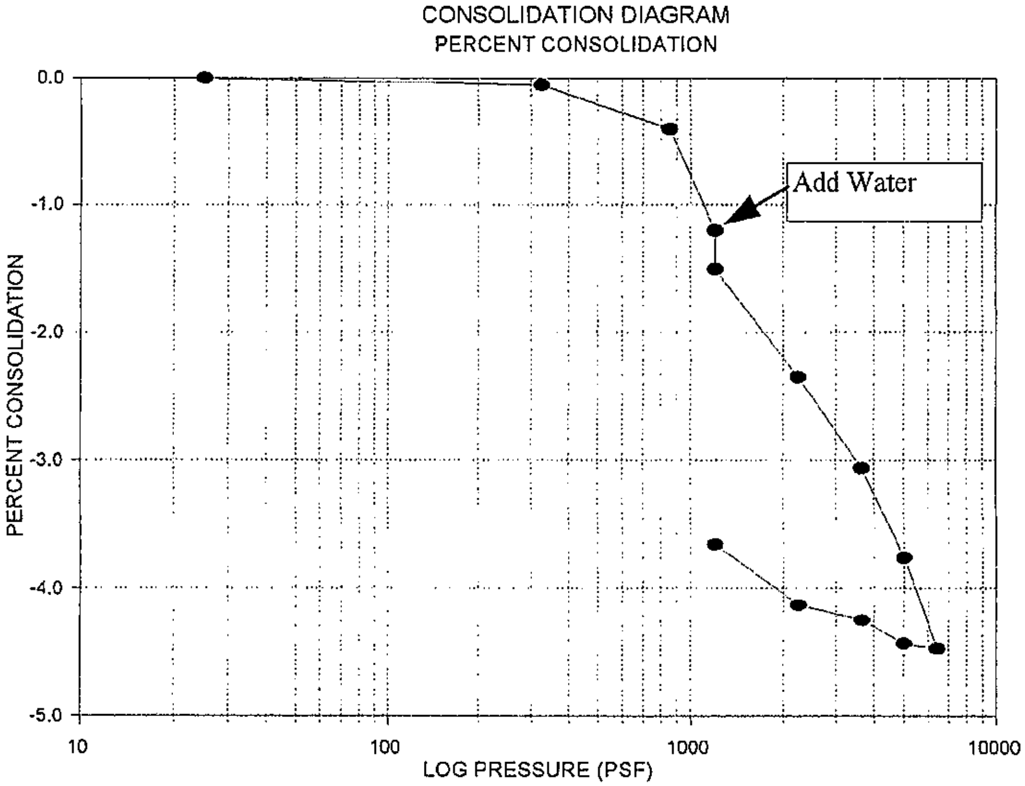
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B12-5'



Dry Density	106.8 pcf	Specific Gravity	2.65
Initial Moisture	19.2%	Initial Void Ratio	0.55
Initial % Saturation	92.8%	C'c	0.05

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

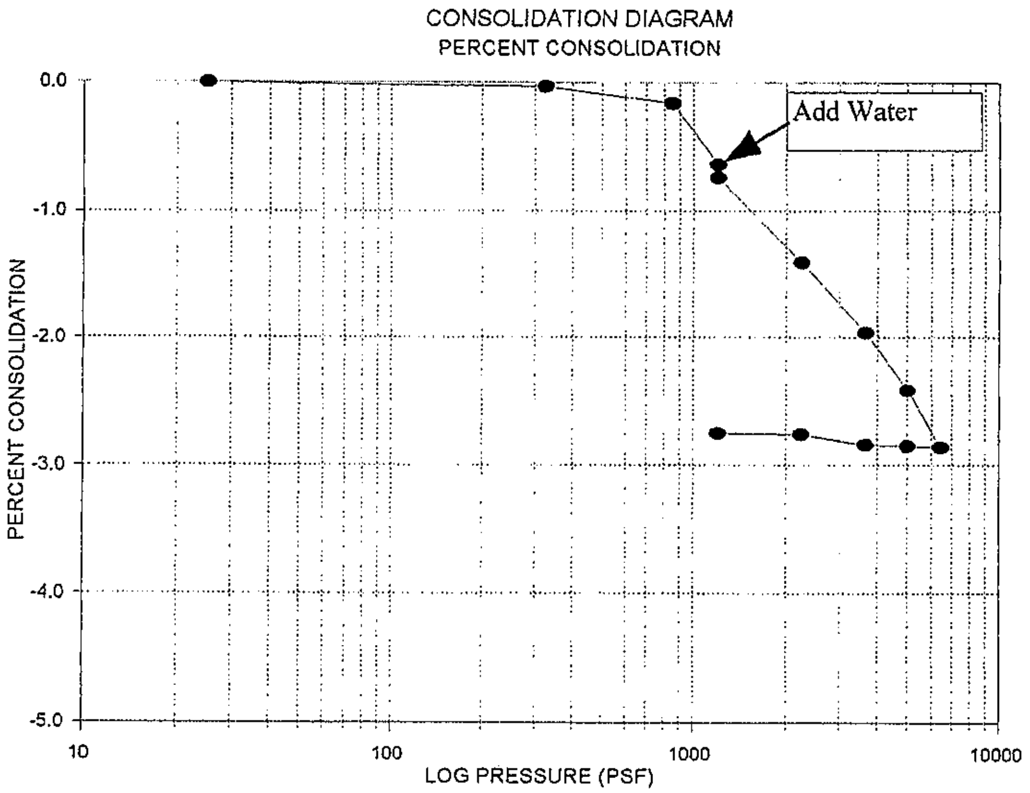
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B10-10'



Dry Density	110.6 pcf	Specific Gravity	2.65
Initial Moisture	12.8%	Initial Void Ratio	0.50
Initial % Saturation	68.5%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

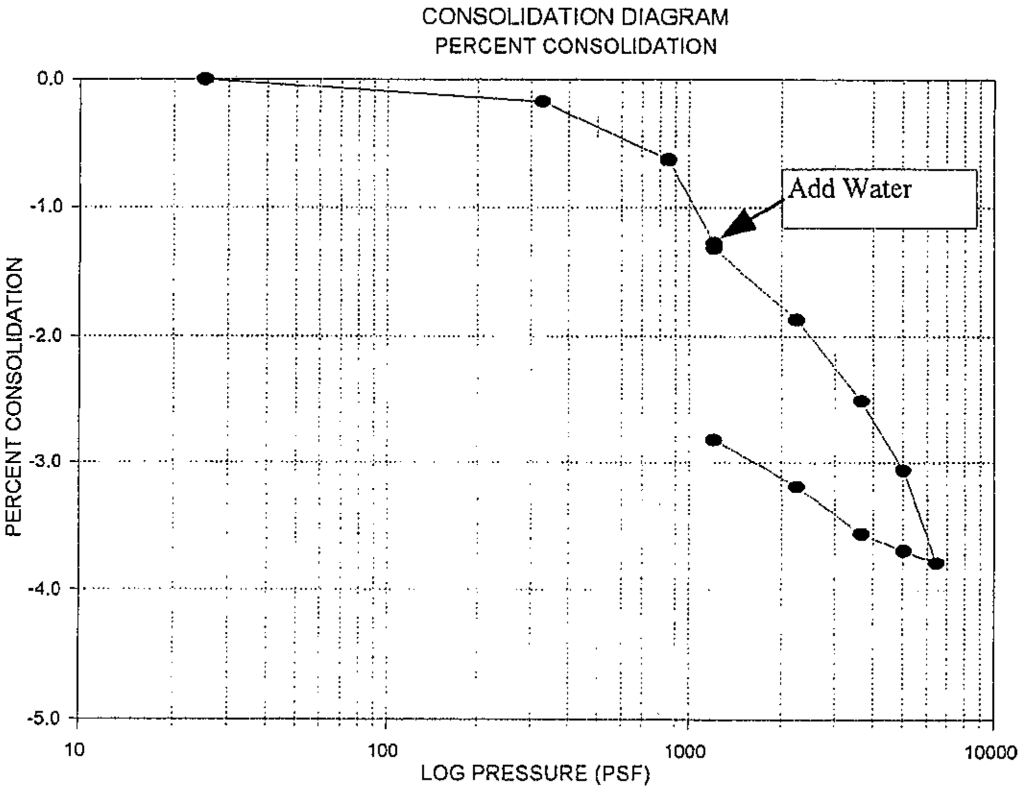
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B9-9'



Dry Density	99.4 pcf	Specific Gravity	2.65
Initial Moisture	23.1%	Initial Void Ratio	0.66
Initial % Saturation	92.2%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

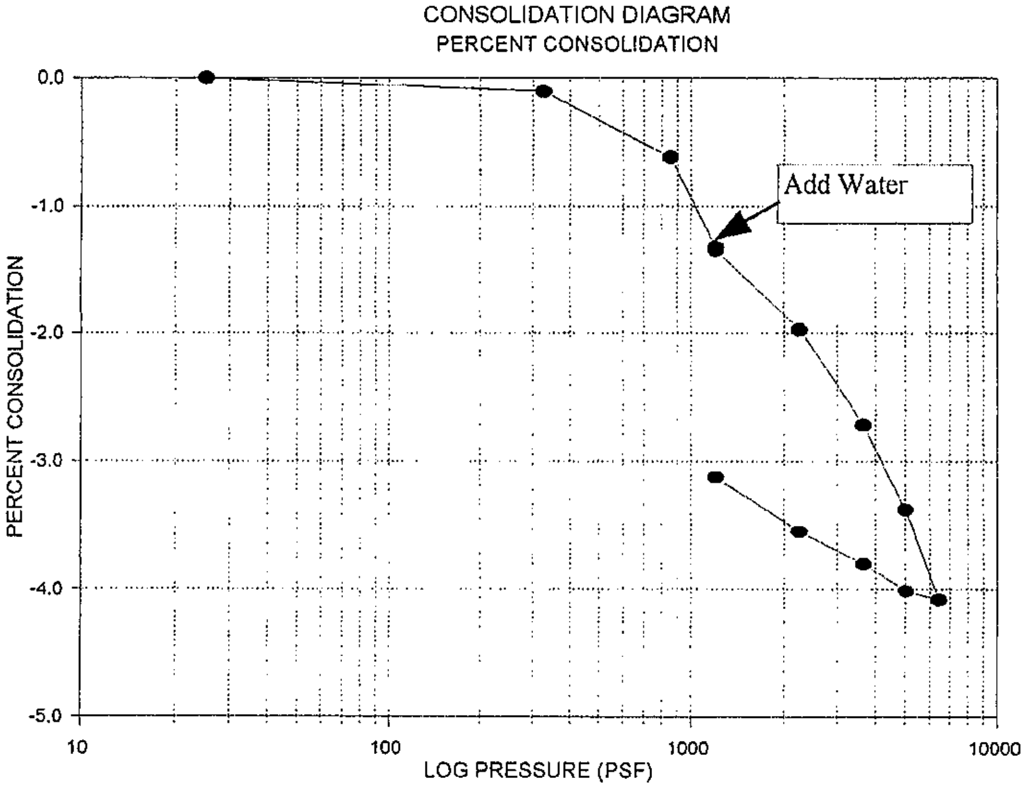
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B12-10'



Dry Density	100.5 pcf	Specific Gravity	2.65
Initial Moisture	20.0%	Initial Void Ratio	0.65
Initial % Saturation	82.1%	C'c	0.04

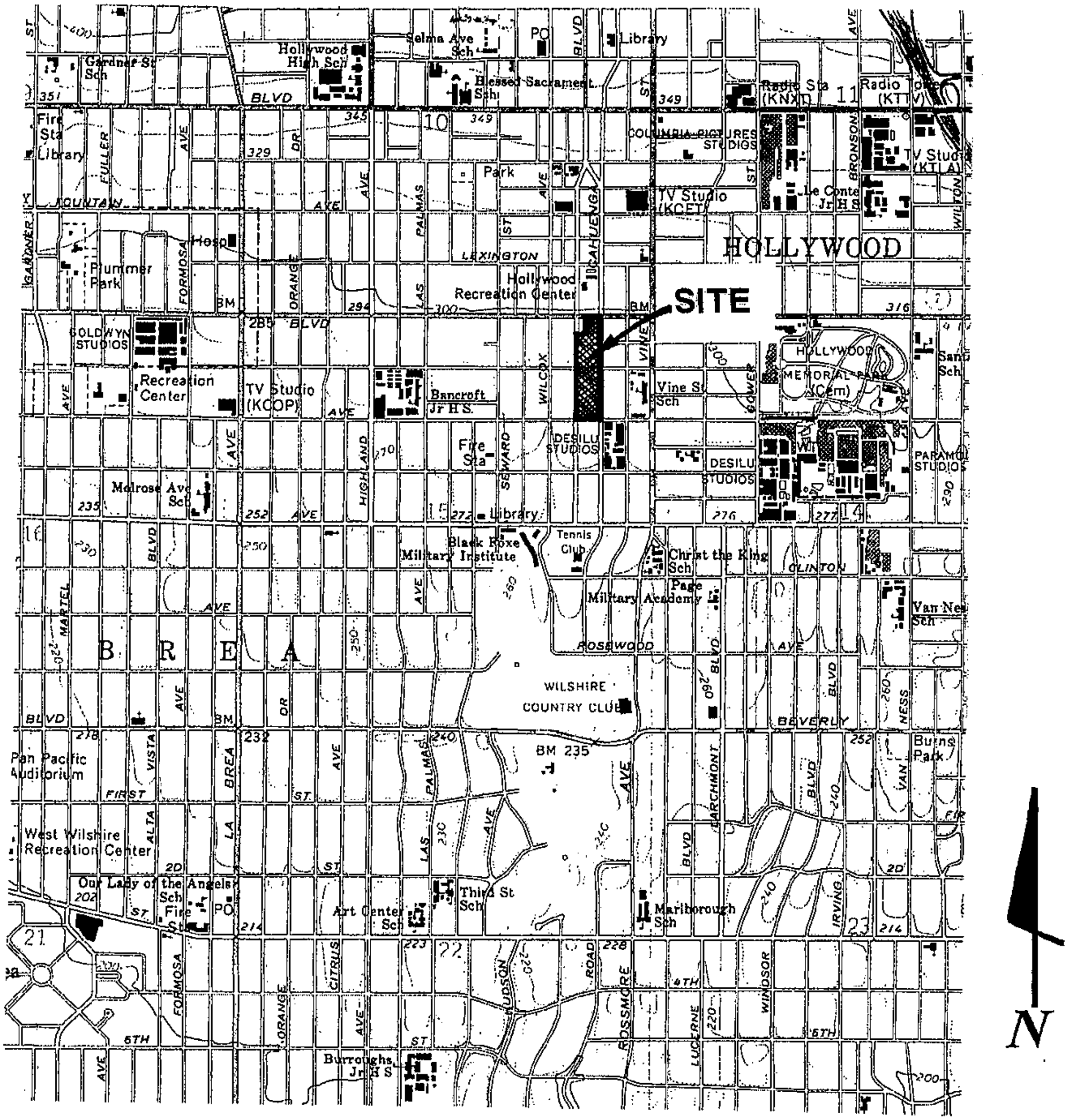
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

VICINITY MAP

JB 18051-B CLIENT: Television Center

CONSULTANT: JWB SCALE: 1" = 2,000'

REFERENCE: *U.S.G.S. 7.5 Minute Quadrangles, Beverly Hills and Hollywood Sheets, photorevised 1981.*



Project No: JB 18051-B

Log of Boring 1

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface **FILL:** Sandy Gravel, brown, red, gray, slightly moist to dry, medium dense to dense, concrete and brick fragments to 4 inches								
-1	1			SW						
-2	2	Sandy Clay, brown, moist, firm								
-3	3	**ALLUVIUM:** Silty Sand, brown to light reddish brown, moist very dense, some clay binder		CL	R	34	16.8	117.4		
-4	4			SM						
-5	5			----	R	50	12.0	120.6		
-6	6									
-7	7	dark brown, slightly porous		----	R	50	15.7	115.4		
-8	8									
-9	9									
-10	10	Gravelly Sand, reddish brown, slightly moist, very dense		----	R	50 6"	10.8	118.4		
-11	11			SW						
-12	12									
-13	13									
-14	14									
-15	15									
-16	16									
-17	17									
-18	18									
-19	19			----	R	50 10"	17.1	110.4		
-20	20	End at 20 Feet; No Water; Fill to 3 Feet;								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 2

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL:** Silty Sand, light brown, slightly moist, medium dense, concrete and brick fragments to 2 inches								
-1	1			SM						
-2	2	Clay, dark brown to black, moist, firm		CL	R	20	18.5	108.8		
-3	3	**ALLUVIUM:** Sandy Clay, dark brown, moist, stiff to very stiff								
-4	4			CL						
-5	5			----	R	45	10.2	121.2		
-6	6	Silty Sand, light reddish brown, slightly moist, very dense, some clay binder, some caliche veins		SM						
-7	7	Silty Sand, light brown to brown, slightly moist very dense								
-8	8									
-9	9									
-10	10	some gravel		----	R	30	10.4	114.7		
-11	11	Sandy Silt, brown to gray brown, moist, very firm								
-12	12			ML						
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet.		----	R	50	18.0	112.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 3

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	FILL: Clay, dark brown to black, moist, firm, some concrete fragments to 2 inches								
-1	1									
-2	2			CL	R	16	19.0	108.6		
-3	3									
-4	4	ALLUVIUM: Sandy Clay, black to brown, moist, very firm								
-5	5			CL	R	17	16.4	112.8		
-6	6									
-7	7	Silty Sand, light reddish brown, slightly moist to moist, dense to very dense, some clay binder		SM	R	33	19.0	108.6		
-8	8	Sandy Silt, light brown to reddish brown, moist very firm		ML						
-9	9									
-10	10	Clayey Silt, reddish brown to grayish brown, moist, very firm, slightly porous		----	R	22	17.9	106.5		
-11	11			ML						
-12	12									
-13	13									
-14	14	Silty Sand, light reddish brown to gray, slightly moist, dense to very dense End at 15 Feet; No Water; Fill to 4 Feet								
-15	15			SM	R	31	13.1	118.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 4

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black to dark brown, moist, firm								
-1	1									
-2	2	Mixed black Clay and brown Silty Sand, moist medium dense		CL	R	18				
-3	3									
-4	4									
-5	5	ALLUVIUM: Clay, black, moist, very firm		---	R	12	24.4	95.0		
-6	6			CL						
-7	7			----	R	18	21.2	104.0		
-8	8									
-9	9									
-10	10	Silt, light gray brown, moist to very moist, firm		----	R	13	27.0	97.4		
-11	11			ML						
-12	12									
-13	13									
-14	14									
-15	15	Clayey Silt, greenish gray, moist to very moist very firm		----	R	18	24.3	102.5		
-16	16									
-17	17	Sandy Silt, reddish brown to gray, moist, very firm								
-18	18									
-19	19			----	R	24	27.9	96.6		
-20	20	End at 20 Feet; Fill to 5 Feet.								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 5

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL:** Clay, black, moist, firm								
-1	1									
-2	2			CL	R	17	22.7	102.3		
-3	3									
-4	4	**ALLUVIUM:** Silty Sand, brown to dark brown, moist, dense								
-5	5			SM	R	15	14.9	112.4		
-6	6	Sandy Silt, dark gray brown, moist, very firm, some caliche veins								
-7	7			----	R	22	21.5	103.5		
-8	8			ML						
-9	9									
-10	10	Silty Clay, gray brown to brown, moist, very firm		----	R	16	30.9	89.8		
-11	11			CL						
-12	12			----	R	12	39.0	77.7		
-13	13									
-14	14									
-15	15	Sand, gray to brownish gray, wet, dense, fine to medium grained		----	R	16	21.9	101.7		
-16	16			SP						
-17	17									
-18	18	medium to coarse								
-19	19	Gravelly Sand, grayish brown to gray, wet, dense to very dense, End at 20 Feet; water at 15 Feet; Fill to 4½ Feet.		SW	R	48	118.5	118.5		
-20	20									

Surface: 2 Inch AC/4 Inch Base

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 6

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface FILL: Silty Sand, brown, moist, dense								
-1	1			SM						
-2	2	ALLUVIUM: Clay, dark gray brown to black, moist, stiff to very stiff								
-3	3			CL						
-4	4			----	R	19	21.3	104.7		
-5	5									
-6	6	Sandy Clay, brown to gray brown, moist, very stiff		----	R	23	19.4	109.2		
-7	7									
-8	8	Silty Sand, reddish brown to brown, moist, very dense, some gravel, some clay binder		----	R	34	17.4	111.8		
-9	9			SM						
-10	10									
-11	11	Gravelly Sand, reddish brown, slightly moist, very dense								
-12	12			SW	R	40	11.4	108.2		
-13	13									
-14	14									
-15	15			----	R	32	12.6	112.5		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-27-99

Size: 8 Inch
Elevation:
Sheet: 1 of 2

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 6

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				—-	R	50 10"	11.4	122.1		
-21	21									
-22	22									Water Measured at 21½ Feet After One Hour
-23	23									
-24	24	Silty Sand, light brown to light reddish brown, moist, dense								
-25	25			SM	SPT	6 9 12				
-26	26									
-27	27									Water Encountered at 26½ Feet
-28	28									
-29	29									
-30	30	Sandy Silt, light brown to gray, moist to very moist, very firm		—-	SPT	8 14 14				
-31	31			ML						
-32	32									
-33	33									
-34	34									
-35	35	Silty Sand, light brown to brownish gray, moist dense to very dense		—-	SPT	15 18 16				
-36	36			SM						
-37	37									
-38	38									
-39	39	End at 40 Feet; Water at 21½ Feet; Fill to 2 Feet		—-	SPT	16 18 19				
-40	40									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 2 of 2

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 7

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black, moist, firm								
-1	1			CL						
-2	2			----	R	17	18.2	110.0		
-3	3									
-4	4									
		ALLUVIUM: Clayey Sand, light reddish brown, moist, very dense, some gravel								
-5	5			----	R	20	18.5	109.5		
-6	6			SC						
-7	7			----	R	36	15.9	113.6		
		Silty Sand, red brown, slightly moist, very dense, some clay binder								
-8	8			SM						
-9	9									
-10	10			----	R	30	9.9	111.9		
-11	11									
-12	12									
-13	13									
-14	14									
		End at 15 Feet; No Water; Fill to 4½ Feet.								
-15	15			----	R	32	18.9	106.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 8

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
-1	1	**FILL:** Gravelly Sand, dark gray brown, slightly moist dense		SW						
-2	2	**ALLUVIUM:** Sandy Clay, black, moist, stiff, slightly porous		—	R	16	20.3	105.2		
-3	3			CL						
-4	4	dark reddish brown, some gravel								
-5	5			----	R	21	14.0	113.9		
-6	6	Silty Sand, reddish brown, moist, dense to very dense, some clay binder, some gravel								
-7	7			SM	R	22	17.4	110.5		
-8	8									
-9	9	dense, slightly porous								
-10	10			----	R	15	13.0	109.8		
-11	11									
-12	12									
-13	13									
-14	14	Silty Sand, dark gray to brownish gray, moist, dense, slightly porous End at 15 Feet; No Water; Fill to 1½ Feet.								
-15	15			----	R	17	17.2	113.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Concrete Driveway

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface FILL: Clay, black, moist, firm	▩							
-1	1			CL						
-2	2	Silty Sand, brown to gray brown, moist, medium dense	░							
-3	3	ALLUVIUM: Sandy Clay, dark reddish brown, moist, stiff, some gravel	▤	SM	R	10	12.3	112.2		
-4	4	slightly porous		CL						
-5	5	Clay, black to dark gray brown, moist, stiff, slightly porous	▨							
-6	6			----	R	17	21.1	104.7		
-7	7									
-8	8									
-9	9	Silty Clay, brown, moist, firm to very firm	▨	----	R	14	23.1	99.4		
-10	10									
-11	11	Sandy Clay, brown to brownish gray, moist, stiff to very stiff	▤							
-12	12			----	R	25	15.6	114.3		
-13	13									
-14	14									
-15	15	Clay, gray to brown, moist, stiff	▨	----	R	15	25.3	98.6		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20	Clayey Sand, brown to reddish brown, moist, dense to very dense with some gravel	░							

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				SC	R	26	21.8	106.3		
-21	21									
-22	22									
-23	23									
-24	24									Water Encountered at 24 Feet
-25	25	Silty Sand, brown to gray, wet, dense, some clay binder		----	R	17	19.6	106.0		
-26	26			SM						
-27	27									
-28	28									
-29	29									
-30	30			----	SPT	7 10 14				
-31	31									
-32	32									
-33	33									
-34	34									
-35	35	Sandy Silt, brown, wet, firm		----	SPT	9 12 16				
-36	36			ML						
-37	37									
-38	38									
-39	39									
-40	40									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 2 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
		Silty Sand, light reddish brown to gray, wet, dense to very dense		----	SPT	14 16 19				
-41	41			SM						
-42	42									
-43	43									
-44	44									
-45	45	Sandy Silt, brown to gray, very moist, very firm		----	SPT	10 14 16				
-46	46			ML						
-47	47									
-48	48									
-49	49									
-50	50	End at 50 Feet; Water at 16 Feet; Fill to 3 Feet.		----	SPT	9 12 15				Water Encountered at 16 Feet after one hour
-51	51									
-52	52									
-53	53									
-54	54									
-55	55									
-56	56									
-57	57									
-58	58									
-59	59									
-60	60									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 3 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 10

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Silty Sand, brown, slightly moist, medium dense								
-1	1	Sandy Clay, dark brown, moist, firm		SM						
-2	2	Clay, black, moist, stiff, slightly porous		CL	R	17	21.4	104.3		
-3	3									
-4	4									
-5	5	Sandy Clay, mottled dark gray brown to brown		—	R	16	19.3	107.5		
-6	6	**ALLUVIUM:** Clayey Sand, reddish brown, moist, dense		SC						
-7	7									
-8	8	Silty Sand, reddish brown, slightly moist, dense to very dense		SM						
-9	9									
-10	10			—	R	36	15.4	110.8		
-11	11									
-12	12	light reddish brown to gray								
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 5½ Feet.		—	R	18	12.8	110.6		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 11

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface **FILL:** Sandy Clay, black, moist, firm, fragments of brick								
-1	1			CL						
-2	2	**ALLUVIUM:** Clay, black, moist, very firm, slightly porous		----	R	15	21.3	103.8		
-3	3									
-4	4	dark brown								
-5	5	Sandy Clay, reddish brown, moist, very stiff, some gravel								
-6	6			----	R	35	14.0	117.4		
-7	7									
-8	8	Silty Sand, reddish brown, moist, very dense, some gravel								
-9	9			SM	R	26	11.0	113.5		
-10	10			SM						
-11	11									
-12	12									
-13	13	Sandy Silt, reddish brown to gray, moist, very firm								
-14	14	End at 15 Feet; No Water; Fill to 2 Feet								
-15	15			----	R	23	24.3	101.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 12

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface **FILL** Clayey Sand, brown to dark brown, moist, dense								
-1	1			SC						
-2	2	**ALLUVIUM:** Clay, black, moist, stiff		----	R	20	27.1	97.9		
-3	3			CL						
-4	4	Sandy Clay, brown, moist, very firm, slightly porous								
-5	5			----	R	19	19.2	106.8		
-6	6									
-7	7						20.8	109.4		
-8	8			----	R	21				
-9	9	Sandy Silt, brown to light brown, moist, firm								
-10	10			ML	R	12	20.0	100.5		
-11	11									
-12	12	Clayey Silt, dark gray brown to brown, moist, firm								
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet		----	R	14	28.6	92.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

THE J. BYER GROUP, INC.

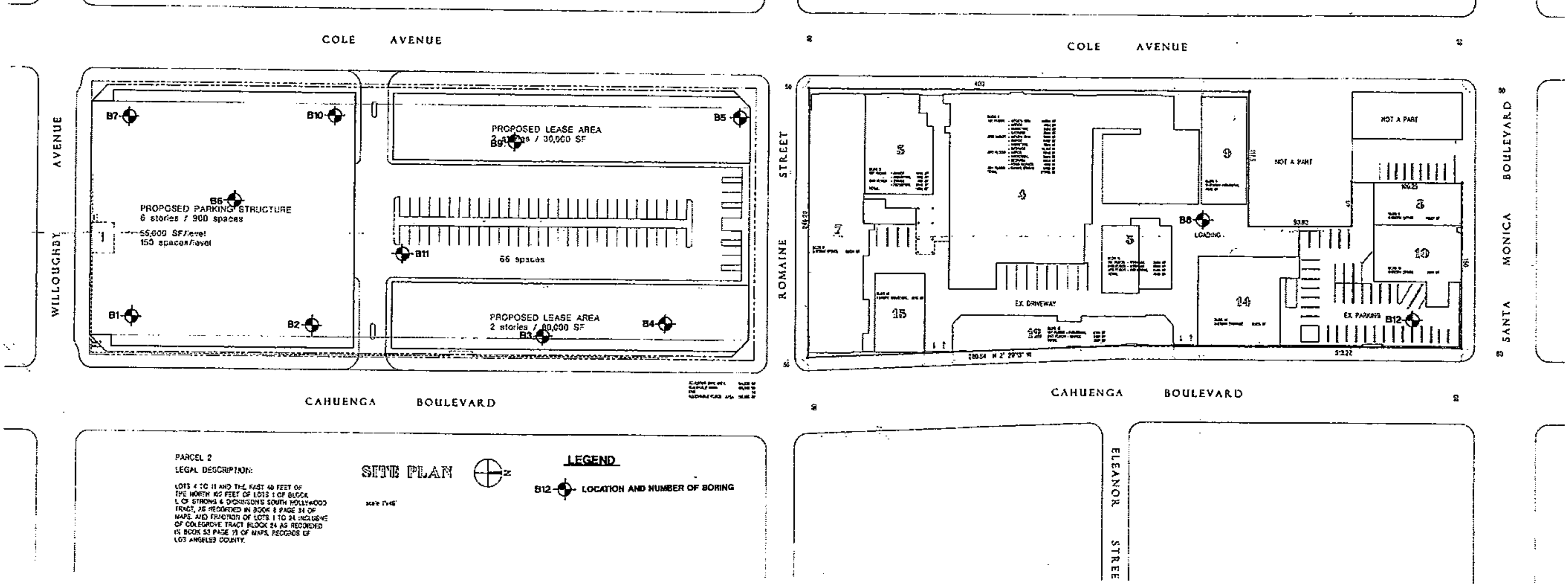
SITE PLAN

JB: 18051-B TELEVISION CENTER

CONSULTANT: JWB

SCALE: 1" = 40'

MAY 18, 1999



PARCEL 2
LEGAL DESCRIPTION:

LOTS 4 TO 11 AND THE EAST 40 FEET OF THE NORTH 100 FEET OF LOTS 1 OF BLOCK L OF STRONG & DICKINSON'S SOUTH HOLLYWOOD TRACT, AS RECORDED IN BOOK 8 PAGE 34 OF MAPS, AND FRACTION OF LOTS 1 TO 24 INCLUSIVE OF COLEGROVE TRACT BLOCK 24 AS RECORDED IN BOOK 53 PAGE 71 OF MAPS, RECORDS OF LOS ANGELES COUNTY.

SITE PLAN

scale 1"=40'

LEGEND

B12 ⊕ LOCATION AND NUMBER OF BORING

Appendix IS-2.2

Geotechnical Engineering Investigation—South Block

Geotechnologies, Inc.
Consulting Geotechnical Engineers
439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

November 29, 2021
Revised November 2, 2023
File Number 22218

Bardas Investment Group
1015 N. Fairfax Avenue
West Hollywood, California 90046

Attention: Alex King

Subject: Geotechnical Engineering Investigation
Proposed TVC South Block Development
6300 Romaine Street, 901 – 955 N. Cahuenga Boulevard,
906 – 956 N. Cole Avenue, and 6113 W. Willoughby Avenue,
Los Angeles, California

Dear Mr. King:

This letter transmits the Geotechnical Engineering Investigation for the property known as the "South Block", 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.

STANLEY SIU-KEI TANG
R.C.E. 56178
EXP. 12/31/24

STANLEY S. TANG
R.C.E. 56178

SST:km

Email to: [aking@bardasig.com]

TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED TVC SOUTH BLOCK DEVELOPMENT
6300 ROMAINE STREET, 901 – 955 N. CAHUENGA BOULEVARD,
906 – 956 N. COLE AVENUE, AND 6113 W. WILLOUGHBY AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed for the property, known as the “South Block”. The purpose of this investigation was to identify the distribution and engineering properties of the earth materials underlying the South Block, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included excavation of three exploratory borings for the South Block, supplemented by ten prior geotechnical borings previously performed for the South Block by The J. Byer Group (JBG), as discussed under the Research section of this report. Additionally, this investigation included collection of representative soil samples from the recent exploratory borings, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan in Appendix I. The exploration logs are presented in Appendix II, and the laboratory testing are presented in Appendix III of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. On the South Block, the Project proposes to construct two soundstage buildings and two creative office buildings containing office and associated production uses. Vehicular parking spaces would be provided on-site in a two to three-level subterranean garage on the South Block. Excavation would occur to a depth of approximately 40 feet below ground surface (bgs) for the proposed subterranean levels and foundation elements on the South Block. Column loads are estimated to be between 600 and 1,200 kips. Wall loads are estimated to be between 5 and 10 kips per lineal foot.

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

SITE CONDITIONS

The South Block is located at 6300 Romaine Street, 901 – 955 N. Cahuenga Boulevard, 906 – 956 N. Cole Avenue, and 6113 W. Willoughby Avenue, in the City of Los Angeles, California. The South Block is bounded by Romaine Street to the north, by N. Cahuenga Boulevard to the east, by Willoughby Avenue to the south, and by Cole Avenue to the west. The site is currently developed with a 5-story parking structure with commercial uses on the first floor, and an at-grade parking lot.

The site slopes downward very gently to the southwest. According to the topographic survey prepared by JRN Civil Engineers, the high site elevation is at 295.9 feet AMSL located at the northeast corner of the site, and the low site elevation is at 288.74 feet AMSL located at the southwest corner of the site. This corresponds to an approximate 7 feet of elevation change diagonally across the site. Drainage across the site is by sheetflow to the City streets. The vegetation on the site consists of isolated trees and planters. The neighboring development consists primarily of commercial development.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

Three exploratory borings were excavated as part of the geotechnical investigation by this firm, between March 24, 2021, and March 26, 2021, for the South Block. The explorations were supplemented by ten prior geotechnical borings (Boring Number 1 through 7, and 9 through 11) previously performed for the South Block by The J. Byer Group (JBG), as discussed under the Research section of this report.

The exploratory borings performed by this firm varied between 80 and 100 feet in depth bgs. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan (Plate VI in Appendix I), and the geologic materials encountered are logged on Plates A-1 through A-3, presented in Appendix II. The prior boring logs relevant to the South Block by JBG are also presented in Appendix II for reference.

The locations of the elevation of the top of the exploratory borings were determined based on interpolation from the Topographic Survey prepared by JRN Civil Engineers. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials underlying the subject site consist of silty sands to sandy clays, which are dark gray to dark brown in color, moist, medium dense to medium firm, fine grained. Fill thickness on the order of 3 feet was encountered in the explorations.

Native soils consist of alluvial deposits consisting primarily of clays with occasional layers of silty and clayey sands, and sands. The native soils are dark brown, grayish brown and dark gray in color, moist to wet, firm to very stiff, dense to very dense, fine grained. The native soils consist predominantly of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual boring logs, which are presented in Appendix II.

Groundwater

Groundwater was encountered between 15.5 and 21.5 feet below the existing ground surface, corresponding to 278.2 to 270.1 feet AMSL. Due to the stratified layers of sands and clays underlying the site, it is likely that the encountered groundwater in the upper zone consists of a confined, perched groundwater layer.

Based on review of the Historically Highest Groundwater Levels Map presented in the Seismic Hazards Zones Report (CGS SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface.[a] A copy of the Historically Highest Groundwater Levels Map is presented on Plate IV in Appendix I of this report. However, since the result of site explorations indicates that the encountered groundwater level is higher, it is recommended that the high groundwater elevation of 278.2 feet AMSL encountered during exploration be utilized for the project design purposes.

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

---

[a] CGS SHZR 026:
https://planning.lacity.org/eir/conventioncntr/DEIR/files/references/California%20Division%20of%20Mines%20and%20Geology,%20%20Hollywood%20Quadrangle,%201998.pdf

Research

Available geotechnical reports for the site were reviewed during the preparation of this investigation. Specifically reviewed is the following report prepared by The J. Byer Group, Inc. (JBG). A copy of this report is presented in Appendix IV for reference.

- Geotechnical Exploration, Proposed Parking Structure, Commercial Buildings, and Additions, 6311 Romaine Street, Hollywood, California, Project No. JB 18051-B, dated May 13, 1999.

A total of twelve exploratory borings were excavated by JBG, extending to depths between 15 to 50 feet below the site grade for both the North Block and the South Block. Ten of the JBG borings (Boring Number 1 through 7, and 9 through 11) are located in the South Block. Between 2 and 5 feet of existing fill materials were encountered at the South Block by JBG during exploration. The fill is underlain by firm natural alluvial deposits. Groundwater was encountered in Borings 5, 6, and 9. According to JBG, seepage of groundwater was observed at a depth of 15 feet in Boring 5, which was excavated adjacent to a broken water main along Cole Avenue that was under repair at the time of exploration. Confined groundwater layer was encountered in Boring 6 at a depth of 26.5 feet below grade, and in Boring 9 at a depth of 24 feet below grade. The groundwater levels rose to 21.5 and 16 feet in Boring 6 and 9, respectively.

The exploratory borings by JBG (Boring Number 1 through 7, and 9 through 11), which are relevant to the South Block, are plotted on the enclosed Plot Plan presented in Appendix I. The boring logs of these relevant boreholes are also presented in Appendix II of this report for reference.

This firm has reviewed the referenced document by JBG, and concurs with the findings provided therein. All JBG borings extended into the underlying native soils similar to those encountered during explorations performed by this firm. The recommendations contained herein shall supersede those previously provided by JBG for the planned development. This firm accepts the prior findings by JBG and the professional responsibility for the project as the geotechnical engineer of record.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The property 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.[b]

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,700 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.[c]

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

---

[b] CGS Note 36: https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-36.pdf
[c] CGS Special Publication 42: https://www.conservation.ca.gov/cgs/documents/publications/special-publications/SP_042.pdf

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.[d]

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 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.[e]

---

[d] CGS Note 31: https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-31.pdf
[e] CGS Special Publication 42: https://www.conservation.ca.gov/cgs/documents/publications/special-publications/SP_042.pdf

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

Ground 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 (Zimas and NavigateLA).[g] 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 Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially "Liquefiable" area (Plate III of Appendix I). This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.[h]

---

[f] Ibid
[g] Zimas website: https://zimas.lacity.org/ and NavigateLA website: https://navigatela.lacity.org/navigatela/
[h] CGS SHZR 026:
https://planning.lacity.org/eir/conventioncntr/DEIR/files/references/California%20Division%20of%20Mines%20and%20Geology,%20%20Hollywood%20Quadrangle,%201998.pdf

A site-specific liquefaction analysis was performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered between 15.5 and 21.5 feet below the existing ground surface, corresponding to 278.2 to 270.1 feet above mean sea level (AMSL). Based on review of the Seismic Hazards Zones Report (CGS SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface.[i] However, since the result of site explorations indicates that the encountered groundwater level is higher than the historically highest groundwater level, a high groundwater level of 15 feet below ground surface was conservatively utilized for the enclosed liquefaction analysis (Appendix III).

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS websites, using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) and the U.S. Seismic Design Maps tool (USGS, 2013). A Site Class “D” (Stiff Soil Profile) and a published shear wave velocity of 230 meters per second were utilized for Vs30 (Tinsley and Fumal, 1985) in the USGS seismic programs. A modal magnitude (M_W) of 6.7 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.98g was obtained using the ASCE Hazard Tool website (https://asce7hazardtool.online/). These parameters are used in the enclosed liquefaction analyses.

The liquefaction analysis, entitled “Empirical Estimation of Liquefaction Potential,” is based on Boring 1. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing

---

[i] Ibid

a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory boring are presented on the enclosed Plates F-1 and F-2 (Appendix III). 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. Furthermore, cohesive soils with PI between 7 and 12 and moisture content greater than 85 percent of the liquid limit are also susceptible to liquefaction.[j]

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

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

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

---

[j] CGS Special Publication 117A: https://www.conservation.ca.gov/cgs/Documents/Publications/Special-Publications/SP_117a.pdf

2002, when the saturated cohesionless sediments with $(N_1)_{60} > 15$, significant displacement is not likely for $M < 8$ earthquakes.[k]

The enclosed liquefaction analysis included in Appendix III of this report, indicates that site soils would not be prone to liquefaction during 2,475-year return period ground motion. Therefore, lateral spreading is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. The South Block is located approximately 11 miles from the Pacific Ocean. 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.[l]

---

[k] Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", ASCE Journal of Geotechnical Engineering, Vol. 128, No. 12, December: https://ascelibrary.org/doi/abs/10.1061/%28ASCE%291090-0241%282002%29128%3A12%281007%29

[l] County of Los Angeles General Plan Plates 1-8: https://planning.lacounty.gov/assets/upl/project/gp_web80-tech-plates-01-to-08.pdf

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately adjacent to the property. The Hollywood Reservoir/Mulholland Dam is located approximately 2 miles north of the South Block.

According to the County of Los Angeles General Plan, the site is within the potential inundation boundary of Hollywood Reservoir/Mulholland Dam.[m] The Hollywood Reservoir/Mulholland Dam as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake for the site as well as other conditions that could undermine the integrity of the dam. Pursuant to these regulations, the Hollywood Reservoir/Mulholland Dam is regularly inspected and meets current safety regulations. In addition, the LADWP has emergency response plans to address any potential impacts to its dams. Therefore, the risk of flooding from a seismically-induced seiche or dam failure is considered to be remote.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

---

[m] Ibid

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

Approximately 3 to 5 feet of existing fill was encountered in the explorations by this firm and by JBG at the South Block. The site is underlain by alluvial deposits consisting primarily of clays with occasional layers of silty and clayey sands, and sands.

Groundwater was encountered between 15.5 and 21.5 feet below the existing ground surface, corresponding to 278.2 to 270.1 feet above mean sea level (AMSL). Due to the stratified layers of sands and clays underlying the site, it is likely that the encountered groundwater in the upper zone consists of a confined, perched groundwater layer. According to the Seismic Hazards Zones Report (CGS SHZR 026) for the Hollywood Quadrangle, the historically highest groundwater level is generally on the order of 20 feet below the existing ground surface. However, since the result of site explorations indicates that the encountered groundwater level is higher, it is recommended that the high groundwater elevation of 278.2 feet AMSL encountered during exploration be utilized for the project design purposes.

On the South Block, the Project proposes to construct two soundstage buildings and two creative office buildings containing office and associated production uses. Vehicular parking spaces would be provided on-site in a two to three-level subterranean garage on the South Block. Excavation would occur to a depth of approximately 40 feet bgs for the proposed subterranean levels and foundation elements on the South Block.

Excavation of the proposed subterranean levels will remove the existing fill materials and expose the underlying firm native soils. Due to the high groundwater level, it is recommended that the subterranean structure be designed for hydrostatic pressure and a mat foundation should be utilized for support of the proposed structure.

The basement walls shall be designed for the soil and hydrostatic pressures based on the existing ground surface. In addition, the proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater elevation of 278.2 feet AMSL. The proposed uplift pressure to be used in the foundation design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the historically highest groundwater level.

Excavation of the proposed subterranean levels will require shoring and temporary dewatering measures to provide a stable and dry excavation due to the depth of the excavation, the presence of groundwater, and the proximity of public right of ways. Pumping of the high-moisture content soils at the bottom of the excavation is anticipated to occur during operation of heavy equipment. Recommendations for stabilizing the wet subgrade is provided in the Wet Soils section of this report.

Foundations for small outlying structures, such as property line walls, planters, trash enclosures, and canopies, which are not tied-in to the proposed structure and are to be constructed immediately adjacent to property lines or adjacent structures, such that the recommended horizontal overexcavation and recompaction cannot be achieved, should be deepened to bear in the dense native soils.

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

SEISMIC DESIGN CONSIDERATIONS

2022 CBC / 2023 LABC Seismic Parameters

Based on information derived from the subsurface investigation, the South Block site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the ASCE 7 Hazard Tool website (https://asce7hazardtool.online/) to calculate ground motion parameters for the site, in accordance with the 2022 California Building Code (CBC) and 2023 Los Angeles Building Code (LABC).

2022 CBC / 2023 LABC SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.083g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.083g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.389g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.746g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.268g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.845g*

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

FILL SOILS

Approximately 3 to 5 feet of existing fill was encountered in the explorations by this firm and by JBG at the South Block. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the high expansion range. The Expansion Index was found to be between 98 and 106 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs-on-Grade" sections of this report.

SUBSIDENCE

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. No large-scale extraction of gas, oil, or geothermal energy currently occurs or is planned at the South Block. Additionally, the proposed structure will be designed to resist hydrostatic pressure, and therefore, no permanent dewatering will be required for the Project.

Temporary construction dewatering will be necessary in order to construct the proposed subterranean structure. The underlying native soils consisting of alluvial deposits comprising primarily of firm to very stiff clays with occasional layers of dense to very dense silty and clayey sands were encountered during explorations. These native soils are typical to this area of Los Angeles County. Additional field explorations and pump tests will be performed at the Project Site to evaluate the groundwater conditions, the proposed temporary dewatering approaches and methods, and subsidence impact (if any) due to construction dewatering. The final dewatering system methods and shoring design, which are subject to regulatory control for safety and subsidence, will be submitted to LADBS for review and approval as part of the building permit processes prior to construction.

HYDROCONSOLIDATION

Hydroconsolidation is a phenomena in which the underlying soils collapse when wetted. Hydroconsolidation could potentially result in significant foundation movements, over a long period of time of wetting.

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

SOIL CORROSIVITY STUDY

The results of soil corrosion potential testing performed by HDR, Inc. indicate that the electrical resistivities of the soils were in the mildly corrosive to corrosive categories with as-received moisture, and in the corrosive to severely corrosive categories when saturated. Soil pH values of the samples ranged between 7.3 and 7.6, indicating neutral to mildly alkaline conditions. The soluble salt content and ammonium of the soil samples is low. The nitrate concentration was high enough to be aggressive to copper. Sulfate concentration is low.

In summary, the soils are classified as severely corrosive to ferrous metals, aggressive to copper, and negligible sulfate attack on concrete. Detailed results, discussion of results and recommended mitigating measures are provided within the report by HDR presented herein. Any questions regarding the results of the soil corrosion report should be addressed to HDR.

METHANE ZONES

Based on review of the Navigate LA (*http://navigatela.lacity.org/NavigateLA/*) website, maintained by the City of Los Angeles, the South Block is located within a Methane Buffer Zone as designated by the City.[n] A copy of the portion of the Methane Zone Risk map covering the South Block is presented on Plate V in Appendix I of this report.

A Methane Soil Gas Investigation was performed by Terra-Petra (TP) for the South Block. The result of the investigation by TP indicated non-detectable readings of methane gas. The testing, summary of findings, and recommendations are presented in the report titled "Report of Methane Soil Gas Investigation" by TP, dated May 5, 2023.

GRADING GUIDELINES

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

Site Preparation

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

---

[n] NavigateLA website: https://navigatela.lacity.org/navigatela/

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

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

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

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

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum 90 percent of the maximum density, except for cohesionless soils having less than 15 percent finer than 0.005 millimeters, which shall be compacted to a minimum 95 percent of the maximum density in accordance with the most recent revision of the Los Angeles Building Code.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

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

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 90. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Wet Soils

At the time of exploration, the soils which will be exposed at the bottom of the excavation were above optimum moisture content. It is anticipated that the excavated material to be placed as compacted fill, and the materials exposed at the bottom of excavated plane will require significant drying and aeration prior to recompaction.

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

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

Mat Foundation

It is recommended that the proposed structure be supported on a mat foundation bearing in the underlying native soils. Excavations on the order of 25 to 30 feet are anticipated for the subterranean parking levels and foundation elements. Preliminarily, it is anticipated that the average bearing pressure will be on the order of 2,000 to 3,000 pounds per square foot. Foundation bearing pressure will vary across the mat footing, with higher concentrated loads up to 4,500 pounds per square foot, located below the central cores of the building.

Given the size of the proposed mat foundation, the average bearing pressure of 3,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average bearing pressure of 3,000 pounds per square foot, with locally higher pressures up to 4,500 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 175 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

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

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

Hydrostatic Considerations for Mat Foundations

The proposed development will be constructed entirely over 2 subterranean parking levels, extending on the order of 20 to 25 feet below the existing site grade, with the lowest finished floor at an approximate elevation of 268.35 feet above mean sea level (AMSL). The subterranean structure will extend below the groundwater level, and therefore, the mat foundations shall be waterproofed and be designed to withstand the hydrostatic uplift pressure based on the high groundwater level.

As discussed in the "Groundwater" section of this report, the proposed mat foundation shall be designed for hydrostatic uplift pressure based on the historically highest groundwater elevation of 278.2 feet AMSL. The proposed uplift pressure to be used in the foundation design would be 62.4(H) psf, where "H" is the depth to the bottom of footing from the historically highest groundwater level. Where necessary, micropiles (tiedown anchors) may be utilized to provide uplift resistance in conjunction with the proposed mat foundation.

Micropiles for Hydrostatic Uplift

Where necessary, an anchoring system consisting of micropiles may be designed to provide resistance against the anticipated hydrostatic uplift pressures acting at the bottom of the mat foundation. The proposed micropiles shall derive support from the underlying native alluvial soils, expected at the subterranean subgrade. It is recommended that a post-grouted micropile system be utilized for support of the potential hydrostatic tension loads. The micropiles shall be a minimum of 10 inches in diameter and shall have a minimum of 30 feet (bonded length) embedded into the underlying native alluvial soils. The reinforcing steel shall be corrosion protected. The micropiles shall only be utilized for tension support and shall not be utilized for support of any lateral loads.

An allowable upward frictional capacity of 7 kips per lineal foot for the bonded length may be utilized in the design of a 10-inch diameter post-grouted micropile. An allowable upward frictional capacity of 8 kips per lineal foot for the bonded length may be utilized in the design of a 12-inch diameter post-grouted micropile. A safety factor of 2 has been applied in determining the allowable frictional capacity. These allowable micropile design capacities shall be considered preliminary and are subject to verification or modification based on a Verification Test Pile Program as discussed below.

A 1/3 increase may be utilized for temporary loads, such as wind and seismic forces. Micropiles should be spaced at a minimum of 3 diameters or 30 inches on centers, whichever is greater. If so spaced, there will be no reduction in the downward capacity of the micropiles due to group action.

The City of Los Angeles requires a steel casing having a minimum thickness of 3/8-inch be installed for the top section of the micropile (the “unbonded zone”) to a depth of 120 percent of the point of zero curvature. The cased section of the micropile shall be considered as the unbounded zone and shall not be considered as contributing to friction.

Based on the enclosed RSPile Analysis (RocScience), depth to zero curvature for a 10-inch diameter micropile is approximately 12 feet for the free-head condition and approximately 14 feet for a fixed-head condition. Therefore, it is recommended that a steel casing be provided for a 10-inch diameter micropile for the upper 14½ and 17 feet for the free-head and fixed head conditions, respectively.

Similarly, depth to zero curvature for a 12-inch diameter micropile is approximately 14 feet for the free-head condition and approximately 16 feet for a fixed-head condition. Therefore, it is recommended that a steel casing be provided for a 12-inch diameter micropile for the upper 17 and 19 feet for the free-head and fixed head conditions, respectively.

Verification Micropile Test Program

A verification test pile program shall be performed in order to verify the design capacities, prior to installation of the production micropiles. Since the proposed micropiles will be utilized only to resist hydrostatic uplift purposes, tension load tests shall be performed during verification test pile program. The verification test piles shall be sacrificial and shall not be utilized as part of the production piles. The number of verification test piles shall be a minimum of 2 test piles, or equivalent to a minimum of 1 percent of the production piles, whichever is greater.

The verification micropiles shall be tested to a minimum of 200 percent of the design load capacity. The load tests shall be performed in accordance with latest version of ASTM D 3689/3689M, with at least one maintained load test. The testing reaction frame shall be sufficiently rigid such that excessive deformation of the testing equipment will not occur. The hydraulic jack, pressure gauges, and dial gauges shall be calibrated prior to performance of the load test. A copy of the calibration certifications shall be provided by the contractor to this firm prior to performance of the load test.

Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment. Pile top movement shall be recorded at the beginning and at the end of each test period.

The total vertical pile top movement during the verification test shall not exceed 1 inch at the design load (DL), and 2 inches at the maximum test load of 200 percent (2*DL). At the completion of the verification test, the test pile may be cut off at a minimum depth of 1 foot below the finished subgrade and abandoned in place.

If a verification tested micropile fails to meet the acceptance criteria, the contractor shall modify the design and/or the construction procedure. All modification and changes shall be submitted to the Structural Engineer and the Geotechnical Engineer for review and approval.

Proof Load Tests

A minimum of 5 percent of the production piles shall be proof tested to a minimum test load of 160 percent of the design load. The proof test shall be made by incrementally loading the micropile in accordance with the ASTM D 3689/3689M. Once the alignment load (AL) is applied, all dial gauges shall be reset to zero. The test load shall be held constant during each test load increment.

Pile top movement shall be recorded at the beginning and at the end of each test period. The total vertical pile top movement during the proof load test shall not exceed 1 inch at the design load.

Pile Integrity Testing

Pile integrity tests shall be performed for all verification test micropiles and reaction piles, as required by LADBS. Due to the slenderness and the anticipated lengths of the micropiles, it is recommended that Thermal Integrity Profiling (TIP) method be utilized for the pile integrity tests. TIP uses the heat generated during the concrete curing process in the foundation pile element, to evaluate the consistency of the concrete and the regularity of its shape. TIP could be used for evaluating the cross-sectional areas and the length of the pile. Due to the slenderness of the micropiles, it is recommended that TIP be performed using embedded thermal sensors, in accordance with Method B of the latest version of ASTM D7949.

Typically, LADBS requires pile integrity tests be performed on all test piles and reaction piles, and a minimum of 5 percent of the production micropiles. In addition, one of the test micropiles shall be exhumed for measurement of the pile diameter and physical examination of the pile integrity. However, in order to minimize disturbance of the underlying soils which will provide support of the proposed mat foundation below the subterranean structure, it is recommended that the requirement of exhumation of a test pile be eliminated, and the non-destructive pile integrity tests be performed on a minimum of 10 percent of the production micropiles.

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structure may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall 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, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended.

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

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

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 0.20 may be used with the dead load forces.

Passive earth pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. It is anticipated that total settlement between 1.5 to 2 inches will occur below the more heavily loaded central core portions of the mat foundation beneath the building. Settlement on the edges of the mat foundation is expected to be between 1 inch. Differential settlement is anticipated to be less than 0.75 inches within 30 feet.

Foundation Observations

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

RETAINING WALL DESIGN

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Due to the historically highest groundwater level for the South Block, it is recommended that the proposed subterranean walls be designed for full hydrostatic pressure based on the existing ground surface, and the code required wall subdrain system may be eliminated. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure with Hydrostatic Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure with Hydrostatic Pressure (pcf)
30 feet	85 pcf	100 pcf

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

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.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 28 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

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 2020-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to 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. As an alternative, the surcharge calculation method provided in the Naval Facilities Design Manual (NAVFAV 7.02) may be followed.

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtainable by the most recent revision of ASTM D 1557. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

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

It is anticipated that excavations on the order of 29 feet in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 12 feet. A uniform sloped excavation does not have a vertical component.

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

It is critical that the soils exposed in the cut slopes are observed by a representative of this office during excavation so that modifications of the slopes can be made if variations in the earth material conditions occur. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Temporary Dewatering

Groundwater was encountered between 15.5 and 21.5 feet below the existing ground surface, corresponding to 278.2 to 270.1 feet above mean sea level (AMSL). Since the proposed subterranean levels and foundation elements will extend below the current groundwater level, it is recommended that a qualified dewatering consultant should be retained during the design phase of the Project. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water. However, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected within the excavation through the use of French drains and sump pumps. The collected water should be pumped to an acceptable disposal area. The exposed subgrade is anticipated to be wet and pumping. Subgrade stabilization and wet soil treatment are provided in the "Wet Soils" section of this report.

Once the temporary dewatering system is discontinued, the groundwater level will likely return to the pre-development level. It is critical that the termination of temporary construction dewatering be coordinated with the project structural engineer to confirm that there is sufficient weight of the structure to resist the high groundwater level prior to discontinuation of dewatering.

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 a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

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

Soldier Piles

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

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.25 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

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

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

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

Lagging

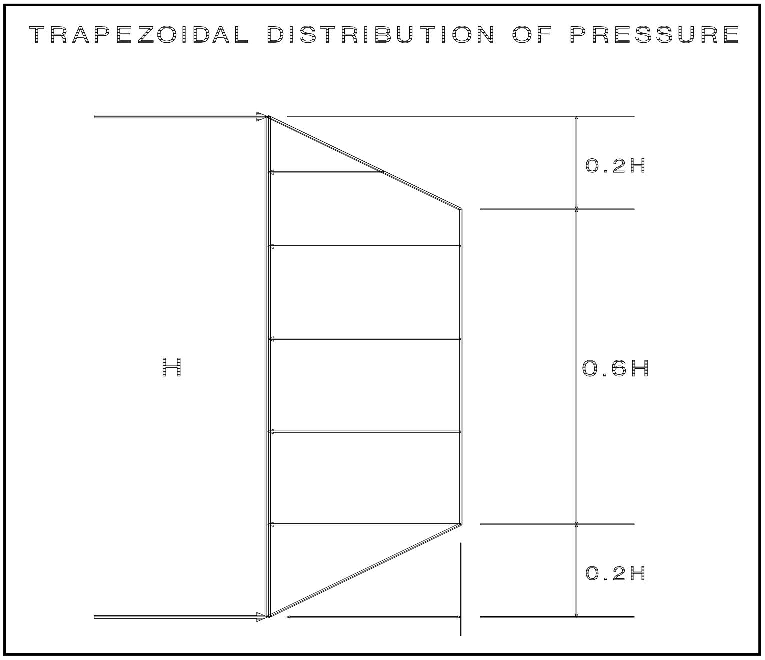
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less.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

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilevered shoring system. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The design of trapezoidal distribution of pressure is shown in the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table:

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
30 feet	45 pcf	30H psf

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



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

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

Tied-Back Anchors

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

Drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

It is recommended that at least 3 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.

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

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

Where applicable, concrete slabs-on-grade shall be a minimum of 5 inches in thickness, and shall be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Slabs-on-grade should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and shall be reinforced with a minimum of #3 steel bars on 12-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

Since the lowest subterranean level will extend below the historically highest groundwater level, the proposed subterranean structure and foundation shall be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for the proposed subterranean structure.

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

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

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	6	9

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Groundwater was encountered between 15.5 and 21.5 feet below the existing ground surface. Due to the high groundwater level encountered at the South Block and the depth of the proposed subterranean levels, stormwater infiltration will not be feasible for the project.

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

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

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

REFERENCES

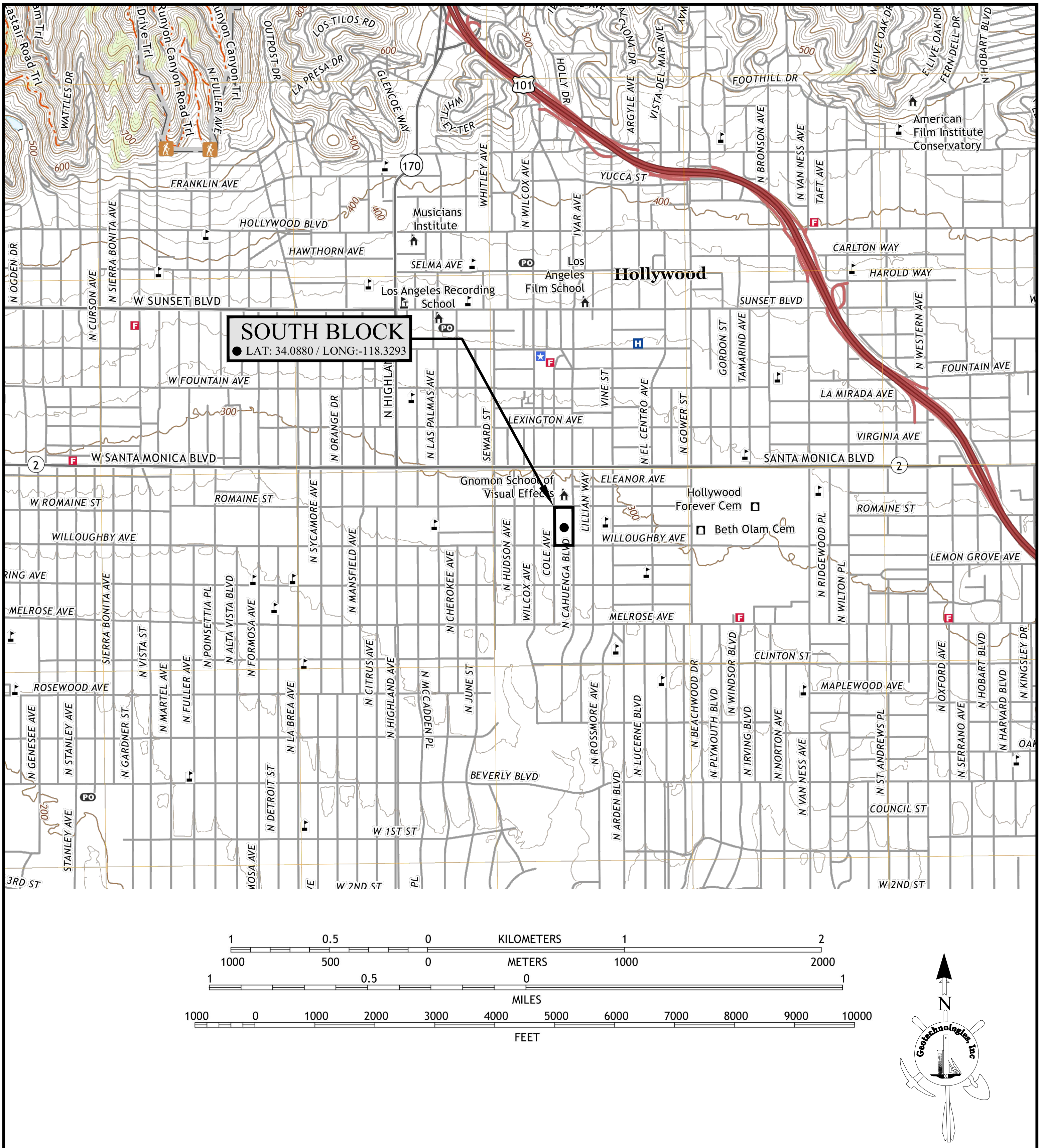
1. American Society of Civil Engineers, 1994, "Settlement Analysis," Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 9.
2. Bartlett, S.F. and Youd, T.L., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, SUNY-Buffalo, Buffalo, NY.
3. Bartlett, S.F. and Youd, T.L., 1995, "Empirical Prediction of Liquefaction-Induced lateral Spread," Journal of Geotechnical Engineering, Vol. 121, No.4, April.
4. Bowles, Joseph E., 1977, "Foundation Analysis and Design," 2nd Edition, McGraw-Hill, New York.
5. California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
6. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.
7. California Geological Survey, 2008, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California," CDMG Special Publication 117A.
8. City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
9. Crook, R., Jr., Proctor, R.J., 1992, The Hollywood and Santa Monica Fault and the Southern Boundary of the Transverse Ranges Province: in Pipkin, B., and Proctor, R.J. (eds.) Engineering Geology Practice in Southern California, Star Publishing Company, Belmont, California.
10. Department of the Navy, NAVFAC Design Manual 7.1, 1982, "Soil Mechanics," Naval Facilities Engineering Command, May.
11. Department of the Navy, NAVFAC Design Manual 7.02, 1986, "Foundations and Earth Structures," Naval Facilities Engineering Command, September.
12. Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) 7.5-Minute Quadrangles, Map No DF-30, map scale 1: 24,000.
13. Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.
14. Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

REFERENCES - continued

15. Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.
16. Southern California Earthquake Center, 1999, “Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California,” March.
17. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.
18. Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.
19. Tokimatsu, K. and Seed, H. B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking,” Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August.
20. Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, “Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement”, Journal of Geotechnical Engineering, Vol. 128, No. 12, December.

APPENDIX I

MAPS AND PLOT PLAN



REFERENCE: USGS TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE 2022

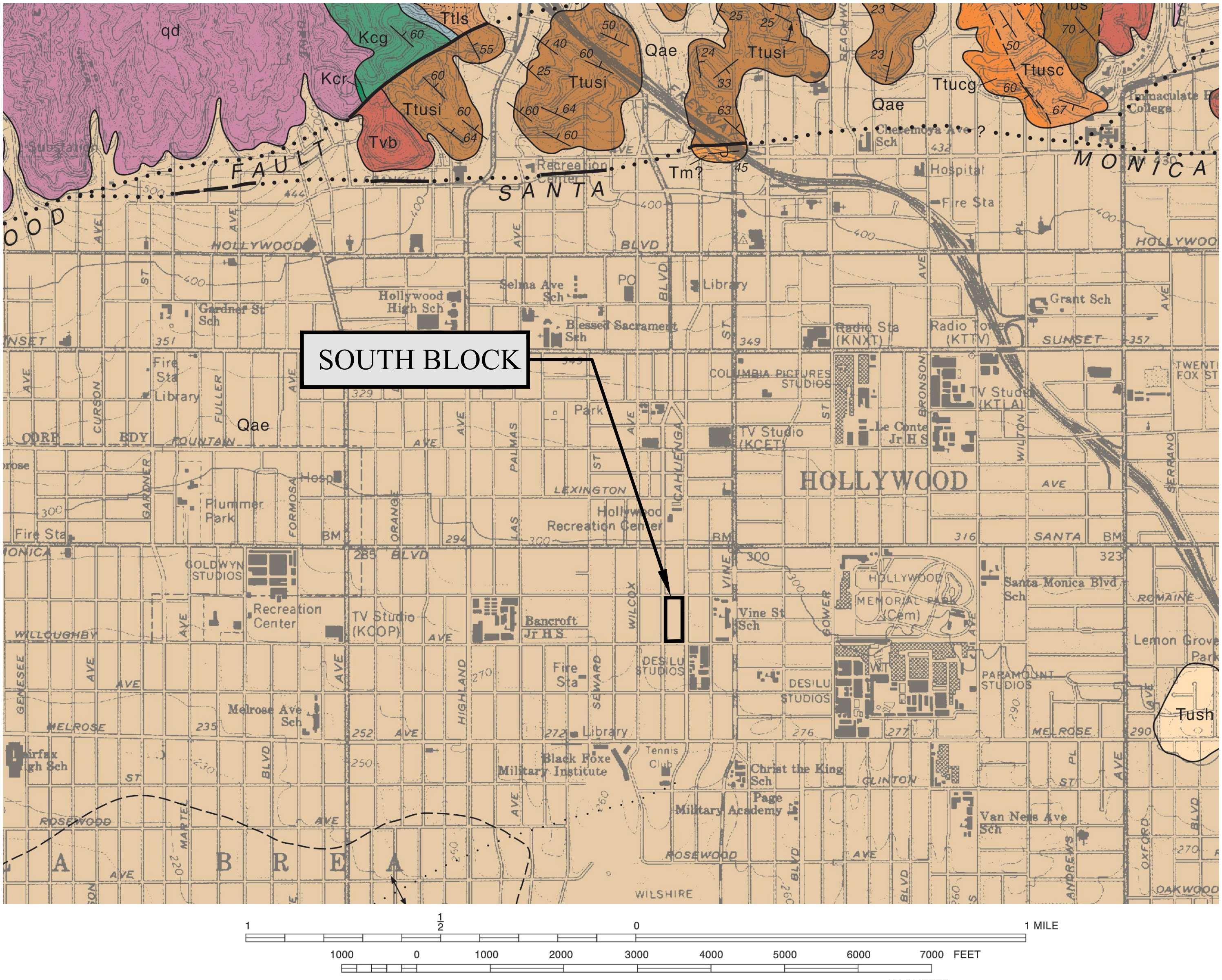
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOUTH BLOCK

FILE NO: 22218

PLATE: I



LEGEND

Qae *Similar to* **Qa***, but slightly elevated and dissected; includes alluvial fan sediments*

Tush *Gray to light brown, thin-bedded silty clay shale, soft and crumbly; locally contains scattered hard calcareous nodules; in places contains laminae of fine grained soft sandstone*

Tm *White-weathering, thin bedded, platy siliceous shale, hard to semi-chalky; at Griffith Park directly overlies granodiorite basement rocks if not in fault (?) contact*

Ttusi *Mostly gray micaceous clay shale or claystone, crumbly where weathered, and thin interbeds of gray to tan semi-friable sandstone*

Ttusc *Light gray massive sandstone, with pebble-cobble conglomerate of detritus as in*
Ttucg *Cahuenga Conglomerate Member (of Dibblee 1989; includes "Cahuenga" and "Griffith" Beds of Neuerburg 1953); light to medium gray, crudely bedded; ranges from coarse pebbly sandstone to cobble-boulder conglomerate composed mostly of granitic detritus (granite to quartz diorite) and some of metavolcanic rocks, quartzite, gneiss, and basalt, in coarse weakly coherent sandstone matrix; grades and intertongues westward and southward into* **Ttusc** *and*

Tvb *Basaltic volcanic rocks: dark gray to black, fine grained, massive to locally vesicular and/or pillowed; composed of mafic minerals (augite and olivine) and plagioclase feldspar;*

Ttls *Tan, moderately hard, thick-bedded arkosic sandstone*

Tsl *Simi Conglomerate Member: gray, vaguely bedded, cobble conglomerate of smooth*

Kcg *Gray to brown, crudely bedded conglomerate of cobbles and pebbles of metavolcanic and granitic rocks and quartzite in brown sandy matrix*

Kcr *"Trabuco" Formation (of Durrell 1954; Colburn, in Fritsche 1973): rusty-brown conglomerate similar to Kcg but locally includes reddish sandstone and claystone; probably nonmarine; base sheared locally; possibly in fault contact with basement rocks (Durrell 1954; Denison and Aguilar 1991)*

qd *Quartz diorite (Lar and Vermont biotite quartz diorite of Neuerburg 1953, in Griffith Park area), medium to light gray, massive to vaguely gneissoid; composed mostly of plagioclase feldspar, and moderate amounts of quartz, biotite, and hornblende; moderately hard to somewhat incoherent where weathered*

REFERENCE: T.W. DIBBLEE (EDITED 2010) GEOLOGIC MAP OF THE HOLLYWOOD & SOUTH HALF BURBANK QUADRANGLES (#DF-30)

N

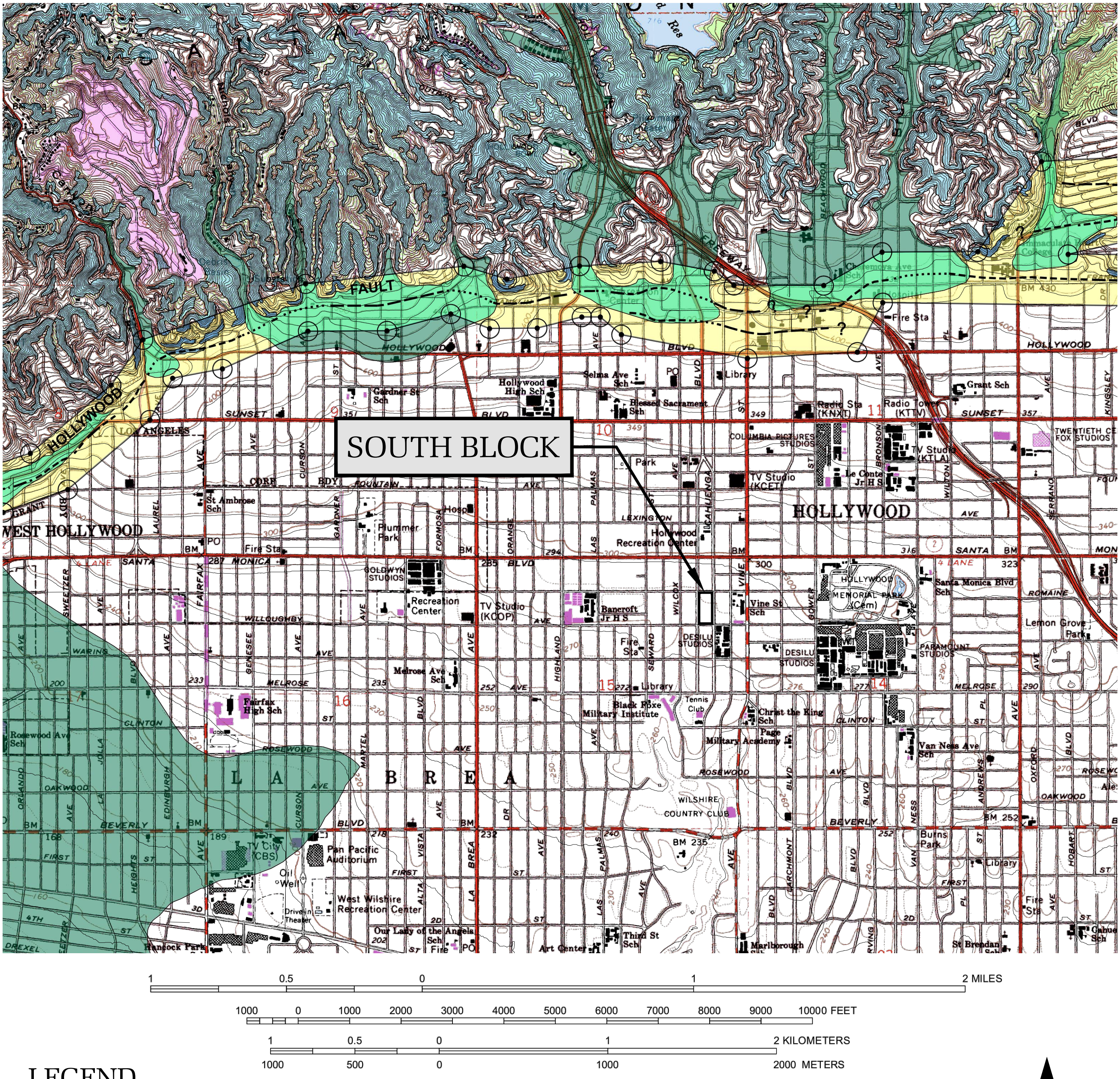
LOCAL GEOLOGIC MAP - DIBBLEE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOUTH BLOCK

FILE NO: 22218

PLATE: II



N

LEGEND

🟨 EARTHQUAKE FAULT ZONES

🟩 LIQUEFACTION ZONES

🟦 EARTHQUAKE-INDUCED LANDSLIDE ZONES

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

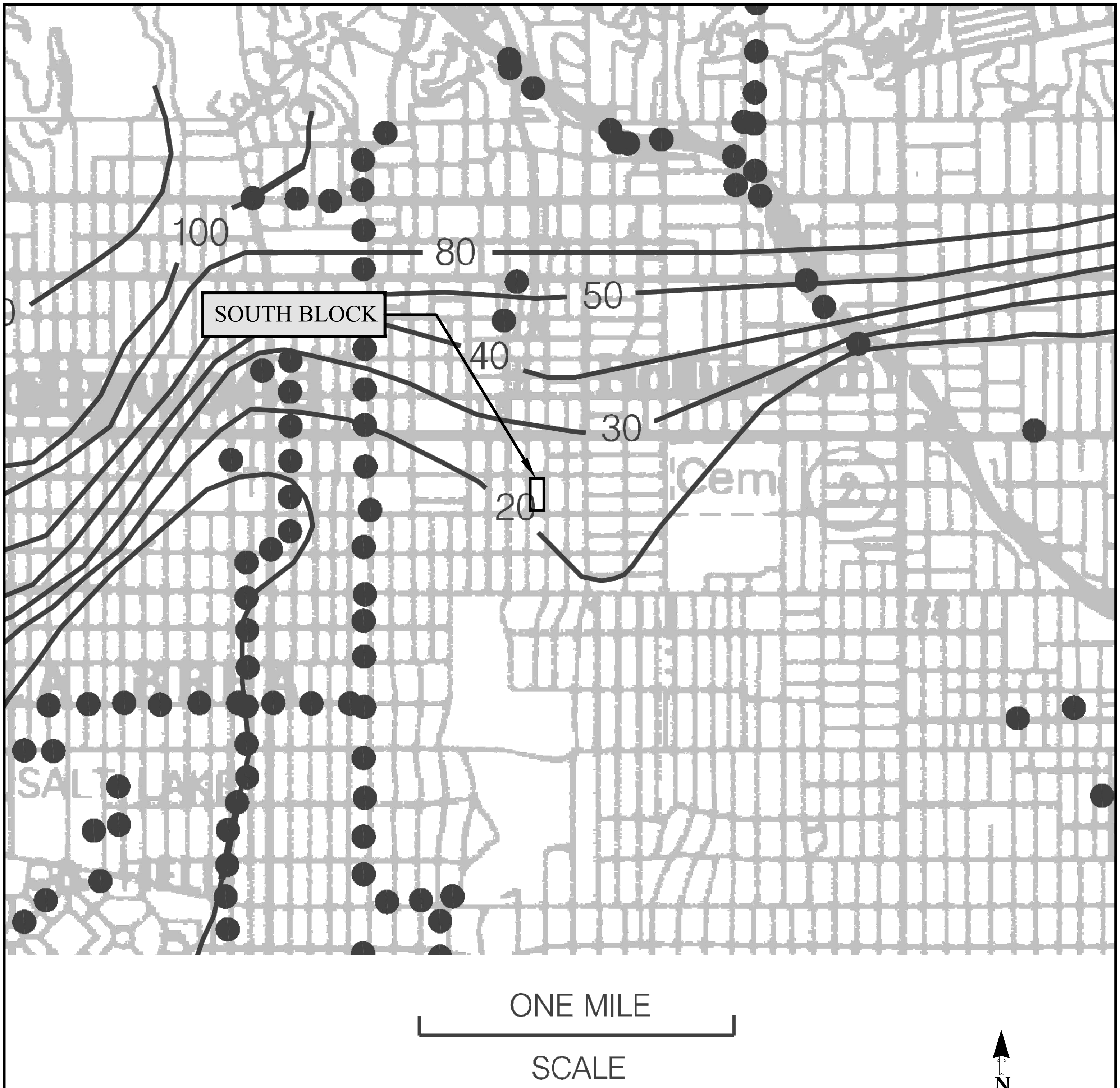
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOURTH BLOCK

FILE NO: 22218

PLATE: III



LEGEND

- ● Borehole Site
- 30 Depth to ground water in feet

REFERENCE: SEISMIC HAZARD ZONE REPORT, 026 HOLLYWOOD, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998

HISTORICALLY HIGHEST GROUND WATER LEVELS

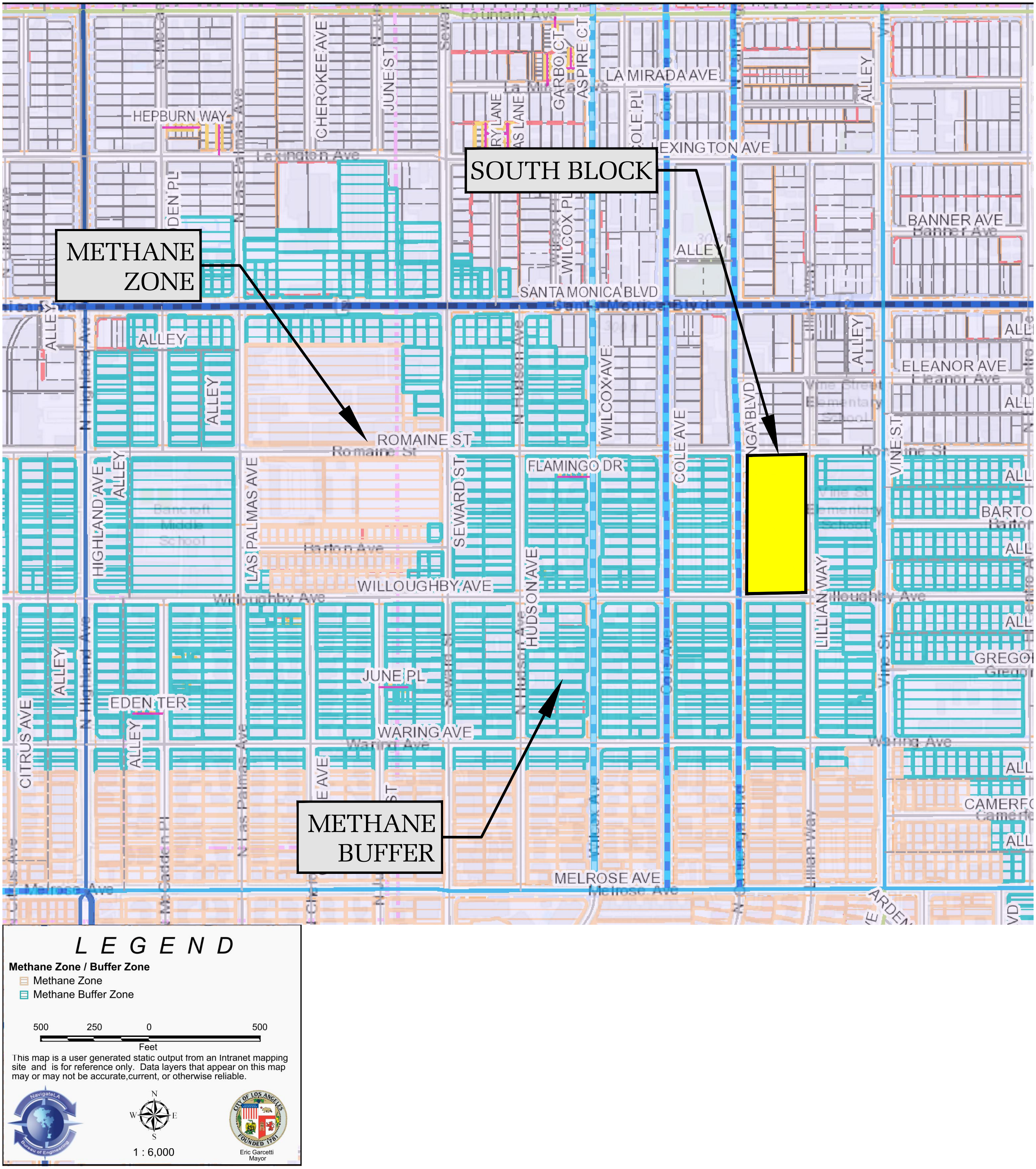
Geotechnologies, Inc.

Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOUTH BLOCK

FILE NO: 22218

PLATE: IV



This map is a user generated static output from an Intranet mapping site and is for reference only. Data layers that appear on this map may or may not be accurate,current, or otherwise reliable.

REFERENCE: https://navigatela.lacity.org/

METHANE ZONE RISK MAP

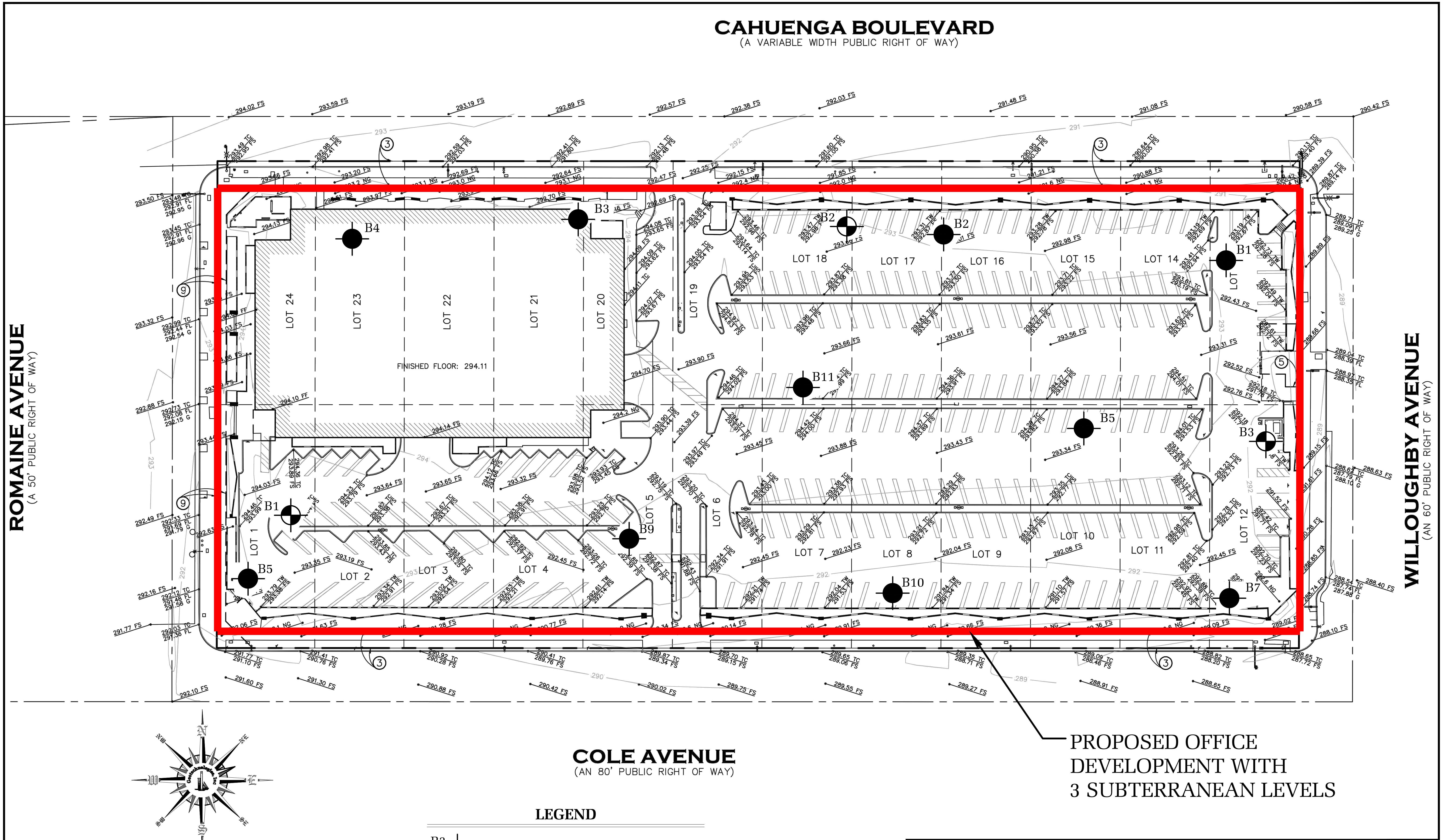
Geotechnologies, Inc.

Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOUTH BLOCK

FILE NO: 22218

PLATE: V



PROPOSED OFFICE DEVELOPMENT WITH 3 SUBTERRANEAN LEVELS

SCALE IN FEET

0 30 50 100

REFERENCE: TOPOGRAPHIC SURVEY BY JRN CIVIL ENGINEERS
DATED: March 9, 2021

LEGEND

B3	LOCATION AND NUMBER OF BORING
B11	LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION BY THE J. BYER GROUP, INC. DATED MAY 3, 1999)

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
TVC SOUTH BLOCK

Drawn by: YD	File No.: 22218
Date: March 2023	Plate: VI

APPENDIX II

BORING LOGS

BORING LOG NUMBER 1

Bardas Investment Group **Date: 03/24/21** **Elevation: 293.7'**

File No. 22218 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3½-inch Asphalt over 3½-inch Base
				-		
				1 --		
				-		FILL: Silty Clay, dark brown, moist, stiff
				2 --		
2.5	28	22.6	101.4	-		
				3 --		
				-	CL	Sandy Clay, dark gray, moist, stiff
				4 --		
				-		
5	11	18.6	SPT	5 --		
				-		Sandy Clay, dark gray to dark brown, moist, stiff
				6 --		
				-		
				7 --		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
7.5	24	21.0	100.6	-		
				8 --		
				-		
				9 --		
				-		
10	13	32.7	SPT	10 --		
				-	CH	Silty Clay, dark brown, moist, firm to stiff
				11 --		
				-		
				12 --		
12.5	34	32.7	90.2	-		
				13 --		
				-		
				14 --		
				-		
15	10	45.5	SPT	15 --		
				-		Silty Clay, dark brown, wet, medium firm, fine grained
				16 --		
				-		
				17 --		
17.5	20	38.9	83.6	-		
				18 --	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, wet, medium dense, fine grained, stiff
				-		
				19 --		
				-		
20	19	18.6	SPT	20 --		
				-	SC	Clayey Sand, dark to grayish brown, wet, medium dense to dense, fine grained
				21 --		
				-		
				22 --		
22.5	51	21.6	104.6	-		
				23 --		
				-		
				24 --		
				-		
25	22	14.2	SPT	25 --		
				-	SP	Sand, dark brown, wet, medium dense, fine to medium grained

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	24	30.5	93.2	-		
				28 --	SC	Clayey Sand, dark to grayish brown, wet, dense, fine grained grained
				-		
				29 --		
				-		
30	19	21.7	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	57	23.1	103.3	-		
				33 --		
				-		
				34 --		
				-		
35	38	15.2	SPT	35 --		
				-	SM/SP	Silty Sand to Sand, dark to yellowish brown, wet, dense, fine grained
				36 --		
				-		
				37 --		
37.5	68	17.7	113.3	-		
				38 --	SM	Silty Sand, dark brown, wet, very dense, fine to medium grained, occasional cobbles
				-		
				39 --		
				-		
40	34	17.7	SPT	40 --		
				-	ML/SM	Sandy Silt to Silty Sand, dark brown, moist, stiff to dense
				41 --		
				-		
				42 --		
42.5	34	21.2	110.9	-		
				43 --	SM	Silty Sand, dark brown, wet, dense, fine grained
				-		
				44 --		
				-		
45	24	25.7	SPT	45 --		
				-	CL	Sandy Clay, dark brown, moist, stiff
				46 --		
				-		
				47 --		
47.5	52	20.0	107.7	-		
				48 --		
				-		
				49 --		
				-		
50	19	30.6	SPT	50 --		
				-		

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	49	33.7	88.0	-		
				53 --	CH	Silty Clay, dark brown, very moist, stiff
				-		
				54 --		
				-		
55	23	27.8	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	83	23.5	103.5	-		
				58 --	CL	Sandy Clay, dark to grayish brown, very moist, stiff
				-		
				59 --		
				-		
60	20	30.6	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	85	21.6	107.1	-		
				63 --		
				-		
				64 --		
				-		
65	23	22.8	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	68 50/5"	17.5	109.4	-		
				68 --	SM/CL	Silty Sand to Sandy Clay, dark brown, moist, very dense to very stiff, fine grained
				-		
				69 --		
				-		
70	24	20.6	SPT	70 --		
				-	CL	Sandy Clay, dark brown to gray, moist, stiff
				71 --		
				-		
				72 --		
72.5	68 50/5"	30.4	94.1	-		
				73 --		
				-		
				74 --		
				-		
75	31	26.4	SPT	75 --		
				-		

BORING LOG NUMBER 1

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	39	17.2	116.5	-		
	50/5"			78 --	SM	Silty Sand to Sandy Clay, dark to grayish brown, very moist, very dense to very stiff
				-		
				79 --		
				-		
80	37	13.7	SPT	80 --		
				-	SM/SP	Silty Sand to Sand, dark brown, wet, very dense, fine to medium grained
				81 --		
				-		
				82 --		
82.5	75	23.8	100.5	-		
	50/4"			83 --	CL	Sandy Clay, dark grayish brown, very moist, very stiff
				-		
				84 --		
				-		
85	38	25.2	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87.5	89	27.5	98.5	-		
				88 --		
				-		
				89 --		
				-		
90	33	20.1	SPT	90 --		
				-		
				91 --		
				-		
				92 --		
92.5	42	24.6	100.1	-		
	50/5"			93 --		Sandy Clay, dark and gray, very moist, very stiff
				-		
				94 --		
				-		
95	46	20.4	SPT	95 --		
				-		
				96 --		
				-		
				97 --		
97.5	92	19.4	109.7	-		
				98 --	SM/CL	Silty Sand to Silty Clay, dark grayish brown mottling, very moist, very dense to very stiff, fine grained
				-		
				99 --		
				-		
100	41	19.1	SPT	100 --		
				-		Total Depth 100 feet Water at 15½ feet Fill to 3 feet

BORING LOG NUMBER 2

Bardas Investment Group **Date: 03/25/21** **Elevation: 292.9'**

File No. 22218 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty Clay, dark brown, moist, stiff, minor brick fragments
				-		
				2 --		
2.5	45	15.1	115.5	-		
				3 --		
				-	CL	Sandy Clay, dark brown, moist, stiff
				4 --		
				-		
5	72	14.8	118.1	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	62	20.2	104.2	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	45	17.1	116.9	15 --		
				-		Sandy Clay, dark brown and gray, moist, medium dense, stiff, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	63	19.5	110.9	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	68	14.6	119.5	25 --		
	50/5"			-	SM	Silty Sand, dark brown to gray, moist, very dense, fine grained

BORING LOG NUMBER 2

Bardas Investment Group

File No. 22218

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	89	15.0	117.9	30 --		
				-	SC	Clayey Sand, dark grayish brown, moist, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	48	18.2	116.0	35 --		
	50/5"			-	SM	Silty Sand, dark brown, very dense, fine grained, with occasional gravel
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	63	20.5	108.5	40 --		
				-		Silty Sand, dark to grayish brown, moist, dense, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	68	22.4	106.0	45 --		
	50/5"			-	SM/CL	Silty Sand to Sandy Clay, dark brown to yellowish brown, moist, very dense to very stiff, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	81	27.5	99.1	50 --		
				-	CL	Sandy Clay, dark brown to gray, very moist, stiff

BORING LOG NUMBER 2

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	78	16.1	114.1	55 --		
				-	SP	Sand, dark brown, wet, very dense, fine to medium grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	70	22.4	100.9	60 --		
				-	SM/ML	Silty Sand to Sandy Clay, dark brown, very moist, very dense to stiff, fine grained
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	94	17.0	112.2	65 --		
				-	SM	Silty Sand, dark brown, wet, very dense, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	100/9"	13.1	111.6	70 --		
				-	SM/SP	Silty Sand to Sand, dark brown, wet, very dense, fine to medium grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	100/8"	20.3	104.7	75 --		
				-	SP	Sand, dark brown, wet, very dense, fine to medium grained

BORING LOG NUMBER 2

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		
				-		
80	100/8"	18.1	106.0	80 --		
				-		Total Depth 80 feet
				81 --		Water at 16½ feet
				-		Fill to 3 feet
				82 --		
				-		
				83 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				84 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				85 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

BORING LOG NUMBER 3

Bardas Investment Group **Date: 03/26/21** **Elevation: 291.6'**

File No. 22218 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Silty Clay, dark gray, moist, stiff
				-		
				2 --		
2.5	27	21.6	104.9	-		
				3 --		
				-	CL	Sandy Clay, dark gray, moist, stiff
				4 --		
				-		
5	54	17.9	109.9	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	90	14.9	116.2	10 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	82	12.4	117.9	15 --		
				-	SM/SP	Silty Sand to Sand, dark to grayish brown, moist, very dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	64	21.6	108.2	20 --		
	50/5"			-	CL	Sandy Clay, dark grayish brown, moist, very stiff, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	81	19.6	107.8	25 --		
				-		

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22218

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	68	19.5	107.2	30 --		
				-	SP	Sand, dark brown, wet, very dense, fine to medium grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	52	24.0	103.4	35 --		
				-	SP/CL	Sand to Sandy Clay, dark brown, wet, dense to very stiff, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	64	20.2	108.7	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	64	18.3	110.2	45 --		
				-	SP	Sand, dark to yellowish brown, wet, very dense, fine to medium grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	80	24.9	99.6	50 --		
				-	SM	Silty Sand, gray, wet, very dense, fine grained

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	61	18.8	111.2	55 --		
	50/5"			-	SM	Silty Sand, dark and gray, wet, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	86	20.3	107.5	60 --		
				-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	44	17.3	115.0	65 --		
	50/4"			-	SW	Sand to Gravelly Sand, wet, very dense, fine to coarse grained, with gravel and cobbles
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	45	22.2	100.1	70 --		
	50/5"			-		BEDROCK: Siltstone, gray to dark gray, moist, moderately hard
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	46	11.9	111.5	75 --		
	50/5"			-		

BORING LOG NUMBER 3

Bardas Investment Group

File No. 22218

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		
				-		
80	100/8"	22.0	103.2	80 --		
				-		Total Depth 80 feet
				81 --		Water at 21.5 feet
				-		Fill to 3 feet
				82 --		
				-		
				83 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				84 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				85 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

Project No: JB 18051-B

Log of Boring 1

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface **FILL:** Sandy Gravel, brown, red, gray, slightly moist to dry, medium dense to dense, concrete and brick fragments to 4 inches								
-1	1			SW						
-2	2	Sandy Clay, brown, moist, firm								
-3	3	**ALLUVIUM:** Silty Sand, brown to light reddish brown, moist very dense, some clay binder		CL	R	34	16.8	117.4		
-4	4			SM						
-5	5			----	R	50	12.0	120.6		
-6	6									
-7	7	dark brown, slightly porous		----	R	50	15.7	115.4		
-8	8									
-9	9									
-10	10	Gravelly Sand, reddish brown, slightly moist, very dense		----	R	50 6"	10.8	118.4		
-11	11			SW						
-12	12									
-13	13									
-14	14									
-15	15									
-16	16									
-17	17									
-18	18									
-19	19			----	R	50 10"	17.1	110.4		
-20	20	End at 20 Feet; No Water; Fill to 3 Feet;								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 2

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL:** Silty Sand, light brown, slightly moist, medium dense, concrete and brick fragments to 2 inches								
-1	1			SM						
		Clay, dark brown to black, moist, firm								
-2	2			CL	R	20	18.5	108.8		
		ALLUVIUM: Sandy Clay, dark brown, moist, stiff to very stiff								
-3	3									
-4	4			CL						
-5	5			----	R	45	10.2	121.2		
		Silty Sand, light reddish brown, slightly moist, very dense, some clay binder, some caliche veins								
-6	6			SM						
-7	7	Silty Sand, light brown to brown, slightly moist very dense								
-8	8									
-9	9									
-10	10	some gravel		----	R	30	10.4	114.7		
-11	11	Sandy Silt, brown to gray brown, moist, very firm								
-12	12			ML						
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet.		----	R	50	18.0	112.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 3

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	FILL: Clay, dark brown to black, moist, firm, some concrete fragments to 2 inches								
-1	1									
-2	2			CL	R	16	19.0	108.6		
-3	3									
-4	4	ALLUVIUM: Sandy Clay, black to brown, moist, very firm								
-5	5			CL	R	17	16.4	112.8		
-6	6	Silty Sand, light reddish brown, slightly moist to moist, dense to very dense, some clay binder								
-7	7			SM	R	33	19.0	108.6		
-8	8	Sandy Silt, light brown to reddish brown, moist very firm		ML						
-9	9									
-10	10	Clayey Silt, reddish brown to grayish brown, moist, very firm, slightly porous		----	R	22	17.9	106.5		
-11	11			ML						
-12	12									
-13	13									
-14	14	Silty Sand, light reddish brown to gray, slightly moist, dense to very dense								
-15	15	End at 15 Feet; No Water; Fill to 4 Feet		SM	R	31	13.1	118.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 4

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black to dark brown, moist, firm								
-2	2	Mixed black Clay and brown Silty Sand, moist medium dense		CL	R	18				
-5	5	ALLUVIUM: Clay, black, moist, very firm		----	R	12	24.4	95.0		
-6	6			CL						
-7	7			----	R	18	21.2	104.0		
-10	10	Silt, light gray brown, moist to very moist, firm		----	R	13	27.0	97.4		
-11	11			ML						
-15	15	Clayey Silt, greenish gray, moist to very moist very firm		----	R	18	24.3	102.5		
-17	17	Sandy Silt, reddish brown to gray, moist, very firm								
-19	19			----	R	24	27.9	96.6		
-20	20	End at 20 Feet; Fill to 5 Feet.								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 5

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL:** Clay, black, moist, firm								
-1	1									
-2	2			CL	R	17	22.7	102.3		
-3	3									
-4	4									
-5	5	**ALLUVIUM:** Silty Sand, brown to dark brown, moist, dense		SM	R	15	14.9	112.4		
-6	6									
-7	7	Sandy Silt, dark gray brown, moist, very firm, some caliche veins		----	R	22	21.5	103.5		
-8	8			ML						
-9	9									
-10	10	Silty Clay, gray brown to brown, moist, very firm		----	R	16	30.9	89.8		
-11	11			CL						
-12	12			----	R	12	39.0	77.7		
-13	13									
-14	14									
-15	15	Sand, gray to brownish gray, wet, dense, fine to medium grained		----	R	16	21.9	101.7		
-16	16			SP						
-17	17									
-18	18	medium to coarse								
-19	19	Gravelly Sand, grayish brown to gray, wet, dense to very dense, End at 20 Feet; water at 15 Feet; Fill to 4½ Feet.		SW	R	48	118.5	118.5		
-20	20									

Surface: 2 Inch AC/4 Inch Base

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 6

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface FILL: Silty Sand, brown, moist, dense								
-1	1			SM						
-2	2	ALLUVIUM: Clay, dark gray brown to black, moist, stiff to very stiff								
-3	3			CL						
-4	4			----	R	19	21.3	104.7		
-5	5									
-6	6	Sandy Clay, brown to gray brown, moist, very stiff		----	R	23	19.4	109.2		
-7	7									
-8	8	Silty Sand, reddish brown to brown, moist, very dense, some gravel, some clay binder		----	R	34	17.4	111.8		
-9	9			SM						
-10	10									
-11	11	Gravelly Sand, reddish brown, slightly moist, very dense								
-12	12			SW	R	40	11.4	108.2		
-13	13									
-14	14									
-15	15			----	R	32	12.6	112.5		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 2

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 6

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				----	R	50 10"	11.4	122.1		
-21	21									
-22	22									Water Measured at 21½ Feet After One Hour
-23	23									
-24	24	Silty Sand, light brown to light reddish brown, moist, dense								
-25	25			SM	SPT	6 9 12				
-26	26									
-27	27									Water Encountered at 26½ Feet
-28	28									
-29	29									
-30	30	Sandy Silt, light brown to gray, moist to very moist, very firm		----	SPT	8 14 14				
-31	31			ML						
-32	32									
-33	33									
-34	34									
-35	35	Silty Sand, light brown to brownish gray, moist dense to very dense		----	SPT	15 18 16				
-36	36			SM						
-37	37									
-38	38									
-39	39			----	SPT	16 18 19				
-40	40	End at 40 Feet; Water at 21½ Feet; Fill to 2 Feet								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 2 of 2

Project No: JB 18051-B

Log of Boring 7

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black, moist, firm	▨							
-1	1		▨	CL						
-2	2		▨	----	R	17	18.2	110.0		
-3	3		▨							
-4	4		▨							
		ALLUVIUM: Clayey Sand, light reddish brown, moist, very dense, some gravel	░							
-5	5		░	----	R	20	18.5	109.5		
-6	6		░	SC						
-7	7		░	----	R	36	15.9	113.6		
		Silty Sand, red brown, slightly moist, very dense, some clay binder	▒							
-8	8		▒	SM						
-9	9		▒							
-10	10		▒	----	R	30	9.9	111.9		
-11	11		▒							
-12	12		▒							
-13	13		▒							
-14	14		▒							
		End at 15 Feet; No Water; Fill to 4½ Feet.	▒							
-15	15			----	R	32	18.9	106.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-27-99

Size: 8 Inch
Elevation:
Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 8

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
-1	1	FILL: Gravelly Sand, dark gray brown, slightly moist dense		SW						
-2	2	ALLUVIUM: Sandy Clay, black, moist, stiff, slightly porous		—	R	16	20.3	105.2		
-3	3			CL						
-4	4	dark reddish brown, some gravel								
-5	5			----	R	21	14.0	113.9		
-6	6	Silty Sand, reddish brown, moist, dense to very dense, some clay binder, some gravel								
-7	7			SM	R	22	17.4	110.5		
-8	8									
-9	9	dense, slightly porous								
-10	10			----	R	15	13.0	109.8		
-11	11									
-12	12									
-13	13									
-14	14	Silty Sand, dark gray to brownish gray, moist, dense, slightly porous End at 15 Feet; No Water; Fill to 1½ Feet.								
-15	15			----	R	17	17.2	113.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Concrete Driveway

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Clay, black, moist, firm								
-1	1			CL						
-2	2	Silty Sand, brown to gray brown, moist, medium dense								
-3	3	**ALLUVIUM:** Sandy Clay, dark reddish brown, moist, stiff, some gravel		SM	R	10	12.3	112.2		
-4	4	slightly porous		CL						
-5	5	Clay, black to dark gray brown, moist, stiff, slightly porous								
-6	6			----	R	17	21.1	104.7		
-7	7									
-8	8									
-9	9	Silty Clay, brown, moist, firm to very firm		----	R	14	23.1	99.4		
-10	10									
-11	11	Sandy Clay, brown to brownish gray, moist, stiff to very stiff								
-12	12			----	R	25	15.6	114.3		
-13	13									
-14	14									
-15	15	Clay, gray to brown, moist, stiff		----	R	15	25.3	98.6		
-16	16									
-17	17									
-18	18									
-19	19									
		Clayey Sand, brown to reddish brown, moist, dense to very dense with some gravel								
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				SC	R	26	21.8	106.3		
-21	21									
-22	22									
-23	23									
-24	24									Water Encountered at 24 Feet
-25	25	Silty Sand, brown to gray, wet, dense, some clay binder		----	R	17	19.6	106.0		
-26	26			SM						
-27	27									
-28	28									
-29	29									
-30	30			----	SPT	7 10 14				
-31	31									
-32	32									
-33	33									
-34	34									
-35	35	Sandy Silt, brown, wet, firm		----	SPT	9 12 16				
-36	36			ML						
-37	37									
-38	38									
-39	39									
-40	40									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 2 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
		Silty Sand, light reddish brown to gray, wet, dense to very dense		----	SPT	14 16 19				
-41	41			SM						
-42	42									
-43	43									
-44	44									
-45	45	Sandy Silt, brown to gray, very moist, very firm		----	SPT	10 14 16				
-46	46			ML						
-47	47									
-48	48									
-49	49									
-50	50	End at 50 Feet; Water at 16 Feet; Fill to 3 Feet.		----	SPT	9 12 15				Water Encountered at 16 Feet after one hour
-51	51									
-52	52									
-53	53									
-54	54									
-55	55									
-56	56									
-57	57									
-58	58									
-59	59									
-60	60									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 3 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 10

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Silty Sand, brown, slightly moist, medium dense	▦							
-1	1	Sandy Clay, dark brown, moist, firm	▤	SM						
-2	2	Clay, black, moist, stiff, slightly porous	▨	CL	R	17	21.4	104.3		
-3	3									
-4	4	Sandy Clay, mottled dark gray brown to brown	▤							
-5	5			—	R	16	19.3	107.5		
-6	6	**ALLUVIUM:** Clayey Sand, reddish brown, moist, dense	░	SC						
-7	7	Silty Sand, reddish brown, slightly moist, dense to very dense	░							
-8	8									
-9	9			SM						
-10	10			—	R	36	15.4	110.8		
-11	11									
-12	12	light reddish brown to gray	░							
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 5½ Feet.		—	R	18	12.8	110.6		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 11

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black, moist, firm, fragments of brick	▩							
-1	1		▩	CL						
-2	2	**ALLUVIUM:** Clay, black, moist, very firm, slightly porous	▨	----	R	15	21.3	103.8		
-3	3		▨							
-4	4	dark brown	▨							
-5	5	Sandy Clay, reddish brown, moist, very stiff, some gravel	▤							
-6	6		▤	----	R	35	14.0	117.4		
-7	7		▤							
-8	8	Silty Sand, reddish brown, moist, very dense, some gravel	▥							
-9	9		▥	SM	R	26	11.0	113.5		
-10	10		▥	SM						
-11	11		▥							
-12	12		▥							
-13	13	Sandy Silt, reddish brown to gray, moist, very firm	▦							
-14	14	End at 15 Feet; No Water; Fill to 2 Feet	▦							
-15	15			----	R	23	24.3	101.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 12

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL** Clayey Sand, brown to dark brown, moist, dense								
-1	1			SC						
-2	2	**ALLUVIUM:** Clay, black, moist, stiff		----	R	20	27.1	97.9		
-3	3			CL						
-4	4	Sandy Clay, brown, moist, very firm, slightly porous								
-5	5			----	R	19	19.2	106.8		
-6	6									
-7	7						20.8	109.4		
-8	8			----	R	21				
-9	9	Sandy Silt, brown to light brown, moist, firm								
-10	10			ML	R	12	20.0	100.5		
-11	11									
-12	12	Clayey Silt, dark gray brown to brown, moist, firm								
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet		----	R	14	28.6	92.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-28-99

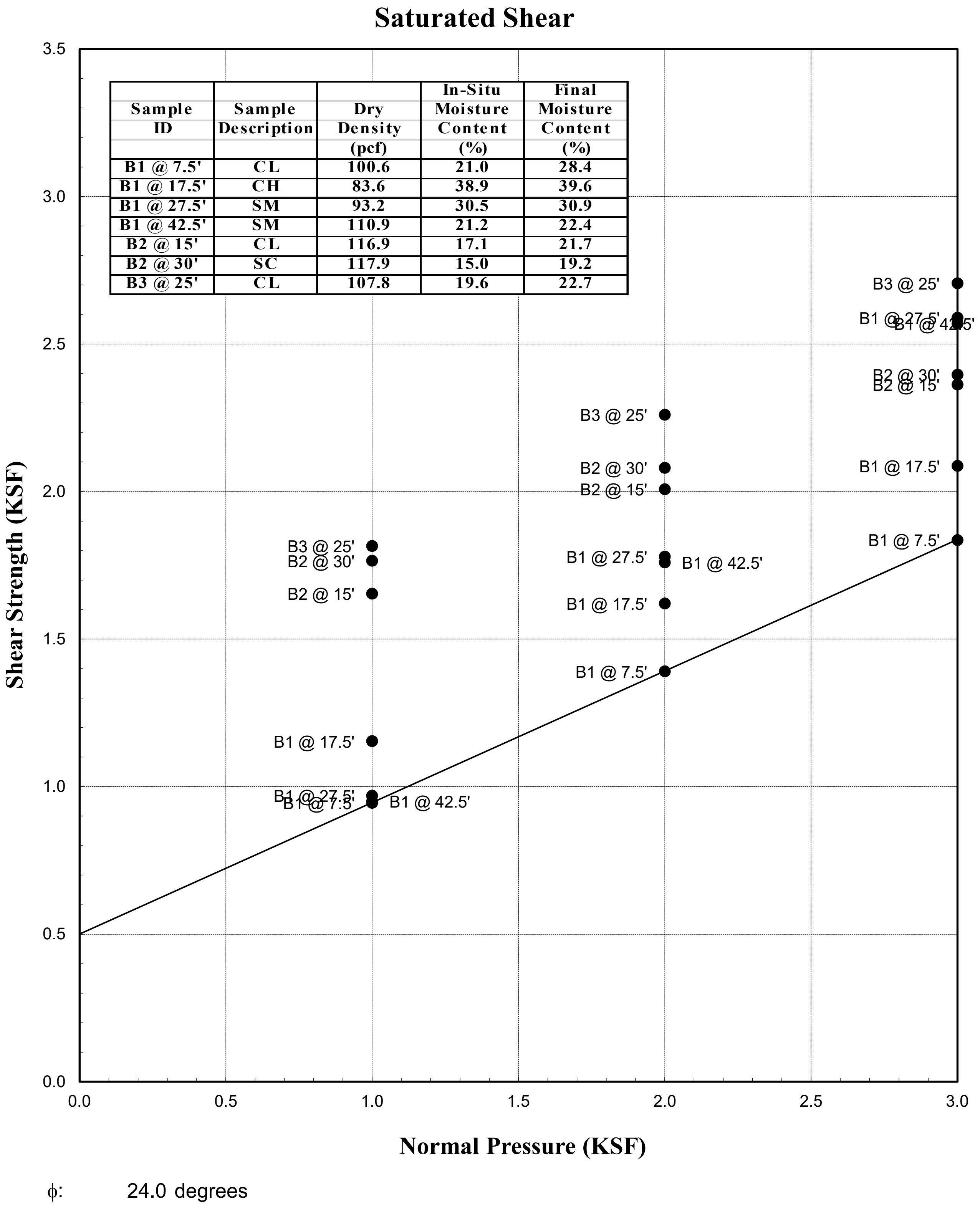
Size: 8 Inch
Elevation:
Sheet: 1 of 1

APPENDIX III

LABORATORY TESTING

SOIL CORROSIVITY REPORT

AND ANALYSIS



Saturated Shear

Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 7.5'	CL	100.6	21.0	28.4
B1 @ 17.5'	CH	83.6	38.9	39.6
B1 @ 27.5'	SM	93.2	30.5	30.9
B1 @ 42.5'	SM	110.9	21.2	22.4
B2 @ 15'	CL	116.9	17.1	21.7
B2 @ 30'	SC	117.9	15.0	19.2
B3 @ 25'	CL	107.8	19.6	22.7

ϕ: 24.0 degrees
c: 500.0 psf

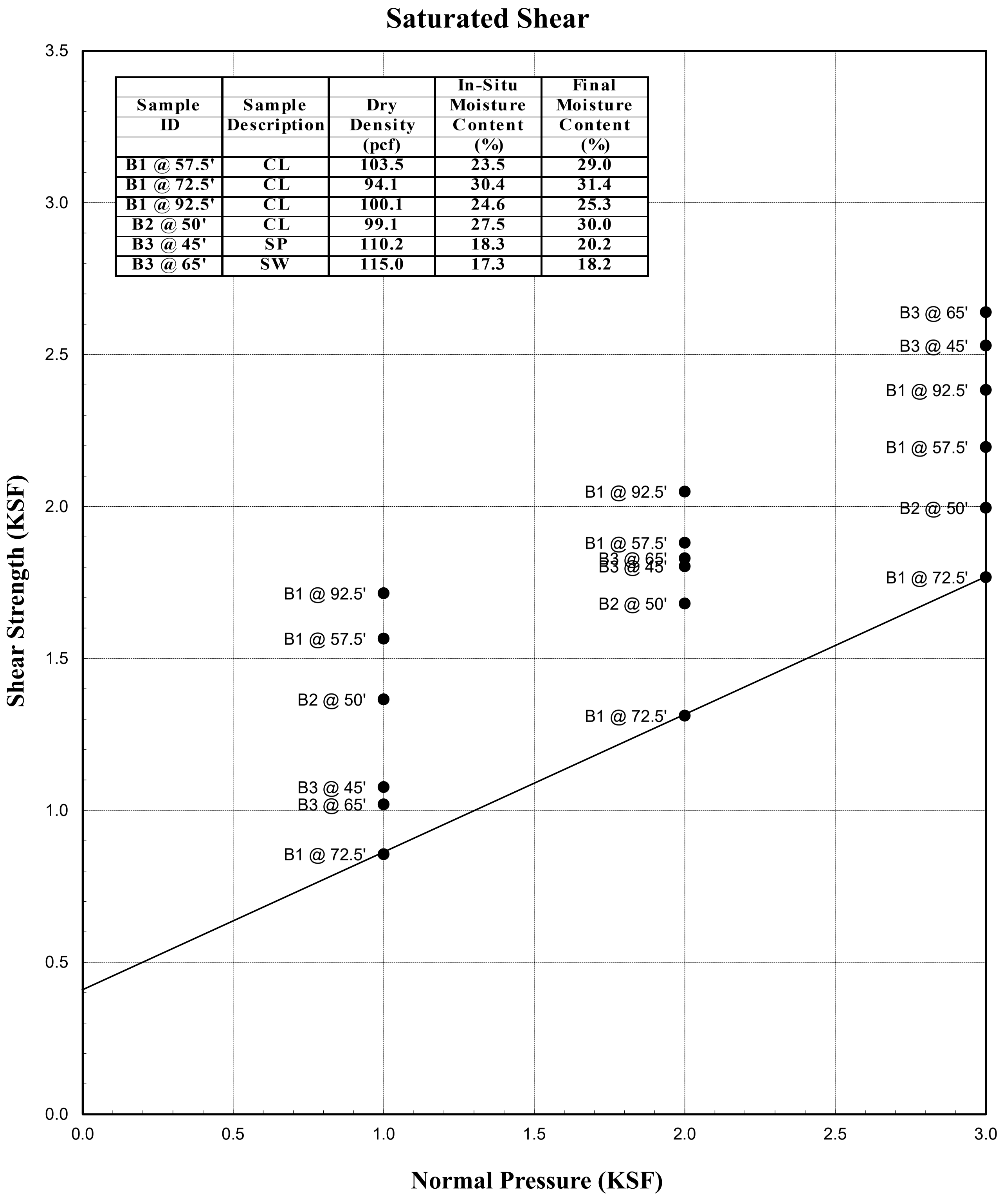
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22218

PLATE: B-1



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 57.5'	CL	103.5	23.5	29.0
B1 @ 72.5'	CL	94.1	30.4	31.4
B1 @ 92.5'	CL	100.1	24.6	25.3
B2 @ 50'	CL	99.1	27.5	30.0
B3 @ 45'	SP	110.2	18.3	20.2
B3 @ 65'	SW	115.0	17.3	18.2

ϕ: 24.5 degrees
c: 400.0 psf

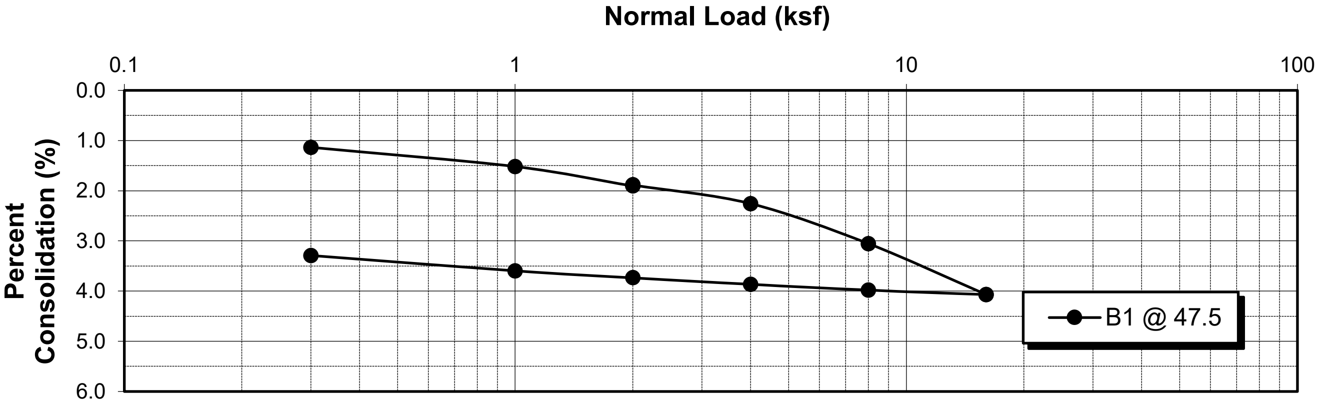
SHEAR TEST DIAGRAM

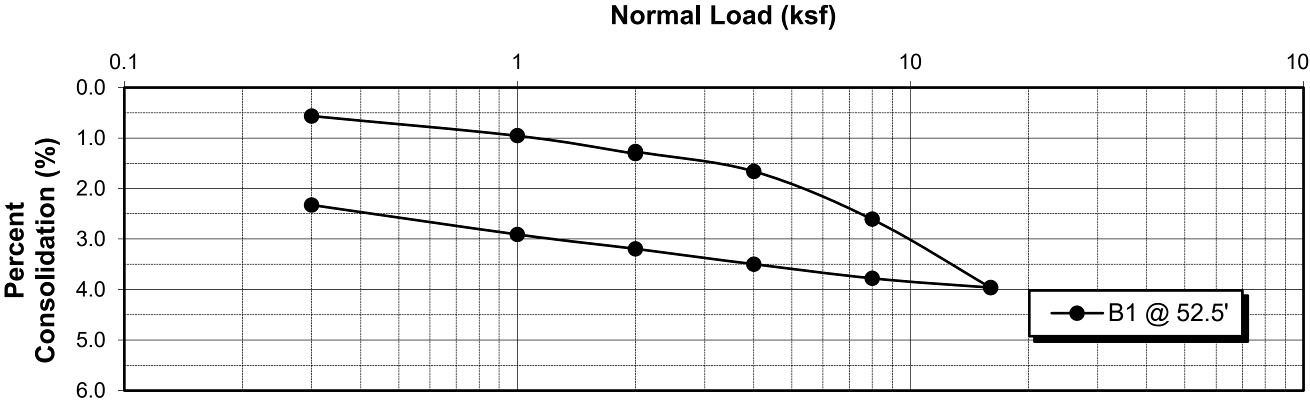
Geotechnologies, Inc.
Consulting Geotechnical Engineers

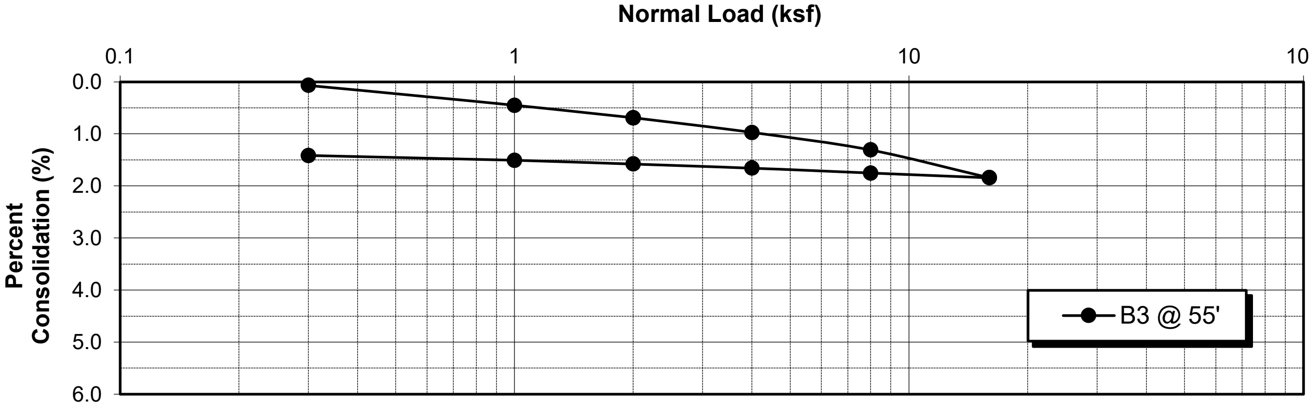
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22218

PLATE: B-2







Water added at 2 KSF

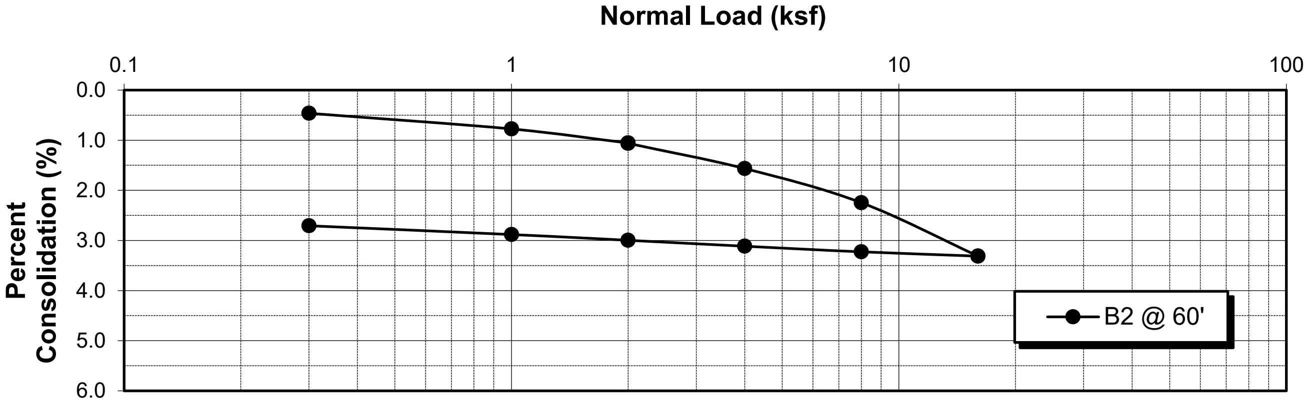
CONSOLIDATION

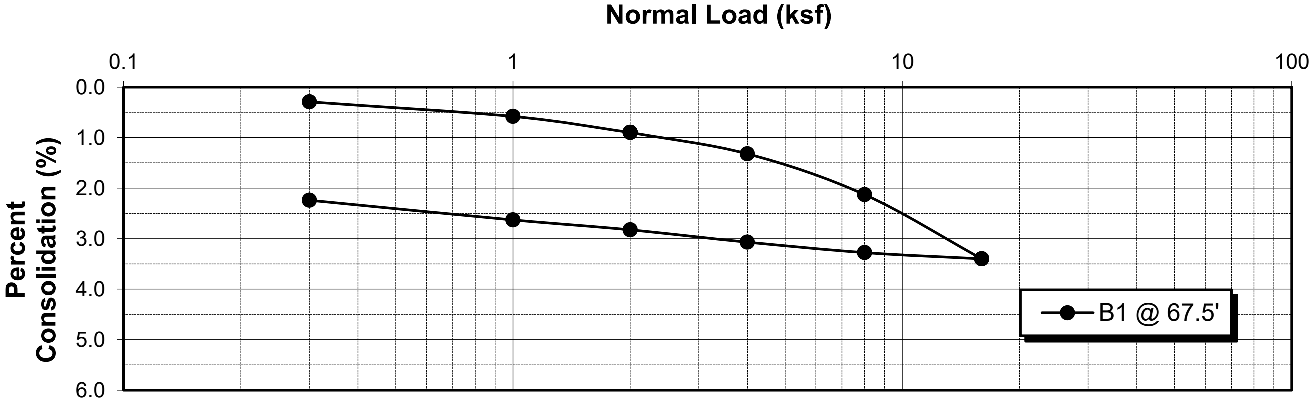
Geotechnologies, Inc.
Consulting Geotechnical Engineers

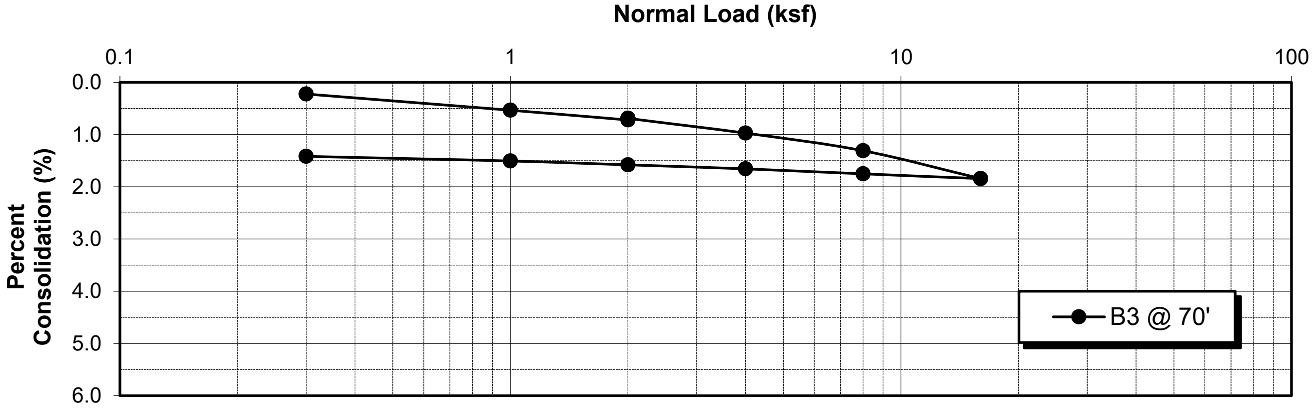
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22218

PLATE: C-1







Water added at 2 KSF

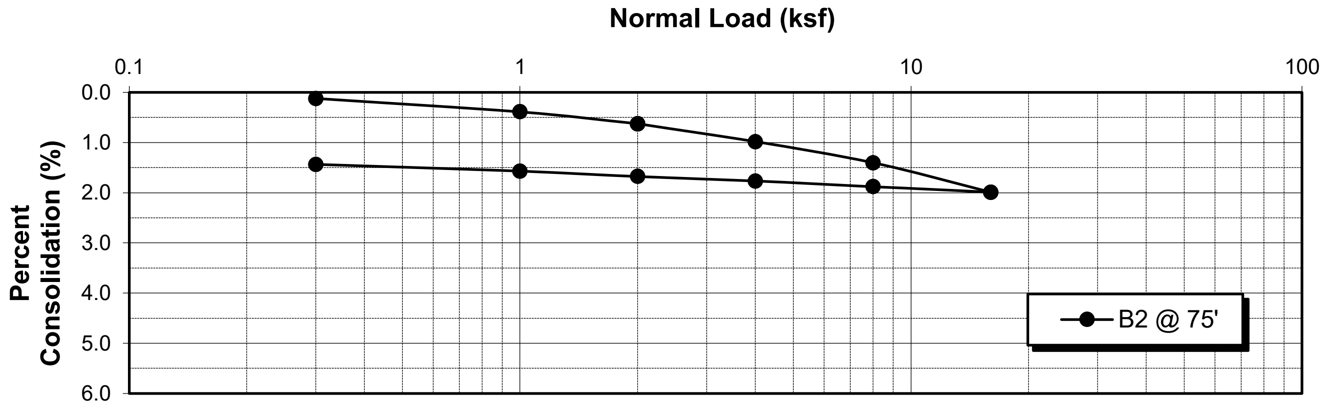
CONSOLIDATION

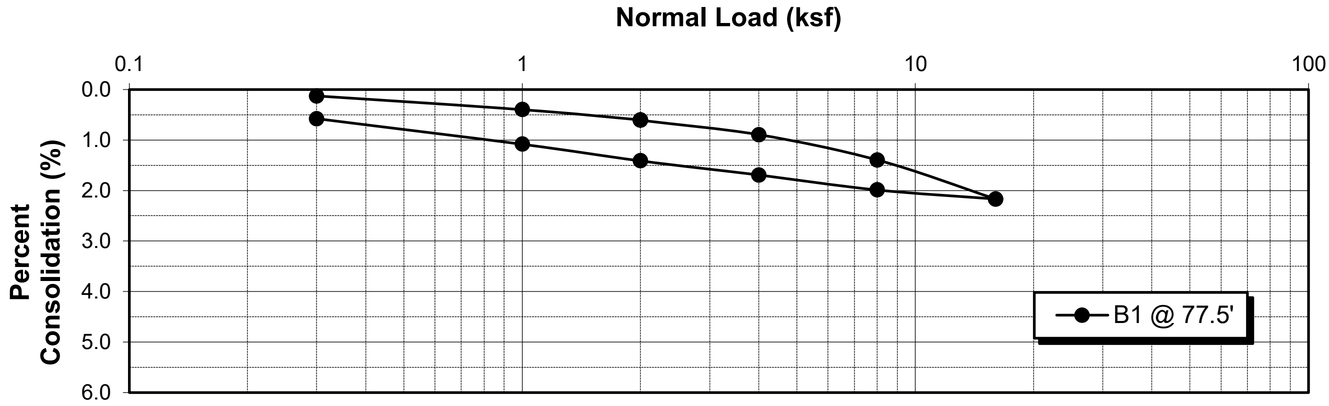
Geotechnologies, Inc.
Consulting Geotechnical Engineers

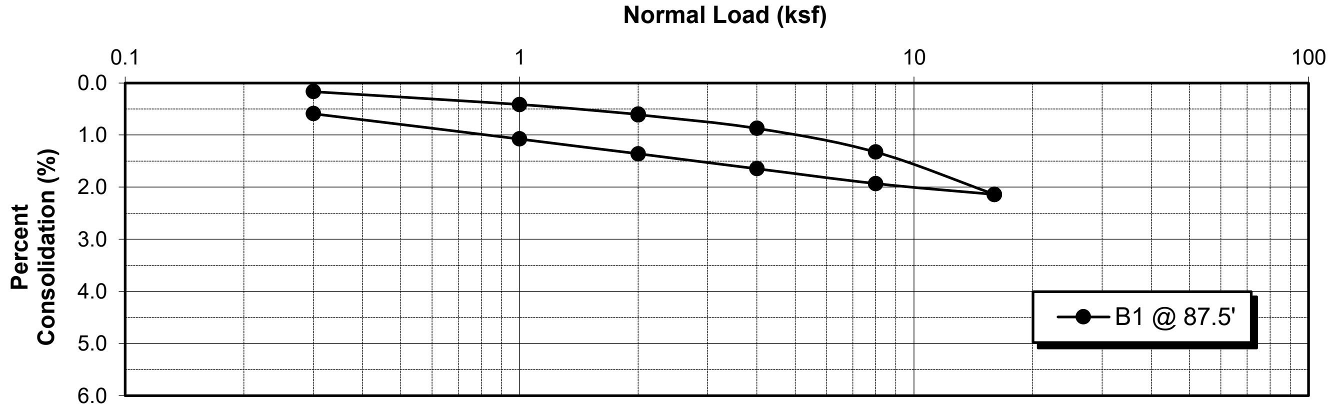
PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22218

PLATE: C-2







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: BARDAS INVESTMENT GROUP

FILE NO.: 22218

PLATE: C-3

ASTM D-1557

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/CL	SM/CL
MAXIMUM DENSITY pcf.	122.5	125.0
OPTIMUM MOISTURE %	12.5	11.5

ASTM D 4829

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SOIL TYPE:	SM/CL	SM/CL
EXPANSION INDEX UBC STANDARD 18-2	106	98
EXPANSION CHARACTER	HIGH	HIGH

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B3 @ 1-5'
SULFATE CONTENT: (percentage by weight)	> 0.15%	> 0.15%

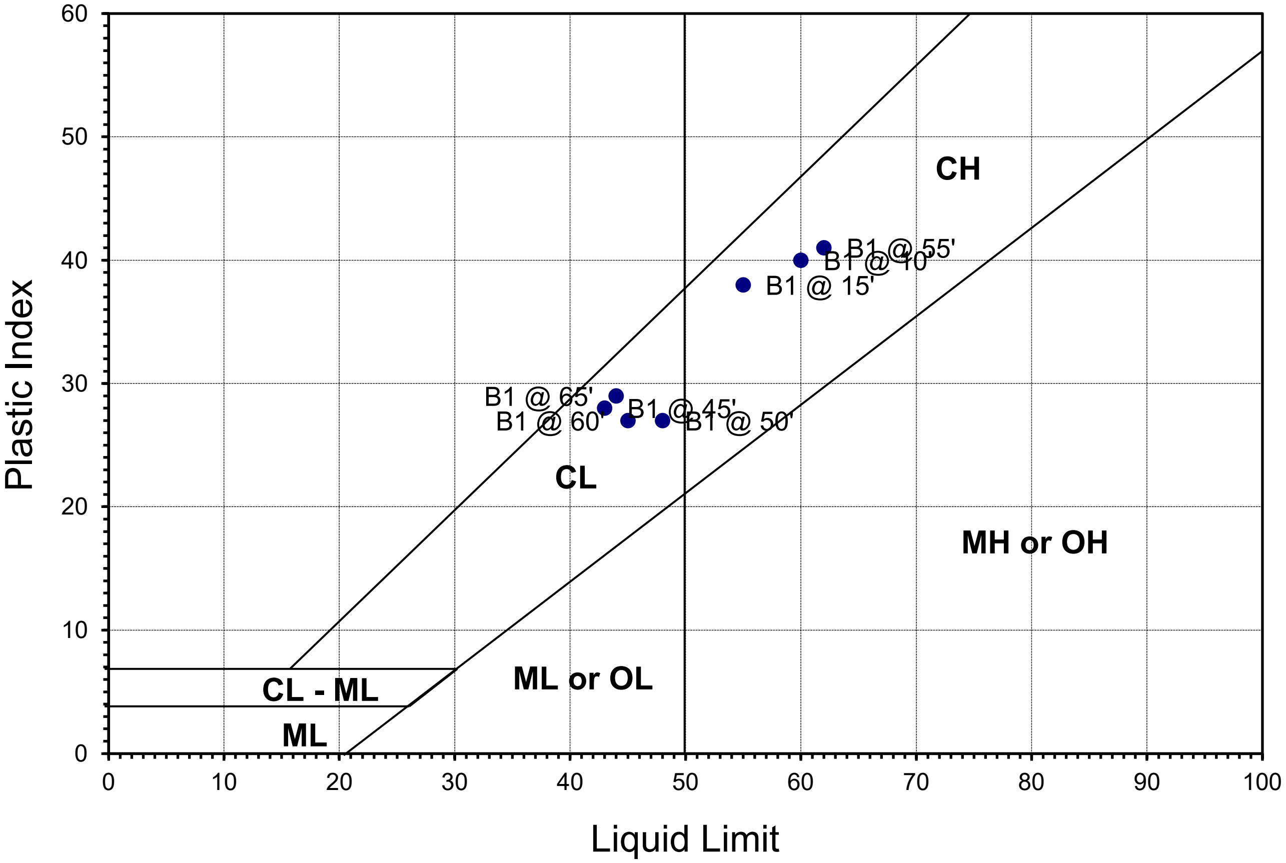
COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BARDAS INVESTMENT GROUP
6300 ROMAINE ST., LOS ANGELES

FILE NO. 22218

PLATE: D



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 10'	CH	88.8	60.0	20.0	40.0
B1 @ 15'	CH	86.7	55.0	17.0	38.0
B1 @ 20'	SC	37.5			
B1 @ 25'	SP	8.6			
B1 @ 30'	SM	26.8			
B1 @ 45'	CL	74.0	43.0	15.0	28.0
B1 @ 50'	CL	78.0	48.0	21.0	27.0
B1 @ 55'	CH	82.0	62.0	21.0	41.0
B1 @ 60'	CL	79.5	45.0	18.0	27.0
B1 @ 65'	CL	63.9	44.0	15.0	29.0

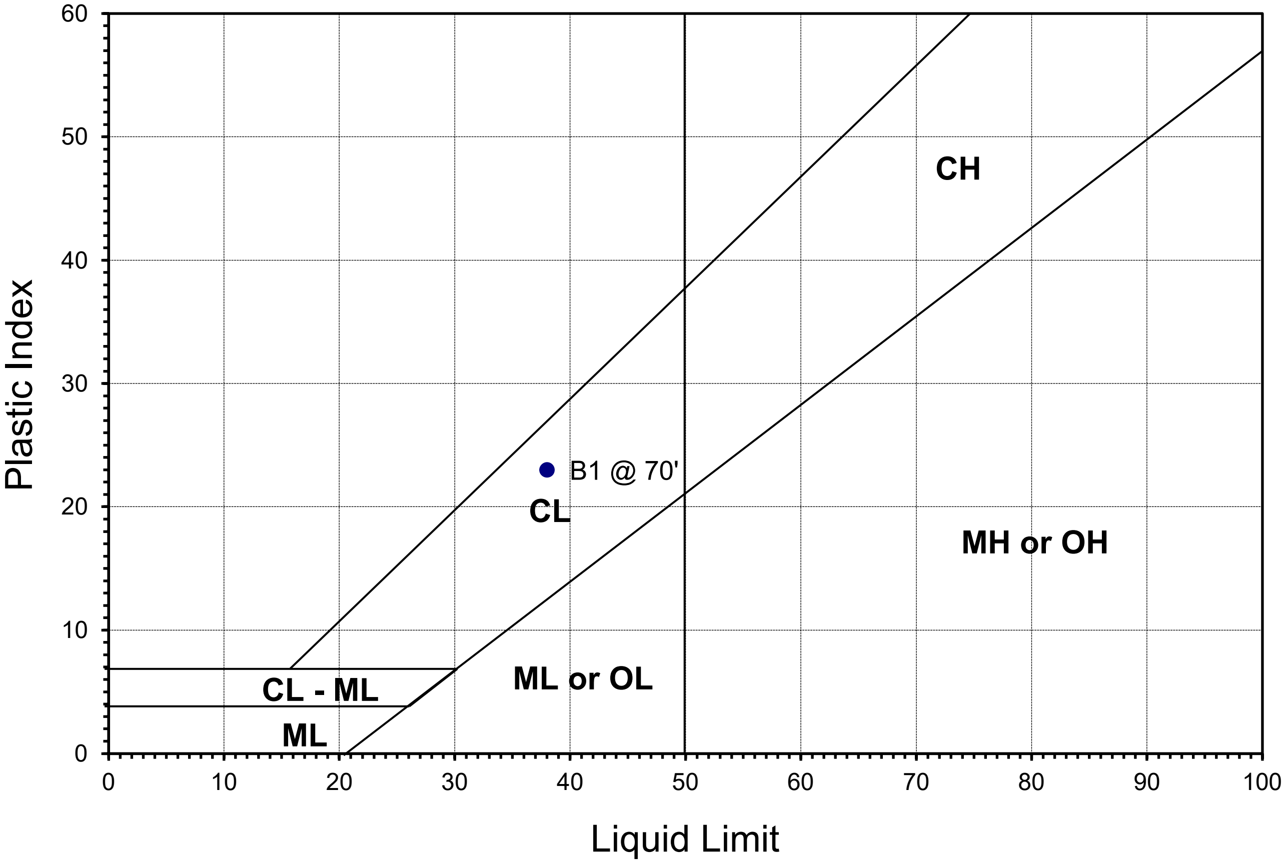
ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: BARDAS INVESTMENT GROUP

FILE NO. 22218

PLATE: F-1



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 70'	CL	66.4	38.0	15.0	23.0

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: BARDAS INVESTMENT GROUP

FILE NO. 22218

PLATE: F-2

April 26, 2021 via email: stang@geoteq.com

Geotechnologies, Inc.
439 Western Ave.
Glendale, CA, 91201

Attention: Mr. Stanley Tang

Re: Soil Corrosivity Study
Metro Plaza Project
Los Angeles, CA
HDR #21-0295SCS, GI #22111

Introduction

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

The proposed structure has six stories and four subterranean levels. The site is located at 6300 Romaine Street in Los Angeles, California, and the water table is reportedly 15 to 16 feet deep.

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

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per *ASTM International (ASTM)* G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and *American Water Works Association (AWWA*) Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory number 20-0295SCS. The full set of test results are shown in the attached Table 1.

Discussion

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil. A correlation between electrical resistivity and corrosivity toward ferrous metals is shown in Table 1.[1]

Table 1: Soil Corrosivity Categories.

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

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

Electrical resistivities were in the mildly corrosive to corrosive categories with as-received moisture. When saturated, the resistivities were in the corrosive to severely corrosive categories. The resistivity from B3 dropped considerably with added moisture because the sample was dry as-received.

Soil pH values varied from 7.3 to 7.6. This range is neutral to mildly alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low.

Per ACI-318, the soil is classified as S0 with respect to sulfate concentration.[3]

The nitrate concentration was high enough to be aggressive to copper. Ammonium was not detected.

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

In conclusion, this soil is classified as severely corrosive to ferrous metals, aggressive to copper, and negligible (S0) for sulfate attack on concrete.

---

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*
[3] *American Concrete Institute (ACI) 318-19 Table 19.3.1.1.*

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion. The following recommendations are based on the evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

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

    a. Dissimilar metals.

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

    c. Above ground steel pipe.

    d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

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

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

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

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

      v. Fusion bonded epoxy per AWWA C213.

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

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

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

      i. Polyethylene encasement per AWWA C105; *or*

      ii. Epoxy coating; *or*

      iii. Polyurethane; *or*

      iv. Wax tape.

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

   b. Apply cathodic protection to ductile iron piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

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

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe. Use the following parameters for clean sand backfill:

   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

   b. pH between 6.0 and 8.0.

   c. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]

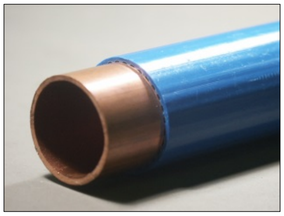
2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.

3. Electrically insulate cold water piping from hot water piping systems.

4. Protect buried copper tubing by one of the following measures:

    a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; or

    b. Install copper pipe with a factory-applied coating that is at least 25 mils in thickness. Use Kamco's Aqua Shield™, Mueller Streamline's Plumbshield™, or equal. The coating must be continuous with no cuts or defects.



    c. Insulate the pipe by installing 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately designed cathodic protection system per NACE SP0169.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

---

[4] *2016 California Plumbing Code (CPC), July 1, 2018 Supplement, Section 604.3.*
[5] *2018 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[6] *2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[7] *2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1*
[8] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

3. Due to the high ground water table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and ground water should be prevented as follows:

    a. For structures that extend below the water table, contact can be prevented with an impermeable waterproofing system. Options include a membrane such as Grace PrePrufe® products, a liquid applied barrier coating, or a waterproofing admixture such as Xypex® Admix. Visqueen, similar rolled barriers, or bentonite-based membranes are not viable waterproofing systems for corrosion protection.

    b. For structures above the water table, contact can be prevented with a gravel capillary break under the concrete and a vapor retarding membrane. Note that per ASTM E1643, "vapor retarders are not intended to provide a waterproofing function."[9] Alternatively, an impermeable waterproofing system may be used.

Post-Tensioned Slabs: Unbonded Single-Stranded Tendons and Anchors

Although chloride levels were relatively low, soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against corrosion by implementing all the following measures:[10,11,12]

1. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.
2. Design all tendons to prevent ingress of moisture. A corrosion-inhibiting coating should be incorporated into the tendon sheaths.
3. Use non-shrink grout mixes for all post-tensioning pockets.
4. Prior to grouting the pocket, apply a protective grease cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity. Ensure the cap fully seats against the face of the standard anchor at the live end.
5. Protect all components from moisture prior to installation and within one working day after installation.
6. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required by the applicable building code.
7. Install caps within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
8. Install pre-cast concrete plug over the grease cap to ensure the live end is sealed from further moisture intrusion.

---

[9] *ASTM E1643-11 (2017): Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. ASTM International, 2017.*
[10] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*
[11] *PTI M10.2-00: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*
[12] *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001*

9. Limit the access of direct runoff onto the anchorage area by designing proper drainage. Do not allow water to pond against anchors.
10. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.
11. Protect post-tensioned slabs from groundwater in accordance with the recommendations for concrete structures and pipe in this report.

Hydraulic Elevators

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

   OPTION 1

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

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

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

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

   OPTION 2

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

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

   OPTION 1

   a. Provide a bonded dielectric coating,

   b. Electrically isolate the pipeline, and

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

   OPTION 2

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

Closure

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

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

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
MARC E. N. WEGNER
No. C73862
Exp. 06/30/21
CIVIL
STATE OF CALIFORNIA

James T. Keegan, MD
Corrosion and Lab Services Section Manager

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

Enc: Table 1

21-0295SCS SCS Final.docx

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Metro Plaza Project
Your #22111, HDR Lab #21-0295SCS
9-Apr-21

Sample ID			B1 @ 2.5'	B2 @ 20'	B3 @ 40'
Resistivity		**Units**			
as-received		ohm-cm	1,160	2,120	19,200
saturated		ohm-cm	920	1,040	1,400
pH			7.3	7.6	7.6
Electrical Conductivity		mS/cm	0.10	0.10	0.04
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	53	36	34
magnesium	Mg^{2+}	mg/kg	20	9.2	4.8
sodium	Na^{1+}	mg/kg	22	33	17
potassium	K^{1+}	mg/kg	10	10	7.2
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	256	192	159
fluoride	F^{1-}	mg/kg	6.1	3.5	4.3
chloride	Cl^{1-}	mg/kg	20	17	11
sulfate	SO_4^{2-}	mg/kg	65	66	26
nitrate	NO_3^{1-}	mg/kg	3.4	71	4.4
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND
Other Tests					
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Geotechnologies, Inc.

Project: BARDAS INVESTMENT GROUP
File No.: 22218
Description: Liquefaction Analysis
Boring Numbe 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.7
Peak Ground Horizontal Acceleration, PGA (g):	0.98
Calculated Mag.Wtg.Factor:	1.234

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	15.5
Historically Highest Groundwater Level* (ft):	15.0
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:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	124.3	Unsaturated	Unsaturated	11	5	0.0	0	124.3	124.3	24.2	1.00	0.640	0.370	Non-Liq.	0.00
2	124.3	Unsaturated	Unsaturated	11	5	0.0	0	248.6	248.6	24.2	1.00	0.638	0.370	Non-Liq.	0.00
3	124.3	Unsaturated	Unsaturated	11	5	0.0	0	372.9	372.9	24.2	1.00	0.635	0.370	Non-Liq.	0.00
4	124.3	Unsaturated	Unsaturated	11	5	0.0	0	497.2	497.2	24.2	0.99	0.633	0.370	Non-Liq.	0.00
5	124.3	Unsaturated	Unsaturated	11	5	0.0	0	621.5	621.5	26.1	0.99	0.631	0.434	Non-Liq.	0.00
6	124.3	Unsaturated	Unsaturated	11	5	0.0	0	745.8	745.8	25.4	0.99	0.628	0.407	Non-Liq.	0.00
7	124.3	Unsaturated	Unsaturated	11	5	0.0	0	870.1	870.1	23.4	0.98	0.626	0.347	Non-Liq.	0.00
8	121.7	Unsaturated	Unsaturated	11	5	0.0	0	991.8	991.8	21.7	0.98	0.623	0.310	Non-Liq.	0.00
9	121.7	Unsaturated	Unsaturated	11	5	0.0	0	1113.5	1113.5	21.7	0.97	0.620	0.307	Non-Liq.	0.00
10	121.7	Unsaturated	Unsaturated	11	5	0.0	0	1235.2	1235.2	20.5	0.97	0.617	0.280	Non-Liq.	0.00
11	121.7	Unsaturated	Unsaturated	13	10	88.8	40	1356.9	1356.9	28.7	0.96	0.614	0.556	Non-Liq.	0.00
12	121.7	Unsaturated	Unsaturated	13	10	88.8	40	1478.6	1478.6	27.7	0.96	0.611	0.489	Non-Liq.	0.00
13	119.6	Unsaturated	Unsaturated	13	10	88.8	40	1598.2	1598.2	26.8	0.96	0.608	0.441	Non-Liq.	0.00
14	119.6	Unsaturated	Unsaturated	13	10	88.8	40	1717.8	1717.8	26.0	0.95	0.605	0.404	Non-Liq.	0.00
15	119.6	Unsaturated	Unsaturated	13	10	88.8	40	1837.4	1837.4	27.9	0.95	0.602	0.483	Non-Liq.	0.00
16	119.6	Saturated	Saturated	10	15	86.7	38	1957.0	1894.6	22.0	0.94	0.619	0.292	Non-Liq.	0.00
17	119.6	Saturated	Saturated	10	15	86.7	38	2076.6	1951.8	21.7	0.93	0.634	0.286	Non-Liq.	0.00
18	116.1	Saturated	Saturated	10	15	86.7	38	2192.7	2005.5	21.5	0.93	0.647	0.280	Non-Liq.	0.00
19	116.1	Saturated	Saturated	10	15	86.7	38	2308.8	2059.2	21.3	0.92	0.660	0.275	Non-Liq.	0.00
20	116.1	Saturated	Saturated	10	15	86.7	38	2424.9	2112.9	21.0	0.92	0.672	0.270	Non-Liq.	0.00
21	116.1	Saturated	Saturated	19	20	37.5	0	2541.0	2166.6	37.8	0.91	0.682	2.000	2.9	0.00
22	116.1	Saturated	Saturated	19	20	37.5	0	2657.1	2220.3	37.5	0.91	0.692	2.000	2.9	0.00
23	127.1	Saturated	Saturated	19	20	37.5	0	2784.2	2285.0	37.1	0.90	0.700	2.000	2.9	0.00
24	127.1	Saturated	Saturated	19	20	37.5	0	2911.3	2349.7	36.7	0.90	0.707	1.952	2.8	0.00
25	127.1	Saturated	Saturated	19	20	37.5	0	3038.4	2414.4	36.4	0.89	0.714	1.774	2.5	0.00
26	127.1	Saturated	Saturated	22	25	8.6	0	3165.5	2479.1	37.2	0.88	0.719	2.000	2.8	0.00
27	127.1	Saturated	Saturated	22	25	8.6	0	3292.6	2543.8	36.8	0.88	0.724	1.941	2.7	0.00
28	121.6	Saturated	Saturated	22	25	8.6	0	3414.2	2603.0	38.9	0.87	0.729	2.000	2.7	0.00
29	121.6	Saturated	Saturated	22	25	8.6	0	3535.8	2662.2	38.6	0.87	0.733	2.000	2.7	0.00
30	121.6	Saturated	Saturated	22	25	8.6	0	3657.4	2721.4	38.2	0.86	0.736	2.000	2.7	0.00
31	121.6	Saturated	Saturated	19	30	26.8	0	3779.0	2780.6	36.1	0.85	0.739	1.603	2.2	0.00
32	121.6	Saturated	Saturated	19	30	26.8	0	3900.6	2839.8	35.8	0.85	0.742	1.497	2.0	0.00
33	127.2	Saturated	Saturated	19	30	26.8	0	4027.8	2904.6	35.5	0.84	0.744	1.395	1.9	0.00
34	127.2	Saturated	Saturated	19	30	26.8	0	4155.0	2969.4	35.2	0.84	0.745	1.306	1.8	0.00
35	127.2	Saturated	Saturated	19	30	26.8	0	4282.2	3034.2	35.0	0.83	0.746	1.226	1.6	0.00
36	127.2	Saturated	Saturated	38	35	0.0	0	4409.4	3099.0	64.2	0.82	0.746	2.000	2.7	0.00
37	127.2	Saturated	Saturated	38	35	0.0	0	4536.6	3163.8	63.8	0.82	0.746	2.000	2.7	0.00
38	133.4	Saturated	Saturated	38	35	0.0	0	4670.0	3234.8	63.5	0.81	0.746	2.000	2.7	0.00
39	133.4	Saturated	Saturated	38	35	0.0	0	4803.4	3305.8	63.1	0.80	0.745	2.000	2.7	0.00
40	133.4	Saturated	Saturated	38	35	0.0	0	4936.8	3376.8	62.7	0.80	0.744	2.000	2.7	0.00
41	133.4	Saturated	Saturated	34	40	0.0	0	5070.2	3447.8	55.8	0.79	0.742	2.000	2.7	0.00
42	133.4	Saturated	Saturated	34	40	0.0	0	5203.6	3518.8	55.5	0.79	0.741	2.000	2.7	0.00
43	134.4	Saturated	Saturated	34	40	0.0	0	5338.0	3590.8	55.2	0.78	0.739	2.000	2.7	0.00
44	134.4	Saturated	Saturated	34	40	0.0	0	5472.4	3662.8	55.0	0.77	0.737	2.000	2.7	0.00
45	134.4	Saturated	Saturated	34	40	0.0	0	5606.8	3734.8	54.7	0.77	0.734	2.000	2.7	0.00
46	134.4	Saturated	Saturated	24	45	74.7	29	5741.2	3806.8	42.8	0.76	0.732	2.000	Non-Liq.	0.00
47	134.4	Saturated	Saturated	24	45	74.7	29	5875.6	3878.8	42.6	0.76	0.729	2.000	Non-Liq.	0.00
48	129.3	Saturated	Saturated	24	45	74.7	29	6004.9	3945.7	42.3	0.75	0.727	2.000	Non-Liq.	0.00
49	129.3	Saturated	Saturated	24	45	74.7	29	6134.2	4012.6	42.1	0.74	0.724	2.000	Non-Liq.	0.00
50	129.3	Saturated	Saturated	24	45	74.7	29	6263.5	4079.5	41.9	0.74	0.721	1.988	Non-Liq.	0.00
51	129.3	Saturated	Saturated	19	50	78.0	27	6392.8	4146.4	31.6	0.73	0.719	0.634	Non-Liq.	0.00
52	129.3	Saturated	Saturated	19	50	78.0	27	6522.1	4213.3	31.4	0.73	0.716	0.616	Non-Liq.	0.00
53	117.6	Saturated	Saturated	19	50	78.0	27	6639.7	4268.5	31.2	0.72	0.713	0.601	Non-Liq.	0.00
54	117.6	Saturated	Saturated	19	50	78.0	27	6757.3	4323.7	31.1	0.71	0.711	0.588	Non-Liq.	0.00
55	117.6	Saturated	Saturated	19	50	78.0	27	6874.9	4378.9	31.0	0.71	0.709	0.576	Non-Liq.	0.00
56	117.6	Saturated	Saturated	23	55	82.0	41	6992.5	4434.1	38.6	0.70	0.706	1.928	Non-Liq.	0.00
57	117.6	Saturated	Saturated	23	55	82.0	41	7110.1	4489.3	38.5	0.70	0.704	1.919	Non-Liq.	0.00
58	127.9	Saturated	Saturated	23	55	82.0	41	7238.0	4554.8	38.3	0.69	0.700	1.908	Non-Liq.	0.00
59	127.9	Saturated	Saturated	23	55	82.0	41	7365.9	4620.3	38.1	0.69	0.697	1.898	Non-Liq.	0.00
60	127.9	Saturated	Saturated	23	55	82.0	41	7493.8	4685.8	37.9	0.68	0.694	1.887	Non-Liq.	0.00
61	127.9	Saturated	Saturated	20	60	79.5	27	7621.7	4751.3	31.9	0.68	0.690	0.642	Non-Liq.	0.00
62	127.9	Saturated	Saturated	20	60	79.5	27	7749.6	4816.8	31.7	0.67	0.687	0.626	Non-Liq.	0.00
63	130.3	Saturated	Saturated	20	60	79.5	27	7879.9	4884.7	31.6	0.67	0.683	0.610	Non-Liq.	0.00
64	130.3	Saturated	Saturated	20	60	79.5	27	8010.2	4952.6	31.4	0.66	0.680	0.595	Non-Liq.	0.00
65	130.3	Saturated	Saturated	20	60	79.5	27	8140.5	5020.5	31.3	0.65	0.676	0.581	Non-Liq.	0.00
66	130.3	Saturated	Saturated	23	65	63.9	29	8270.8	5088.4	37.0	0.65	0.673	1.582	Non-Liq.	0.00
67	130.3	Saturated	Saturated	23	65	63.9	29	8401.1	5156.3	36.8	0.64	0.669	1.518	Non-Liq.	0.00
68	128.6	Saturated	Saturated	23	65	63.9	29	8529.7	5222.5	36.6	0.64	0.666	1.459	Non-Liq.	0.00
69	128.6	Saturated	Saturated	23	65	63.9	29	8658.3	5288.7	36.5	0.64	0.663	1.405	Non-Liq.	0.00
70	128.6	Saturated	Saturated	23	65	63.9	29	8786.9	5354.9	36.3	0.63	0.659	1.354	Non-Liq.	0.00
71	128.6	Saturated	Saturated	24	70	66.4	23	8915.5	5421.1	38.2	0.63	0.656	1.781	Non-Liq.	0.00
72	128.6	Saturated	Saturated	24	70	66.4	23	9044.1	5487.3	38.1	0.62	0.653	1.772	Non-Liq.	0.00
73	122.7	Saturated	Saturated	24	70	66.4	23	9166.8	5547.6	37.9	0.62	0.650	1.764	Non-Liq.	0.00
74	122.7	Saturated	Saturated	24	70	66.4	23	9289.5	5607.9	37.8	0.61	0.647	1.757	Non-Liq.	0.00
75	122.7	Saturated	Saturated	24	70	66.4	23	9412.2	5668.2	37.7	0.61	0.644	1.749	Non-Liq.	0.00
76	122.7	Saturated	Saturated	31	75	0.0	0	9534.9	5728.5	44.5	0.60	0.641	1.741	2.7	0.00
77	122.7	Saturated	Saturated	31	75	0.0	0	9657.6	5788.8	44.4	0.60	0.638	1.733	2.7	0.00
78	136.6	Saturated	Saturated	31	75	0.0	0	9794.2	5863.0	44.3	0.60	0.635	1.724	2.7	0.00
79	136.6	Saturated	Saturated	31	75	0.0	0	9930.8	5937.2	44.1	0.59	0.631	1.715	2.7	0.00
80	136.6	Saturated	Saturated	31	75	0.0	0	10067.4	6011.4	44.0	0.59	0.628	1.706	2.7	0.00
81	136.6	Saturated	Saturated	37	80	0.0	0	10204.0	6085.6	52.3	0.59	0.625	1.697	2.7	0.00
82	136.6	Saturated	Saturated	37	80	0.0	0	10340.6	6159.8	52.2	0.58	0.622	1.688	2.7	0.00
83	124.4	Saturated	Saturated	37	80	0.0	0	10465.0	6221.8	52.0	0.58	0.619	1.681	2.7	0.00
84	124.4	Saturated	Saturated	37	80	0.0	0	10589.4	6283.8	51.9	0.57	0.617	1.674	2.7	0.00
85	124.4	Saturated	Saturated	37	80	0.0	0	10713.8	6345.8	51.7	0.57	0.614	1.667	2.7	0.00
86	124.4	Saturated	Saturated	38	85	0.0	0	10838.2	6407.8	53.0	0.57	0.612	1.659	2.7	0.00
87	124.4	Saturated	Saturated	38	85	0.0	0	10962.6	6469.8	52.9	0.56	0.610	1.652	2.7	0.00
88	125.6	Saturated	Saturated	38	85	0.0	0	11088.2	6533.0	52.7	0.56	0.608	1.645	2.7	0.00
89	125.6	Saturated	Saturated	38	85	0.0	0	11213.8	6596.2	52.6	0.56	0.605	1.638	2.7	0.00
90	125.6	Saturated	Saturated	38	85	0.0	0	11339.4	6659.4	52.5	0.56	0.603	1.631	2.7	0.00
91	125.6	Saturated	Saturated	33	90	0.0	0	11465.0	6722.6	45.5	0.55	0.601	1.624	2.7	0.00
92	125.6	Saturated	Saturated	33	90	0.0	0	11590.6	6785.8	45.3	0.55	0.599	1.618	2.7	0.00
93	124.8	Saturated	Saturated	33	90	0.0	0	11715.4	6848.2	45.2	0.55	0.597	1.611	2.7	0.00
94	124.8	Saturated	Saturated	33	90	0.0	0	11840.2	6910.6	45.1	0.55	0.596	1.604	2.7	0.00
95	124.8	Saturated	Saturated	33	90	0.0	0	11965.0	6973.0	45.0	0.54	0.594	1.598	2.7	0.00
96	124.8	Saturated	Saturated	46	95	0.0	0	12089.8	7035.4	62.6	0.54	0.592	1.591	2.7	0.00
97	124.8	Saturated	Saturated	46	95	0.0	0	12214.6	7097.8	62.5	0.54	0.591	1.585	2.7	0.00
98	131.0	Saturated	Saturated	46	95	0.0	0	12345.6	7166.4	62.3	0.54	0.589	1.578	2.7	0.00
99	131.0	Saturated	Saturated	46	95	0.0	0	12476.6	7235.0	62.2	0.54	0.588	1.571	2.7	0.00
100	131.0	Saturated	Saturated	46	95	0.0	0	12607.6	7303.6	62.0	0.53	0.586	1.564	2.7	0.00

Total Liquefaction Settlement, S =	**0.00 inches**

APPENDIX IV

PRIOR REPORT

[redacted]

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVE. # 201, GLENDALE, CA [illegible]
818•549•9959 TEL 818•543•3747 FAX

TRANSMITTAL

May 13, 1999
JB 18051-B

Television Center, Inc.
6311 Romaine Street
Hollywood, California 90038

Attention: Ana Ramirez

Subject

Geotechnical Exploration
Proposed Parking Structure, Commercial Buildings, and Additions
6311 Romaine Street
Hollywood, California

Gentlepersons:

Transmitted herewith is our geotechnical engineering report with respect to construction of the parking structure, commercial buildings, and additions at Television Center in Hollywood. Our tests borings indicate that the site is underlain by a surface layer of fill placed during past development of the site. The fill is on the order of two to five feet thick. Below the fill are natural alluvial soils that are strong and capable of supporting the proposed development. Conventional spread footings are recommended for support of the structures. The property is not subject to liquefaction and is not within any special studies zone for active earthquake faults.

The reviewing agency for this document is the City of Los Angeles Building Department. They require three copies of the report along with an application form and a filing fee. City review is expected to take four weeks. Copies of the report have been distributed as follows:

(1) Television Center, Inc., Attention: Ana Ramirez
(6) Turkel Architecture Group, Attention: Steven Turkel

It is our understanding that your architect will file the report with the City of Los Angeles. It is suggested that you read the report carefully prior to submittal to any reviewing agency. Any questions concerning the data or interpretation of the report should directed to the undersigned. The J. Byer Group appreciates the opportunity to offer our consultation and advice on this project. An invoice for this work has been included with the report copy sent to Television Center, Inc.

Very Truly Yours,
THE J. BYER GROUP, INC.

John W. Byer
President

xc: (1) Addressee
(1) Steve Turkel

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVE. # 201, GLENDALE, CA 91206
818•549•9959 TEL 818•543•3747 FAX

GEOTECHNICAL EXPLORATION
PROPOSED PARKING GARAGE, COMMERCIAL STRUCTURES AND ADDITIONS
EXISTING TELEVISION CENTER
6311 ROMAINE STREET
HOLLYWOOD, CALIFORNIA
FOR TELEVISION CENTER, INC.
THE J. BYER GROUP, INC. PROJECT NUMBER JB 18051-B
MAY 13, 1999

GEOTECHNICAL EXPLORATION
PROPOSED PARKING GARAGE, COMMERCIAL STRUCTURES AND ADDITIONS
EXISTING TELEVISION CENTER
6311 ROMAINE STREET
HOLLYWOOD, CALIFORNIA
FOR TELEVISION CENTER, INC.
THE J. BYER GROUP, INC. PROJECT NUMBER JB 18051-B
MAY 13, 1999

INTRODUCTION

Per your authorization, The J. Byer Group has performed geotechnical exploration at the property. The following report summarizes findings of the exploration and provides recommendations for development of the site. The purpose of this study is to evaluate the nature, distribution, and engineering properties of the earth materials underlying the site with respect to construction of a six story parking garage, two story commercial buildings, and additions to the existing facility.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

EXPLORATION

The scope of the field exploration was determined following our initial site visit and consultation with Mr. Steve Turkel, Project Architect. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Site Plan. Conditions affecting portions of the property outside the area explored, are beyond the scope of this report.

Exploration was conducted on April 27 and 28, 1999 with the aid of a hollow stem auger drill rig. It included advancing 12 borings to depths ranging from 15 to 50 feet below the ground surface. Samples of the earth materials were obtained at frequent intervals and returned to our soils engineering laboratory for testing and analysis.

Office tasks included laboratory testing of selected soil samples, review of our files with respect to geotechnical explorations in the area, review of State of California Seismic Hazard Maps, engineering analysis, preparation of the enclosed Site Plan, and the preparation of this report. The earth materials encountered in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project and the locations of the borings are shown on the Site Plan.

PROPOSED DEVELOPMENT

Information concerning the proposed project was provided by Mr. Steve Turkel, Architect. It is proposed to construct two small additions to the existing facility. One at the corner of Cahuenga Boulevard and Santa Monica Boulevard, and one in the interior area. These additions are planned to be one or two story wood frame structures. In the southern portion of the property, it is proposed to construct a six story parking structure. To the north of that structure will be two story wood frame structures. Foundation loads for the parking structure are expected to be relatively high. Foundation loads for the small commercial buildings and additions are expected to be low. The proposed project is considered preliminary at this time and formal plans have not been prepared.

SITE DESCRIPTION

The subject property is located in the Hollywood area of the City of Los Angeles. It fronts on Santa Monica Boulevard to the north, Cahuenga Boulevard on the east, Willoughby Avenue to the south and Cole Avenue on the west. The property consists of two square blocks divided by Romaine Street. The northern block is developed with numerous buildings, including the original Technicolor reinforced concrete vault building. Other additions are one and two story, small production studios. To the south is a surface level asphalt parking lot, which covers the entire block. The parking lot is essentially level but is elevated two to three feet above the surrounding streets. The surface of the parking lot is in moderate to poor condition. Drainage on the property is generally by sheetflow runoff down the natural slope of the land to the fronting streets.

GROUNDWATER

Groundwater was encountered in Borings 5, 6, and 9. In Boring 5, groundwater was seeping into the boring a depth of 15 feet. This boring is adjacent to a broken water main in Cole Avenue, which was under repair at the time of this exploration. It is probable that leakage from the water line is the source of the water. A confined layer of groundwater was found in Boring 6 at a depth of 26½ feet below grade. The water rose to 21½ feet below grade one hour after completion of the boring. A similar confined layer was encountered in Boring 9 at a depth of 24 feet below grade. The water rose to 16 feet below grade one hour after completion of the boring. The groundwater levels discussed are confined to sandy layers within the natural soil and do not represent a permanent water table. As indicated by the borings, fluctuations in groundwater levels may also occur due to conditions not evident at the time of exploration. Fluctuations may also occur across the site.

EARTH MATERIALS

Fill

A thin layer of fill, associated with previous site development and demolition, covers the entire property. Two feet of fill was encountered in Borings 8 and 12 where the proposed additions are planned. Two to five feet of fill was found below the existing asphalt parking lot in the southern portion of the property. The fill consists of various blends of silty sands, sandy gravel, and clay and contains construction and demolition debris, including concrete and brick fragments.

Alluvium

Natural alluvial soils derived as outwash from the south flank of the eastern Santa Monica Mountains underlies the entire property. The alluvium consists of an upper layer of dark brown to black sandy clay that is moist and firm to stiff. The upper clay rich layer is two to three feet thick. Below the clay, the alluvium becomes sandier and grades to silty sand and clayey sand that are brown to reddish brown, moist and dense. A persistent silt layer is found between 10 and 15 feet below the surface grade. The silt is light gray brown, moist to very moist, and firm to stiff. The silt bed rests on coarser sands, silty sands, and gravelly sands beginning at a depth of 15 feet. These sandy layers then grade into clayey sand and silty sand with depth.

Bedrock

Bedrock was not encountered in the borings to the depths explored. Sedimentary bedrock typical of the eastern Santa Monica Mountains is exposed approximately one mile north of the property.

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Division of Mines and Geology, private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies are shifting their focus to earthquake resistant structures as opposed to prediction. The purpose of the code seismic design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Within the past 25 years, southern California and vicinity have experienced an increase in seismic activity beginning with the San Fernando earthquake in 1971. In 1987, a moderate earthquake struck the Whittier area and was located on a previously unknown fault. Ground shaking from this event caused substantial damage to the City of Whittier, and surrounding cities.

The January 17, 1994, Northridge earthquake was initiated along a previously unrecognized fault below the San Fernando Valley. The energy released by the earthquake propagated to the southeast, northwest, and northeast in the form of shear and compression waves, which caused the strong ground shaking in portions of the San Fernando Valley, Simi Valley, City of Santa Clarita, and City of Santa Monica.

Southern California faults are classified as: active, potentially active, or inactive. Faults from past geologic periods of mountain building, but do not display any evidence of recent offset, are considered "potentially active". Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults". There are no known active faults within close vicinity of the subject property.

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including strapping water heaters and securing furniture. It is likely that the subject property will be shaken by future earthquakes produced in southern California. However, secondary effects such as surface rupture, lurching, liquefaction, consolidation, ridge shattering, and landsliding should not occur at the subject property.

Liquefaction

The subject property is not located within a liquefaction zone as indicated on the Hollywood Quadrangle (Official Map) of the State of California Seismic Hazard Zones, released March 25, 1999. Liquefaction is a condition where soil experiences deformation at constant low residual stress or with a low residual resistance due to build up and maintenance of high pore water pressures. Liquefaction can occur in either a static or dynamic condition and the possibility of occurrence depends upon the void ratio, relative density, and confining pressure of the soil. There are four general conditions necessary for liquefaction to occur. These include a high groundwater table, fine grained cohesionless soils, a low relative density, and strong ground shaking. The borings indicate that the subject property is underlain by dense alluvial soils that are not subject to liquefaction. The groundwater levels encountered are perched and confined layers that are locally discontinuous. It is the opinion of The J. Byer Group that the subject property is not subject to liquefaction.

SITE SEISMIC CONSIDERATIONS

Southern California is placed in Seismic Zone IV per the Uniform Building Code. The nearest mapped fault is the Hollywood, located approximately one mile north of the project. The Hollywood fault is not currently zoned as an active fault as determined by the Alquist-Priolo

Special Studies Zone Maps. The Hollywood fault is currently being studied by local universities and recent data may indicate the fault is active. The Hollywood fault has no history of seismic activity within the past 200 years. The nearest capable fault is considered the Newport-Inglewood fault, which crosses through the Baldwin Hills approximately five miles south of the property. The Newport-Inglewood fault can produce earthquakes in the moment magnitude range of 6.0 to 7.0.

The alluvial soils on the site are characterized as S_2 stiff soil per Table 16-J of 1994 Uniform Building Code. The alluvial soils are considered very dense soil S_c per Table 16-J of the 1997 Uniform Building Code. The nearest source factors N_a and N_v are 1.3 and 1.6, respectively for a type "B" fault (Hollywood) less than two kilometers north of the site.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon 12 borings, research of records, analysis of laboratory test data, consultation with your architect, and over 30 years experience providing geotechnical explorations on similar properties on similar settings. It is the finding of The J. Byer Group, Inc. that construction of the proposed project is feasible from a geotechnical engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is the natural alluvial soil found at depths ranging between two and five feet below the existing surface grade. Removal and recompaction of the surface fill may be required in the area of the additions depending upon the type of construction. For slab-on-grade, the fill inside of the building line should be removed to the natural grade and replaced as approved compacted fill. The proposed parking garage area may also require remedial grading for slab support depending upon the final floor elevation. The proposed lease areas may require removal and recompaction of all of the fill in order to provide foundation support. The following section provides general grading recommendations with respect to site preparation.

SITE PREPARATION

Surficial materials consisting of fill are present on the site. Remedial grading may be necessary to improve site conditions for foundation and floor slab support.

General Grading Specifications

The following guidelines may be used in preparation of the grading plan and job specifications. The J. Byer Group would appreciate the opportunity of reviewing the plans to insure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The site to receive compacted fill should be prepared by removing all existing fill. The exposed excavated area should be observed by the soils engineer prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum density.

B. If the compacted fill is to be utilized for foundation support the proposed building site shall be excavated to a minimum depth of three feet below the bottom of all footings or the depth of fill, whichever is deeper. The excavation should extend five feet beyond the building footprint as shown on the Site Plan. The excavated areas should be observed by the soils engineer prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts and compacted in six inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. The maximum density shall be determined by ASTM D 1557-91 or equivalent.

E. Field observation and testing shall be performed by the soils 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 90 percent compaction is obtained. One compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

F. It is estimated that the fill, when removed and replaced as compacted fill, will shrink in volume approximately five to ten percent. Imported soil may be necessary to complete the grading and achieve the finished grades.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads.

Spread Footings

Continuous and pad footings may be used to support the proposed additions, commercial buildings, and parking structure provided they are founded in the natural alluvial soil or approved compacted fill. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24 inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Alluvium	18	2,500	0.3	250	5,000
Future Compacted Fill	18	2,000	0.3	200	4,000

Increases in the bearing value are allowable at a rate of 20 percent for each additional foot of footing width or depth to a maximum of 4,000 pounds per square foot for compacted fill and 5,000 pounds per square foot for natural alluvium. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing values shown 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. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

All continuous footings should be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geotechnical engineer prior to placing forms, steel or concrete.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated. Differential settlement should not exceed ¼ inch.

RETAINING WALLS

General Design

Currently, retaining walls are not proposed for the project. However, should retaining walls be used the following design recommendations should be incorporated into the plans. Retaining walls up to 10 feet high, and with a level backslope may be designed for a minimum equivalent fluid pressure of 43 pounds per cubic foot. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾ inch crushed gravel.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾ inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted fill blanket to the surface. Retaining wall backfill should be capped with a paved surface drain.

Foundation Design

Retaining wall footings may be sized per the"Spread Footings" section of this report.

FLOOR SLABS

For slab-on-grade construction, the existing fill should be removed to expose the natural alluvium, and replaced as approved compacted fill. In the City of Los Angeles, slab-on-grade construction supported by approved compacted fill requires a minimum reinforcement of #4 steel bars at 16 inches on center, each way. Slabs which will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be covered with a thin layer of sand, about one inch, to prevent punctures and aid in the concrete cure.

PAVING

Prior to placing paving, the existing fill should be removed to the natural alluvial soil, and replaced as approved compacted fill, in accordance with the Site Preparation section of this report. Any trench backfill below paving, should be compacted to 90 percent of the laboratory maximum density. Irrigation water should be prevented from migrating below paving. For light passenger cars the recommend paving section is three inches of asphalt over four inches of compacted base. For heavy use, including truck lanes and storage, the recommended paving section is four inches of asphalt over six inches of compacted base.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised floor type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾ inch crushed gravel to help the collection of water. Yard areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

Construction of raised floor buildings where the grade under the floor has been lowered for joist clearance can also lead to moisture problems. Surface moisture can seep through the footing and pond in the underfloor area. Positive drainage away from the footings, waterproofing the footings, compaction of trench backfill and subdrains can help to reduce moisture intrusion.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by The J. Byer Group. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The Building Department requires that the geotechnical company provide site observations during construction. The observations include bottoms prior to placing fill, compaction of fill, and foundation excavations. All fill that is placed should be tested for compaction and approved by the soils engineer prior to use for support of engineered structures. The City of Los Angeles requires that all retaining wall subdrains be observed by a representative of the geotechnical company and the City Inspector.

Please advise The J. Byer Group, Inc. at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the jobsite and available to our representative. The project consultant will perform the observation and post a notice at the jobsite of his visit and findings. This notice should be given to the agency inspector.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions and excavation characteristics described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications or recommendations during construction requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

The J. Byer Group appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
THE J. BYER GROUP, INC.

John W. Byer
President

Robert I. Zweigler
G.E. 2120

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 6-30-02
GEOTECHNICAL
STATE OF CALIFORNIA

JWB:RIZ:flh
G:\FINAL\REPORTS\18051-B.RPT

Enc: Appendix I - Laboratory Testing
Shear Test Diagrams (2)
Consolidation Curves (12)
Vicinity Map
Log of Borings (15 Pages)

In Pocket: Site Plan

xc: (1) Addressee
(6) Steve Turkel, Architect

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill and alluvium were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring lined barrel sampler conforming to ASTM D-3550 with successive drops of the hammer. Experience has shown that sampling causes some disturbance of the sample, however the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The central portions of the samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D-2937. The moisture content of the samples was determined using the procedures outlined in ASTM D-2216. The results are shown on the Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill was determined by remolding a bulk sample of the existing fill using the procedures outlined in ASTM D 1557, a five-layer standard. Remolded samples were prepared at 90 percent of the maximum dry density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
1	2	Brown Sandy Clay	129.0	14.8	48

Expansion Test

To find the expansiveness of the future compacted fill, an expansion index test was performed. Based upon the testing, the future compacted fill will be slightly to moderately expansive.

Shear-Tests

Shear tests were performed on samples of the remolded fill and natural alluvium using the procedures outlined in ASTM D-3080 and a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the "Shear Test Diagrams".

Consolidation

Consolidation tests were performed on insitu samples of the alluvium. Results are graphed on the "Consolidation Curves".

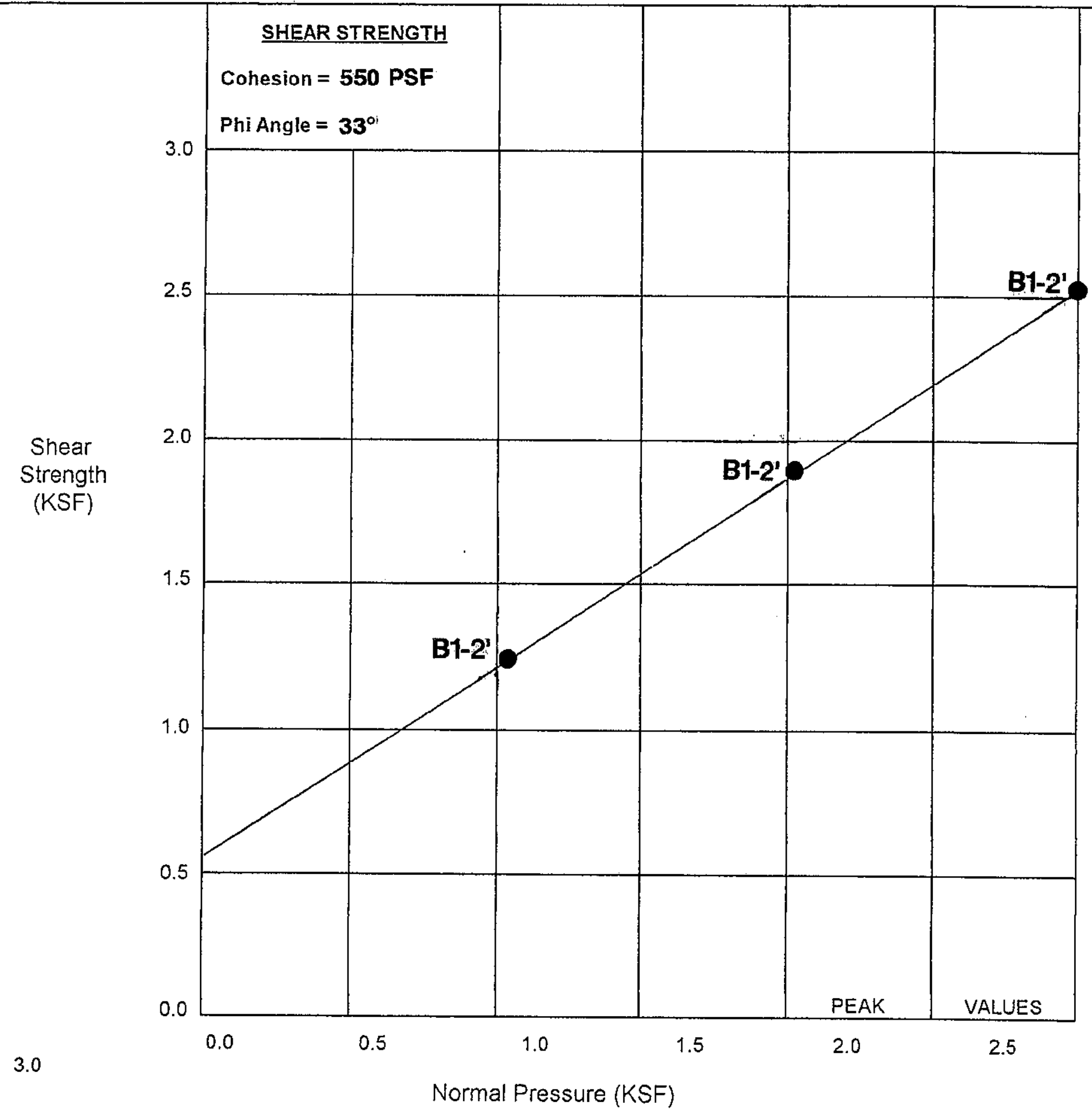
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

SHEAR TEST DIAGRAM

JB: 18051-B Television Center

SAMPLE: Future Compacted Fill

Samples remolded to 90% of maximum density



SHEAR STRENGTH

Cohesion = 550 PSF

Phi Angle = 33°

○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **19.5**

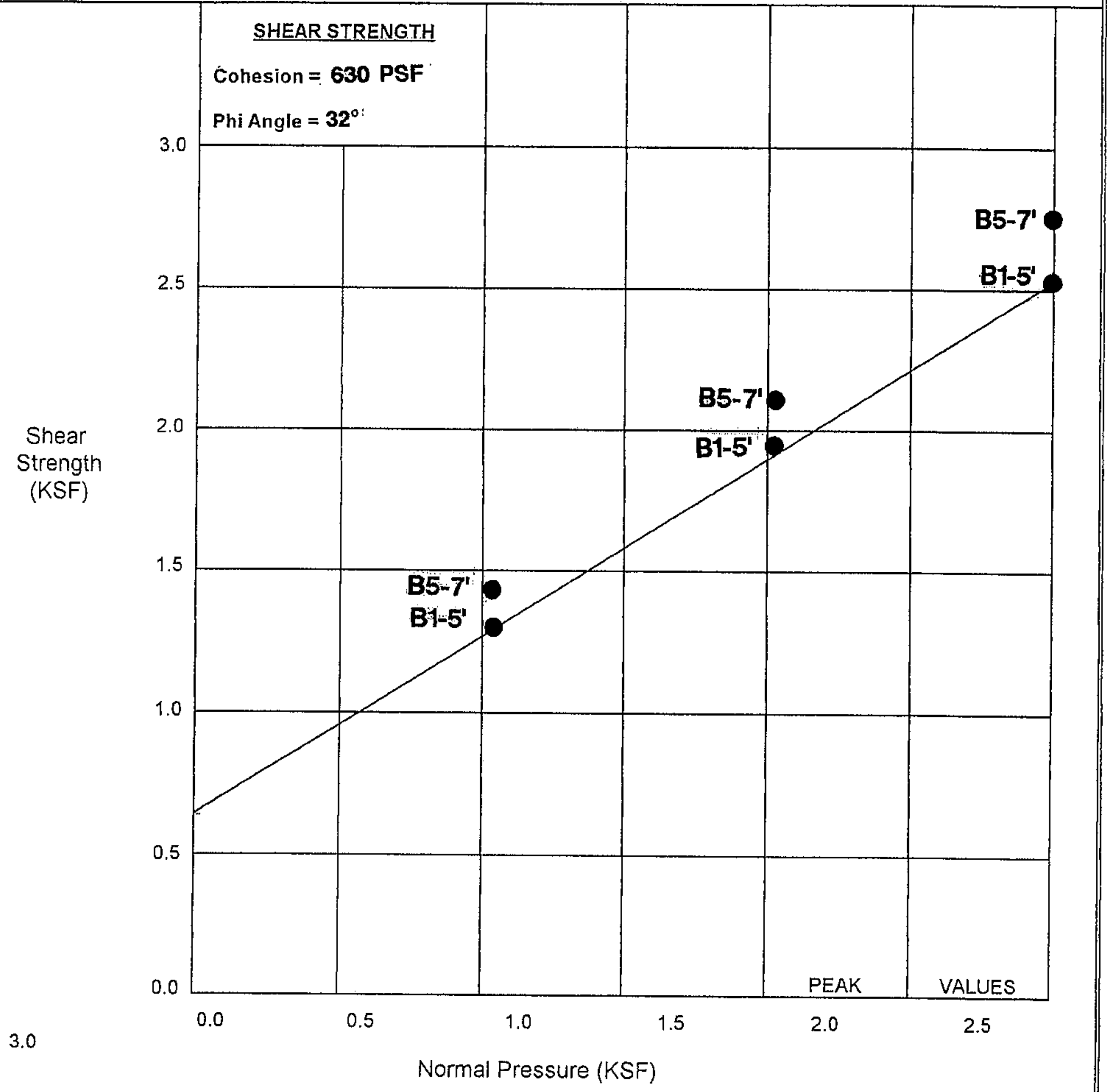
Dry Density (pcf) = **118.0**

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

SHEAR TEST DIAGRAM

JB: 18051-B Television Center

SAMPLE: Alluvium



SHEAR STRENGTH

Cohesion = **630 PSF**

Phi Angle = **32°**

○ Direct Shear (Field Moisture)

● Direct Shear (Saturated)

Moisture Content (%) = **18.6**

Dry Density (pcf) = **120.6**

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

CONSOLIDATION DIAGRAM

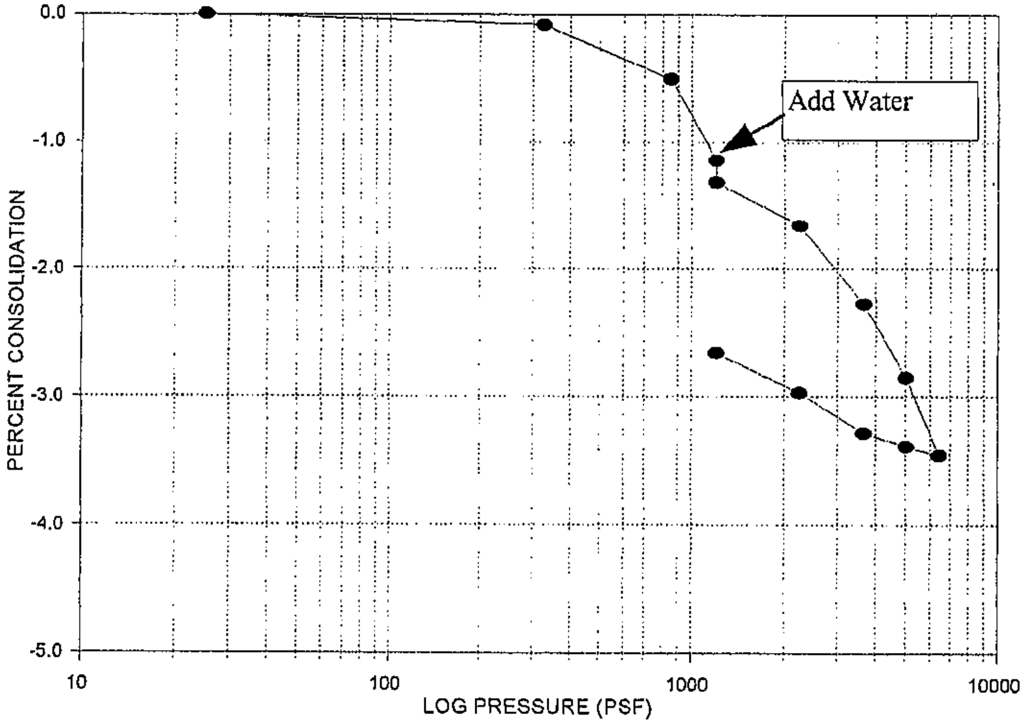
JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B2-5'

CONSOLIDATION DIAGRAM
PERCENT CONSOLIDATION



Dry Density	97.4 pcf	Specific Gravity	2.65
Initial Moisture	10.2%	Initial Void Ratio	0.70
Initial % Saturation	38.7%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

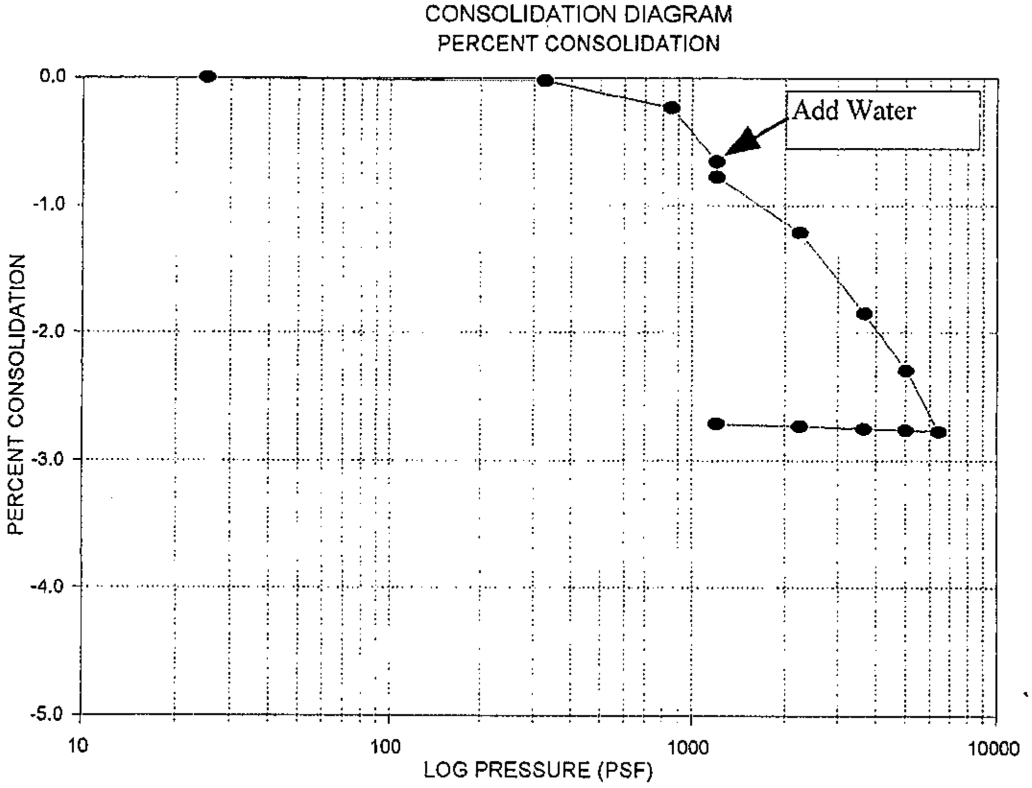
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B1-3'



Dry Density	115.4 pcf	Specific Gravity	2.65
Initial Moisture	15.7%	Initial Void Ratio	0.43
Initial % Saturation	96.1%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

CONSOLIDATION DIAGRAM

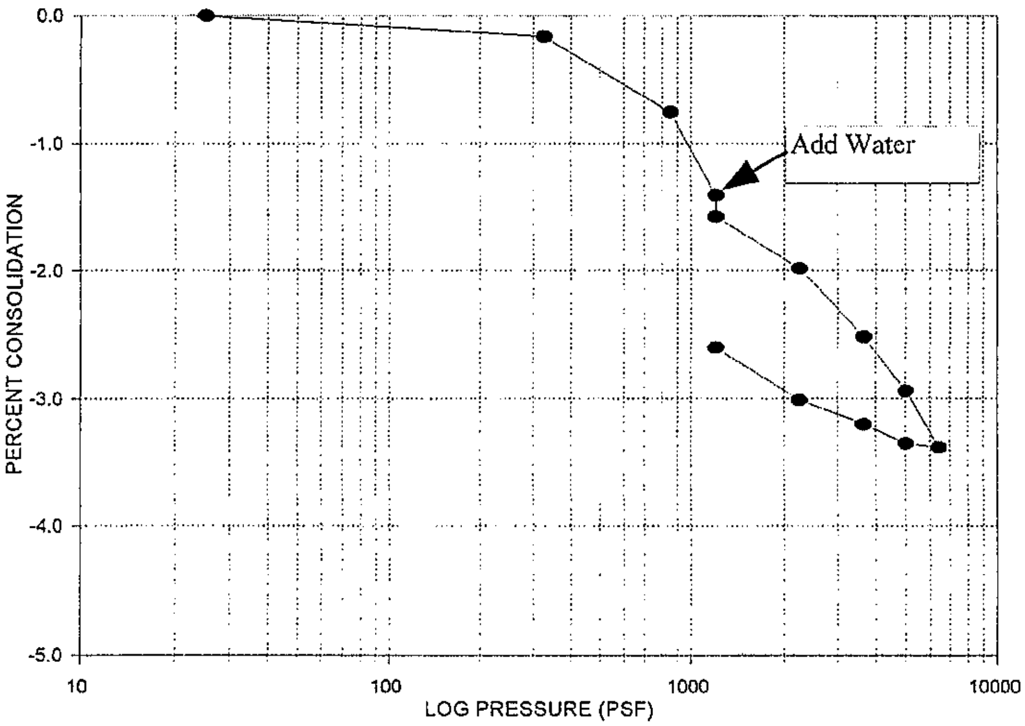
JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B4-7'

CONSOLIDATION DIAGRAM
PERCENT CONSOLIDATION



Dry Density	104.0 pcf	Specific Gravity	2.65
Initial Moisture	21.2%	Initial Void Ratio	0.59
Initial % Saturation	95.2%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

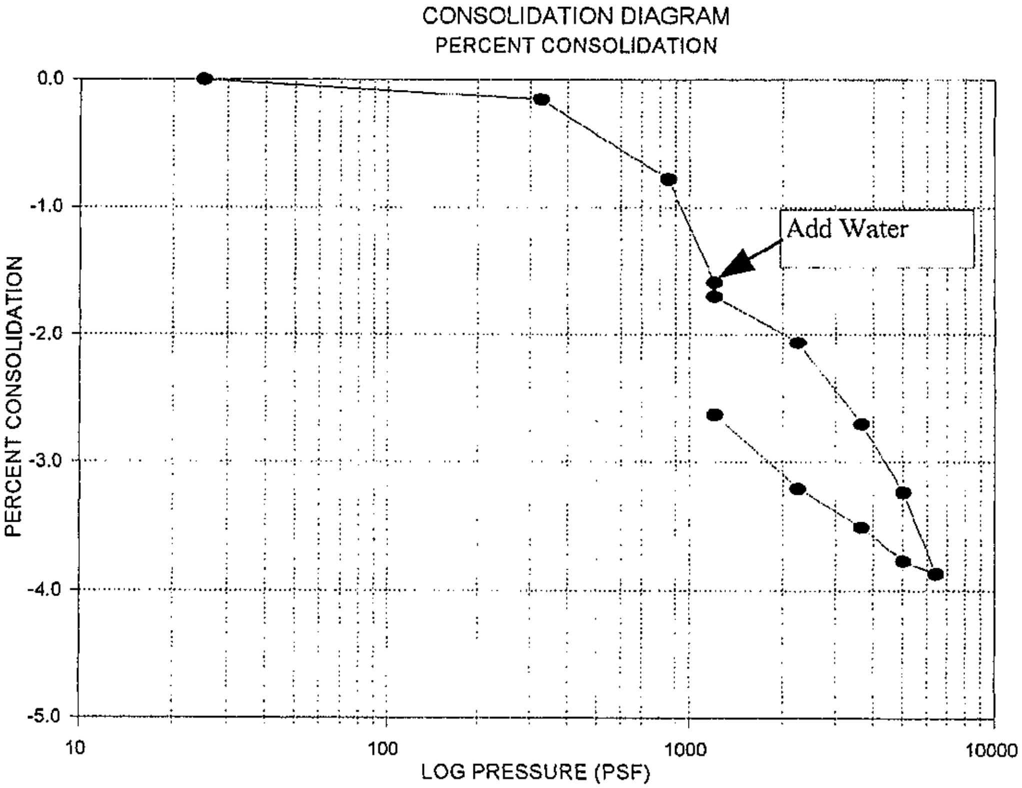
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B4-10'



Dry Density	97.4 pcf	Specific Gravity	2.65
Initial Moisture	27.0%	Initial Void Ratio	0.70
Initial % Saturation	102.5%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

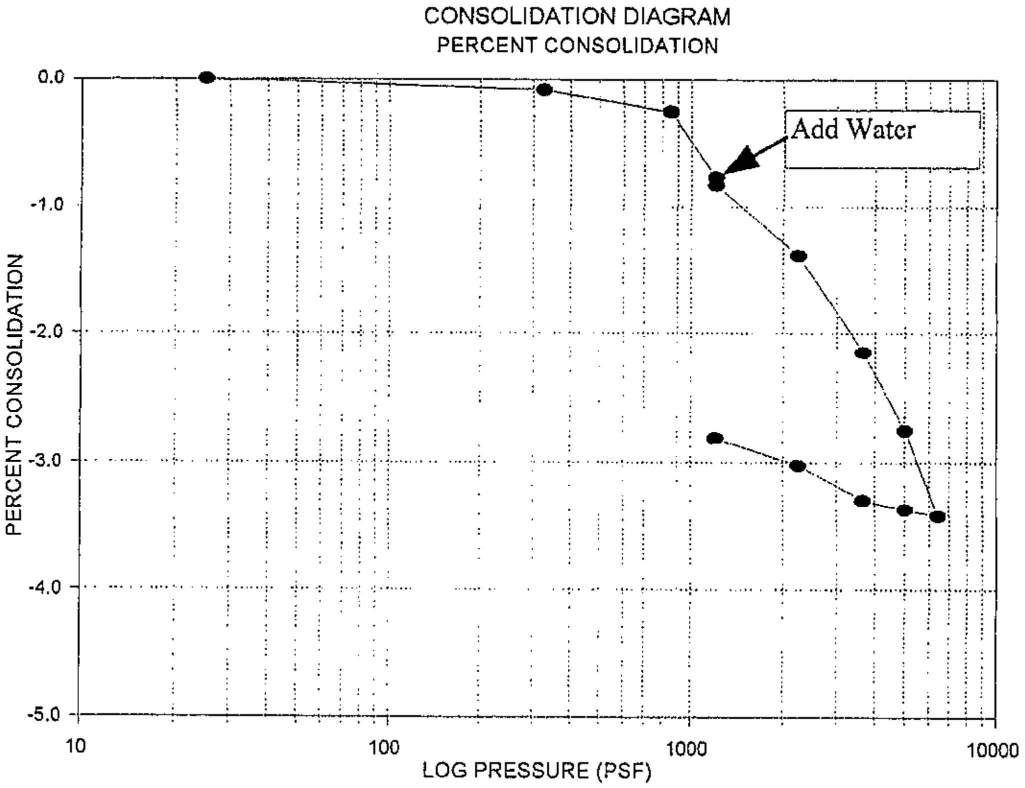
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B5-5'



Dry Density	112.4 pcf	Specific Gravity	2.65
Initial Moisture	14.9%	Initial Void Ratio	0.47
Initial % Saturation	83.8%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

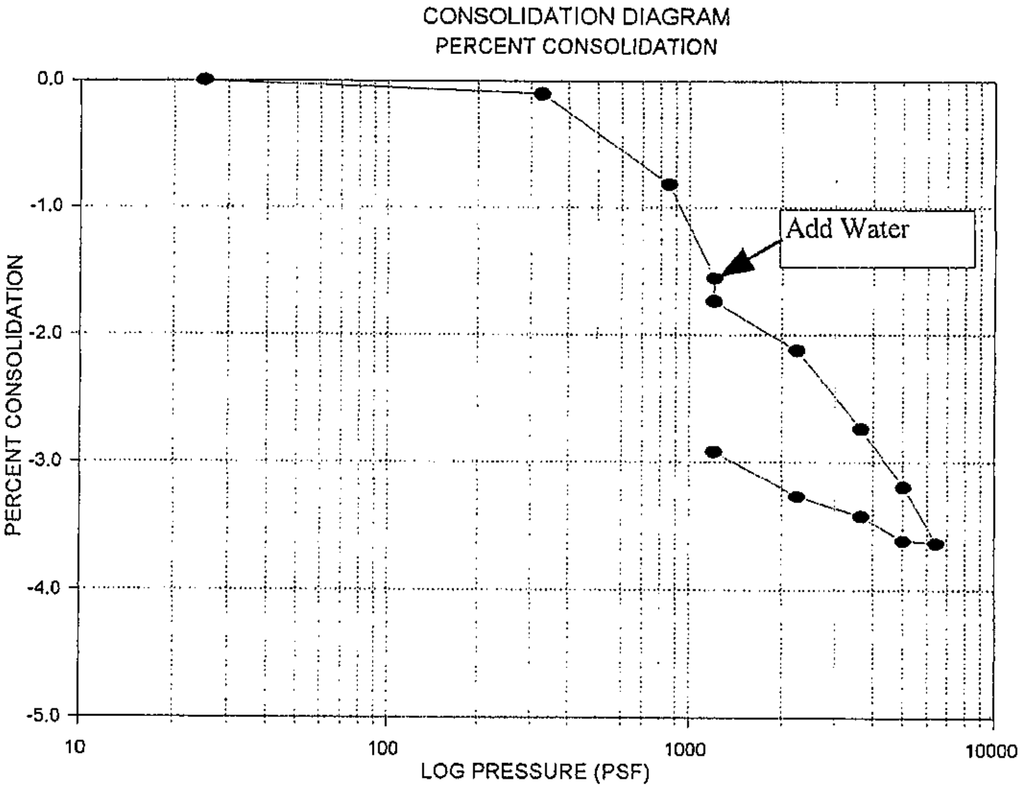
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B6-6'



Dry Density	109.2 pcf	Specific Gravity	2.65
Initial Moisture	19.4%	Initial Void Ratio	0.51
Initial % Saturation	100.0%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

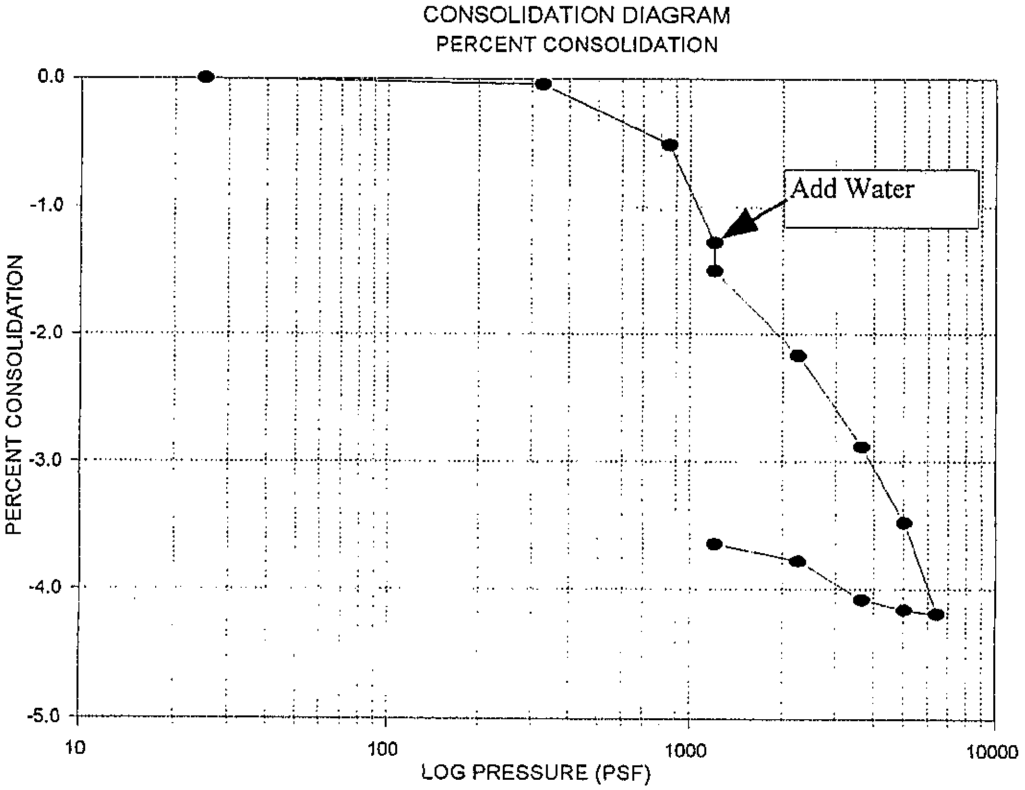
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B8-2'



Dry Density	105.2 pcf	Specific Gravity	2.65
Initial Moisture	20.3%	Initial Void Ratio	0.57
Initial % Saturation	94.1%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

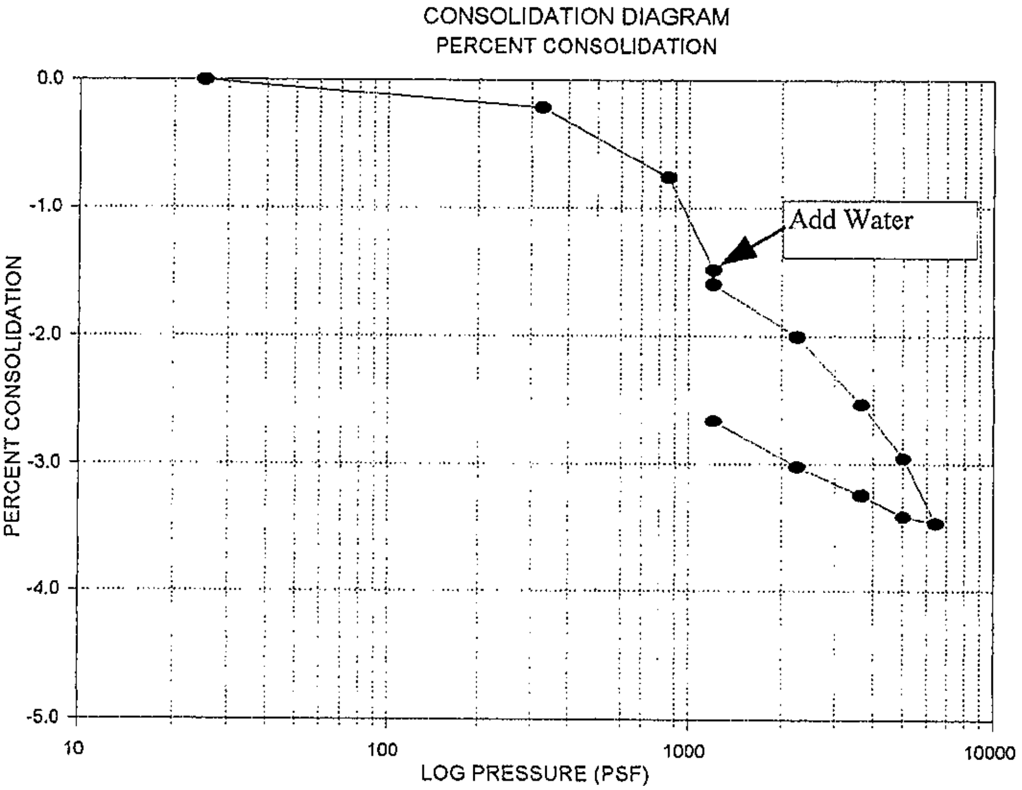
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B9-6'



Dry Density	104.7 pcf	Specific Gravity	2.65
Initial Moisture	21.1%	Initial Void Ratio	0.58
Initial % Saturation	96.5%	C'c	0.03

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

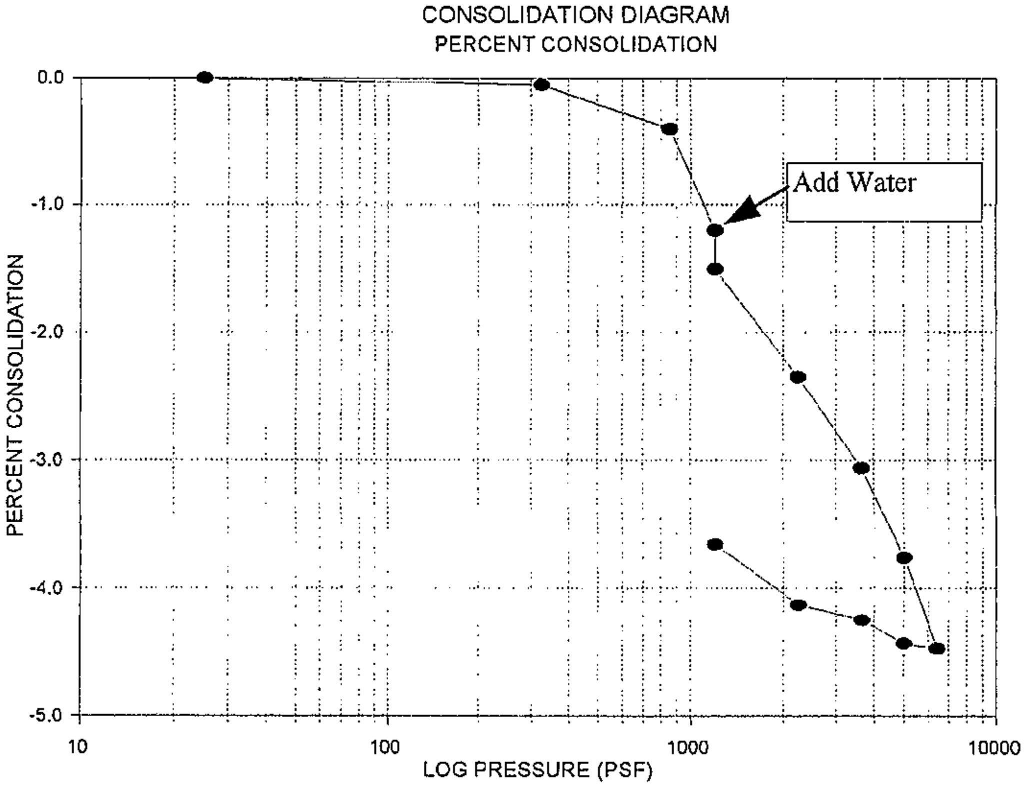
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B12-5'



Dry Density	106.8 pcf	Specific Gravity	2.65
Initial Moisture	19.2%	Initial Void Ratio	0.55
Initial % Saturation	92.8%	C'c	0.05

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

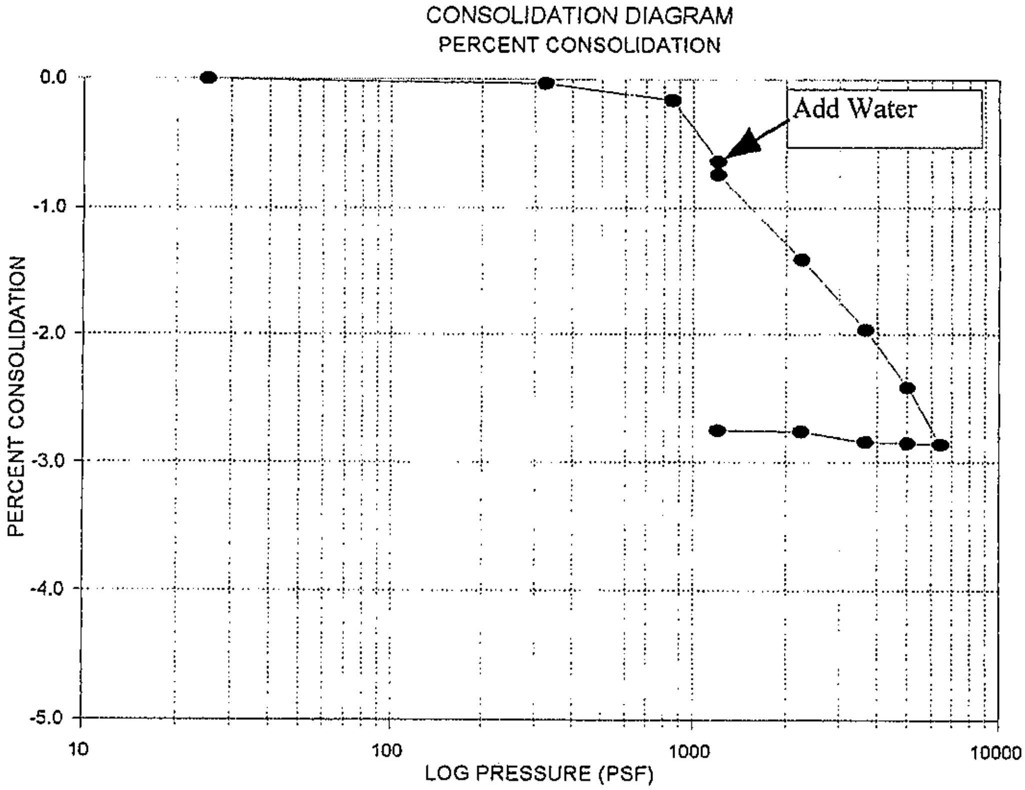
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B10-10'



Dry Density	110.6 pcf	Specific Gravity	2.65
Initial Moisture	12.8%	Initial Void Ratio	0.50
Initial % Saturation	68.5%	C'c	0.03

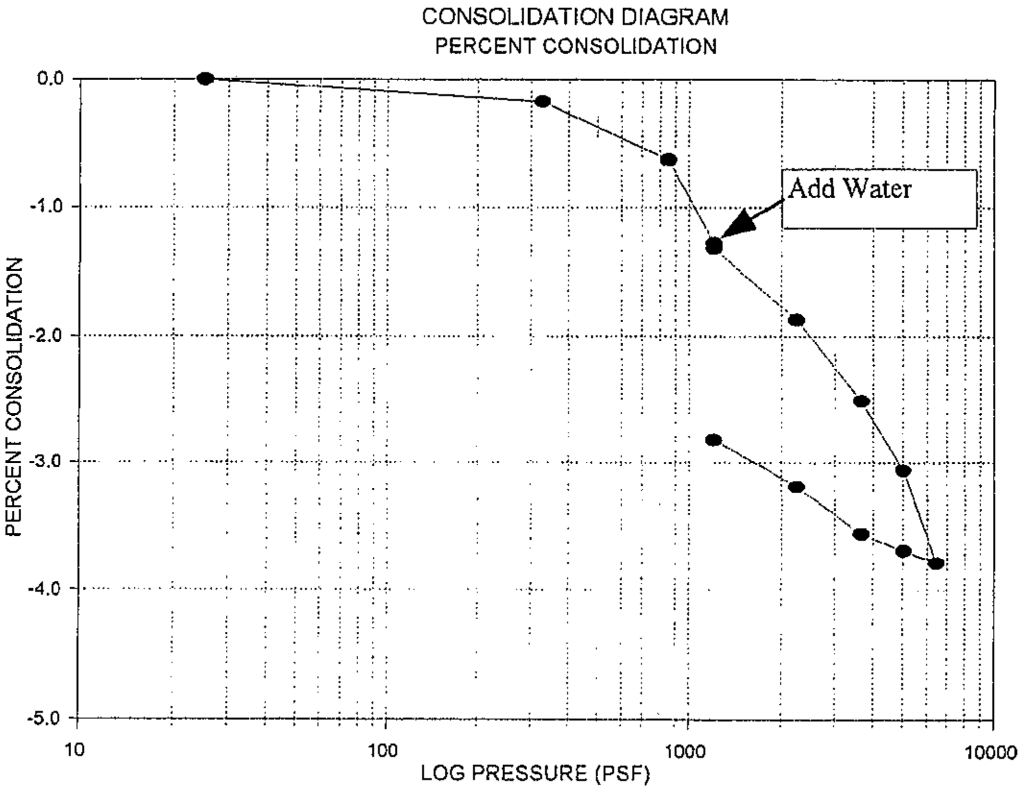
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B9-9'



Dry Density	99.4 pcf	Specific Gravity	2.65
Initial Moisture	23.1%	Initial Void Ratio	0.66
Initial % Saturation	92.2%	C'c	0.04

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
(818) 549-9959 FAX: (818) 543-3747

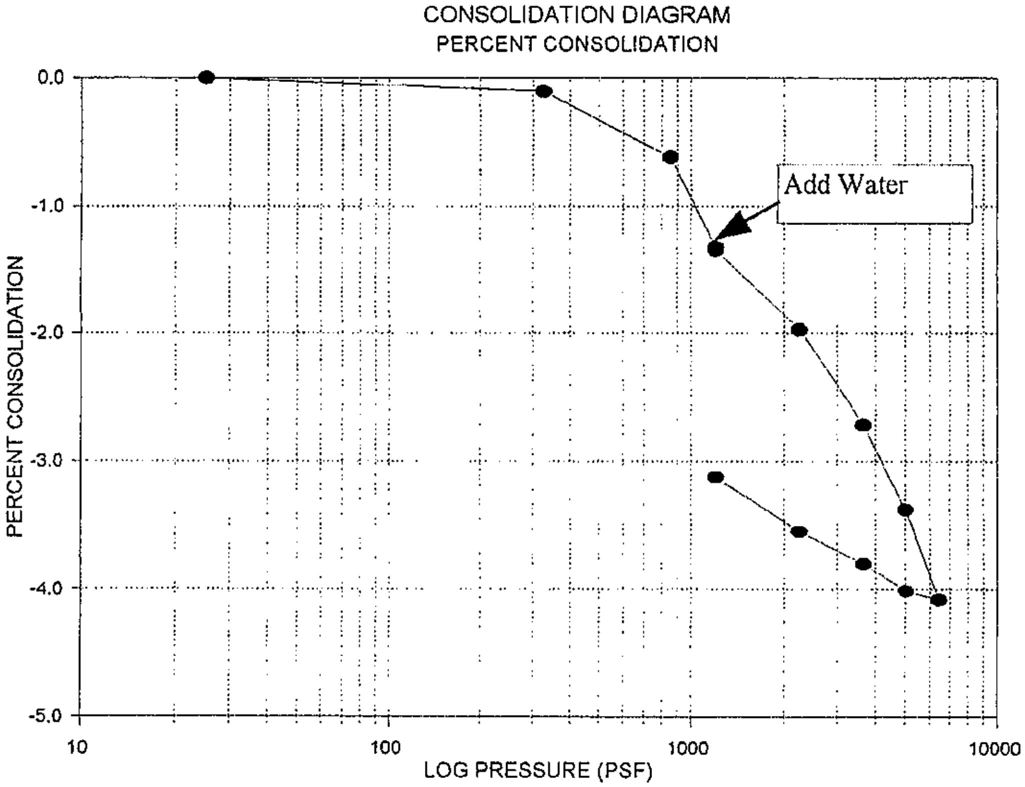
CONSOLIDATION DIAGRAM

JB: 18051-B Television Center

CONSULTANT: JWB

EARTH MATERIAL: Alluvium

LOCATION: B12-10'



Dry Density	100.5 pcf	Specific Gravity	2.65
Initial Moisture	20.0%	Initial Void Ratio	0.65
Initial % Saturation	82.1%	C'c	0.04

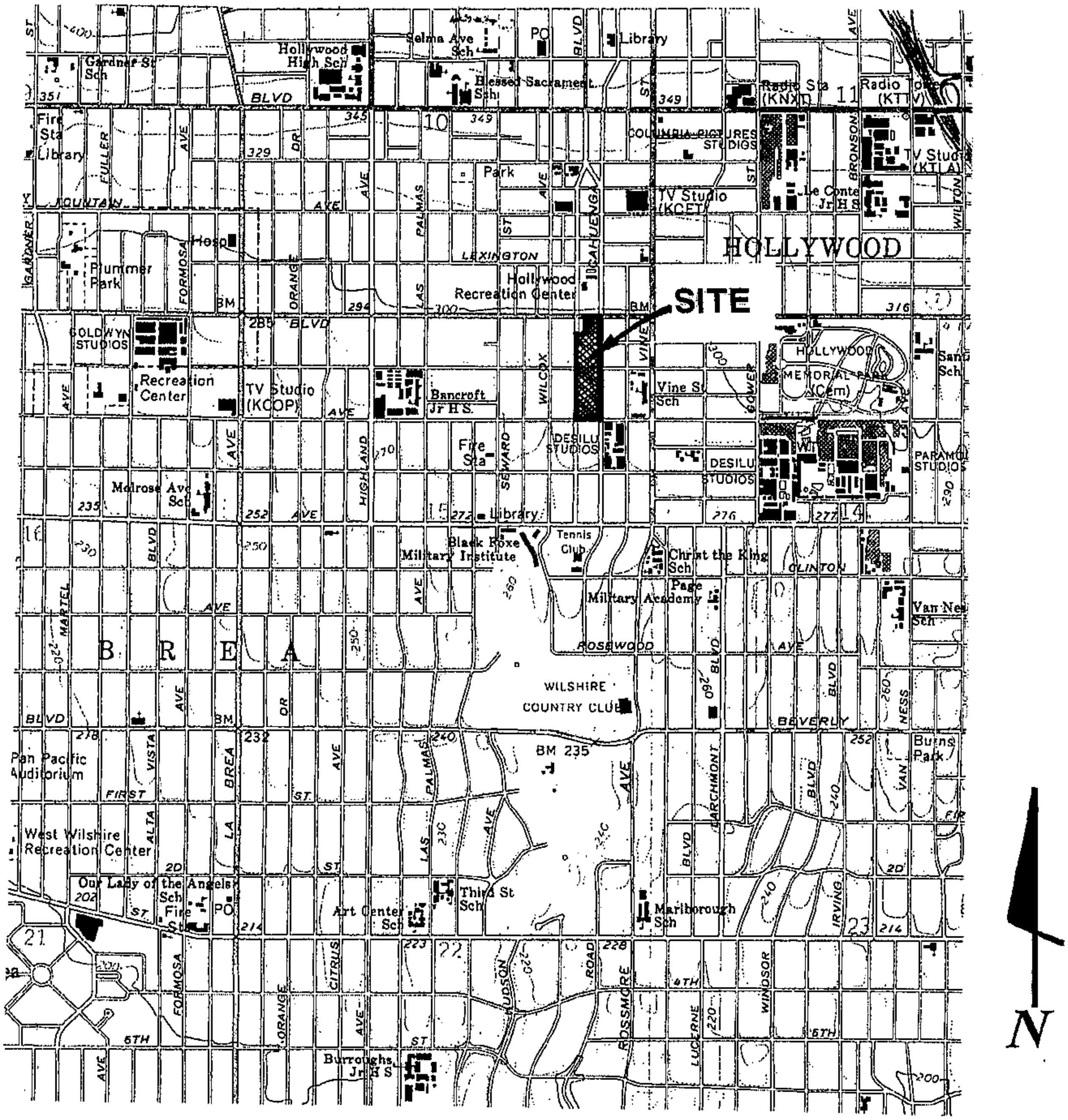
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

VICINITY MAP

JB 18051-B CLIENT: Television Center

CONSULTANT: JWB SCALE: 1" = 2,000'

REFERENCE: *U.S.G.S. 7.5 Minute Quadrangles, Beverly Hills and Hollywood Sheets, photorevised 1981.*



Project No: JB 18051-B

Log of Boring 1

Client: Television Center, Inc.

Location: 6311 Romaine Street **By:** JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
-1	1	**FILL:** Sandy Gravel, brown, red, gray, slightly moist to dry, medium dense to dense, concrete and brick fragments to 4 inches		SW						
-2	2	Sandy Clay, brown, moist, firm								
-3	3	**ALLUVIUM:**		CL	R	34	16.8	117.4		
-4	4	Silty Sand, brown to light reddish brown, moist very dense, some clay binder		SM						
-5	5			----	R	50	12.0	120.6		
-6	6									
-7	7	dark brown, slightly porous		----	R	50	15.7	115.4		
-8	8									
-9	9									
-10	10	Gravelly Sand, reddish brown, slightly moist, very dense		----	R	50 6"	10.8	118.4		
-11	11			SW						
-12	12									
-13	13									
-14	14									
-15	15									
-16	16									
-17	17									
-18	18									
-19	19			----	R	50 10"	17.1	110.4		
-20	20	End at 20 Feet; No Water; Fill to 3 Feet;								

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-27-99

Size: 8 Inch
Elevation:
Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 2

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
-1	1	**FILL:** Silty Sand, light brown, slightly moist, medium dense, concrete and brick fragments to 2 inches		SM						
-2	2	Clay, dark brown to black, moist, firm **ALLUVIUM:**		CL	R	20	18.5	108.8		
-3	3	Sandy Clay, dark brown, moist, stiff to very stiff								
-4	4			CL						
-5	5	Silty Sand, light reddish brown, slightly moist, very dense, some clay binder, some caliche veins		----	R	45	10.2	121.2		
-6	6			SM						
-7	7	Silty Sand, light brown to brown, slightly moist very dense								
-8	8									
-9	9									
-10	10	some gravel		----	R	30	10.4	114.7		
-11	11	Sandy Silt, brown to gray brown, moist, very firm								
-12	12			ML						
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 2 Feet.		----	R	50	18.0	112.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 3

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	FILL: Clay, dark brown to black, moist, firm, some concrete fragments to 2 inches								
-1	1									
-2	2			CL	R	16	19.0	108.6		
-3	3									
-4	4	ALLUVIUM: Sandy Clay, black to brown, moist, very firm								
-5	5			CL	R	17	16.4	112.8		
-6	6									
-7	7	Silty Sand, light reddish brown, slightly moist to moist, dense to very dense, some clay binder		SM	R	33	19.0	108.6		
-8	8	Sandy Silt, light brown to reddish brown, moist very firm		ML						
-9	9									
-10	10	Clayey Silt, reddish brown to grayish brown, moist, very firm, slightly porous		----	R	22	17.9	106.5		
-11	11			ML						
-12	12									
-13	13									
-14	14	Silty Sand, light reddish brown to gray, slightly moist, dense to very dense								
-15	15	End at 15 Feet; No Water; Fill to 4 Feet		SM	R	31	13.1	118.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 4

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black to dark brown, moist, firm								
-2	2			CL	R	18				
		Mixed black Clay and brown Silty Sand, moist medium dense								
-5	5			----	R	12	24.4	95.0		
		ALLUVIUM: Clay, black, moist, very firm								
-6	6			CL						
-7	7			----	R	18	21.2	104.0		
-10	10	Silt, light gray brown, moist to very moist, firm		----	R	13	27.0	97.4		
-11	11			ML						
-15	15	Clayey Silt, greenish gray, moist to very moist very firm		----	R	18	24.3	102.5		
-17	17	Sandy Silt, reddish brown to gray, moist, very firm								
-19	19			----	R	24	27.9	96.6		
-20	20	End at 20 Feet; Fill to 5 Feet.								

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-27-99

Size: 8 Inch
Elevation:
Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 5

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	FILL: Clay, black, moist, firm								
-1	1									
-2	2			CL	R	17	22.7	102.3		
-3	3									
-4	4									
-5	5	ALLUVIUM: Silty Sand, brown to dark brown, moist, dense		SM	R	15	14.9	112.4		
-6	6									
-7	7	Sandy Silt, dark gray brown, moist, very firm, some caliche veins		----	R	22	21.5	103.5		
-8	8			ML						
-9	9									
-10	10	Silty Clay, gray brown to brown, moist, very firm		----	R	16	30.9	89.8		
-11	11			CL						
-12	12			----	R	12	39.0	77.7		
-13	13									
-14	14									
-15	15	Sand, gray to brownish gray, wet, dense, fine to medium grained		----	R	16	21.9	101.7		
-16	16			SP						
-17	17									
-18	18	medium to coarse								
-19	19	Gravelly Sand, grayish brown to gray, wet, dense to very dense, End at 20 Feet; water at 15 Feet; Fill to 4½ Feet.		SW	R	48	118.5	118.5		
-20	20									

Surface: 2 Inch AC/4 Inch Base

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 6

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	FILL: Silty Sand, brown, moist, dense	▩							
-1	1			SM						
-2	2	ALLUVIUM: Clay, dark gray brown to black, moist, stiff to very stiff	▨							
-3	3			CL						
-4	4			----	R	19	21.3	104.7		
-5	5									
-6	6	Sandy Clay, brown to gray brown, moist, very stiff	▤	----	R	23	19.4	109.2		
-7	7									
-8	8	Silty Sand, reddish brown to brown, moist, very dense, some gravel, some clay binder	⁙	----	R	34	17.4	111.8		
-9	9			SM						
-10	10									
-11	11	Gravelly Sand, reddish brown, slightly moist, very dense	●							
-12	12			SW	R	40	11.4	108.2		
-13	13									
-14	14									
-15	15			----	R	32	12.6	112.5		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 1 of 2

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 6

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				—	R	50 10"	11.4	122.1		
-21	21									
-22	22									Water Measured at 21½ Feet After One Hour
-23	23									
-24	24	Silty Sand, light brown to light reddish brown, moist, dense								
-25	25			SM	SPT	6 9 12				
-26	26									Water Encountered at 26½ Feet
-27	27									
-28	28									
-29	29									
-30	30	Sandy Silt, light brown to gray, moist to very moist, very firm		—	SPT	8 14 14				
-31	31			ML						
-32	32									
-33	33									
-34	34									
-35	35	Silty Sand, light brown to brownish gray, moist dense to very dense		—	SPT	15 18 16				
-36	36			SM						
-37	37									
-38	38									
-39	39			—	SPT	16 18 19				
-40	40	End at 40 Feet; Water at 21½ Feet; Fill to 2 Feet								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-27-99

Size: 8 Inch

Elevation:

Sheet: 2 of 2

Project No: JB 18051-B

Log of Boring 7

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black, moist, firm	▨							
-1	1		▨	CL						
-2	2		▨	----	R	17	18.2	110.0		
-3	3		▨							
-4	4		▨							
		ALLUVIUM: Clayey Sand, light reddish brown, moist, very dense, some gravel	░							
-5	5		░	----	R	20	18.5	109.5		
-6	6		░	SC						
-7	7	Silty Sand, red brown, slightly moist, very dense, some clay binder	░	----	R	36	15.9	113.6		
-8	8		░	SM						
-9	9		░							
-10	10		░	----	R	30	9.9	111.9		
-11	11		░							
-12	12		░							
-13	13		░							
-14	14		░							
		End at 15 Feet; No Water; Fill to 4½ Feet.	░							
-15	15			----	R	32	18.9	106.1		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot
Drill Method: Hollow-Stem Auger
Drill Date: 4-27-99

Size: 8 Inch
Elevation:
Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 8

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface **FILL:** Gravelly Sand, dark gray brown, slightly moist dense								
-1	1			SW						
-2	2	**ALLUVIUM:** Sandy Clay, black, moist, stiff, slightly porous		—	R	16	20.3	105.2		
-3	3			CL						
-4	4	dark reddish brown, some gravel								
-5	5			----	R	21	14.0	113.9		
-6	6	Silty Sand, reddish brown, moist, dense to very dense, some clay binder, some gravel								
-7	7			SM	R	22	17.4	110.5		
-8	8									
-9	9	dense, slightly porous								
-10	10			----	R	15	13.0	109.8		
-11	11									
-12	12									
-13	13									
-14	14	Silty Sand, dark gray to brownish gray, moist, dense, slightly porous End at 15 Feet; No Water; Fill to 1½ Feet.								
-15	15			----	R	17	17.2	113.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Concrete Driveway

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 9

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
-1	1	**FILL:** Clay, black, moist, firm		CL						
-2	2	Silty Sand, brown to gray brown, moist, medium dense								
-3	3	**ALLUVIUM:** Sandy Clay, dark reddish brown, moist, stiff, some gravel		SM	R	10	12.3	112.2		
-4	4	slightly porous		CL						
-5	5	Clay, black to dark gray brown, moist, stiff, slightly porous								
-6	6			----	R	17	21.1	104.7		
-7	7									
-8	8									
-9	9	Silty Clay, brown, moist, firm to very firm		----	R	14	23.1	99.4		
-10	10									
-11	11	Sandy Clay, brown to brownish gray, moist, stiff to very stiff								
-12	12			----	R	25	15.6	114.3		
-13	13									
-14	14									
-15	15	Clay, gray to brown, moist, stiff		----	R	15	25.3	98.6		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20	Clayey Sand, brown to reddish brown, moist, dense to very dense with some gravel								

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
				SC	R	26	21.8	106.3		
-21	21									
-22	22									
-23	23									
-24	24									Water Encountered at 24 Feet
-25	25	Silty Sand, brown to gray, wet, dense, some clay binder		----	R	17	19.6	106.0		
-26	26			SM						
-27	27									
-28	28									
-29	29									
-30	30			----	SPT	7 10 14				
-31	31									
-32	32									
-33	33									
-34	34									
-35	35	Sandy Silt, brown, wet, firm		----	SPT	9 12 16				
-36	36			ML						
-37	37									
-38	38									
-39	39									
-40	40									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 2 of 3

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 9

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
		Silty Sand, light reddish brown to gray, wet, dense to very dense		----	SPT	14 16 19				
-41	41			SM						
-42	42									
-43	43									
-44	44									
-45	45	Sandy Silt, brown to gray, very moist, very firm		----	SPT	10 14 16				
-46	46			ML						
-47	47									
-48	48									
-49	49									
-50	50	End at 50 Feet; Water at 16 Feet; Fill to 3 Feet.		----	SPT	9 12 15				Water Encountered at 16 Feet after one hour
-51	51									
-52	52									
-53	53									
-54	54									
-55	55									
-56	56									
-57	57									
-58	58									
-59	59									
-60	60									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 3 of 3

Project No: JB 18051-B

Log of Boring 10

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Silty Sand, brown, slightly moist, medium dense								
-1	1	Sandy Clay, dark brown, moist, firm		SM						
-2	2	Clay, black, moist, stiff, slightly porous		CL	R	17	21.4	104.3		
-3	3									
-4	4									
-5	5	Sandy Clay, mottled dark gray brown to brown		—	R	16	19.3	107.5		
-6	6	**ALLUVIUM:** Clayey Sand, reddish brown, moist, dense		SC						
-7	7									
-8	8	Silty Sand, reddish brown, slightly moist, dense to very dense								
-9	9			SM						
-10	10			—	R	36	15.4	110.8		
-11	11									
-12	12	light reddish brown to gray								
-13	13									
-14	14									
-15	15	End at 15 Feet; No Water; Fill to 5½ Feet.		—	R	18	12.8	110.6		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Client: Television Center, Inc.

Location: 6311 Romaine Street

Log of Boring 11

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE

Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0	0	Ground Surface								
		FILL: Sandy Clay, black, moist, firm, fragments of brick								
-1	1			CL						
-2	2	**ALLUVIUM:** Clay, black, moist, very firm, slightly porous		----	R	15	21.3	103.8		
-3	3									
-4	4	dark brown								
-5	5	Sandy Clay, reddish brown, moist, very stiff, some gravel								
-6	6			----	R	35	14.0	117.4		
-7	7									
-8	8	Silty Sand, reddish brown, moist, very dense, some gravel								
-9	9			SM	R	26	11.0	113.5		
-10	10			SM						
-11	11									
-12	12									
-13	13	Sandy Silt, reddish brown to gray, moist, very firm								
-14	14	End at 15 Feet; No Water; Fill to 2 Feet								
-15	15			----	R	23	24.3	101.4		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

Project No: JB 18051-B

Log of Boring 12

Client: Television Center, Inc.

Location: 6311 Romaine Street

By: JET

The J. Byer Group, Inc.
512 East Wilson Ave., Suite 201
Glendale, CA 91206
(818) 549-9959

SUBSURFACE PROFILE										
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count Per Foot	Moisture Content (%)	Dry Density (pcf)	% Saturation	Remarks
0		Ground Surface								
	0	**FILL** Clayey Sand, brown to dark brown, moist, dense	▩							
-1	1		▩	SC						
-2	2	**ALLUVIUM:** Clay, black, moist, stiff	▨	----	R	20	27.1	97.9		
-3	3		▨	CL						
-4	4	Sandy Clay, brown, moist, very firm, slightly porous	▤							
-5	5		▤	----	R	19	19.2	106.8		
-6	6		▤							
-7	7		▤				20.8	109.4		
-8	8		▤	----	R	21				
-9	9	Sandy Silt, brown to light brown, moist, firm	░							
-10	10		░	ML	R	12	20.0	100.5		
-11	11		░							
-12	12	Clayey Silt, dark gray brown to brown, moist, firm	▧							
-13	13		▧							
-14	14		▧							
-15	15	End at 15 Feet; No Water; Fill to 2 Feet	▧	----	R	14	28.6	92.2		
-16	16									
-17	17									
-18	18									
-19	19									
-20	20									

Surface: Parking Lot

Drill Method: Hollow-Stem Auger

Drill Date: 4-28-99

Size: 8 Inch

Elevation:

Sheet: 1 of 1

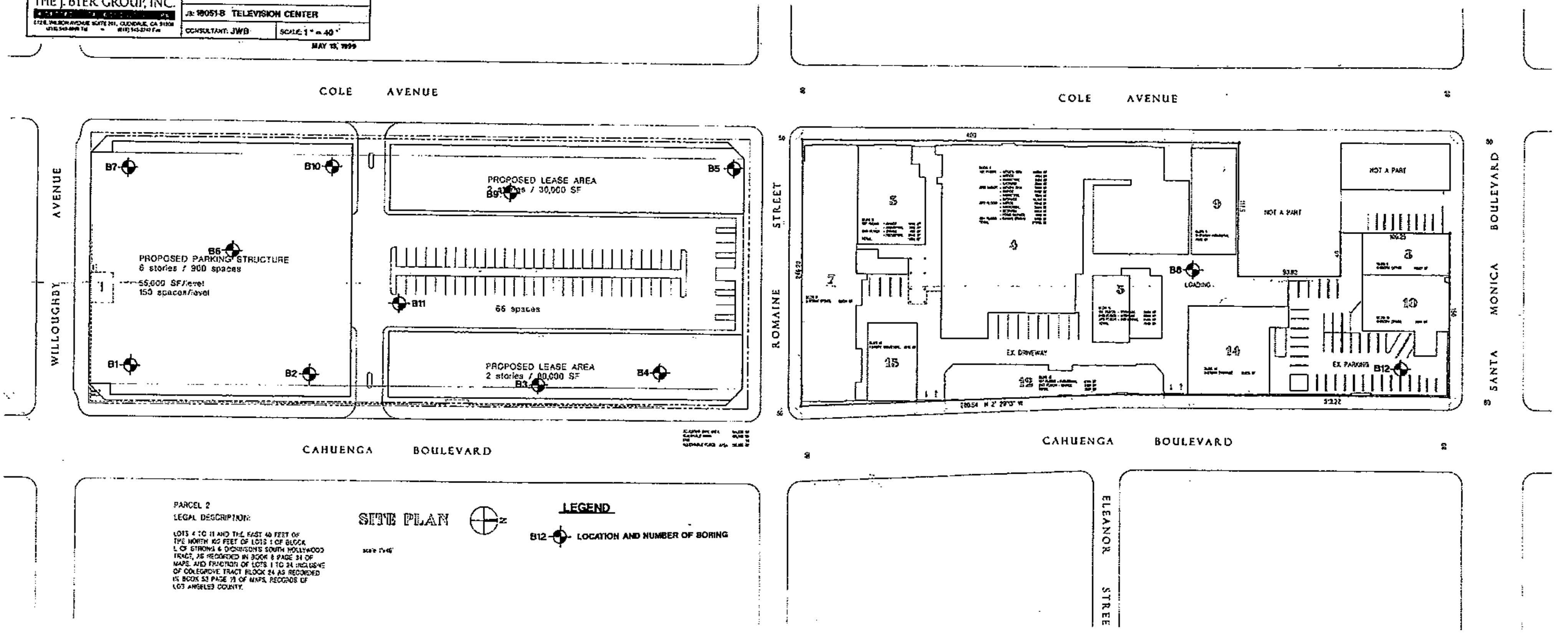
THE J. BYER GROUP, INC.

SITE PLAN

J#: 18051-B TELEVISION CENTER

CONSULTANT: JWB | SCALE: 1" = 40'

MAY 18, 1999



PARCEL 2

LEGAL DESCRIPTION:

LOTS 4 TO 11 AND THE EAST 40 FEET OF THE NORTH 100 FEET OF LOTS 1 OF BLOCK L OF STRONG & DICKINSON'S SOUTH HOLLYWOOD TRACT, AS RECORDED IN BOOK 8 PAGE 31 OF MAPS, AND FRACTION OF LOTS 1 TO 24 INCLUSIVE OF COLEGROVE TRACT BLOCK 24 AS RECORDED IN BOOK 53 PAGE 71 OF MAPS, RECORDS OF LOS ANGELES COUNTY.

LEGEND

B12 ⊕ LOCATION AND NUMBER OF BORING