C-1: GEOTECHNICAL ENGINEERING INVESTIGATION

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Jia Yuan USA Co., Inc.
801 S. Figueroa Street, Suite 1800
Los Angeles, California 90017

Attention: Tracy Chu

Subject: Geotechnical Engineering Investigation
Proposed City Center Hotel and Residential Development
1020 South Figueroa Street, Los Angeles, California

Ladies and Gentlemen:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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1020 SOUTH FIGUEROA STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This investigation included excavation of eight 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.

SITE CONDITIONS

The property is located at 1020 South Figueroa Street, in the City of Los Angeles, California. The site consists of an "L" shaped lot, which is currently developed with a 9-story hotel, a 2-story parking structure, and at-grade parking lots. All of the existing buildings will be demolished as part of the planned development.

The property is bounded by Olympic Boulevard to the north, by Flower Street to the east, by 11th Street to the south, and by Figueroa Street to the west. An 11-story building and a 1 to 2-story restaurant are located immediately adjacent to the subject property at the northeast corner of the site.

The site slopes downward very gently to the south, with approximately 4 feet of elevation change. Drainage across the site is by sheetflow to the city streets. The vegetation on the site consists of isolated trees and planters. The neighboring development consists primarily of residential and commercial structures.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The site is proposed to be developed with two residential towers and a hotel tower. The towers will vary between 30 and 39 stories in height, and will be interconnected with 8-story podium structures. The entire development will be constructed over 3 subterranean parking levels, extending on the order of 35 feet below the existing site grade.

Structural design data were provided by the project structural engineer, Saiful-Bouquet Structural Engineers. The towers will be supported on mat foundations, and the podium structures will be supported on conventional spread footings. Average bearing pressures for the towers will be on the order of 6,000 to 8,000 psf. Typical column footing loads for the podium structures will be on the order of 2,250 kips. Grading will consist of excavations on the order of 45 to 50 feet in depth for the subterranean parking levels and foundation elements.

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

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between February 16, 2015, and February 20, 2015, by excavating eight exploratory borings. The exploratory borings varied between 80 to 130 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. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-8.

Geologic Materials

Fill materials underlying the subject site consist of silty sands to sands, and sandy silts, which are dark brown in color, slightly moist to moist, medium dense to medium firm, fine to medium grained, with occasional gravel. Fill thickness ranging from 3 to 8 feet was encountered in the exploratory borings.

Native soils consist of silty sands to gravelly sands, with occasional layers of sandy and clayey silts. The native soils are yellowish brown to dark grayish brown in color, slightly moist to moist, very dense 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 logs.

Groundwater

Groundwater was not encountered during exploration. 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 100 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

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

Surface Rupture

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

CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

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

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

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

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

Groundwater was not encountered during exploration. 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 100 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 structures.

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

Tsunamis, Seiches and Flooding

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

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 finding of Geotechnologies, Inc. that construction of the proposed high-rise hotel and residential development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Between 3 and 8 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 45 to 50 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 soils. The proposed towers may be supported on mat foundations bearing in the underlying dense native soils. The podium structures may be supported on conventional foundations bearing in the underlying dense native soils.

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

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 1,460 feet/second was measured between 0 and 100 feet, and an average shear wave velocity of 1,660 feet/second was measured between 30 and 130 feet. A Ground Motion Evaluation study will be prepared by GeoPentech for the project.

2013 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.

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	C
Mapped Spectral Acceleration at Short Periods (S_S)	2.315g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.315g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.543g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.814g
Site Coefficient (F_v)	1.3
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.058g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.705g

FILL SOILS

The maximum depth of fill encountered on the site was 8 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 the low to moderate expansion range. The Expansion Index was found to be between 20 and 73 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.

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 Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane Zone designation. A copy of the portion of the map covering the Project Site is included herein.

GRADING GUIDELINES

The following 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. Based on the laboratory test results performed by this firm, the granular soils encountered at the site would require the 95 percent compaction requirement.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 95 percent 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 95 percent compaction is obtained.

Acceptable Materials

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

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 95 percent 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

The proposed towers may be supported on mat foundations bearing in the underlying dense native soils at the level of the planned excavation. The podium structures may be supported on conventional foundations bearing in the underlying dense native soils at the level of the planned excavation.

Mat Foundation

The proposed towers 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. Average bearing pressures for the tower mat foundations will be on the order of 6,000 to 8,000 psf. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

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 8,000 pounds per square foot may be utilized. The mat foundation may be designed utilizing a modulus of subgrade reaction of 200 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

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 500 pounds per square foot. The bearing capacity increase for each additional foot of depth is 750 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 structures, 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 0.35 may be used with the dead load forces.

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

Foundation Settlement

It is anticipated that total settlement on the order of 2½ inches will occur below the more heavily loaded central core portions of the mat foundation beneath the residential tower. Settlement on the edges of the mat foundation is expected to be on the order of 1½ inch.

The maximum settlement of a typical column footing (approximately 2,250 kips) below the podium structures is expected to be less than ¾ 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 ¾ 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)
35 feet	42 pcf	65 pcf
40 feet	45 pcf	65 pcf
45 feet	47 pcf	65 pcf
50 feet	49 pcf	65 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 26 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 a minimum of 95 percent of the maximum density as determined by the latest revision of ASTM D 1557.

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 95 percent of the maximum density obtainable by the latest revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 130 feet which corresponds to 100 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 45 to 50 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 30 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.

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 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 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

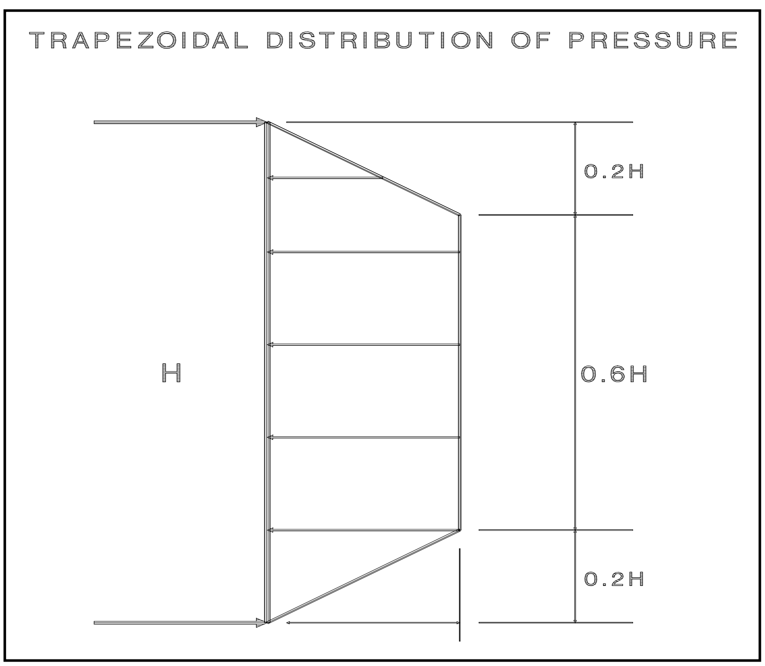
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less. 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
40 feet	40 pcf	26H psf
50 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 (h:v) 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 (h:v) 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 95 percent of the maximum dry density.

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

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore 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 trimable, 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 95 percent relative compaction.

Slab Reinforcing

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

SOIL CORROSIVITY STUDY

A soil corrosivity study was performed by HDR, Inc. The results of soil corrosion potential testing indicate that the electrical resistivities of the soils were in the mildly corrosive to corrosive categories in the as-received moisture, and in the moderately to severely corrosive categories when saturated. Soil pH values of the samples ranged between 7.5 and 8.6, indicating mild to strongly alkaline conditions. The chemical content of the samples was low and moderate. Sulfate content is negligible.

In summary, the soils are classified as severely corrosive to ferrous metals. Detailed results, discussion of results and recommended mitigating measures are provided in the HDR report presented at the end of this report. Any questions regarding the results of the soil corrosion report should be addressed to HDR, Inc.

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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SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
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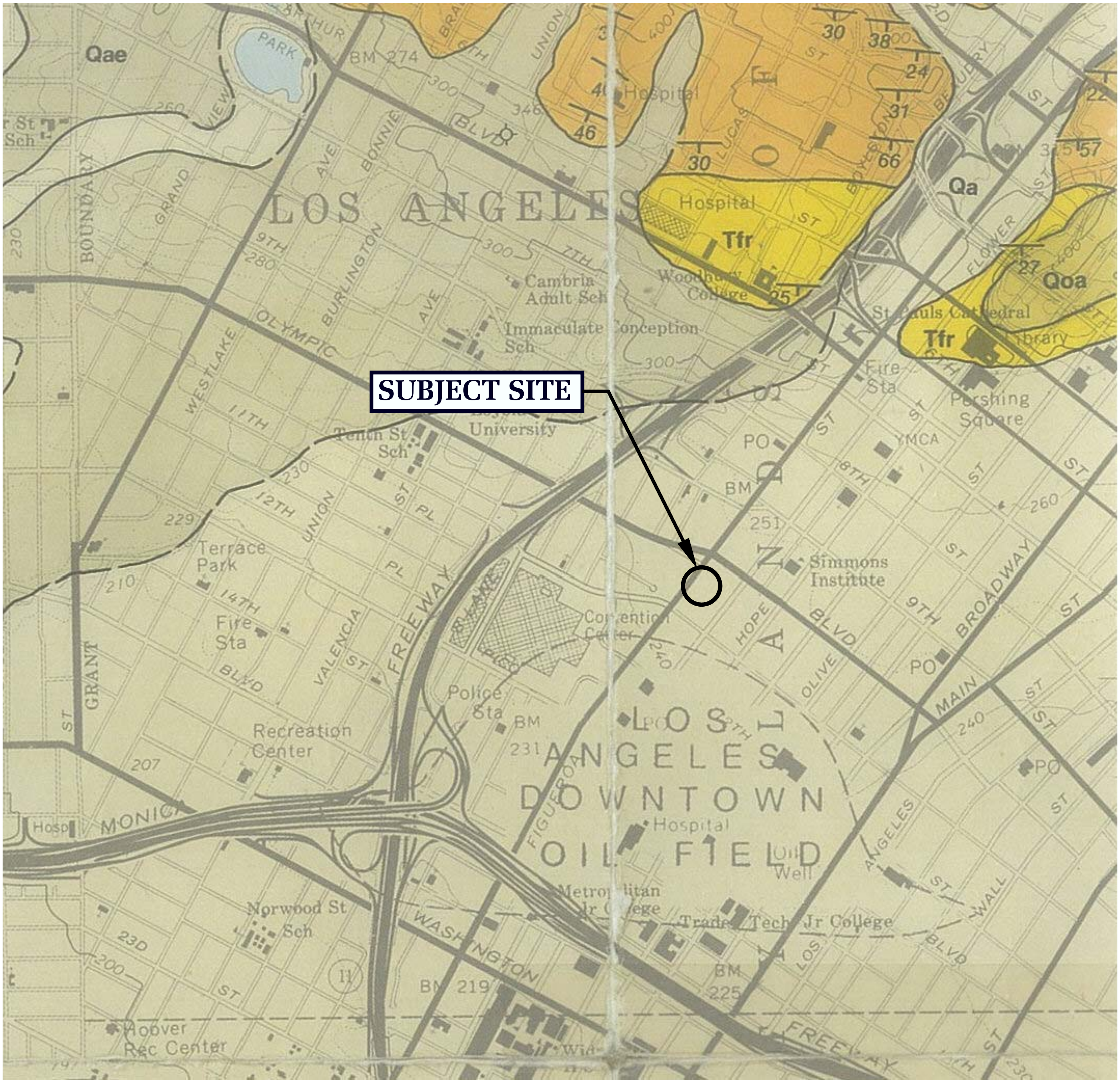
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
HOLLYWOOD, CA QUADRANGLE

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

JIA YUAN USA COMPANY, INC.

FILE NO. 20766



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 of detritus from Santa Monica Mtns.

Tfr: Fernando Formation - Repetto member (Lamar 1970) consolidated but crumbly gray to greenish-gray claystone-siltstone, in part sandy

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

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

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LOCAL GEOLOGIC MAP - DIBBLEE

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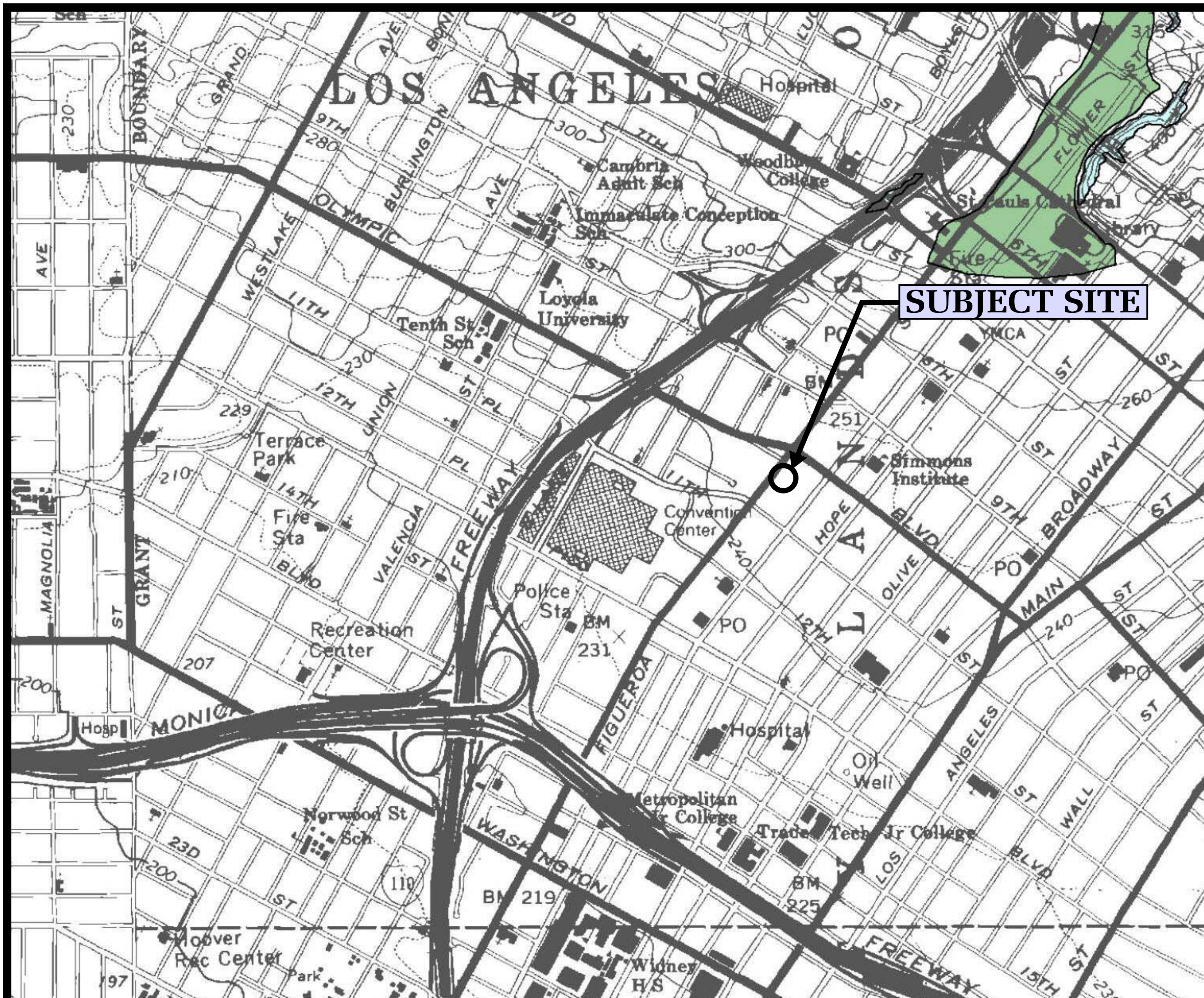
Consulting Geotechnical Engineers

JIN YUAN USA COMPANY, INC.

FILE NO. 20766



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

LIQUEFACTION AREA

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

SEISMIC HAZARD ZONE MAP

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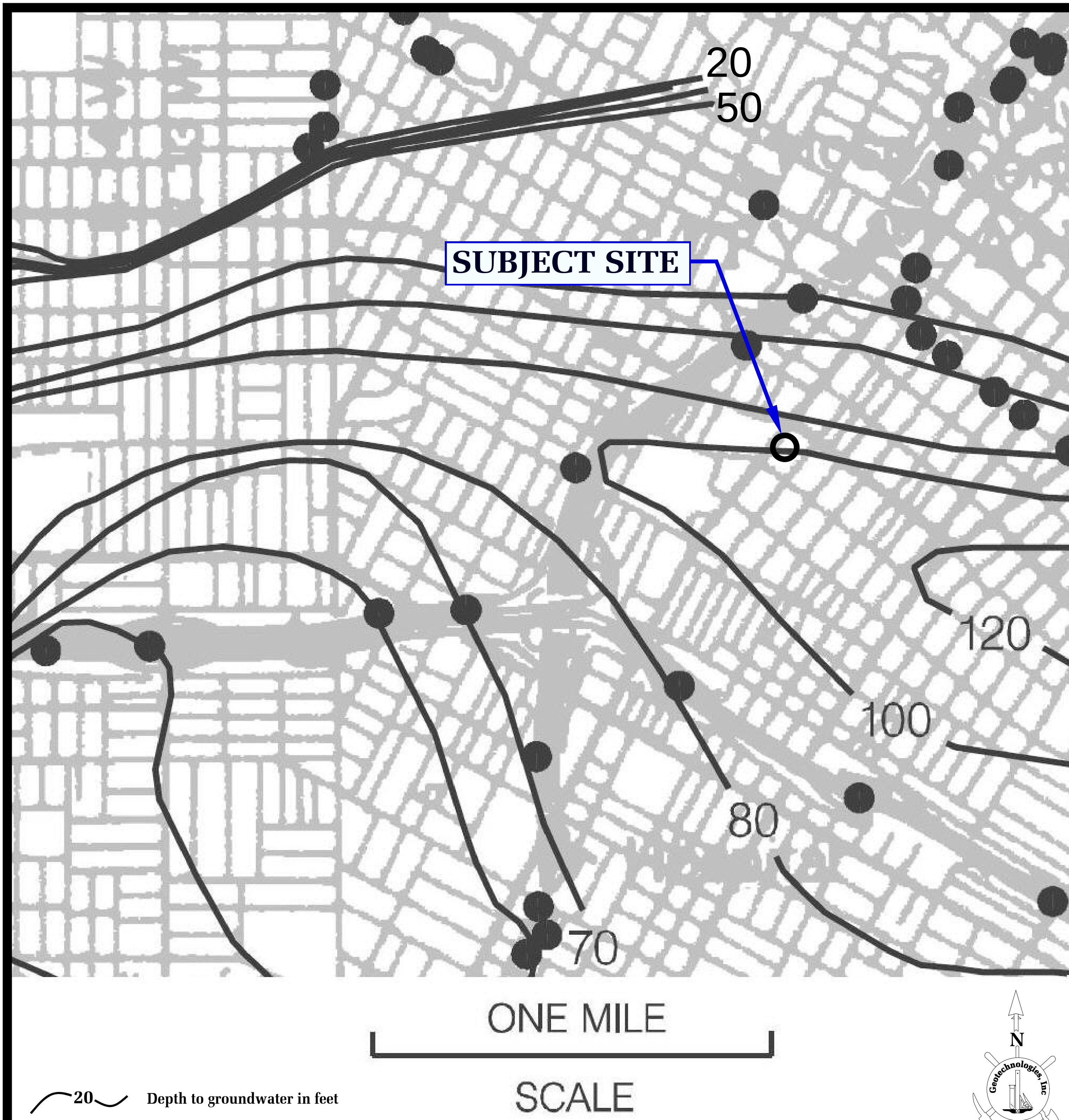

20 Depth to groundwater in feet

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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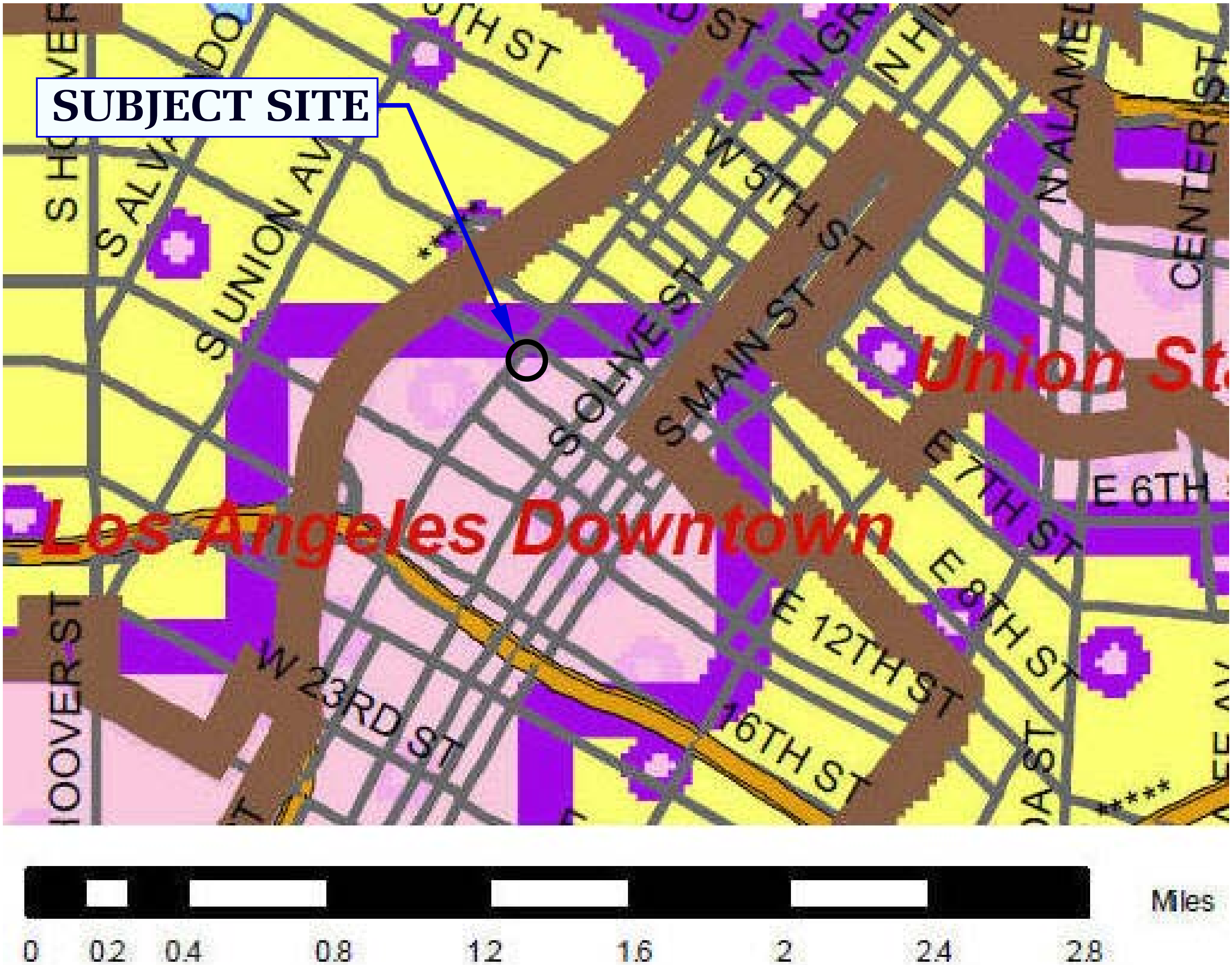

LEGEND

- Methane Zone
- Methane Buffer Zone
- Council District Boundary

REFERENCE: GIS Mapping, Bureau of Engineering, Department fo Public Works - 09/24/03

METHANE ZONE RISK MAP

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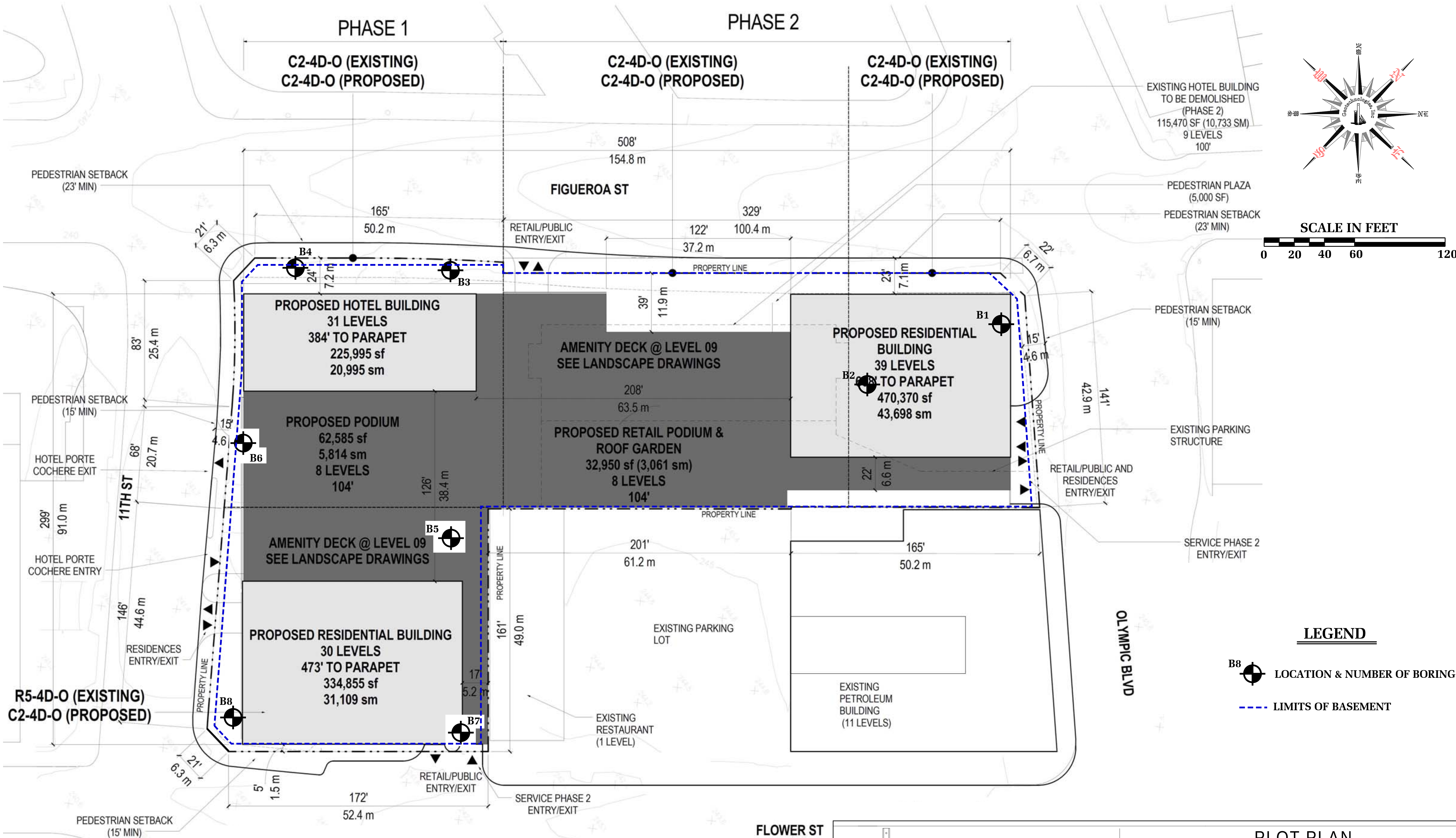
FILE NO. 20766



REFERENCE: PLOT PLAN BY GENSLER MARCH 26, 2015

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PLOT PLAN

JIA YUAN USA COMPANY, INC.

File No.: 20766

Date: April '15

BORING LOG NUMBER 1

Hazens Investment, LLC **Date: 02/18/15** **Elevation: 245.5'**

File No. 20766 **Method: 8-inch diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 4-inches Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, stiff to medium dense, fine grained
				-		
				2 --		
2.5	24	19.2	107.8	-		
				3 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				4 --		
				-		
5	36	3.3	121.0	5 --		
	50/4"			-	SM/SW	Silty Sand to Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained, with gravel
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	27	2.7	139.1	10 --		
	50/5"			-	SW	Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	37	4.8	119.5	15 --		
	50/3"			-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	42	3.7	113.9	20 --		
	50/4"			-		Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained, with cobbles
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	41	7.9	129.9	25 --		
	50/4"			-	SP	Sand, dark to yellowish brown, slightly moist, very dense, fine to medium grained.

BORING LOG NUMBER 1

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	44	7.5	119.6	30 --		
				-		Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	52	13.2	120.4	35 --		
				-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	68	19.8	109.9	40 --		
				-	ML	Sandy Silt, dark grayish brown, moist, very stiff, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	43	14.1	119.2	45 --		
	50/6"			-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	42	18.3	111.2	50 --		
				-	SM/ML	Sandy Silt to Silty Sand, dark brown, moist, very dense to stiff, fine grained

BORING LOG NUMBER 1

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51		
				52		
				53		
				54		
55	58	12.9	119.6	55		
					SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				56		
				57		
				58		
				59		
60	32 50/6"	14.3	113.3	60		
						Silty Sand, dark grayish brown, moist, very dense, fine grained
				61		
				62		
				63		
				64		
65	28 50/6"	16.1	118.2	65		
				66		
				67		
				68		
				69		
70	39 50/4"	10.2	126.9	70		
					SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				71		
				72		
				73		
				74		
75	41 50/3"	5.2	110.3	75		

BORING LOG NUMBER 1

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
				78		
				79		
80	44 50/3"	2.8	114.8	80		
				81		Total Depth 80 feet 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		
				85		Used 8-inch diameter Hollow-Stem Auger 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		

BORING LOG NUMBER 2

Hazens Investment, LLC **Date: 02/16/15** **Elevation: 246.5'**

File No. 20766 **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 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained, occasional gravel
				-		
				2 --		
2.5	19	11.1	111.9	-		
				3 --		
				-	ML	Sandy Silt, dark to medium brown, moist, stiff, fine grained
				4 --		
				-		
5	23	16.9	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	88	13.0	120.6	-		
				8 --	SM	Silty Sand, dark brown, moist, very dense, fine grained, occasional cobbles
				-		
				9 --		
				-		
10	80	2.3	SPT	10 --		
				-	SW	Gravelly Sand, dark brown to yellowish brown, slightly moist, very dense, fine to coarse grained
				11 --		
				-		
				12 --		
12.5	100/8"	2.0	126.6	-		
				13 --		
				-		
				14 --		
				-		
15	78	1.9	SPT	15 --		
				-		Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				16 --		
				-		
				17 --		
17.5	67 50/4"	3.0	119.8	-		
				18 --		
				-		
				19 --		
				-		
20	81	3.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	62	15.6	108.0	-		
				23 --	ML	Sandy Silt, dark grayish brown, moist, stiff, fine grained
				-		
				24 --		
				-		
25	41	11.8	SPT	25 --		
				-	SP/SM	Sand to Silty Sand, dark grayish brown, moist, dense, fine grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

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	10.5	113.8			
				28		Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained
				29		
30	79	5.5	SPT	30		
					SP/SW	Sand to Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				31		
				32		
32.5	38 50/3"	3.6	114.0			
				33		
				34		
35	41	13.2	SPT	35		
					SM	Silty Sand, dark grayish brown, moist, dense, fine grained
				36		
				37		
37.5	77	12.7	121.2			
				38		
				39		
40	45	15.1	SPT	40		
				41		
				42		
42.5	47 50/5"	17.4	112.1			
				43	ML	Sandy Silt to Clayey Silt, dark grayish brown, moist, very stiff
				44		
45	34	16.3	SPT	45		
				46		
				47		
47.5	48	17.7	116.0			
				48		Sandy to Clayey Silt, dark and grayish brown mottling, moist, stiff
				49		
50	23	18.5	SPT	50		
					SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	83	16.7	113.9	-		
				53 --		Sandy Silt to Silty Sand, dark and grayish brown, moist, very dense to very stiff, fine grained
				-		
				54 --		
				-		
55	49	17.9	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	72	20.9	106.9	-		
				58 --	ML	Sandy Silt, dark grayish brown, moist, very stiff
				-		
				59 --		
				-		
60	25	18.5	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	90	17.8	113.1	-		
				63 --	SM	Silty Sand, dark brown, moist, very dense, fine grained
				-		
				64 --		
				-		
65	43	16.4	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	68	15.3	118.6	-		
				68 --		
				-		
				69 --		
				-		
70	47	16.0	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	39	6.8	121.4	-		
	50/5"			73 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				74 --		
				-		
75	38	2.1	SPT	75 --		
	50/3"			-	SW	Gravelly Sand, dark to medium brown, slightly moist, very dense, fine to coarse grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	24	3.5	114.4	-		
	50/3"			78 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				79 --		
				-		
80	81	3.2	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	100/9"	3.4	112.2	-		
				83 --		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				84 --		
				-		
85	78	2.8	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87.5	80	21.0	106.8	-		
				88 --	ML	Sandy Silt, dark to yellowish brown, moist, very stiff, fine grained
				-		
				89 --		
				-		
90	86	5.6	SPT	90 --		
				-	SP	Sand, yellow to grayish brown, slightly moist, very dense, fine grained
				91 --		
				-		
				92 --		
92.5	100/7"	2.8	112.5	-		
				93 --	SW	Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained
				-		
				94 --		
				-		
95	35	2.8	SPT	95 --		
	50/4"			-		
				96 --		
				-		
				97 --		
97.5	100/7"	7.0	116.0	-		
				98 --	SP	Sand, yellowish brown, slightly moist, very dense, fine grained
				-		
				99 --		
				-		
100	40	2.7	SPT	100 --		
	50/5"			-	SW	Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

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"	5.3	121.7	-		
				103 --		Sand to Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained
				-		
				104 --		
				-		
105	85	3.2	SPT	105 --		
				-	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				106 --		
				-		
				107 --		
107.5	100/7"	9.5	106.8	-		
				108 --		
				-		
				109 --		
				-		
110	40	2.7	SPT	110 --		
	50/5"			-	SW	Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				111 --		
				-		
				112 --		
112.5	100/8"	12.3	113.3	-		
				113 --	ML	Sandy Silt, yellowish brown, moist, very stiff, fine grained
				-		
				114 --		
				-		
115	52	15.5	SPT	115 --		
	50/4"			-		
				116 --		
				-		
				117 --		
117.5	32	14.2	113.9	-		
	50/3"			118 --	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				-		
				119 --		
				-		
120	65	13.6	SPT	120 --		
				-		
				121 --		
				-		
				122 --		
122.5	100/8"	4.4	115.8	-		
				123 --	SP	Sand, yellow to grayish brown, slightly moist, very dense, fine grained
				-		
				124 --		
				-		
125	30	2.8	SPT	125 --		
	50/5"			-		

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				126 --		
				-		
				127 --		
127.5	100/7"	5.4	110.3	-		
				128 --		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				129 --		
				-		
130	43 50/3"	2.9	SPT	130 --		
				-		Total Depth 130 feet
				131 --		No Water
				-		Fill to 3 feet
				132 --		
				-		
				133 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				134 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				135 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				136 --		
				-		SPT=Standard Penetration Test
				137 --		
				-		
				138 --		
				-		
				139 --		
				-		
				140 --		
				-		
				141 --		
				-		
				142 --		
				-		
				143 --		
				-		
				144 --		
				-		
				145 --		
				-		
				146 --		
				-		
				147 --		
				-		
				148 --		
				-		
				149 --		
				-		
				150 --		
				-		

BORING LOG NUMBER 3

Hazens Investment, LLC **Date: 02/17/15** **Elevation: 243.0'**

File No. 20766 **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 --		3½-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense,
				-		
				2 --		
2.5	45	4.3	131.9	-		
				3 --		
				-	SM	Silty Sand, dark brown, slightly moist, dense, fine grained,
				4 --		occasional cobbles
				-		
5	71	3.2	132.0	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	68	2.6	135.5	10 --		
	50/5"			-	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, slightly moist,
				11 --		very dense, fine to coarse grained, with cobbles
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	41	2.8	125.0	15 --		
	50/3"			-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/8"	3.9	124.3	20 --		
				-	SW	Gravelly Sand, yellowish brown, slightly moist, very dense, fine to
				21 --		coarse grained, with cobbles
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	73	18.6	110.1	25 --		
				-	SM/SP	Silty Sand to Sand, dark grayish brown, moist, very dense, fine
						grained

BORING LOG NUMBER 3

Hazens Investment, LLC

File No. 20766

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	45	23.6	100.8	30 --		
				-		Silty Sand to Sand, dark to yellowish brown, moist, dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	41	19.7	110.2	35 --		
				-	SM/ML	Silty Sand to Sandy and Clayey Silt, dark grayish brown, moist, very stiff to very dense, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	61	19.4	109.5	40 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark grayish brown, moist, very stiff
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	39	21.8	107.8	45 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very stiff to dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	76	14.2	119.3	50 --		
				-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained

BORING LOG NUMBER 3

Hazens Investment, LLC

File No. 20766

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	66	18.1	115.5	55 --		- - - - - - - - - - - -
				-		Silty Sand, dark brown to grayish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	74 50/5"	13.1	123.2	60 --		
				-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	71	18.5	111.1	65 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	58	20.1	110.5	70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	35 50/5"	11.3	121.5	75 --		
				-	SM	Silty Sand to Sand, dark brown, moist, very dense, fine grained

BORING LOG NUMBER 3

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76 --		
				77 --		
				78 --		
				79 --	SP	Sand, dark to yellowish brown, slightly moist, very dense, fine grained
80	46 50/3"	3.8	110.7	80 --		
				81 --		Total Depth 80 feet 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 --		
				85 --		Used 8-inch diameter Hollow-Stem Auger 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 --		

BORING LOG NUMBER 4

Hazens Investment, LLC **Date: 02/19/15** **Elevation: 241.5'**

File No. 20766 **Method: 8-inch diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 8-inches Base
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff, fine grained
				2 --		
2.5	75/9"	6.9	129.0	-		
				3 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, slightly moist, very dense, very stiff, fine grained, with cobbles
				4 --		
				-		
5	83	3.3	128.8	5 --		
				-	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	93	3.2	127.9	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	51	5.6	125.2	15 --		
				-	SP/ML	Sand to Sandy Silt, dark grayish brown, slightly moist, very dense to very stiff, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	28	6.6	112.8	20 --		
	50/6"			-	SP	Sand, yellowish brown, slightly moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	86	6.8	112.8	25 --		
				-		

BORING LOG NUMBER 4

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	44	7.1	113.9	30 --		
	50/4"			-		Sand, yellowish brown, slightly moist, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	46	15.6	116.9	35 --		
				-	SM/ML	Silty Sand to Clayey Silt, dark brown to grayish brown, moist, dense to stiff, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	54	17.9	113.4	40 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, very stiff
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	87	12.3	123.7	45 --		
				-	SM/SP	Silty Sand to Sand, dark to grayish brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	78	16.1	116.9	50 --		
				-	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff

BORING LOG NUMBER 4

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	66	17.0	113.2	55 --		
				-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	67	20.2	107.2	60 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very dense to very stiff, fine grained
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	23 50/6"	12.7	123.0	65 --		
				-		
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	70	16.8	114.2	70 --		
				-		Silty Sand to Sandy Silt, dark grayish brown, moist, stiff to very dense, fine grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	30 50/6"	12.2	111.2	75 --		
				-	SP	Sand, yellowish brown, moist, very dense, fine to medium grained

BORING LOG NUMBER 4

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		Sand, yellowish brown, slightly moist, very dense, fine grained,
				-		with cobbles
80	42	4.2	100.6	80 --		
	50/3"			-		Total Depth 80 feet
				81 --		No Water
				-		Fill to 3 feet
				82 --		
				-		
				83 --		
				-		
				84 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				85 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				86 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

BORING LOG NUMBER 5

Hazens Investment, LLC **Date: 02/19/15** **Elevation: 244'**

File No. 20766 **Method: 8-inch diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0		5-inches Asphalt over 3-inches Base
				1 – 4		FILL: Silty Sand to Clayey Silt, dark brown to grayish brown, moist, medium dense to stiff, fine grained, with occasional gravel
5	61	10.7	124.7	5	ML	Sandy to Clayey Silt, dark brown, moist, stiff, fine grained
				6	SM	Silty Sand, dark brown, slightly moist, very dense, fine grained, with cobbles
10	37 50/5"	— —	— —	10		
15	61	12.3	123.5	15		Silty Sand, dark brown, moist, very dense, fine grained
20	30 50/5"	10.9	128.7	20	SP	Sand, dark brown, moist, very dense, fine grained
25	33	19.3	103.1	25	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, stiff to dense, fine grained

BORING LOG NUMBER 5

Hazens Investment, LLC

File No. 20766
sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	30	18.4	103.9	30 --		
	50/6"			-	SP	Sand, dark to yellowish brown, moist, very dense, fine to medium grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	62	17.1	113.4	35 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, very dense to very stiff, fine grained
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	50	18.1	111.6	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	25	18.2	113.6	45 --		
	50/4"			-	ML/CL	Sandy Silt to Silty Clay, dark grayish brown, moist, very stiff
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	51	15.3	118.1	50 --		
				-	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff

BORING LOG NUMBER 5

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 51 -- - 52 -- - 53 -- - 54 -- -		
55	77	12.5	121.4	55 -- - 56 -- - 57 -- - 58 -- - 59 -- -	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
60	36	18.6	111.2	60 -- - 61 -- - 62 -- - 63 -- - 64 -- -		
65	25 50/4"	14.5	117.7	65 -- - 66 -- - 67 -- - 68 -- - 69 -- -	SM/SP	Silty Sand to Sand, grayish brown, moist, very dense, fine to medium grained, occasional gravel
70	48	19.1	110.3	70 -- - 71 -- - 72 -- - 73 -- - 74 -- -	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, very stiff
75	43 50/4"	2.6	113.7	75 -- -	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained

BORING LOG NUMBER 5

Hazens Investment, LLC

File No. 20766
sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
				-		
				78 --		
				-		
				79 --		Gravelly Sand, yellowish brown, slightly moist, very dense,
				-	SW	fine to coarse grained
80	45	2.0	116.8	80 --		
	50/4"			-		Total Depth 80 feet
				81 --		No Water
				-		Fill to 5 feet
				82 --		
				-		
				83 --		
				-		NOTE: The stratification lines represent the approximate
				84 --		boundary between earth types; the transition may be gradual.
				-		
				85 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				86 --		Modified California Sampler used unless otherwise noted
				-		
				87 --		
				-		
				88 --		
				-		
				89 --		
				-		
				90 --		
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
				95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
				100 --		
				-		

BORING LOG NUMBER 6

Hazens Investment, LLC **Date: 02/20/15** **Elevation: 241.5'**

File No. 20766 **Method: 8-inch Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inches Asphalt over 8-inches Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Sand, dark brown, moist, stiff to medium
				2 --		dense, fine grained
2.5	21	12.4	123.4	-		
				3 --		Sandy Silt to Silty Sand, dark brown, moist, stiff to medium
				-		dense, fine grained, occasional brick fragments
				4 --		
				-		
5	27	12.9	122.3	5 --		
				-		Silty Sand to Sandy Silt, dark brown, moist, stiff to medium
				6 --		dense, fine grained, occasional brick and concrete fragments
				-		
				7 --		
7.5	40 50/3"	4.1	118.4	-		
				8 --		
				-	SM/SP	Silty Sand to Sand, dark to yellowish brown, slightly moist, very
				9 --		dense, fine to medium grained, occasional gravel
				-		
10	57	7.7	121.1	10 --		
				-	SP	Sand, medium brown, slightly moist, very dense, fine to medium
				11 --		grained, occasional gravel
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	45	28.5	93.8	15 --		
				-	ML/CL	Clayey Silt to Silty Clay, gray to yellowish brown, moist, very stiff,
				16 --		occasional cobbles
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	59	12.0	125.2	20 --		
				-	SM	Silty Sand, reddish brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	30 50/6"	10.0	128.7	25 --		
				-		Silty Sand, medium brown, moist, very dense, fine grained

BORING LOG NUMBER 6

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	51	15.7	115.3	30 --		
				-		Silty Sand, yellowish brown, moist, very dense, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	32	4.4	117.5	35 --		
	50/6"			-	SP	Sand to Gravelly Sand, medium brown, slightly moist, very dense, fine to medium grained, occasional gravel
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	77	12.0	121.5	40 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, very dense, fine grained
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	53	15.1	116.9	45 --		
				-	SM	Silty Sand, dark to grayish brown, moist, very dense, fine grained
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	74	15.3	116.7	50 --		
				-	ML/CL	Sandy Silt to Silty Clay, dark grayish brown, moist, very stiff

BORING LOG NUMBER 6

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	58	18.5	110.2	55 --		
				-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	54	16.1	115.6	60 --		
				-		
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	80	13.8	120.8	65 --		
				-		Silty Sand, dark and grayish brown mottling, moist, dense, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	36	17.4	113.5	70 --		
				-	ML	Sandy Silt, dark brown, moist, very stiff
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	55	17.3	112.9	75 --		
				-	SM	Silty Sand, dark brown, moist, very dense, fine grained

BORING LOG NUMBER 6

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76 --		
				77 --		
				78 --		
				79 --	SP	Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained, with cobbles
80	45 50/4"	3.5	114.3	80 --		Total Depth 80 feet
				81 --		No Water Fill to 8 feet
				82 --		
				83 --		
				84 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				85 --		
				86 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				87 --		
				88 --		
				89 --		
				90 --		
				91 --		
				92 --		
				93 --		
				94 --		
				95 --		
				96 --		
				97 --		
				98 --		
				99 --		
				100 --		

BORING LOG NUMBER 7

Hazens Investment, LLC **Date: 02/20/15** **Elevation: 243'**

File No. 20766 **Method: 8-inch Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inches Asphalt over 7-inches Base
				-		
				1 --		
				-		FILL: Sandy Silt to Silty Sand, dark brown, moist, medium dense to stiff, fine grained
				2 --		
2.5	35	13.5	118.6	-		
				3 --		
				-	SM/ML	Silty Sand to Clayey Silt, dark brown, moist, dense to stiff, fine grained, with cobbles
				4 --		
				-		
5	43	6.9	129.1	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	30	15.9	91.9	10 --		
	50/3"			-	ML	Sandy to Clayey Silt, dark brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	45	30.0	93.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	61	11.9	125.1	20 --		
				-	SM	Silty Sand, medium brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	39	14.3	119.6	25 --		
				-		

BORING LOG NUMBER 7

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	56	14.3	119.6	30		Silty Sand, dark to grayish brown, moist, very dense, fine grained
				31		
				32		
				33		
				34		
35	72	12.3	114.4	35	SP	Sand, dark to yellowish brown, moist, very dense, fine to medium grained
				36		
				37		
				38		
				39		
40	58	13.5	117.5	40	SM	Silty Sand, dark to grayish brown, moist, very dense, fine grained, occasional gravel
				41		
				42		
				43		
				44		
45	27 50/6"	15.2	117.6	45	SM/SP	Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained, with cobbles
				46		
				47		
				48		
				49		
50	48	14.3	116.0	50	ML	Sandy to Clayey Silt, dark to grayish brown, moist, stiff

BORING LOG NUMBER 7

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				52 --		
				53 --		
				54 --		
55	33	12.8	121.3	55 --		
	50/6"			-	SM/SP	Silty Sand to Sand, dark brown, moist, very dense, fine grained
				56 --		
				57 --		
				58 --		
				59 --		
60	48	17.4	112.0	60 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				61 --		
				62 --		
				63 --		
				64 --		
65	48	18.8	111.8	65 --		
				66 --		
				67 --		
				68 --		
				69 --		
70	71	21.8	106.2	70 --		
				-	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff
				71 --		
				72 --		
				73 --		
				74 --		
75	28	10.7	106.5	75 --		
	50/5"			-	SP	Sand, yellowish brown, moist, very dense, fine grained

BORING LOG NUMBER 7

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
				78		
				79	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained, with cobbles
80	45 50/4"	2.2	113.0	80		
				81		Total Depth 80 feet No Water Fill to 3 feet
				82		
				83		
				84		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				85		Used 8-inch diameter Hollow-Stem Auger 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		

BORING LOG NUMBER 8

Hazens Investment, LLC **Date: 02/20/15** **Elevation: 241.5'**

File No. 20766 **Method: 8-inch Diameter Hollow Stem Auger**

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inches Asphalt over 15-inches Base
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense to stiff, fine grained
				2 --		
2.5	59	7.5	131.5	-		
				3 --		
				-	SM/SP	Silty Sand to Sand, dark brown, slightly moist, dense, fine grained
				4 --		
				-		
5	76	2.5	127.0	5 --		
				-	SP/SW	Sand to Gravelly Sand, dark brown, slightly moist, very dense, fine to coarse grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	24 50/4"	3.8	123.2	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	25	34.9	86.9	15 --		
				-	ML	Sandy to Clayey Silt, dark to grayish brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	57	16.6	113.9	20 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark grayish brown, moist, stiff to very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	36	18.1	109.4	25 --		
				-		

BORING LOG NUMBER 8

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	57	16.8	112.5	30		Silty Sand to Sandy Silt, dark grayish brown, moist, very dense to stiff, fine grained
				31		
				32		
				33		
				34		
35	53	13.7	118.8	35	SM	Silty Sand, dark brown, moist, very dense, fine grained
				36		
				37		
				38		
				39		
40	52	23.2	104.8	40	CL/ML	Clayey Silt to Silty Clay, dark to grayish brown, moist, very stiff
				41		
				42		
				43		
				44		
45	28 50/6"	15.6	115.7	45	SP	Sand, dark brown, moist, very dense, fine grained
				46		
				47		
				48		
				49		
50	31 50/6"	16.9	115.3	50	ML	Sandy Silt, dark brown, moist, very stiff

BORING LOG NUMBER 8

Hazens Investment, LLC

File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	37	13.1	118.1	55 --		
	50/6"			-	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	63	22.8	104.3	60 --		
				-	ML	Sandy to Clayey Silt, dark grayish brown, moist, very stiff
				61 --		
				-		
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
65	32	17.0	113.2	65 --		
	50/6"			-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				66 --		
				-		
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
70	26	11.8	123.6	70 --		
	50/6"			-	SM/SP	Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
75	42	2.9	114.3	75 --		
	50/4"			-	SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained

BORING LOG NUMBER 8

Hazens Investment, LLC

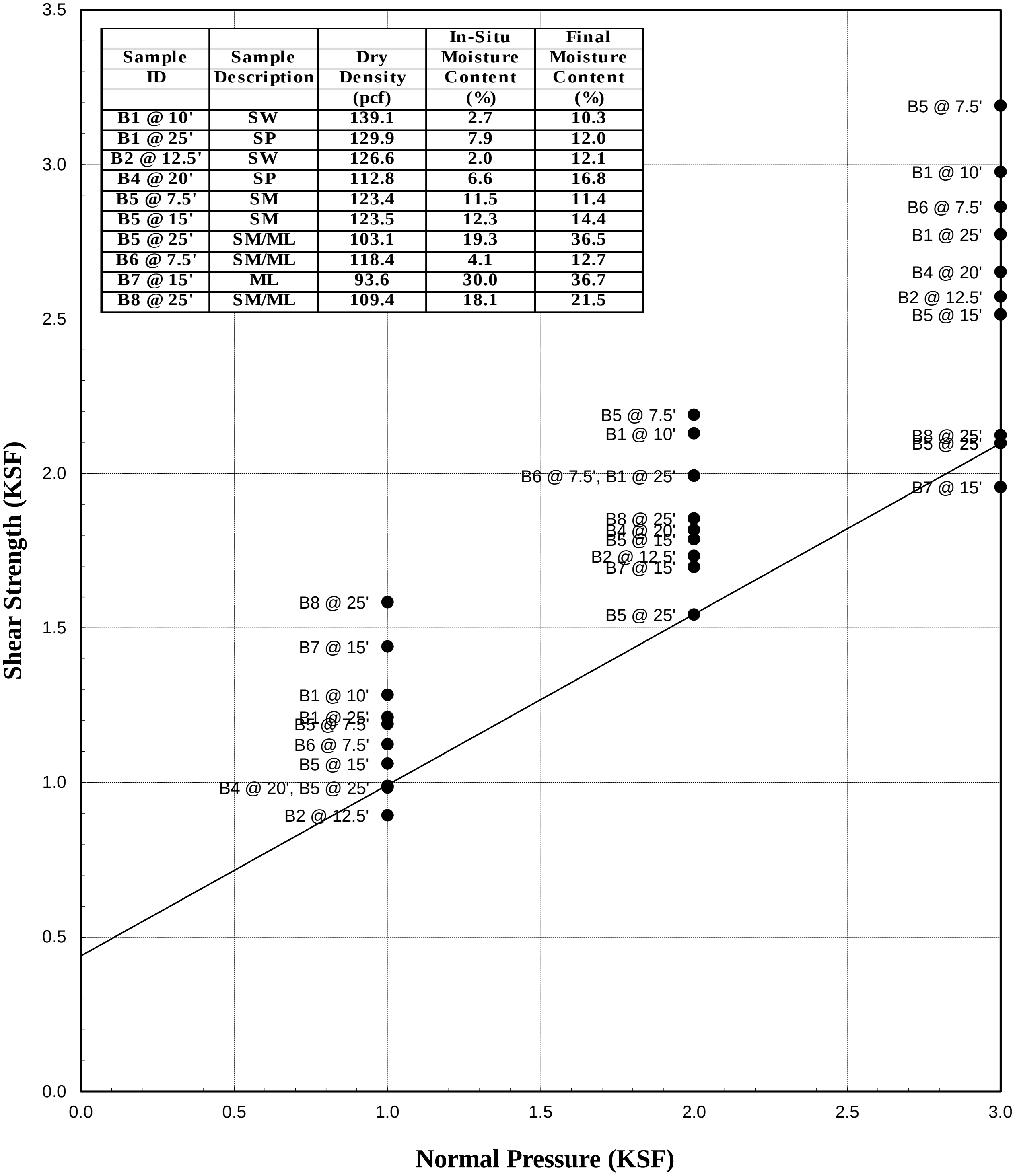
File No. 20766

sa

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76 --		
				77 --		
				78 --		
				79 --		Sand, yellowish brown, slightly moist, very dense, fine to coarse grained
80	39 50/6"	2.9	115.6	80 --		
				81 --		Total Depth 80 feet No Water Fill to 3 feet
				82 --		
				83 --		
				84 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				85 --		
				86 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				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 @ 10'	SW	139.1	2.7	10.3
B1 @ 25'	SP	129.9	7.9	12.0
B2 @ 12.5'	SW	126.6	2.0	12.1
B4 @ 20'	SP	112.8	6.6	16.8
B5 @ 7.5'	SM	123.4	11.5	11.4
B5 @ 15'	SM	123.5	12.3	14.4
B5 @ 25'	SM/ML	103.1	19.3	36.5
B6 @ 7.5'	SM/ML	118.4	4.1	12.7
B7 @ 15'	ML	93.6	30.0	36.7
B8 @ 25'	SM/ML	109.4	18.1	21.5



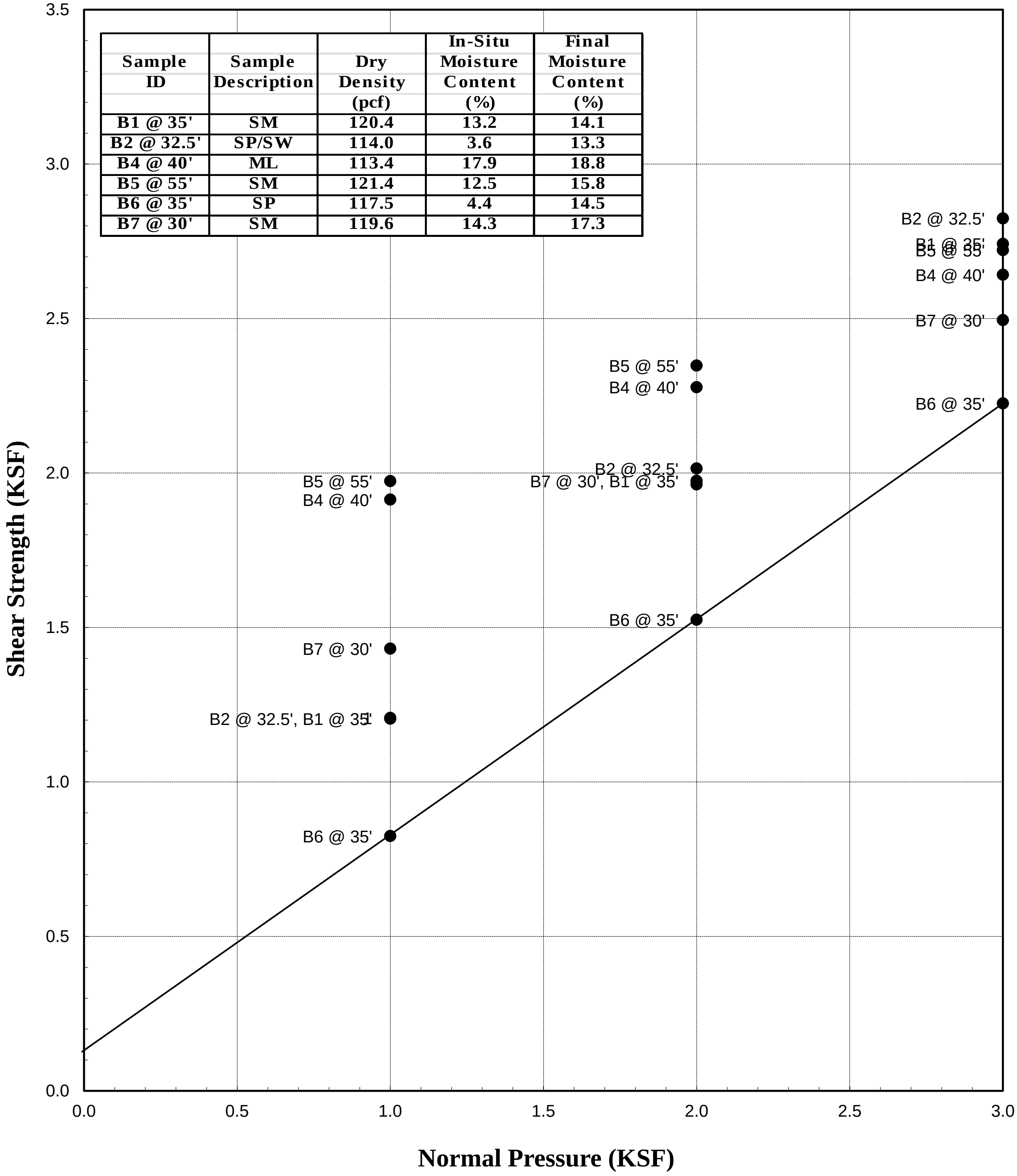
ϕ: 29.0 degrees
c: 435.0 psf

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JIA YUAN USA CO., INC.	
FILE NO.: 20766	PLATE: B-1

Saturated Shear



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 35'	SM	120.4	13.2	14.1
B2 @ 32.5'	SP/SW	114.0	3.6	13.3
B4 @ 40'	ML	113.4	17.9	18.8
B5 @ 55'	SM	121.4	12.5	15.8
B6 @ 35'	SP	117.5	4.4	14.5
B7 @ 30'	SM	119.6	14.3	17.3

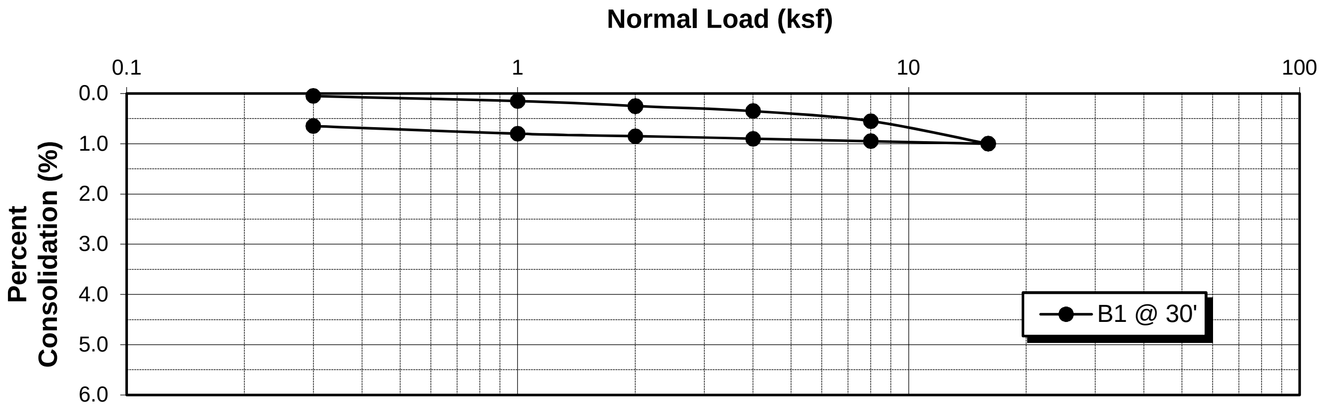
ϕ: 35.0 degrees
c: 125.0 psf

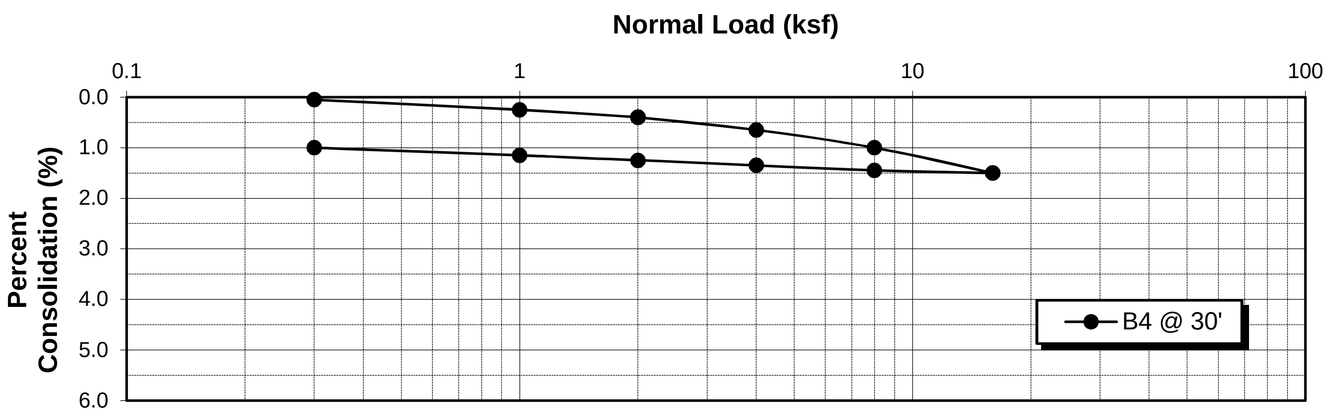
SHEAR TEST DIAGRAM

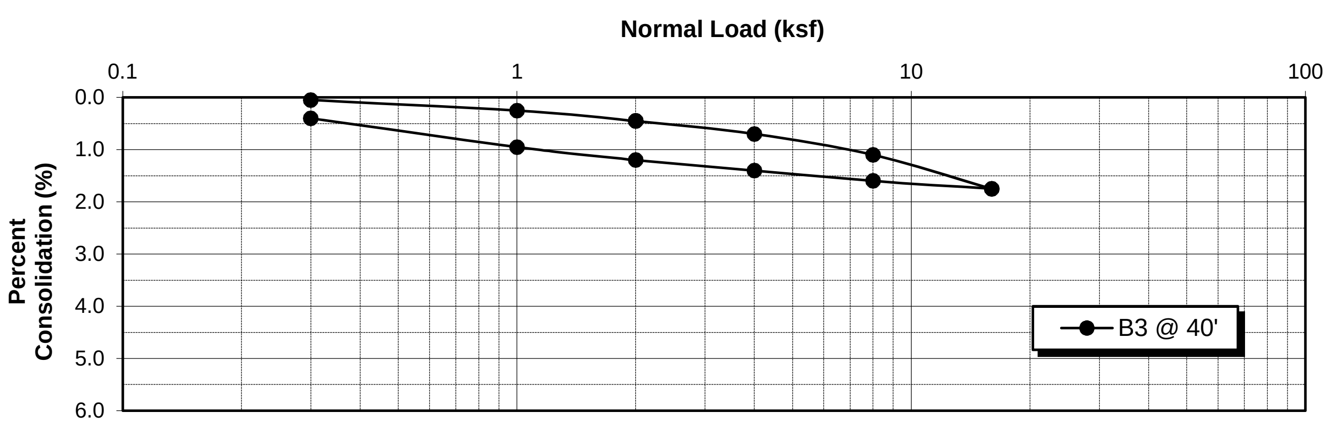
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JIA YUAN USA CO., INC.

FILE NO.: 20766 | PLATE: B-2





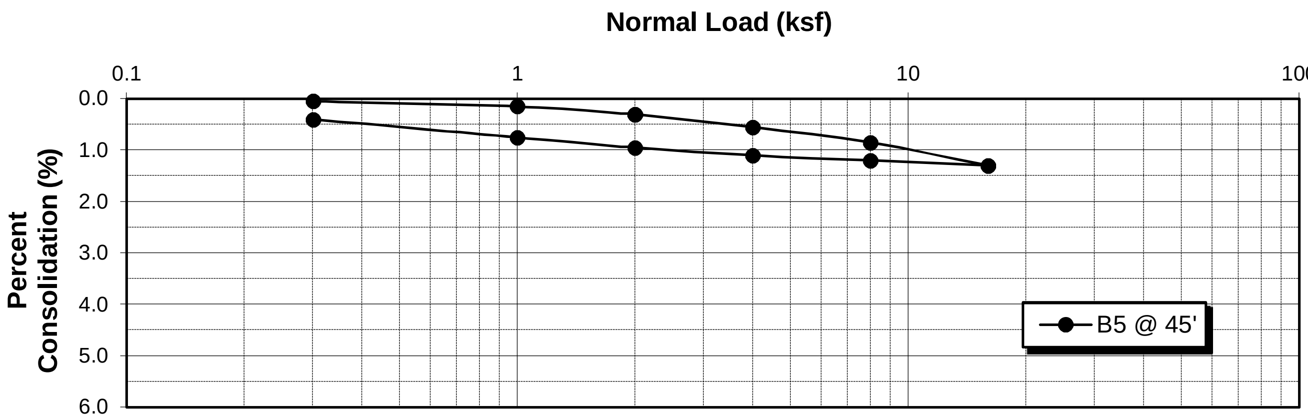


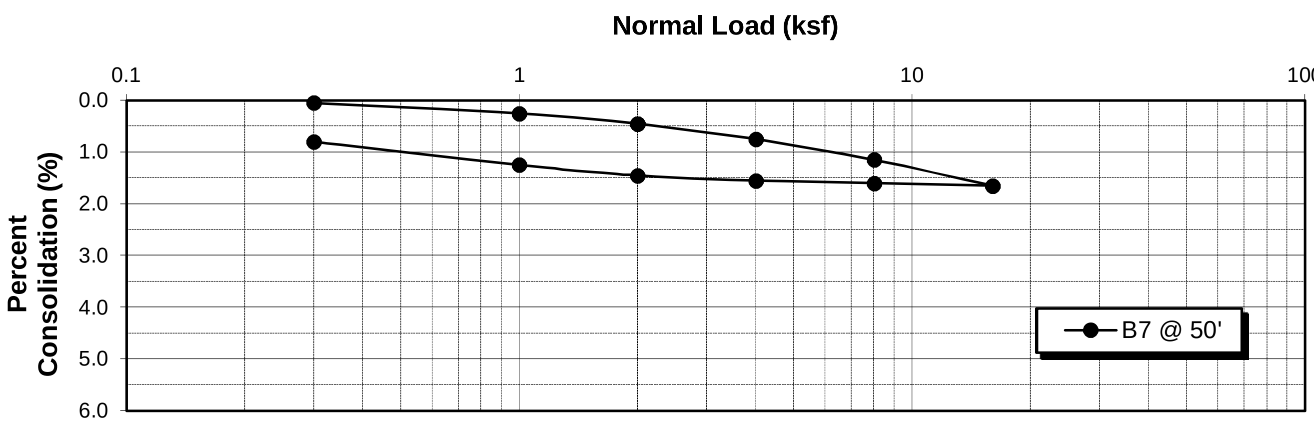
Water added at 2 KSF

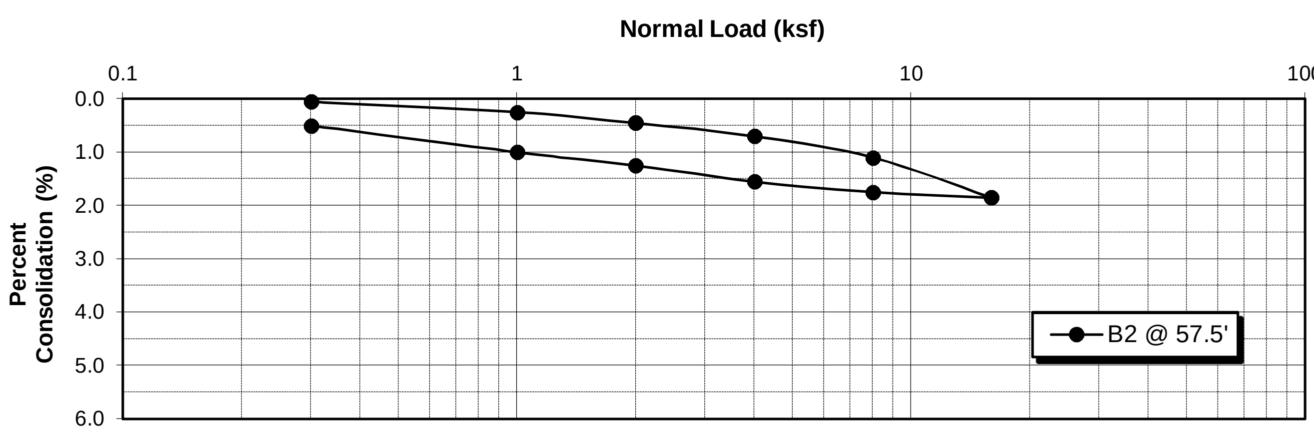
CONSOLIDATION

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JIA YUAN USA CO., INC.	
FILE NO. 20766	PLATE: C-1



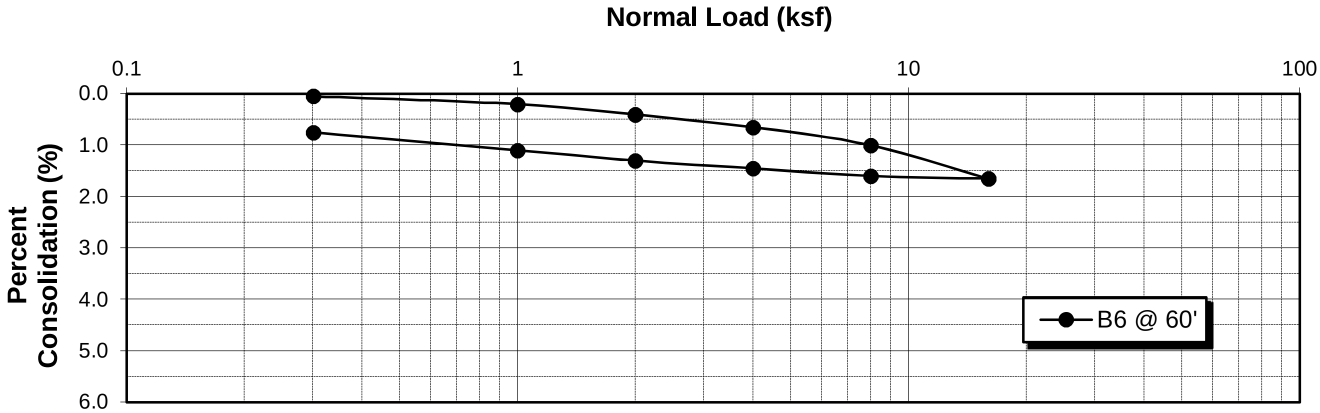


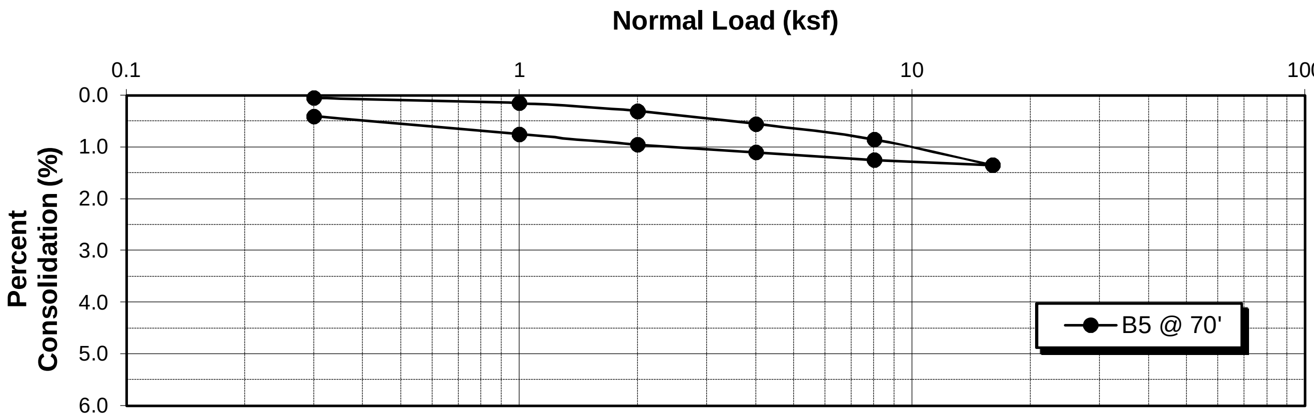


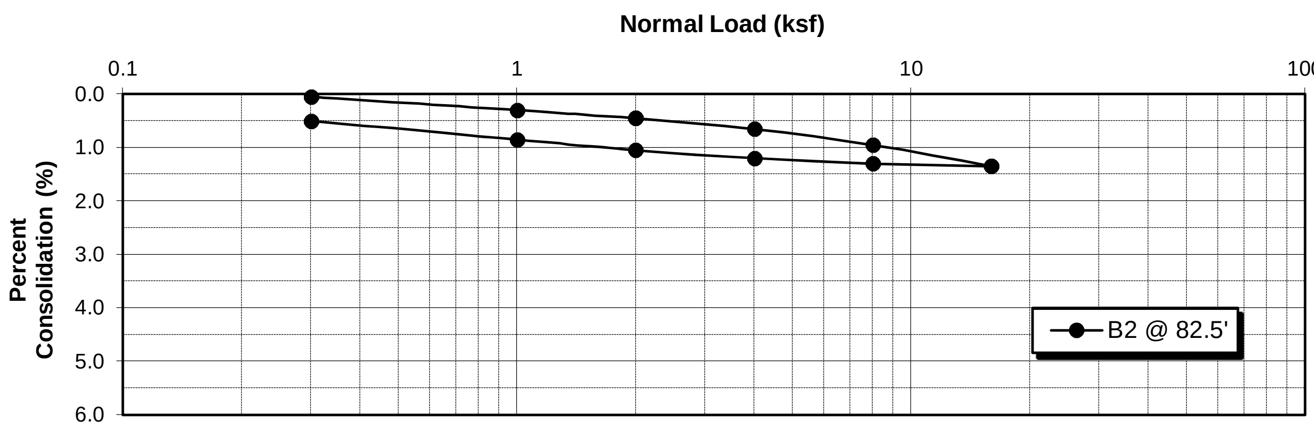
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc. CONSULTING GEOTECHNICAL ENGINEERS	PROJECT: JIA YUAN USA CO., INC.	
	FILE NO. 20766	PLATE: C-2







Water added at 2 KSF

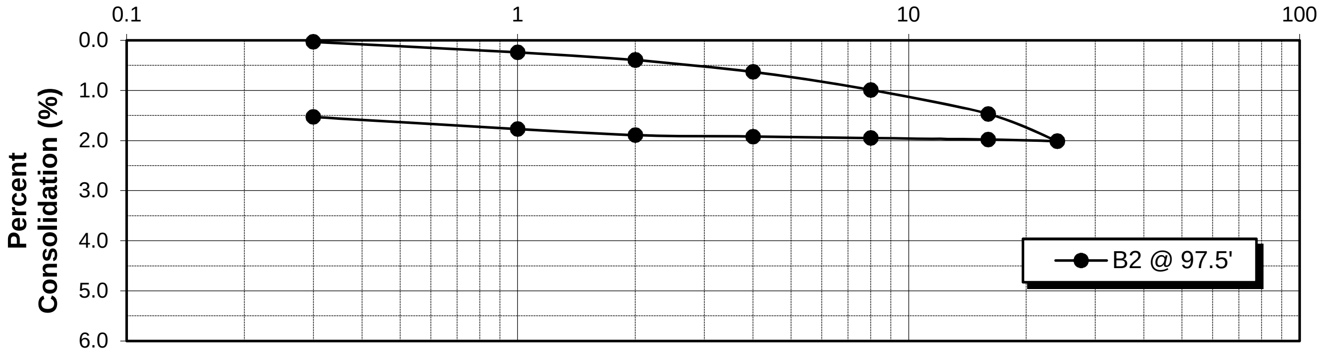
CONSOLIDATION

Geotechnologies, Inc.

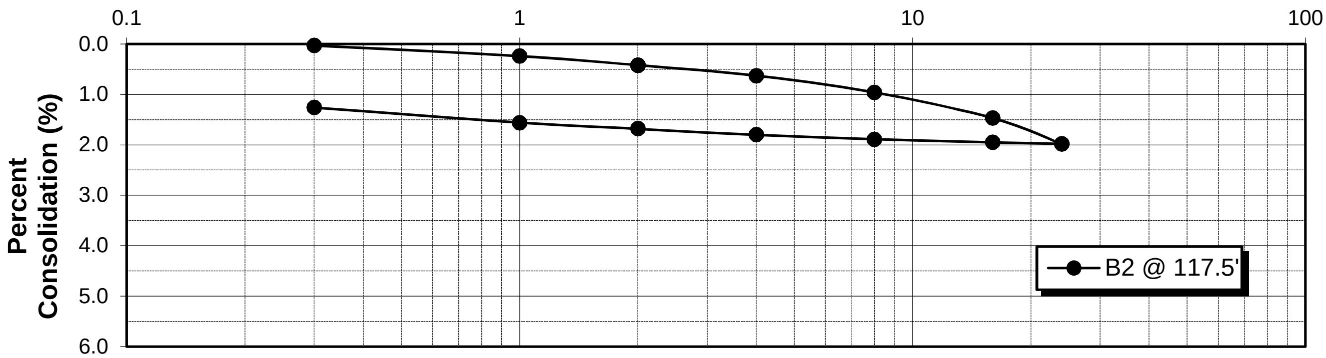
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JIA YUAN USA CO., INC.	
FILE NO. 20766	PLATE: C-3

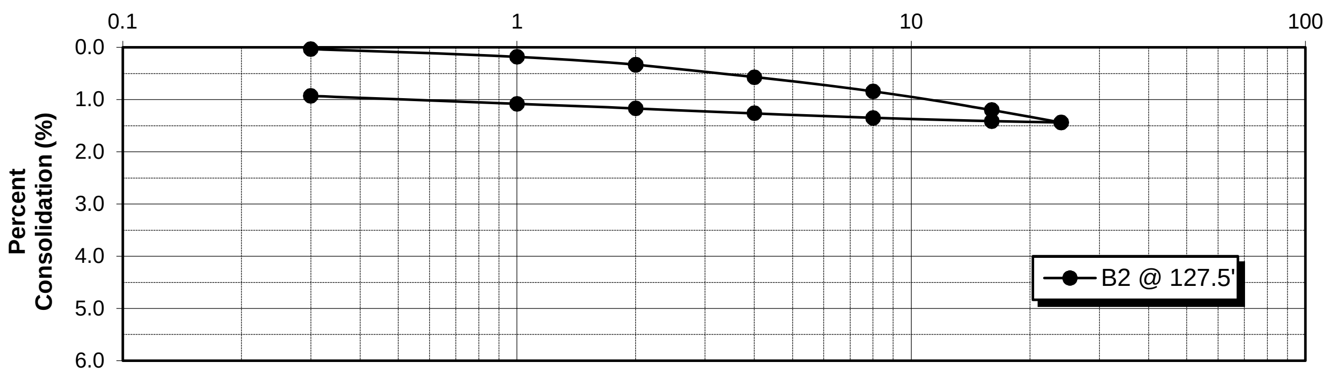
Normal Load (ksf)



Normal Load (ksf)



Normal Load (ksf)



Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc. CONSULTING GEOTECHNICAL ENGINEERS	PROJECT: JIA YUAN USA CO., INC.	
	FILE NO. 20766	PLATE: C-4

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

Jia Yuan USA Co., Inc.
File No. 20766

COMPACTION/EXPANSION/SULFATE DATA SHEET

ASTM D-1557

Sample	B1 @ 1' – 5'	B4 @ 1' – 5'	B7 @ 1' – 5'
Soil Type	SM	SM	SM
Maximum Density (pcf)	128.5	138.5	129.0
Optimum Moisture Content (percent)	10.0	8.0	10.5
Percent finer than 0.005mm (percent)	<15%	<15%	<15%

EXPANSION INDEX

Sample	B1 @ 1' – 5'	B4 @ 1' – 5'	B7 @ 1' – 5'
Soil Type	SM	SM	SM
Expansion Index – UBC Standard 18-2	66	20	73
Expansion Characteristic	Moderate	Low	Moderate

SULFATE CONTENT

Sample	B1 @ 1' – 5'	B4 @ 1' – 5'	B7 @ 1' – 5'
Sulfate Content (ppm)	<250	<250	<250

PLATE D

June 2, 2015 via email: Stang@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Avenue
Glendale, CA 91201

Attention: Mr. Stanley Tang

Re: Soil Corrosivity Study
Jia Yuan USA Co., Inc.
Los Angeles, California
HDR #257874, GI #20766

Introduction

Laboratory tests have been completed on four soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed construction consists of a hotel and residential towers with two subterranean levels. The site is located at 1020 South Figueroa Street in Los Angeles and the water table is reportedly more than 100 feet deep.

The scope of this study is limited to a determination soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory analysis was performed under HDR laboratory number 15-0366SCS and the test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive and corrosive categories with as-received moisture. When saturated, the resistivities were in the moderately to severely corrosive categories. The resistivities dropped considerably with added moisture because the samples were dry as-received.

---

[1] *American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

Soil pH values varied from 7.5 to 8.6. This range is mildly to strongly alkaline.[3] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low.

Nitrate was detected in low concentrations.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as severely corrosive to ferrous metals.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

---

[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

   a. Dissimilar metals.

   b. Dissimilarly coated piping (cement-mortar vs. dielectric).

   c. Above ground steel pipe.

   d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevator

Implement *all* the following measures:

1. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.

2. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders as described above for steel pipe, item #4, option 1.

   b. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   a. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

3. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

   b. Electrically isolate the pipeline.

   c. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   a. Place the oil line in a PVC casing pipe with solvent-welded joints to prevent contact with soil and soil moisture.

Iron Pipe

Implement *all* the following measures:

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of any casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable coating intended for underground use such as:

        i. Polyethylene encasement per AWWA C105; *or*

        ii. Epoxy coating; *or*

        iii. Polyurethane; *or*

        iv. Wax tape.

        NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

    b. Apply cathodic protection to cast and ductile iron piping as per NACE SP0169.

    OPTION 2

    a. As an alternative to coating systems described in Option 1 and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

Copper Tubing

Implement *all* the following measures:

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.
2. Electrically insulate cold water piping from hot water piping systems.
3. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3,000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.
4. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:
    a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*
    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.
2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

Post Tensioning Slabs: Unbonded Single-Stranded Tendons and Anchors

1. Soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against corrosion by implementing *all* the following measures:[8,9,10]

    a. Prior to grouting the pocket, apply a corrosion protection cap filled with corrosion protection material to the strand end that fully encapsulates the strand end and wedge cavity such as Tiger Industries' PocketCap or equal. Ensure the cap fully seats against the anchor face.

---

[4] *2012 International Building Code (IBC) Section 1904.3*

[5] *2012 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[6] *2013 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318 Table 19.3.2.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

[8] *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*

[9] *Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*

[10] *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001*

b. All components exposed to the job site should be protected within one working day after their exposure during installation.

c. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required by the applicable building code.

d. Caps and sleeves should be installed within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.

e. Inspect the following to ensure the encapsulated system is completely watertight:

    i. Sheathing: Verify that all damaged areas, including pin-holes, are repaired.

    ii. Stressing tails: After removal, ensure they are cut to a length for proper installation of P/T coating filled end caps.

    iii. End caps: Ensure proper installation before patching the pocket former recesses.

    iv. Patching: Ensure the patch is of an approved material and mix design, and installed void-free.

f. Limit the access of direct runoff onto the anchorage area by designing proper drainage.

g. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

Jose Peña

Brien L. Clark, PE

PROFESSIONAL ENGINEER
LICENSED
BRIEN L. CLARK
No. CH 6291
Exp. 12-31-15
CHEMICAL
STATE OF CALIFORNIA

Enc: Table 1

257874_SCS_Rpt_JP_Rev00_BC

6-2-15

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Jia Yuan USA Co., Inc
Your #20766, HDR Lab #15-0366SCS
11-May-15

Sample ID			B1 @ 5' SM/SW	B4 @ 10' SP/SW	B5 @ 30' SP	B7 @ 10' ML
Resistivity		**Units**				
as-received		ohm-cm	800,000	328,000	10,000	1,480
saturated		ohm-cm	8,400	8,000	2,880	800
pH			8.6	7.9	7.7	7.5
Electrical Conductivity		mS/cm	0.05	0.13	0.04	0.04
Chemical Analyses						
Cations						
calcium	Ca^{2+}	mg/kg	14	35	14	17
magnesium	Mg^{2+}	mg/kg	3.5	11	4.0	5.1
sodium	Na^{1+}	mg/kg	59	128	37	32
potassium	K^{1+}	mg/kg	1.3	7.6	4.4	2.9
Anions						
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	110	149	58	64
fluoride	F^{1-}	mg/kg	3.2	4.3	0.9	1.4
chloride	Cl^{1-}	mg/kg	3.5	23	3.8	3.3
sulfate	SO_4^{2-}	mg/kg	29	111	32	20
phosphate	PO_4^{3-}	mg/kg	7.1	19	3.2	6.7
Other Tests						
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	ND	0.6	22	ND
sulfide	S^{2-}	qual	na	na	na	na
Redox		mV	na	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

Geotechnologies, Inc.

Project: Jia Yuan USA Company, Inc.

File No.: 20766

Settlement Calculation - Mat Footing

Description: Tower Mat Foundation

Soil Unit Weight	125.0 pcf	Mat Footing	
Bearing Value	8000.0 psf	245000 kips	
Depth of Footing	50.0 feet		
Width of Footing	175.0 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													
	55.0	5.0	35.0	94%	7492	6875	14367	B7 @ 50'	1.50	1.00	0.50	10.0	0.60
60.0													
	65.0	15.0	11.7	89%	7113.2	8125	15238.2	B6 @ 60'	1.50	1.00	0.50	10.0	0.60
70.0													
	80.0	30.0	5.8	77%	6180.4	10000	16180.4	B2 @ 82.5'	1.35	1.15	0.20	20.0	0.48
90.0													
	95.0	45.0	3.9	68%	5452	11875	17327	B2 @ 97.5'	1.50	1.20	0.30	10.0	0.36
100.0													
	110.0	60.0	2.9	59%	4690.4	13750	18440.4	B2 @ 117.5'	1.55	1.30	0.25	20.0	0.60
120.0													
	235.0	185.0	0.9	19%	1490	29375	30865	B2 @ 127.5'	1.55	1.52	0.03	230.0	0.83
350.0													

Settlement: 3.47

Reduction: 0.67

Total Settlement in inches: ***2.31***

Geotechnologies, Inc.

Project: Jia Yuan USA Company, Inc.

File No.: 20766

Settlement Calculation - Column Footing

Description: 2250 kips Column Footing

Soil Unit Weight	125.0 pcf		Column Footing
Bearing Value	10000.0 psf		2250 kips
Depth of Footing	50.0 feet		
Width of Footing	15.0 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													
	55.0	5.0	3.0	61%	6086	6875	12961	B7 @ 50'	1.50	1.00	0.50	10.0	0.60
60.0													
	65.0	15.0	1.0	22%	2165	8125	10290	B6 @ 60'	1.20	1.00	0.20	10.0	0.24
70.0													
	75.0	25.0	0.6	10%	999	9375	10374	B5 @ 70'	1.00	0.95	0.05	10.0	0.06
80.0													

Settlement: 0.90

Reduction: 0.67

Total Settlement in inches: ***0.60***

April 2, 2015

Project No. 14072A

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

**SUBJECT: DOWNHOLE SEISMIC TEST RESULTS
BORING NUMBER 2
1020 SOUTH FIGUEROA STREET
LOS ANGLES, CALIFORNIA**

Dear Mr. Tang,

Per your request and in accordance with the provisions of our proposal, dated December 18, 2014, we performed downhole seismic tests within Boring Number 2 at the subject property located at 1020 South Figueroa Street in Los Angeles, California. The log of Boring Number 2 provided by Geotechnologies, Inc. is provided in Attachment 1 and indicates that the subsurface materials are composed of (1) Artificial Fill from a depth of 0 to approximately 3 feet below ground surface and (2) Alluvium predominantly consisting of sand with occasional gravelly zones (SM, SP and SW) and occasional sandy silt (ML) layers from approximately 3 to 130 feet (bottom of the borehole). Additionally, no groundwater was observed during borehole drilling. Downhole seismic tests were performed within Boring Number 2 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 2 on March 18, 2015. 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".

Boring Number 2 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. The annular space between the 8-inch diameter hole and 2-inch diameter casing was backfilled with 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 (130 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 4.1.1.7. 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, a 56 Hz low-cut filter and 209 Hz high-cut filter were applied to the P waveforms, and a 16 Hz low-cut filter and 107 Hz high-cut filter was 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 2 are presented on Figure 1. Figure 1 shows (1) a table of the measured P and SH-wave travel-times and depths; (2) a table of the interpreted P and SH-wave layer velocities and depth ranges; (3) a table of the calculated P and SH-wave interval velocities; and (4) a plot of the P and SH-wave travel-times as a function of depth showing the interpreted layer velocities.

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 2, as logged by Geotechnologies, Inc. As shown on Table 1, the measured SH-wave velocity from 0 to 10 feet below ground surface is approximately 1,150 ft/sec; from 10 to 45 feet is approximately 1,470 ft/sec; from 45 to 60 feet is approximately 1,100 ft/sec, from 60 to 90 feet is approximately 1,700 ft/sec, and from 90 to 130 feet is approximately 2,120 ft/sec.

Also, the measured P-wave velocity within Boring Number 2 from 0 to 10 feet below ground surface is approximately 2,090 ft/sec; from 10 to 45 feet is approximately 2,340 ft/sec; from 45 to 75 feet is approximately 4,830 ft/sec, from 75 to 90 feet is approximately 3,300 ft/sec, and from 90 to 130 feet is approximately 3,540 ft/sec. The measured P-wave velocity suggests the material adjacent to the borehole is saturated between 45 to 75 feet. No groundwater was observed during drilling; therefore, it is possible that this high P-wave velocity is due to the borehole annular grout.

TABLE 1
SUMMARY OF SH-WAVE AND P-WAVE VELOCITY LAYERS WITHIN BORING NUMBER 2

<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 very dense Sand (SM and SP) and stiff sandy Silt (ML) [Fill/Alluvium]</td><td>0 to 10</td><td>1,150</td><td>2,090</td></tr>
<tr><td>Dense to very dense Sand (SM, SP and SW) with occasional gravel and stiff to very stiff sandy to clayey Silt (ML) layers [Alluvium]</td><td>10 to 45</td><td>1,470</td><td>2,340</td></tr>
<tr><td>Dense to very dense Sand (SM) and stiff to very stiff sandy Silt (ML) [Alluvium]</td><td>45 to 60</td><td>1,100</td><td rowspan="2">4,830*</td></tr>
<tr><td rowspan="3">Very dense Sand (SM, SP and SW) with occasional gravel and some very stiff sandy Silt (ML) layers [Alluvium]</td><td>60 to 75</td><td rowspan="2">1,700</td></tr>
<tr><td>75 to 90</td><td>3,300</td></tr>
<tr><td>90 to 130</td><td>2,120</td><td>3,540</td></tr>
</table>

* It is noted that the measured P-wave velocity for this layer may be influenced by borehole annular grout material.

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 2 was calculated between a depth of 0 to 100 feet and 30 to 130 feet. The results are summarized on Table 2.

TABLE 2
CALCULATED V_{s30} WITHIN BORING NUMBER 2

DEPTH RANGE (ft, below ground surface)	V_{s30} (ft/sec)
0 to 100	1,460
30 to 130	1,660

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
Senior Project Geophysicist
GP 1013

John A. Barneich
Principal
GE 116

SEISMIC WAVE TRAVEL TIMES

Depth (ft)	P-time (ms)	P-layer	SH-wave (ms)	SH-layer
0	0	1	0	1
5	3.47	1	6.14	1
10	5.34	12	9.75	12
15	7.60	2	12.77	2
20	9.75	2	16.72	2
25	11.64	2	19.56	2
30	14.20	2	23.01	2
35	16.44	2	26.09	2
40	18.07	2	29.62	2
45	19.43	23	32.81	23
50	20.39	3	37.02	3
55	21.41	3	41.79	3
60	22.32	3	46.32	34
65	23.45	3	49.44	4
70	24.39	3	53.14	4
75	25.64	34	55.81	4
80	27.27	4	58.25	4
85	28.80	4	60.94	4
90	30.16	45	64.18	45
95	31.41	5	66.63	5
100	32.96	5	69.70	5
105	34.57	5	71.78	5
110	35.71	5	73.74	5
115	37.09	5	75.54	5
120	38.59	5	78.19	5
125	39.94	5	80.40	5
130	41.49	5	83.73	5

Source Offset (ft)
5

Vs30 (fps)	Depth Range (ft, bgs)
1,460	0 to 100
1,660	30 to 130

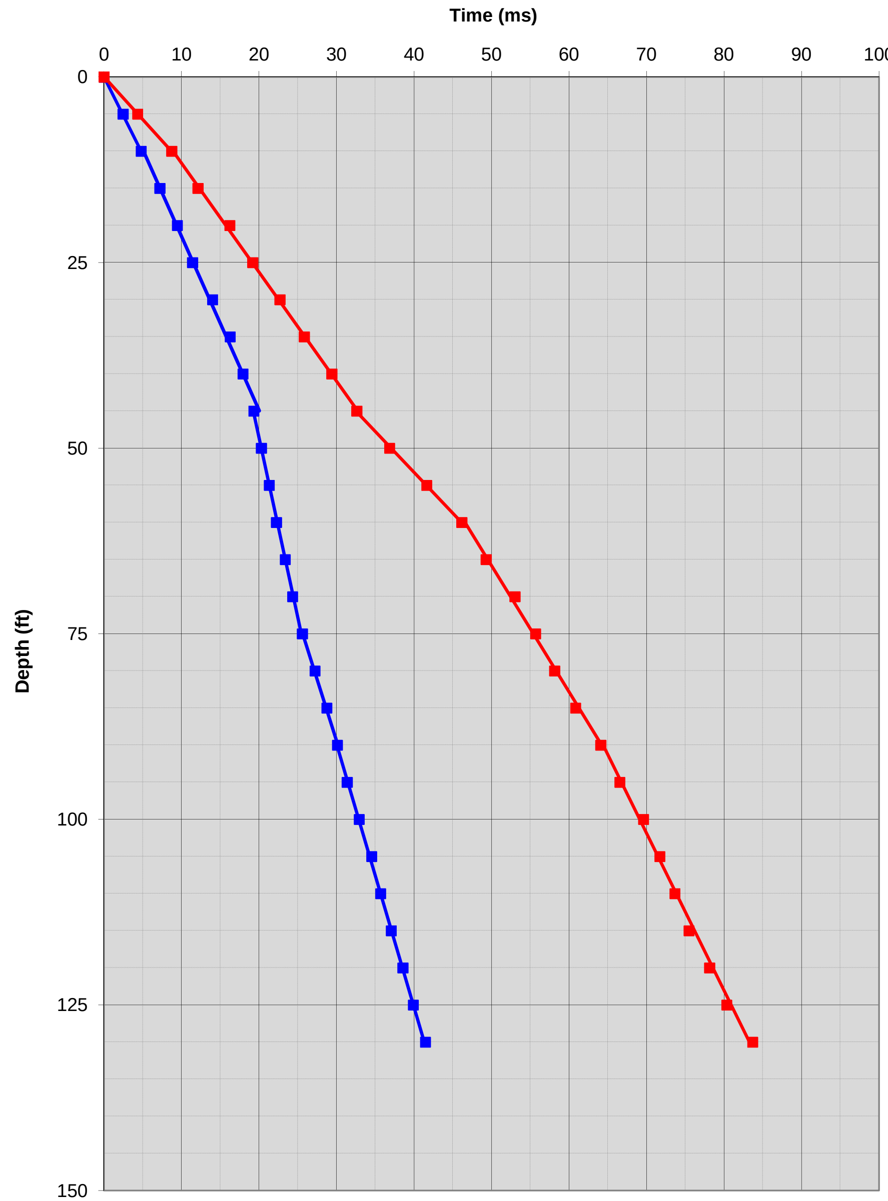
LAYER VELOCITES

Layer	P-Velocity (fps)	P-Depth (ft)	SH-Velocity (fps)	SH-Depth (ft)
1	2,090	0 to 10	1,150	0 to 10
2	2,340	10 to 45	1,470	10 to 45
3	4,830	45 to 75	1,100	45 to 60
4	3,300	75 to 90	1,700	60 to 90
5	3,540	90 to 130	2,120	90 to 130
6				
7				
8				
9				
10				

INTERVAL VELOCITIES

Depth Range (ft)	P-Velocity (fps)	SH-Velocity (fps)
0 to 5	2,040	1,150
5 to 10	2,150	1,140
10 to 15	2,060	1,470
15 to 20	2,220	1,220
20 to 25	2,560	1,690
25 to 30	1,920	1,420
30 to 35	2,210	1,600
35 to 40	3,030	1,400
40 to 45	3,610	1,550
45 to 50	5,100	1,180
50 to 55	4,860	1,050
55 to 60	5,430	1,100
60 to 65	4,390	1,590
65 to 70	5,300	1,350
70 to 75	3,990	1,870
75 to 80	3,040	2,040
80 to 85	3,280	1,850
85 to 90	3,650	1,540
90 to 95	3,990	2,040
95 to 100	3,230	1,630
100 to 105	3,100	2,400
105 to 110	4,360	2,540
110 to 115	3,620	2,770
115 to 120	3,340	1,880
120 to 125	3,700	2,270
125 to 130	3,220	1,500

TRAVEL TIME PLOT



GeoPentech

SEISMIC DOWNHOLE RESULTS
BOREHOLE-2

PROJECT: **1020 SOUTH FIGUEROA STREET**

PROJECT #: **14072A** | DATE: **APR 2015** | FIGURE: **1**

ATTACHMENT 1

BORING LOG NUMBER 2
GEOTECHNOLOGIES, INC.

BORING LOG NUMBER 2

Hazens Investment, LLC **Date: 02/16/15**

File No. 20766 **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 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine to medium grained, occasional gravel
				-		
				2 --		
2.5	19	11.1	111.9	-		
				3 --		
				-	ML	Sandy Silt, dark to medium brown, moist, stiff, fine grained
				4 --		
				-		
5	23	16.9	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	88	13.0	120.6	-		
				8 --	SM	Silty Sand, dark brown, moist, very dense, fine grained, occasional cobbles
				-		
				9 --		
				-		
10	80	2.3	SPT	10 --		
				-	SW	Gravelly Sand, dark brown to yellowish brown, slightly moist, very dense, fine to coarse grained
				11 --		
				-		
				12 --		
12.5	100/8"	2.0	126.6	-		
				13 --		
				-		
				14 --		
				-		
15	78	1.9	SPT	15 --		
				-		Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				16 --		
				-		
				17 --		
17.5	67 50/4"	3.0	119.8	-		
				18 --		
				-		
				19 --		
				-		
20	81	3.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	62	15.6	108.0	-		
				23 --	ML	Sandy Silt, dark grayish brown, moist, stiff, fine grained
				-		
				24 --		
				-		
25	41	11.8	SPT	25 --		
				-	SP/SM	Sand to Silty Sand, dark grayish brown, moist, dense, fine grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

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	10.5	113.8	- 28 -- - 29 -- -		Silty Sand to Sand, dark grayish brown, moist, very dense, fine grained
30	79	5.5	SPT	30 -- - 31 -- - 32 --	SP/SW	Sand to Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
32.5	38 50/3"	3.6	114.0	- 33 -- - 34 -- -		
35	41	13.2	SPT	35 -- - 36 -- - 37 --	SM	Silty Sand, dark grayish brown, moist, dense, fine grained
37.5	77	12.7	121.2	- 38 -- - 39 -- -		
40	45	15.1	SPT	40 -- - 41 -- - 42 --		
42.5	47 50/5"	17.4	112.1	- 43 -- - 44 -- -	ML	Sandy Silt to Clayey Silt, dark grayish brown, moist, very stiff
45	34	16.3	SPT	45 -- - 46 -- - 47 --		
47.5	48	17.7	116.0	- 48 -- - 49 -- -		Sandy to Clayey Silt, dark and grayish brown mottling, moist, stiff
50	23	18.5	SPT	50 -- -	SM/ML	Silty Sand to Sandy Silt, dark brown, moist, dense to stiff, fine grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	83	16.7	113.9	-		
				53 --		Sandy Silt to Silty Sand, dark and grayish brown, moist, very dense to very stiff, fine grained
				-		
				54 --		
				-		
55	49	17.9	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	72	20.9	106.9	-		
				58 --	ML	Sandy Silt, dark grayish brown, moist, very stiff
				-		
				59 --		
				-		
60	25	18.5	SPT	60 --		
				-		
				61 --		
				-		
				62 --		
62.5	90	17.8	113.1	-		
				63 --	SM	Silty Sand, dark brown, moist, very dense, fine grained
				-		
				64 --		
				-		
65	43	16.4	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	68	15.3	118.6	-		
				68 --		
				-		
				69 --		
				-		
70	47	16.0	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72.5	39	6.8	121.4	-		
	50/5"			73 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				74 --		
				-		
75	38	2.1	SPT	75 --		
	50/3"			-	SW	Gravelly Sand, dark to medium brown, slightly moist, very dense, fine to coarse grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				76 --		
				-		
				77 --		
77.5	24	3.5	114.4	-		
	50/3"			78 --	SP	Sand, dark brown, slightly moist, very dense, fine to medium grained
				-		
				79 --		
				-		
80	81	3.2	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82.5	100/9"	3.4	112.2	-		
				83 --		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				-		
				84 --		
				-		
85	78	2.8	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87.5	80	21.0	106.8	-		
				88 --	ML	Sandy Silt, dark to yellowish brown, moist, very stiff, fine grained
				-		
				89 --		
				-		
90	86	5.6	SPT	90 --		
				-	SP	Sand, yellow to grayish brown, slightly moist, very dense, fine grained
				91 --		
				-		
				92 --		
92.5	100/7"	2.8	112.5	-		
				93 --	SW	Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained
				-		
				94 --		
				-		
95	35	2.8	SPT	95 --		
	50/4"			-		
				96 --		
				-		
				97 --		
97.5	100/7"	7.0	116.0	-		
				98 --	SP	Sand, yellowish brown, slightly moist, very dense, fine grained
				-		
				99 --		
				-		
100	40	2.7	SPT	100 --		
	50/5"			-	SW	Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

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"	5.3	121.7			
				103		Sand to Gravelly Sand, yellowish brown, slightly moist, very dense, fine to coarse grained
				104		
105	85	3.2	SPT	105		
					SP	Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				106		
				107		
107.5	100/7"	9.5	106.8			
				108		
				109		
110	40 50/5"	2.7	SPT	110		
					SW	Gravelly Sand, dark to yellowish brown, slightly moist, very dense, fine to coarse grained
				111		
				112		
112.5	100/8"	12.3	113.3			
				113	ML	Sandy Silt, yellowish brown, moist, very stiff, fine grained
				114		
115	52 50/4"	15.5	SPT	115		
				116		
				117		
117.5	32 50/3"	14.2	113.9			
				118	SM	Silty Sand, dark grayish brown, moist, very dense, fine grained
				119		
120	65	13.6	SPT	120		
				121		
				122		
122.5	100/8"	4.4	115.8			
				123	SP	Sand, yellow to grayish brown, slightly moist, very dense, fine grained
				124		
125	30 50/5"	2.8	SPT	125		

BORING LOG NUMBER 2

Hazens Investment, LLC

File No. 20766

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				126		
				127		
127.5	100/7"	5.4	110.3			
				128		Sand, yellowish brown, slightly moist, very dense, fine to medium grained
				129		
130	43 50/3"	2.9	SPT	130		
				131		Total Depth 130 feet No Water Fill to 3 feet
				132		
				133		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				134		
				135		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				136		
				137		SPT=Standard Penetration Test
				138		
				139		
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				150		