Appendix IS-4

Preliminary Geotechnical Engineering Investigation

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File Number 21358

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633 W. 5th Street, Suite 2600
Los Angeles, California 90071

Attention: Jeff Chang

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Mixed-Use Development
754 S. Hope Street and 609-625 W. 8th Street, 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, shoring and foundation design.

Due to the conceptual phase of the project, this report is preliminary in nature due to the lack of structural information. This report is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when additional data is available, and the development plan achieves refinement.

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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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.

Due to the conceptual phase of the project, this report is preliminary in nature due to the lack of structural information. This report is preliminary in nature, and is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when additional data is available, and the development plan achieves refinement.

This investigation included excavation of five 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 development team. The proposed project consists of demolishing the existing surface parking lot and multi-level parking structure, and constructing a 45-story mixed-use project comprised of approximately 562,696 square feet of floor area, with a maximum of 547 residential dwelling units, approximately 7,499 square feet of ground floor commercial/retail space and potentially a dedicated space to a school use ("Project").

The common open space elements of the project are provided in a tiered terrace arrangement in several locations throughout the vertical levels of the building. The tower is organized around the concept of stepped massing with multiple amenity decks located at 3 different elevations within the tower (Level 6, Level 17, and Level 31), one at each step in the massing. Each amenity level provides a mix of outdoor landscaped decks and indoor amenity rooms.

The Project includes seven levels of parking (three of which will be below-grade and four above-grade). Preliminarily, it is anticipated that the tower will either be supported on pile foundations or a mat foundation. If the tower will be supported on a mat footing, it is anticipated that the mat footing will be approximately 10 to 12 feet thick, and will have an average bearing pressure on the order of 12,000 psf. Typical column footing loads for the podium structure will be between 1,500 and 2,000 kips. Grading will consist of excavations on the order of 60 to 65 feet in depth for the proposed subterranean parking levels and foundation elements.

When the project achieves more definition, the structural loads shall be confirmed by the project structural engineer, and provided to this firm for review and analysis. 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 property is located at 754 South Hope Street, and 609-625 West 8th Street, in the City of Los Angeles, California. The project site is bounded by an 8-story building and a 4-story parking structure to the north, by Grand Avenue to the east, by 8th Street to the south, and by Hope Street to the west. The western portion of the site is currently developed with a 3-story parking structure, and the eastern portion is developed with a surface parking lot.

The site is relatively level with no pronounced highs or lows. Drainage across the site is by sheetflow to the city streets. The vegetation on the site is virtually non-existent. The neighboring development consists primarily of residential and commercial structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between December 22, 2016, and December 29, 2016, by excavating five exploratory borings. The exploratory borings varied between 65 to 150 feet in depth below the existing site grade. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers.

Geologic Materials

The explorations encountered existing fill underlain by natural alluvium. Fill materials underlying the subject site consist of silty sands and sandy silts, which are dark brown in color, slightly moist to moist, medium dense to dense, and medium firm to stiff, fine grained, and locally with abundant brick and concrete fragments. Fill thickness ranging from 3 to 6 feet was encountered in the exploratory borings.

Native soils consist of silty sand and gravelly sands, with occasional layers of sandy and clayey silts. The native soils are yellowish to dark brown and gray in color, slightly moist to moist, dense to very dense, stiff to very stiff, fine to coarse grained, with varying amount of gravel and cobbles. The native soils consist predominantly of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed soil profiles may be obtained from individual boring and logs.

Groundwater

Groundwater was encountered at a depth of 130 feet below the existing site grade in Boring Number 1. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 70 feet below the existing 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 could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

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 faults or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Seismic Hazards Maps of the State of California (CDMG, 1999), does not classify the site as part of the potentially "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake.

Groundwater was encountered at a depth of 130 feet below the existing site grade in Boring Number 1. The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 70 feet below the existing site grade.

Based on the dense nature of the underlying soils, and the depth to historic highest groundwater level, the potential for liquefaction occurring at the 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 structure.

Some seismically-induced dry settlement of the proposed and existing improvements should be expected as a result of strong ground-shaking. However, due to the uniform nature of the upper earth 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.

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 does not lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Due to the conceptual phase of the project, this report is preliminary in nature due to the lack of structural information. This report is preliminary in nature, and is therefore not intended for submission for building permit purposes. A comprehensive report should be prepared when additional data is available, and the development plan achieves refinement.

Between 3 and 6 feet of existing fill materials was encountered during exploration at the site. Due to the variable nature and the varying depths of the existing fill materials, the existing fill materials are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill.

The proposed development will be constructed over 3 subterranean parking levels. It is anticipated that excavations on the order of 60 to 65 feet in depth will be required for the proposed subterranean parking levels including the foundation elements. Excavation of the proposed subterranean levels will remove the existing fill materials and expose the underlying dense native soil.

Due to the conceptual design stage of the proposed development, the structural loading of the development is not available. When the project achieves more definition, the structural loads shall be confirmed by the project structural engineer, and provided to this firm for review and analysis. The design information provided in this report shall be considered preliminary and are subject to be confirmed and/or modified when the structural loads are available.

Preliminarily, depending on the structural loading, the resulting settlement, and the effects of settlement on the adjacent neighboring structures, the proposed tower may either be supported on a system of Auger Cast Pile foundation system or a mat foundation bearing in the underlying dense native soil. The podium structure may be supported on conventional foundations bearing in the underlying dense native soils.

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

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

SEISMIC DESIGN CONSIDERATIONS

Seismic Velocity Measurements

A downhole seismic velocity measurement was performed by GeoPentech at the project site. The result of the seismic velocity measurements is presented at the end of this report. According to the seismic downhole results, an average shear wave velocity of 1,470 feet/second was measured between 0 and 100 feet, and an average shear wave velocity of 2,040 feet/second was measured between 50 and 150 feet.

2016 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class C, which corresponds to a “Very Dense Soil or Soft Rock” 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	C
Mapped Spectral Acceleration at Short Periods (S_S)	2.349g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.349g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.566g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.825g
Site Coefficient (F_v)	1.3
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.072g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.715g

FILL SOILS

The maximum depth of fill encountered on the site was 6 feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in moderate expansion range. The Expansion Index was found to be between 74 and 98 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs-on-Grade" sections of this report.

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 source of natural sulfate minerals in soils includes 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.

HYDROCONSOLIDATION

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

The underlying native soils are very dense, and contain abundant slate fragments. Soil samples collected from the underlying native soils are subject to a very minor degree of hydroconsolidation strains, on the order of 0.1 percent. The property owner shall maintain proper drainage of the subject site throughout the life of the structure. All utility and irrigation lines and drainage devices should be checked periodically and maintained. In addition, landscape irrigation should be properly controlled, in order to reduce the amount of water infiltration into the underlying soils, which provide support to the proposed structure. The Site

Drainage section below should be followed and implemented into the final construction documents.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane and Methane Buffer Zones map. Based on this review it appears that the subject property is located within a Methane Buffer Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane (Buffer) Zone designation. A copy of the portion of the map covering the Project Site is included herein.

GRADING GUIDELINES

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

Site Preparation

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

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

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

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

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

Compaction

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

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

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

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

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

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

Preliminarily, depending on the structural loading, the resulting settlement, and the effects of settlement on the adjacent neighboring structures, the proposed tower may either be supported on a system of Auger Cast Pile foundation system or a mat foundation bearing in the underlying dense native soil. The podium structure may be supported on conventional foundations bearing in the underlying dense native soils. When the project achieves more definition, the structural loads shall be confirmed by the project structural engineer, and provided to this firm for review and analysis.

AUGER CAST PILE (ACP) DESIGN

The proposed tower may be supported on a system of Auger Cast Pile (ACP). ACP piles are created by rotating a continuous flight auger into the ground to a specified depth. Subsequently, the augers are slowly withdrawn while cementitious grout is pumped under pressure to create the pile as the auger is being retracted. As the auger is retracted, the spoils on the auger are continuously removed by a small excavator or loader. Once the pile has been grouted and auger completely removed from the pile, reinforcing cages will then be wet set into the previously placed concrete.

The ACP piles are installed by using a closed tip auger tool. The proposed piles shall be a minimum of 18 inches in diameter, and shall be drilled to derive support from the underlying dense native soils. The following table presents the allowable axial capacity (with a minimum safety factor of 2) for design using the ACP piles with a minimum of 50-foot embedment into the underlying native soils.

Depth of Embedment (feet)	Allowable Axial Capacity (kips)*
50	225

* Uplift capacity may be designed using 50 percent of the downward capacity.

An indicator pile program, including compression and tension load tests, shall be performed at the site to verify the pile design capacities. The allowable pile capacities presented herein are considered to be preliminary, and are subject to be confirmed or modified depending on the results of the indicator pile load test program. In addition to the indicator pile program, Low Strain Pile Integrity Tests (PIT) shall be performed on a minimum of 10 percent of the production piles to verify the structural integrity of the piles. A more detailed specification is presented in the Appendix of this report.

A one-third increase may be used for transient loading such as wind or seismic forces. Allowable uplift capacity may be designed using 50 percent of the allowable downward capacity indicated in the above table. For ultimate compression and tension design, the pile capacities may be doubled.

Where pile groups are required, the piles should be spaced a minimum of 3 diameters on centers. If so spaced, there will be no reduction in the downward or upward capacity of the piles due to group action.

Lateral Design

Lateral loads may be resisted by the piles, and by the passive resistance of the soils against the pile caps. The passive resistance of the soils against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in this value may be used for wind or seismic loads. The resistance of the piles and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

Maximum recommended allowable lateral capacities for 0.5-inch deflection for single, isolated, fixed-head and free-head piles are presented in the Appendix. No factors of safety have been applied to the lateral load values calculated to induce the calculated lateral deflection. Lateral capacities provided are for concrete piles embedded into the underlying native soils encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 pounds per square inch (psi).

Single isolated piles may be classified as piles spaced at or greater than 8 widths on center. For pile groups where piles will be spaced closer than 8 diameters on center in the direction of loading, the following reduction factor may be utilized to determine the allowable lateral pile capacities to maintain the 0.5-inch pile deflection.

Pile Spacing	Percentage of Lateral Passive Resistance
7B	85%
6B	70%
5B	55%
4B	40%
3B	25%

Where B is the diameter of the proposed piles

The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Pile Foundation Settlement

The maximum settlement of pile-supported foundations is not expected to exceed ½ inch. Differential settlement is not expected to exceed ¼ inch.

Piling Equipment

The piling equipment used for the project shall conform to the specifications below.

- *Piling Rig* – The contractor shall use equipment of adequate torque, crowd force, and power, to achieve the design tip elevation. As a minimum, the piling rig shall be capable of providing a minimum torque of 150,000 ft-lbs, and 25 tons of down crowd thrust.
- *Automated Monitoring Equipment* – The drilling rig shall be equipped with an automated monitoring equipment (AME) designed to monitor the pile installation process. During the drilling process, the AME shall record auger depth, drill torque, and elapsed time. During the grouting process, the AME shall record the auger depth, grout pressure, and elapsed time.
- *Augers* – The augers shall be capable of creating a minimum 18-inch diameter pile.
- *Grouting Equipment* – A grout port shall be located near the tip of the displacement auger. A continuous system of grout mixing, pumping, and agitating equipment shall be utilized. Equipment shall be maintained in good working order to maintain a continuous flow of concrete during auger withdrawal. The grout pump shall be capable of developing displacement pressures of 250-psi.

Pile Installation Procedures

The following installation procedures may be followed to install the CFA piles.

1. Contractor is responsible for using equipment of adequate torque, crowd, and power to achieve the design tip elevation. The piling rig and the flight augers used for the production pile installation shall be of identical design to that used for the indicator pile test program.

2. The flight auger is advanced until it reaches the design tip elevation. The grout port in the auger tool shall be closed with a plug that prevents soil and/or water from entering the hollow shaft while the auger is advanced into the ground.

3. The flight auger shall be capable of creating a smooth walled shaft with a minimum of 18 inches in diameter (both test piles and production piles shall be a minimum of 18 inches in diameter).

4. A minimum delivery pressure of 250 psi plus the hydraulic pressure developed by the grout column in the drill stem shall be applied to create the pile. The operator shall

maintain positive rotation of the displacement auger continuously throughout the grouting process until the displacement element is completely retracted from the ground.

5. The piling rig shall be equipped with automated monitoring equipment (AME) to record the auger depth, drill torque, grout pressure, and elapsed time. All recorded data shall be provided for review.

6. Once the grouted pile shaft is filled with concrete, the steel reinforcing cage shall be inserted into the wet concrete pile. All reinforcing elements shall be fitted with centralizers or clip spacers.

Indicator Test Pile Program

An indicator pile test program must be performed and approved by the City of Los Angeles prior to installation of the production piles. The number of indicator test piles shall be a minimum of 2 test piles, or equivalent to 1 percent of the total number of production piles, whichever is greater. Pile load tests shall be performed from the proposed subgrade elevation.

Compression load tests will be performed on all indicator test piles. Axial compressive load test shall be performed in accordance with ASTM D1143. The test piles and reaction piles shall be considered sacrificial and shall not be utilized for foundation support of the proposed buildings.

The allowable pile capacities and pile lengths presented herein are subject to be confirmed, or altered depending on the results of the indicator pile load test program. Additional foundation piles may be necessary if the actual load tests do not meet the recommended allowable loads presented in this report.

Below is a summary of the indicator pile load test program. Detailed test requirements are presented in Table 1 of the Specification for Auger Cast Grouted Piles.

- The number of indicator test piles shall be a minimum of 2 test piles, or equivalent to 1 percent of the total number of production piles, whichever is greater.

- Load tests shall be performed on sacrificial test piles in accordance with ASTM D1143 (Axial Compressive Load). The design load shall be held until the measured creep does not exceed 0.005 inch per hour. Piles with a settlement rate exceeding 0.005 inch/hour under the design load during a pile test will be rejected.
- Pile load tests shall be performed to a minimum load equivalent to the ultimate capacity, which is two times the allowable capacity.
- Test piles and reaction piles shall be sacrificial and shall not be incorporated as foundation piles. Sacrificial test piles and reaction piles shall be cut off 3 feet below the finished grade and abandoned in place following the completion of the testing program.
- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.
- One test pile shall be exhumed from the ground to physically examine the pile integrity.
- Results of the pile load testing will be submitted as a summary letter to the LADBS Grading Division for review and approval.

<u>Geotechnical Pile Inspections</u>

During pile installation, a City of Los Angeles Deputy Grading Inspector shall record and maintain data for each pile, including the following:

- Pile Number
- Installed pile length
- Auger torque vs. depth
- Head pressure inside the tremie pipe vs. depth
- Drilling rate vs. depth
- Concrete volume vs. depth
- Unanticipated site conditions if any

<u>Non-Destructive Testing</u>

None-destructive testing methods shall be employed to evaluate the integrity of the piles installed to provide quality control and assurance of the pile construction method.

- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.

- Low Strain Pile Integrity Tests (PIT) shall be performed on 10 percent of the production piles.
- If any PIT test indicates a discontinuity within a tested pile, that pile shall be evaluated by the geotechnical and structural engineers. Unsatisfactory piles may be abandoned in place and shall be replaced with replacement piles.

MAT FOUNDATION AND CONVENTIONAL FOUNDATION DESIGN

Mat Foundation

Depending on the structural loading, the resulting settlement, and the effects of settlement on the adjacent neighboring structures, it may be possible to support the proposed tower on a mat foundation bearing in the underlying dense native soil.

The proposed tower will be constructed over 3 subterranean parking levels extending on the order of 45 to 50 feet below the existing site grade, including the foundation elements. Preliminarily, it is anticipated that an average bearing pressure for the tower mat foundation will be on the order of 12,000 psf. Foundation bearing pressure will vary across the mat footing, with the highest concentrated loads located at the central cores of the mat foundation.

Given the size of the proposed mat foundation, these average bearing pressures are well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average allowable bearing pressure of 12,000 pounds per square foot may be utilized. 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
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

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

Conventional Foundation

Continuous foundations may be designed for a bearing capacity of 4,000 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 4,500 pounds per square foot, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade and 24 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 200 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 10,000 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 structure, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. The client should be aware that miscellaneous structures constructed in this manner may potentially be damaged and will require replacement should liquefaction occurs during a major seismic event.

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 0.30 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 2,500 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

It is anticipated that total settlement on the order of 3½ inches will occur below the more heavily loaded central core portions of the mat foundation beneath the residential tower. Settlement on the edges of the mat foundation is expected to be on the order of 1½ to 1¾ inch. A more detailed settlement analysis will be required when the project achieves more definition and the structural loads become available.

The maximum settlement of a typical column footing below the podium structure is expected to be less than ¾ to 1 inch.

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

Foundation Observations

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

RETAINING WALL DESIGN

Cantilever retaining walls 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. 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)
65 feet	52½ pcf	60 pcf

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

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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 25 pounds per cubic foot. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Surcharge from Adjacent Structures

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

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2008-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

where:

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

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

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 Drainage

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

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are 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 density obtainable by the ASTM D1557.

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

Where retaining walls are to be constructed adjacent to property lines, there is usually not enough space for placement of a standard perforated pipe and gravel drainage system. Under these circumstances, 2-inch diameter weepholes may be placed at the 8 feet on center along the base of the wall. The wall shall be backfilled with a minimum of 1 foot of gravel above the base of the retaining wall. The gravel may consist of three-quarter inch to one inch crushed rocks.

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.

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 ASTM D1557. 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

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered during exploration at a depth of 130 feet in Boring Number 1, which corresponds to approximately 80 feet below the base of the proposed structure. Therefore the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 10 gallons per minute may be assumed.

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 65 feet in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, or structures should be shored.

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

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

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

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

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.30 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 450 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

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

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

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

Lagging

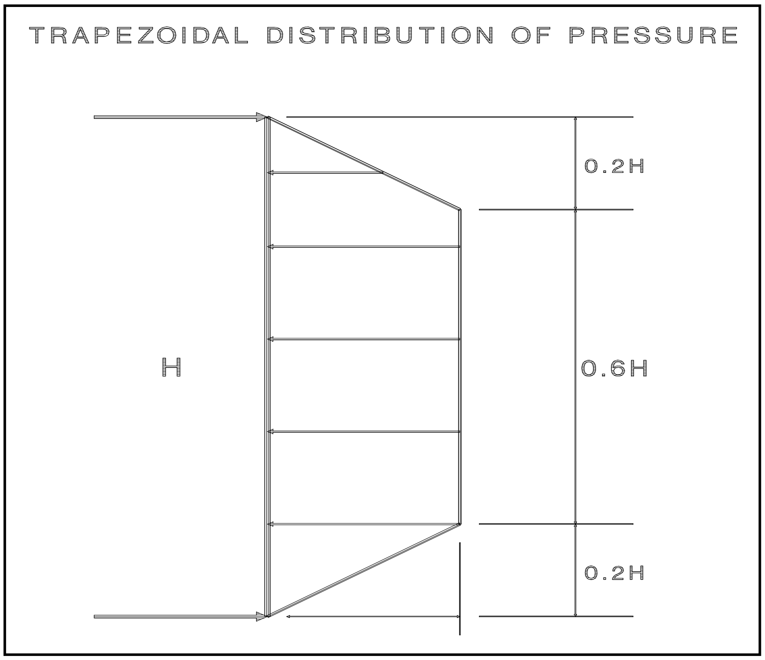
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less.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

Lateral Pressures

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

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

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



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

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

Tied-Back Anchors

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

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

It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity. The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied.

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

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation.

Monitoring

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

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

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. 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. 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 15 feet should not be exceeded. Lesser spacing would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

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

Slab Reinforcing

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

PAVEMENTS

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

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

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

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

SITE DRAINAGE

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

All site drainage, 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

Introduction

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

Percolation Testing

In order to establish a percolation rate for the site soils, Boring Number 5, which was excavated with the aid of a hollow-stem drill rig to a depth of 80 feet, was utilized for a percolation test. A 2-inch diameter casing was placed within the center of the borehole for the purpose of performing a percolation test. The casing consisted of solid PVC pips between the ground surface and a depth of 70 feet, and perforated pipes below a depth of 70 feet. The boring was presoaked for a minimum of 2 hours prior to the test. After the presoak, the boring was refilled with water and the absorption of the soils was measured.

Based on results of the percolation tests, a percolation rate of 45 feet per hour was obtained. Using a safety factor of 3, an infiltration rate of 15 feet per hour may be utilized for design purposes. It is recommended that stormwater should only percolate into native soils.

The Proposed System

It is recommended that a deep “dry well” type stormwater infiltration system be utilized for the proposed development. The proposed stormwater infiltration should be designed to infiltrate below the western portion of the building where the structural loads will be less. The edge of the proposed infiltration system shall maintain minimum horizontal distance of 5 feet from any property line, public right of ways, and foundations. In addition, the proposed stormwater infiltration system shall be designed to infiltrate a minimum of 20 feet below the base of the proposed foundation system, or below a depth of 85 feet below the existing site grade, whichever is greater.

Native soils, consisting of silty sands to poorly graded and well graded sands, were generally encountered below a depth of 85 feet below the existing site grade. The granular soils encountered on the site should allow stormwater to percolate in a generally vertical manner. Therefore, there is no potential for creating a perched water condition.

The soils encountered below the base of the proposed structure are in the very low to low expansion range. The onsite soils are not susceptible to significant hydroconsolidation. Stormwater infiltration should not cause any damage or settlement to any building. The site is not located in a hillside area and no slopes are nearby.

Groundwater was encountered at a depth of 130 feet below the existing site grade. Many building officials have decided that stormwater should not be infiltrated within 10 feet of the existing or historically high groundwater level. Therefore, the bottom of the proposed stormwater infiltration system should not exceed a depth of 120 feet below the existing site grade.

Recommendations

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

- Open infiltration basins have many negative associated issues. Such a design must consider attractive nuisance, impacts to growing vegetation, impacts to air quality and vector control.

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

- All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

- Excavations proposed for the installation of stormwater facilities should comply with the "Temporary Excavations" sections of this (the referenced) reports well as CalOSHA Regulations where applicable.

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

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

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

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

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15. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.

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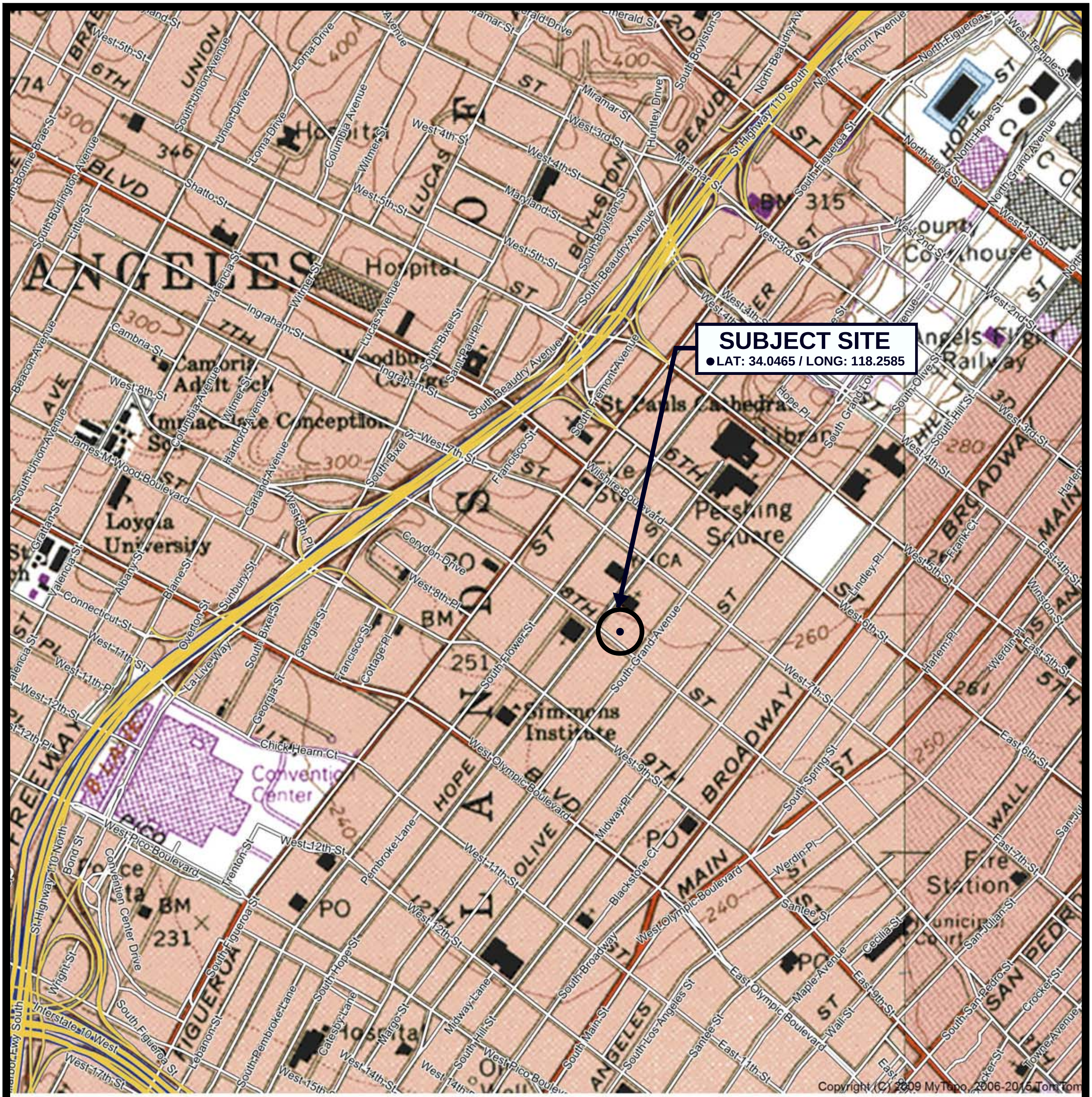


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FFFT

N

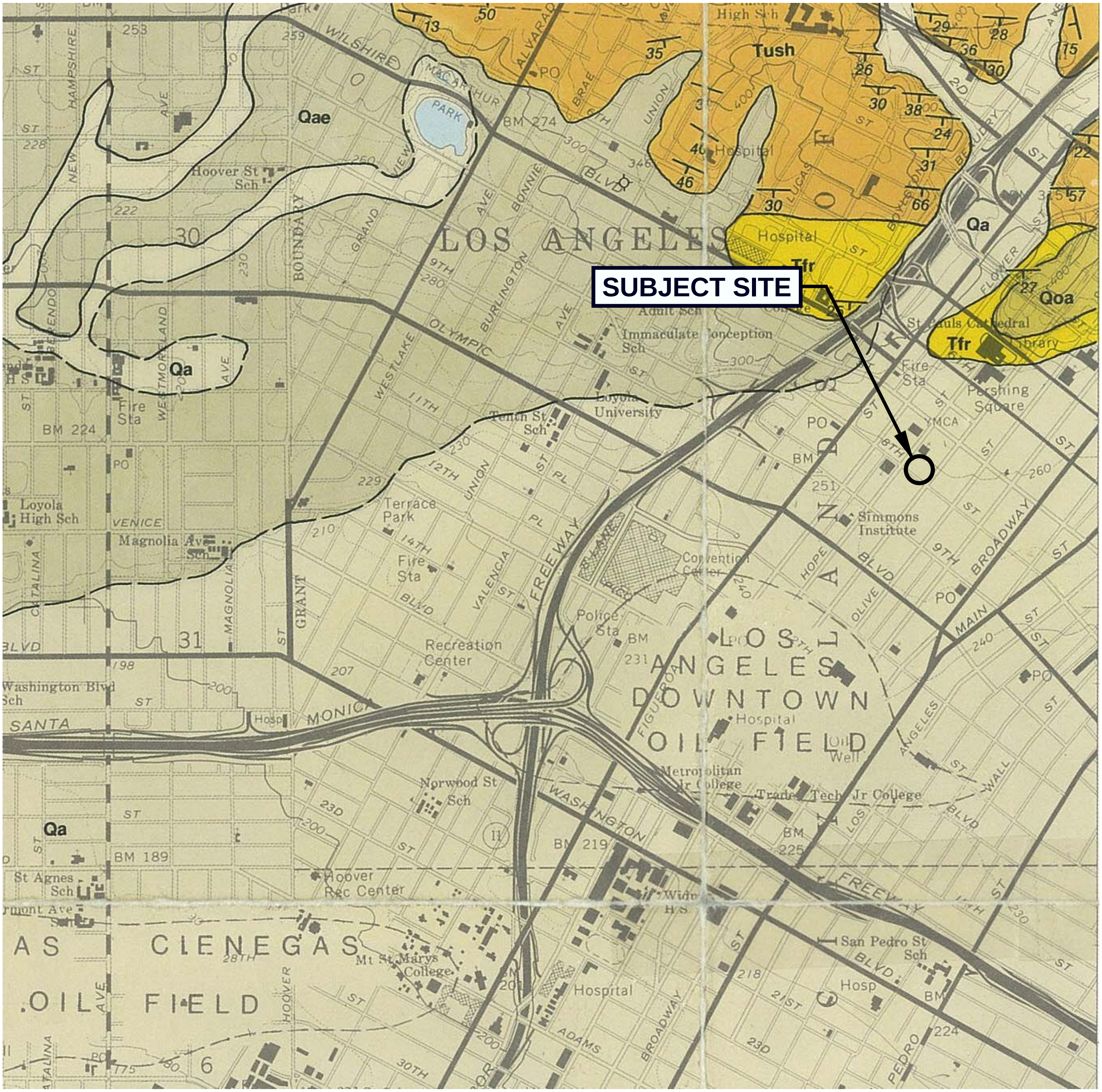
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MITSUI FUDOSAN

FILE NO. 21358



SCALE IN FEET
0 500 1000 2000

LEGEND

Qa: Surficial Sediments - alluvium; gravel, sand and clay
Qae: Older Surficial Sediments - similar to Qa, but slightly elevated and dissected
Qoa: Older Surficial Sediments - older alluvium, gray to light brown pebble-gravel, sand, silt, and clay, similar to Qae
Tush: Unnamed Shale - gray to light brown, thin-bedded silty clay shale, soft and crumbly
Tush: Fernando Formation - Repetto Member (Lamar 1970): consolidated but crumbly gray to greenish-gray claystone-siltstone
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: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE HOLLYWOOD & BURBANK (SOUTH HALF) QUADRANGLES (#DF-30)

LOCAL GEOLOGIC MAP - DIBBLEE

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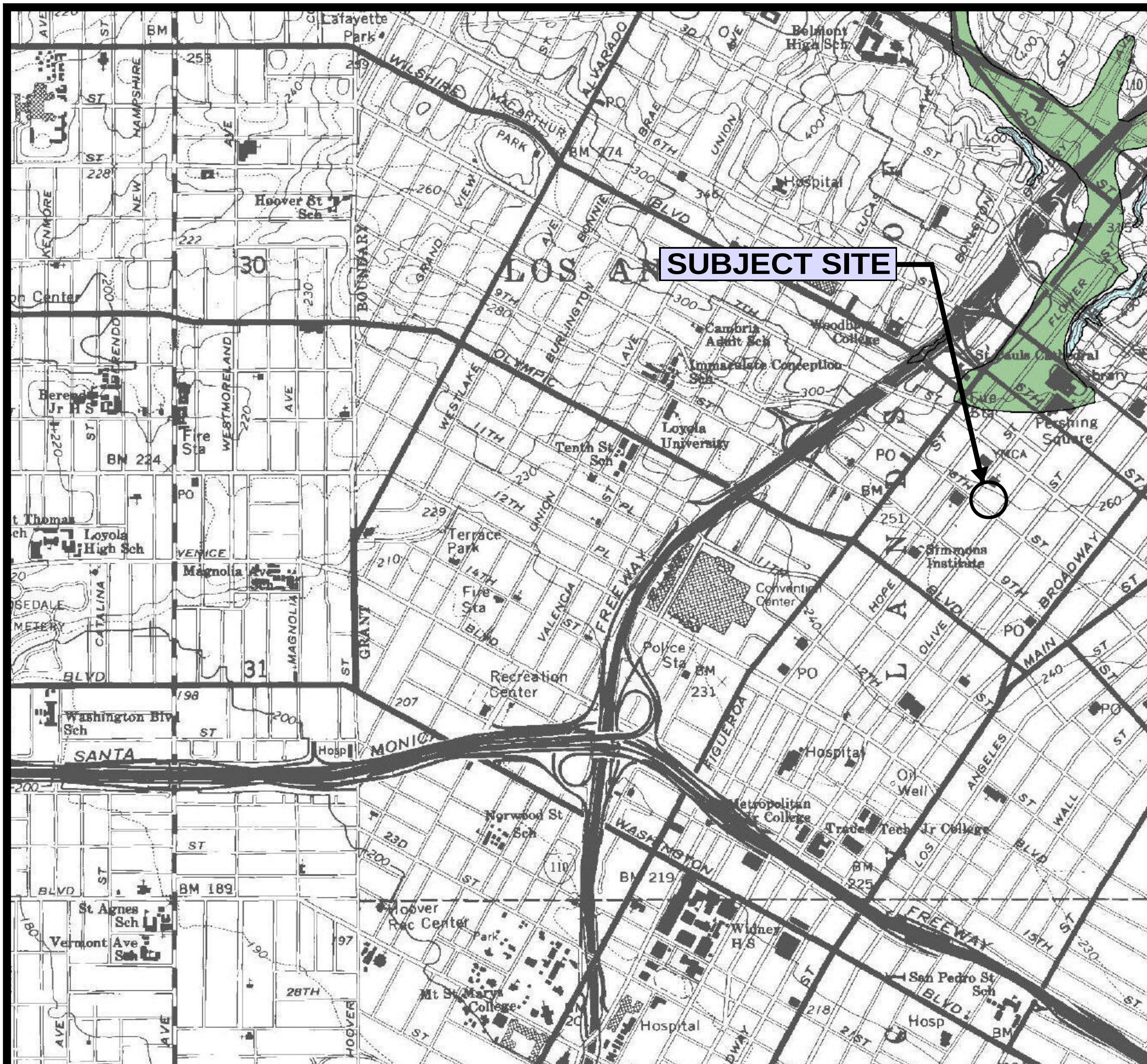
FILE NO. 21358



SCALE 1:24,000

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

N

Geotechnologies, Inc

LIQUEFACTION AREA

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

SEISMIC HAZARD ZONE MAP

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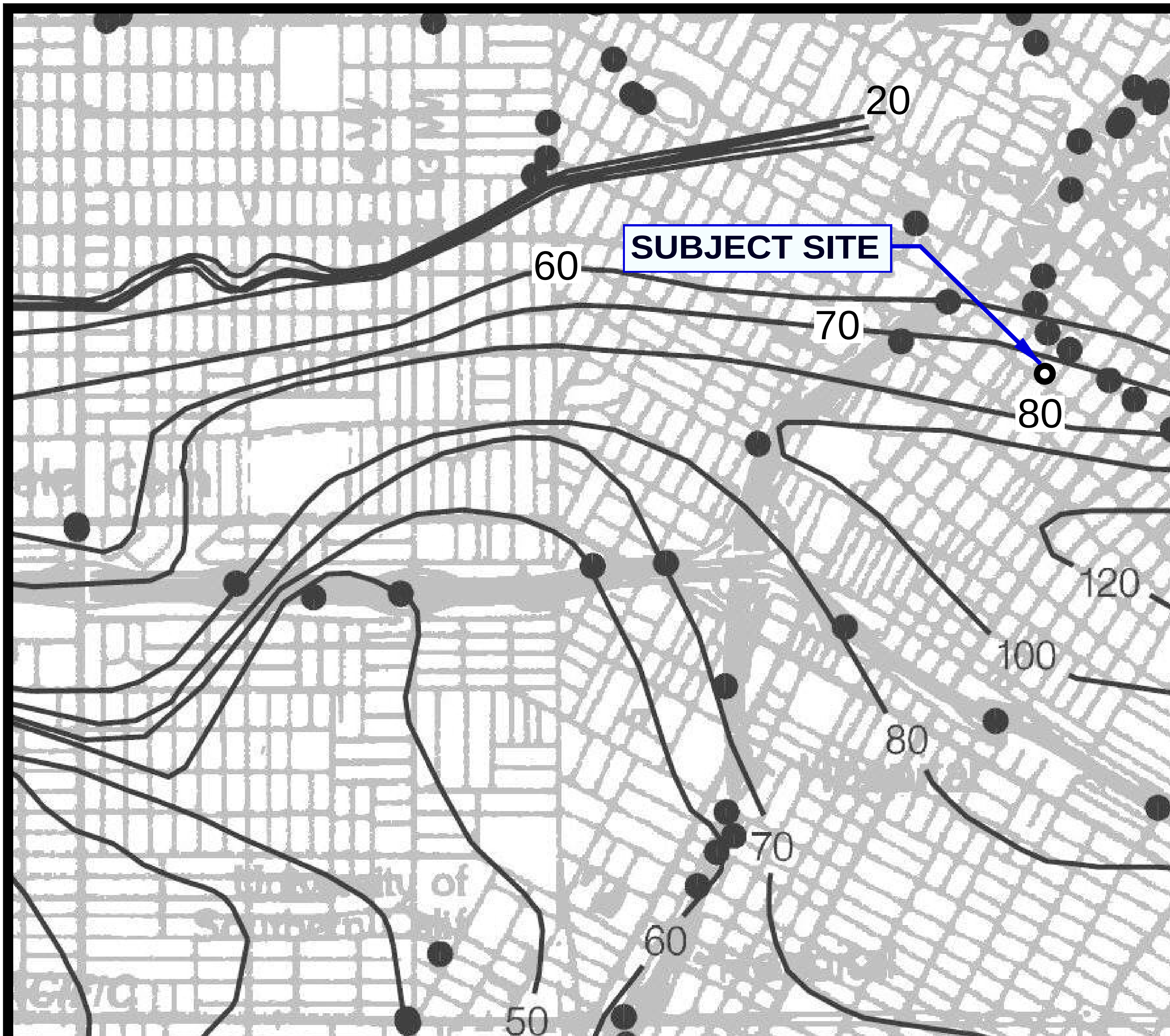
FILE NO. 21358



ONE MILE

SCALE

N

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

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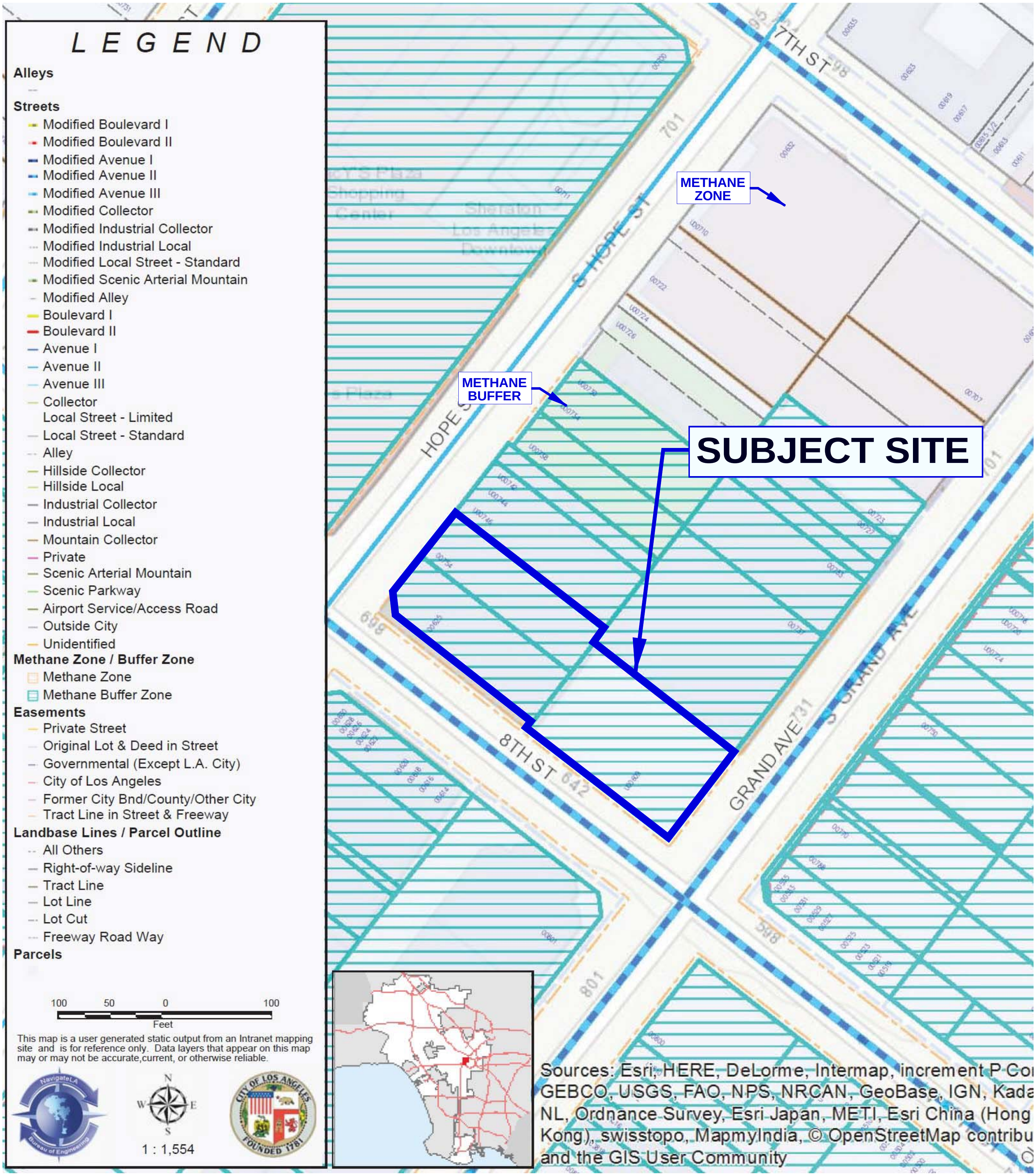
FILE NO. 21358



REFERENCE: http://navigatela.lacity.org/NavigateLA/

METHANE ZONE RISK MAP

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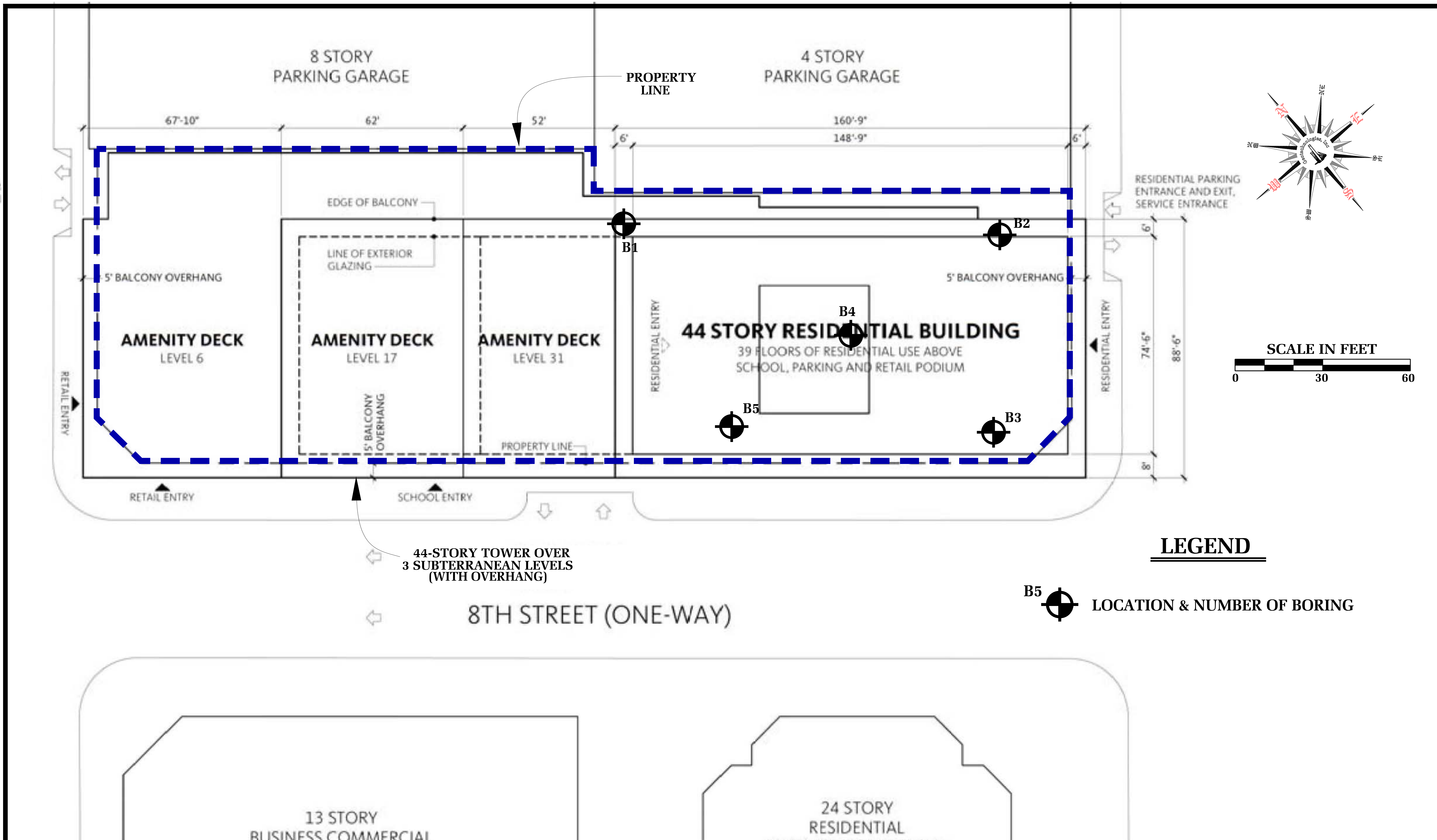
FILE NO. 21358



REFERENCE: SITE PLAN BY GENSLER
DATED 10/30/2018

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PLOT PLAN

MITSUI FUDOSAN

File No.: 21358

Date: March '17

BORING LOG NUMBER 1

Mitsui Fudosan **Date: 12/27/16**

File No. 21358 **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
				0 --		2½-inch Asphalt over 1½-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to dense, fine grained
				-		
				2 --		
2.5	24	16.7	110.4	-		
				3 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				4 --		
				-		
5	28	13.9	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	80	3.4	122.0	-		
				8 --	SP/SW	Sand to Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				-		
				9 --		
				-		
10	51	2.5	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	25	2.0	126.6	-		
	50/5"			13 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				14 --		
				-		
15	65	3.7	SPT	15 --		
				-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				16 --		
				-		
				17 --		
17.5	70	2.4	128.6	-		
				18 --		
				-		
				19 --		
				-		
20	70	3.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	45	1.6	93.9	-		
	50/4"			23 --		
				-		
				24 --		
				-		
25	37	4.2	SPT	25 --		
				-	SP/SM	Sand to Silty Sand, dark brown, slightly moist, dense, fine grained

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	74	11.1	119.1	-		
				28 --		Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained
				-		
				29 --		
				-		
30	34	9.8	SPT	30 --		
				-	SP	Sand, dark brown, moist, dense, fine to medium grained
				31 --		
				-		
				32 --		
32.5	48	16.1	116.9	-		
	50/5"			33 --	ML	Sandy Silt, dark gray, moist, very stiff
				-		
				34 --		
				-		
35	43	16.4	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	24	13.8	117.0	-		
	50/5"			38 --		
				-		
				39 --		
				-		
40	51	13.1	SPT	40 --		
				-	SM	Silty Sand, dark grayish brown, moist, dense, fine grained
				41 --		
				-		
				42 --		
42.5	82	12.6	119.6	-		
				43 --		
				-		
				44 --		
				-		
45	32	16.2	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	65	19.4	102.9	-		
				48 --		
				-		
				49 --		
				-		
50	31	27.6	SPT	50 --		
				-	ML	Clayey Silt, dark brown to gray, moist, very stiff

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
52.5	70	18.2	112.4	52		
				53	SM/ML	Silty Sand, dark gray, moist, very dense, fine grained
				54		
55	53	16.8	SPT	55		
				56		
57.5	80	4.1	106.5	57		
				58	SP	Sand, dark brown, slightly moist, very dense, fine to medium dense, occasional gravel
				59		
60	37 50/5"	11.7	SPT	60	SM	Silty Sand, dark gray, moist, very dense, fine to medium grained
				61		
62.5	100/8"	3.6	110.5	62		
				63	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				64		
65	40 50/3"	2.2	SPT	65		
				66		
67.5	100/7.5"	2.4	112.7	67		
				68		
				69		
70	80	2.1	SPT	70		
				71		
72.5	100/9"	2.5	109.6	72		
				73		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				74		
75	90	2.3	SPT	75		

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	100/9"	2.3	106.6	-		
				78 --		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				79 --		
				-		
80	30 50/6"	2.5	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	100/8"	3.7	111.3	-		
				83 --		
				-		
				84 --		
				-		
85	45 50/5"	6.8	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87.5	100	14.3	105.9	-		
				88 --	SM/SW	Silty Sand to Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
				-		
				89 --		
				-		
90	80	3.6	SPT	90 --		
				-	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained, occasional gravel
				91 --		
				-		
				92 --		
92.5	100/9"	4.4	108.9	-		
				93 --		
				-		
				94 --		
				-		
95	55	14.5	SPT	95 --		
				-	SP/ML	Sand to Sandy Silt, dark gray, moist, dense to stiff, fine to grained
				96 --		
				-		
				97 --		
97.5	100/8"	7.3	113.9	-		
				98 --	SP/SW	Sand to Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained, occasional gravel
				-		
				99 --		
				-		
100	70	4.5	SPT	100 --		
				-	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained, occasional gravel

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				101 --		
				102 --		
102.5	100/6"	5.8	109.6	-		
				103 --		Sand, yellow to grayish brown, slightly moist, very dense, fine to medium grained, occasional gravel
				104 --		
105	40 50/5"	25.0	SPT	105 --		
				-	ML	Sandy Silt, dark grayish brown, moist, very stiff, fine grained
				106 --		
				107 --		
107.5	100/6"	9.6	115.8	-		
				108 --	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				109 --		
110	89	9.1	SPT	110 --		
				111 --		
				112 --		
112.5	100/8"	6.5	109.5	-		
				113 --		
				114 --		
115	75/7"	17.5	SPT	115 --		
				-	ML	Sandy Silt, dark grayish brown, moist, very stiff, fine to medium grained, occasional gravel
				116 --		
				117 --		
117.5	100/6"	21.8	102.2	-		
				118 --		
				119 --		
120	45 50/4"	17.5	SPT	120 --		
				121 --		
				122 --		
122.5	100/7"	6.3	103.8	-		
				123 --	SP	Sand, gray, slightly moist, very dense, fine grained
				124 --		
125	50/6"	9.4	SPT	125 --		
				-	SM	Silty Sand, dark gray, moist, very dense, fine grained

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				126 --		
				-		
				127 --		
127.5	100/6.5"	4.6	122.2	-		
				128 --	SP	Sand, gray, slightly moist, very dense, fine grained
				-		
				129 --		
				-		
130	40	6.4	SPT	130 --		
	50/3"			-		
				131 --		
				-		
				132 --		
132.5	40	25.4	94.9	-		
	50/3"			133 --	SM/SP	Silty Sand to Sand, gray, wet, very dense, fine grained
				-		
				134 --		
				-		
135	35	15.0	SPT	135 --		
	50/5"			-		
				136 --		
				-		
				137 --		
137.5	38	30.9	88.7	-		
	50/5"			138 --	SP	Sand, gray, wet, very dense, fine grained
				-		
				139 --		
				-		
140	60	25.4	SPT	140 --		
	50/5"			-	SM/SP	Silty Sand to Sand, gray, wet, very dense, fine grained
				141 --		fine grained
				-		
				142 --		
142.5	100/7"	25.7	98.2	-		
				143 --		
				-		
				144 --		
				-		
145	38	23.9	SPT	145 --		
	50/5.5"			-		NOTE: The stratification lines represent the approximate
				146 --		boundary between earth types; the transition may be gradual.
				-		
				147 --		Used 8-inch diameter Hollow-Stem Auger
147.5	100/6"	22.1	105.9	-		140-lb. Automatic Hammer, 30-inch drop
				148 --		Modified California Sampler used unless otherwise noted
				-		
				149 --		SPT=Standard Penetration Test
				-		
150	38	24.7	SPT	150 --		
	50/5"			-		Total Depth 150 feet
						After waiting 30 minutes Water level was 130'
						Fill to 3 feet

BORING LOG NUMBER 2

Mitsui Fudosan **Date: 12/29/16**

File No. 21358 **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
				0 --		2-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to medium firm, fine grained
				-		
				2 --		
2.5	27	10.2	110.8	-		
				3 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff, fine grained
				4 --		
				-		
5	37	16.6	114.8	5 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	72	2.4	125.0	10 --		
				-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	40	7.0	Disturbed	15 --		
	50/2"			-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/7"	1.0	126.1	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100/11.5"	5.3	126.0	25 --		
				-		Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles

BORING LOG NUMBER 2

Mitsui Fudosan

File No. 21358

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	100/12"	5.4	130.2	30 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	90	13.5	121.3	35 --		
				-		Silty Sand, dark grayish brown, moist, very dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	57	14.5	116.1	40 --		
	50/5"			-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	38	12.7	118.0	45 --		
	50/4"			-	SM/SP	Silty Sand to Sand, dark gray, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	100/12"	13.3	123.1	50 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained

BORING LOG NUMBER 2

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	100/8"	5.6	107.8	55 --		
				-	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained, occasional cobbles
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	100/8"	4.7	126.9	60 --		
				-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	100/8"	5.3	102.8	65 --		
				-		Total Depth 65 feet
				66 --		No Water
				-		Fill to 3 feet
				67 --		
				-		
				68 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				69 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				70 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 3

Mitsui Fudosan | **Date: 12/29/16**

File No. 21358 | **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
				0 --		2½-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	38	10.8	110.9	-		
				3 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				4 --		
				-		
5	42	10.5	116.3	5 --		
				-		Sandy Silt, dark brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	30	8.9	123.6	10 --		
	50/5"			-	ML/SW	Sandy Silt to Gravelly Sand, dark brown, moist, very stiff to very dense, fine to coarse grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	100/8"	1.7	118.1	15 --		
				-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/7"	1.7	126.0	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	48	21.6	104.9	25 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff

BORING LOG NUMBER 3

Mitsui Fudosan

File No. 21358

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	55	22.7	102.8	30 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	55	12.7	117.6	35 --		
	50/3"			-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very
				36 --		dense to very stiff, fine grained
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	80	18.4	113.4	40 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	80	18.7	111.0	45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

BORING LOG NUMBER 3

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
				53 --		
				54 --		
55	24	24.4	102.9	55 --		
	50/5"			-		Clayey Silt, dark gray, moist, very stiff
				56 --		
				57 --		
				58 --		
				59 --		
				60 --		
				61 --		
				62 --		
				63 --		
				64 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained
65	40	12.1	115.6	65 --		
	50/5"			-		Total Depth 65 feet
				66 --		No Water
				-		Fill to 3 feet
				67 --		
				68 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				69 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				70 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

BORING LOG NUMBER 4

Mitsui Fudosan **Date: 12/23/16**

File No. 21358 **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
				0 --		2½-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Sand, dark to yellowish brown, moist, medium dense, fine grained, with abundant brick and concrete fragments
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	70	15.8	109.2	5 --		
				-		
				6 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				7 --		
7.5	68	11.4	116.9	-		
	50/2"			8 --	SM	Silty Sand, dark brown, moist, dense, fine grained, occasional gravel
				-		
				9 --		
				-		
10	86	7.1	125.0	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	100/8"	3.7	101.5	15 --		
				-	SW	Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	0.4	119.6	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	100.8"	3.3	120.4	25 --		
				-		

BORING LOG NUMBER 4

Mitsui Fudosan

File No. 21358

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	100/10"	7.6	110.9	30 --		
				-	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	53	15.6	106.3	35 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	73	20.5	99.6	40 --		
				-	ML	Sandy Silt, dark grayish brown, moist, stiff
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	35	13.9	112.7	45 --		
	50/5.5"			-	SM	Silty Sand, dark gray, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	90	12.6	118.8	50 --		
				-		

BORING LOG NUMBER 4

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	72	17.1	115.9	55 --		
				-	SM	Silty Sand, dark gray, moist, very dense
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	75	20.2	109.0	60 --		
				-		Silty Sand, dark gray, moist, dense, fine grained
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	83	22.3	100.1	65 --		
				-		Total Depth 65 feet
				66 --		No Water
				-		Fill to 6 feet
				67 --		
				-		
				68 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				69 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				70 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 5

Mitsui Fudosan **Date: 12/22/16**

File No. 21358 **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
				0 --		2-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	49	8.0	108.4	-		
				3 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				4 --		
				-		
5	76	8.3	126.1	5 --		
				-	SM	Silty Sand, dark brown, slightly moist, dense, fine grained
				6 --		
				-		
				7 --		
7.5	82	3.2	120.4	-		
				8 --	SP/SW	Sand to Gravelly Sand, dark brown, moist, very dense, fine to coarse grained
				-		
				9 --		
				-		
10	86	2.4	119.7	10 --		
				-	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained, occasional gravel
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	100/9"	2.4	119.7	15 --		
				-	SW	Gravelly Sand, dark brown, slightly moist, very dense
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	4.5	109.5	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	68	6.8	110.0	25 --		
				-	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained, occasional cobbles

BORING LOG NUMBER 5

Mitsui Fudosan

File No. 21358

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	90	8.1	112.7	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	72	18.1	114.1	35 --		
				-	ML	Sandy Silt, dark to yellowish brown, moist, very stiff
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	47	15.5	116.6	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	75	18.0	114.4	45 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, dense to stiff, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	48	15.4	115.5	50 --		
				-		

BORING LOG NUMBER 5

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	57	16.7	112.4	55 --		
				-		Silty Sand to Sandy Silt, grayish brown, moist, dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	48	19.1	106.9	60 --		
				-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	41	18.3	109.0	65 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	30	12.5	107.9	70 --		
	50/4"			-		very dense
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	40	15.4	101.6	75 --		
	50/3"			-	SP	Sand, dark brown, moist, very dense, fine grained

BORING LOG NUMBER 5

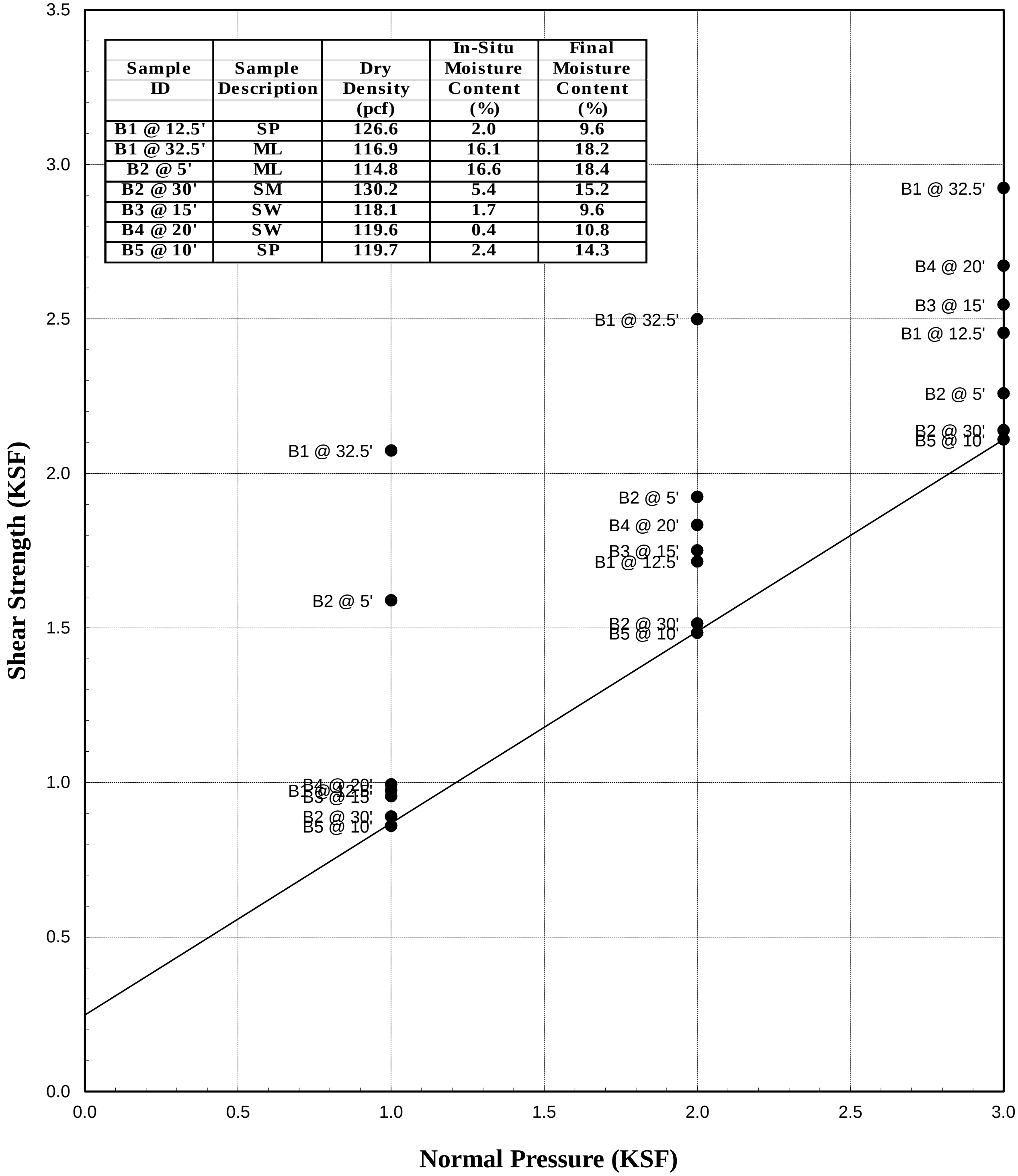
Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		
				-		
80	38	8.2	104.5	80 --		
	50/3"			-		Total Depth 80 feet
				81 --		No Water
				-		Fill to 3 feet
				82 --		
				-		
				83 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				84 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				85 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				86 --		
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

Saturated Shear



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 12.5'	SP	126.6	2.0	9.6
B1 @ 32.5'	ML	116.9	16.1	18.2
B2 @ 5'	ML	114.8	16.6	18.4
B2 @ 30'	SM	130.2	5.4	15.2
B3 @ 15'	SW	118.1	1.7	9.6
B4 @ 20'	SW	119.6	0.4	10.8
B5 @ 10'	SP	119.7	2.4	14.3

ϕ: 32.0 degrees
c: 235.0 psf

SHEAR TEST DIAGRAM

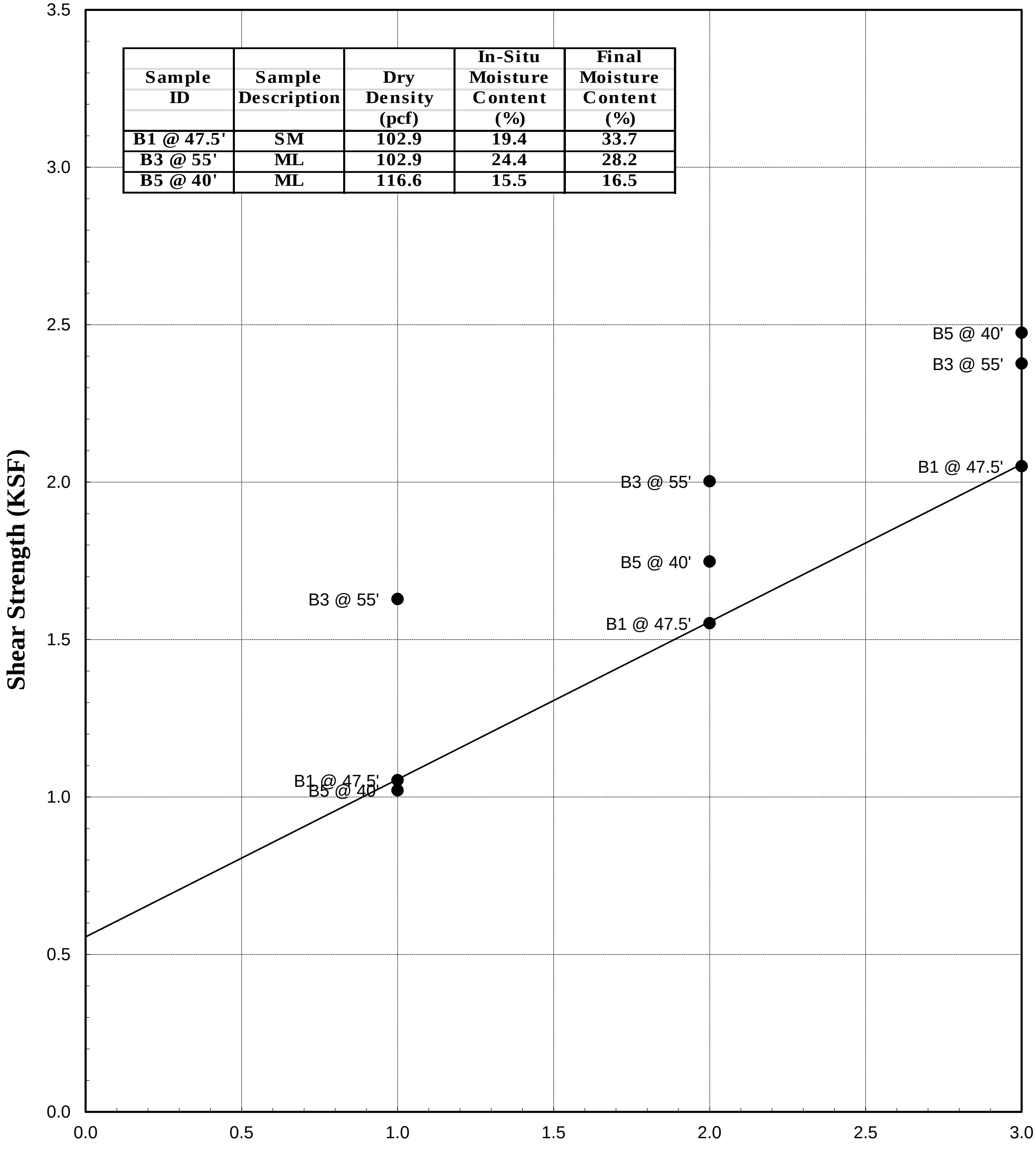
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: MITSUI FUDOSAN

FILE NO.: 21358

PLATE: B-1

Saturated Shear



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 47.5'	SM	102.9	19.4	33.7
B3 @ 55'	ML	102.9	24.4	28.2
B5 @ 40'	ML	116.6	15.5	16.5

ϕ: 26.5 degrees
c: 555.0 psf

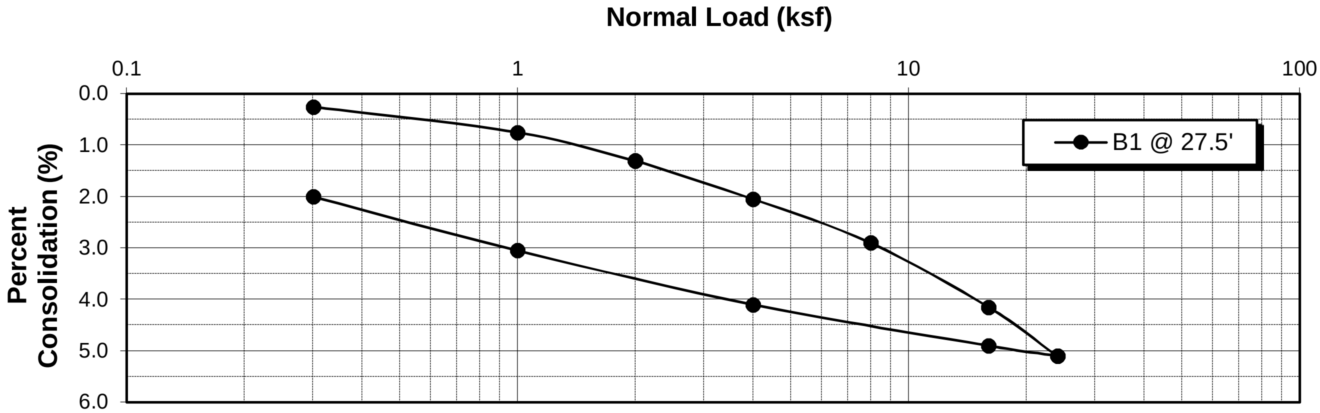
SHEAR TEST DIAGRAM

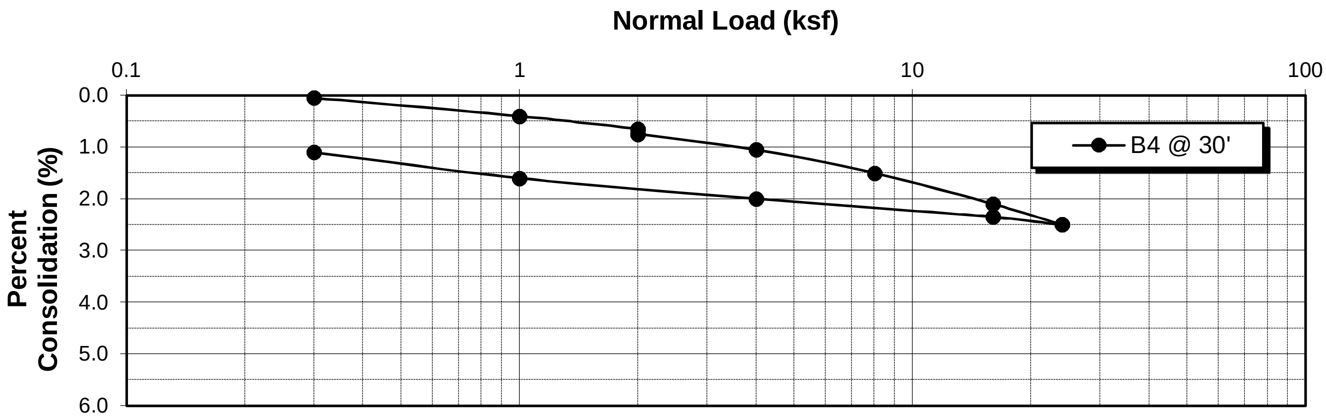
Geotechnologies, Inc.
Consulting Geotechnical Engineers

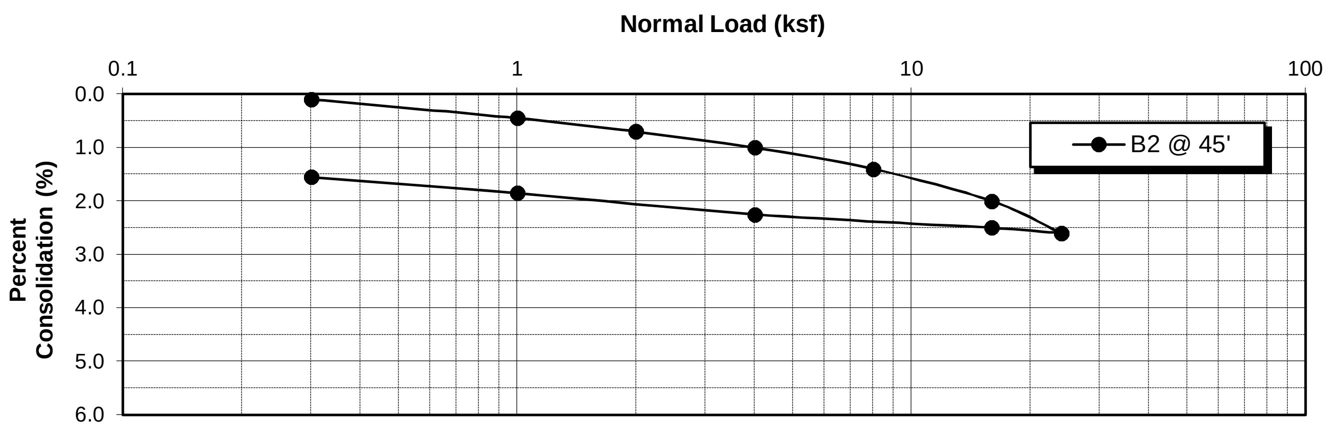
PROJECT: MITSUI FUDOSAN

FILE NO.: 21358

PLATE: B-2







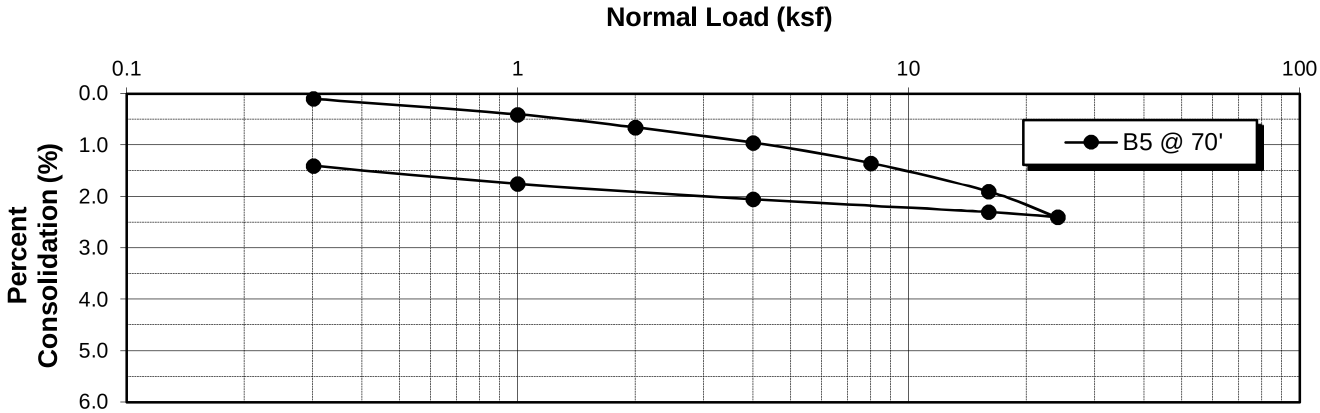
Water added at 2 KSF

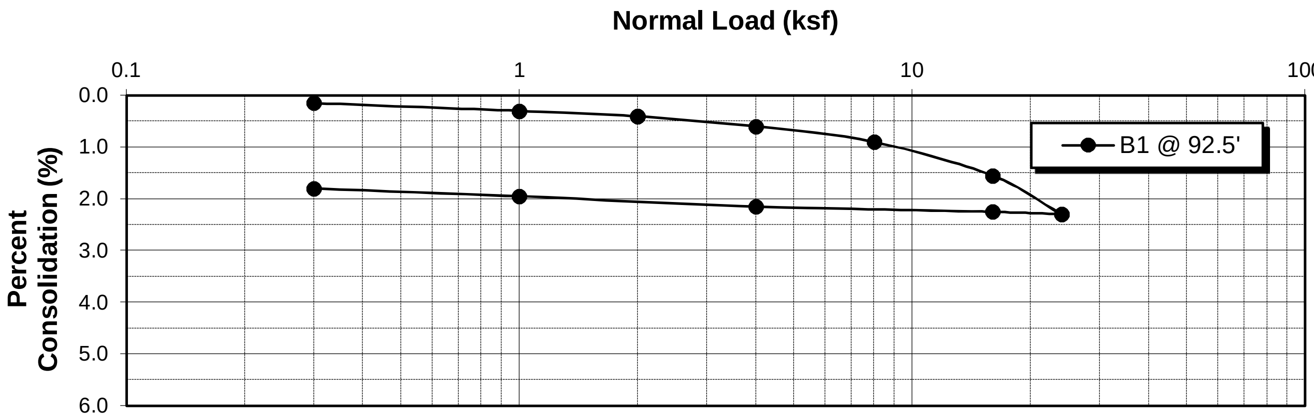
CONSOLIDATION

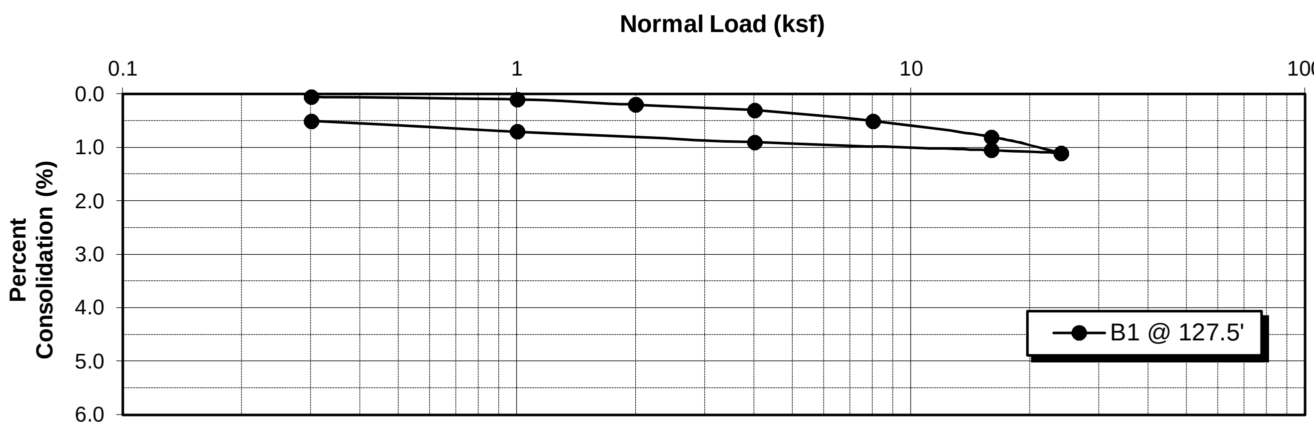
Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: MITSUI FUDOSAN	
FILE NO. 21358	PLATE: C-1







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: MITSUI FUDOSAN	
FILE NO. 21358	PLATE: C-2

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

Mitsui Fudosan
File No. 21358

COMPACTION/EXPANSION/SULFATE DATA SHEET

ASTM D-1557

Sample	B3 @ 1' – 5'	B4 @ 5' – 10'
Soil Type	SM	SM
Maximum Density (pcf)	124.0	124.0
Optimum Moisture Content (percent)	11.5	11.0
Percent finer than 0.005mm (percent)	<15%	<15%

EXPANSION INDEX

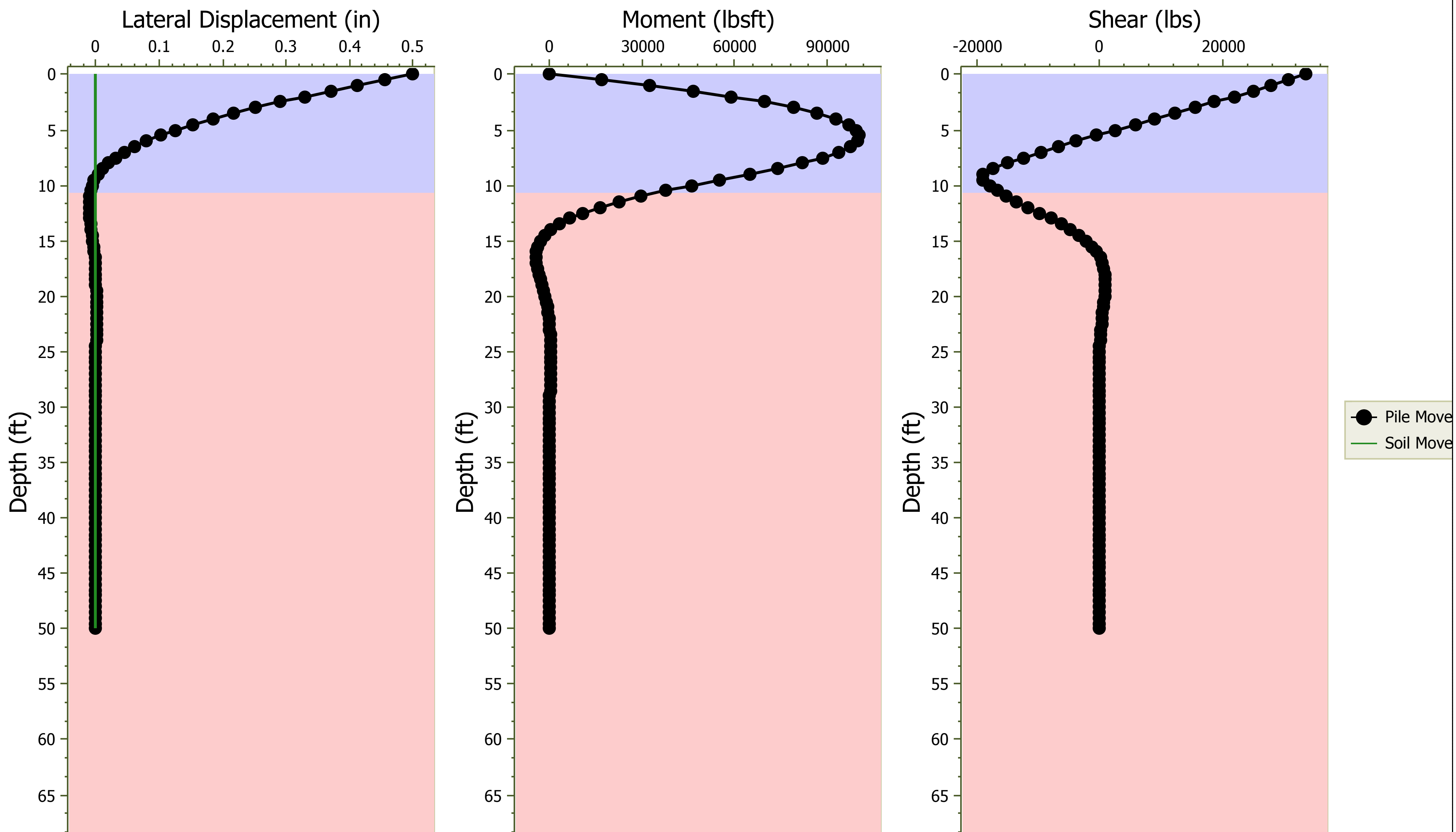
Sample	B3 @ 1' – 5'	B4 @ 5' – 10'
Soil Type	SM	SM
Expansion Index – UBC Standard 18-2	98	74
Expansion Characteristic	High	Moderate

SULFATE CONTENT

Sample	B3 @ 1' – 5'	B4 @ 5' – 10'
Sulfate Content (ppm)	<250	<250

PLATE D

Lateral Pile Analysis - Free Head Condition



RSPILE 1.004	*Project* File No. 21358, Mitsui Fudosan		
	Analysis Description Lateral Pile Capacity		
	Drawn By SST	*Company* Geotechnologies, Inc.	
	Date 4/3/2017, 2:40:35 PM	*File Name* 18-inch diameter CFA pile (0.5d-free).rspile	

Lateral Pile Analysis Information

Project Summary

File Name 18-inch diameter CFA pile (0.5d-free)
File Version 0.000

Project Title File No. 21358, Mitsui Fudosan
Analysis Lateral Pile Capacity
Author SST
Company Geotechnologies, Inc.
Date Created 4/3/2017, 2:40:35 PM

Comments

18-inch diameter CFA pile

Results

Lateral Displacement, Moment and Shear Results

Soil Layer #	Pile Depth (ft)	Deflection (in)	Moment (lbsft)	Shear (lbs)
1	0.00	0.5000	0.0000	33585.4804
1	0.50	0.4562	16934.4853	30810.7186
1	1.00	0.4129	32443.7028	27923.3620
1	1.50	0.3705	46464.9590	24929.9243
1	2.00	0.3294	58939.3933	21848.0630
1	2.50	0.2899	69823.5597	18699.4076
1	3.00	0.2524	79081.8711	15500.3674
1	3.50	0.2171	86688.9759	12270.0188
1	4.00	0.1843	92630.0802	9019.2289
1	4.50	0.1540	96892.4379	5775.0617
1	5.00	0.1264	99490.0714	2566.4583
1	5.50	0.1016	100440.9278	-605.7377
1	6.00	0.0796	99761.6080	-3718.9951
1	6.50	0.0604	97494.3075	-6726.1686
1	7.00	0.0439	93704.4587	-9614.0669
1	7.50	0.0301	88449.0780	-12368.1226
1	8.00	0.0187	81809.7311	-14941.0515
1	8.50	0.0096	73892.2116	-17197.3685
1	9.00	0.0025	64914.9650	-19011.8453
1	9.50	-0.0027	55110.2383	-18982.3893
2	10.00	-0.0064	46099.5586	-17677.1265
2	10.50	-0.0088	37547.0640	-16530.6869
2	11.00	-0.0101	29638.9958	-15084.4825
2	11.50	-0.0106	22497.5024	-13423.2552
2	12.00	-0.0105	16223.3437	-11596.0578
2	12.50	-0.0099	10888.7592	-9715.1239
2	13.00	-0.0091	6481.3286	-7898.3446
2	13.50	-0.0080	2954.4221	-6204.0068
2	14.00	-0.0069	236.3852	-4672.8772
2	14.50	-0.0057	-1761.0636	-3330.0315

2	15.00	-0.0047	-3135.4559	-2187.0416
2	15.50	-0.0037	-3987.3491	-1244.3403
2	16.00	-0.0028	-4415.3080	-493.6140
2	16.50	-0.0020	-4512.0722	79.9022
2	17.00	-0.0014	-4361.8373	495.3174
2	17.50	-0.0008	-4038.5366	774.2315
2	18.00	-0.0004	-3604.9861	939.1712
2	18.50	-0.0001	-3112.7408	1012.3325
2	19.00	0.0001	-2602.5092	1014.6225
2	19.50	0.0002	-2104.9792	964.9840
2	20.00	0.0003	-1641.9195	879.9719
2	20.50	0.0004	-1227.4384	773.5462
2	21.00	0.0004	-869.3010	657.0417
2	21.50	0.0004	-570.2257	539.2751
2	22.00	0.0003	-329.1006	426.7522
2	22.50	0.0003	-142.0771	323.9392
2	23.00	0.0002	-3.5181	233.5705
2	23.50	0.0002	93.2131	156.9671
2	24.00	0.0002	155.1217	94.3462
2	24.50	0.0001	189.1018	45.1084
2	25.00	0.0001	201.5923	8.0930
2	25.50	0.0001	198.3523	-18.2045
2	26.00	0.0000	184.3357	-35.4548
2	26.50	0.0000	163.6449	-45.3557
2	27.00	0.0000	139.5451	-49.5295
2	27.50	-0.0000	114.5215	-49.4533
2	28.00	-0.0000	90.3648	-46.4157
2	28.50	-0.0000	68.2715	-41.4973
2	29.00	-0.0000	48.9501	-35.5682
2	29.50	-0.0000	32.7246	-29.2981
2	30.00	-0.0000	19.6304	-23.1759
2	30.50	-0.0000	9.4996	-17.5335
2	31.00	-0.0000	2.0326	-12.5725
2	31.50	-0.0000	-3.1433	-8.3912
2	32.00	-0.0000	-6.4283	-5.0089
2	32.50	-0.0000	-8.2169	-2.3892
2	33.00	-0.0000	-8.8746	-0.4588
2	33.50	-0.0000	-8.7239	0.8766
2	34.00	-0.0000	-8.0369	1.7199
2	34.50	-0.0000	-7.0341	2.1735
2	35.00	-0.0000	-5.8857	2.3327
2	35.50	0.0000	-4.7168	2.2820
2	36.00	0.0000	-3.6136	2.0928
2	36.50	0.0000	-2.6296	1.8229
2	37.00	0.0000	-1.7930	1.5169
2	37.50	0.0000	-1.1127	1.2075
2	38.00	0.0000	-0.5839	0.9172
2	38.50	0.0000	-0.1930	0.6596
2	39.00	0.0000	0.0786	0.4420
2	39.50	0.0000	0.2519	0.2663
2	40.00	0.0000	0.3477	0.1309
2	40.50	0.0000	0.3853	0.0320
2	41.00	0.0000	0.3817	-0.0356
2	41.50	0.0000	0.3513	-0.0776

2	42.00	0.0000	0.3054	-0.0995
2	42.50	0.0000	0.2528	-0.1065
2	43.00	-0.0000	0.1995	-0.1032
2	43.50	-0.0000	0.1500	-0.0932
2	44.00	-0.0000	0.1065	-0.0796
2	44.50	-0.0000	0.0705	-0.0645
2	45.00	-0.0000	0.0420	-0.0495
2	45.50	-0.0000	0.0209	-0.0357
2	46.00	-0.0000	0.0062	-0.0237
2	46.50	-0.0000	-0.0030	-0.0138
2	47.00	-0.0000	-0.0077	-0.0060
2	47.50	-0.0000	-0.0091	-0.0004
2	48.00	-0.0000	-0.0082	0.0032
2	48.50	-0.0000	-0.0060	0.0050
2	49.00	0.0000	-0.0033	0.0051
2	49.50	0.0000	-0.0010	0.0034
2	50.00	0.0000	-0.0000	0.0000

General Properties

Pile Loading

Property	Value
Axial Load (ksf)	225000
Moment (lbsft)	0
Deflection (in)	0.5

Lateral Soil Loading

Soil Displacement(in) 0
Sliding Depth (ft) 0

Pile Properties

Property	Value
Name	18-in ACP
Pile Type:	Cylindrical
Material Type:	Elastic
Young’s Modulus (psf)	432000000.00
Diameter (ft)	1.50

Soil Layer Properties

Layers

Layer Name	Layer Type	Thickness (ft)
Soil Layer 1	Dry Stiff Clay	10.00
Soil Layer 2	Sand	60.00

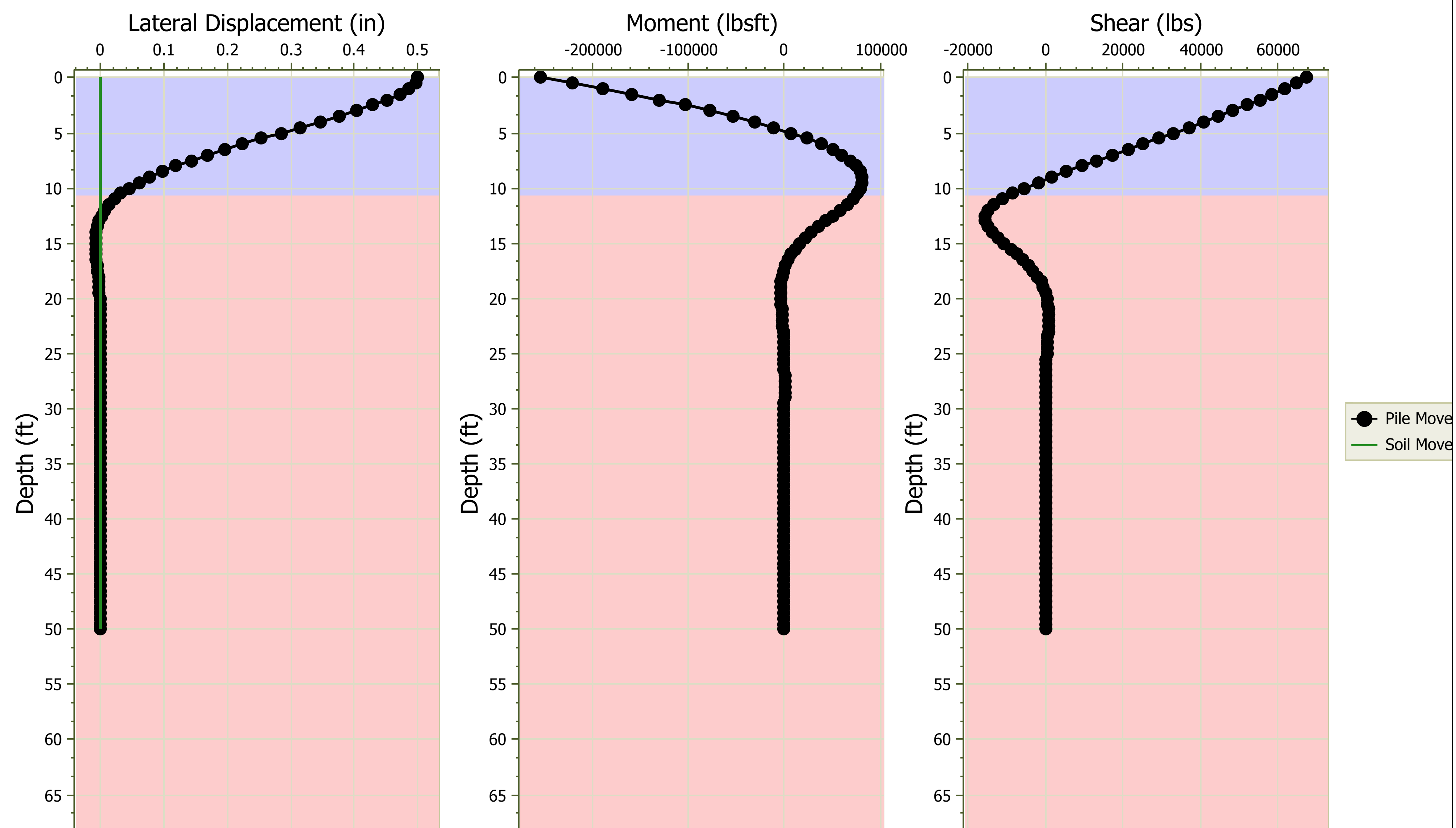
Soil Layer 1

Property	Value
Soil Type	Dry Stiff Clay
Thickness (ft)	10.00
Unit Weight (lbs/ft3)	120.00
Strain Factor (E50)	0.01
Undrained Shear Strength (psf)	2000.00

Soil Layer 2

Property	Value
Soil Type	Sand
Thickness (ft)	60.00
Unit Weight (lbs/ft3)	120.00
Friction Angle (degrees)	34.00
Top Kpy (lbs/ft3)	345600.00

Lateral Pile Analysis - Fixed Head Condition



Project	File No. 21358, Mitsui Fudosan		
Analysis Description	Lateral Pile Capacity		
Drawn By	SST	Company	Geotechnologies, Inc.
Date	4/3/2017, 2:40:35 PM	File Name	18-inch diameter CFA pile (0.5d-fixed).rspile

RSPILE 1.004

Lateral Pile Analysis Information

Project Summary

File Name 18-inch diameter CFA pile (0.5d-fixed)
File Version 0.000

Project Title File No. 21358, Mitsui Fudosan
Analysis Lateral Pile Capacity
Author SST
Company Geotechnologies, Inc.
Date Created 4/3/2017, 2:40:35 PM

Comments

18-inch diameter CFA pile

Results

Lateral Displacement, Moment and Shear Results

Soil Layer #	Pile Depth (ft)	Deflection (in)	Moment (lbsft)	Shear (lbs)
1	0.00	0.5000	-253943.1634	67581.2884
1	0.50	0.4965	-220765.1739	64778.0196
1	1.00	0.4867	-188916.4124	61806.6308
1	1.50	0.4717	-158495.1520	58677.1189
1	2.00	0.4523	-129593.8703	55399.5559
1	2.50	0.4293	-102299.2238	51983.8567
1	3.00	0.4034	-76692.1370	48439.5691
1	3.50	0.3753	-52847.9926	44775.6868
1	4.00	0.3458	-30836.9133	41007.2222
1	4.50	0.3154	-10717.3854	37156.8978
1	5.00	0.2847	7465.1426	33241.1295
1	5.50	0.2542	23670.6062	29275.6147
1	6.00	0.2244	37871.3053	25282.1790
1	6.50	0.1957	50051.0874	21273.7750
1	7.00	0.1683	60197.3140	17268.1533
1	7.50	0.1426	68313.7078	13299.0344
1	8.00	0.1188	74423.4801	9383.8142
1	8.50	0.0972	78549.8642	5519.0475
1	9.00	0.0777	80714.7167	1739.3381
1	9.50	0.0604	80977.9420	-1903.9782
2	10.00	0.0454	79414.7567	-5354.9936
2	10.50	0.0327	76142.9261	-8493.7088
2	11.00	0.0221	71359.5185	-11178.5728
2	11.50	0.0134	65325.5375	-13379.0835
2	12.00	0.0066	58270.0009	-14963.9064
2	12.50	0.0014	50586.4369	-15696.4607
2	13.00	-0.0023	42741.3088	-15604.6159
2	13.50	-0.0049	35100.6890	-14882.4340
2	14.00	-0.0065	27936.9563	-13707.6401
2	14.50	-0.0073	21438.1007	-12237.4327

2	15.00	-0.0075	15718.7044	-10606.0469
2	15.50	-0.0073	10831.7657	-8923.8500
2	16.00	-0.0068	6780.6547	-7277.7097
2	16.50	-0.0060	3530.6280	-5732.3604
2	17.00	-0.0052	1019.4636	-4332.4986
2	17.50	-0.0044	-833.0854	-3105.3524
2	18.00	-0.0036	-2117.2013	-2063.5008
2	18.50	-0.0028	-2926.3529	-1207.7515
2	19.00	-0.0021	-3352.0769	-529.9226
2	19.50	-0.0015	-3480.1099	-15.4134
2	20.00	-0.0010	-3387.7447	354.5171
2	20.50	-0.0006	-3142.2487	600.8101
2	21.00	-0.0003	-2800.1691	745.0245
2	21.50	-0.0001	-2407.3450	808.1098
2	22.00	0.0001	-1999.4516	809.5107
2	22.50	0.0002	-1602.9175	766.5722
2	23.00	0.0002	-1236.0751	694.2003
2	23.50	0.0003	-910.4253	604.7368
2	24.00	0.0003	-631.9218	507.9981
2	24.50	0.0003	-402.2025	411.4381
2	25.00	0.0002	-219.7171	320.3929
2	25.50	0.0002	-80.7172	238.3742
2	26.00	0.0002	19.9069	167.3835
2	26.50	0.0001	87.9480	108.2236
2	27.00	0.0001	129.3556	60.7908
2	27.50	0.0001	149.8501	24.3388
2	28.00	0.0001	154.6594	-2.2958
2	28.50	0.0000	148.3597	-20.5064
2	29.00	0.0000	134.7996	-31.7689
2	29.50	0.0000	117.0891	-37.5314
2	30.00	-0.0000	97.6346	-39.1365
2	30.50	-0.0000	78.2064	-37.7726
2	31.00	-0.0000	60.0236	-34.4483
2	31.50	-0.0000	43.8474	-29.9846
2	32.00	-0.0000	30.0739	-25.0211
2	32.50	-0.0000	18.8224	-20.0314
2	33.00	-0.0000	10.0129	-15.3440
2	33.50	-0.0000	3.4337	-11.1657
2	34.00	-0.0000	-1.2045	-7.6062
2	34.50	-0.0000	-4.2253	-4.7006
2	35.00	-0.0000	-5.9550	-2.4305
2	35.50	-0.0000	-6.7004	-0.7417
2	36.00	-0.0000	-6.7348	0.4410
2	36.50	-0.0000	-6.2905	1.2024
2	37.00	-0.0000	-5.5565	1.6284
2	37.50	-0.0000	-4.6800	1.7998
2	38.00	0.0000	-3.7694	1.7884
2	38.50	0.0000	-2.8998	1.6552
2	39.00	0.0000	-2.1188	1.4499
2	39.50	0.0000	-1.4520	1.2107
2	40.00	0.0000	-0.9083	0.9658
2	40.50	0.0000	-0.4851	0.7345
2	41.00	0.0000	-0.1720	0.5287
2	41.50	0.0000	0.0457	0.3545

2	42.00	0.0000	0.1847	0.2140
2	42.50	0.0000	0.2618	0.1059
2	43.00	0.0000	0.2925	0.0272
2	43.50	0.0000	0.2905	-0.0264
2	44.00	0.0000	0.2673	-0.0595
2	44.50	0.0000	0.2320	-0.0765
2	45.00	0.0000	0.1915	-0.0816
2	45.50	-0.0000	0.1508	-0.0786
2	46.00	-0.0000	0.1132	-0.0704
2	46.50	-0.0000	0.0806	-0.0593
2	47.00	-0.0000	0.0540	-0.0472
2	47.50	-0.0000	0.0334	-0.0353
2	48.00	-0.0000	0.0186	-0.0245
2	48.50	-0.0000	0.0088	-0.0154
2	49.00	-0.0000	0.0031	-0.0081
2	49.50	-0.0000	0.0005	-0.0030
2	50.00	-0.0000	-0.0000	0.0000

General Properties

Pile Loading

Property	Value
Axial Load (ksf)	225000
Slope (degrees)	0
Deflection (in)	0.5

Lateral Soil Loading

Soil Displacement(in) 0
Sliding Depth (ft) 0

Pile Properties

Property	Value
Name	18-in ACP
Pile Type:	Cylindrical
Material Type:	Elastic
Young's Modulus (psf)	432000000.00
Diameter (ft)	1.50

Soil Layer Properties

Layers

Layer Name	Layer Type	Thickness (ft)
Soil Layer 1	Dry Stiff Clay	10.00
Soil Layer 2	Sand	60.00

Soil Layer 1

Property	Value
Soil Type	Dry Stiff Clay
Thickness (ft)	10.00
Unit Weight (lbs/ft3)	120.00
Strain Factor (E50)	0.01
Undrained Shear Strength (psf)	2000.00

Soil Layer 2

Property	Value
Soil Type	Sand
Thickness (ft)	60.00
Unit Weight (lbs/ft3)	120.00
Friction Angle (degrees)	34.00
Top Kpy (lbs/ft3)	345600.00

Geotechnologies, Inc.

Project: Mitsui Fudosan
File No.: 21358

Settlement Calculation - Mat Footing

Description: Mat Foundation (167'x105')
Gridline:

Soil Unit Weight	125.0 pcf		Mat Footing	
Bearing Value	12000.0 psf		334668 kips	
Depth of Footing	50.0 feet			
Width of Footing	167.0 feet			

* Influence Values are based on Westergaard's Analyses (Ref: Sowers)

Depth Below Ground Surface (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
50.0													
	62.5	12.5	13.4	90%	10809.3	7812.5	18621.8	B5 @ 70'	2.00	1.40	0.60	25.0	1.80
75.0													
	87.5	37.5	4.5	70%	8365.2	10937.5	19302.7	B1 @ 92.5'	1.80	1.15	0.65	25.0	1.95
100.0													
	217.5	167.5	1.0	19%	2235	27187.5	29422.5	B1 @ 127.5'	1.25	1.20	0.05	235.0	1.41
335.0													

Settlement: 5.16
Reduction: 0.67

Total Settlement in inches: ***3.44***

GeoPentech

April 4, 2017

Project No. 16106A

Mr. Stan Tang
Geotechnologies, Inc.
439 Western Avenue
Glendale, California 91201

**SUBJECT: DOWNHOLE SEISMIC TEST RESULTS
BORING NUMBER 1
NORTHEAST CORNER OF 8TH STREET AND HOPE STREET
LOS ANGELES, CALIFORNIA**

Dear Mr. Tang,

Per your request and in accordance with the provisions of our proposal, dated December 14, 2016, we performed downhole seismic tests within Boring Number 1 drilled by Geotechnologies for the property located on the northeast corner of 8th Street and Hope Street in Los Angeles, California. The log of Boring Number 1 provided by Geotechnologies, Inc. is included in Attachment 1 and indicates that the subsurface materials are composed of

1. Fill primarily consisting of silty sand to sandy silt (SM to ML) from the ground surface to approximately 3 feet below ground surface; and
2. Alluvium predominantly consisting of sand (SM, SP, and SW) with occasional gravel and some clayey to sandy silt (ML) from approximately 3 to 150 feet (bottom of the borehole).

Additionally, the groundwater surface was noted at a depth of 130 feet during borehole drilling on December 27, 2016. Downhole seismic tests were performed within Boring Number 1 to assist Geotechnologies, Inc. with their evaluation of the site. This letter summarizes the results of the downhole seismic tests and the evaluation of V_{s30}.

Seismic Downhole Methods and Procedures

Downhole seismic tests were collected within Boring Number 1 on March 2, 2017. The downhole seismic test method makes direct measurements of in-situ vertically propagating compression (P) and horizontally polarized shear (SH) wave velocities as a function of depth within the geologic material adjacent to a borehole. Measurement procedures followed ASTM D7400-08, "Standard Test Methods for Downhole Seismic Testing". The geophysical data were collected, processed, and interpreted by a California-licensed Professional Geophysicist (PGp).

Boring Number 1 was drilled with an 8-inch diameter bit using hollow stem auger drilling methods and a 2-inch diameter PVC casing was installed under the direction of Geotechnologies, Inc. as part of their geotechnical investigation program. The annular space between the 8-inch diameter hole and 2-inch diameter casing was backfilled with bentonite-cement grout, which was assumed to be formulated to approximate the density of the surrounding geologic material and pumped in from the base of the borehole to completely fill the annular space.

A seismic source was used to generate a seismic wave (P or SH) at the ground surface. The seismic source was offset horizontally from the borehole a distance of 5 feet. The P-wave seismic source consisted of a ground plate that was struck vertically with a sledgehammer. The SH-wave seismic source consisted of an 8-foot long by 6-inch wide by 4-inch high wood beam capped on both ends with a steel plate and loaded in place by the front end of a vehicle that was parked on top of the beam. The ends of this beam were positioned equidistant from the borehole. Initially, one end of the beam was struck horizontal with a sledgehammer to produce an SH-wave (forward hit). Next, the opposite end of the beam was struck horizontally with a sledgehammer to produce an opposite polarity SH-wave (reverse hit). The combination of the two opposite polarity SH-waves were used to determine SH travel times.

A downhole receiver positioned at a selected depth within the borehole was used to record the arrival of the seismic wave (P or SH). A three component triaxial borehole geophone (one vertical-channel and two orthogonal horizontal channels), which could be firmly pneumatically fixed against the PVC casing sidewall, was used to collect the downhole seismic measurements. Multiple downhole seismic measurements were performed at successive receiver depths within the borehole. The receiver depth was referenced to ground surface, and measurements were made at receiver intervals of 5 feet from the ground surface to the bottom of the hole (150 ft).

A Geometrics S12 signal enhancing seismograph was used to record the response of the downhole receiver. The seismic source (sledgehammer) contained a trigger that was connected to and initiated the seismograph recording, thus measuring the travel time between seismic source and downhole receiver. Downhole seismic test records were digitally recorded and stored with a 0.062 ms sample interval.

The recorded digital downhole seismic records were analyzed using the OYO Corporation program PickWin Version 5.1.1.2. The digital waveforms were analyzed to identify arrival times. The first prominent departure of the vertical receiver trace was identified as the P-wave first arrival. The SH-wave forward and reverse hits recorded on the two horizontal receiver channels were superimposed. The SH-wave first arrival was identified at the location of the first prominent relatively low-frequency departure of the forward hit and an 180^{o} polarity change is noted to have occurred on the reverse hit. For analysis, 17 Hz low-cut and 250 Hz high-cut filters were applied to the P waveforms, and 25 Hz low-cut and 134 Hz high-cut filters were applied to the SH waveforms.

After correcting the P and SH-wave travel time for the source offset, the P and SH-wave travel-times were plotted versus depth. P and SH layer and interval velocities were calculated as the slope of lines drawn through the plotted data.

Seismic Downhole Results

The results of the seismic downhole measurements collected within Boring Number 1 are presented on Figure 1. Figure 1 shows (1) a table of the measured P and SH-wave travel-times and depths; (2) a plot of the P and SH-wave travel-times as a function of depth showing the interpreted layer velocities; (3) a table of the calculated P and SH-wave interval velocities and depth ranges; (4) a table of the interpreted P and SH-wave layer velocities and depth ranges; and (5) a plot of the layer and interval velocity models as a function of depth.

Table 1 below summarizes the interpreted P and SH layer velocities and depths shown on Figure 1 for the various geologic units within Boring Number 1, as logged by Geotechnologies, Inc. It is noted that the groundwater level was measured at a depth of 130 feet during borehole drilling on December 27, 2016. The measured P-wave velocities suggest the material adjacent to the borehole below a depth of approximately 115 feet is saturated.

TABLE 1
SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS WITHIN BORING NUMBER 1

<table>
<tr><th>PREDOMINANT LITHOLOGY</th><th>Depth Range (ft)</th><th>SH-WAVE Velocity (ft/sec)</th><th>P-WAVE Velocity (ft/sec)</th></tr>
<tr><td>Medium dense to dense, silty SAND to sandy SILT (SM to ML) [Fill]</td><td>0 to 5</td><td>650</td><td>1,160</td></tr>
<tr><td>Dense to very dense, SAND (SM, SP, and SW) [Alluvium]</td><td>5 to 30</td><td rowspan="2">1,300</td><td>2,160</td></tr>
<tr><td>Very stiff, sandy to clayey SILT (ML) and dense to very dense, SAND (SM and SP) [Alluvium]</td><td>30 to 55</td><td rowspan="2">3,220</td></tr>
<tr><td>Very dense, SAND (SM, SP, and SW) with trace very stiff, sandy SILT (ML) [Alluvium]</td><td>55 to 115</td><td>2,040</td></tr>
<tr><td>Very stiff, sandy SILT (ML) and very dense, SAND (SM and SP) [Alluvium]</td><td>115 to 125</td><td>2,750</td><td rowspan="2">4,630</td></tr>
<tr><td>Very dense, SAND (SM and SP) [Alluvium]</td><td>125 to 150</td><td>2,050</td></tr>
</table>

The V_{s30} was calculated based on the procedures outlined in the 2010 California Building Code, "2010 California Existing Building Code, Title 24, Part 10, Section 1613A.5.5 – Site Classification for Seismic Design." The V_{s30} was calculated from Equation 16A-40 of this reference which states:

$$v_s = \frac{\sum_{i=1}^{n} di}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where:

i = distinct different soil and/or rock layer between 1 and n
v_{si} = shear wave velocity in feet per second of layer i
d_i = thickness of any layer within the 100 foot interval
$\sum_{i=1}^{n} d_i$ = 100 feet

Based on this procedure, the V_{s30} for Boring Number 1 was calculated between various depth ranges. The results are summarized on Table 2.

TABLE 2
CALCULATED V_{s30} WITHIN BORING NUMBER 1

DEPTH RANGE (ft, below ground surface)	V_{s30} (ft/sec)
0 to 100	1,470
10 to 110	1,620
20 to 120	1,720
30 to 130	1,830
40 to 140	1,930
50 to 150	2,040

Limitations

The above information is based on limited observations and geophysical measurements made as described above. GeoPentech does not guarantee the performance of the project, only that the information provided meets the standard of care of the profession at this time under the same scope limitations imposed by the project. In this regard, our scope of work included making the P and SH-wave velocity measurements in one borehole under the direction of Geotechnologies, Inc. personnel. We relied upon the assumption that the annular space between the PVC casing and the borehole wall was properly filled with bentonite-cement grout so that PVC casing and the borehole wall were in continuous contact and that the grout was formulated to approximate the density of the surrounding geologic material.

We trust the contents of this letter will meet your current needs. If you have questions or require additional information, please call.

Very Truly Yours,

GeoPentech

Steven K. Duke
Geophysicist
GP 1013

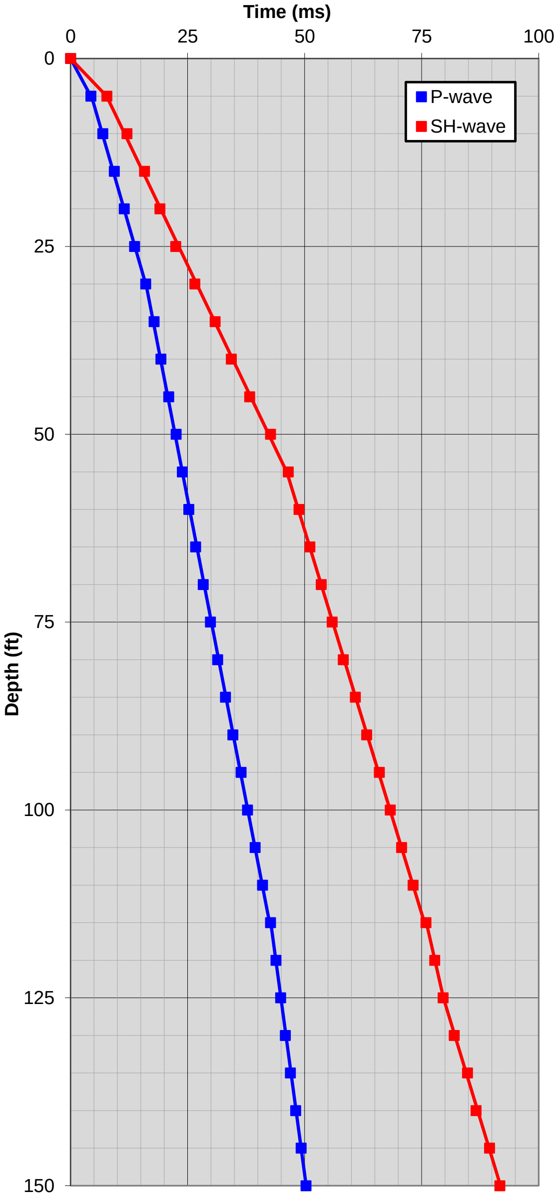
Sarkis Tatusian
Principal
GE 2118

SEISMIC WAVE TRAVEL TIMES

Depth (ft)	P-time (ms)	P-layer	SH-wave (ms)	SH-layer
0	0	1	0	1
5	6.11	12	10.95	12
10	7.66	2	13.44	2
15	9.83	2	16.63	2
20	11.79	2	19.65	2
25	13.91	2	22.90	2
30	16.26	23	26.92	2
35	18.03	3	31.17	2
40	19.41	3	34.60	2
45	21.06	3	38.49	2
50	22.67	3	42.93	2
55	23.96	3	46.66	23
60	25.31	3	48.96	3
65	26.80	3	51.28	3
70	28.39	3	53.66	3
75	29.93	3	55.98	3
80	31.46	3	58.36	3
85	33.13	3	60.94	3
90	34.71	3	63.35	3
95	36.48	3	66.06	3
100	37.85	3	68.38	3
105	39.43	3	70.76	3
110	41.04	3	73.23	3
115	42.75	34	75.99	34
120	43.92	4	77.83	4
125	44.92	4	79.61	45
130	45.92	4	82.01	5
135	47.00	4	84.83	5
140	48.11	4	86.69	5
145	49.29	4	89.54	5
150	50.31	4	91.74	5

Source Offset (ft)
5

TRAVEL TIME PLOT



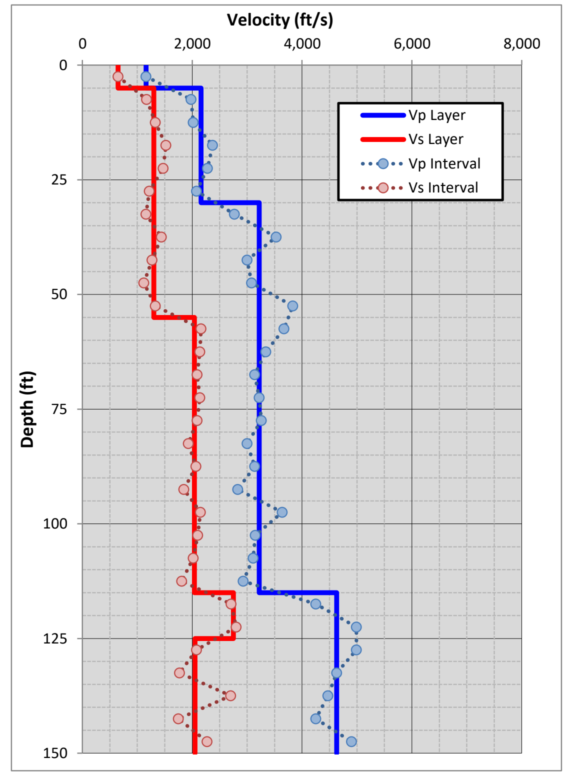
INTERVAL VELOCITES

Depth Range	P-Velocity (ft/s)	SH-Velocity (ft/s)
0 to 5	1,160	650
5 to 10	1,980	1,170
10 to 15	2,020	1,330
15 to 20	2,370	1,520
20 to 25	2,280	1,470
25 to 30	2,080	1,220
30 to 35	2,770	1,160
35 to 40	3,530	1,440
40 to 45	3,000	1,270
45 to 50	3,080	1,120
50 to 55	3,830	1,330
55 to 60	3,670	2,160
60 to 65	3,340	2,140
65 to 70	3,140	2,090
70 to 75	3,220	2,140
75 to 80	3,260	2,090
80 to 85	3,000	1,930
85 to 90	3,140	2,070
90 to 95	2,830	1,850
95 to 100	3,640	2,150
100 to 105	3,150	2,100
105 to 110	3,110	2,020
110 to 115	2,930	1,810
115 to 120	4,250	2,710
120 to 125	4,990	2,800
125 to 130	4,990	2,080
130 to 135	4,630	1,770
135 to 140	4,470	2,700
140 to 145	4,250	1,750
145 to 150	4,900	2,270

LAYER VELOCITES

Layer	P-Depth (ft)	P-Velocity (ft/s)	SH-Depth (ft)	SH-Velocity (ft/s)
1	0 to 5	1,160	0 to 5	650
2	5 to 30	2,160	5 to 55	1,300
3	30 to 115	3,220	55 to 115	2,040
4	115 to 150	4,630	115 to 125	2,750
5			125 to 150	2,050
6				
7				
8				
9				
10				

VELOCITY MODEL



V_{S30} CALCULATION

Vs30 (ft/s)	Depth (ft)
1,470	0 to 100
2,040	50 to 150

GeoPentech

SEISMIC DOWNHOLE TEST RESULTS
BORING NUMBER 1

PROJECT: 8TH AND HOPE

PROJECT #: 16106A | DATE: APR 2017 | FIGURE: 1

ATTACHMENT 1

BORING LOG NUMBER 1
GEOTECHNOLOGIES, INC.

BORING LOG NUMBER 1

Mitsui Fudosan | **Date: 12/27/16**

File No. 21358 | **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
				0 --		2½-inch Asphalt over 1½-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to dense, fine grained
				-		
				2 --		
2.5	24	16.7	110.4	-		
				3 --		
				-	SM	Silty Sand, dark brown, moist, dense, fine grained
				4 --		
				-		
5	28	13.9	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	80	3.4	122.0	-		
				8 --	SP/SW	Sand to Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				-		
				9 --		
				-		
10	51	2.5	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	25	2.0	126.6	-		
	50/5"			13 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				14 --		
				-		
15	65	3.7	SPT	15 --		
				-	SW	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
				16 --		
				-		
				17 --		
17.5	70	2.4	128.6	-		
				18 --		
				-		
				19 --		
				-		
20	70	3.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	45	1.6	93.9	-		
	50/4"			23 --		
				-		
				24 --		
				-		
25	37	4.2	SPT	25 --		
				-	SP/SM	Sand to Silty Sand, dark brown, slightly moist, dense, fine grained

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	74	11.1	119.1	-		
				28 --		Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained
				-		
				29 --		
				-		
30	34	9.8	SPT	30 --		
				-	SP	Sand, dark brown, moist, dense, fine to medium grained
				31 --		
				-		
				32 --		
32.5	48	16.1	116.9	-		
	50/5"			33 --	ML	Sandy Silt, dark gray, moist, very stiff
				-		
				34 --		
				-		
35	43	16.4	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	24	13.8	117.0	-		
	50/5"			38 --		
				-		
				39 --		
				-		
40	51	13.1	SPT	40 --		
				-	SM	Silty Sand, dark grayish brown, moist, dense, fine grained
				41 --		
				-		
				42 --		
42.5	82	12.6	119.6	-		
				43 --		
				-		
				44 --		
				-		
45	32	16.2	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	65	19.4	102.9	-		
				48 --		
				-		
				49 --		
				-		
50	31	27.6	SPT	50 --		
				-	ML	Clayey Silt, dark brown to gray, moist, very stiff

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	70	18.2	112.4	-		
				53 --	SM/ML	Silty Sand, dark gray, moist, very dense, fine grained
				-		
				54 --		
				-		
55	53	16.8	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	80	4.1	106.5	-		
				58 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium dense, occasional gravel
				-		
				59 --		
				-		
60	37	11.7	SPT	60 --		
	50/5"			-	SM	Silty Sand, dark gray, moist, very dense, fine to medium grained
				61 --		
				-		
				62 --		
62.5	100/8"	3.6	110.5	-		
				63 --	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				64 --		
				-		
65	40	2.2	SPT	65 --		
	50/3"			-		
				66 --		
				-		
				67 --		
67.5	100/7.5"	2.4	112.7	-		
				68 --		
				-		
				69 --		
				-		
70	80	2.1	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	100/9"	2.5	109.6	-		
				73 --		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				74 --		
				-		
75	90	2.3	SPT	75 --		
				-		

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 76 -- - 77 --		
77.5	100/9"	2.3	106.6	- 78 -- - 79 -- -		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
80	30 50/6"	2.5	SPT	80 -- - 81 -- - 82 --		
82.5	100/8"	3.7	111.3	- 83 -- - 84 -- -		
85	45 50/5"	6.8	SPT	85 -- - 86 -- - 87 --		
87.5	100	14.3	105.9	- 88 -- - 89 -- -	SM/SW	Silty Sand to Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
90	80	3.6	SPT	90 -- - 91 -- - 92 --	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained, occasional gravel
92.5	100/9"	4.4	108.9	- 93 -- - 94 -- -		
95	55	14.5	SPT	95 -- - 96 -- - 97 --	SP/ML	Sand to Sandy Silt, dark gray, moist, dense to stiff, fine to grained
97.5	100/8"	7.3	113.9	- 98 -- - 99 -- -	SP/SW	Sand to Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained, occasional gravel
100	70	4.5	SPT	100 -- -	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained, occasional gravel

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				101 --		
				102 --		
102.5	100/6"	5.8	109.6	-		
				103 --		Sand, yellow to grayish brown, slightly moist, very dense, fine to medium grained, occasional gravel
				104 --		
105	40 50/5"	25.0	SPT	105 --		
				-	ML	Sandy Silt, dark grayish brown, moist, very stiff, fine grained
				106 --		
				107 --		
107.5	100/6"	9.6	115.8	-		
				108 --	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				109 --		
110	89	9.1	SPT	110 --		
				111 --		
				112 --		
112.5	100/8"	6.5	109.5	-		
				113 --		
				114 --		
115	75/7"	17.5	SPT	115 --		
				-	ML	Sandy Silt, dark grayish brown, moist, very stiff, fine to medium grained, occasional gravel
				116 --		
				117 --		
117.5	100/6"	21.8	102.2	-		
				118 --		
				119 --		
120	45 50/4"	17.5	SPT	120 --		
				121 --		
				122 --		
122.5	100/7"	6.3	103.8	-		
				123 --	SP	Sand, gray, slightly moist, very dense, fine grained
				124 --		
125	50/6"	9.4	SPT	125 --		
				-	SM	Silty Sand, dark gray, moist, very dense, fine grained

BORING LOG NUMBER 1

Mitsui Fudosan

File No. 21358

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				126 --		
				127 --		
127.5	100/6.5"	4.6	122.2	128 --	SP	Sand, gray, slightly moist, very dense, fine grained
				129 --		
130	40 50/3"	6.4	SPT	130 --		
				131 --		
				132 --		
132.5	40 50/3"	25.4	94.9	133 --	SM/SP	Silty Sand to Sand, gray, wet, very dense, fine grained
				134 --		
135	35 50/5"	15.0	SPT	135 --		
				136 --		
				137 --		
137.5	38 50/5"	30.9	88.7	138 --	SP	Sand, gray, wet, very dense, fine grained
				139 --		
140	60 50/5"	25.4	SPT	140 --	SM/SP	Silty Sand to Sand, gray, wet, very dense, fine grained fine grained
				141 --		
				142 --		
142.5	100/7"	25.7	98.2	143 --		
				144 --		
145	38 50/5.5"	23.9	SPT	145 --		
				146 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				147 --		Used 8-inch diameter Hollow-Stem Auger
147.5	100/6"	22.1	105.9	148 --		140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				149 --		SPT=Standard Penetration Test
150	38 50/5"	24.7	SPT	150 --		Total Depth 150 feet After waiting 30 minutes Water level was 130' Fill to 3 feet