Attachment N

Geotechnical Investigation

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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

March 29, 2018
File Number 21560

Bastion Development Corporation
11955 West Washington Boulevard
Suite 103
Culver City, California 90066

Attention: Reid Kaufmann

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
12727 and 12753 West Washington Boulevard, Culver City, California;
3984 and 3988 South Meier Street, and 12740 and 12750 West Zanja Street,
Los Angeles, California

Dear Mr. Kaufmann:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/19
CIVIL
STATE OF CALIFORNIA

GREGORIO VARELA
R.C.E. 81201

GV:km

Distribution: (4) Addressee

Email to: [rkaufmann@bastion.ca]

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
12727 AND 12753 WEST WASHINGTON BOULEVARD
CULVER CITY, CALIFORNIA;
3984 AND 3988 SOUTH MEIER STREET
AND 12740 AND 12750 WEST ZANJA STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This investigation included four exploratory borings, 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.

It should be noted that the northern portion of the site is located within the limits of the City of Los Angeles, while the southern portion of the site is located within the limits of the City of Culver City. At this time, it is unknown which of the two jurisdictions will review the proposed project.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The site is proposed to be developed with a mixed-use structure. The structure is proposed to be six stories in height, constructed over one subterranean parking level. The exact depth of the proposed subterranean parking level is not known at this time. However, based on the experience of this firm, it is anticipated that the subterranean level may extend to a depth between 10 and 12 feet below the existing grade. The enclosed Plot Plan shows the anticipated location and alignment of the proposed structure.

Column loads are estimated to be between 400 and 1,000 kips. Wall loads are estimated to be between 5 and 30 kips per lineal foot. These loads reflect the dead plus live load. Grading is expected to consist of excavations on the order of 12 to 16 feet for construction of the proposed subterranean level, including foundation elements.

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

SITE CONDITIONS

The site is triangular in shape, and approximately 1¼ acres in area, delimited by Zanja Street to the north, Washington Boulevard to the south-east, and Meier Street to the west. The site is bisected by the boundary between the City of Los Angeles and the City of Culver City. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

Based on review of the Topographic Survey prepared by Cal Vada Surveying, Inc., dated October 30, 2017, the site grade descends gently to the southwest. A topographic relief on the order of 2 feet is observed across the site. The site is currently developed with two single-story commercial buildings, and an asphalt-paved parking lot.

Vegetation at the site is limited, and consists of mature trees and shrubs, contained in manicured planter islands. Drainage across the site appears to be by sheetflow to the city streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on February 19 and 20, 2018 by drilling four borings. The borings were drilled to depths ranging between 40 and 68 feet below grade, with the aid of a truck-mounted drilling machine 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 through A-4.

The location of exploratory excavations was determined by information furnished from hardscape features shown on the attached Plot Plan. Elevations of the exploratory excavations were obtained by review of the Topographic Survey prepared by Cal Vada Surveying, Inc., dated October 30, 2017. 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 all four exploratory borings, to depths ranging between 2½ and 3 feet below the existing grade. The fill consists of sandy silt and silty clay, which is dark brown, slightly moist, and stiff.

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

Groundwater

Groundwater was encountered in all exploratory borings, to depths ranging between 35 and 37 feet below the existing grade. Based on elevations presented in the Topographic Survey prepared by Cal Vada Surveying, Inc., dated October 30, 2017, the observed groundwater depths correspond to approximate elevations ranging between 1.8 and 3.6 feet.

According to groundwater data provided in the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle, the historically highest groundwater level for the site was on the order of 14 feet below the ground surface (CDMG, 1998, Revised 2006). A copy of the historically highest groundwater map is enclosed herein. Based on an average site elevation of 38.5 feet, it is the opinion of this firm that the historically highest groundwater level for the site corresponds to elevation 24.5 feet.

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

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. 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 active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

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

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

Ground 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 active or potentially active faults underlie the subject site. In addition, the subject 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.

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

Groundwater was encountered during exploration, at a depth of 35 to 37 feet below the ground surface. Based on review of the seismic hazard zone report of the Venice 7½-minute quadrangle (CDMG, 1998, revised 2006), the historic-high groundwater level for the site was 14 feet below the ground surface. Both the historic highest groundwater level and the current groundwater level were utilized for the enclosed liquefaction analysis.

Section 11.8.3 of ASCE 7-10 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the USGS U.S. Seismic Design Maps tool, this corresponds to a PGA_M of 0.67g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008) indicates a PGA of 0.68g (2 percent in 50 years ground motion) and a mean magnitude of 6.8 for the site. The liquefaction potential evaluation was performed by utilizing a magnitude 6.8 earthquake, and a peak horizontal acceleration of 0.68g.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring 2. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. Fines content, as defined by percentage passing the #200 sieve were utilized for the fines correction factor in computing the corrected blow count of selected soil layers. Fine contents results are present in Plate E of this report.

The site-specific liquefaction analysis included in the Appendix, indicates that the site soils would not be prone to liquefaction during the ground motion expected during the design basis earthquake.

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 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.

Review of the City of Los Angeles Inundation and Tsunami Hazard Areas map indicates the site does lie within mapped inundation boundaries for the Stone Canyon, Lower Franklin, and Hollywood Reservoirs. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Fill materials were encountered during exploration to depths ranging between 2½ and 3 feet below the existing site grade. The existing fill materials are unsuitable for support of new foundations and concrete slabs-on-grade. It is however anticipated that the existing fill will be removed during excavation of the proposed subterranean parking level. The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean level.

Groundwater was observed in all four exploratory borings, to depths ranging between 35 and 37 feet below the existing grade. These groundwater levels correspond to elevations ranging between 3.6 and 1.8 feet. The historically highest groundwater level for the site is on the order of 14 feet below grade. Based on the average site elevation observed across the footprint of the proposed structure, it is the opinion of this firm that the historically highest groundwater level for the project may be considered to correspond to elevation 24.5 feet.

Where elements of a proposed development extend below the historically highest groundwater level, the structure should either be designed to resist potential hydrostatic forces, or a permanent dewatering system should be installed so that external water pressure does not develop against the proposed retaining walls and slabs-on-grade. While the exact depth of the proposed subterranean parking level is unknown at this time, it is anticipated that its finished grade would be above elevation 24.5 feet. Recommendations provided herein assume that the finished floor elevation of the lowest subterranean level will be located at or above the historically highest groundwater level (elevation 24.5 feet). Therefore, the proposed subterranean retaining walls may be designed for a drained condition, provided that a subdrain system is installed. In the event that the subterranean level will extend below the historically highest groundwater elevation, please contact this firm so the appropriate recommendations are provided.

The proposed subterranean level will extend adjacent to the property lines. Therefore the excavation for the proposed subterranean level will require temporary shoring in order to provide a stable excavation. Shoring recommendations are provided in the "Excavations" section of this report.

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

SEISMIC DESIGN CONSIDERATIONS

2016 California Building Code Seismic Parameters

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

2016 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.790g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.790g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.193g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.675g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.013g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.675g

EXPANSIVE SOILS

The onsite geologic materials are in the very low to high expansion range. The Expansion Index was found to be between 2 and 102 for representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

WATER-SOLUBLE SULFATES

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

The 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 American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

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 Methane Buffer Zone, as designated by the City.

GRADING GUIDELINES

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

Site Preparation

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

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

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

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

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

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density.

All fill should be mechanically compacted in layers not more than 8 inches thick. Based on the very low to high expansion index of the site soils, it is recommended that fill materials are moisture conditioned to approximately 3 to 5 percent over optimum moisture content before recompaction.

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 30. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Shrinkage

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

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather.

These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

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

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

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

Abandoned Seepage Pits

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

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

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

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.

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

FOUNDATION DESIGN

Conventional

The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean level. Continuous foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

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

The bearing capacity increase for each additional foot of width is 150 pounds per square foot. The bearing capacity increase for each additional foot of depth is 400 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls, trash enclosures or canopies, which will not be rigidly connected to the proposed structure may bear in native soils, or a properly compacted fill pad. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material. No bearing capacity increases are recommended.

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

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces.

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

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

Foundation Settlement

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

Foundation Observations

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

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

RETAINING WALL DESIGN

As mentioned before, the exact depth of the proposed subterranean level is unknown at this time. Based on the experience of this firm, it is anticipated that the finished grade of the proposed subterranean level would extend to a depth between 10 and 12 feet below the existing grade. As a preventive measure, recommendations for the design of retaining walls up to 14 feet in height are provided herein. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

The recommendations provided herein assume that the finished grade for the proposed subterranean level will not extend below the historically highest groundwater level, which was determined to be elevation 24.5 feet. In the event that the finished grade elevation will extend deeper than his elevation, this office shall be contacted so the appropriate recommendations are provided.

Additional pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. Based on review of the enclosed Plot Plan, it is not anticipated that the proposed retaining walls will be surcharged by existing structures. However, vehicular traffic is expected in the vicinity of the proposed structure. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Drained Cantilever Retaining Walls

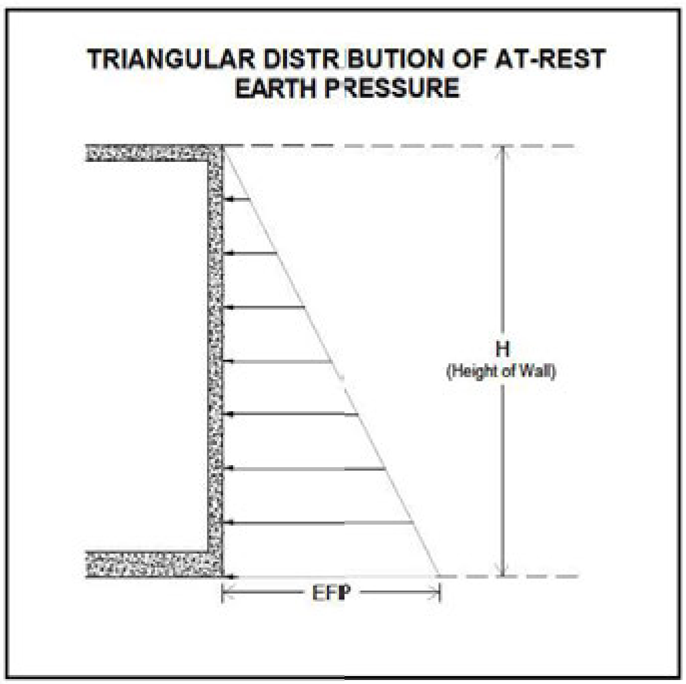
Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed utilizing the following table:

HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 14	45

The highly expansive properties of the on-site soils have been considered in the development of the recommended lateral earth pressures. These lateral earth pressures assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. For the purpose of designing restrained retaining walls up to 14 feet in height, the at-rest pressure would be 61 pounds per cubic foot.



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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 19 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Surcharge from Adjacent Structures

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2014-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing or proposed structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

where:

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

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

Retaining Wall Drainage

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

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

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. According to the Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site was approximately 14 feet below the existing ground surface. Groundwater was encountered during exploration at depths ranging between 35 and 37 feet below the existing site grade during exploration.

It is anticipated that the proposed retaining walls will not extend below the historically highest or the current groundwater levels. Therefore the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices. Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

In the event that the proposed underground retaining walls will extend deeper than anticipated, this office shall be contacted so the appropriate recommendations are provided.

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations up to a depth of 16 feet below the existing grade may be anticipated for construction of the proposed subterranean parking level and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 16 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

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

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

Soldier Pile Installation below Groundwater

Groundwater was encountered during exploration at depths between 35 and 37 feet below the existing site grade. If the proposed soldier beams will extend into the existing water level, caving of the saturated earth materials below the groundwater level may occur during drilling of piles. Casing or polymer drilling fluid will most likely be required during drilling in order to maintain open shafts. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 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 insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

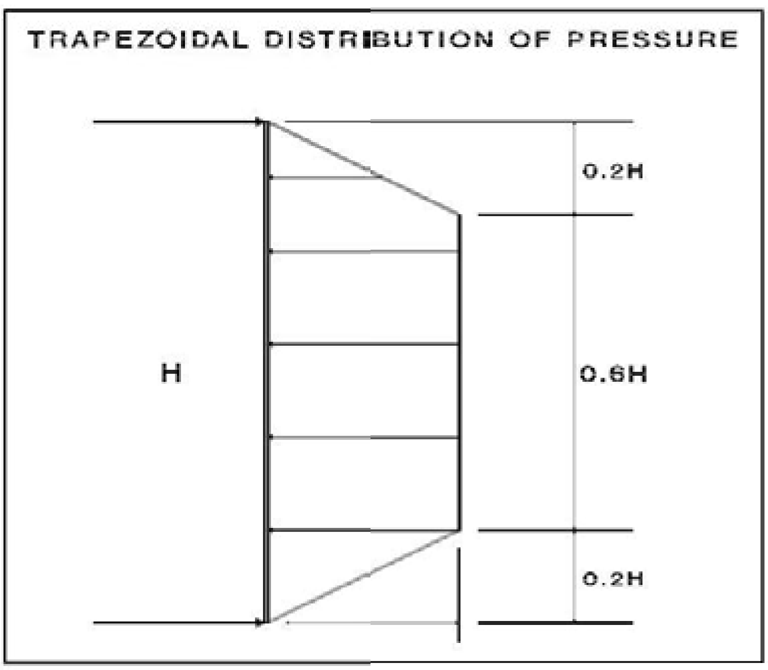
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure, but should be limited to a maximum of 400 pounds per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

Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 16	28

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



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

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 16	18H

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.

Tied-Back Anchors

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

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

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

Anchor Installation

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

Tieback Anchor Testing

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

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

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

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

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

Internal Bracing

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. An allowable bearing pressure of 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 24 inches in width and length as well as 24 inches in depth into native alluvial soils. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

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 recommended that shoring deflection be limited 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. 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.

Monitoring

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

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

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. This assumes that the subterranean slab-on-grade will be built above the historically highest groundwater level (elevation 24.5 feet). Slabs-on-grade should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

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

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

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

Concrete Crack Control

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

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

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

Slab Reinforcing

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

PAVEMENTS

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

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Car Traffic	3	4
Medium Truck Traffic	4	6

Concrete paving may also be utilized for the project. For concrete paving, the following sections are recommended:

Service	Concrete Pavement Thickness Inches	Base Course Inches
Passenger Car and Medium Truck Traffic	6	4

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

For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete pavement should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

Percolation testing of the on-site soils was not conducted as part of this investigation. It is anticipated that the proposed structure will extend adjacent to the property lines, which would not allow for the required horizontal offset distance between shallow infiltration systems and structures or property lines. Based on the anticipated depth of the proposed structure, the current groundwater level, and the required minimum vertical offset of 10 feet between the current groundwater level and the bottom of infiltration systems, the disposal of stormwater by means of a deep drywell system would saturate the soils located within the primary zone of foundation influence. Saturation of these soils is not recommended because it would result in a change in their engineering properties.

Based on the above considerations, it is the opinion of this firm that the disposal of stormwater by infiltration into the onsite soils is not suitable for the proposed project.

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

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

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. 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. Results are presented on Plate D of this report.

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

California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2006), Seismic Hazard Zone Report of the Venice 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 036, map scale 1:24,000.

California Department of Conservation, Division of Mines and Geology, 1999, Seismic Hazard Zones Map, Venice 7½-minute Quadrangle, CDMG Seismic Hazard Zone Mapping Act of 1990.

Fain, Hsai-Yang, ed., 1991, Foundation Engineering Handbook, Second Edition: New York, Van Nostrand Reinhold, pp. 630-631.

Ishihara, K. 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, Volume 1, P. 321-376, August.

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

O'Rourke, T.D., Pease, J.W. (1997), Mapping Liquefiable Layer Thickness for Seismic Hazard Assessment, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 123, no. 1, pp. 46-56.

Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

Stewart, J.P., Blake, T.F., and Hollingsworth, R.A., 2003, a screen analysis procedure of seismic slope stability: Earthquake Spectra, v. 19, n. 3, p. 697-712.

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

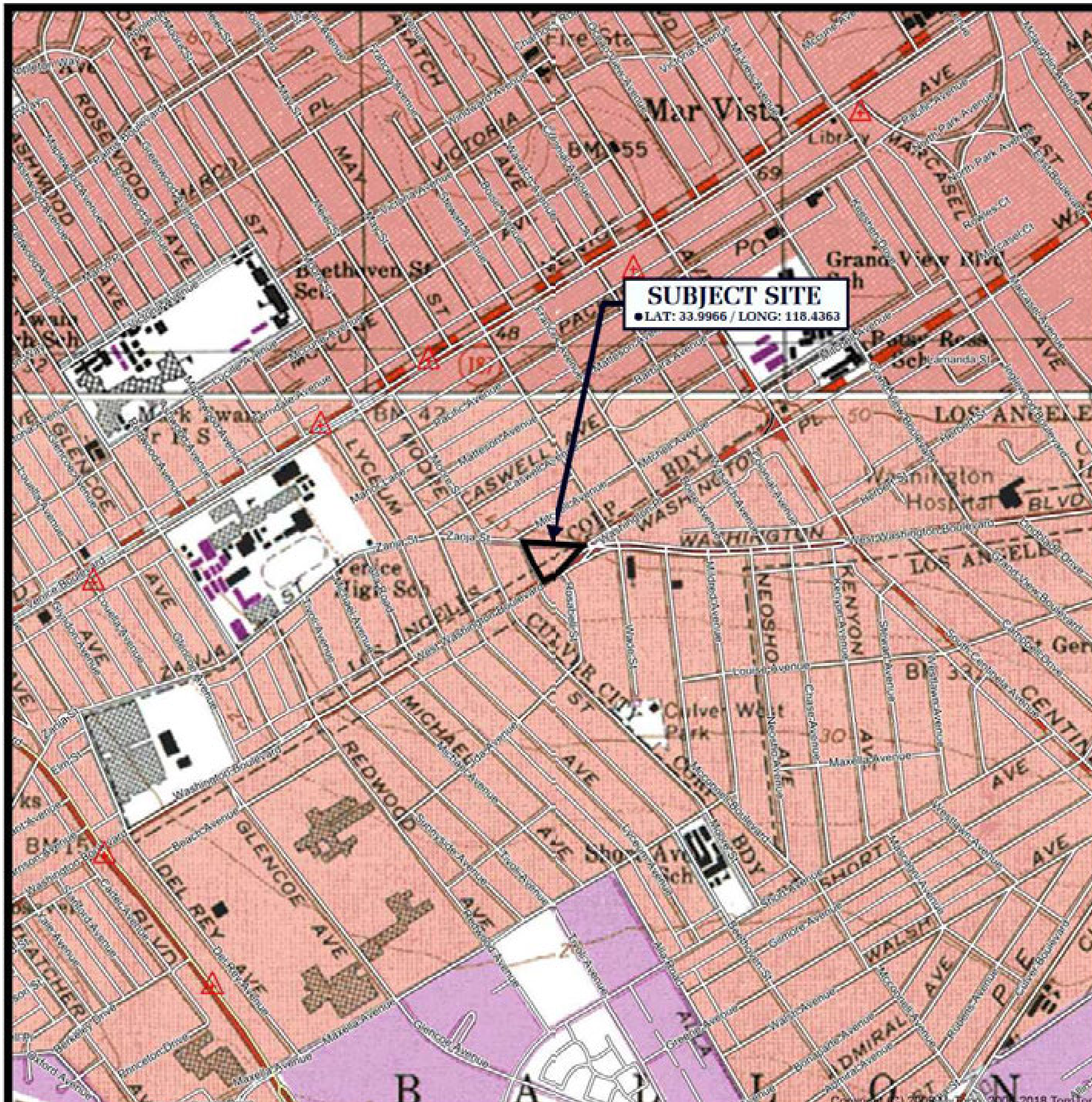
Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.

Tokimatsu, K. and Seed, H. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.

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

United States Geological Survey, 2014, U.S.G.S. U.S. Seismic Design Maps tool http://earthquake.usgs.gov/designmaps/us/application.php.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., Vedder, J.G., Geology of the Los Angeles Basin, Southern California-An Introduction, U.S. Geological Professional Paper 420-A.



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

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

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION

FILE NO. 21560

LEGEND

B4 LOCATION & NUMBER OF BORING

LIMITS OF PROPOSED SUBTERRANEAN PARKING GARAGE

REFERENCE: BASEMENT PARKING PLAN BY PK ARCHITECTURE NOT DATED

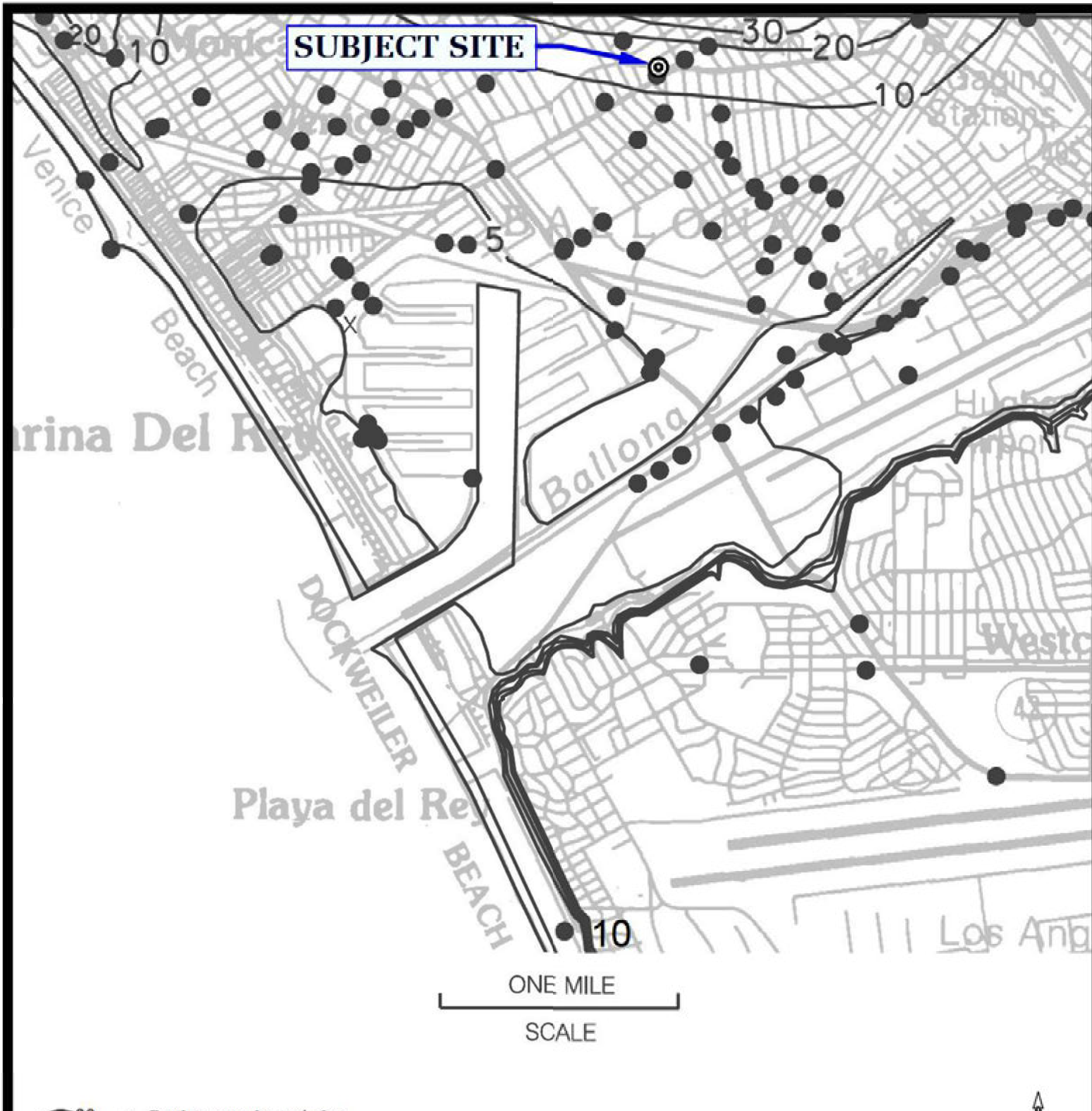
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PLOT PLAN

BASTION DEVELOPMENT CORPORATION

File No.: 21560

Date: March '18



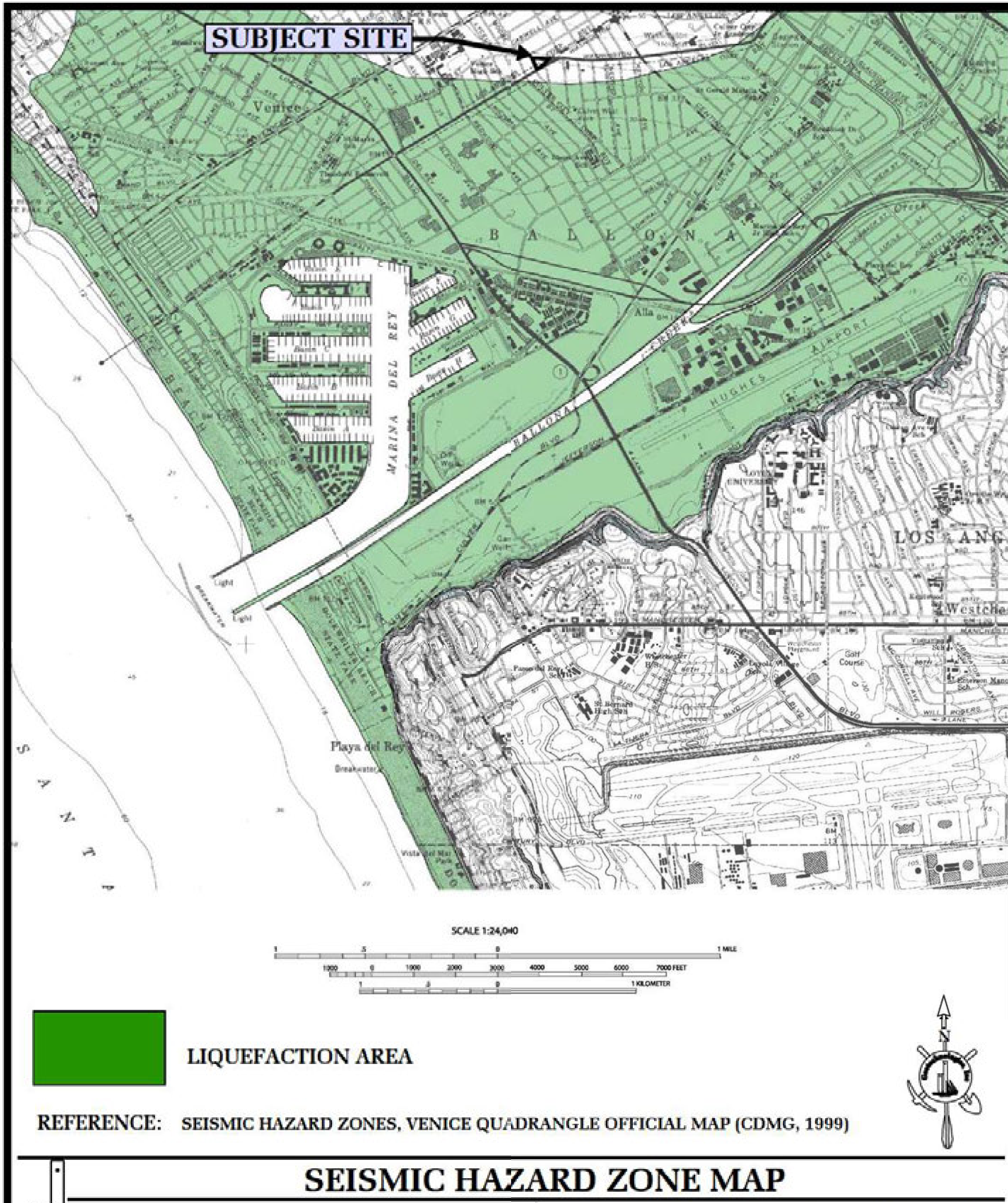
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 036
VENICE 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION

FILE NO. 21560



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, VENICE QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION

FILE NO. 21560

BORING LOG NUMBER 1

Bastion Development Corp. **Date: 02/19/18** **Elevation: 37.5'***

File No. 21560 **Method: 8-inch diameter Hollow Stem Auger**

km/ae

*Reference: Topographic Survey by Calvada Surveying, Inc., dated 10/30/2017

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class	Description
						Surface Conditions: Asphalt
				0		4-inch Asphalt over 7-inch Base
				1		
				2		FILL: Sandy Clay, dark brown, slightly moist, stiff
2.5	18	15.9	113.5			
				3	CL	NATIVE SOILS: Silty Clay, dark brown, slightly moist, stiff
				4		
5	22	17.8	109.3	5		
					CL/ML	Silty Clay to Clayey Silt, dark brown, slightly moist, stiff
				6		
				7		
				8		
				9		
10	17	5.9	108.7	10		
					ML/SM	Clayey Silt to Silty Sand, dark brown, slightly moist, stiff or medium dense, fine grained
				11		
				12		
				13		
				14		
15	21	5.5	125.4	15		
					SM	Silty Sand, dark brown, slightly moist, medium dense, fine grained
				16		
				17		
				18		
				19		
20	42	7.5	123.0	20		
					SW	Gravelly Sand, dark brown to gray, slightly moist, medium dense, fine to coarse grained
				21		
				22		
				23		
				24		
25	53 50/6"	6.7	117.7	25		

BORING LOG NUMBER 1

Bastion Development Corp.

File No. 21560

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	100/7"	5.5	117.4	30		
				31	SW/CL	Gravelly Sand to Silty Clay, orange gray brown, wet, very dense or very stiff, fine to coarse grained
				32		
				33		
				34		
35	100/6"	10.0	123.9	35		
				36	SM	Silty Sand, dark gray brown, wet, very dense, fine grained, gravel
				37		
				38		
				39	ML	Silt, light brown with gray, slightly moist to moist, very stiff
40	54	26.0	98.9	40		
				41		Total Depth 40 feet Water at 35 feet No Caving Fill to 2½ feet
				42		
				43		
				44		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				45		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 2

Bastion Development Corp. **Date: 02/19/18** **Elevation: 38.6'***

File No. 21560 **Method: 8-inch diameter Hollow Stem Auger**

km/ae *Reference: Topographic Survey by Calvada Surveying, Inc., dated 10/30/2017

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		4-inch Asphalt over 7-inch Base
				1		FILL: Sandy Silt with Clay, dark brown, slightly moist, stiff
				2		
				3	ML	NATIVE SOILS: Clayey Silt, dark brown, slightly moist, stiff
				4		
5	7	16.9	SPT	5		
				6		
				7		
7.5	14	15.1	112.1			
				8	CL	Silty Clay, medium brown to very dark brown, slightly moist, stiff
				9		
10	12	7.2	SPT	10	ML	Sandy Silt, dark brown, slightly moist, stiff
				11		
				12		
12.5	26	13.9	121.6			
				13	ML/SM	Sandy Silt to Silty Sand with Gravel, dark brown, slightly moist, stiff to dense, slate fragments
				14		
15	19	7.5	SPT	15		
				16		
				17		
17.5	30 50/6"	4.5	122.8			
				18	SW	Gravelly Sand with Silt, dark brown, slightly moist, very dense, fine to medium grained
				19		
20	20	5.5	SPT	20		
				21		
				22		
22.5	100/8"	3.3	121.3			
				23	SW/CL	Gravelly Sand with Silty Clay, very dark brown, slightly moist, very dense, fine to medium grained
				24		
25	62	5.4	SPT	25		
					SW	Gravelly Sand, orange brown, slightly moist, very dense, fine to medium grained

BORING LOG NUMBER 2

Bastion Development Corp.

File No. 21560

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	100/7"	7.3	116.0			
				28 --	SW/CL	Gravelly Sand with Silty Clay, orange gray brown, slightly moist, very dense, very stiff, fine to medium grained
				29 --		
30	28 50/6"	7.1	SPT	30 --	SP	Sand, dark olive brown, slightly moist, very dense, fine grained
				31 --		
				32 --		
32.5	100/9"	4.3	120.7			
				33 --		gravel
				34 --		
35	34 50/6"	7.2	SPT	35 --	SW	Gravelly Sand, orange gray brown, wet, very dense, fine to medium grained
				36 --		
				37 --		
37.5	38 50/6"	6.3	130.5			
				38 --		
				39 --		
40	48	9.9	SPT	40 --	SP	Sand with Gravel, medium gray brown, wet to saturated, dense, fine grained
				41 --		
				42 --		
42.5	40 50/5"	59.1	57.4	43 --	CL	Silty Clay, dark gray to black, wet, very stiff
				44 --		
45	25	46.1	SPT	45 --		
				46 --		
				47 --		
47.5	100/10"	26.8	97.6			
				48 --	SP/SM	Sand to Silty Sand, medium gray, saturated, very dense, coarse grained
				49 --		
50	49	28.2	SPT	50 --		
					ML	Silt, dark gray, very moist, very stiff

BORING LOG NUMBER 2

Bastion Development Corp.

File No. 21560

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
52.5	48	34.9	85.0			
				53		
				54		
55	29	38.3	SPT	55		
				56		
				57		
57.5	72	32.7	91.1			
				58	CL	Silty Clay, dark gray, slightly moist to moist, stiff
				59		
60	26	28.8	SPT	60		
				61		
				62		
62.5	40 50/5"	23.0	98.9			
				63	SP	Sand, dark gray to dark tan brown, wet, very dense, fine grained
				64		
65	51	21.2	SPT	65		
				66		
				67		
67.5	40 50/4"	18.4	104.3			
				68		
						Total Depth 68 feet Water at 35 feet No Caving Fill to 3 feet
				69		
				70		
				71		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				72		
				73		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				74		
				75		SPT=Standard Penetration Test

BORING LOG NUMBER 3

Bastion Development Corp. — **Date: 02/20/18** — **Elevation: 38.9'***

File No. 21560 — **Method: 8-inch diameter Hollow Stem Auger**

km/ae — *Reference: Topographic Survey by Calvada Surveying, Inc., dated 10/30/2017

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class	Description Surface Conditions: Asphalt
				0		3-inch Asphalt, No Base
				1		FILL: Sandy Silt, dark brown, slightly moist, stiff
				2		
2.5	54	9.3	111.8			
				3	ML	NATIVE SOILS: Sandy Silt, medium brown, slightly moist, very stiff
				4		
5	56	7.8	114.1	5		
				6		
				7		
				8		
				9		
10	49	2.2	118.8	10	SM	Silty Sand, medium brown, slightly moist, dense, fine grained, gravel
				11		
				12		
				13		
				14		
15	40	3.8	121.9	15	SW	Gravelly Sand, dark brown, slightly moist, dense, fine to coarse grained
				16		
				17		
				18		
				19		
20	71	4.1	121.2	20		
				21		
				22		
				23		
				24		
25	100/8"	8.9	124.8	25	SW/SC	Gravelly Sand to Clayey Sand, dark gray brown, slightly moist, very dense, fine to coarse grained

BORING LOG NUMBER 3

Bastion Development Corp.

File No. 21560

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	100/8"	4.0	124.8	30		
				31	SP	Sand with Gravel, dark orange brown, slightly moist, very dense, fine grained
				32		
				33		
				34		
35	100/7"	2.5	112.7	35		
				36	SW	Gravelly Sand, olive brown, slightly moist, very dense, fine to coarse grained
				37		
				38		
				39		
40	39 50/5"	7.5	127.4	40		
				41		Total Depth 40 feet Water at 37 feet No Caving Fill to 3 feet
				42		
				43		
				44		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				45		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 4

Bastion Development Corp. **Date: 02/20/18** **Elevation: 38.6'***

File No. 21560 **Method: 8-inch diameter Hollow Stem Auger**

km/ae

*Reference: Topographic Survey by Calvada Surveying, Inc., dated 10/30/2017

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class	Description
						Surface Conditions: Asphalt
				0		2-inch Asphalt, No Base
				1		FILL: Sandy Silt, dark brown, slightly moist, very stiff
				2		
2.5	60	13.4	124.5			
				3	ML	NATIVE SOILS: Sandy Silt, medium brown, slightly moist, stiff
				4		
5	31	12.5	111.8	5		
				6		
				7		
7.5	35	11.6	121.1			
				8	SM	Silty Sand, medium brown, slightly moist, medium dense, fine grained, gravel
				9		
10	45	4.3	112.3	10	SW	Gravelly Sand, medium brown, slightly moist, medium dense, fine to coarse grained
				11		
				12		
				13		
				14		
15	64	2.8	120.5	15		
				16		
				17		
				18		
				19		
20	39	3.5	116.4	20		
				21		
				22		
				23		
				24		
25	28 50/6"	3.3	120.3	25		very dense

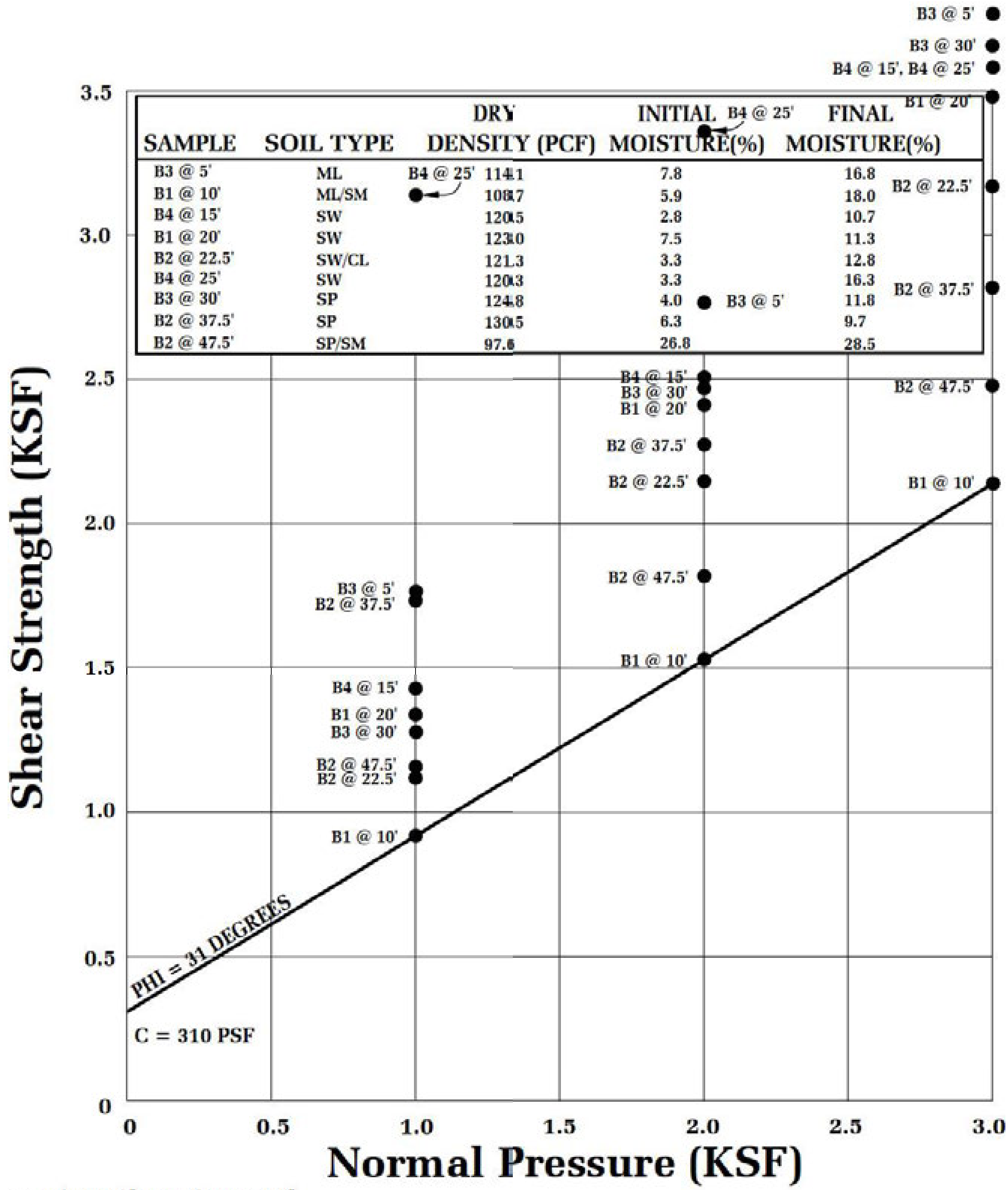
BORING LOG NUMBER 4

Bastion Development Corp.

File No. 21560

km/ae

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class	Description
				26		
				27		
				28		
				29		
30	100/7"	5.5	112.0	30		
				31	SW/ML	Gravelly Sand with Clayey Silt, orange gray brown, slightly moist, very dense or stiff, fine to coarse grained
				32		
				33		
				34		
35	100/8"	8.1	113.7	35		
				36	SM/SP	Silty Sand to Sand with Gravel, dark olive brown, moist, very dense, fine grained
				37		
				38		
				39	SW	Gravelly Sand, dark olive brown, saturated, very dense, coarse grained
40	100/8"	11.0	120.8	40		
				41		Total Depth 40 feet Water at 37 feet No Caving Fill to 3 feet
				42		
				43		
				44		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				45		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46		
				47		
				48		
				49		
				50		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B3 @ 5'	ML	114.1	7.8	16.8
B1 @ 10'	ML/SM	108.7	5.9	18.0
B4 @ 15'	SW	120.5	2.8	10.7
B1 @ 20'	SW	123.0	7.5	11.3
B2 @ 22.5'	SW/CL	121.3	3.3	12.8
B4 @ 25'	SW	120.3	3.3	16.3
B3 @ 30'	SP	124.8	4.0	11.8
B2 @ 37.5'	SP	130.5	6.3	9.7
B2 @ 47.5'	SP/SM	97.6	26.8	28.5

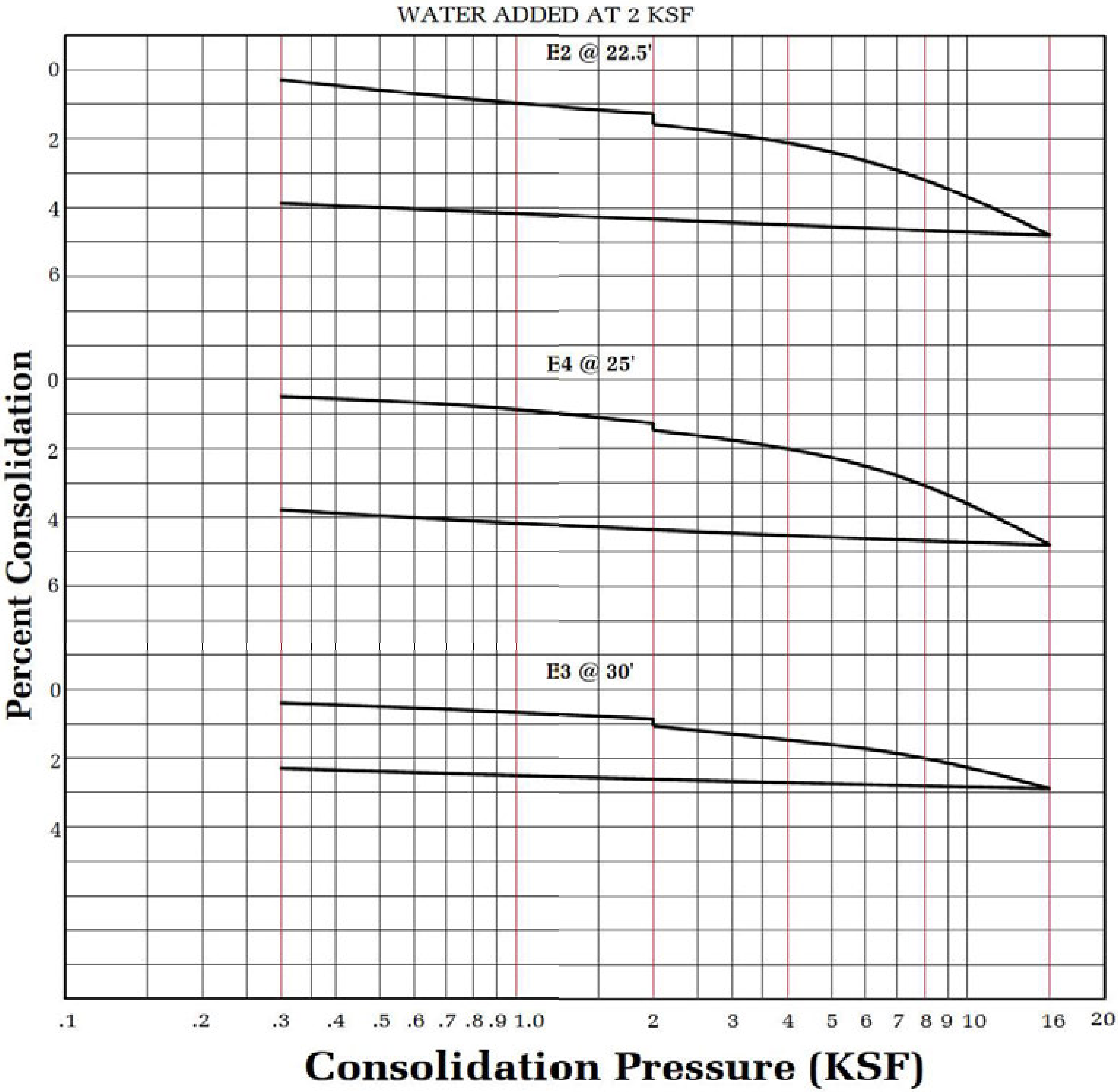
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION

FILE NO. 21560 | PLATE: B

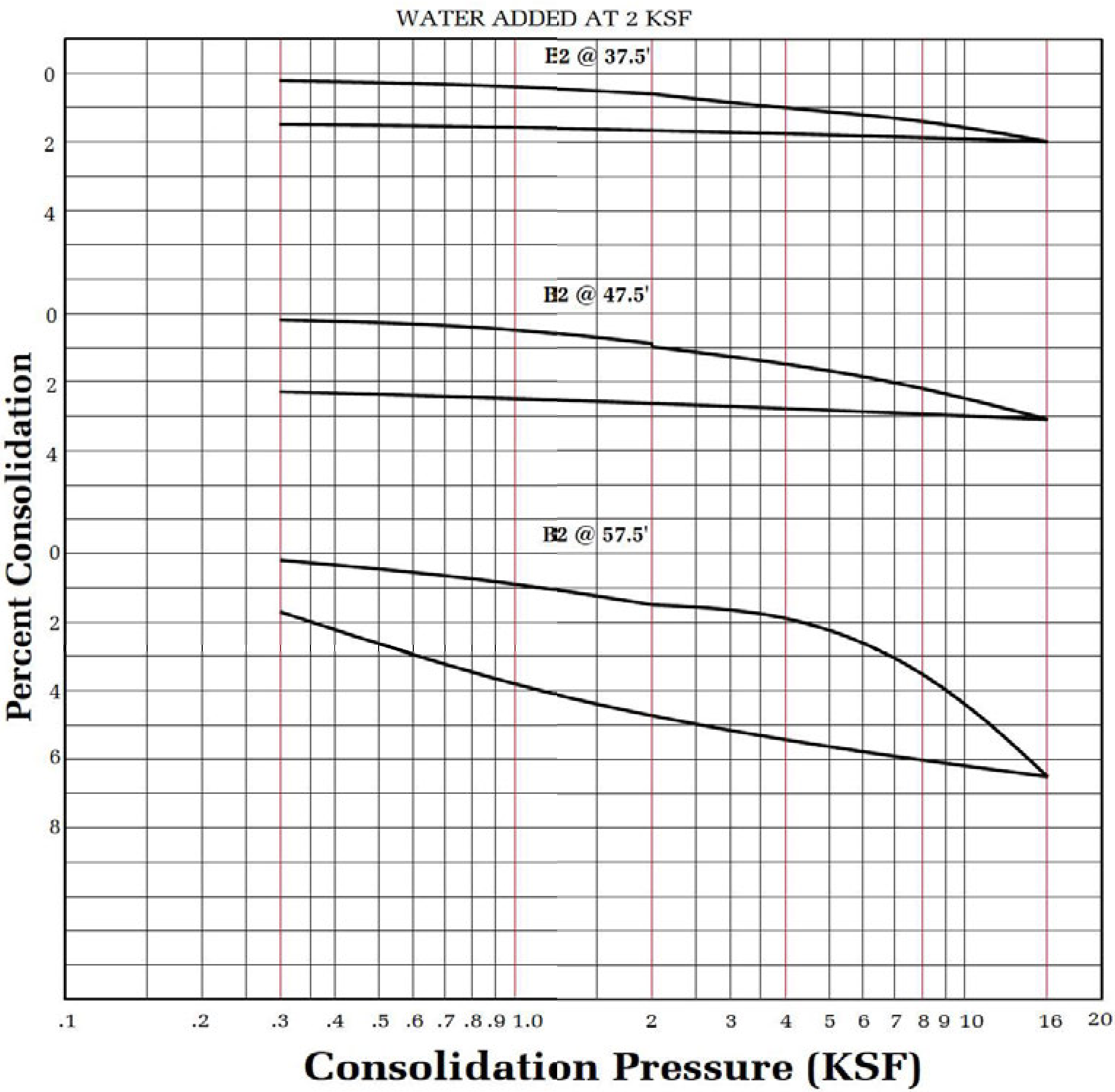


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION

FILE NO. 21560	PLATE: C-1



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION	
FILE NO. 21560	PLATE: C-2

ASTM D-1557

SAMPLE	B1 @ 1-5'	B4 @ 1-5'
SOIL TYPE:	CL	ML
MAXIMUM DENSITY pcf.	128.1	126.5
OPTIMUM MOISTURE %	9.6	11.0

ASTM D 4829-03

SAMPLE	B1 @ 1- 5'	B4 @ 1-5'	B3 @ 10'	B3 @ 20'
SOIL TYPE:	CL	ML	SM	SW
EXPANSION INDEX UBC STANDARD 18-2	54	102	2	3
EXPANSION CHARACTER	MODERATE	HIGH	VERY LOW	VERY LOW

SULFATE CONTENT

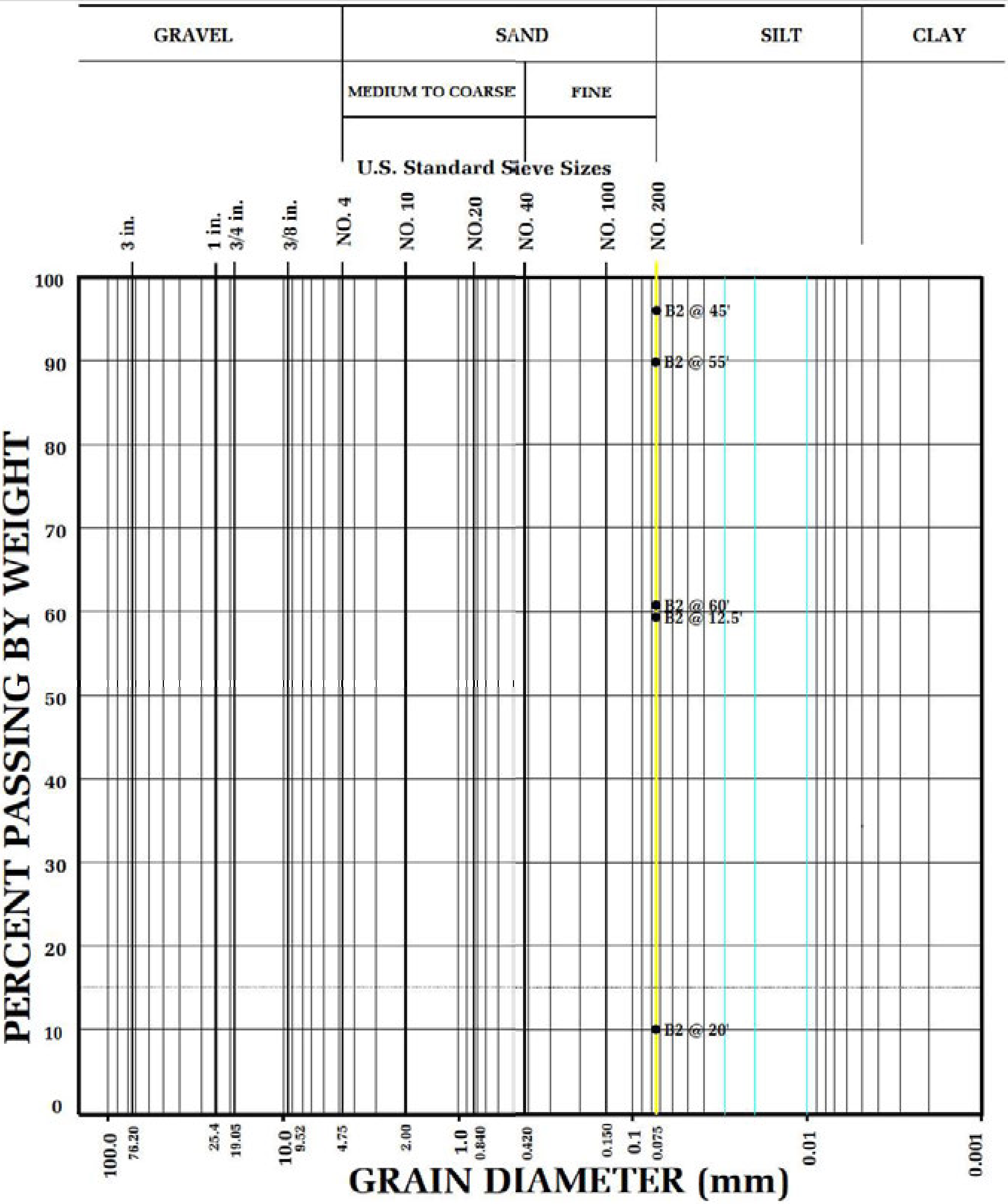
SAMPLE	B1 @ 1-5'	B4 @ 1-5'	B3 @ 10'	B2 @ 12.5'	B2 @ 17.5'	B3 @ 20'
SULFATE CONTENT: (percentage by weight)	< 0.1 %	< 0.1 %	< 0.1 %	< 0.1 %	< 0.1 %	< 0.1 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT COPORATION

FILE NO. 21560 | PLATE: D



GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BASTION DEVELOPMENT CORPORATION	
FILE NO. 21560	PLATE: E

Geotechnologies, Inc.

Project: Bastion Development Corporation
File No.: 21560
Description: Liquefaction Analysis
Boring Numbe 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.8
Peak Ground Horizontal Acceleration, PGA (g):	0.68
Calculated Mag.Wtg.Factor:	1.203

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	35.0
Historically Highest Groundwater Level* (ft):	14.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

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

LIQUEFACTION BOUNDARY:

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

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vo} (psf)	Effective Vert. Stress σ_{vo}' (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlement ΔS (inches)
1	129.0	Unsaturated	Unsaturated	7	5	0.0	0	129.0	129.0	14.5	1.00	0.444	0.201	Non-Liq.	0.00
2	129.0	Unsaturated	Unsaturated	7	5	0.0	0	258.0	258.0	14.5	1.00	0.442	0.201	Non-Liq.	0.00
3	129.0	Unsaturated	Unsaturated	7	5	0.0	0	387.0	387.0	14.5	1.00	0.441	0.201	Non-Liq.	0.00
4	129.0	Unsaturated	Unsaturated	7	5	0.0	0	516.0	516.0	14.5	0.99	0.439	0.201	Non-Liq.	0.00
5	129.0	Unsaturated	Unsaturated	7	5	0.0	0	645.0	645.0	15.6	0.99	0.438	0.213	Non-Liq.	0.00
6	129.0	Unsaturated	Unsaturated	7	5	0.0	0	774.0	774.0	15.6	0.99	0.436	0.213	Non-Liq.	0.00
7	129.0	Unsaturated	Unsaturated	7	5	0.0	0	903.0	903.0	14.6	0.98	0.435	0.200	Non-Liq.	0.00
8	129.0	Unsaturated	Unsaturated	7	5	0.0	0	1032.0	1032.0	13.4	0.98	0.433	0.185	Non-Liq.	0.00
9	129.0	Unsaturated	Unsaturated	7	5	0.0	0	1161.0	1161.0	13.3	0.97	0.431	0.182	Non-Liq.	0.00
10	129.0	Unsaturated	Unsaturated	7	5	0.0	0	1290.0	1290.0	12.5	0.97	0.429	0.172	Non-Liq.	0.00
11	138.5	Unsaturated	Unsaturated	12	10	0.0	0	1428.5	1428.5	20.7	0.97	0.427	0.272	Non-Liq.	0.00
12	138.5	Unsaturated	Unsaturated	12	10	0.0	0	1567.0	1567.0	19.7	0.96	0.425	0.253	Non-Liq.	0.00
13	138.5	Unsaturated	Unsaturated	19	15	59.3	0	1705.5	1705.5	37.1	0.96	0.423	2.000	Non-Liq.	0.00
14	138.5	Unsaturated	Unsaturated	19	15	59.3	0	1844.0	1844.0	36.0	0.95	0.421	1.729	Non-Liq.	0.00
15	138.5	Unsaturated	Saturated	19	15	59.3	0	1982.5	1920.1	39.7	0.95	0.432	2.000	4.6	0.00
16	138.5	Unsaturated	Saturated	19	15	0.0	0	2121.0	1996.2	33.5	0.94	0.443	1.006	2.3	0.00
17	138.5	Unsaturated	Saturated	19	15	0.0	0	2259.5	2072.3	32.9	0.94	0.452	0.906	2.0	0.00
18	128.4	Unsaturated	Saturated	19	15	0.0	0	2387.9	2138.3	32.5	0.93	0.460	0.835	1.8	0.00
19	128.4	Unsaturated	Saturated	19	15	0.0	0	2516.3	2204.3	32.1	0.93	0.468	0.774	1.7	0.00
20	128.4	Unsaturated	Saturated	19	15	0.0	0	2644.7	2270.3	31.6	0.92	0.475	0.722	1.5	0.00
21	128.4	Unsaturated	Saturated	20	20	10.0	0	2773.1	2336.3	34.4	0.92	0.481	1.159	2.4	0.00
22	128.4	Unsaturated	Saturated	20	20	10.0	0	2901.5	2402.3	34.0	0.91	0.487	1.067	2.2	0.00
23	125.4	Unsaturated	Saturated	20	20	10.0	0	3026.9	2465.3	33.7	0.91	0.492	0.992	2.0	0.00
24	125.4	Unsaturated	Saturated	20	20	10.0	0	3152.3	2528.3	33.3	0.90	0.496	0.926	1.9	0.00
25	125.4	Unsaturated	Saturated	20	20	10.0	0	3277.7	2591.3	33.0	0.89	0.500	0.868	1.7	0.00
26	125.4	Unsaturated	Saturated	62	25	0.0	0	3403.1	2654.3	103.6	0.89	0.504	2.000	4.0	0.00
27	125.4	Unsaturated	Saturated	62	25	0.0	0	3528.5	2717.3	103.0	0.88	0.507	2.000	3.9	0.00
28	124.4	Unsaturated	Saturated	62	25	0.0	0	3652.9	2779.3	107.8	0.88	0.510	2.000	3.9	0.00
29	124.4	Unsaturated	Saturated	62	25	0.0	0	3777.3	2841.3	107.1	0.87	0.512	2.000	3.9	0.00
30	124.4	Unsaturated	Saturated	62	25	0.0	0	3901.7	2903.3	106.5	0.87	0.514	2.000	3.9	0.00
31	125.8	Unsaturated	Saturated	88	30	0.0	0	4027.5	2966.7	150.3	0.86	0.516	2.000	3.9	0.00
32	125.8	Unsaturated	Saturated	88	30	0.0	0	4153.3	3030.1	149.5	0.85	0.518	2.000	3.9	0.00
33	125.8	Unsaturated	Saturated	88	30	0.0	0	4279.1	3093.5	148.7	0.85	0.519	2.000	3.9	0.00
34	125.8	Unsaturated	Saturated	88	30	0.0	0	4404.9	3156.9	147.9	0.84	0.520	2.000	3.8	0.00
35	125.8	Unsaturated	Saturated	88	30	0.0	0	4530.7	3220.3	147.1	0.84	0.520	2.000	3.8	0.00
36	138.7	Saturated	Saturated	84	35	0.0	0	4669.4	3296.6	139.6	0.83	0.520	2.000	3.8	0.00
37	138.7	Saturated	Saturated	84	35	0.0	0	4808.1	3372.9	138.7	0.82	0.519	2.000	3.8	0.00
38	138.7	Saturated	Saturated	84	35	0.0	0	4946.8	3449.2	137.9	0.82	0.519	2.000	3.9	0.00
39	138.7	Saturated	Saturated	84	35	0.0	0	5085.5	3525.5	137.1	0.81	0.518	2.000	3.9	0.00
40	138.7	Saturated	Saturated	84	35	0.0	0	5224.2	3601.8	136.4	0.81	0.517	2.000	3.9	0.00
41	138.7	Saturated	Saturated	48	40	0.0	0	5362.9	3678.1	77.5	0.80	0.516	2.000	3.9	0.00
42	138.7	Saturated	Saturated	48	40	0.0	0	5501.6	3754.4	77.1	0.79	0.515	1.996	3.9	0.00
43	91.3	Saturated	Saturated	25	45	96.0	0	5592.9	3783.3	45.1	0.79	0.515	1.990	3.9	0.00
44	91.3	Saturated	Saturated	25	45	96.0	0	5684.2	3812.2	45.0	0.78	0.516	1.985	3.8	0.00
45	91.3	Saturated	Saturated	25	45	96.0	0	5775.5	3841.1	44.9	0.78	0.516	1.980	3.8	0.00
46	123.8	Saturated	Saturated	25	45	96.0	0	5899.3	3902.5	44.7	0.77	0.515	1.968	3.8	0.00
47	123.8	Saturated	Saturated	25	45	96.0	0	6023.1	3963.9	44.4	0.76	0.514	1.957	3.8	0.00
48	123.8	Saturated	Saturated	25	45	96.0	0	6146.9	4025.3	44.2	0.76	0.512	1.946	3.8	0.00
49	123.8	Saturated	Saturated	25	45	96.0	0	6270.7	4086.7	44.0	0.75	0.511	1.936	3.8	0.00
50	114.6	Saturated	Saturated	25	45	96.0	0	6385.3	4138.9	43.8	0.75	0.510	1.927	3.8	0.00
51	114.6	Saturated	Saturated	49	50	0.0	0	6499.9	4191.1	78.4	0.74	0.508	1.918	3.8	0.00
52	114.6	Saturated	Saturated	49	50	0.0	0	6614.5	4243.3	78.2	0.74	0.507	1.909	3.8	0.00
53	114.6	Saturated	Saturated	49	50	0.0	0	6729.1	4295.5	75.9	0.73	0.506	1.900	3.8	0.00
54	114.6	Saturated	Saturated	49	50	0.0	0	6843.7	4347.7	75.7	0.72	0.504	1.892	3.8	0.00
55	114.6	Saturated	Saturated	49	50	0.0	0	6958.3	4399.9	75.5	0.72	0.503	1.883	3.7	0.00
56	114.6	Saturated	Saturated	29	55	89.8	0	7072.9	4452.1	50.0	0.71	0.501	1.875	3.7	0.00
57	114.6	Saturated	Saturated	29	55	89.8	0	7187.5	4504.3	49.9	0.71	0.499	1.867	3.7	0.00
58	120.9	Saturated	Saturated	29	55	89.8	0	7308.4	4562.8	49.8	0.70	0.497	1.858	3.7	0.00
59	120.9	Saturated	Saturated	29	55	89.8	0	7429.3	4621.3	49.6	0.70	0.495	1.848	3.7	0.00
60	120.9	Saturated	Saturated	29	55	89.8	0	7550.2	4679.8	49.5	0.69	0.493	1.840	3.7	0.00
61	120.9	Saturated	Saturated	26	60	60.8	0	7671.1	4738.3	44.0	0.69	0.491	1.831	3.7	0.00
62	120.9	Saturated	Saturated	26	60	60.8	0	7792.0	4796.8	43.9	0.68	0.489	1.822	3.7	0.00
63	121.7	Saturated	Saturated	26	60	60.8	0	7913.7	4856.1	43.7	0.68	0.487	1.813	3.7	0.00
64	121.7	Saturated	Saturated	26	60	60.8	0	8035.4	4915.4	43.5	0.67	0.485	1.805	3.7	0.00
65	121.7	Saturated	Saturated	26	60	60.8	0	8157.1	4974.7	43.4	0.67	0.483	1.796	3.7	0.00
66	121.7	Saturated	Saturated	51	70	0.0	0	8278.8	5034.0	75.8	0.66	0.481	1.788	3.7	0.00
67	121.7	Saturated	Saturated	51	70	0.0	0	8400.5	5093.3	75.6	0.66	0.479	1.779	3.7	0.00
68	123.5	Saturated	Saturated	51	70	0.0	0	8524.0	5154.4	75.3	0.65	0.477	1.771	3.7	0.00

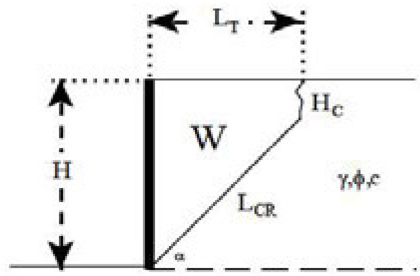
Geotechnologies, Inc.

Project: Bastion Development
File No.: 21560
Description: Retaining Wall up to 14 feet High

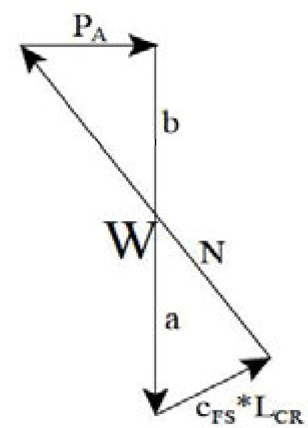
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	14.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	31.0 degrees
Cohesion of Retained Soils	(c)	310.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.8 degrees
	(c_{FS})	206.7 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.4	92	11524.4	11.8	7250.2	4274.2	1402.8
41	6.2	91	11334.6	11.9	6952.3	4382.2	1523.5
42	6.0	89	11114.9	12.0	6660.7	4454.1	1636.2
43	5.8	87	10873.4	12.0	6378.7	4494.7	1740.7
44	5.7	85	10616.3	12.0	6108.1	4508.2	1837.0
45	5.5	83	10348.1	12.0	5849.9	4498.2	1925.2
46	5.4	81	10072.3	12.0	5604.3	4468.0	2005.2
47	5.3	78	9791.6	11.9	5371.3	4420.3	2077.2
48	5.2	76	9507.9	11.8	5150.6	4357.3	2141.3
49	5.1	74	9222.8	11.8	4941.6	4281.2	2197.4
50	5.1	71	8937.5	11.7	4743.8	4193.7	2245.8
51	5.0	69	8652.7	11.6	4556.5	4096.3	2286.5
52	5.0	67	8369.3	11.5	4379.1	3990.3	2319.6
53	4.9	65	8087.6	11.4	4210.8	3876.8	2345.1
54	4.9	62	7808.1	11.2	4051.1	3756.9	2363.1
55	4.9	60	7530.8	11.1	3899.3	3631.5	2373.7
56	4.9	58	7256.0	11.0	3754.8	3501.2	2376.8
57	4.9	56	6983.8	10.9	3617.0	3366.8	2372.4
58	4.9	54	6714.1	10.7	3485.3	3228.9	2360.6
59	4.9	52	6447.0	10.6	3359.1	3087.9	2341.3
60	5.0	49	6182.3	10.4	3238.0	2944.4	2314.5
61	5.0	47	5920.0	10.3	3121.3	2798.7	2280.1
62	5.1	45	5659.9	10.1	3008.6	2651.3	2238.1
63	5.1	43	5401.8	9.9	2899.4	2502.4	2188.4
64	5.2	41	5145.7	9.8	2793.1	2352.5	2130.9
65	5.3	39	4891.1	9.6	2689.3	2201.9	2065.5



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\ max}$ 2376.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$
EFP 24.3 pcf

Design Wall for an Equivalent Fluid Pressure: **45 pcf** **(Based on a High Espanion Potential)**

Geotechnologies, Inc.

Project: Bastion Development
File No.: 21560

Soil Weight	γ	125 pcf
Internal Friction Angle	ϕ	31 degrees
Cohesion	c	0 psf
Height of Retaining Wall	H	14 feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

	$K_o = 1 - \sin\phi$	0.485
	$\sigma'_v = \gamma H$	1750.0 psf

$\sigma'_h =$	848.7 psf	
EFP =	60.6 pcf	
$P_o =$	5940.8 lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 61 pcf

Geotechnologies, Inc.

Project: Bastion Development

File No.: 21560

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	14.0	feet
Retained Soil Unit Weight:	(γ)	125.0	pcf
Horizontal Ground Acceleration:	(k_h)	0.22	g

Seismic Increment (ΔP_{AE}):

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

$\Delta P_{AE} =$ 2021.3 lbs/ft

Force applied at 0.6H above the base of the wall

Transfer load to 2/3 of the height of the wall

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

T = 1819.1 lbs/ft

$EFP = 2*T/H^2$

EFP = 19 pcf

triangular distribution of pressure

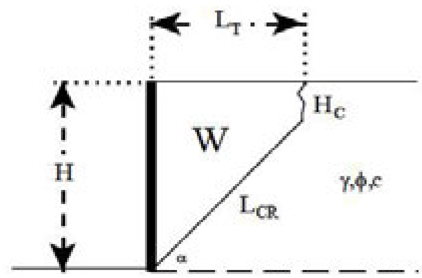
Geotechnologies, Inc.

Project: Bastion Development
File No.: 21560
Description: Temporary Shoring Wall up to 16 feet High

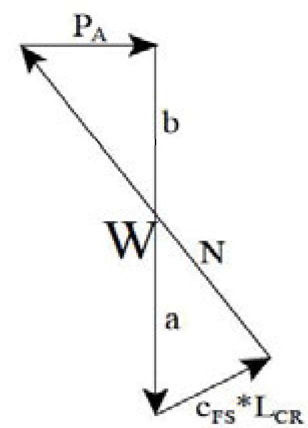
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	16.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	31.0 degrees
Cohesion of Retained Soils	(c)	310.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	25.7 degrees
	(c_{FS})	248.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	9.4	100	12440.2	10.2	9228.1	3212.1	820.4
41	9.0	101	12629.2	10.7	9069.4	3559.8	975.6
42	8.6	101	12684.4	11.1	8841.5	3843.0	1125.7
43	8.2	101	12640.9	11.4	8573.1	4067.8	1269.1
44	7.9	100	12523.6	11.7	8283.1	4240.5	1404.6
45	7.6	99	12351.1	11.8	7983.9	4367.2	1531.7
46	7.4	97	12136.8	11.9	7683.5	4453.3	1649.7
47	7.2	95	11891.2	12.0	7387.2	4503.9	1758.4
48	7.0	93	11621.7	12.1	7098.3	4523.4	1857.7
49	6.9	91	11334.5	12.1	6818.8	4515.8	1947.3
50	6.8	88	11034.0	12.1	6549.7	4484.3	2027.3
51	6.6	86	10723.7	12.0	6291.6	4432.0	2097.6
52	6.5	83	10406.3	12.0	6044.7	4361.6	2158.2
53	6.5	81	10083.9	11.9	5808.6	4275.3	2209.2
54	6.4	78	9758.2	11.9	5583.1	4175.1	2250.6
55	6.4	75	9430.3	11.8	5367.7	4062.7	2282.4
56	6.3	73	9101.4	11.7	5161.8	3939.6	2304.6
57	6.3	70	8772.0	11.5	4964.7	3807.3	2317.3
58	6.3	68	8442.8	11.4	4775.9	3666.9	2320.5
59	6.3	65	8114.2	11.3	4594.7	3519.5	2314.2
60	6.3	62	7786.3	11.2	4420.4	3365.9	2298.4
61	6.4	60	7459.4	11.0	4252.3	3207.1	2273.0
62	6.4	57	7133.6	10.8	4089.8	3043.8	2238.1
63	6.5	54	6808.7	10.7	3932.1	2876.7	2193.6
64	6.6	52	6484.9	10.5	3778.5	2706.4	2139.4
65	6.7	49	6161.9	10.3	3628.3	2533.5	2075.7



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\,max}$ 2320.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

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

EFP 18.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf