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Consulting Geotechnical Engineers

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December 19, 2018
File Number 21704

Frontier Holdings West, LLC;
Regal Group, LLC;
Main Fund Associates, LLC;
Main Street Tower
888 South Figueroa Street, Suite 1900
Los Angeles, California 90017

Attention: Daniel Taban

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Mixed-Use Tower
1123 through 1161 South Main Street, Los Angeles, California

Dear Mr. Taban:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides preliminary geotechnical recommendations for the development of the site, including earthwork, seismic design, excavations, retaining walls, shoring and foundation design.

This report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should prepare a comprehensive geotechnical engineering investigation, suitable for submission to the building official. Supplemental subsurface exploration and laboratory testing, as well as the re-evaluation of the design parameters provided herein, will be required in order to prepare a comprehensive geotechnical engineering investigation. Engineering for the proposed project should not begin until approval of the comprehensive 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
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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE TOWER
1123 THROUGH 1161 SOUTH MAIN STEET
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the preliminary 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 report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should prepare a comprehensive geotechnical engineering investigation, suitable for submission to the building official. Supplemental subsurface exploration and laboratory testing, as well as the re-evaluation of the design parameters provided herein, will be required in order to prepare a comprehensive geotechnical engineering investigation.

This investigation included excavation of two 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.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. In addition, the Concept Design drawings, prepared by MVE and Partners, dated September 13, 2018, were reviewed for the preparation of this report. The proposed project consists of a 30-story mixed-use tower. The four lower levels of the structure will consist of a podium, occupied by parking and retail space. The tower will be located in the central area of the podium, and will be occupied by residential space. The proposed structure will be built at-grade. The alignment of the proposed structure is shown in the enclosed Plot Plan.

Structural information for the proposed structure is not available at this time. Grading is expected to consist of removal and recompaction of existing unsuitable soils. Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The site is located at 1123 through 1161 South Main Street, in the Downtown area of the City of Los Angeles, California. The subject site is rectangular in shape, and just over one acre in area. The site is bounded by a parking lot to the north, Main Street to the east, 12th Street to the south, and a city alleyway to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site is relatively level, with no pronounced highs or lows. The site is currently developed with four single-story commercial buildings and a paved parking lot. Vegetation at the site is non-existent. Drainage across the site appears to be by sheetflow to the city streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on November 6, 2018 by drilling two borings. The borings were drilled to a depth of 30 and 60 feet, respectively, 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 and A-2.

The location of exploratory borings was determined by hardscaped features shown in the enclosed Plot Plan. The location of the exploratory borings should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in the exploratory borings to a depth of 3 feet below the existing grade. The fill consists of silty sands, which are dark brown in color, moist, medium dense, and fine grained.

The fill is in turn underlain by native alluvial soils, consisting of interlayered mixtures of sands and silty sands. The native alluvial soils are yellowish brown, olive brown and dark brown in color, moist, medium dense to very dense, and fine to coarse grained, with cobbles and gravel. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during exploration, conducted to a maximum depth of 60 feet below the existing site grade. The historically highest groundwater level was established by review of the Hollywood 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 2006). Review of this plate indicates that the historically highest groundwater level at the site was on the order of 115 feet below grade. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map.

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 will most likely experience caving.

CITY OF LOS ANGELES METHANE ZONE

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is located within the limits of a City of Los Angeles Methane Zone. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane Zone designation.

OIL WELLS

Based on review of the California State Division of Oil, Gas and Geothermal Resources (DOGGR) On-line Mapping System, the site is located within the limits of the Los Angeles Downtown Oil Field. A copy of this map has been enclosed in the Appendix as the Oil Field and Oil Well Map.

Review of the DOGGR On-line Mapping System also indicates that no oil or gas wells were drilled within the subject site. As shown on the enclosed Oil Field and Oil Well Map, the closest wells were drilled approximately 700 feet to the southwest of the site.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject site is located in the Los Angeles Basin of 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.

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)	2.288g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.288g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.525g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.804g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.207g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.804g

Deaggregated Seismic Source Parameters

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). The results are based on a 2 percent in 50 years ground motion (2,475 year return period). A shear wave velocity of 259 meters per second was utilized for Vs30. The deaggregation program indicates a PGA of 0.79g and a modal magnitude of 6.5 for the site.

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.

Review of the California Seismic Hazards Zones Map for the Hollywood Quadrangle (CDMG 1999), indicates that the subject site is not located within a “Liquefiable” area. This determination is based on groundwater records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map has been enclosed to this report.

Groundwater was not encountered during exploration, conducted to a maximum depth of 60 feet below the existing site grade. The historically highest groundwater level for the site is reported to be on the order of 115 feet below grade. Based on the density of the soils underlying the site, and the mapped depth to the historically highest groundwater level, the soils underlying the site are not considered capable of liquefaction during the ground motion expected during the design-based 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, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within the potential mapped inundation boundaries of the Hansen and Sepulveda Reservoirs, should the dam retaining these reservoirs fail during a seismic event. 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 preliminary 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.

This report is preliminary in nature since the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should be contacted so the design parameters provided herein are revised and/or re-evaluated, and a comprehensive geotechnical engineering investigation suitable for submission to the building official is prepared. Supplemental subsurface exploration and analyses will be required in order to prepare a comprehensive geotechnical engineering investigation.

During exploration, fill materials were observed to extend to a depth of 3 feet below the existing grade. The existing fill materials are unsuitable for support of new foundations and concrete slabs-on-grade, but they may be re-used for the preparation of compacted fill subgrades.

It is recommended that the tower portion of the structure is supported on a mat foundation bearing in undisturbed native alluvial soils. The portion of the podium extending beyond the footprint of the tower may be supported by conventional foundations bearing in a properly compacted fill pad. For the preparation of a compacted fill pad, all existing fill materials and upper alluvial soils shall be removed and recompacted to a minimum depth of 3 feet below the bottom of the proposed foundations. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundation, whichever is greater.

It is expected that temporary grading and foundation excavations will extend immediately adjacent to the property lines. These temporary excavations will require the use of slot cuts or shoring, to provide a stable excavation. Slot cut and shoring recommendations are provided in the Temporary 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.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to low expansion range. The Expansion Index was found to be 7 and 35 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.

GRADING GUIDELINES

Site Preparation

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

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

Recommended Overexcavation

For the portions of the podium extending beyond the footprint of the tower, where conventional foundations will bear on a compacted fill pad, all existing fill and upper native alluvial soils shall be excavated to a minimum depth of 3 feet below the bottom of the proposed foundations. In addition, the excavation shall extend horizontally at least 3 feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundations, whichever is greater.

Over-excavation will not be required within the footprint of the proposed tower, since the mat foundation supporting this portion of the structure will bear in undisturbed native soils.

Compaction

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

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 95 percent compaction is obtained.

Acceptable 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. Cobbles should be expected within the materials to be reused as controlled fill. Where cobbles are encountered, the size of the cobbles shall be limited to a maximum of 6 inches in dimension.

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 40. 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 95 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 95 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 two 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.

LEED Considerations

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

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

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

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

FOUNDATION DESIGN

It is recommended that the tower portion of the structure is supported on a mat foundation bearing in undisturbed native alluvial soils. The portion of the podium extending beyond the footprint of the tower may be supported by conventional foundations, bearing in a properly compacted fill pad.

Mat Foundation for Tower

The mat foundation shall bear in undisturbed native alluvial soils. For design purposes, an average bearing pressure of up to 5,000 pounds per square foot, with locally higher pressures up to 10,000 pounds per square foot may be utilized in the mat foundation design.

The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

where K = Reduced Subgrade Modulus
K1 = Unit Subgrade Modulus
B = Foundation Width (feet)

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

<u>Conventional Foundations for Podium</u>

Continuous foundations may be designed for a bearing capacity of 3,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

Column foundations may be designed for a bearing capacity of 4,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

The bearing capacity increase for each additional foot of width is 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

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

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed hotel structure, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. 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 0.35 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 1,800 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

It is anticipated that total settlement on the order of 2½ inches will occur below the more heavily loaded central core portions of the mat foundation beneath the tower. Settlement on the edges of the mat foundation is not expected to exceed 1¼ inch.

The maximum settlement of a typical column footing below the podium portion of the structure is expected to be on the order of ¾-inch.

Differential settlement between the podium column footings and the edges of the tower mat foundation is expected to be on the order of ½ inch. Differential settlement between columns for the podium is not expected to exceed ½ inch.

Foundation Observations

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

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

RETAINING WALL DESIGN

The proposed structure will be built at-grade. Therefore the only retaining walls anticipated would be associated with the construction of elevator pits, or planters.

Cantilever Retaining Walls

Miscellaneous cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 30 pounds per cubic foot for walls retaining up to 5 feet of earth.

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

Dynamic (Seismic) Earth Pressure

Based on the 2016 California Building Code, retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. Miscellaneous retaining walls anticipated for the proposed project are not expected to exceed 5 feet in height. Therefore the dynamic earth pressure may be omitted.

Retaining Wall Drainage

Retaining walls should be provided with a subdrain covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The onsite geologic materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 95 percent of the maximum density as determined by the most recent revision of ASTM D 1557.

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. Subdrainage pipes should outlet to an acceptable location.

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.

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.

Waterproofing

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

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 95 percent of the maximum density 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

Based on the maximum depth of fill observed during exploration, temporary excavations on the order of 5 feet in height are anticipated during grading and conventional foundation excavation. Depending on the final thickness of the mat foundation, deeper excavations may be required during construction of the mat.

The on-site fill and native soils are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic, structures or property lines. Surcharged and unsurcharged vertical excavations may be performed to a maximum height of 7 feet with the aid of slot-cuts, as recommended in the following section. Temporary shoring will be required for vertical excavations exceeding a height of 7 feet.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 12 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.

Slot Cutting

Based on the alignment of the proposed structure, as shown in the enclosed Plot Plan, it is anticipated that grading and foundation excavations will extend adjacent to the property lines. Where a property line, or vehicular traffic, will surcharge a temporary excavation, the slot cutting method may be utilized to maintain a stable excavation. The slot cutting method may also be utilized for the deepening of foundations. The height of the excavation is limited to 7 feet. The "A-B-C" slot-cutting procedure is recommended.

The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation consists of excavating the "A" slots. Alternate "A" slots of 8 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should be 8 feet in width for a combined intervening length of 16 feet. The "A" slots should be properly backfilled, before the "B" slots are excavated. The height of the slots shall not exceed 8 feet in height. Calculations indicating that slots 8 feet in width will be stable for the maximum recommended height of 7 feet have been included in the appendix of this report.

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.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot, up to a maximum of 3,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.5 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Caving should be expected to occur during drilling in the native granular soils underlaying the site. Where caving occurs, it will be necessary to utilize casing or polymer drilling fluid to maintain open pile 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. Large sized materials should also be anticipated during drilling (i.e. gravels, cobbles, and possibly boulders).

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 is 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 10	28

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.

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. Slabs-on-grade should be cast over properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 95 percent of the maximum dry density.

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

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

Where any dampness would be objectionable it is recommended that floor slabs should be waterproofed. A qualified waterproofing consultant should be engaged in order to recommend a product and/or method which would provide protection from unwanted moisture.

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

Based on ACI 302.2R-30, Chapter 7, for projects with vapor sensitive coverings, a vapor barrier should be provided. Figure 7.1 shows that the slab should be poured on the vapor barrier. Where humidity-controlled areas are proposed and the base materials and slabs will not be within a water-tight system, Figure 7.1 shows that the barrier should be covered with a 4 inch layer of dry granular material. ACI notes that the decision whether to locate the material in direct contact with the slab or beneath a layer of granular fill should be made on a case by case basis. The necessity of a vapor retarder as well as the use of dry granular material, as discussed above, is not a geotechnical issue and should be confirmed by qualified members of the design team.

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

Concrete Crack Control

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

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

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 95 percent 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 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the exposed grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 percent 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 control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-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.

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

Percolation testing was not conducted as part of this preliminary investigation. It is the opinion of this firm that on-site stormwater infiltration is feasible at the site. The use of a deep infiltration system, such a drywell, would be suitable for the project. Percolation testing will be required to provide geotechnical recommendations to aid in the design of a stormwater infiltration system.

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 in 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. Results are presented in Plate D of this report.

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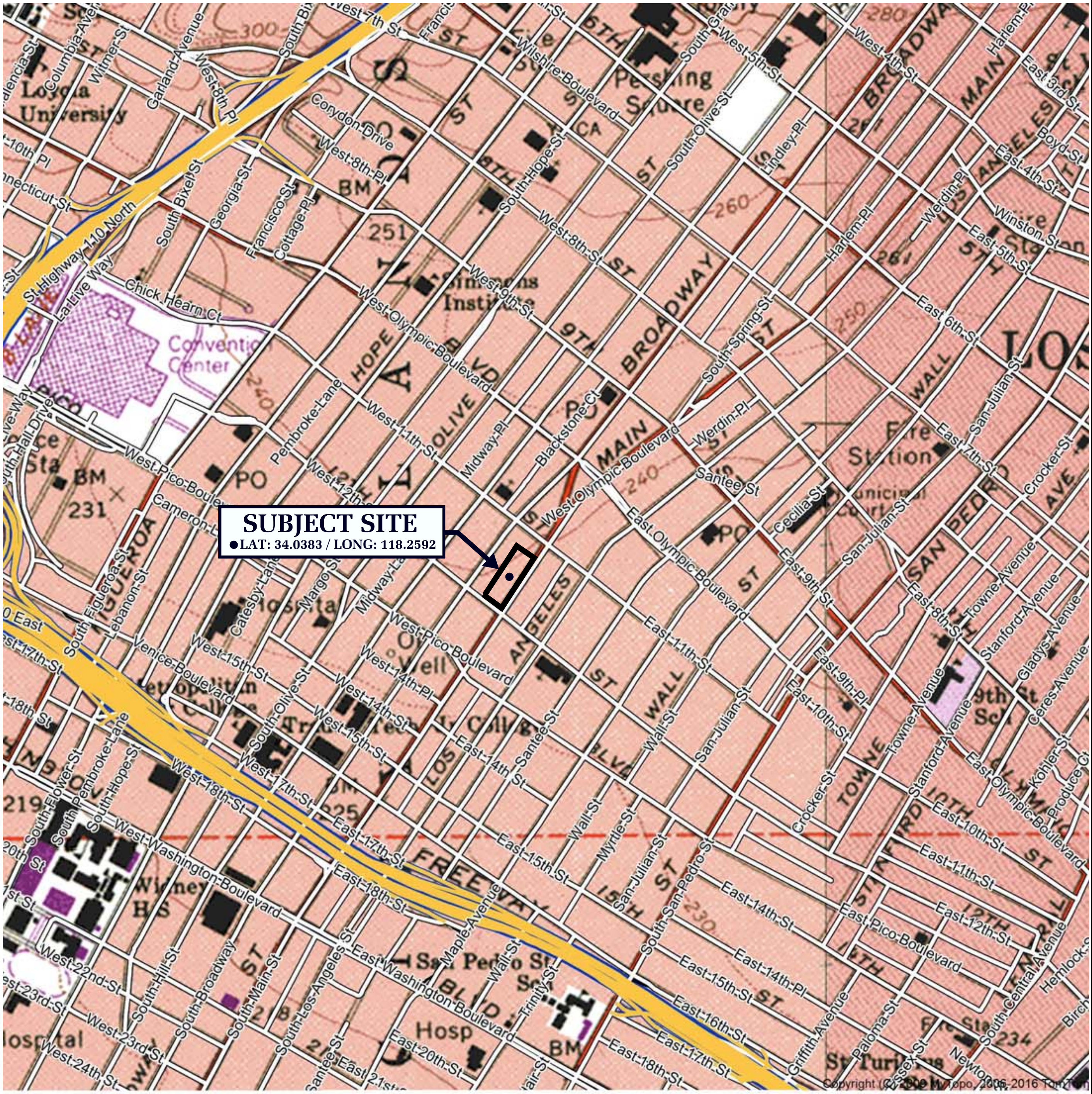

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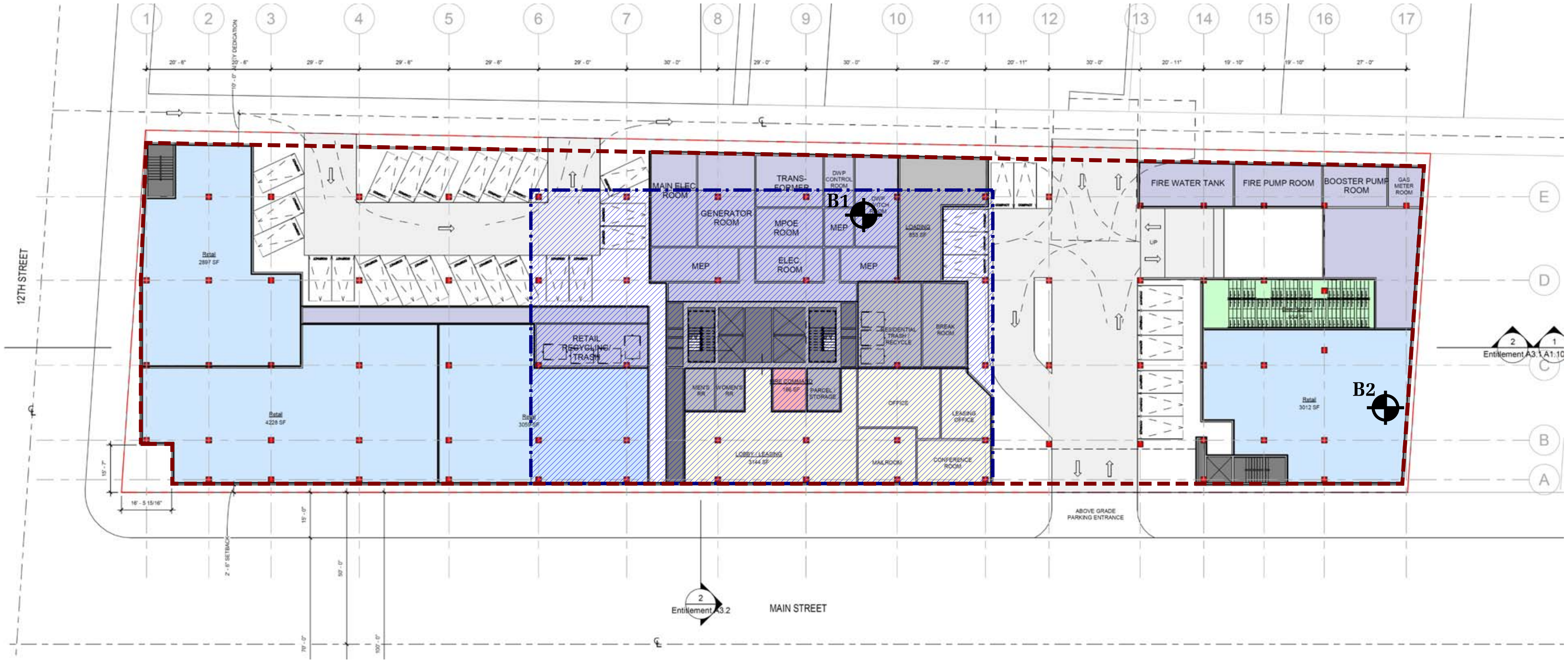
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704



LEGEND

B3 LOCATION & NUMBER OF BORING

LIMITS OF PROPOSED PODIUM

PROPOSED TOWER

SCALE IN FEET

0 40 80

Geotechnologies, Inc.
Consulting Geotechnical Engineers

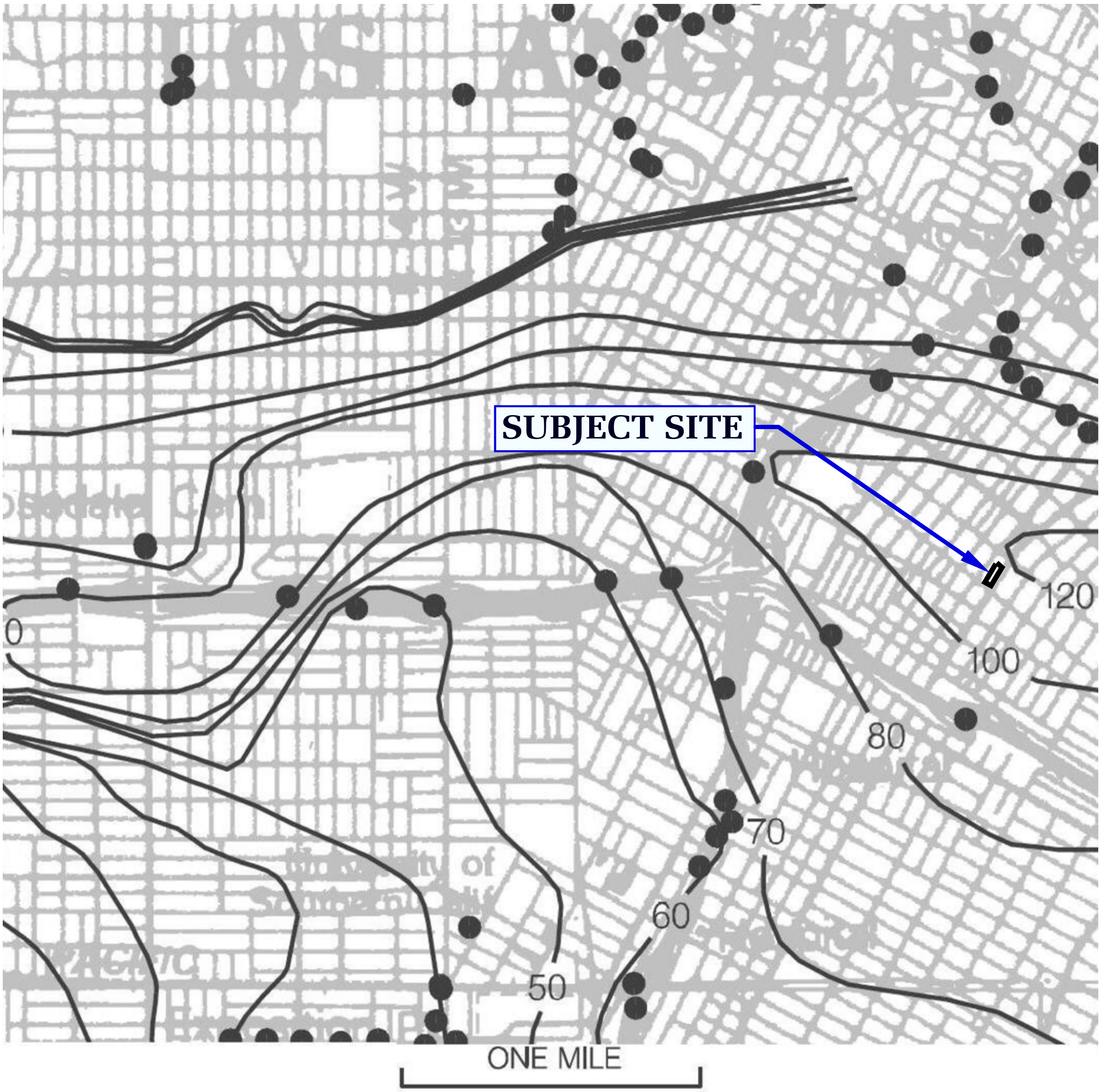
PLOT PLAN

FRONTIER HOLDINGS WEST, LLC

File No.: 21704

Date: December '18

REFERENCE: GROUND FLOOR PLAN BY MVE & PARTNERS DATED SEPTEMBER 13, 2018



20 Depth to groundwater in feet

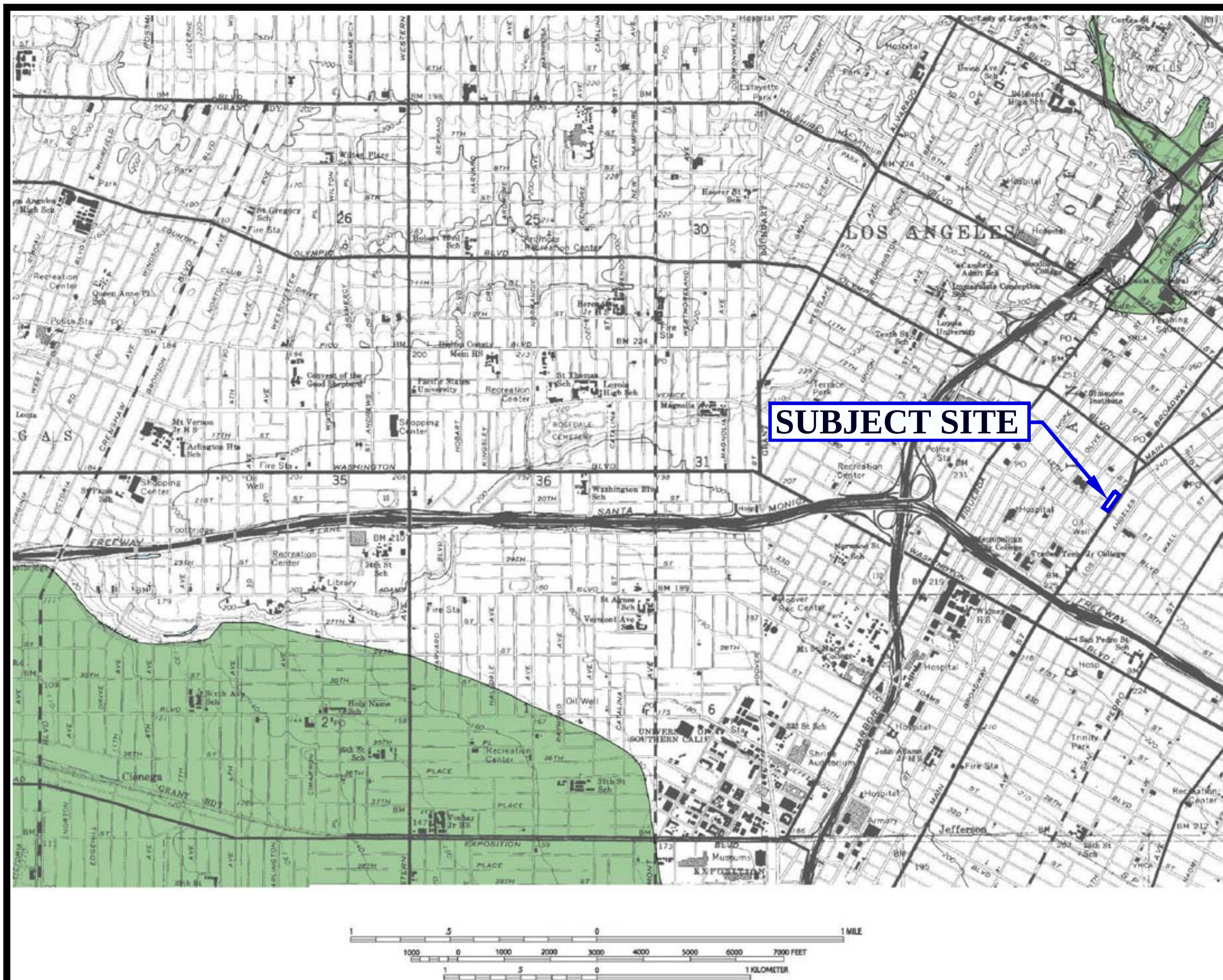
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 026
HOLLYWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE No. 21704



LIQUEFACTION AREA

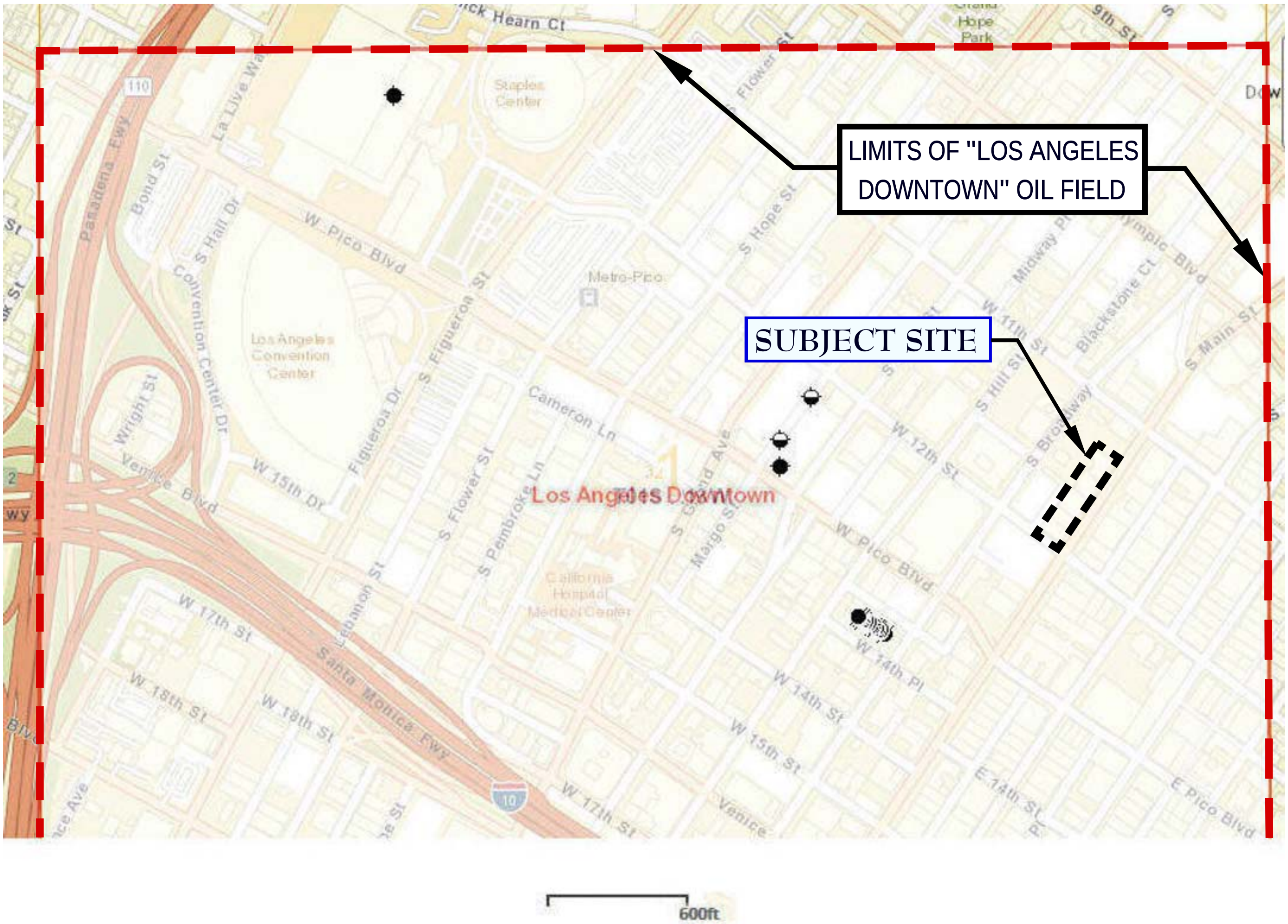
REFERENCE: SEISMIC HAZARD ZONE MAP, HOLLYWOOD QUADRANGLE, 1999, DIVISION OF MINES AND GEOLOGY

SEISMIC HAZARD ZONE MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704



LEGEND

Oil/Gas Well Location

REFERENCE: STATE OF CALIFORNIA DEPARTMENT OF CONSERVATION, DIVISION OF OIL, GAS, AND GEOTHERMAL RESOURCES ONLINE MAPPING SYSTEM

OIL FIELD / OIL WELL MAP

GEOTECHNOLOGIES, INC.
CONSULTING GEOTECHNICAL ENGINEERS

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704

BORING LOG NUMBER 1

Frontier Holdings West, LLC **Date: 11/06/18**

File No. 21704 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		2½-inch Asphalt over 2½-inch Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	32	4.6	122.1	-		
				3 --		
				-	SM/SP	NATIVE SOILS: Silty Sand to Sand, dark and yellowish brown, moist, medium dense, fine to coarse grained
				4 --		
				-		
5	53	1.0	SPT	5 --		
				-	SW	Cobbley Sand, dark brown, moist, dense, fine to coarse grained
				6 --		
				-		
				7 --		
7.5	59	2.7	126.7	-		
				8 --		
				-		
				9 --		
				-		
10	33	2.4	SPT	10 --		
				-	SP	Sand, dark brown, moist, medium dense, fine to medium grained, minor gravel
				11 --		
				-		
				12 --		
12.5	40	8.1	124.3	-		
				13 --	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained, few cobbles
				-		
				14 --		
				-		
15	41	4.9	SPT	15 --		
				-	SP	Sand, dark brown, moist, medium dense to dense, fine to medium grained, few gravel
				16 --		
				-		
				17 --		
17.5	39 50/3"	4.9	117.8	-		
				18 --		
				-		
				19 --		
				-		
20	37	6.9	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	39 50/4"	6.1	126.1	-		
				23 --		very dense, fine to medium grained, few gravel
				-		
				24 --		
				-		
25	79	4.9	SPT	25 --		
				-	SP/SW	Sand to Gravelly Sand, dark and yellowish brown, moist, very dense, fine to coarse grained

BORING LOG NUMBER 1

Frontier Holdings West, LLC

File No. 21704

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	42	7.2	125.9	-		
	50/5"			28 --		
				-		
				29 --		
				-		
30	60	4.0	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	4.5	1.7	126.6	-		
	50/4"			33 --		
				-		
				34 --		
				-		
35	50	8.9	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	39	4.9	126.7	-		
	50/2"			38 --		
				-		
				39 --		
				-		
40	36	11.1	SPT	40 --		
				-	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, medium dense, fine grained
				41 --		
				-		
				42 --		
42.5	18	18.3	115.7	-		
	50/5"			43 --	SP	Sand, dark brown, moist, very dense, fine to medium grained, minor gravel
				-		
				44 --		
				-		
45	60	2.9	SPT	45 --		
	50/2"			-		dark and olive brown, fine to coarse grained
				46 --		
				-		
				47 --		
47.5	100/10"	2.7	119.7	-		
				48 --		dark and yellowish brown, few gravel
				-		
				49 --		
				-		
50	79	2.8	SPT	50 --		
				-		

BORING LOG NUMBER 1

Frontier Holdings West, LLC

File No. 21704

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	23	3.6	118.1	-		
	50/5"			53 --		
				-		
				54 --		
				-		
55	85	8.0	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	46	3.9	118.0	-		
	50/2"			58 --		
				-		
				59 --		
				-		
60	49	2.5	SPT	60 --		
	50/3"			-		Total Depth 60 feet
				61 --		No Water
				-		Fill to 3 feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		SPT=Standard Penetration Test
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 2

Frontier Holdings West, LLC **Date: 11/06/18**

File No. 21704 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		2½-inch Asphalt over 3½-inch Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	44 50/3"	2.6	122.3	-		
				3 --		
				-	SM/SW	NATIVE SOILS: Sand to Cobbley Sand, dark brown, moist, very dense, fine to coarse grained
				4 --		
				-		
5	24 50/4"	2.0	118.1	5 --		
				-	SP	Sand, dark brown, moist, very dense, fine to coarse grained, few cobbles
				6 --		
				-		
				7 --		
7.5	66	2.1	127.3	-		
				8 --	SP/SW	Sand to Cobbley Sand, dark brown, moist, very dense, fine to coarse grained
				-		
				9 --		
				-		
10	45 50/3"	2.0	119.7	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	46 50/4"	2.3	126.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	28 50/4"	3.6	117.3	20 --		
				-	SP	Sand, dark brown, moist, very dense, fine to medium grained, few cobbles
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/8"	4.8	117.7	25 --		
				-		Cobbles

BORING LOG NUMBER 2

Frontier Holdings West, LLC

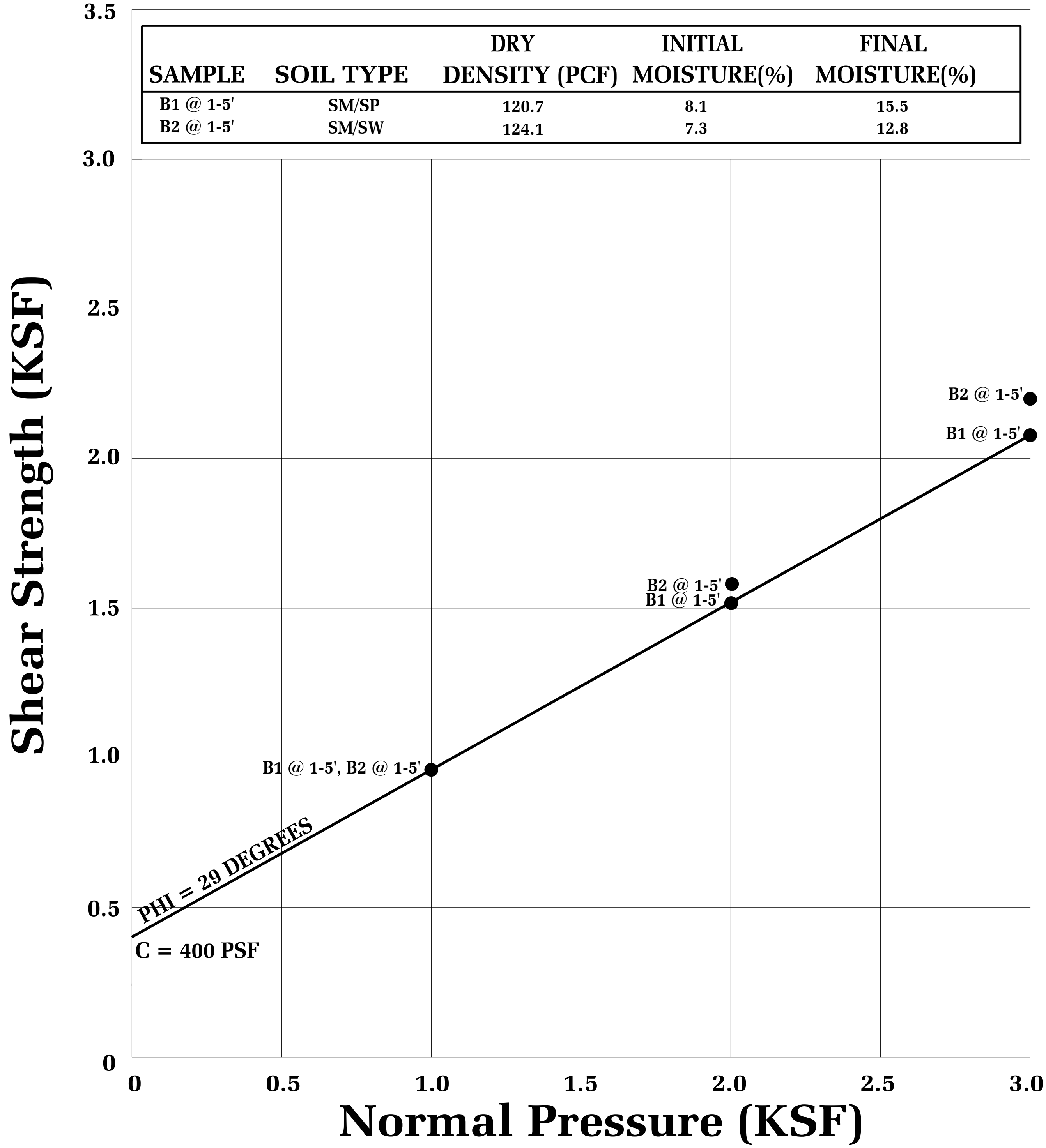
File No. 21704

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
				28 --		
				29 --		
30	44 50/4"	2.6	112.0	30 --		
				31 --		Total Depth 30 feet No Water Fill to 3 feet
				32 --		
				33 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				34 --		
				35 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				36 --		
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
				45 --		
				46 --		
				47 --		
				48 --		
				49 --		
				50 --		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM/SP	120.7	8.1	15.5
B2 @ 1-5'	SM/SW	124.1	7.3	12.8

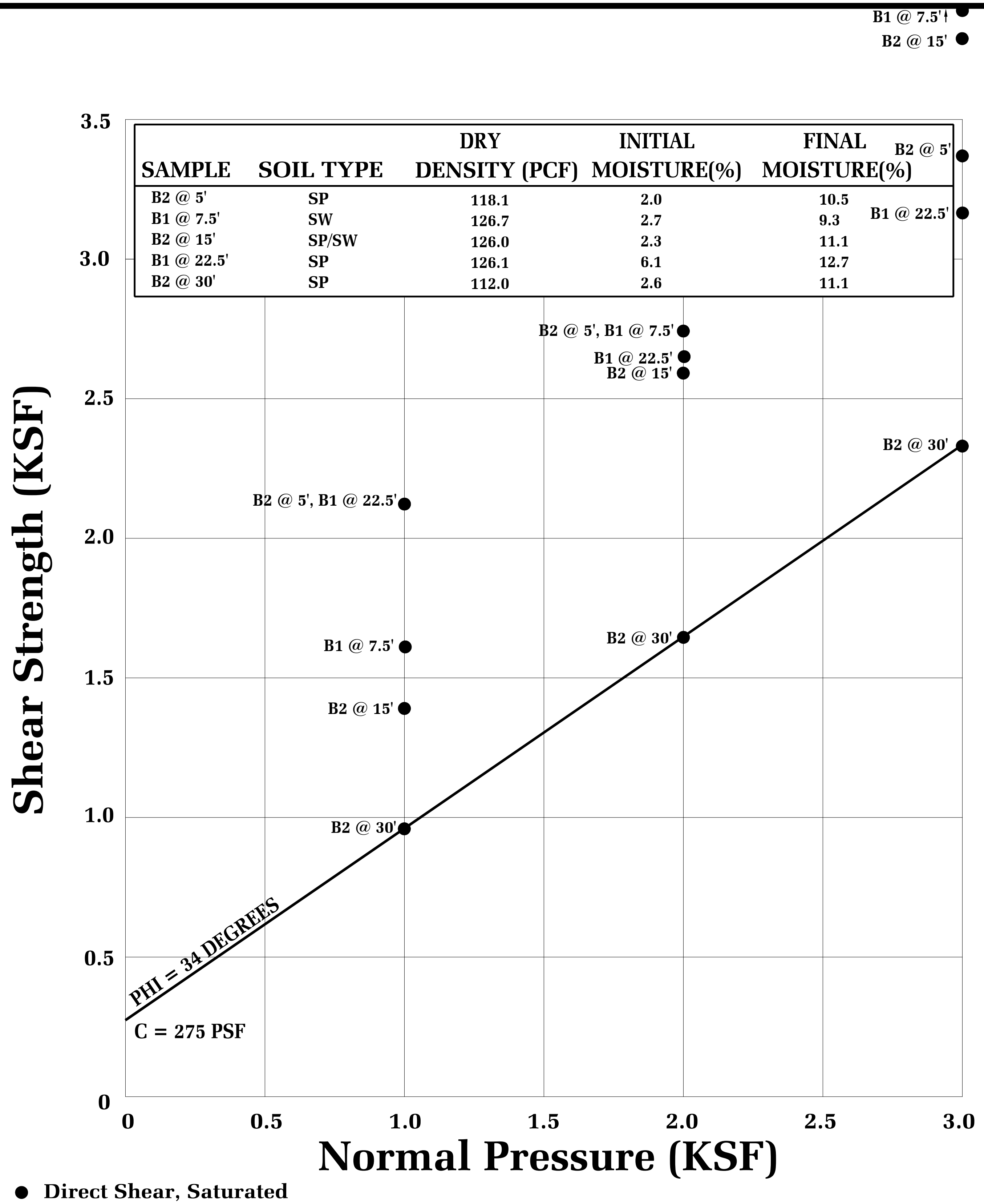


● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC	
FILE NO. 21704	PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 5'	SP	118.1	2.0	10.5
B1 @ 7.5'	SW	126.7	2.7	9.3
B2 @ 15'	SP/SW	126.0	2.3	11.1
B1 @ 22.5'	SP	126.1	6.1	12.7
B2 @ 30'	SP	112.0	2.6	11.1

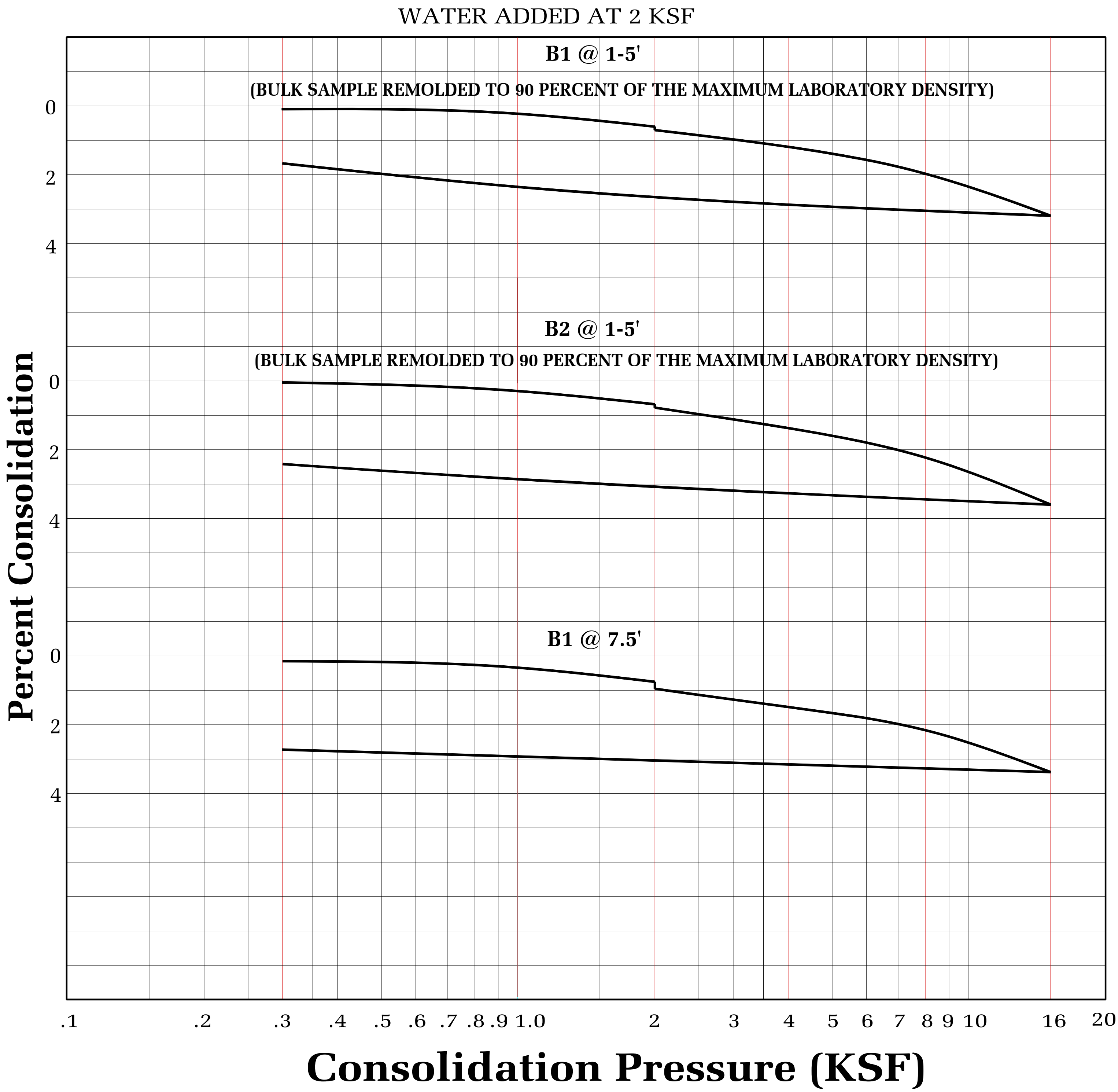
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704 | PLATE: B-2

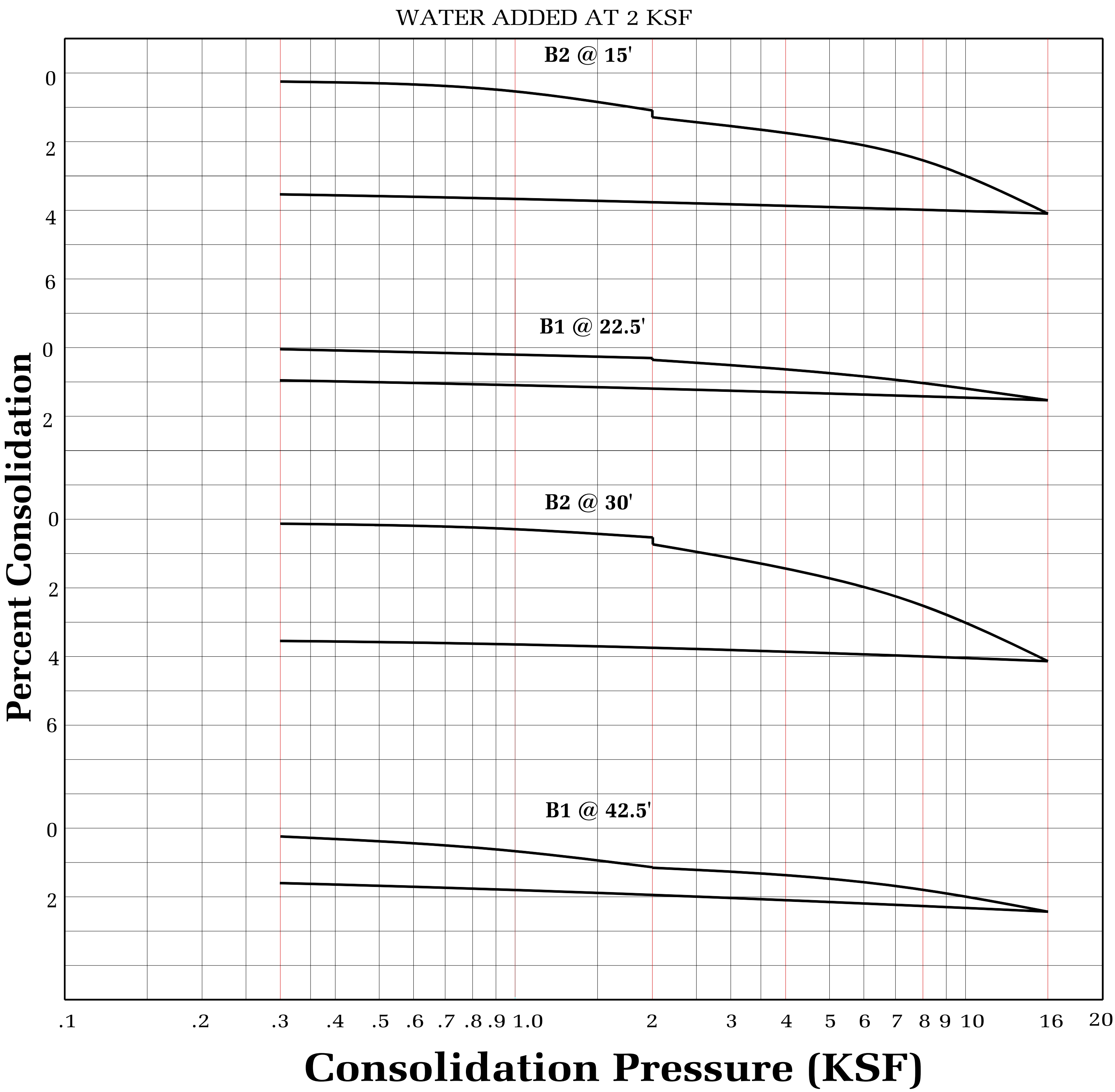


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704	**PLATE: C-1**



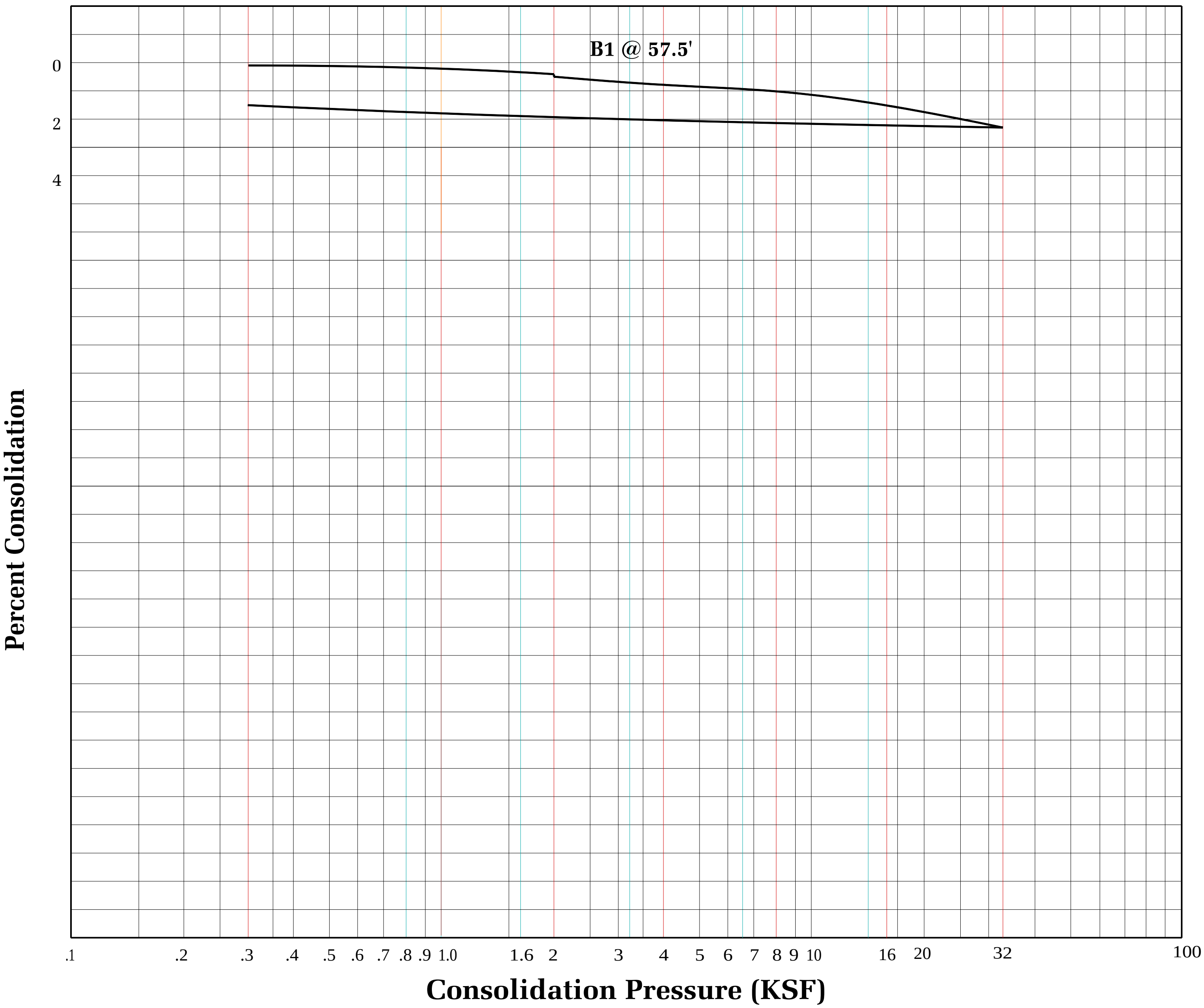
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704	PLATE: C-2

(WATER ADDED AT 2 KSF)



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704	PLATE: C-3

ASTM D-1557

SAMPLE	B1 @ 1-5'	B2 @ 1-5'
SOIL TYPE:	SM/SP	SM/SW
MAXIMUM DENSITY pcf.	120.7	124.1
OPTIMUM MOISTURE %	8.1	7.3

ASTM D 4829

SAMPLE	B1 @ 1-5'	B2 @ 1-5'
SOIL TYPE:	SM/SP	SM/SW
EXPANSION INDEX UBC STANDARD 18-2	35	7
EXPANSION CHARACTER	LOW	VERY LOW

SULFATE CONTENT

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

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FRONTIER HOLDINGS WEST, LLC

FILE NO. 21704 | PLATE: D

Geotechnologies, Inc.

Project: Frontier Holdings West, LLC

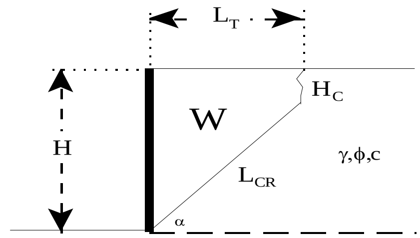
File No.: 21704

Description: Retaining Wall up to 5 feet High

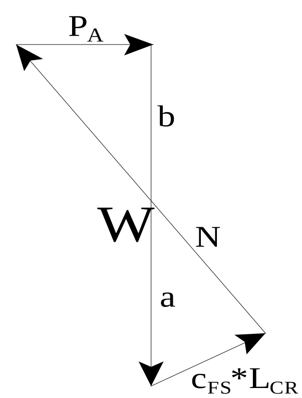
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	5.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	275.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	24.2	degrees
	(c_{FS})	183.3	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.4	-10	-1205.9	-2.2	-1355.7	149.8	0.0
41	6.1	-7	-910.0	-1.7	-1002.9	92.9	0.0
42	5.9	-5	-674.4	-1.3	-729.7	55.2	0.0
43	5.7	-4	-486.0	-1.0	-516.9	30.9	0.0
44	5.5	-3	-334.7	-0.7	-350.4	15.7	0.0
45	5.3	-2	-213.2	-0.5	-220.0	6.8	0.0
46	5.2	-1	-115.6	-0.3	-117.7	2.1	0.0
47	5.1	0	-37.5	-0.1	-37.7	0.2	0.0
48	5.0	0	24.5	0.1	24.4	0.1	0.0
49	4.9	1	73.3	0.2	72.3	1.0	0.5
50	4.8	1	111.1	0.3	108.6	2.5	1.2
51	4.7	1	139.6	0.4	135.5	4.1	2.1
52	4.7	1	160.2	0.4	154.5	5.6	3.0
53	4.6	1	174.1	0.5	167.2	7.0	3.8
54	4.6	1	182.3	0.5	174.4	8.0	4.6
55	4.6	1	185.6	0.5	177.0	8.6	5.1
56	4.5	1	184.6	0.6	175.7	8.9	5.5
57	4.5	1	179.8	0.6	171.1	8.8	5.6
58	4.5	1	171.7	0.5	163.4	8.3	5.6
59	4.6	1	160.7	0.5	153.1	7.5	5.2
60	4.6	1	146.9	0.5	140.4	6.5	4.7
61	4.6	1	130.7	0.4	125.4	5.3	4.0
62	4.7	1	112.2	0.4	108.2	4.1	3.2
63	4.7	1	91.6	0.3	88.8	2.8	2.3
64	4.8	1	69.0	0.3	67.4	1.6	1.4
65	4.8	0	44.4	0.2	43.7	0.7	0.6



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 5.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 0.5 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Frontier Holdings West, LLC

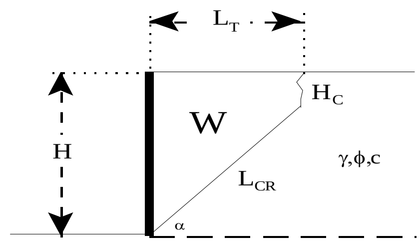
File No.: 21704

Description: Temporary Shoring up to 10 feet High

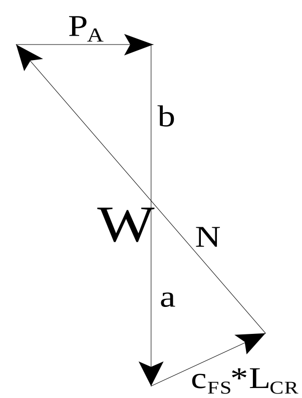
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	275.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	28.4	degrees
	(c_{FS})	220.0	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	10.0	0	-21.2	0.0	-21.3	0.0	0.0
41	9.4	7	873.9	1.0	845.6	28.3	6.4
42	8.8	12	1525.7	1.7	1431.2	94.6	23.0
43	8.4	16	2001.7	2.4	1824.6	177.1	46.3
44	8.0	19	2347.8	2.9	2084.4	263.4	73.8
45	7.6	21	2596.5	3.3	2250.1	346.4	103.6
46	7.4	22	2771.0	3.7	2348.6	422.4	134.4
47	7.1	23	2888.1	4.0	2398.8	489.3	165.1
48	6.9	24	2960.6	4.2	2414.2	546.3	195.1
49	6.7	24	2997.7	4.4	2404.3	593.4	223.6
50	6.5	24	3006.9	4.5	2376.1	630.8	250.4
51	6.4	24	2993.6	4.6	2334.6	659.0	275.0
52	6.3	24	2962.2	4.7	2283.5	678.7	297.2
53	6.2	23	2916.1	4.8	2225.6	690.4	316.8
54	6.1	23	2857.9	4.8	2163.0	695.0	333.7
55	6.0	22	2789.9	4.9	2096.9	693.0	347.7
56	6.0	22	2713.7	4.9	2028.7	685.0	358.9
57	5.9	21	2630.7	4.9	1958.9	671.8	367.0
58	5.9	20	2542.0	4.8	1888.2	653.7	372.1
59	5.9	20	2448.4	4.8	1816.9	631.5	374.2
60	5.9	19	2350.7	4.7	1745.3	605.4	373.2
61	5.9	18	2249.4	4.7	1673.3	576.1	369.1
62	6.0	17	2145.0	4.6	1601.1	543.9	362.1
63	6.0	16	2037.8	4.5	1528.5	509.3	352.0
64	6.1	15	1928.0	4.4	1455.3	472.6	339.0
65	6.1	15	1815.7	4.3	1381.5	434.2	323.1



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 374.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 7.5 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf

Geotechnologies, Inc.

Project: Frontier West Holdings

File No.: 21704

Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	7.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	34.0	degrees
Cohesion of Soils	(c)	275.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	1000.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

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

$A = 0.5*H*b$

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

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

$F_2 = d*L$

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

$R_2 = 2*\Delta F$

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

FS = Resistance Force/Driving Force

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

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
65	3.3	11	1428.1	930.0	1190.2	4355.4	8.0
66	3.1	11	1363.5	878.2	1120.8	4158.6	8.0
67	3.0	10	1300.0	827.2	1054.0	3964.7	8.0
68	2.8	10	1237.3	777.1	989.5	3773.7	8.0
69	2.7	9	1175.6	727.9	927.4	3585.4	8.0
70	2.5	9	1114.7	679.6	867.5	3399.6	8.0
71	2.4	8	1054.5	632.4	809.7	3216.1	8.0
72	2.3	8	995.1	586.3	754.0	3034.8	8.0
73	2.1	7	936.3	541.4	700.2	2855.6	8.0
74	2.0	7	878.2	497.6	648.2	2678.3	8.0
75	1.9	7	820.6	455.1	598.1	2502.7	8.0
76	1.7	6	763.6	414.0	549.6	2328.8	8.0
77	1.6	6	707.0	374.2	502.7	2156.4	8.0
78	1.5	5	651.0	335.8	457.3	1985.3	8.0
79	1.4	5	595.3	298.8	413.4	1815.6	8.0
80	1.2	4	540.0	263.4	370.8	1646.9	8.0
81	1.1	4	485.1	229.5	329.4	1479.4	8.0
82	1.0	3	430.4	197.1	289.2	1312.7	8.0
83	0.9	3	376.0	166.4	250.1	1146.8	8.0
84	0.7	3	321.9	137.4	212.1	981.7	8.0
85	0.6	2	267.9	110.1	174.9	817.2	8.0
86	0.5	2	214.2	84.5	138.6	653.1	8.0
87	0.4	1	160.5	60.7	103.0	489.5	8.0
88	0.2	1	106.9	38.6	68.1	326.2	8.0
89	0.1	0	53.5	18.4	33.8	163.0	8.0
90	0.0	0	0.0	0.0	0.0	0.0	8.0

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

d_{allow} 8.0 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 7 Feet in Height, with a Factor of Safety Equal or Exceeding 1.25.