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File Number 22403

ROM Investments, Inc.
6464 Sunset Boulevard, Suite 610
Los Angeles, California 90028

Attention: Leeor Maciborski

Subject: Geotechnical Engineering Investigation
Proposed Residential Complex
5416-5430 Carlton Way, Los Angeles, California

Dear Mr. Maciborski:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL COMPLEX
5416-5430 CARLTON WAY
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the design team. The site is proposed to be developed with a residential complex. The structure is proposed to be seven or eight stories over two subterranean levels. Preliminarily, the proposed subterranean levels will extend on the order of 20 feet below the existing site grade. Column loads are estimated to be between 700 and 1,000 kips. Wall loads are estimated to be between 6 and 10 kips per lineal foot. Grading will consist of excavations as deep as 25 feet in depth for the subterranean levels and foundation elements.



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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 Project Site is located at 5416-5430 Carlton Way, in the Hollywood area of the City of Los Angeles, California. The site is relatively level with approximately five feet of elevation change. The site is bounded by Carlton Way to the north, and by neighboring developments to the east, south, and west.

The site is currently developed with one to two story residential structures, and paved parking lot. The vegetation on the site consists of small trees and shrubs in landscape planters.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on July 19 and 20, 2023 by excavating two exploratory borings. The explorations varied in depth from 50 to 80 feet. The explorations were prosecuted with the aid of a limited access track mounted rig and a truck-mounted drilling rig using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 and A-2.

The location of exploratory excavations was determined by information furnished by the client. Elevations of the exploratory excavations were determined by hand level or interpolation from data provided. 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 were encountered in both of the explorations. The fill was found to be approximately 3 feet in depth and consists of sandy silts, which are dark brown, moist, and stiff, with minor construction debris. The upper native soils underlying the site consist of medium dense to very dense silty sands and clayey sands underlain by stiff to very stiff silty clays, sandy silts, and clay with various amounts of sand.

The geologic materials consist of (older) detrital 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 at a depth of 65 feet below the existing site grade in Boring B-2. Groundwater was not encountered in Boring B-1, which was excavated to a depth of 50 feet below the existing site grade.

The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled “Historically Highest Ground Water Contours”. Review of this plate indicates that the historically highest groundwater level is greater than 50 feet below grade.

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



Caving

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

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.



REGIONAL FAULTING

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

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines



“Holocene-active” Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

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

Ground 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 and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site.

The closest fault to the site which could cause surface rupture is the Hollywood Fault which is located approximately 0.35 miles north of the Project Site. The Hollywood Fault is part of the Transverse Ranges Southern Boundary fault system. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood–Beverly Hills area to the Los Feliz area of Los Angeles. The Hollywood fault is the eastern segment of the reverse oblique Santa Monica–Hollywood fault. Based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies, this fault is classified as active.

Until recently, the approximately 9.3-mile-long Hollywood Fault was considered to be expressed as a series of linear ground-surface geomorphic expressions and south-facing ridges along the south margin of the eastern Santa Monica Mountains and the Hollywood Hills. Multiple recent fault rupture hazard investigations have shown that the Hollywood Fault is located south of the



ridges and bedrock outcroppings along portions of Sunset Boulevard. The Hollywood Fault has not produced any damaging earthquakes during the historical period and has had relatively minor micro-seismic activity. It is estimated that the Hollywood fault is capable of producing a maximum 6.7 magnitude earthquake. In 2014, the California Geological Survey established an Earthquake Fault Zone for the Hollywood Fault.

The Project Site is not located within an Alquist-Priolo Earthquake Fault Zone. 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. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

Groundwater was encountered at a depth of 65 feet below ambient site grade in Boring B-2. The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report 026 Plate 1.2 entitled “Historically Highest Ground Water Contours”. Review of this plate indicates that the historically highest groundwater level is greater than 50 feet below grade.

The proposed development will be constructed over 2 subterranean levels, which will extend on the order of 25 feet below the existing site grade when including the foundation system. Additionally, the proposed development will be designed to infiltrate stormwater at an



approximate depth of 15 feet below the foundation system (which will be discussed in a later section of this report). Therefore, for the purpose of performing a liquefaction evaluation, a historic high groundwater of 40 feet has been conservatively assumed for the enclosed liquefaction evaluation.

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). The enclosed liquefaction analysis was performed using the semi-empirical method based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

The modal magnitude (M_W) and the peak ground acceleration (PGA_M) were obtained from the USGS Probabilistic Seismic Hazard Deaggregation program and the ASCE 7 Hazard Tool program, respectively. Based on these programs, a M_W of 6.9 and a PGA_M of 0.998g were obtained for the Project Site. These seismic parameters are utilized for the enclosed liquefaction analysis.

The enclosed liquefaction evaluation is based on blowcount data collected from Boring B2. 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 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 F-Plate. Based on the collected SPT data, the enclosed liquefaction analysis indicates that the soils underlying the site would not be capable of liquefaction during the maximum considered earthquake ground motion.



Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. No major water-retaining structures are located immediately adjacent to the Project Site. Therefore, the risk of flooding from a seismically-induced seiche is considered to be remote.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a breached upgradient reservoir.



Landsliding

The CGS Seismic Hazards Zones Map of the Hollywood Quadrangle indicates that the site is not located within an Earthquake-Induced Landslide Zone (CGS, 2014). 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 multi-story residential structure is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Up to 3 feet of existing fill materials was encountered in the explorations. The existing fill materials are not suitable for support of the proposed foundations, floor slabs or additional fill. The proposed development will be constructed over 2 subterranean levels extending on the order of 25 feet below the existing site grade when considering the proposed foundation system. Therefore, it is anticipated that excavation of the proposed subterranean levels and foundation elements will remove the existing fill and expose the underlying dense native soils. The proposed structure may be supported on conventional foundations bearing in the underlying dense native soils.

Due to the location of the proposed structure relative to property lines, public way, and existing structures, the excavation of the proposed subterranean level will require shoring measures to provide a stable excavation.

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



SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE 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)	2.113g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.113g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.408g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.759g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.290g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.860g*

* 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

Up to 3 feet of existing fill materials was encountered during explorations. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and be wasted from the Project Site.

EXPANSIVE SOILS

The onsite geologic materials are in the moderate expansion range. The Expansion Index was found to be 50 for a bulk sample 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.

WATER-SOLUBLE SULFATES

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

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

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on the most recent revision to American Concrete Institute (ACI) Standard 318, the sulfate exposure is classified as S0, and is considered to be negligible for sulfate



attack on concrete. Therefore, there are no restriction on the cement types which may be utilized for concrete foundations in contact with the site soils.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on this review it appears that the subject property is not located within a Methane Zone or a Methane Buffer Zone as designated by the City.

GRADING GUIDELINES

The following grading guidelines may be utilized 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.
- 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 recompact 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 recompact 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 comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. The granular soils tested by this firm would require the 95 percent compaction requirement. Comparative compaction is defined, for purposes of these guidelines, as the ratio of the in-place density to the maximum density as determined by applicable ASTM testing.

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. 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. in general accordance with the most recent revision of ASTM D 1557.

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

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported



materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Shrinkage

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



Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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



FOUNDATION DESIGN

Conventional

The proposed structure may be supported on conventional foundations bearing in the underlying dense native soils. Continuous foundations may be designed utilizing an allowable bearing capacity of 3,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 18 inches into the recommended bearing material.

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

The allowable bearing capacity increase for each additional foot of width is 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 7,000 pounds per square foot.

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

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 foundation, and two should be placed near the bottom.



Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed residential complex may bear in properly compacted fill and/or the native soils. Continuous footings may be designed for a bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Lateral Design

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

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

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



Foundation Observations

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

RETAINING WALL DESIGN

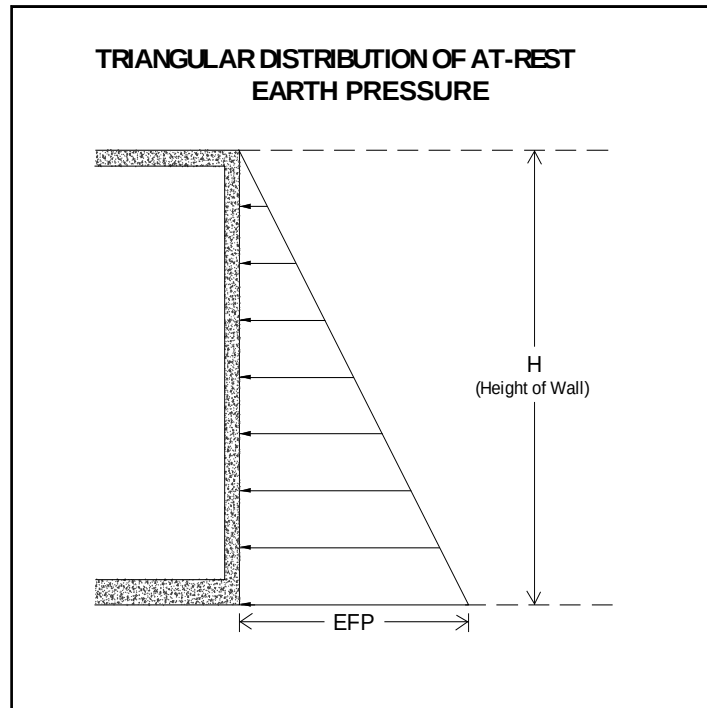
Cantilever Retaining Walls

Retaining walls may be designed utilizing a triangular distribution of active earth pressure. Cantilever retaining walls up to 20 feet in height supporting a level backslope may be designed for 45 pounds per cubic foot. For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes would be 70 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.





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

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



Dynamic (Seismic) Earth Pressure

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

Surcharge from Adjacent Structures

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

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2020-83, 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.



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

Waterproofing

Moisture 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.

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

Retaining Wall Drainage

Retaining wall subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rock. As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to once inch crushed rock, wrapped in filter fabric.



Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

Where shoring will not allow the installation of a standard subdrainage system outside the wall, rock pockets may be utilized. The rock pockets with should drain through the wall. The pockets should be a minimum of 12 inches in length, width, and depth. The pocket should be filled with gravel. The rock pockets should be spaced no more than 8 feet on center. A collector is placed within the gravel which directs collected waters through the wall to a sump or standard collector pipe system constructed under the slab. This method should be approved by the retaining wall designer prior to implementation.

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 in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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



Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered during exploration to a depth of 65 feet which corresponds to 40 feet below the base of the proposed structure. Therefore, the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

TEMPORARY EXCAVATIONS

It is anticipated that excavations up to 25 feet in vertical height will be required for the 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 or structures should be shored.

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

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from



entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than two 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 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Casing may be required should caving be experienced in the granular (saturated) geologic 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 p.s.i. of 1,000 over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

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.

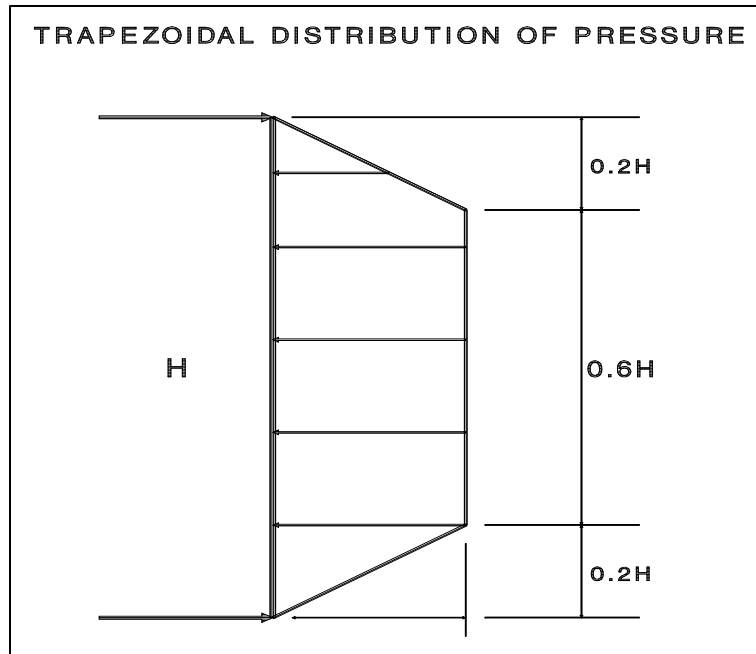
Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
25	37½

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





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

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (pounds per square foot)
25	24H

Where H is the height of the wall 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. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.



Tieback Anchors

Tieback 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 350 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 25-foot high shoring, a tieback anchor elevation 6 feet below grade and a minimum 20 foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchors should be placed at least 6 feet on center to be considered isolated. It is recommended that at least 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.



Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of 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.

Pre-Construction Survey

Prior to excavation of the proposed basement levels, it is recommended the surrounding structures and improvements be surveyed to provide a documented record of their condition. It is recommended this include video and/or photographic documentation as well. Such a survey would aid in the resolution of any disputes that may arise concerning damage to adjacent facilities caused by the proposed construction.

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

Concrete floor slabs should be a minimum of 5 inches in thickness, and should 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 or 95 percent of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, should be reinforced with a minimum of #3 steel bars on 18-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 or 95 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

Based on ACI 302.2R-30, Chapter 7, for projects which do not have vapor sensitive coverings or humidity-controlled areas, a vapor retarder/barrier is not necessary. Where a vapor retarder/barrier is considered necessary, the design of the slab and the installation of the vapor retarder/barrier should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder/barrier should comply with ASTM E 1745 Class A requirements. The necessity of a vapor retarder/barrier is not a geotechnical issue and should be confirmed by qualified members of the design team.



Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor retarder/barrier should be provided. Figure 7.1 shows that the slab should be poured on the vapor retarder/barrier. The ACI guide notes in 5.2.3.2 that the decision to locate the vapor retarder/barrier in direct contact with the slab's underside had long been debated. Experience has shown, however, that the greatest level of protection for floor coverings, coating, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above is not a geotechnical issue and should be confirmed by qualified members of the design team.

Where a vapor retarder/barrier is used, it should be placed on a level and compact subgrade. Precautions should be taken to protect the vapor retarder/barrier from damage during installation of reinforcing, utilities and concrete. The use of stakes driven through the vapor retarder/barrier should be avoided. Repair any damaged areas of the vapor retarder/barrier prior to concrete placement.

Groundwater was encountered on the subject site at depths deeper than 65 feet. Proposed concrete slabs-on-grade do not need to be supported on a layer of compacted aggregate to provide a capillary break.

Concrete Crack Control

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



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

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompact 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 recompact to 90 percent of the maximum density as determined by the most recent revision of ASTM D 1557. The client and the design team 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

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 consist of Crushed Aggregate Base which conform with Section 200-2.2 of the most recent edition of “Standard Specifications for Public Works Construction”, (Green Book).

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress.

SITE DRAINAGE

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

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



within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

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

Percolation Testing

Percolation testing was conducted in Boring B-1. Boring B-1 was drilled to a depth of 50 feet. At the completion of drilling, a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a slotted PVC pipe within the lower 15 feet of the borehole, and solid PVC pipe to the top of the borehole. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

Prior to testing, the borehole was filled with water for the purpose of pre-soaking for 2 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first.



A field percolation rate of 16.5 inches was obtained from the percolation test. Using a safety factor of 3, an infiltration rate of 5½ inches may be utilized for the design of the proposed stormwater infiltration system.

At the completion of the percolation testing, the PVC casing was removed from the percolation testing well, and the resulting hole was backfilled with on-site soils to the ground surface. An asphalt patch was placed.

The Proposed System

Since the proposed development will be constructed over 2 subterranean levels extending on the order of 25 feet below the ground surface, it is recommended that a deep drywall system be utilized for the proposed infiltration system. The infiltration system shall be designed to infiltrate at a depth of 15 feet below the bottom of the proposed foundation system. Additionally, groundwater was encountered at a depth of 65 feet below the existing site grade during exploration, therefore, the bottom of the proposed infiltration system should not extend below a depth of 55 feet below the existing grade.

The final location and design of the proposed infiltration system shall be reviewed and approved by this office prior to construction to evaluate whether the intent of the recommendations provided by this firm are satisfied.

Recommendations

The design and construction of stormwater infiltration facilities is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:



- Open infiltration basins have many negative associated issues. Such a design must consider attractive nuisance, impacts to growing vegetation, impacts to air quality and vector control.
- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.
- All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.
- Excavations proposed for the installation of stormwater facilities should comply with the “Temporary Excavations” sections of this (the referenced) reports well as CalOSHA Regulations where applicable.

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 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 effect 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 general accordance with 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 in general accordance with 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.

General accordance with 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.



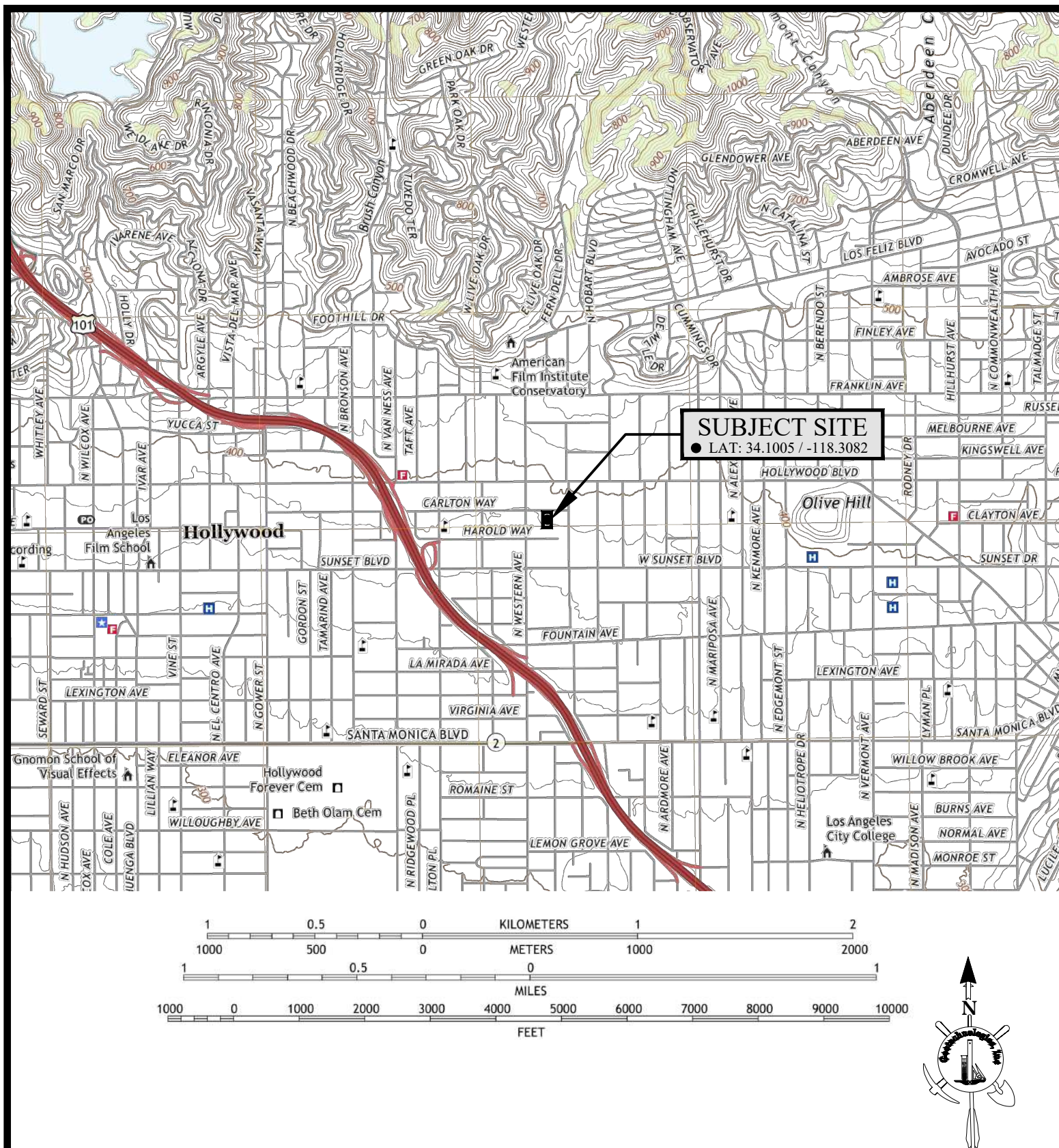
REFERENCES

- 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.
- California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
- City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
- 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.
- Division of Oil Gas and Geothermal Resources (DOGGR), 2001, Regional Wildcat Map, Northern Los Angeles Basin, map W1-5, map scale 1:48,000.
- Dolan, J.F., Sieh, K., Rockwell, T.K., Guphill, P., and Miller, G., 1997, Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California, GSA Bulletin, v. 109: no 12, p1595-1616.
- Hart, E.W. and Bryant, W.A., 1999 (updated 2005), Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.
- Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.



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www.geotek.com



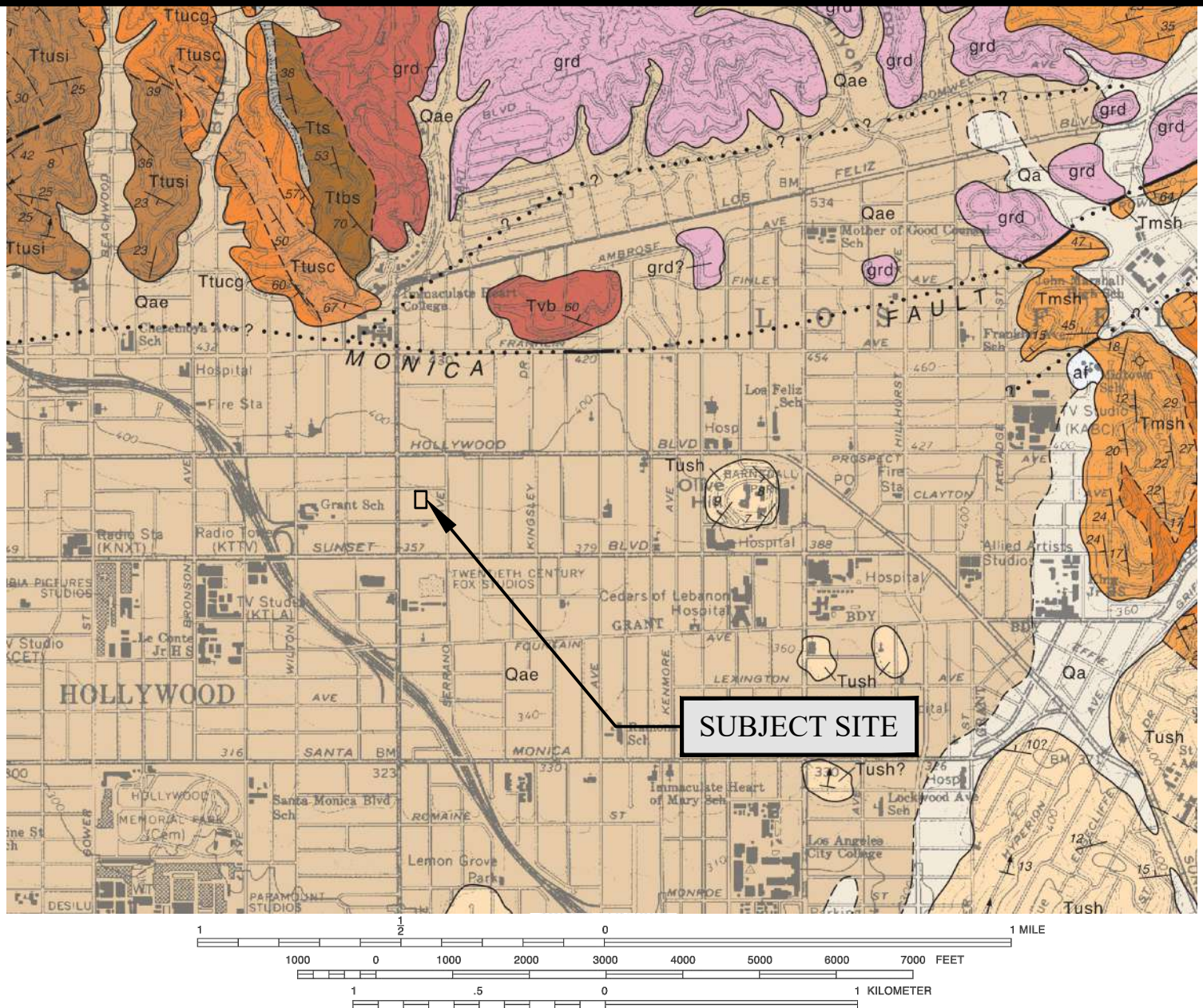
REFERENCE: USGS TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE 2022

VICINITY MAP

Geotechnologies, Inc.
 Consulting Geotechnical Engineers

5416 - 5430 CARLTON WAY, LOS ANGELES

FILE NO: 22403



LEGEND

- Qa: Surficial Sediments - alluvium: clay, sand & gravel
- Qae: Older Surficial Sediments - similar to Qa, but slightly elevated and dissected
- Tush: Unnamed Shale - gray to light brown, thin-bedded silty clay shale, soft and crumbly
- Tmsh: White-weathering, thin bedded, platy, siliceous shale, locally porcelaneous and silty
- Tmss: Tan to light gray semi-friable sandstone; includes thin interbeds of micaceous silty clay shale
- 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
- Ttucg: Light to medium gray, crudely bedded; ranges from coarse pebbly sandstone to cobble-boulder conglomeration composed mostly of granitic detritus
- Tvb: Basaltic volcanic rocks: dark gray to black, fine grained, massive to locally vesicular and/or pillowed
- grd: Granodiorite, light gray, massive moderately hard, composed mostly of plagioclase feldspar, lesser amounts of quartz

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

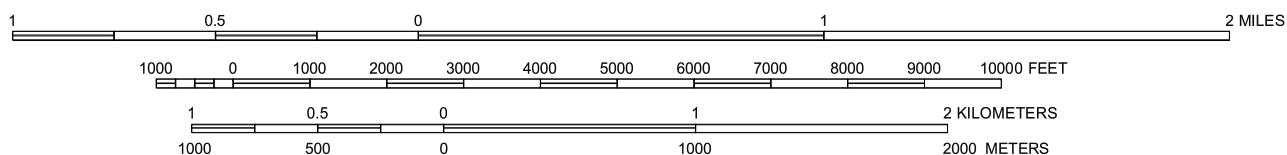
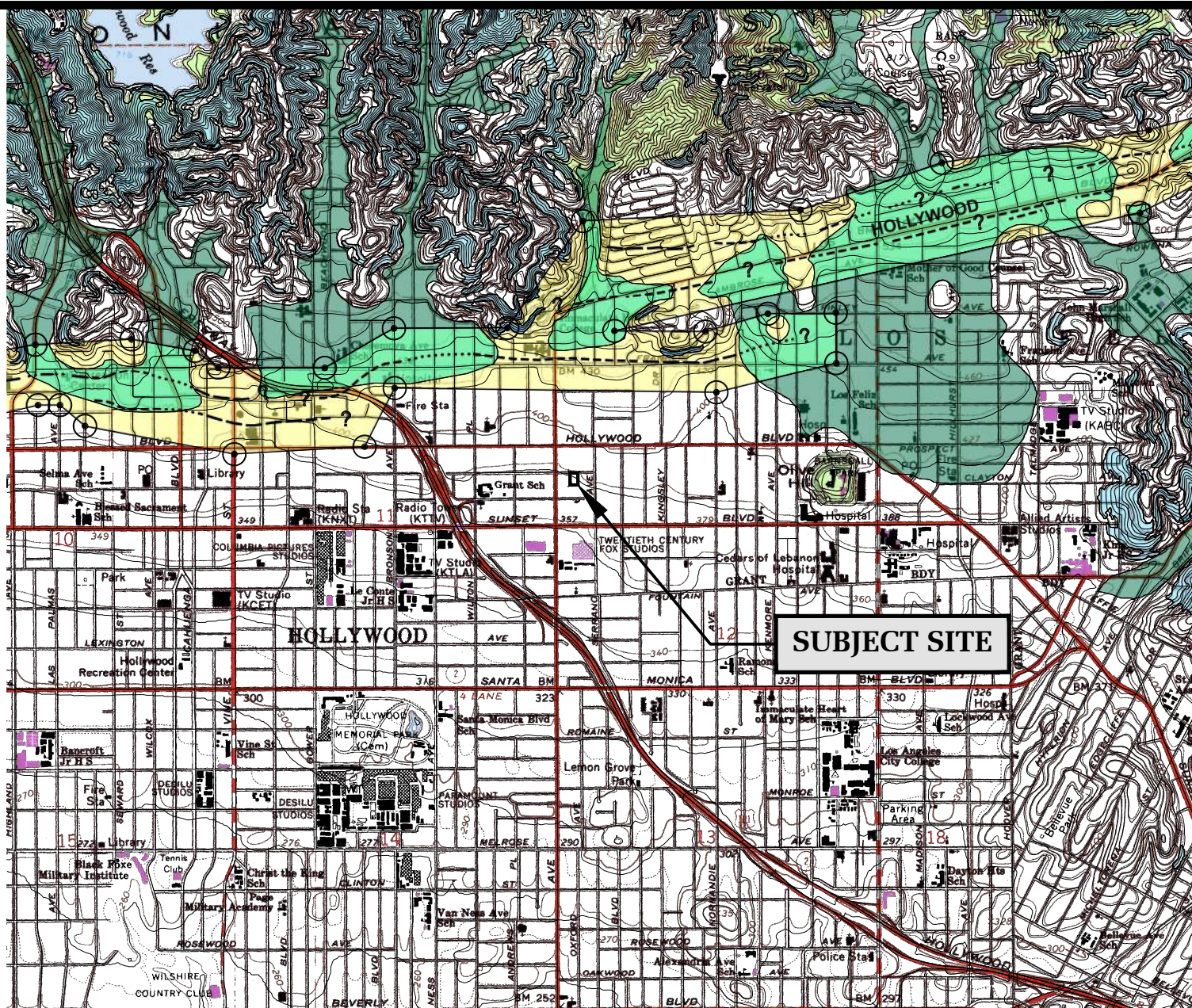
GEOLOGIC MAP (DIBBLEE)



Geotechnologies, Inc.
Consulting Geotechnical Engineers

5416 - 5430 CARLTON WAY, LOS ANGELES

FILE NO: 22403



LEGEND

-  EARTHQUAKE FAULT ZONES
-  LIQUEFACTION ZONES
-  EARTHQUAKE-INDUCED LANDSLIDE ZONES

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



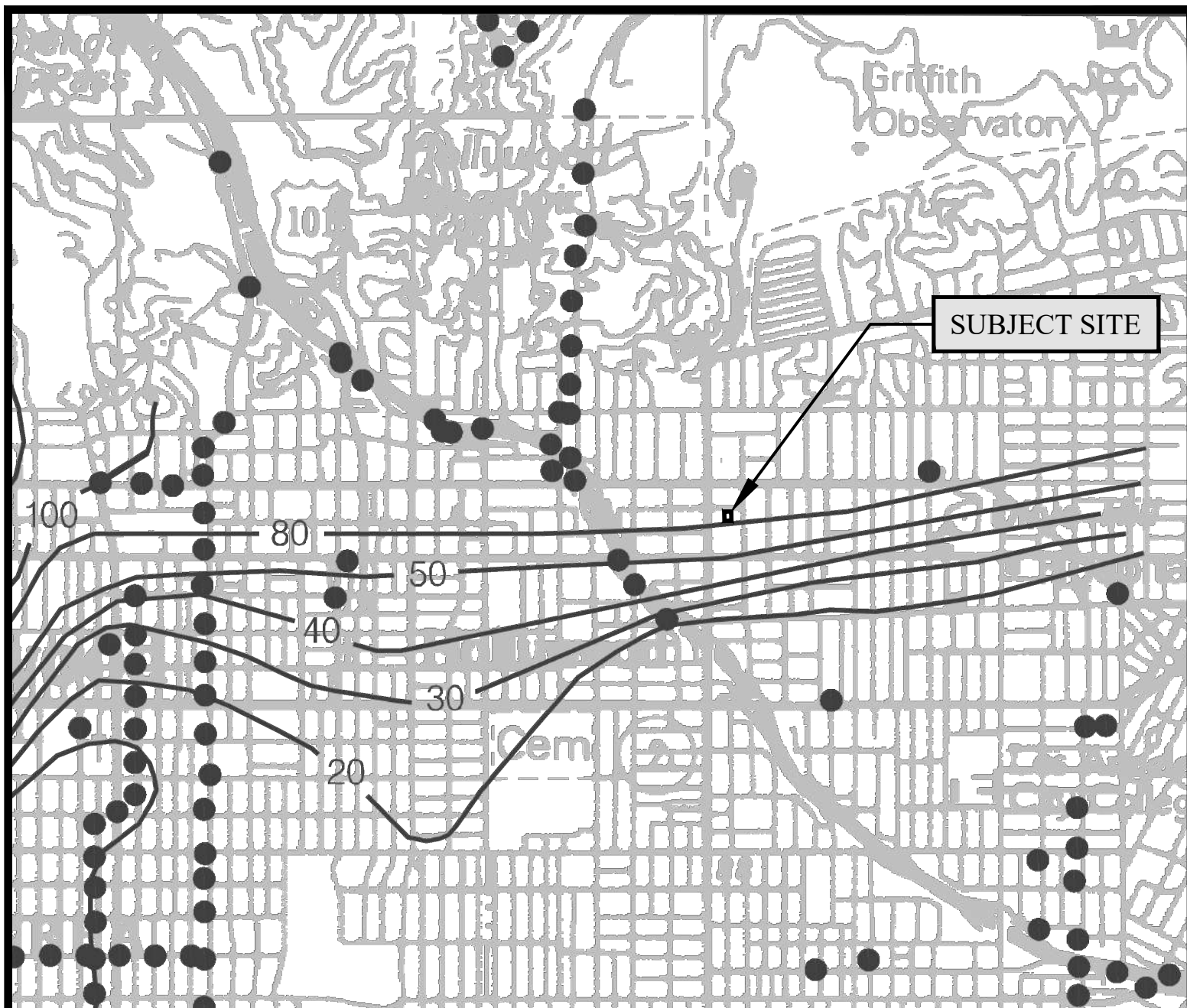
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

5416 - 5430 CARLTON WAY, LOS ANGELES

FILE NO: 22403





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

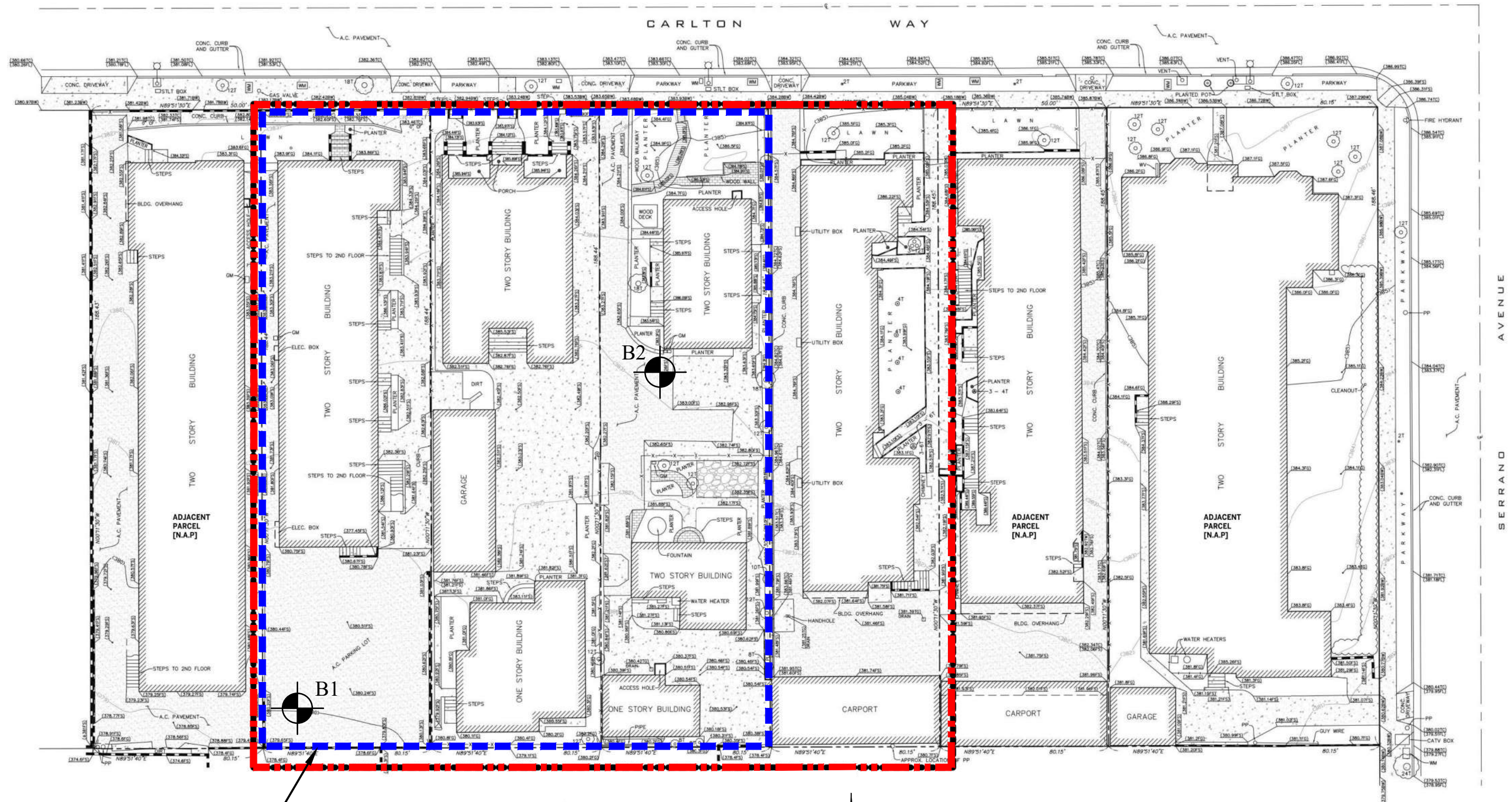
HISTORICALLY HIGHEST GROUNDWATER LEVELS



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Consulting Geotechnical Engineers

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FILE NO: 22403

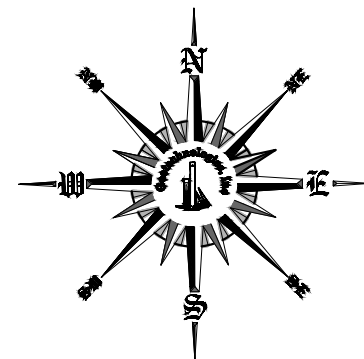


PROJECT SITE

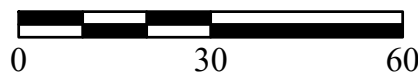
LEGEND



LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

 PROPERTY LINE

SCALE IN FEET



PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

ROM INVESTMENTS

Drawn by: JD	File No.: 22403
--------------	-----------------

Date: August 2023

BORING LOG NUMBER 1

Rom Investments, Inc.

Date: 07/19/23

Elevation: 380'

File No. 22403

Method: 8-inch diameter Hollow Stem Auger

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt for Parking
				-		2-inch Asphalt, No Base
				1 --		FILL: Silty Sand, dark brown, moist, medium dense
				-		
2.5	7	14.0	104.1	2 --		
				-		
				3 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				4 --		
				-		
5	6	12.6	SPT	5 --		
				-		
				6 --		
				-		
7.5	32	27.7	87.7	7 --		
				-		
				8 --	ML	Sandy Silt, dark brown, moist, stiff, fine grained
				-		
				9 --		
				-		
10	20	21.4	SPT	10 --		
				-		
				11 --	CH	Silty Clay, dark brown, moist, very stiff
				-		
				12 --		
12.5	27	20.6	96.4	-		
				13 --	ML	Sandy Silt, dark brown, moist, stiff
				-		
				14 --		
				-		
15	20	18.7	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	21	15.6	105.3	-		
				18 --	SM	Silty Sand, dark brown, moist, loose, fine grained
				-		
				19 --		
				-		
20	18	15.7	SPT	20 --		
				-		
				21 --	CL-ML	Silty Clay, dark brown, moist, stiff, fine grained
				-		
				22 --		
22.5	20	18.8	94.0	-		
				23 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				24 --		
				-		
25	10	13.1	SPT	25 --		
				-		

BORING LOG NUMBER 1

Rom Investments, Inc.

File No. 22403

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
27.5	52	12.5	122.7	- 26 -- -		
30	23	14.1	SPT	27 -- - 28 -- - 29 -- -		
32.5	51	11.5	124.9	30 -- -	ML	Sandy Silt, dark brown, moist, very stiff
35	21	11.8	SPT	31 -- -		
37.5	51	17.9	116.0	32 -- -	SM	Silty Sand, dark brown, moist, medium dense, fine grained
40	21	17.9	SPT	33 -- -		
42.5	48	17.9	114.1	34 -- -	SC	Clayey Sand, dark brown, moist, medium dense, fine grained
45	24	16.5	SPT	35 -- -		
47.5	52	14.3	110.2	36 -- -	CL-ML	Silty Clay, dark brown, moist, very stiff
50	33	13.2	SPT	37 -- -		
				38 -- -		
				39 -- -		
				40 -- -		
				41 -- -		
				42 -- -		
				43 -- -	ML	Sandy Silt, dark brown, moist, very stiff
				44 -- -		
				45 -- -		
				46 -- -	SC	Clayey Sand, dark and yellowish brown, moist, medium dense to very dense
				47 -- -		
				48 -- -		
				49 -- -		
				50 -- -		
						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
						Total Depth 50 feet No Water Fill to 3 feet

BORING LOG NUMBER 2

Rom Investments Inc.

Date: 07/20/23

Elevation: 383'

File No. 22403

Method: 8-inch diameter Hollow Stem Auger

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				0 --		Surface Conditions: Asphalt for Parking
				-		2½-inch Asphalt, No Base
				1 --		FILL: Silty Sand, dark brown, moist, medium dense
				-		
2.5	16	15.8	93.2	2 --		
				-		
				3 --	SM	Silty Sand, dark brown, moist, stiff
				-		
				4 --		
				-		
5	13	14.2	SPT	5 --		
				-		
				6 --	SC	Clayey Sand, dark brown, moist, medium dense
				-		
7.5	53	23.2	92.1	7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	26	14.1	SPT	10 --		
				-		
				11 --	ML	Sandy Silt, dark brown, moist, very stiff
				-		
				12 --		
				-		
12.5	63	14.3	93.7	13 --		
				-		
				14 --		
				-		
15	24	12.1	SPT	15 --		
				-		
				16 --	SM	Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
				-		
17.5	53	7.3	114.1	17 --		
	50/5.5"			-		
				18 --		dark brown, moist, very dense, fine grained
				-		
				19 --		
				-		
20	25	12.9	SPT	20 --		
				-		
				21 --		
				-		
22.5	62	16.8	90.3	22 --		
	50/5"			-		
				23 --		dark and yellowish brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	64	11.0	SPT	25 --		
				-		
						dark brown, moist, very dense, fine grained

BORING LOG NUMBER 2

Rom Investments Inc.

File No. 22403

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
27.5	100/6.5"	9.1	115.3	27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	68	9.5	SPT	30 --		dark and yellowish brown
				-		
				31 --		
				-		
32.5	48 50/5"	4.8	106.2	32 --		
				-		
				33 --	SP	Sand, dark and yellowish brown, moist, very dense, fine to medium grained
				-		
				34 --		
				-		
35	48	5.5	SPT	35 --		
				-		dense
				36 --		
				-		
37.5	42 50/5"	5.3	111.8	37 --		
				-		
				38 --		very dense, fine grained
				-		
				39 --		
				-		
40	54	12.1	SPT	40 --		
				-	ML	Sandy Silt, dark and yellowish brown, moist, fine grained, very stiff
				41 --		
				-		
				42 --		
42.5	72	17.2	111.6	-		
				43 --		
				-		
				44 --		
				-		
45	47	14.6	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	45 50/4"	15.5	114.5	-		
				48 --		
				-		
				49 --		
				-		
50	49	16.2	SPT	50 --		
				-		

BORING LOG NUMBER 2

Rom Investments Inc.

File No. 22403

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
52.5	32 50/4"	13.6	112.1	53 --	SM	Silty Sand, dark brown, moist, very dense, fine grained
				-		
				54 --		
				-		
55	30	20.4	SPT	55 --		
				-		
				56 --	ML	Sandy Silt, dark brown, moist, fine grained, stiff to very stiff
				-		
				57 --		
				-		
57.5	56	21.7	104.2	58 --		
				-		
				59 --		
				-		
60	48	16.2	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	29 50/5"	18.9	108.5	63 --		
				-		
				64 --		
				-		
65	40	18.7	SPT	65 --		
				-		
				66 --	SC	Clayey Sand, dark brown, moist, dense
				-		
				67 --		
				-		
67.5	33 50/4"	17.6	106.5	68 --	ML	Sandy Silt, dark brown, very moist, very stiff
				-		
				69 --		
				-		
70	38	23.5	SPT	70 --		
				-		
				71 --	CL	Silty Clay, dark brown, wet, fine grained, very stiff
				-		
				72 --		
				-		
72.5	45	20.3	106.3	73 --	SM	Silty Sand, dark brown, wet, medium dense, fine grained
				-		
				74 --		
				-		
75	45	18.6	SPT	75 --		
				-	SC	Clayey Sand, dark brown, wet, dense to very dense

BORING LOG NUMBER 2

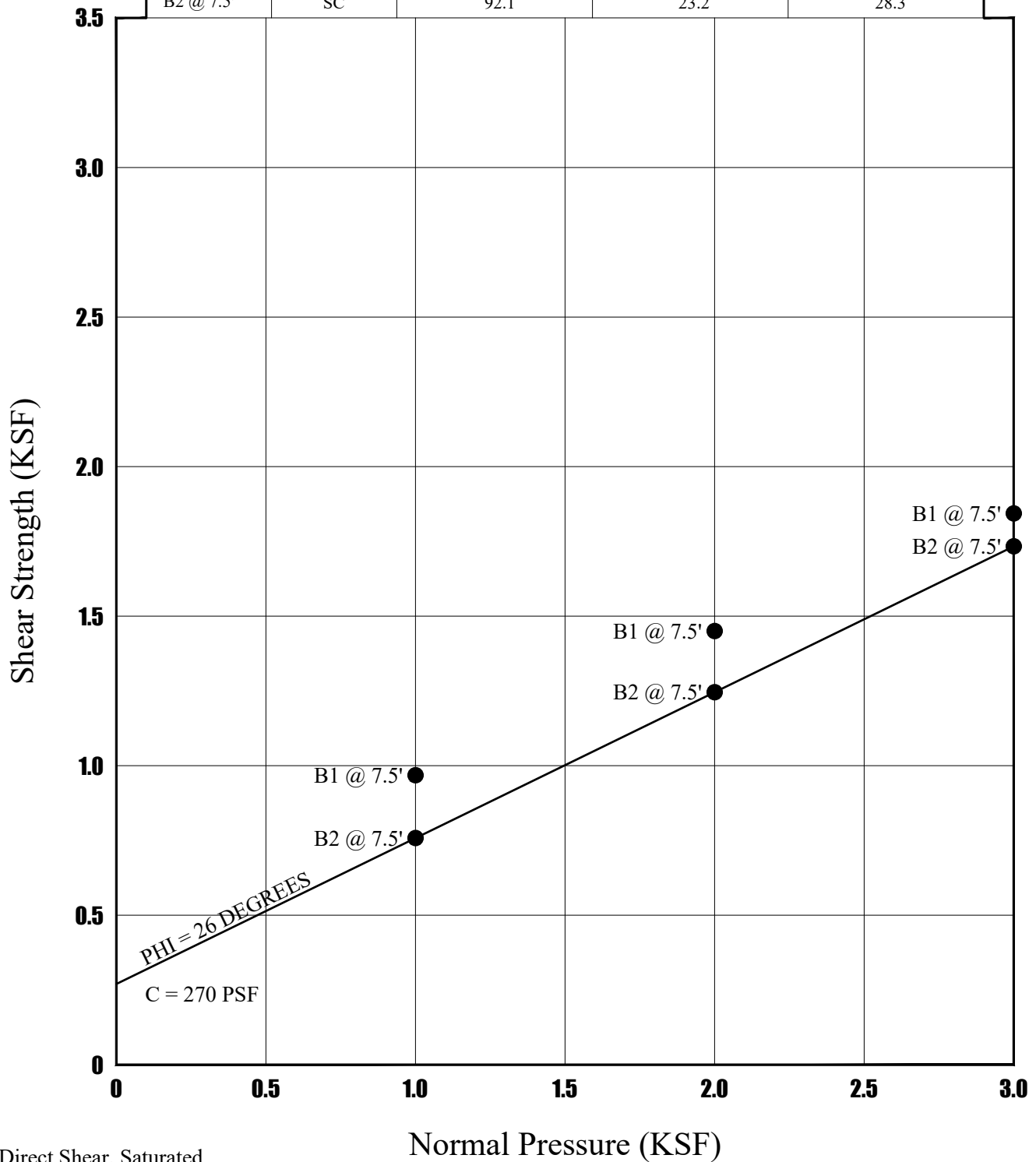
Rom Investments Inc.

File No. 22403

kk/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
77.5	47 50/4"	15.3	116.7	- 76 -- - 77 -- - 78 -- - 79 -- -		
80	57	22.2	SPT	80 -- - 81 -- - 82 -- - 83 -- - 84 -- - 85 -- - 86 -- - 87 -- - 88 -- - 89 -- - 90 -- - 91 -- - 92 -- - 93 -- - 94 -- - 95 -- - 96 -- - 97 -- - 98 -- - 99 -- - 100 -- -		Total Depth 80 feet Water at 65 feet Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 7.5'	ML	87.7	27.7	31.6
B2 @ 7.5'	SC	92.1	23.2	28.3



SHEAR TEST DIAGRAM (ASTM D3080)



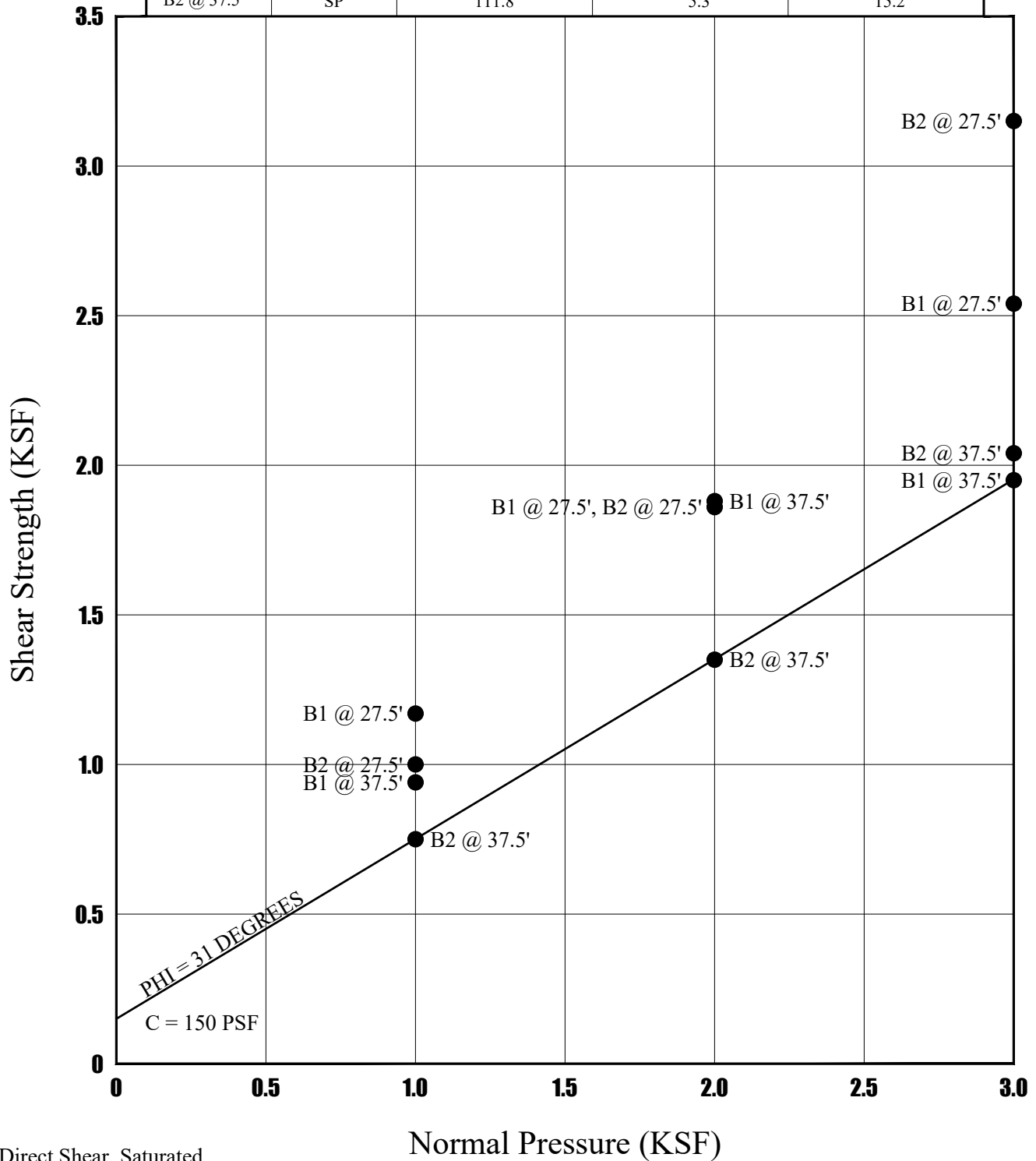
Geotechnologies, Inc.
Consulting Geotechnical Engineers

ROM INVESTMENTS

FILE NO: 22403

PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 27.5'	SM	122.7	12.5	14.5
B2 @ 27.5'	SM	115.3	9.1	16.0
B1 @ 37.5'	CL-ML	116.0	17.9	23.6
B2 @ 37.5'	SP	111.8	5.3	15.2



SHEAR TEST DIAGRAM (ASTM D3080)

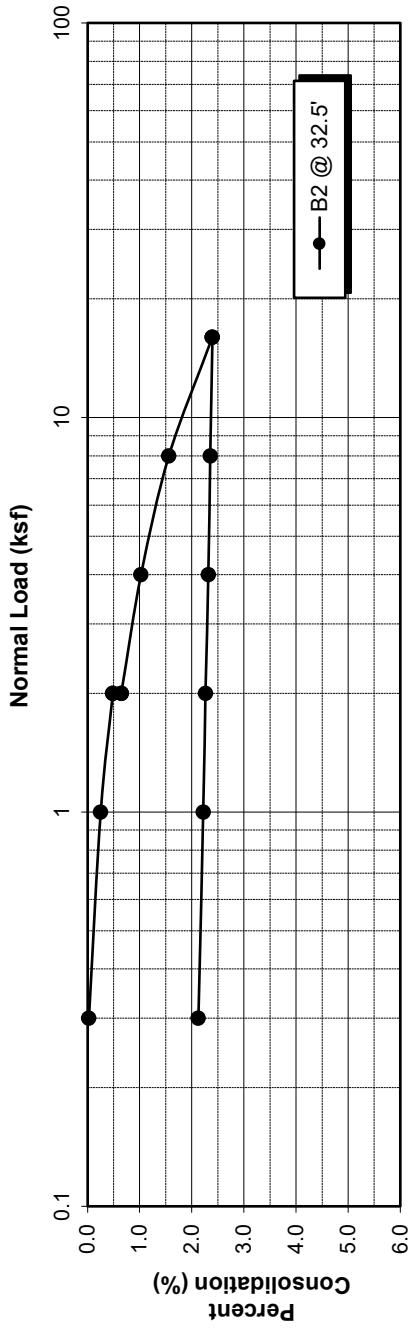
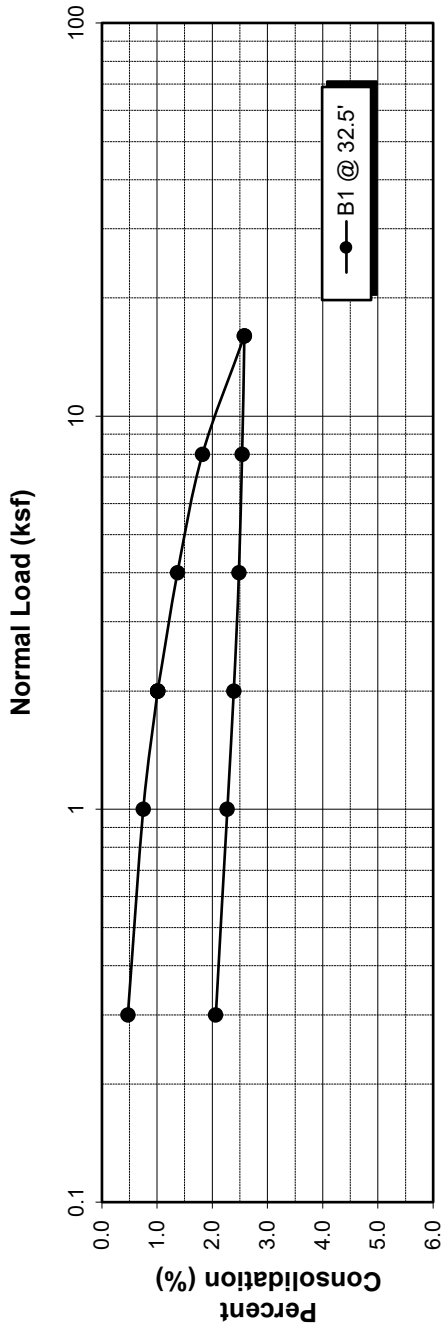
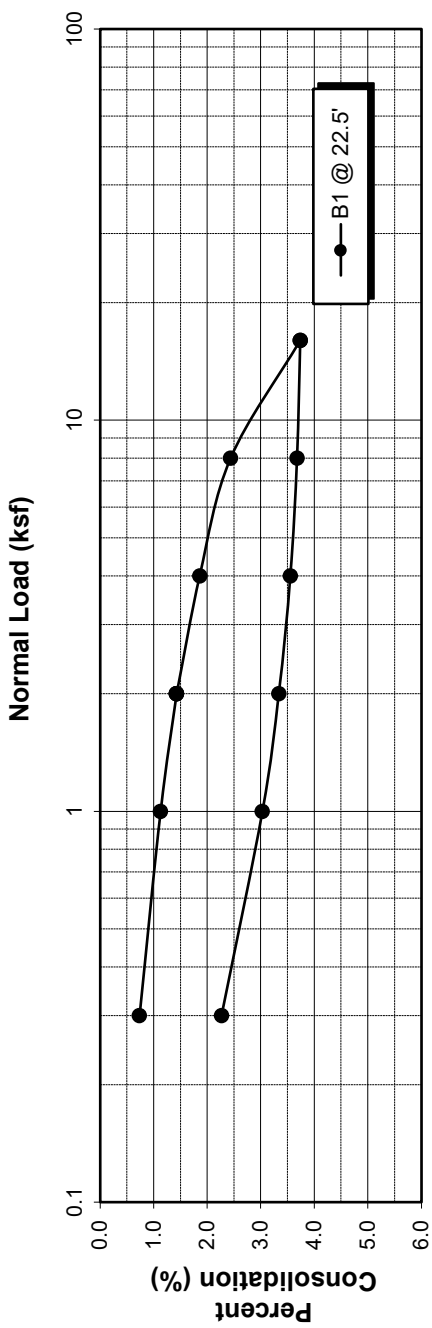


Geotechnologies, Inc.
Consulting Geotechnical Engineers

ROM INVESTMENTS

FILE NO: 22403

PLATE: B-2



Water added at 2 KSF



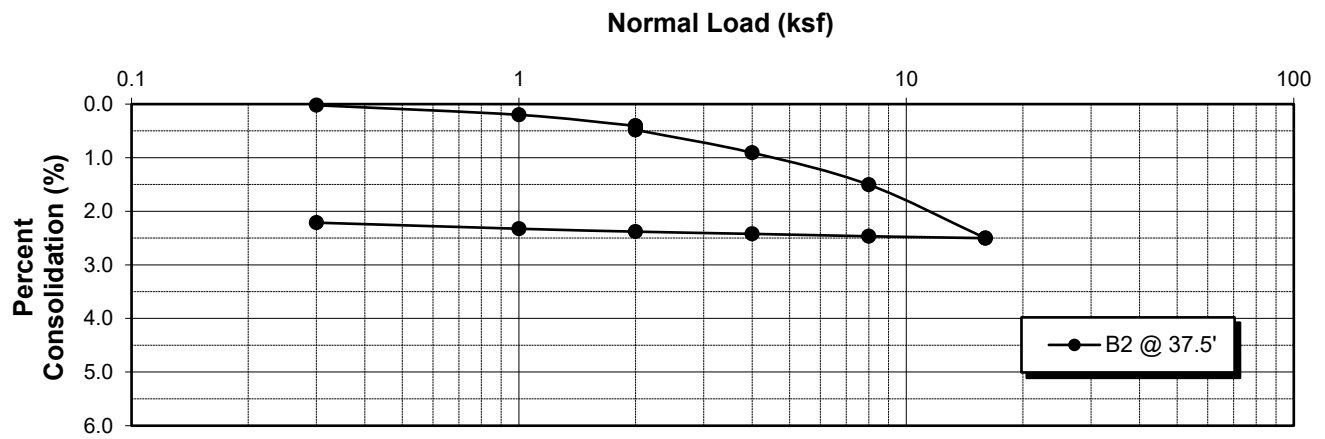
CONSOLIDATION (ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: ROM INVESTMENTS, INC.

FILE NO.: 22403

PLATE: C-1



Water added at 2 KSF



CONSOLIDATION (ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: ROM INVESTMENTS, INC.

FILE NO.: 22403

PLATE: C-2

LABORATORY COMPACTION CHARACTERISTICS (ASTM D1557)	
SAMPLE	B1 @ 1'-5'
SOIL TYPE	ML
MAXIMUM DENSITY PCF.	127.3
OPTIMUM MOISTURE %	10.4

EXPANSION INDEX (ASTM D4829)	
SAMPLE	B1 @ 1'-5'
SOIL TYPE	ML
EXPANSION INDEX UBC STANDARD 18-2	50
EXPANSION CHARACTER	<u>MODERATE</u>

SULFATE CONTENT (CALIFORNIA TEST 417)	
SAMPLE	B1 @ 1'-5'
SULFATE CONTENT: (Percentage by Weight)	<0.1%

COMPACTION/EXPANSION/SULFATE DATA SHEET

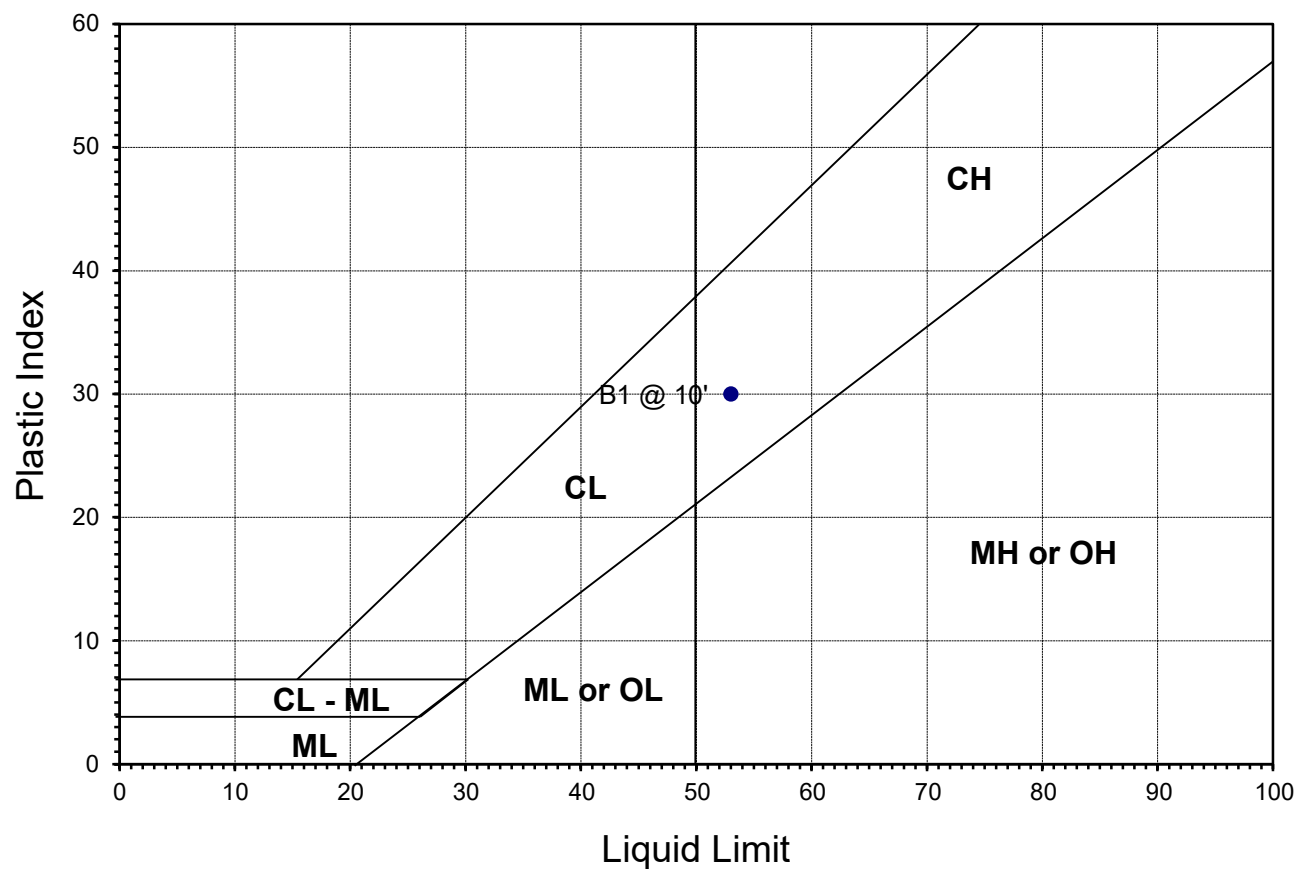


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ROM INVESTMENTS, INC.

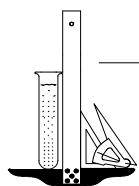
FILE NO: 22403

PLATE: D



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B2 @ 5'	SC	40.0			
B1 @ 10'	CH	70.2	53.0	23.0	30.0
B2 @ 55'	SC	40.2			

ATTERBERG LIMITS (ASTM D4318)



Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: ROM INVESTMENTS, INC.

FILE NO.: 22403

PLATE: E



Project: **ROM**
File No.: **22403**
Description: **Liquefaction Analysis**
Boring No: **B2**

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

Current Groundwater Level (ft.):		65.0
Historically Highest Groundwater Level* (ft.):		40.0
Unit Weight of Water (pcf):		62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

LIQUEFACTION EVALUATION (Idris & Boulanger, EERI NO 12)

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):		8
SPT Sampler with Room for Limer (V/N):		Y
LIQUEFACTION BOUNDARY:		18
Plastic Index Cut Off (PI):		1
Minimum Liquefaction FS:		

Depth to Base Layer (feet)	Weight Unit (pcf)	Water Level (feet)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount (feet)	Depth of SPT (feet)	Fines Content (%)	Fines Content (%)	Vertical Stress σ _v (psf)	Vertical Stress σ _v (psf)	Effective Vertical Stress σ _{v'} (psf)	Excess Pore Water Pressure (psf)	Stress Reduction Coeff. r _d	Cyclic Shear Ratio (CSR)	Cyclic Resistance Ratio (CRR)	Factor of Safety (F.S.)	Liquefaction AS (inches)
1	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	107.9	29.5	1.00	0.587	0.587	0.653	0.587	0.00	0.00
2	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	215.8	215.8	1.00	0.587	0.587	0.649	0.587	0.00	0.00
3	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	323.7	323.7	1.00	0.587	0.587	0.644	0.587	0.00	0.00
4	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	431.6	431.6	1.00	0.587	0.587	0.647	0.587	0.00	0.00
5	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	539.5	539.5	1.00	0.587	0.587	0.644	0.587	0.00	0.00
6	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	647.4	647.4	1.00	0.587	0.587	0.647	0.587	0.00	0.00
7	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	755.3	755.3	1.00	0.587	0.587	0.640	0.587	0.00	0.00
8	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	868.9	868.9	1.00	0.587	0.587	0.638	0.587	0.00	0.00
9	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	982.5	982.5	1.00	0.587	0.587	0.635	0.587	0.00	0.00
10	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1096.1	1096.1	1.00	0.587	0.587	0.632	0.587	0.00	0.00
11	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1209.7	1209.7	1.00	0.587	0.587	0.629	0.587	0.00	0.00
12	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1323.3	1323.3	1.00	0.587	0.587	0.626	0.587	0.00	0.00
13	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1430.4	1430.4	1.00	0.587	0.587	0.624	0.587	0.00	0.00
14	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1537.5	1537.5	1.00	0.587	0.587	0.621	0.587	0.00	0.00
15	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1644.6	1644.6	1.00	0.587	0.587	0.618	0.587	0.00	0.00
16	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1751.7	1751.7	1.00	0.587	0.587	0.615	0.587	0.00	0.00
17	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1858.8	1858.8	1.00	0.587	0.587	0.611	0.587	0.00	0.00
18	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	1961.3	1961.3	1.00	0.587	0.587	0.608	0.587	0.00	0.00
19	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2063.8	2063.8	1.00	0.587	0.587	0.605	0.587	0.00	0.00
20	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2166.3	2166.3	1.00	0.587	0.587	0.602	0.587	0.00	0.00
21	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2268.8	2268.8	1.00	0.587	0.587	0.598	0.587	0.00	0.00
22	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2371.3	2371.3	1.00	0.587	0.587	0.595	0.587	0.00	0.00
23	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2473.8	2473.8	1.00	0.587	0.587	0.592	0.587	0.00	0.00
24	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2576.3	2576.3	1.00	0.587	0.587	0.589	0.587	0.00	0.00
25	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2678.8	2678.8	1.00	0.587	0.587	0.586	0.587	0.00	0.00
26	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2781.3	2781.3	1.00	0.587	0.587	0.583	0.587	0.00	0.00
27	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2883.8	2883.8	1.00	0.587	0.587	0.580	0.587	0.00	0.00
28	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	2986.3	2986.3	1.00	0.587	0.587	0.577	0.587	0.00	0.00
29	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3088.8	3088.8	1.00	0.587	0.587	0.574	0.587	0.00	0.00
30	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3191.3	3191.3	1.00	0.587	0.587	0.571	0.587	0.00	0.00
31	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3293.8	3293.8	1.00	0.587	0.587	0.568	0.587	0.00	0.00
32	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3396.3	3396.3	1.00	0.587	0.587	0.565	0.587	0.00	0.00
33	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3498.8	3498.8	1.00	0.587	0.587	0.562	0.587	0.00	0.00
34	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3601.3	3601.3	1.00	0.587	0.587	0.559	0.587	0.00	0.00
35	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3703.8	3703.8	1.00	0.587	0.587	0.556	0.587	0.00	0.00
36	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3806.3	3806.3	1.00	0.587	0.587	0.553	0.587	0.00	0.00
37	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	3908.8	3908.8	1.00	0.587	0.587	0.550	0.587	0.00	0.00
38	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4011.3	4011.3	1.00	0.587	0.587	0.547	0.587	0.00	0.00
39	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4113.8	4113.8	1.00	0.587	0.587	0.544	0.587	0.00	0.00
40	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4216.3	4216.3	1.00	0.587	0.587	0.541	0.587	0.00	0.00
41	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4318.8	4318.8	1.00	0.587	0.587	0.538	0.587	0.00	0.00
42	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4421.3	4421.3	1.00	0.587	0.587	0.535	0.587	0.00	0.00
43	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4523.8	4523.8	1.00	0.587	0.587	0.532	0.587	0.00	0.00
44	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4626.3	4626.3	1.00	0.587	0.587	0.529	0.587	0.00	0.00
45	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4728.8	4728.8	1.00	0.587	0.587	0.526	0.587	0.00	0.00
46	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4831.3	4831.3	1.00	0.587	0.587	0.523	0.587	0.00	0.00
47	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	4933.8	4933.8	1.00	0.587	0.587	0.520	0.587	0.00	0.00
48	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5036.3	5036.3	1.00	0.587	0.587	0.517	0.587	0.00	0.00
49	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5138.8	5138.8	1.00	0.587	0.587	0.514	0.587	0.00	0.00
50	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5241.3	5241.3	1.00	0.587	0.587	0.511	0.587	0.00	0.00
51	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5343.8	5343.8	1.00	0.587	0.587	0.508	0.587	0.00	0.00
52	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5446.3	5446.3	1.00	0.587	0.587	0.505	0.587	0.00	0.00
53	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5548.8	5548.8	1.00	0.587	0.587	0.502	0.587	0.00	0.00
54	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5651.3	5651.3	1.00	0.587	0.587	0.499	0.587	0.00	0.00
55	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5753.8	5753.8	1.00	0.587	0.587	0.496	0.587	0.00	0.00
56	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5856.3	5856.3	1.00	0.587	0.587	0.493	0.587	0.00	0.00
57	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	5958.8	5958.8	1.00	0.587	0.587	0.490	0.587	0.00	0.00
58	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6061.3	6061.3	1.00	0.587	0.587	0.487	0.587	0.00	0.00
59	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6163.8	6163.8	1.00	0.587	0.587	0.484	0.587	0.00	0.00
60	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6266.3	6266.3	1.00	0.587	0.587	0.481	0.587	0.00	0.00
61	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6368.8	6368.8	1.00	0.587	0.587	0.478	0.587	0.00	0.00
62	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6471.3	6471.3	1.00	0.587	0.587	0.475	0.587	0.00	0.00
63	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6573.8	6573.8	1.00	0.587	0.587	0.472	0.587	0.00	0.00
64	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6676.3	6676.3	1.00	0.587	0.587	0.469	0.587	0.00	0.00
65	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6778.8	6778.8	1.00	0.587	0.587	0.466	0.587	0.00	0.00
66	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6881.3	6881.3	1.00	0.587	0.587	0.463	0.587	0.00	0.00
67	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	6983.8	6983.8	1.00	0.587	0.587	0.460	0.587	0.00	0.00
68	107.9	Unsat	Unsat	Unsat	13	5	13	0.0	7086.3	7086.3	1.00	0.587	0.587	0.457	0.587	0.00	0.00
69	107.9	Unsat	Unsat	Unsat													