Appendix E

Geotechnical Report

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September 8, 2026
File No. 22659

Shatto Emet QOZB LP
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Subject: Addendum II – Bulking Factor
Proposed Self-Storage Facility
515, 517, 525, and 531 S. Virgil Avenue, 518 and 526 S. Westmoreland Avenue
Los Angeles, California

References: *Report by Geotechnologies, Inc.:*
Geotechnical Engineering Investigation, revised November 18, 2025;
Addendum I – Additional Geotechnical Design Recommendations, revised August 3, 2026.

Correspondences by the City of Los Angeles, Department of Building and Safety:
Soils Report Approval Letter (Log # 138654), dated January 2, 2026;
Soils Report Approval Letter (Log # 142325), dated September 1, 2026.

Ladies and Gentlemen:

This letter has been prepared to provide a bulking factor for the Project Site. It is anticipated that a bulking factor of 25 to 30 percent may be utilized for estimating the export quantity.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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July 17, 2025
Revised November 18, 2025
File Number 22659

Shatto Emet QOZB LP
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Subject: Preliminary Geotechnical Engineering Investigation
Proposed Self-Storage Facility
515, 517, 525, and 531 S. Virgil Avenue, 518 and 526 S. Westmoreland Avenue
Los Angeles, California

Ladies and Gentlemen:

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

This report is preliminary in nature due to the conceptual nature of the Project. It is recommended that additional exploration be performed to achieve a thorough investigation of the Project Site once the development plans achieve refinement. The recommendations contained herein shall be considered to be preliminary and are subject to confirmation based on review of the design plans and the result of the additional exploration.

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED STORAGE FACILITY
515, 517, 525, AND 531 S. VIRGIL AVENUE
518 AND 526 S. WESTMORELAND AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This report is preliminary in nature due to the conceptual nature of the Project. It is recommended that additional exploration be performed to achieve a thorough investigation of the Project Site once the development plans achieve refinement. The recommendations contained herein shall be considered to be preliminary and are subject to confirmation based on review of the design plans and the result of the additional exploration.

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. The proposed project will consist of construction of a new 8-story storage facility which will be constructed at

or near the existing site grade. Preliminary, column loads are estimated to be between 800 and 1,200 kips. Wall loads are estimated to be between 10 and 15 kips per lineal foot. Grading will consist of excavation on the order of 6 to 8 feet for the removal and recompaction of the existing unsuitable site soils for foundation and slab support.

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

SITE CONDITIONS

The Project Site is located at 515, 517, 525, and 531 S. Virgil Avenue, 518 and 526 S. Westmoreland Avenue, in the City of Los Angeles, California. The Project Site is bounded by a 2 to 3-story medical building and a paved parking lot to the north, S. Virgil Avenue to the east, paved parking lots to the south, and S. Westmoreland Avenue to the west.

The site is currently developed with a 4-story office building and associated at-grade parking lot located to the west of the office building. The existing building and associated site improvements will be demolished prior to the proposed construction.

The vegetation at the Project Site consists of isolated shrubs and plants located along the east side of the existing office building. Based on the available survey by Ware Malcomb, a high site elevation of 274.0 feet is recorded near the central portion of the site and a lower site elevation of 268.5 feet is recorded at the southeast corner of the site, corresponding to 5½ feet of elevation change across the Project Site. Drainage across the site is by sheetflow to the city streets. The neighboring development consists primarily of commercial and residential buildings.

RESEARCH

As a part of this geotechnical investigation, this firm requested past geotechnical reports performed for the Project site from the City of Los Angeles, Department of Building and Safety (LADBS). The following report by Feffer Geological Consultants (FGC) and approval letter were obtained from LADBS:

- Geotechnical Investigation, Proposed Seven Story and Eight Story Building Over Two Subterranean Levels, 515-531 S. Virgil Avenue and 518-526 S. Westmoreland Avenue, Los Angeles, California (File No. 2126-84), dated April 6, 2018.
- Soil Report Approval Letter by City of Los Angeles, Department of Building and Safety (Log # 105164), dated October 2, 2018.

A geotechnical investigation was previously performed by FGC for the development of an 8-story and 7-story buildings over 2 levels of subterranean parking garage. Based on the referenced geotechnical investigation report by FGC, the Project Site was explored on March 15, 2018, by excavating four exploratory borings. Boring B1 was excavated to the depth of 46½ feet, and borings B2 through B4 were excavated to the depths of 21½ feet. Approximately 6½ to 7½ feet of fill materials were encountered in boring by FGC. Fill materials consisted of predominately of clayey silt and sand. Fill materials were underlain by Alluvium, consisting of sand, silt, and silty clay, which are very dense and hard.

FGC reported that Puente Formation bedrock was encountered below the Alluvium, at depths ranging between 12½ and 14½ feet below the existing site grade. As described by FGC, the upper bedrock consists of sands with gravel and sandy siltstone, with fine sands and gravelly silty sands. Siltstone was encountered by FGC at a depth of 25 feet in Boring No. 1. The other FGC borings terminated at 21½ feet below the ground surface. Groundwater was not encountered by FGC to the maximum depth of 46½ feet.

The referenced FGC report was reviewed and approved by LADBS under Log # 105164). The locations of borings by FGC are shown on the enclosed Plot Plan. Boring Logs by FGC are enclosed in the Appendix of this report.

Geotechnologies has reviewed the referenced report by FGC and is in general concurrence with their findings, except as noted in the Geologic Materials section of this report. Geotechnologies accepts the professional responsibility as the geotechnical engineer of record for the Project.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on May 13, 2025, by excavating two exploratory borings. The borings were excavated to a depth of 30 feet below the existing site grade, with the aid of a bucket-type drilling machine equipped using a 24-ich diameter drilling bucket. Downhole logging was performed by a representative of this firm in both borings to provide a more detailed measurement of the soil and bedrock contact. The exploratory boring locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-2.

The locations of exploratory borings were determined from hardscape features shown on the attached Plot Plan. The elevations of exploratory borings are estimated based on the elevation contour lines obtained from the NavigateLA website. The locations and elevations of the exploratory borings should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials to a depth of 6 feet were encountered in both exploratory borings excavated by this firm. The existing fill consists of sandy silt to silty clay, dark brown and gray, moist, stiff.

Fill materials are underlain by Older Alluvium, consisting of clayey silt and silty clay, silty sand, sand, and gravelly sand, which are predominantly dark grayish brown to yellowish and reddish brown in color, moist, stiff, dense to very dense, fine to coarse grained.

Bedrock was encountered in both exploratory borings performed by this firm at approximately 25 feet below existing site grade. The bedrock consists of siltstone, which are dark grayish brown in color, moist to very moist, moderately hard, weathered in the upper portion.

Based on review of the referenced geotechnical investigation report by FGC, FGC reported that Puente Formation bedrock was encountered below the Alluvium, at depths ranging between 12½ and 14½ feet below the existing site grade. The upper bedrock, as described by FGC, consisted of sands with gravel and sandy siltstone, with fine sands and gravelly silty sands. Siltstone was encountered by FGC at a depth of 25 feet in Boring No. 1. However, the other FGC borings were terminated at 21½ feet below the ground surface in the "silty sandstone" materials.

It is the opinion of this firm that FGC has misidentified the upper bedrock material as sandstone, and should be correctly identified as a dense, older, granular alluvial deposits. The borings performed by FGC consisted of hollow-stem borings in which the materials could only be described by soil samples collected from the sampler. Whereas, the boreholes excavated by this firm were excavated by a bucket-type drill rig and were downhole logged by a representative of this firm to verify the geologic materials and depth to the Puente Formation siltstone bedrock, which occurs at an approximate depth of 25 feet below the existing site grade. More detailed description of the earth materials encountered in exploratory borings can be obtained from individual boring logs provided in the Enclosures.

Groundwater

Groundwater seepage was encountered in exploratory borings at depths ranging from 25 to 29½ feet below the existing site grade. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report of Hollywood Quadrangle (1998, revised 2006), Plate 1.2 entitled "Historically Highest Ground Water Contours". Review of this plate indicates that the historically highest groundwater level is on the order of 20 feet below site grade.

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

Caving

Caving was not encountered in the exploratory borings. 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 Project Site is located in the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

The 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 CGS, faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

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

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature, no known Holocene-active or Pre-Holocene faults underlie the Project Site. Based on review of the Earthquake Zones of Required Investigation for Hollywood 7.5-Minute Quadrangle (CGS, 2014),

the Project Site is not located within the Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the Project 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 Earthquake Zones of Required Investigation for Hollywood 7.5-Minute Quadrangle (CGS, 2014) 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.

Based on the stiff and dense to very dense nature of older alluvium, and presence of relatively shallow bedrock, the potential for liquefaction occurring at the Project Site is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

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

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

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within the potential inundation boundaries of Silver Lake Reservoir and Rowena Reservoir. 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 approximate elevation difference across the site is only on the order of 3 feet. Additionally, based on review of Earthquake Zones of Required Investigation for Hollywood 7.5-Minute Quadrangle (CGS, 2014), the site is not located within area designated as “Earthquake Induced Landslide Zone”. Based on these considerations, the potential of landslides negatively affecting the proposed development is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the preliminary finding of Geotechnologies, Inc. that construction of the proposed storage facility 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 due to the conceptual nature of the Project. It is recommended that additional exploration be performed to achieve a thorough investigation of the Project Site once the development plans achieve refinement. The recommendations contained herein shall be considered to be preliminary and are subject to confirmation based on review of the design plans and the result of the additional exploration.

Approximately 6 to 7½ feet of existing fill materials were encountered in exploratory borings during site explorations by this firm and also previously by FGC. Groundwater seepage was encountered in the exploratory borings at depths ranging from 25 to 29½ feet below the existing site grade. The historically highest groundwater level is on the order of 20 feet below site grade.

Based on the conceptual drawings prepared by Ware Malcomb, the proposed development will be constructed at or near the existing site grade. The existing fill materials are considered to be unsuitable for foundation and slab support. All existing fill materials and soils disturbed as a result of demolition shall be properly removed and recompacted for support of the proposed foundations and slabs-on-grade. The proposed fill pad shall extend a minimum of 6 feet below the existing site grade or 3 feet below the bottom of the proposed foundations, whichever is greater. In addition, the proposed fill pad shall extend a minimum of 3 feet horizontally beyond the edges of the foundation system or equivalent to the depth of fill below the foundations, whichever is greater. The proposed structure may be supported by conventional foundations bearing in the properly compacted fill pad.

Temporary excavations between 6 to 8 feet may be required for the recommended removal and recompaction. Temporary excavations up to 8 feet in height adjacent to property lines may be excavated using the "A-B-C" slot cut method.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings, or which may result from changes in subsurface conditions. Once the project design achieves more definition, the proposed development plans shall be provided to this firm for review. Any changes in the design, as outlined in this report, shall 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

2025 California Building Code Seismic Parameters

According to Section 11.4.2.1 of ASCE 7-22, the risk-targeted Maximum Considered Earthquake (MCE_R) spectral response accelerations shall be based on the most critical spectral acceleration at each period of Site Class C, CD, and D. The Project Site is not liquefiable and does not fall in the Site Class DE, E, or F categories. A "Default" Site Class was utilized for determining the multi-period design response spectrum in accordance with Section 11.4.5.1 of the ASCE 7-22 (2025 CBC). The MCE_R spectral acceleration and the design response spectral acceleration are presented in the Appendix of this report.

FILL SOILS

Approximately 6 to 7½ feet of existing fill materials were encountered in exploratory borings during site explorations by this firm and also previously by FGC. The existing fill material and any fill generated during demolition shall be properly removed and recompacted as compacted fill for support of the proposed building.

EXPANSIVE SOILS

The onsite geologic materials are in the high expansion range. The Expansion Index was found to be between 94 and 110 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation" and "Slab-on-Grade" sections of this report.

OIL WELLS

According to the California Department of Conservation, Geologic Energy Management Division (CalGEM) Well Finder map, the Project Site is located within the Los Angeles City Oil Field. One of the oil wells (API 0403726259) is located in the area of the existing 4-story building. A plan showing the location of nearby wells is presented in the enclosed Oil Well Location Map. Geotechnologies does not practice in the field of environmental engineering. A qualified environmental consultant should be consulted regarding any regulatory requirements and implications of the presence of the oil well (if any).

METHANE ZONES

According to the City of Los Angeles Methane and Methane Buffer Zones Map, the Project Site is located within a Methane Zone as designated by the City. A qualified methane consultant should

be retained to consider the requirements and implications of the City's Methane Zone designation. A copy of the portion of the map covering the Project Site is included herein.

WATER-SOLUBLE SULFATES

The Portland cement portion of concrete is subject to attack when exposed to water-soluble sulfates. Usually, the two most common sources of exposure are from soil and marine environments. The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on the most recent revision to American Concrete Institute (ACI) Standard 318, the sulfate exposure is classified as S0, which is considered to be negligible, for geologic materials with less than 0.1%, and therefore, there are no restrictions on cement type 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, associated root system, 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.

- 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

All existing fill materials shall be properly removed and recompacted for support of the proposed structure. The proposed building area shall be excavated to a minimum depth of 6 feet below the existing site grade, or 3 feet below the bottom of the proposed foundations, whichever is greater. The excavation shall extend a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structure are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum 90 percent of the maximum density, except for cohesionless soils having less than 15 percent finer than 0.005 millimeters, which shall be compacted to a minimum 95 percent of the maximum density of the material used. 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. Due to the high expansion potential of the onsite soils, it is recommended that the materials placed should be moisture conditioned to a minimum of three percent over the optimum moisture content of the particular material placed. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum

laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

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

Acceptable Materials

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005

millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Soils

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

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

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

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. If on-site soils are found suitable for use as compacted fill and/or backfill by environmental

consultant, 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 at least three percent over the optimum moisture content of the particular material used.

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

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by

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

FOUNDATION DESIGN

Based on the conceptual drawings prepared by Ware Malcomb, the proposed structure will be constructed at or near the existing site grade. The proposed structure may be supported by conventional foundations bearing in the recommended compacted fill pad.

Conventional Foundations

Continuous foundations bearing in the compacted fill soils may be designed for a bearing capacity of 2,000 pounds per square foot and should be a minimum of 12 inches in width and 24 inches into the recommended bearing material.

Column foundations bearing in native alluvial soils may be designed for a bearing capacity of 2,500 pounds per square foot and should be a minimum of 24 inches in width and 24 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 100 pounds per square foot. The bearing capacity increase for each additional foot of depth is 300 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

Due to a high expansion potential for the onsite soils, all bearing walls must be supported on continuous footings. Perimeter footings for the at-grade portion of the building shall extend a minimum of 3 feet below the lowest adjacent grade, and must be continuous enclosing the entire

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

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 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.

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 0.25 may be used with the dead load forces.

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

Foundation Settlement

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

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

Building walls and miscellaneous site retaining walls up to 5 feet in height may be required for the proposed project. Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Preliminary, retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure (pcf)
Up to 5 feet	35 pcf	70 pcf

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. 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.

Additional pressure should be added for the surcharge condition due to vehicular traffic or adjacent structures. For traffic surcharge, the upper ten 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.

Dynamic (Seismic) Earth Pressure

Seismic wall design pressure is not required for retaining walls less than 6 feet in height. Retaining 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 additional seismic loads, with an equivalent fluid pressure of 30 pounds per cubic foot. When utilizing the code load combination formulas, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained building walls under seismic loading condition.

Retaining Wall Waterproofing

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

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

Retaining Wall Drainage

Subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rock. As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension and may consist of ¾-inch to once inch crushed rock, wrapped in filter fabric.

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

Retaining Wall Backfill

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

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

Sump Pump Design

Groundwater seepage was encountered in exploratory borings at depths ranging from 25 to 29½ feet below the existing site grade. Since the proposed development will be constructed at or near the existing site grade, the retaining wall backdrainage system is not expected to experience an appreciable flow of water. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

TEMPORARY EXCAVATIONS

It is anticipated that excavations between 6 to 8 feet in vertical height will be required for the recommended grading for support of the proposed structure. Temporary excavations are expected to remove existing fill materials and expose native soil, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Unsurcharged vertical excavations exceeding 5 feet in height, and any excavations near adjacent buildings or property lines shall be made using the slot-cutting method.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Temporary excavations over 5 feet in height should be excavated at a uniform 1:1 (h:v) slope gradient in their entirety, up to a maximum depth of 10 feet. A uniform sloped excavation is sloped from bottom to top and does not have any vertical component.

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

Slot Cuts

Slot cutting method may be utilized for excavations up to 8 feet in height, to maintain stable excavations adjacent to property lines or existing buildings. The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation is made at a uniform 1:1 (h:v) slope. Alternate "A" slots of 8 feet in width may be worked. The remaining earth buttresses ("B" and "C" slots) should each 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. After completing the backfilling in the "B" slots, finally the "C" slots may be excavated.

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.

SLABS ON GRADE

Concrete Slabs-on Grade

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

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

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Due to the highly expansive nature of the onsite soils, and the shallow depth to bedrock (which are considered to be relatively impermeable), it is the opinion of this firm that stormwater infiltration is not feasible for the Project Site.

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. California sedimentary bedrock is known for containing variable layers, such as abundant gravel, cobbles, and boulders. Additionally, bedrock can contain concretion. Concretions are typically lenticular and follow the bedding. 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.

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 borings were collected and transported to the laboratory. Undisturbed samples of soil were obtained with the aid of a hand sampler. Soil samples were transported to the laboratory for testing. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hours 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. The results of expansion testing are plotted on D-Plate.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. 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. The results of laboratory compaction testing are plotted on D-Plate.

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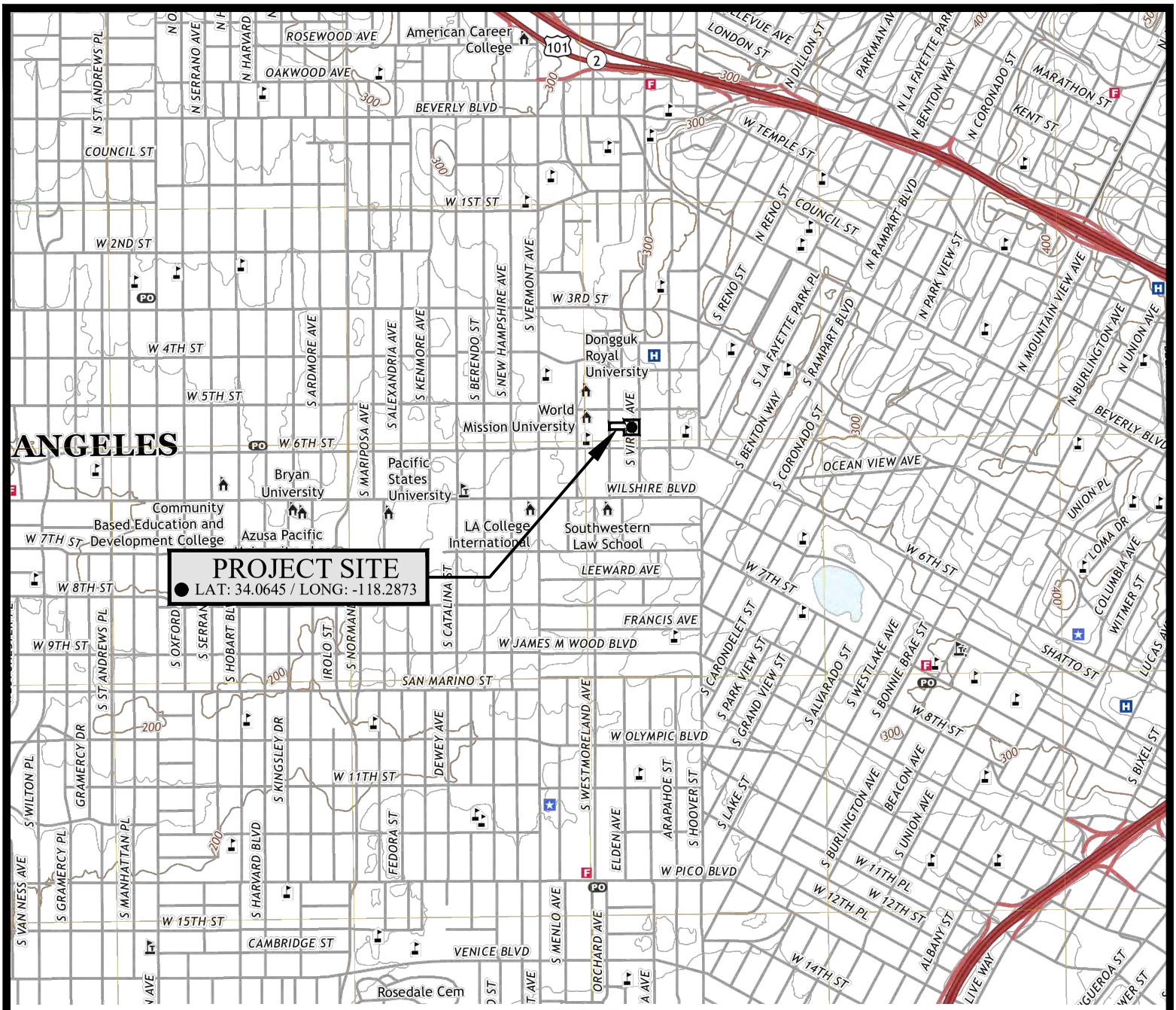
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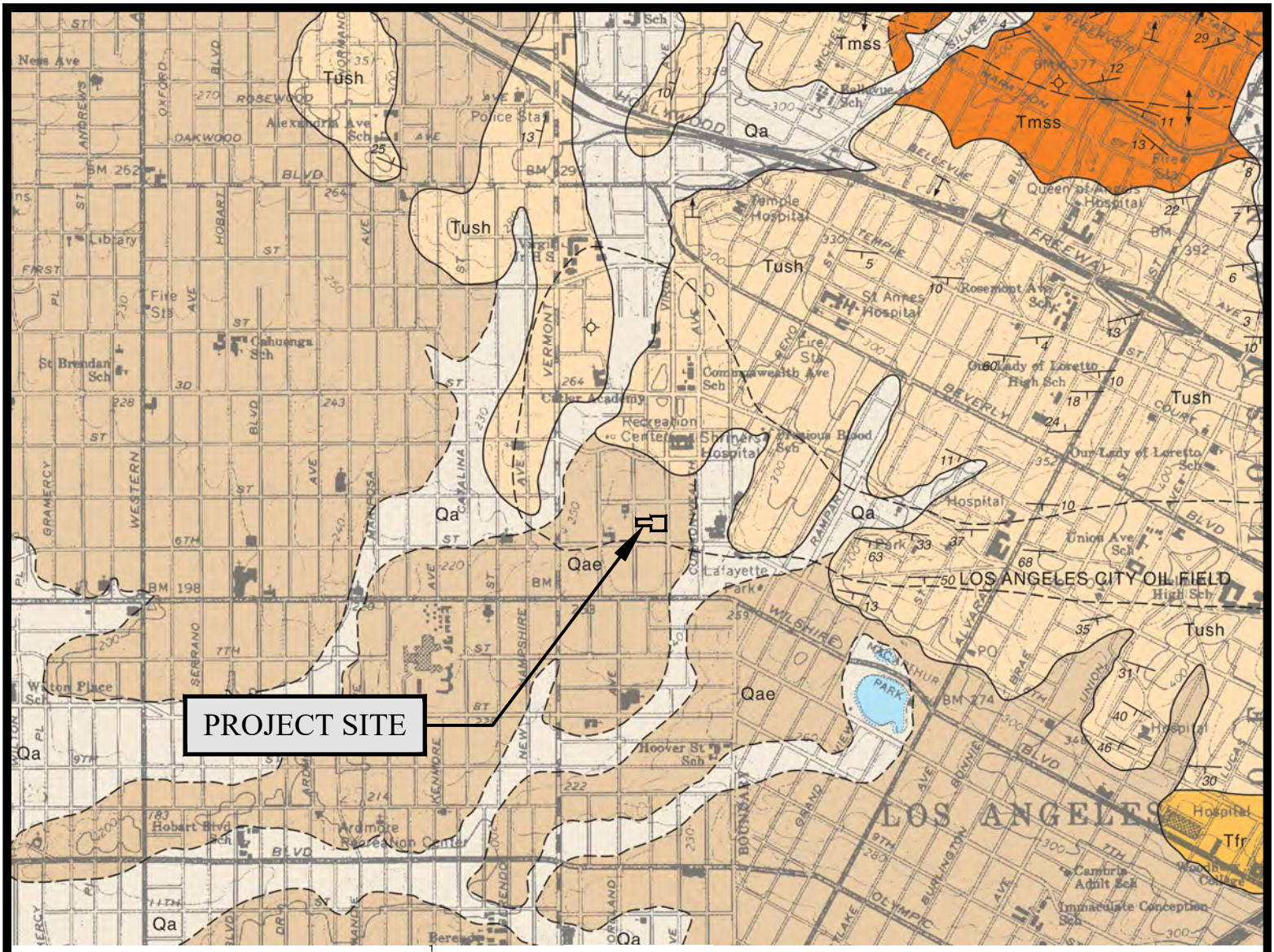
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 - MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE 2022

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

SHATTO EMET QOZB LP
515-531 S VIRGIL AVE., 518-526 S WESTMORELAND AVE., LOS ANGELES

FILE NO: 22659



LEGEND

Qa: Surficial Sediments - Alluvium: sand and clay, includes gravel and sand of minor stream channels

Qae: Older Surficial Sediments - Similar to Qa, but slightly elevated and dissected

Tfr: Fernando Formation - Repetto Member: In downtown Los Angeles, consolidated but crumbly gray to greenish-gray claystone-siltstone, in part sandy; vaguely bedded, locally fossiliferous

Tush: Unnamed Shale - Gray to light brown, thin-bedded silty clay shale, soft and crumbly

Tmss: Monterey Formation - Tan to light gray semi-friable sandstone; includes thin interbeds of micaceous silty clay shale; forms Elysian deep sea fan of Dibblee 1989

Folds - arrow on axial trace of fold indicates direction of plunge

Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

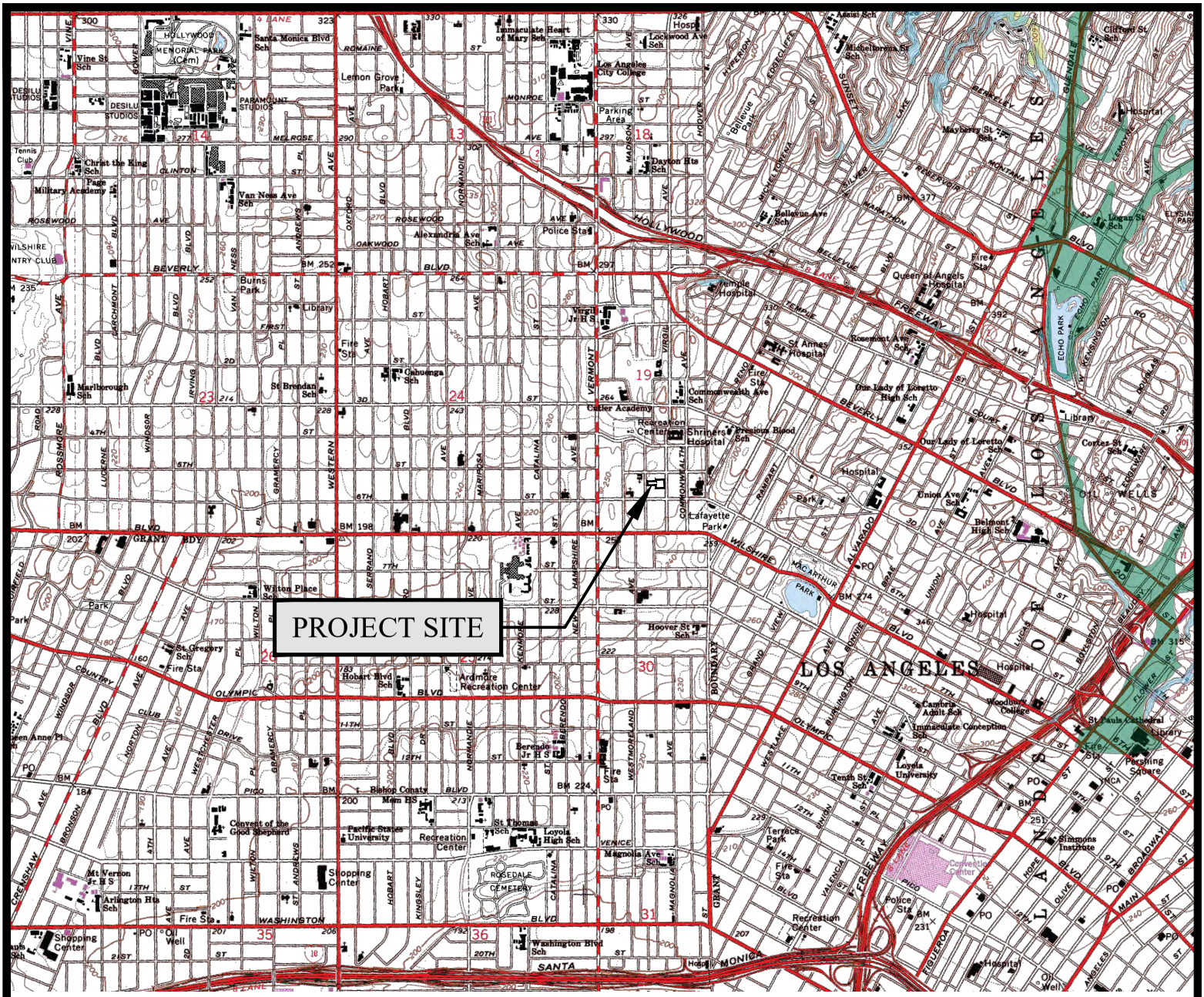
REFERENCE: T.W. DIBBLEE (EDITED 2010) GEOLOGIC MAP OF THE HOLLYWOOD & SOUTH HALF BURBANK QUADRANGLES (#DF-30)

GEOLOGIC MAP (DIBBLEE)

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FILE NO: 22659



LEGEND

LIQUEFACTION ZONES

EARTHQUAKE-INDUCED LANDSLIDE ZONES

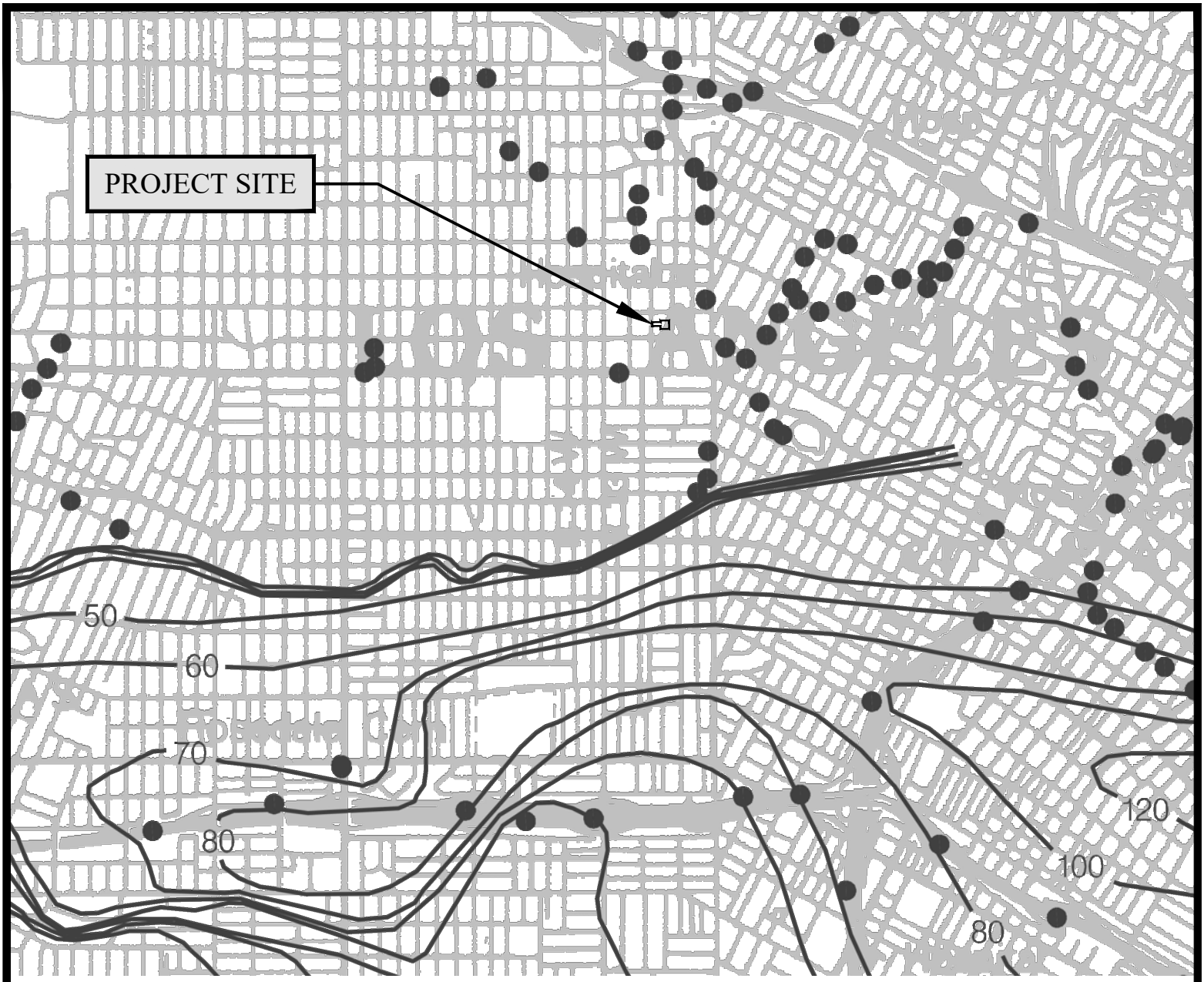
REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, HOLLYWOOD QUADRANGLE (CGS, 2014)

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

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Consulting Geotechnical Engineers

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FILE NO: 22659



ONE MILE

SCALE

LEGEND

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

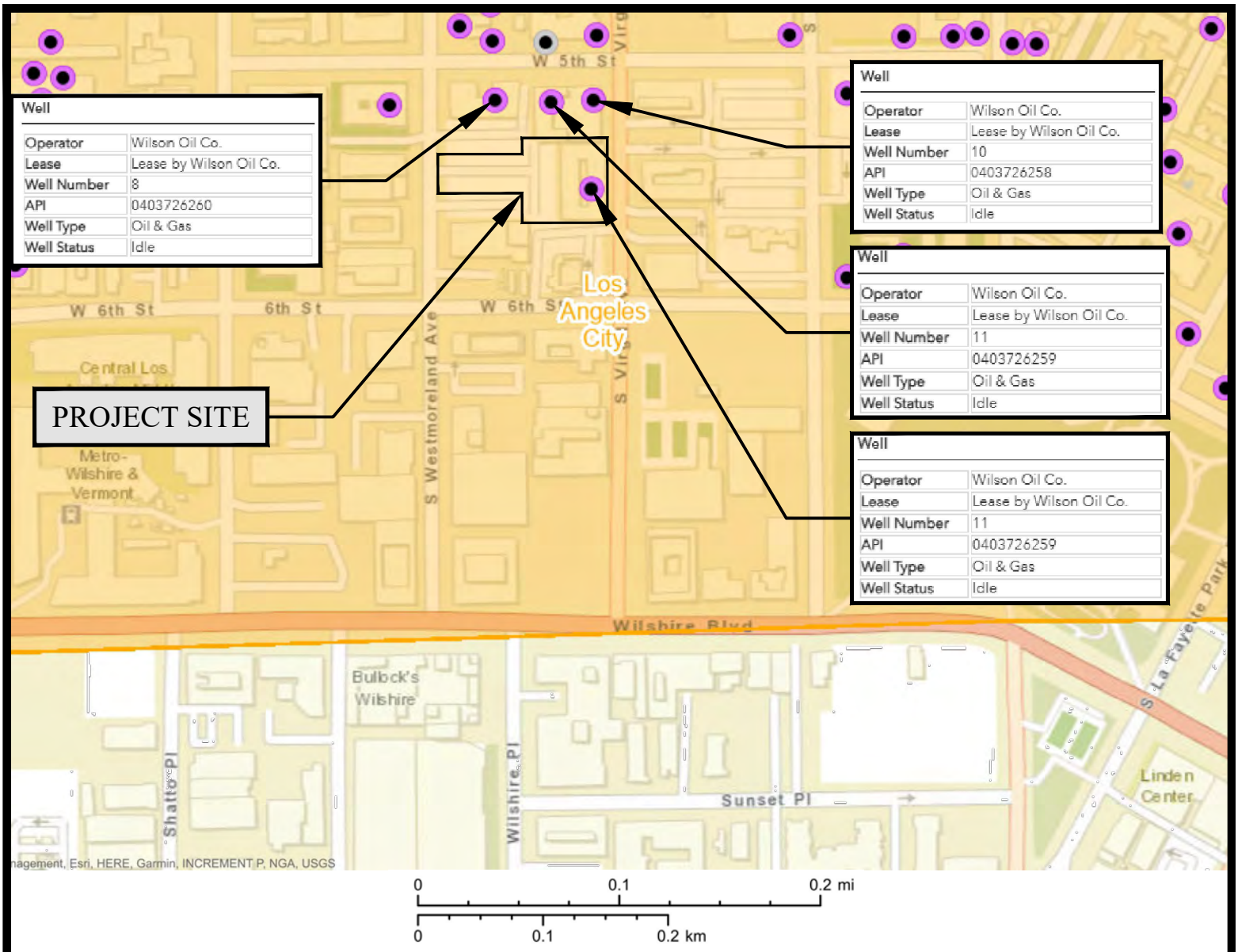
REFERENCE: SEISMIC HAZARD ZONE REPORT, 026 HOLLYWOOD, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998 (REV 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS MAP

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FILE NO: 22659



LEGEND

OIL WELLS

OIL/ GAS FIELDS

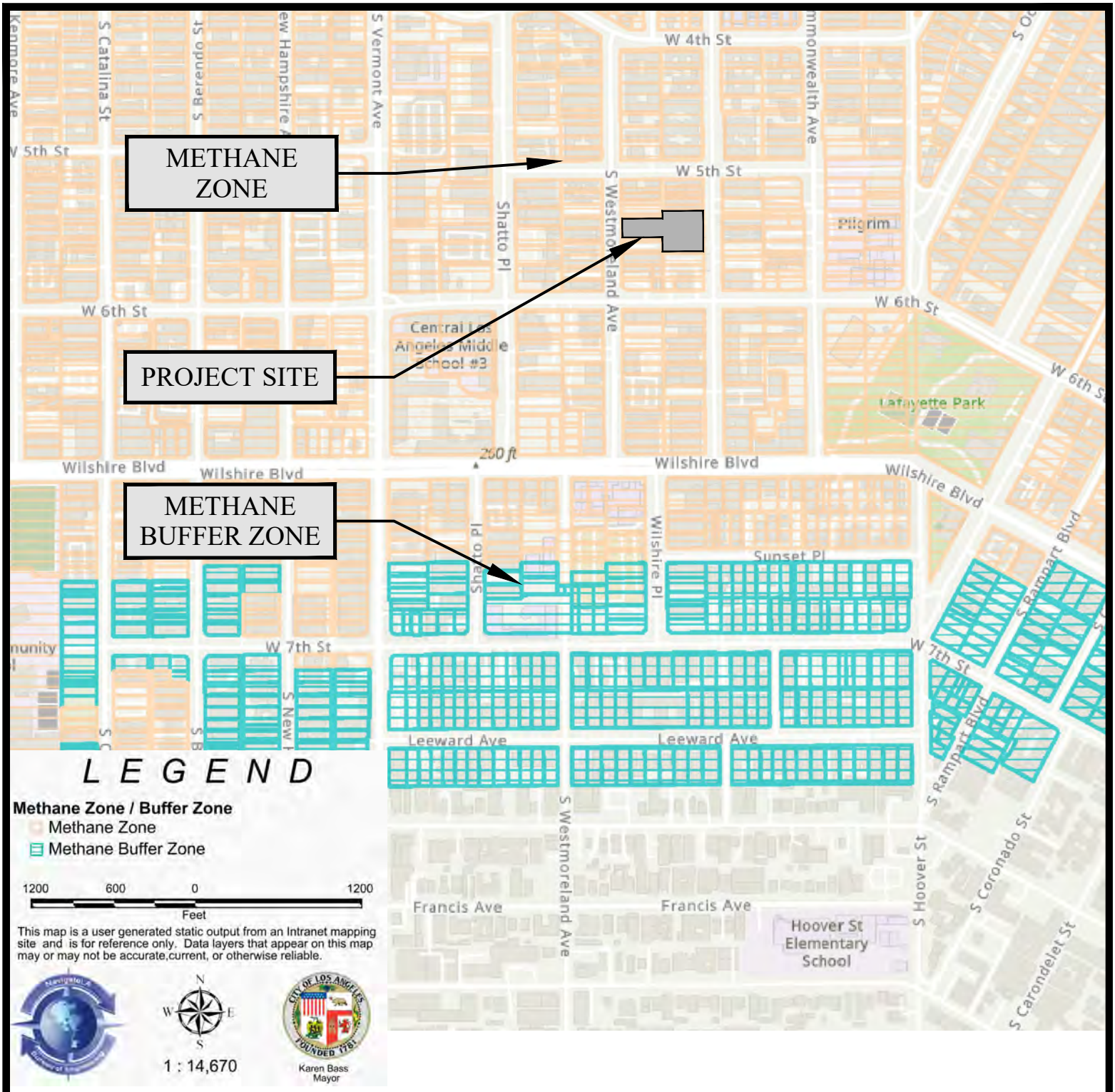
REFERENCE: CDEMD, Well Finder (https://maps.conservation.ca.gov/doggr/wellfinder/)

OIL WELL LOCATION MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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515-531 S VIRGIL AVE., 518-526 S WESTMORELAND AVE., LOS ANGELES

FILE NO: 22659



REFERENCE: City of Los Angeles, Department of Public Works, Navigate LA (https://navigatela.lacity.org/)

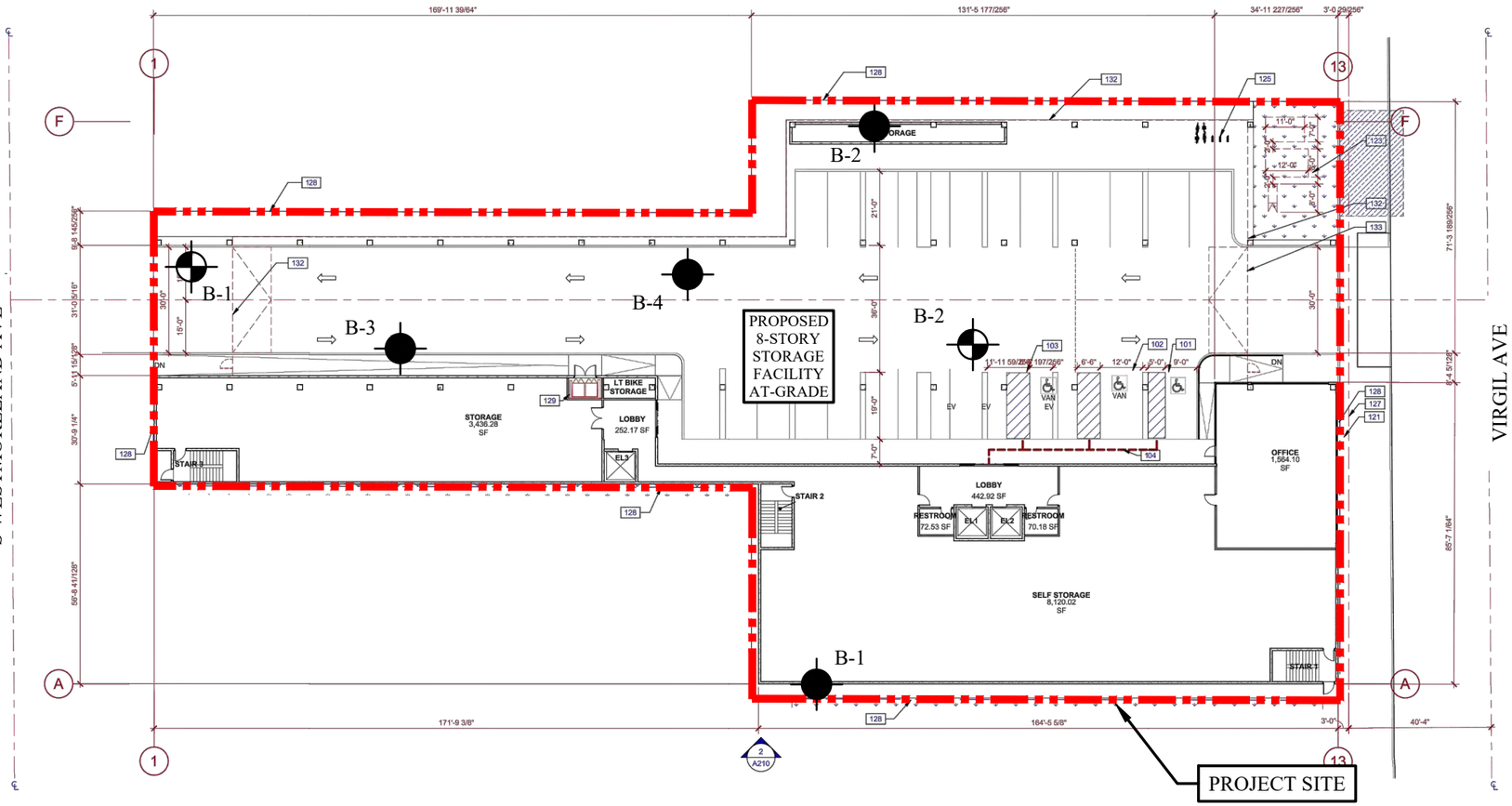
METHANE ZONE RISK MAP

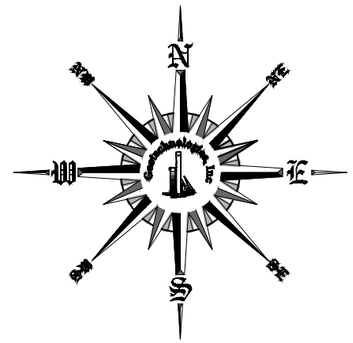
Geotechnologies, Inc.
Consulting Geotechnical Engineers

SHATTO EMET QOZB LP
515-531 S VIRGIL AVE., 518-526 S WESTMORELAND AVE., LOS ANGELES

FILE NO: 22659

Plot Date: Nov. 17, 2025
z:\shared\design cad_old\autocad\2025 projects\22659\22659 plot plan.dwg





SCALE IN FEET

0 40 80

LEGEND

B-2 — LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

B-4 — LOCATION AND NUMBER OF TEST PIT (PREVIOUS INVESTIGATION BY FEFFER GEOLOGICAL CONSULTING JOB NO. 2126-84, DATED 4/6/2018)

REFERENCE: SITE PLAN BY WARE MALCOMB DATED 10/28/2025

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

SHATTO EMET QOZB LP
515-531 S VIRGIL AVE., 518-526 S WESTMORELAND AVE., LOS ANGELES

Drawn by: JD | File No.: 22659

Date: November 2025

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BORING LOG NUMBER 1

Shatto Emet QOZB LP **Date: 05/13/25**

File No. 22659 **Method: 24-inch Bucket Boring**

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4½-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt, dark brown and gray, moist, stiff
				-		
				2 --		
2.5	2/12"	24.5	100.2	-		
				3 --		
				-		
				4 --		
				-		
5	2/6"	24.0	99.8	5 --		
				-		
				6 --		
				-	CL	OLDER ALLUVIUM: Clayey Silt to Silty Clay, grayish to yellowish and reddish brown, moist, stiff
				7 --		
				-		
				8 --		
				-		
				9 --		
				-	SM/ML	Silty Sand to Clayey Silt, dark grayish brown, moist, dense to stiff, fine grained, stiff
10	3/6"	18.4	110.8	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-	SM/SP	Silty Sand to Sand, yellowish brown, slightly moist, very dense, fine grained
				14 --		
				-		
15	4/6"	6.1	107.7	15 --		
				-	SP	Sand, yellowish brown, slightly moist, very dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	5/6"	7.5	111.5	20 --		
				-		
				21 --		
				-		
				22 --		
				-	SW	Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
				23 --		
				-		
				24 --		
				-		
25	3/6"	34.3	88.7	25 --		
				-		BEDROCK: Siltstone, dark brown, moist, moderately hard, weathered

BORING LOG NUMBER 1

Shatto Emet QOZB LP

File No. 22659

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		@27' Bedding [N60W/20S]
30	30 5/6"	39.8	76.6	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		very moist Total Depth 30 feet Water Seepage at 27 and 29½ feet Fill to 6 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 24-inch diameter Bucket Auger Kelly Bar Weight: 0-25' @ 3390lbs 25'-52' @ 2230 lbs.

BORING LOG NUMBER 2

Shatto Emet QOZB LP **Date: 05/13/25**

File No. 22659 **Method: 24-inch Bucket Boring**

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt to Silty Clay, dark brown, moist, stiff
				-		
				2 --		
2.5	2/6"	18.1	109.0	-		
				3 --		
				-		
				4 --		
				-		
5	1/6"	22.7	101.6	5 --		
				-		
				6 --		
				-	SM/ML	OLDER ALLUVIUM: Silty Sand to Clayey Silt, dark to grayish brown, moist, dense to stiff, fine grained
				7 --		
				-		
				8 --		
				-	SM/SP	Silty Sand to Sand, yellow to grayish brown, slightly moist to dense, fine grained
				9 --		
				-		
10	4/6"	7.6	108.8	10 --		
				-		
				11 --		
				-		
				12 --		
				-	SP	Sand, yellowish brown, slightly moist, very dense, fine grained
				13 --		
				-		
				14 --		
				-		
15	4/6"	11.5	102.3	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	5/6"	12.9	123.4	20 --		
				-	SW	Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	2/6"	35.4	86.5	25 --		
				-		BEDROCK: Siltstone, grayish brown, moist, moderately hard

BORING LOG NUMBER 2

Shatto Emet QOZB LP

File No. 22659

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		@27' Bedding [N70W/20S] very moist
30	5/6"	34.2	86.3	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		Total Depth 30 feet Water Seepage at 25 and 27½ feet Fill to 6 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 24-inch diameter Bucket Auger Kelly Bar Weight: 0-25' @ 3390lbs 25'-52' @ 2230 lbs.

LOG OF EXPLORATORY BORING

Job Number: 2126-84
Project:Virgil Avenue (Thomas Safran & Associates)

Date Performed: 3/15/18

Boring No:1
Boring Location: A/C Parking Lot
515-531 S Virgilr Avenue, Los Angeles
Drill Type: CME-75 8"Hollow Stem Drill Rig

Depth in Feet	Blows per 6"	Sample Type: Undisturbed	Sample Type: Bulk	Bedrock/ Soil Description	Color	Density	Moisture
0				4" Asphalt, 4" Base			
				Artificial Fill (Af) @8"Silty with little clay, low plasticity, sharp contact	Dark grayish brown 10YR 5/2	*	Moist
2.5				@2.5'Sand with little silt, sands fine to medium Silt with some sand, non plastic	Yellowish brown 10YR 5/4	*	Moist
5	10/11/20	R			Brown 10YR 4/3	Stiff	Moist
7.5				Quaternary Alluvium (Qa):			
10	16/21/50 For 6"	R		Sand with few silt, sands fine-medium	Very pale brown, 10YR 7/4	Very dense	Moist
15	35/50 For 5"	R		Bedrock: Sand trace fine gravel to 2"	Pale brown, 10YR 6/3	Very dense	Moist
17				@15' possible weathered bedrock based on 15 & 20 blow counts. Thereafter the material would be a silty sandstone		Very dense	
20	30/50 For 6"	R		Sandy siltlstone, weak cementation, sands very fine-fineGravelly silty sand and sandy silt	Very pale brown, 10 YR 7/3	Hard	Moist
25	12/25/40	R		Siltstone, abundant iron staining as veins, moderate cementation, trace clay, very fine sand	Olive gray 10 YR 5/2	Hard	Moist
30	20/32/50 For 6"	R		Siltstone, sands few & very fine-fine, moderate cementation	Dark gray 10 YR 4/1	Hard	Moist
35	20/35/50 For 6"	R		Clayey siltstone, moderate cementation	Dark grayish brown 10 YR 4/2, sparse mottling, pale brown, tan, yellow	Hard	Moist
40							

Feffer Geological Consulting

Figure

L 0 6 L 1 1 5 2 0 L 8 6 6 9 3 7

LOG OF EXPLORATORY BORING

Job Number: 2126-84
Project:Virgil Avenue (Thomas Safran & Associates)

Date Performed: 3/15/18

Boring No:1
Boring Location: A/C Parking Lot
515-531 S Virgilr Avenue, Los Angeles
Drill Type: CME-75 8"Hollow Stem Drill Rig

Depth in Feet	Blows per 6"	Sample Type: Undisturbed	Sample Type: Bulk	Bedrock/ Soil Description	Color	Density	Moisture
40	50 For 5"	R		Sandy siltstone, lamination present in 1/4"-1/2" layers,moderate cementation	Grayish brown to dark gray 2.5Y 5/25/41	Hard	Moist
45	50 For 5"	R		Sandy siltstone, lamination present in 1/4"-1/2" layers, strong cementation	Grayish brown to dark gray 2.5Y 5/25/41	Hard	Moist
46.5				End At 46.5', Artificial Fill To 7.5', No Ground Water, No Caving			
50							
55							
60							
65							
70							
75							
80							

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Figure

1 0 6 1 1 1 5 2 0 1 8 6 6 9 3 7

LOG OF EXPLORATORY BORING

Job Number: 2126-84
Project:Virgil Avenue (Thomas Safran & Associates)

Date Performed: 3/15/18

Boring No:2
Boring Location: A/C Parking Lot
515-531 S Virgilr Avenue, Los Angeles
Drill Type: CME-75 8"Hollow Stem Drill Rig

Depth in Feet	Blows per 6"	Sample Type: Undisturbed	Sample Type: Bulk	Bedrock/ Soil Description	Color	Density	Moisture
0				4" Asphalt, 3" Base Artificial Fill (Af) @7"-5' Mixed fills, clayey silt with sand	Dark gray, medium brown	*	Moist
2.5						*	
5	10/12/15	R		Clayey silt, trace fine grained sand, low plasticity	Dark grayish brown 10YR 4/2	Very stiff	Moist
7.5							Moist
10	10/17/25	R		Quaternary Alluvium (Qa) Silt-clay, low to medium plasticity	Gray, motteling 10YR 5/1	Hard	Moist
12.5				Bedrock:			
15	23/37/50 For 5"	R		Silty sandstone, sands very fine-fine with few medium sand, weak cementation	Light gray, 10YR 7/2	Very dense	Moist
20	30/50 For 6"	R		Very silty sandstone, weak cementation	Light yellowish brown, 10YR 6/4	Very dense	Very moist not wet
25				End At 21.5', Artificial Fill To 7.5', No Ground Water, No Caving			
30							
35							
40							

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Figure

1861152018869397

LOG OF EXPLORATORY BORING

Sheet 1 of 1

Job Number: 2126-84
Project:Virgil Avenue (Thomas Safran & Associates)

Date Performed: 3/15/18

Boring No:3
Boring Location: A/C Parking Lot
515-531 S Virgilr Avenue, Los Angeles
Drill Type: CME-75 8"Hollow Stem Drill Rig

Depth in Feet	Blows per 6"	Sample Type: Undisturbed	Sample Type: Bulk	Bedrock/ Soil Description	Color	Density	Moisture
0				4" Asphalt, No Base Artificial Fill (Af)			
2.5							
5	10/12/18	R		Silt with little sand, sands very fine to fine, possible native	Yellowishbrown 10Y 5/4		Moist
7.5				Quaternary Alluvium (Qa)			
10	27/50 For 6"	R		Silt with little sand, CaCo3 yellow hard veins moderate	Olive gray, yellow 5Y 5/2		Moist
12.5				Bedrock:			
15	25/50 For 6"	R		Silty sandstone, sands very fine-fine, very weak cementation	Light gray, 10YR 4/2	Very dense	Moist
20	30/50 For 6"	R		Silty sandstone, sands very fine-fine, weak cementation	Light gray, 10YR 4/2	Very dense	Moist
25				End At 21.5', Artificial Fill To 7.5', No Ground Water, No Caving			
30							
35							
40							

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Figure

1061115201866937

LOG OF EXPLORATORY BORING

Sheet 1 of 1

Job Number: 2126-84
Project:Virgil Avenue (Thomas Safran & Associates)

Date Performed: 3/15/18

Boring No:4
Boring Location: A/C Parking Lot
515-531 S Virgilr Avenue, Los Angeles
Drill Type: CME-75 8"Hollow Stem Drill Rig

Depth in Feet	Blows per 6"	Sample Type: Undisturbed	Sample Type: Bulk	Bedrock/ Soil Description	Color	Density	Moisture
0				4" Asphalt, No Base Artificial Fill (Af)			
2.5							
5	12/21/33	R		Silt with some sand, sands fine to medium, possible Af	Yellowish brown 10Y 5/4	Hard	Moist
				Quaternary Alluvium (Qa)			
7.5							
10	8/15/25	R		Silt with some sand trace clay, very fine to medium, yellow CaCo3 veins	Olive gray, yellow 5Y 5/2	Hard	Moist
12.5							
	21/50 For 6"			Bedrock:			
15		R		Silty sandstone, sands very fine-fine, weak cementation	Light gray, 10YR 7/2	Very dense	Moist
20	30/50 For 5"	R		Silty sandstone, sands very fine-fine, weak cementation	Light gray, 10YR 4/2	Very dense	Moist
				End At 21.5', Artificial Fill To 6.5', No Ground Water, No Caving			
25							
30							
35							
40							

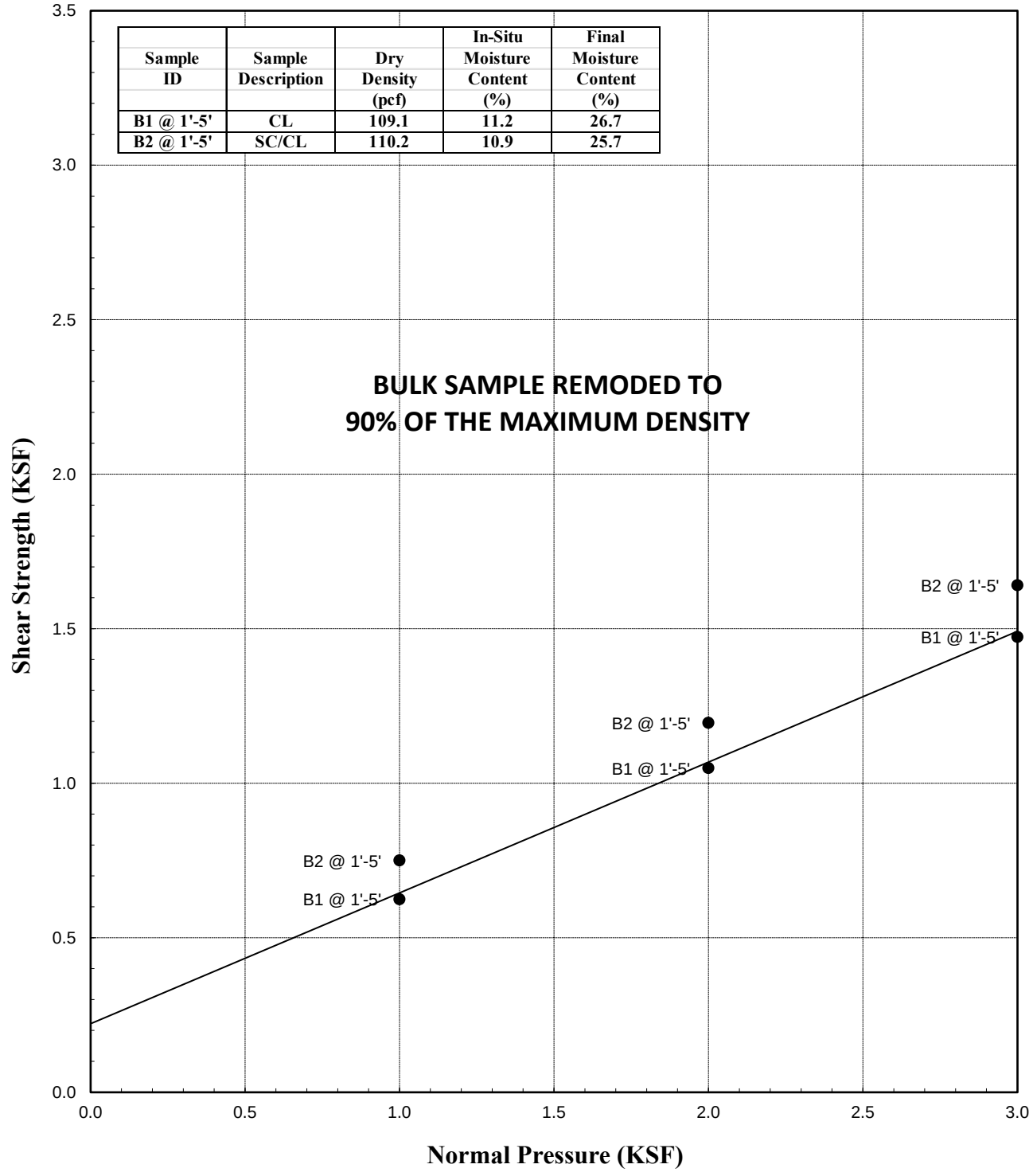
Feffer Geological Consulting

Figure

10611152018669937

Saturated Shear

Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 1'-5'	CL	109.1	11.2	26.7
B2 @ 1'-5'	SC/CL	110.2	10.9	25.7



ϕ: 23.0 degrees
c: 200.0 psf

SHEAR TEST DIAGRAM

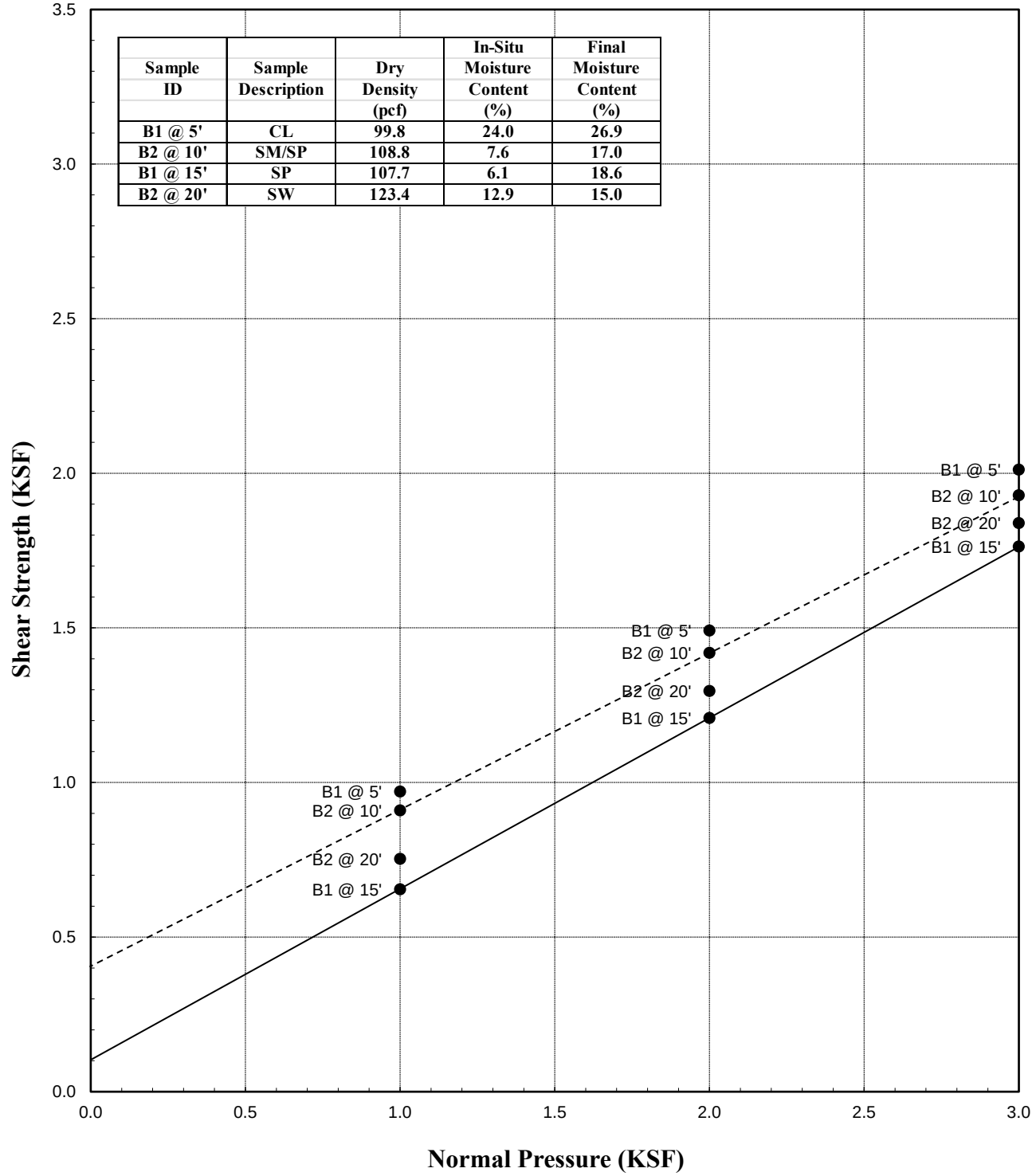
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Consulting Geotechnical Engineers

PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659 | PLATE: B-1

Saturated Shear

Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 5'	CL	99.8	24.0	26.9
B2 @ 10'	SM/SP	108.8	7.6	17.0
B1 @ 15'	SP	107.7	6.1	18.6
B2 @ 20'	SW	123.4	12.9	15.0



Shoring & RW:	ϕ:	27.0 degrees	Foundation:	ϕ:	29.0 degrees
	c:	400.0 psf		c:	100.0 psf

SHEAR TEST DIAGRAM

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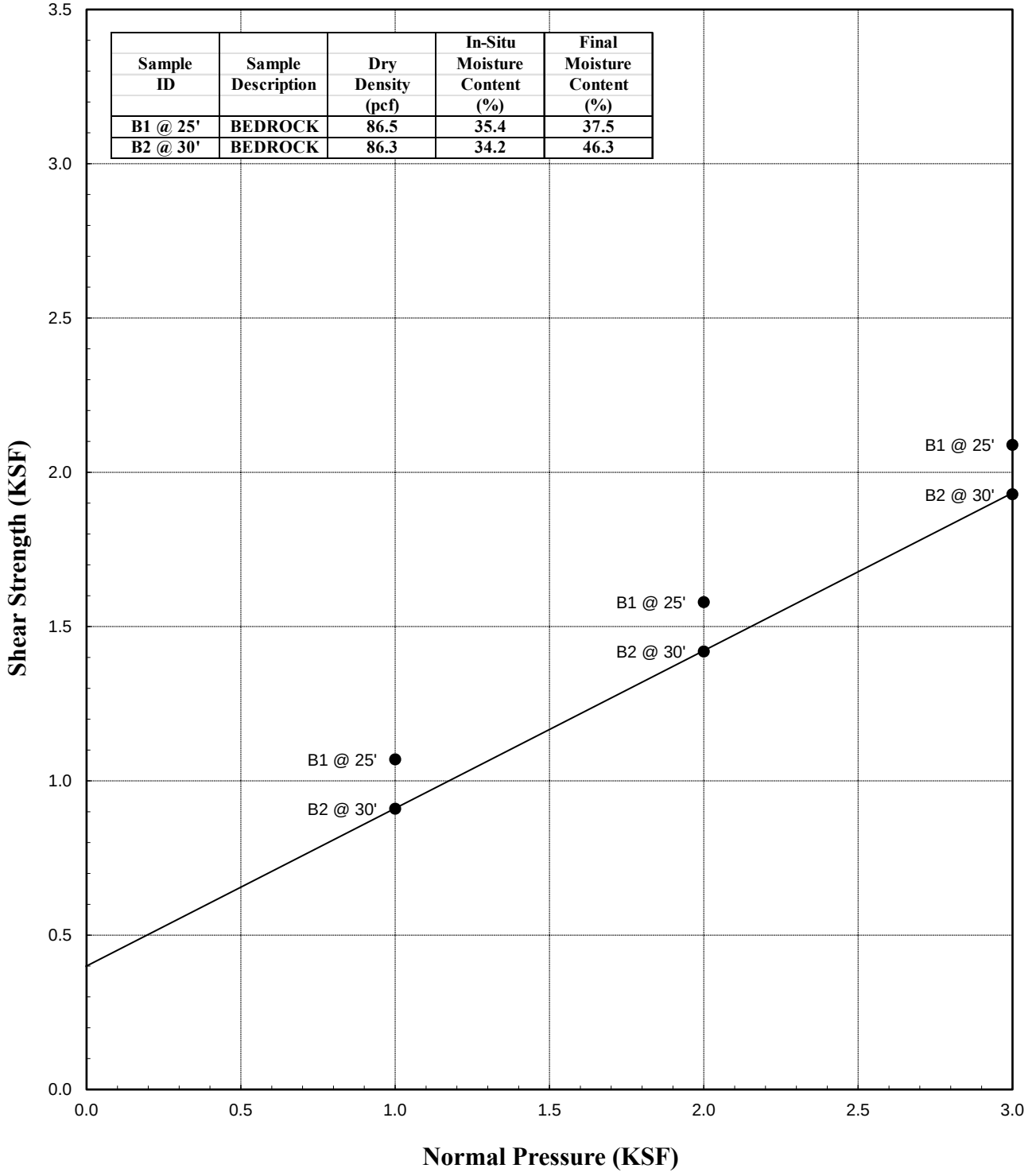
PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659

PLATE: B-2

Saturated Shear

Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 25'	BEDROCK	86.5	35.4	37.5
B2 @ 30'	BEDROCK	86.3	34.2	46.3



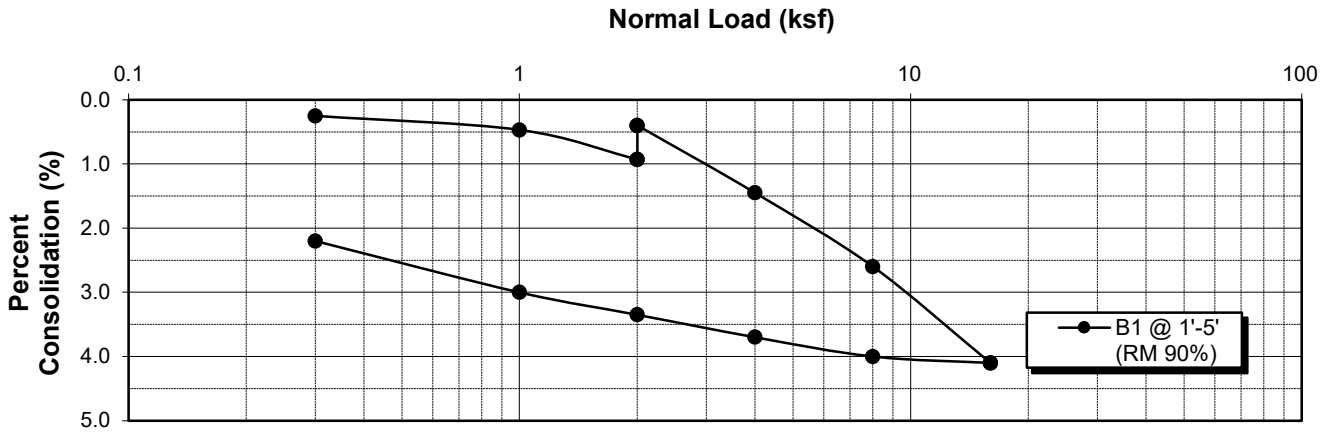
ϕ: 27.0 degrees
c: 400.0 psf

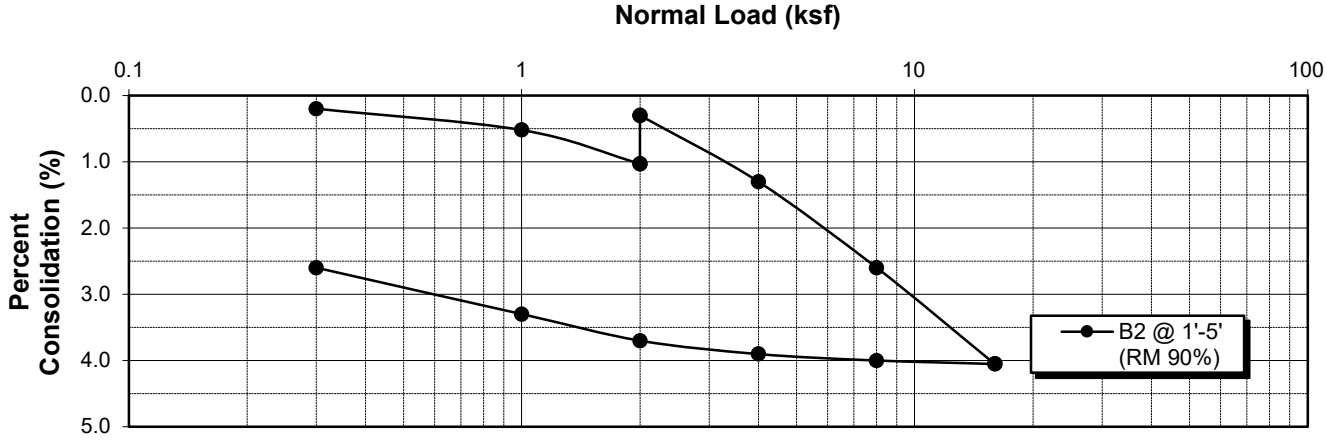
SHEAR TEST DIAGRAM

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PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659 | PLATE: B-3





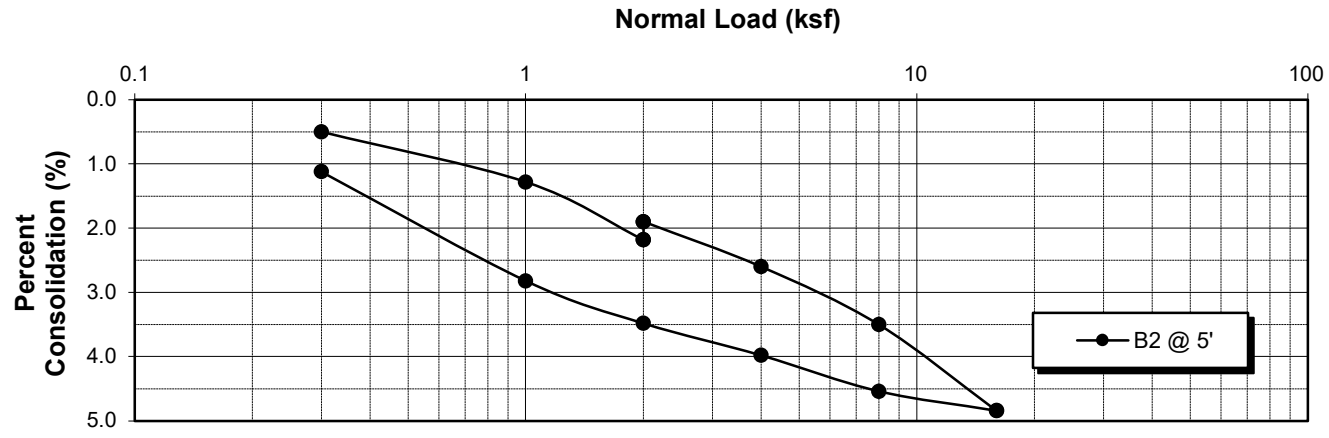
Water added at 2 KSF

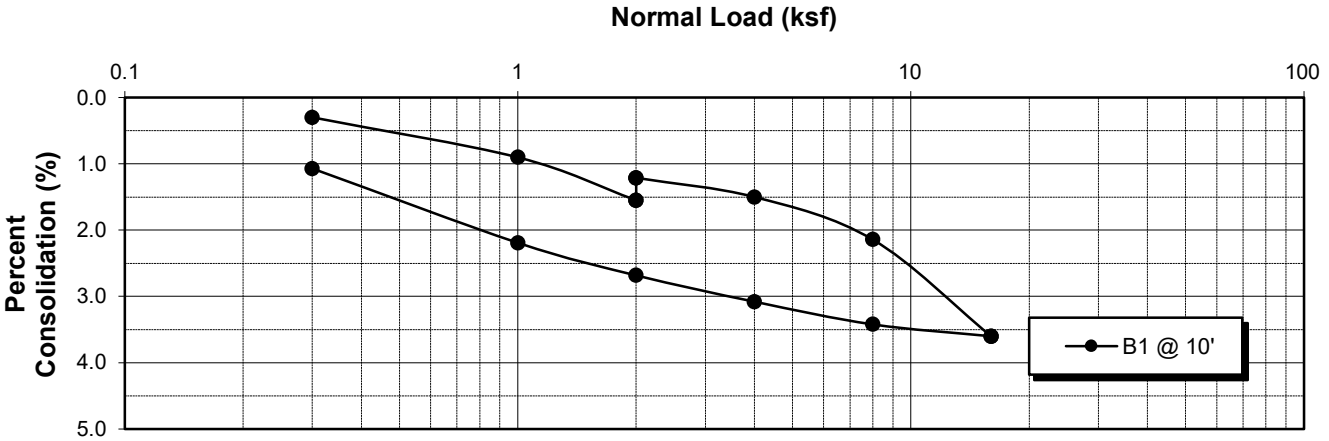
CONSOLIDATION (ASTM D2435)

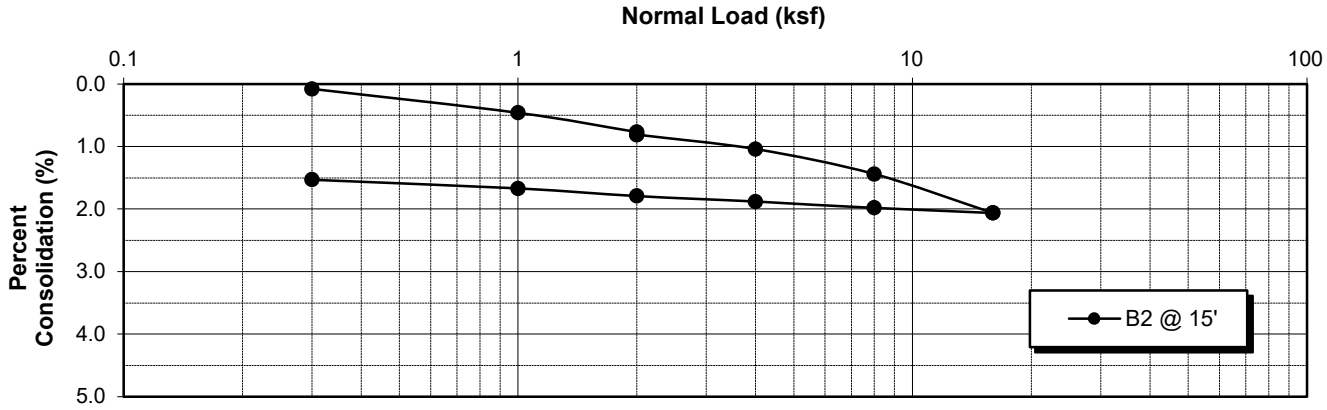
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PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659 | PLATE: C-1







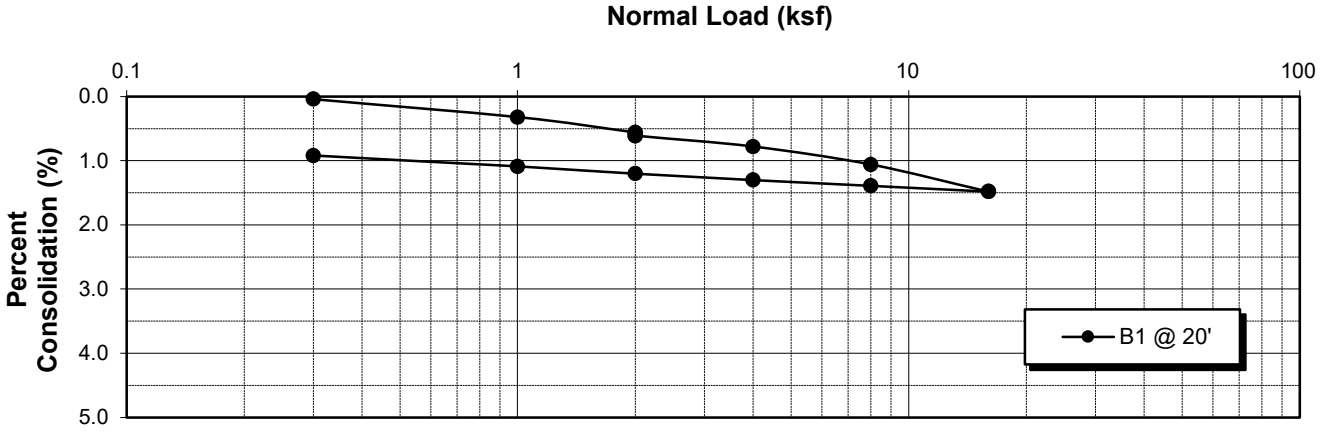
Water added at 2 KSF

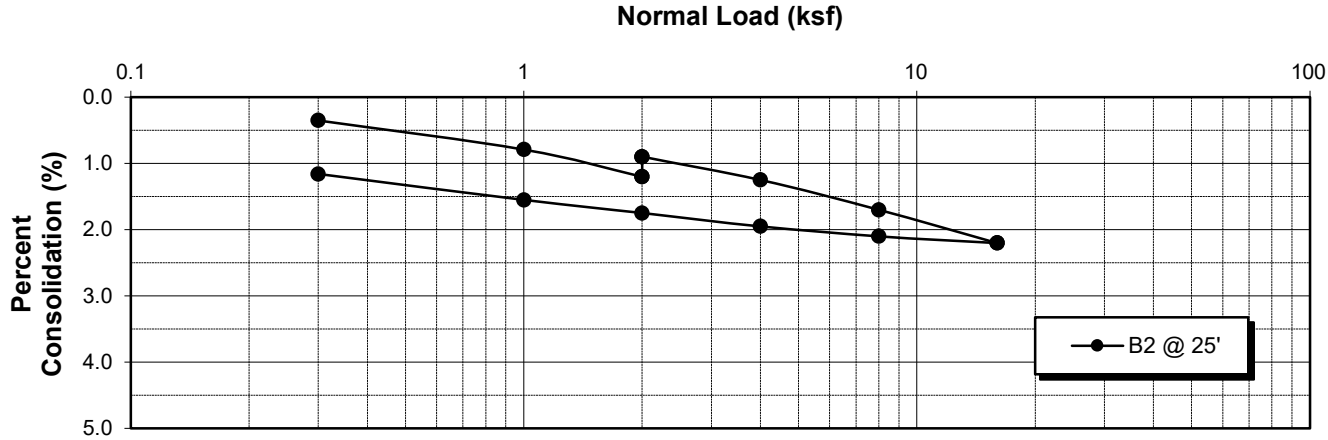
CONSOLIDATION(ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659 | PLATE: C-2





Water added at 2 KSF

CONSOLIDATION(ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: Shatto Emet QOZB LP

FILE NO.: 22659 | PLATE: C-3

LABORATORY COMPACTION CHARACTERISTICS
(ASTM D1557)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	ML	ML/CL
MAXIMUM DRY DENSITY (PCF)	121.2	122.4
OPTIMUM MOISTURE %	11.2	10.9

EXPANSION INDEX
(ASTM D4829)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	ML	ML/CL
EXPANSION INDEX UBC STANDARD 18-2	94	110
EXPANSION CHARACTER	HIGH	HIGH

SULFATE CONTENT
(CALIFORNIA TEST 417)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SULFATE CONTENT: (Percentage by Weight)	<0.1%	<0.1%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

SHATTO EMET QOZB LP
515-531 S VIRGIL AVE., 518-526 S WESTMORELAND AVE., LOS ANGELES

FILE NO: 22659 | PLATE: D

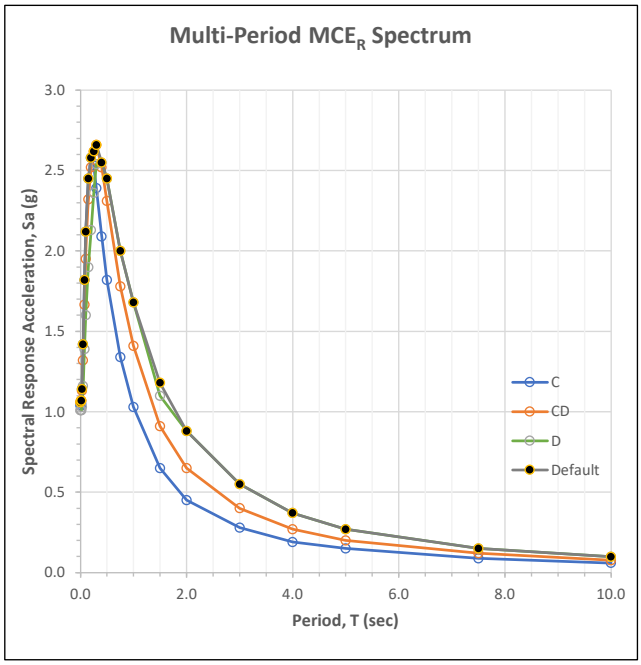
Geotechnologies, Inc.

File No. 22659
Client: Shatto Emet QOZB LP
Lattitude: 34.064503 **Longitude:** -118.287679
V_{S30}: N/A ft/s
Site Class: Default

ASCE 7-22 MCE_R and Design Spectrum

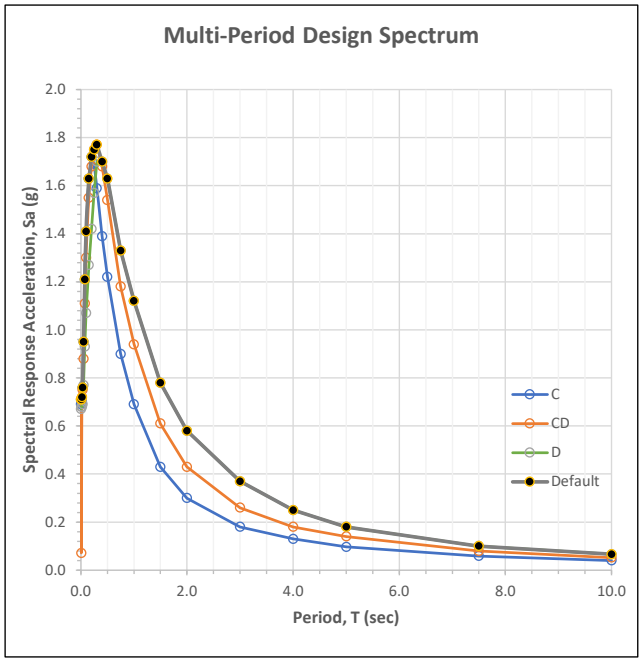
Multi-Period MCE_R Spectrum

	Site Class			
Period	**C**	**CD**	**D**	**Default**
0	1.04	1.06	1.01	**1.06**
0.01	1.04	1.07	1.01	**1.07**
0.02	1.05	1.07	1.01	**1.07**
0.03	1.14	1.13	1.03	**1.14**
0.05	1.42	1.32	1.16	**1.42**
0.075	1.82	1.666	1.39	**1.82**
0.1	2.12	1.95	1.6	**2.12**
0.15	2.45	2.32	1.9	**2.45**
0.2	2.58	2.52	2.13	**2.58**
0.25	2.53	2.62	2.36	**2.62**
0.3	2.39	2.66	2.54	**2.66**
0.4	2.09	2.52	2.55	**2.55**
0.5	1.82	2.31	2.45	**2.45**
0.75	1.34	1.78	2	**2**
1	1.03	1.41	1.68	**1.68**
1.5	0.65	0.91	1.1	**1.18**
2	0.45	0.65	0.88	**0.88**
3	0.28	0.4	0.55	**0.55**
4	0.19	0.27	0.37	**0.37**
5	0.15	0.2	0.27	**0.27**
7.5	0.088	0.12	0.15	**0.15**
10	0.06	0.077	0.099	**0.099**



Multi-Period Design Spectrum

	Site Class			
Period	**C**	**CD**	**D**	**Default**
0	0.69	0.71	0.67	**0.71**
0.01	0.69	0.071	0.68	**0.71**
0.02	0.7	0.72	0.68	**0.72**
0.03	0.76	0.75	0.69	**0.76**
0.05	0.95	0.88	0.77	**0.95**
0.075	1.21	1.11	0.93	**1.21**
0.1	1.41	1.3	1.07	**1.41**
0.15	1.63	1.55	1.27	**1.63**
0.2	1.72	1.68	1.42	**1.72**
0.25	1.69	1.75	1.57	**1.75**
0.3	1.59	1.77	1.7	**1.77**
0.4	1.39	1.68	1.7	**1.7**
0.5	1.22	1.54	1.63	**1.63**
0.75	0.9	1.18	1.33	**1.33**
1	0.69	0.94	1.12	**1.12**
1.5	0.43	0.61	0.78	**0.78**
2	0.3	0.43	0.58	**0.58**
3	0.18	0.26	0.37	**0.37**
4	0.13	0.18	0.25	**0.25**
5	0.097	0.14	0.18	**0.18**
7.5	0.059	0.079	0.1	**0.1**
10	0.04	0.052	0.066	**0.066**



Geotechnologies, Inc.

Project: Shatto Emet QOZB LP
File No.: 22659
Description: Slot Cut at Property Line

Slot Cut Calculation

Input:

Height of Slots	(H)	8.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	27.0	degrees
Cohesion of Soils	(c)	400.0	psf
Factor of Safety	(FS)	1.50	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	2000.0	psf
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
55	5.6	22	2800.8	2255.7	3045.4	10865.4	64.3
56	5.4	22	2698.0	2178.0	2907.0	10466.7	58.1
57	5.2	21	2597.6	2100.1	2773.0	10077.2	53.4
58	5.0	20	2499.5	2022.1	2643.4	9696.4	49.8
59	4.8	19	2403.4	1944.0	2517.9	9323.8	46.8
60	4.6	18	2309.4	1866.0	2396.5	8959.0	44.5
61	4.4	18	2217.2	1788.2	2278.8	8601.5	42.6
62	4.3	17	2126.8	1710.7	2164.9	8250.8	41.1
63	4.1	16	2038.1	1633.4	2054.6	7906.5	40.0
64	3.9	16	1950.9	1556.7	1947.6	7568.4	39.1
65	3.7	15	1865.2	1480.5	1843.9	7235.9	38.4
66	3.6	14	1780.9	1404.9	1743.4	6908.8	38.0
67	3.4	14	1697.9	1330.0	1646.0	6586.8	37.7
68	3.2	13	1616.1	1256.0	1551.4	6269.5	37.7
69	3.1	12	1535.5	1182.8	1459.7	5956.6	37.9
70	2.9	12	1455.9	1110.7	1370.7	5647.9	38.2
71	2.8	11	1377.3	1039.6	1284.2	5343.1	38.8
72	2.6	10	1299.7	969.8	1200.3	5041.9	39.6
73	2.4	10	1222.9	901.1	1118.7	4744.2	40.7
74	2.3	9	1147.0	833.8	1039.4	4449.6	42.1
75	2.1	9	1071.8	767.9	962.3	4157.9	43.9
76	2.0	8	997.3	703.6	887.2	3868.9	46.0
77	1.8	7	923.5	640.8	814.2	3582.5	48.7
78	1.7	7	850.2	579.6	743.0	3298.3	52.1
79	1.6	6	777.5	520.2	673.5	3016.3	56.5
80	1.4	6	705.3	462.6	605.8	2736.1	62.1

* Width of Slots to achieve a minimum o f 1.5 Factor of Saferty, with a Maximum Allowable Slot Width of 8-feet.

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

d_{allow} 37.7 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 8 Feet in Height, with a Factor of Safety Equal or Exceeding 1.5.