Appendix E1

Geotechnical Engineering Investigation

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December 29, 2016
File Number 21324

LIG-900, 910 & 926 E. 4th St.
405-411 S. Hewitt, LLC
6315 Bandini Boulevard
Commerce, California 90040

Attention: Dilip Bhavnani

<u>Subject</u>: Geotechnical Engineering Investigation
Proposed Mixed Use Structure
405-411 South Hewitt Street, and 900-926 East 4th Street, and 412 Colyton Street
Los Angeles, California

Dear Mr. Bhavnani:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

TABLE OF CONTENTS

SECTION **PAGE**

GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED USE STRUCTURE
405-411 SOUTH HEWITT STREET, AND 900-926 EAST 4TH STREET, AND 412 COLYTON STREET
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This investigation included six exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was obtained by review of the architectural plans prepared by Gensler, dated December 16, 2016, as well as communication with the office of Walter P. Moore. The site is proposed to be developed with a mixed-use structure. The structure is proposed to be eleven stories in height, and will be built over three subterranean parking levels. It is anticipated that the finished floor elevation of the lowest subterranean parking levels will extend to an approximate depth of 29 feet below the existing grade. In addition, an underground utility vault will be built to the west of the proposed structure. It is anticipated that the finished floor elevation of this utility vault will extend approximately 11 feet below the existing grade. The enclosed Plot Plan and Cross Sections A-A' and B-B' illustrate the location, alignment and depth of the proposed structures.

Column loads are estimated to be up to a maximum 2,400 kips. This load reflects the dead plus live load. Grading is expected to consist of excavations as deep as 33 feet in depth for construction of the proposed subterranean parking garage, underground vault, 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.

SITE CONDITIONS

The subject is located at 405-411 South Hewitt Street, and 900-926 East 4th Street, and 412 Colyton Street, in the City of Los Angeles, California. The site is bounded by 4th Street to the North, Hewitt Street to the east, several single-story warehouse structures to the south, and Colyton Street to the west. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site is currently developed with four single-story structures, and three paved parking lots. As illustrated in the enclosed Plot Plan, one of the existing stories will remain, while the rest will be demolished prior to construction. The site is relatively level, with a maximum elevation relief on the order of 3 feet across the site. Vegetation at the site is limited, and consists of a couple of mature trees, and shrubbery. Drainage across the site appears to be by sheetflow to the city streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on November 8 and 9, 2016, by drilling six exploratory borings. The borings ranged from 50 to 80 feet in depth, and were prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-6.

The location of exploratory excavations was determined from hardscape features shown on the attached Plot Plan. Elevations of the exploratory excavations were determined from elevations presented in the Land Title Survey prepared by JRN Civil Engineers, dated July 31, 2015. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in all exploratory excavations to depths ranging between 2½ and 5 feet below the existing grade. The fill consists of a mixture of silty sands and sands, which are dark yellowish brown to dark brown in color, moist, medium dense, and fine grained.

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

Groundwater

Groundwater was encountered during drilling of Boring 3, at an approximate depth of 78 feet below the existing grade. The historically highest groundwater level was established by review of the Los Angeles 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 2006). Review of this plate indicates that the historically highest groundwater level at the site was on the order of 84 feet below grade. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map.

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

Caving

Caving could not be directly observed during exploration due to the continuously-cased design of the excavation equipment utilized. However, based on the experience of this firm, caving should be expected in large diameter excavations performed in the granular native soil layers.

OIL WELLS

Based on review of the California State Division of Oil, Gas and Geothermal Resources (DOGGR) Online Mapping System, the site is located within the limits of the Union Station Oil Field. The DOGGR On-line Mapping System also indicates that no oil or gas wells were drilled at the subject site; the closest oil and gas well was drilled approximately 1,000 feet to the southwest of the subject site.

METHANE ZONES

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

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

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

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

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

Liquefaction

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

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

Groundwater was encountered during exploration at an approximate depth of 78 feet below grade. The historically highest groundwater level for the site is reported to be on the order of 84 feet below grade. Based on the density of the soils underlying the site, and the current and historically highest groundwater levels, it is the opinion of this firm that the soils underlying the site are not considered capable of liquefaction during the ground motion expected during the design-based earthquake.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

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

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site lies within the potential mapped inundation boundaries of the Hansen and Sepulveda Reservoirs, should the dam retaining these reservoirs fail during a seismic event. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Fill materials were encountered during exploration to depths ranging between 2½ and 5 feet below the existing site grade. The existing fill materials are unsuitable for support of new foundations and concrete slabs-on-grade. It is anticipated that the existing fill will be removed during excavation for the proposed subterranean parking levels, which are expected to extend to a depth of 29 feet below the existing site grade. The proposed mixed-use structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean levels.

A new underground utility vault is also being proposed to be built to the west of the mixed-use structure. The bottom of this vault is expected to extend to a depth of 11 feet below the existing grade. This underground utility vault may also be supported by conventional foundations bearing in the native soils expected at its subgrade.

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

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

2016 California Building Code Seismic Parameters

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

2016 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.375g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.375g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.583g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.831g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.247g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.831g

Deaggregated Seismic Source Parameters

The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). The results are based on a 2 percent in 50 years ground motion (2,475 year return period). A shear wave velocity of 310 meters per second, selected from published values, was utilized for Vs30 (Tinsley and Fumal, 1985). The deaggregation program indicates a PGA of 0.88g and a modal magnitude of 6.6 for the site.

EXPANSIVE SOILS

The onsite geologic materials are in the very low expansion range. The Expansion Index was found to be 3 and 4 for representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

WATER-SOLUBLE SULFATES

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

The sources of natural sulfate minerals in soils include the sulfates of calcium, magnesium, sodium, and potassium. When these minerals interact and dissolve in subsurface water, a sulfate concentration is created, which will react with exposed concrete. Over time sulfate attack will destroy improperly proportioned concrete well before the end of its intended service life.

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on American Concrete Institute (ACI) Standard 318-08, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1% and Type I cement may be utilized for concrete foundations in contact with the site soils.

GRADING GUIDELINES

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

Site Preparation

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

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

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

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

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

Compaction

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

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. 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 30. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

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

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

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

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

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

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

Conventional

The proposed mixed-use structure and underground utility vault may be supported by conventional foundations bearing in the native alluvial soils expected at the subgrade of the proposed subterranean levels.

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, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended native alluvial soils.

Column foundations may be designed for a bearing capacity of 5,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended native alluvial soils.

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 1,000 pounds per square foot. The maximum recommended bearing capacity is 10,000 pounds per square foot.

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

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may bear in native alluvial soils, or in properly compacted fill materials. Continuous footings may be designed for a bearing capacity of 2,000 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 capacity increases are recommended.

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

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.45 may be used with the dead load forces.

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

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

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. Based on the enclosed settlement calculation, the maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement 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

Based on the proposed depth of the subterranean levels, it is anticipated that retaining walls on the order of 29 feet in height may be required for the project. As a precautionary measure, recommendations for the design of underground retaining walls up to a height of 32 feet have been provided herein. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional pressure should be added to the retaining wall design, for a surcharge condition due to vehicular traffic or adjacent structures. Based on review of the enclosed Plot Plan, it is anticipated that the proposed retaining walls will be surcharged by the existing single-story warehouse structures located to the south and west of the site. In addition, a portion of the subterranean garage retaining walls may be surcharged by the new underground utility vault. Information regarding the depth, configuration and loading of adjacent foundations will be required in order to determine the additional surcharge loading.

Vehicular traffic is expected in the vicinity of the proposed structure. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Cantilever Retaining Walls

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

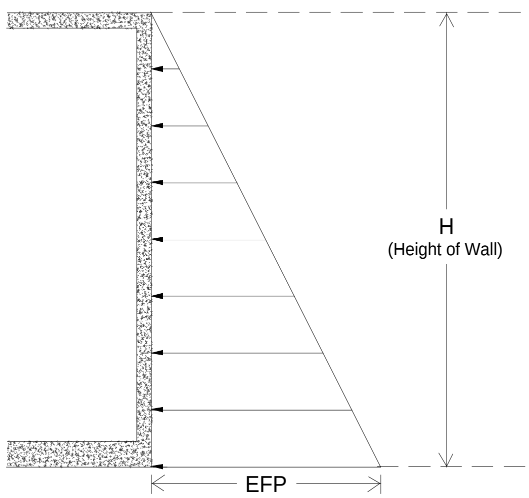
HEIGHT OF WALL (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 8	30
8 to 15	39
15 to 32	46

The lateral earth pressures recommended assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes would be 55 pounds per cubic foot, for walls up to 32 feet in height. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

TRIANGULAR DISTRIBUTION OF AT-REST EARTH PRESSURE



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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 24 pounds per cubic foot. When using the load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. The dynamic earth pressure may be omitted where the retaining wall is 6 feet in height or less.

Surcharge from Adjacent Structures

It is anticipated that the proposed retaining walls and temporary shoring walls will be surcharged by existing neighboring structures located to the south and west of the site. The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2014-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

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

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

where:

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

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

Retaining Wall Drainage

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

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 4 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected waters to a sump

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official. The City of Los Angeles only allows the use of flat drainage products when in conjunction with a conventional perforated subdrain pipe and gravel, or gravel pockets and weepholes.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was encountered during exploration at a depth of 78 feet below the existing grade. Based on the depth of the proposed development, the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will affect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

Waterproofing

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

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

Retaining Wall Backfill

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

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

TEMPORARY EXCAVATIONS

Excavations on the order of 13 to 33 feet in height are expected for construction of the proposed subterranean parking levels, underground utility vault, and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Vertical excavations exceeding 5 feet, or excavations which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum depth of 20 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

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

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.45 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Caving should be expected to occur during drilling in the native granular soils underlaying the site. Where caving occurs, it will be necessary to utilize casing or polymer drilling fluid to maintain open pile shafts. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Large sized materials should also be anticipated during drilling (i.e. gravels, cobbles, and possibly boulders).

Lagging

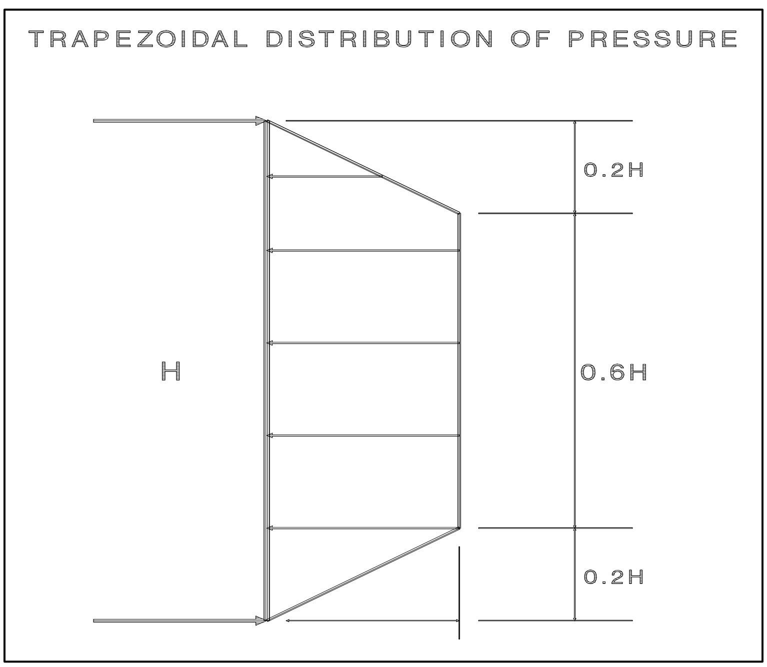
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but is limited to a maximum of 400 pounds per square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

Lateral Pressures

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 12	28
12 to 20	33
20 to 35	38

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



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

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 12	18H
12 to 20	21H
20 to 35	24H

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

Tied-Back Anchors

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

Drilled friction anchors may be designed for a skin friction of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,000 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within saturated 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.

Tieback Anchor Testing

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

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

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

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

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

Internal Bracing

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. An allowable bearing pressure of 5,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 24 inches in width and length as well as 18 inches in depth into native alluvial soils. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that shoring deflection be limited to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed, provided there are no structures within a 1:1 plane drawn upward from the base of the excavation. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

Monitoring

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

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

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over undisturbed native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 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 native alluvial soils or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 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 spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

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

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the 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 Car Traffic	3	4
Medium Truck Traffic	4	6

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

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

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

For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

At this time, stormwater infiltration at the subject site has not been proposed, and percolation testing has not been performed by this firm. It is recommended that this office is notified should stormwater infiltration be considered for the proposed project so that adequate testing and recommendations are provided.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of stormwater infiltration and/or filtration systems. Please be advised that stormwater infiltration and treatment is a relatively new requirement by the various jurisdictions and has been subject to change without notice.

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.

If corrosion sensitive improvements are planned, it is recommended that a comprehensive corrosion study should be commissioned. The study will develop recommendations to avoid premature corrosion of buried pipes and concrete structures in direct contact with the soils.

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 D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented on Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined 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. Results are presented on Plate D of this report.

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

VICINITY MAP

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Consulting Geotechnical Engineers

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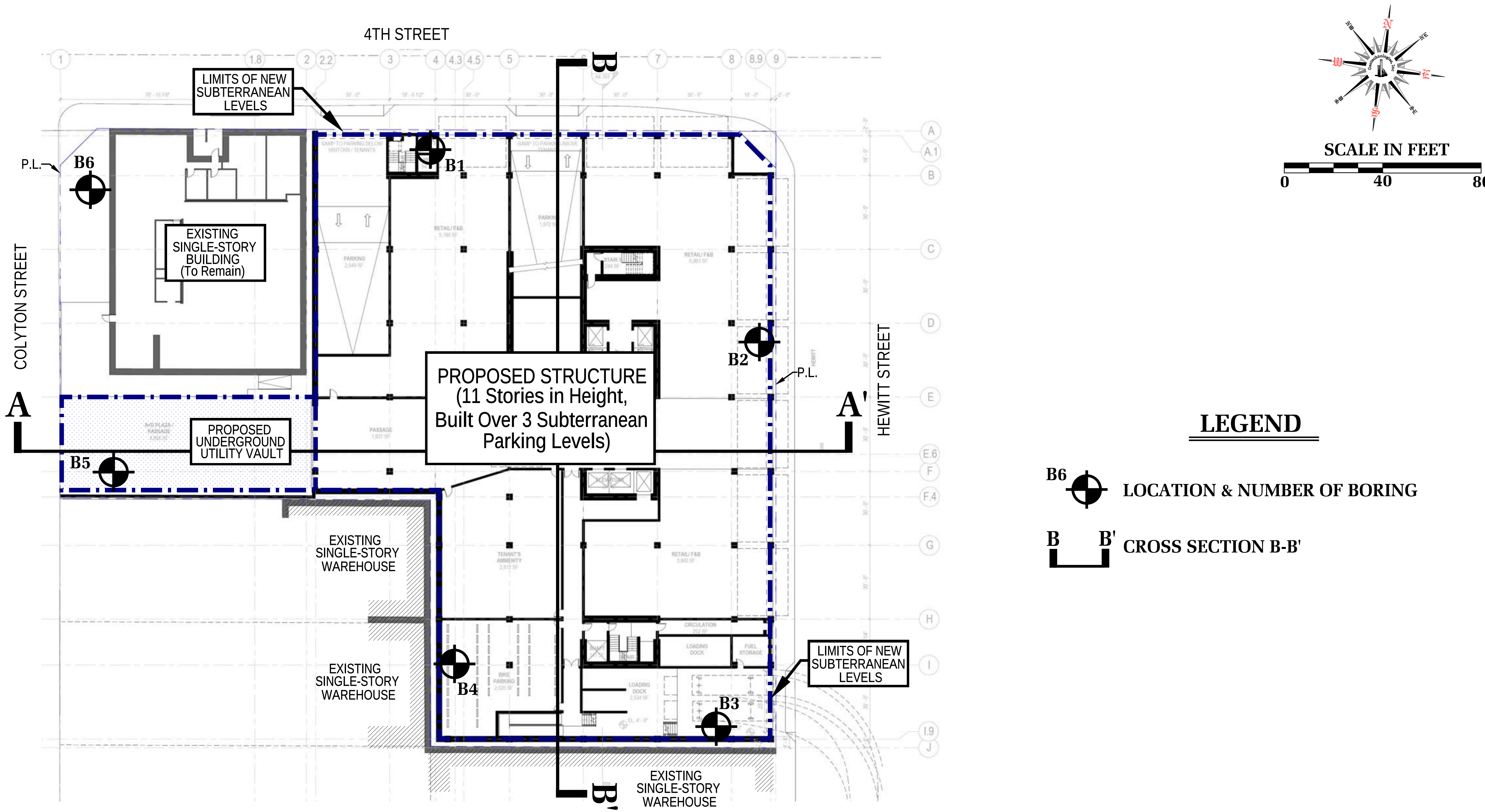

REFERENCE: OVERALL GROUND FLOOR PLAN BY GENSLER
PRINT DATE 12/16/2016

PLOT PLAN

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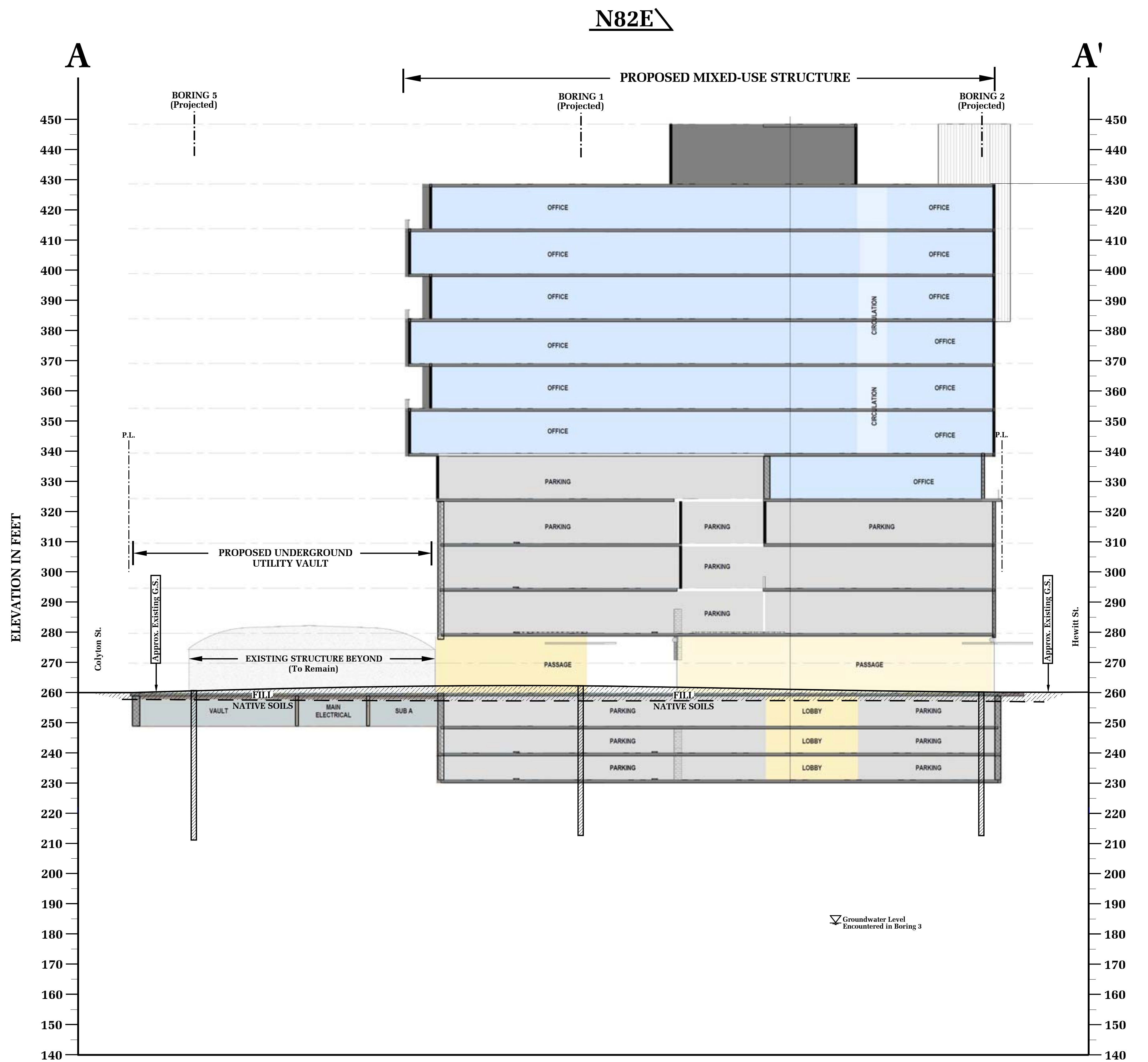
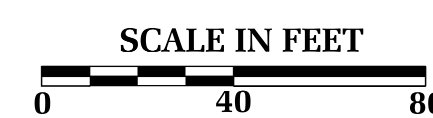
December '16





CROSS SECTION A-A'

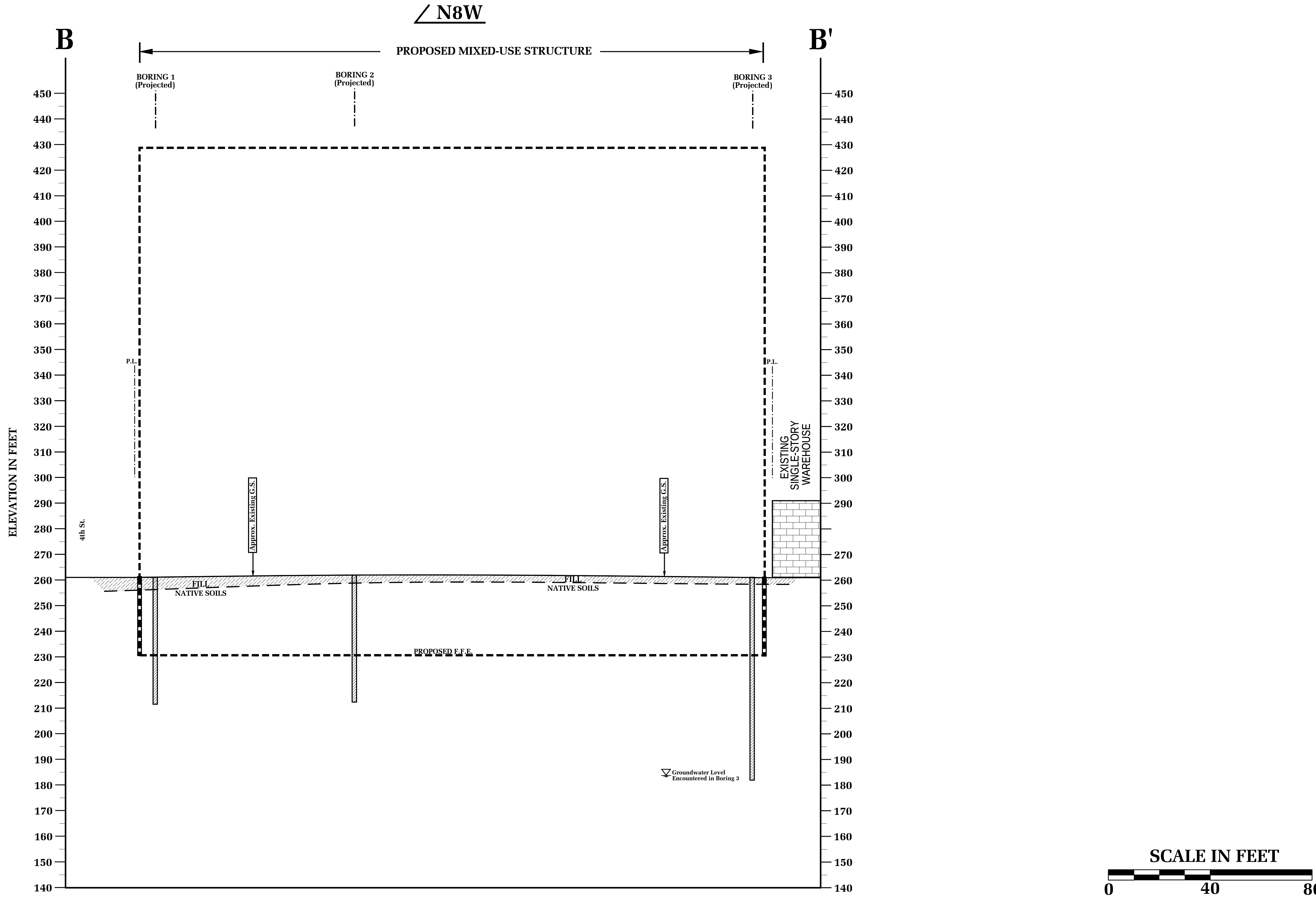
Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG-900, 910 & 925 E. 4TH ST., 405-411 S. HEWITT LLC

FILE No. 21324

December '16

REFERENCE: SECTION 2 BY GENSLER
PRINT DATE 12/16/2016



CROSS SECTION B-B'

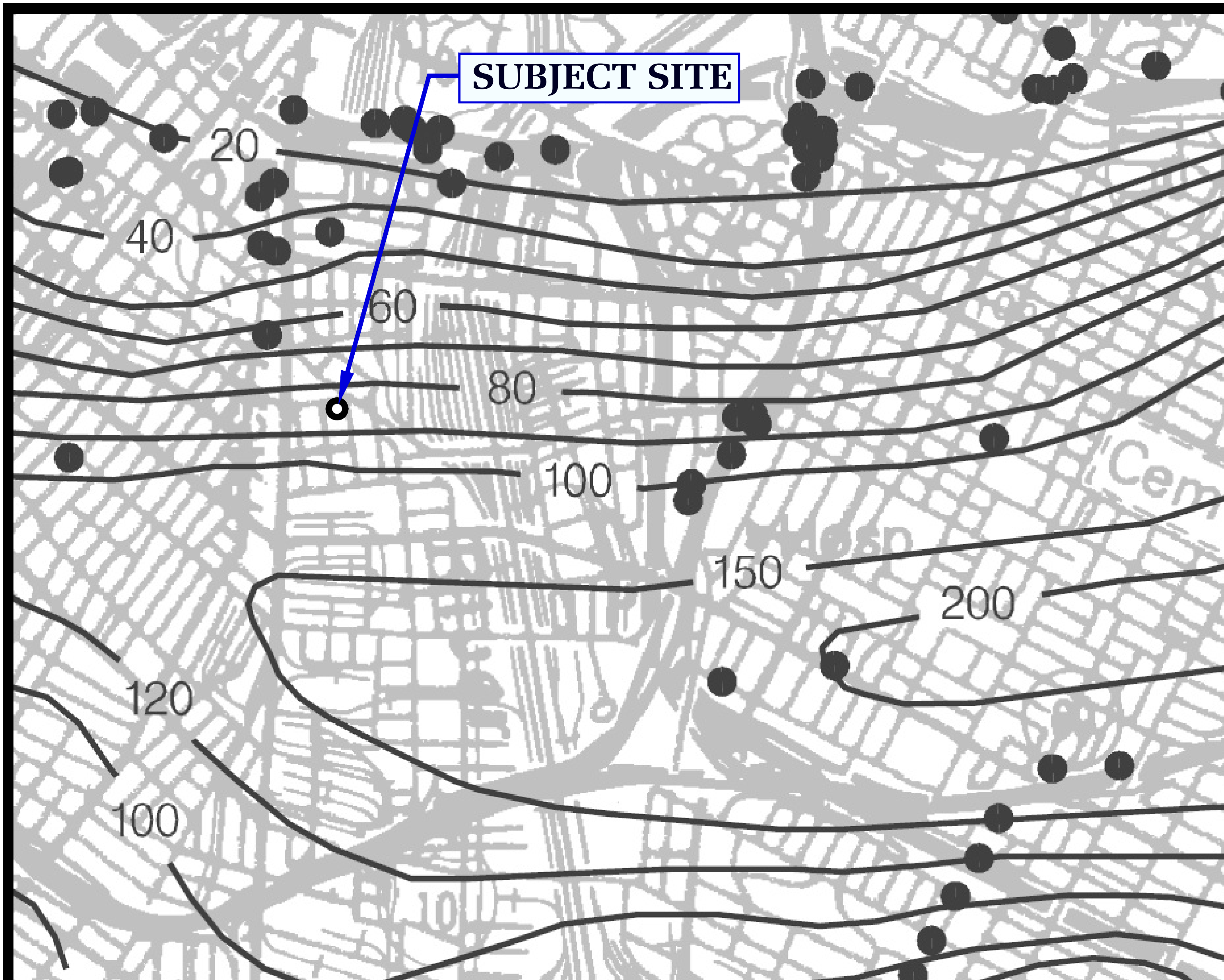
Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG-900, 910 & 925 E. 4TH ST., 405-411 S. HEWITT LLC

FILE No. 21324

December '16

REFERENCE: OVERALL GROUND FLOOR PLAN BY GENSLER PRINT DATE 12/16/2016



ONE MILE

SCALE

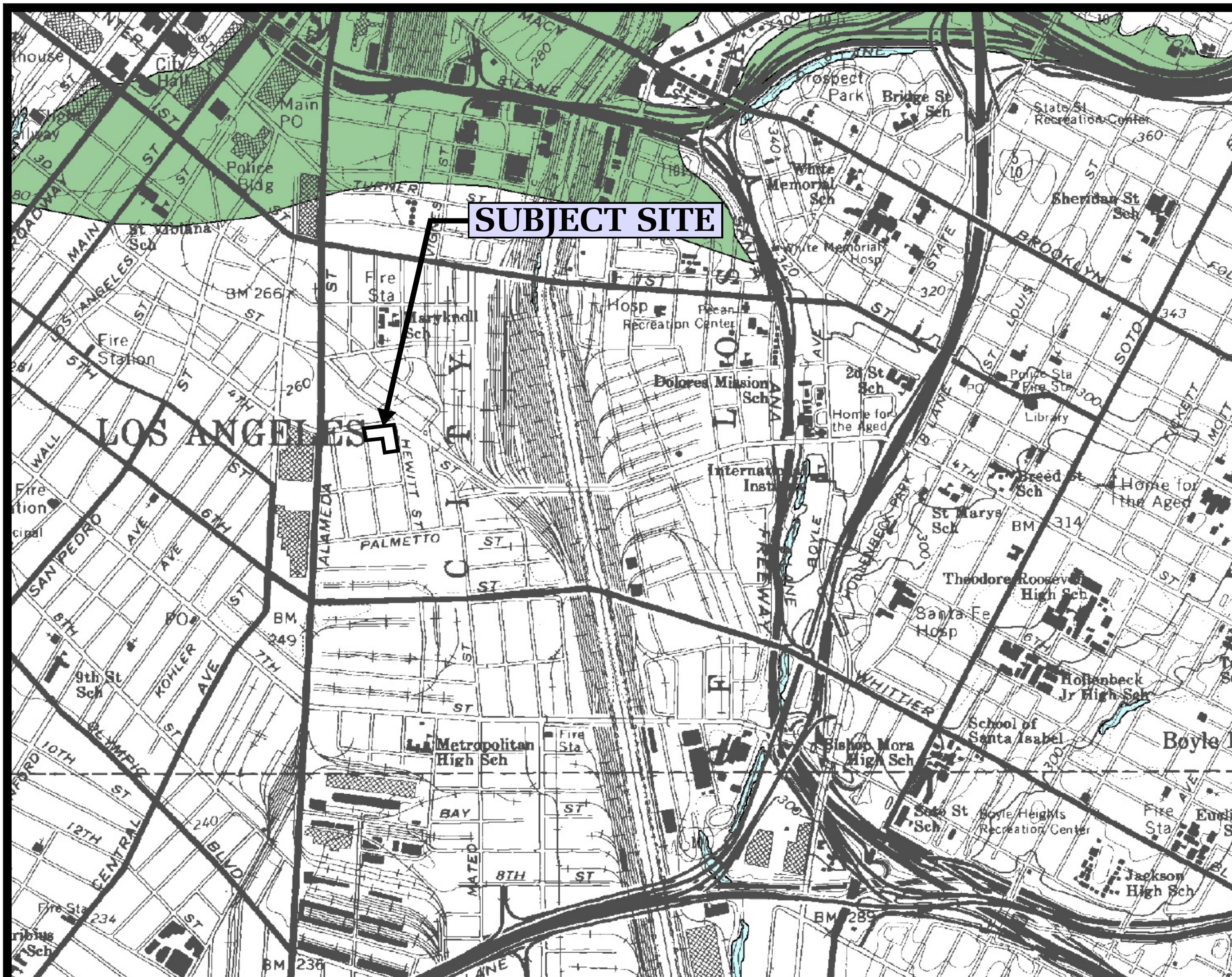
N

20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 029
LOS ANGELES 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

Geotechnologies, Inc. *Consulting Geotechnical Engineers*	**LIG - 900, 910 & 925 E. 4TH ST., 405-411 S. HEWITT LLC**
	FILE No. 21324



SCALE 1:24,000

1 .5 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

LIQUEFACTION AREA

REFERENCE: **SEISMIC HAZARD ZONES, LOS ANGELES QUADRANGLE OFFICIAL MAP (CDMG, 1999)**

SEISMIC HAZARD ZONE MAP

LIG - 900, 910 & 925 E. 4TH ST.,
405-411 S. HEWITT LLC

FILE NO. 21324

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BORING LOG NUMBER 1

LIG-900, 910 & 925 E. 4th Street **Date: 11/09/16** **Elevation: 261.3'***

File No. 21324 **Method: 8-inch diameter Hollow Stem Auger**

km *Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		6-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Sand to Sand, dark yellowish brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	11	6.1	105.9	-		
				3 --		
				-		
				4 --		
				-		
5	19	3.4	113.6	5 --		
				-	SP/SM	NATIVE SOILS: Sand to Silty Sand, dark yellowish brown, moist, medium dense, fine to medium grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	50	2.1	116.6	10 --		
				-	SW/SM	Sand to Silty Sand, dark gray, moist, medium dense to dense, fine to coarse grained, with cobbles
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	72	2.6	119.5	15 --		
				-	SP/SM	Sand to Silty Sand, yellow to light brown, moist, dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	68	1.7	107.7	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	83	3.3	109.3	25 --		
				-		yellowish brown, very dense, fine grained

BORING LOG NUMBER 1

LIG-900, 910 & 925 E. 4th Street

File No. 21324

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	40	1.5	126.7	30 --		
	50/5"			-	SP	Sand, gray to yellowish brown, slightly moist, very dense, fine to medium grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	100/7"	2.1	126.1	35 --		
				-	SW	Sand, yellow and grayish brown, moist, very dense, fine to coarse grained, with gravel
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	100/7"	1.4	121.6	40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	100/8"	2.0	123.2	45 --		
				-		yellowish gray, slightly moist, with cobbles
				46 --		
				-		
				47 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				48 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				49 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
50	100/5"	0.5	Disturbed	50 --		
				-		Total Depth 50 feet No Water Fill to 5 feet

BORING LOG NUMBER 2

LIG-900, 910 & 925 E. 4th Street **Date: 11/09/16** **Elevation: 261.5'***

File No. 21324 **Method: 8-inch diameter Hollow Stem Auger**

km
*Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete
				0 --		4-inch Concrete, No Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	19	3.9	105.1	-		
				3 --		
				-	SM/SP	NATIVE SOILS: Silty Sand to Sand, yellowish brown, slightly moist, medium dense, fine grained
				4 --		
				-		
5	18	10.6	110.2	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	36	1.8	124.8	10 --		
				-	SP/SM	Sand to Silty Sand, yellowish gray, slightly moist, medium dense, fine to medium grained, gravel
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	48	4.3	103.2	15 --		
				-		yellowish brown, moist, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	75	3.1	109.1	20 --		
				-	SP/SW	Sand, yellow and grayish brown, moist, dense, fine to coarse grained, with gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 2

LIG-900, 910 & 925 E. 4th Street

File No. 21324

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	78	2.6	118.6	30 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	100/7"	2.6	106.4	40 --		
				-		very dense
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		NOTE: The stratification lines represent the approximate
				45 --		boundary between earth types; the transition may be gradual.
				-		
				46 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				47 --		Modified California Sampler used unless otherwise noted
				-		
				48 --		
				-		Total Depth 48 feet by refusal
				49 --		No Water
				-		Fill to 3 feet
				50 --		
				-		

BORING LOG NUMBER 3

LIG-900, 910 & 925 E. 4th Street **Date: 11/08/16** **Elevation: 260.0'***

File No. 21324 **Method: 8-inch diameter Hollow Stem Auger**

km *Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete
				0 --		6½-inch Concrete, No Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	16	1.8	110.4	-		
				3 --	SP/SM	NATIVE SOILS: Sand to Silty Sand, yellowish brown, slightly moist, medium dense, fine grained
				-		
				4 --		
				-		
5	11	10.9	106.1	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
				10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	20	4.2	117.0	15 --		
	50/5"			-		very dense, few cobbles
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	26	3.9	117.0	25 --		
	50/5"			-		yellow and grayish brown, moist

BORING LOG NUMBER 3

LIG-900, 910 & 925 E. 4th Street

File No. 21324

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 --		
				31 --		
				32 --		
				33 --		
				34 --		
35	70	0.7	134.0	35 --		
				-	SP/SW	Sand, yellowish to grayish brown, moist, very dense, fine to coarse grained, with cobbles
				36 --		
				37 --		
				38 --		
				39 --		
				40 --		
				41 --		
				42 --		
				43 --		
				44 --		
45	100/11.5"	0.5	145.3	45 --		
				-	SW	Sand, gray, moist, very dense, fine to coarse grained, with cobbles
				46 --		
				47 --		
				48 --		
				49 --		
50	50/5"	0.2	SPT	50 --		
				-		

BORING LOG NUMBER 3

LIG-900, 910 & 925 E. 4th Street

File No. 21324

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	100/4"	No Recovery		-		
				53 --		
				-		
				54 --		
				-		
55	50/6"	1.0	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	100/7"	0.6	124.9	-		
				58 --		
				-		
				59 --		
				-		
60	47	12.9	SPT	60 --		
				-	SM/SP	Silty Sand to Sand, dark gray, moist, medium dense to dense, fine grained
				61 --		
				-		
				62 --		
62.5	33	2.5	104.6	-		
	50/4"			63 --	SP	Sand, dark yellowish brown, moist, very dense, fine grained
				-		
				64 --		
				-		
65	70	3.3	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67.5	35	2.8	104.0	-		
	50/4"			68 --		
				-		
				69 --		
				-		
70	37	15.6	SPT	70 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				71 --		
				-		
				72 --		
72.5	30	5.8	104.9	-		
	50/5"			73 --	SP	Sand, dark yellowish brown, moist, very dense, fine grained
				-		
				74 --		
				-		
75	36	21.2	SPT	75 --		
				-	SM	Silty Sand, dark yellowish brown, moist, medium dense, fine grained

BORING LOG NUMBER 3

LIG-900, 910 & 925 E. 4th Street

File No. 21324

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				76		
				77		
77.5	100/7"	2.7	114.4			
				78	SP	Sand, dark gray, moist, very dense, fine to medium grained, minor gravel
				79		
						wet
80	84	10.9	SPT	80		
				81		Total Depth 80 feet Water at 78 feet 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		SPT=Standard Penetration Test
				88		
				89		
				90		
				91		
				92		
				93		
				94		
				95		
				96		
				97		
				98		
				99		
				100		

BORING LOG NUMBER 4

LIG-900, 910 & 925 E. 4th Street **Date: 11/08/16** **Elevation: 260.8'***

File No. 21324 **Method: 8-inch diameter Hollow Stem Auger**

km *Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 5-inch Base
				-		
				1 --		
				-		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				2 --		
2.5	22	2.3	105.0	-		
				3 --		
				-	SP	NATIVE SOILS: Sand to Silty Sand, yellowish brown, moist,
				4 --		medium dense, fine grained, few gravel
				-		
5	16	2.2	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	19	6.0	101.7	-		
				8 --		yellow and grayish brown, slightly moist, fine to medium grained
				-		
				9 --		
				-		
10	12	4.3	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	50	1.8	113.8	-		
				13 --		few cobbles
				-		
				14 --		
				-		
15	24	2.0	SPT	15 --		
				-	SW/SM	Sand to Silty Sand, yellow and grayish brown, slightly moist,
				16 --		medium dense, fine to coarse grained, with gravel
				-		
				17 --		
17.5	42	3.9	115.3	-		
				18 --	SP/SM	Sand to Silty Sand, yellowish brown, slightly moist, medium dense,
				-		fine to medium grained
				19 --		
				-		
20	31	2.9	SPT	20 --		
				-		yellowish gray, few gravel
				21 --		
				-		
				22 --		
22.5	40	3.8	116.2	-		
	50/5"			23 --		dense
				-		
				24 --		
				-		
25	27	4.9	SPT	25 --		
				-	SP	Sand, yellowish gray, moist, medium dense, fine to medium grained

BORING LOG NUMBER 4

LIG-900, 910 & 925 E. 4th Street

File No. 21324

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	48	5.0	111.5			
				28		
				29		
30	80	2.9	SPT	30		
					SP/SW	Sand, gray, moist, very dense, fine to coarse grained, with gravel
				31		
				32		
32.5	60	5.4	98.8			
				33	SP	Sand, yellowish brown, slightly moist, dense, fine to medium grained
				34		
35	20 50/5.5"	1.8	SPT	35		
					SP/SW	Sand, yellow to grayish brown, moist, very dense, fine to coarse grained, with gravel
				36		
				37		
37.5	47 50/4"	4.8	105.1			
				38	SP	Sand, yellowish brown moist, very dense, fine grained
				39		
40	50/5"	1.7	SPT	40		
				41		
				42		
42.5	40 50/4"	2.0	122.9			
				43		fine to medium grained, few cobbles
				44		
45	53 50/4"	2.1	SPT	45		
					SP/SW	Sand, yellow to grayish brown, moist, very dense, fine to coarse grained, with gravel
				46		
47	100/5"	No Recovery		47		
				48		
				49		
50	50/2"	No Recovery	SPT	50		Total Depth 50 feet No Water Fill to 3 feet

NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.

Used 8-inch diameter Hollow-Stem Auger
140-lb. Automatic Hammer, 30-inch drop
Modified California Sampler used unless otherwise noted
SPT=Standard Penetration Test

BORING LOG NUMBER 5

LIG-900, 910 & 925 E. 4th Street — **Date: 11/09/16** — **Elevation: 261.0'***

File No. 21324 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		2-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	20	4.7	106.5	-		
				3 --		
				-	SM/SP	NATIVE SOILS: Silty Sand to Sand, dark yellowish brown, moist, medium dense, fine grained
				4 --		
				-		
5	29	4.4	112.5	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	34	3.5	Disturbed	10 --		
				-	SP/SM	Sand to Silty Sand, yellowish brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
				15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	72	2.6	101.5	20 --		
				-	SP	Sand, yellowish brown, moist, dense, fine to medium grained, few cobbles
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 5

LIG-900, 910 & 925 E. 4th Street

File No. 21324

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	88	4.9	118.0	30		
						very dense
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
40	100/7"	2.5	109.7	40		
				41		
				42		
				43		
				44		
				45		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				46		Used 8-inch diameter Hollow-Stem Auger
						140-lb. Automatic Hammer, 30-inch drop
				47		Modified California Sampler used unless otherwise noted
				48		
				49		
						few cobbles
50	100/9"	1.7	124.1	50		
						Total Depth 50 feet No Water Fill to 3 feet

BORING LOG NUMBER 6

LIG-900, 910 & 925 E. 4th Street — **Date: 11/09/16** — **Elevation: 260.0'***

File No. 21324 — **Method: 8-inch diameter Hollow Stem Auger**

km — *Reference: Land Title Survey by JRN Civil Engineers, dated 7/31/15

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		2-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Sand to Sand, dark yellowish brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	34	2.9	107.7	-		
				3 --		
				-	SP	NATIVE SOILS: Sand, yellowish brown, moist, medium dense, fine grained
				4 --		
				-		
5	27	15.8	106.1	5 --		
				-	SM/SP	Silty Sand to Sand, dark yellowish brown, slightly moist, medium dense, fine grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
				10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	40	1.8	114.2	15 --		
	50/3"			-	SP/SM	Sand to Silty Sand, olive brown, moist, very dense, fine grained, few cobbles
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	38	4.4	110.4	25 --		
	50/5"			-		yellowish brown

BORING LOG NUMBER 6

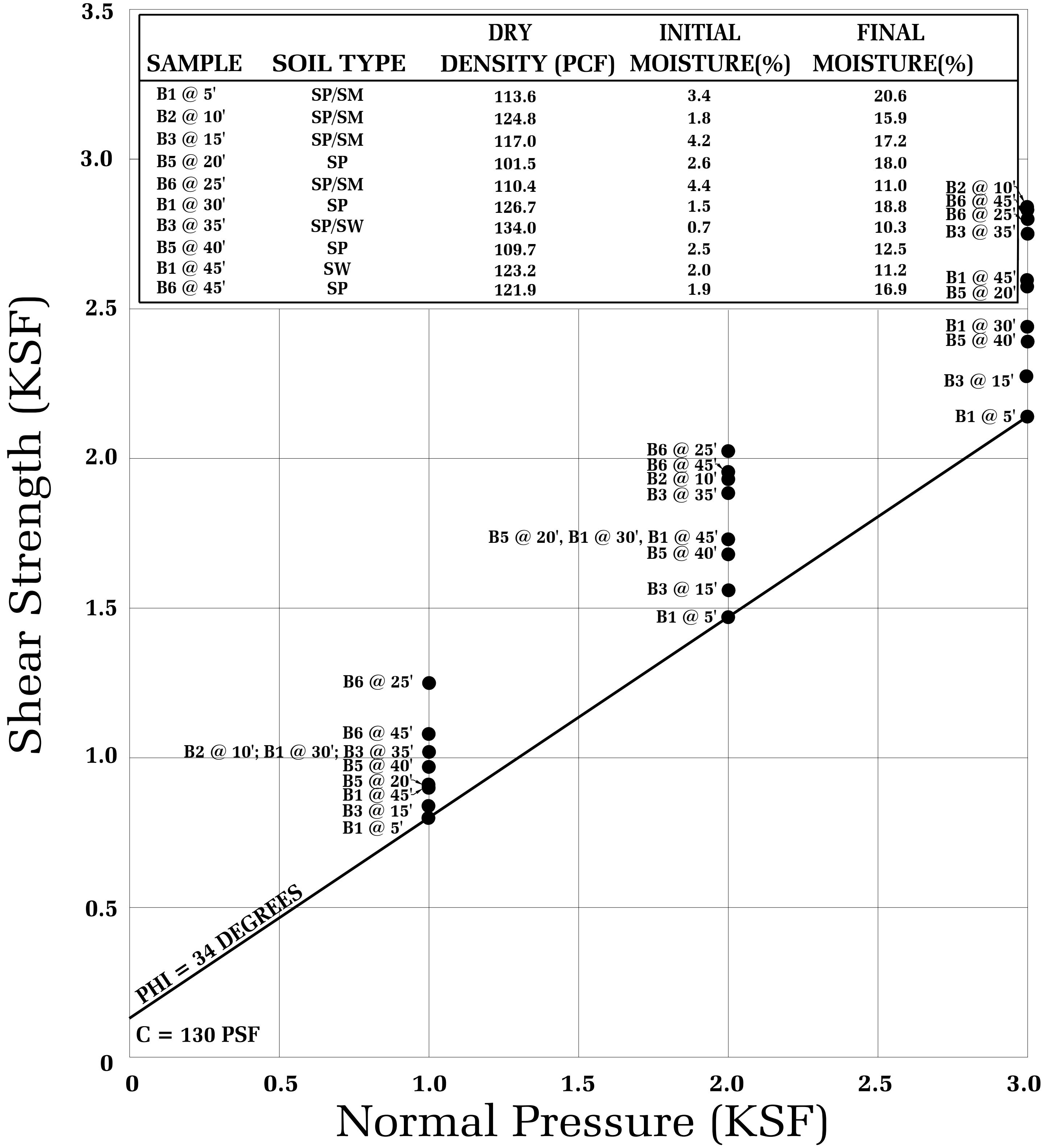
LIG-900, 910 & 925 E. 4th Street

File No. 21324

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 --		
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	40	2.0	123.7	35 --		
	50/4"			-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	100/8"	1.9	121.9	45 --		
				-	SP	Sand, dark brown, moist, very dense, fine to coarse grained, with cobbles
				46 --		
				-		
				47 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				48 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				49 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
50	100/8"	1.5	129.3	50 --		
				-		Total Depth 50 feet No Water Fill to 3 feet

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 5'	SP/SM	113.6	3.4	20.6
B2 @ 10'	SP/SM	124.8	1.8	15.9
B3 @ 15'	SP/SM	117.0	4.2	17.2
B5 @ 20'	SP	101.5	2.6	18.0
B6 @ 25'	SP/SM	110.4	4.4	11.0
B1 @ 30'	SP	126.7	1.5	18.8
B3 @ 35'	SP/SW	134.0	0.7	10.3
B5 @ 40'	SP	109.7	2.5	12.5
B1 @ 45'	SW	123.2	2.0	11.2
B6 @ 45'	SP	121.9	1.9	16.9



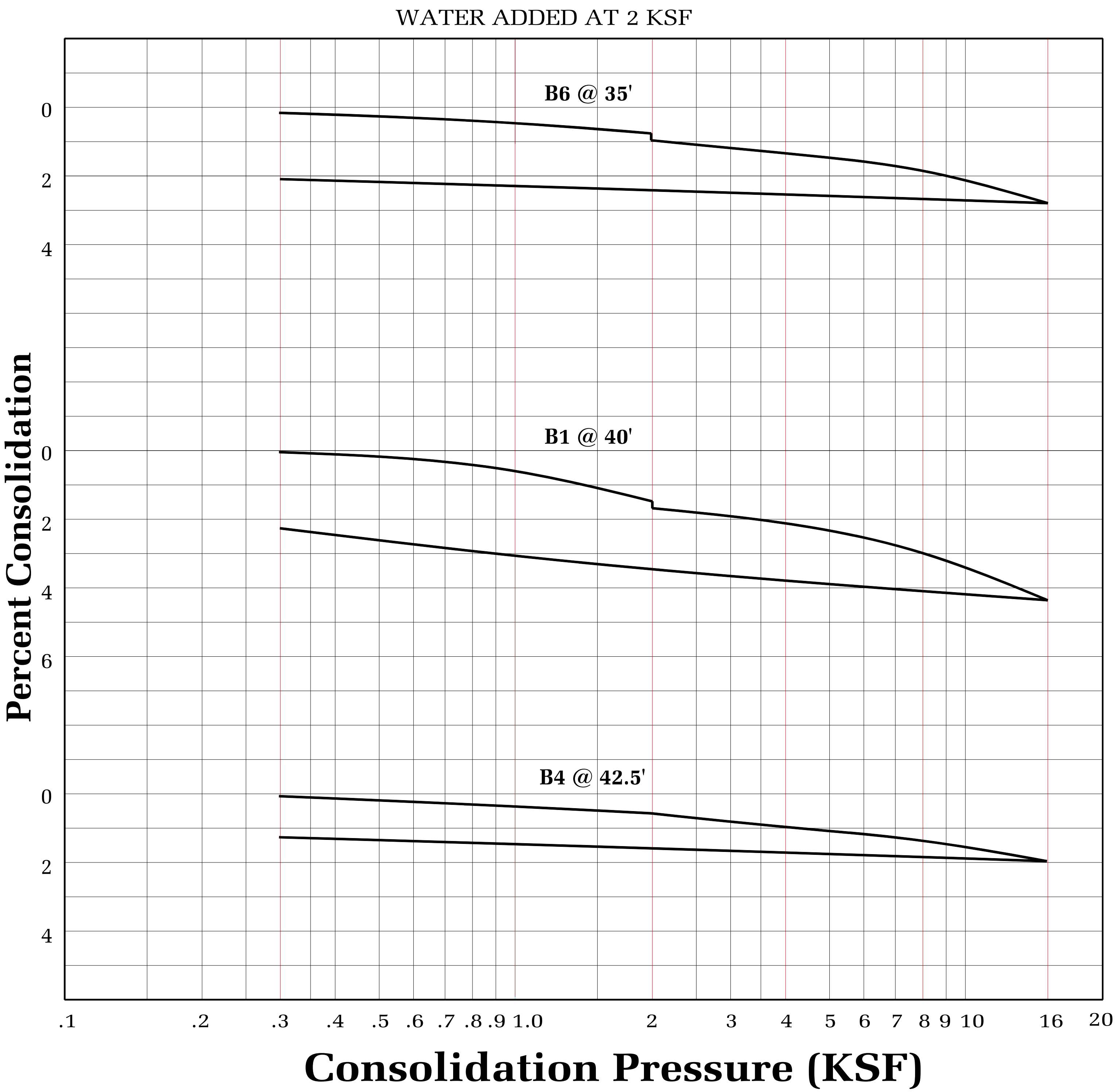
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG - 900, 910 & 925 E. 4TH ST.,
405-411 S. HEWITT LLC

FILE NO. 21324 | PLATE: B



CONSOLIDATION TEST

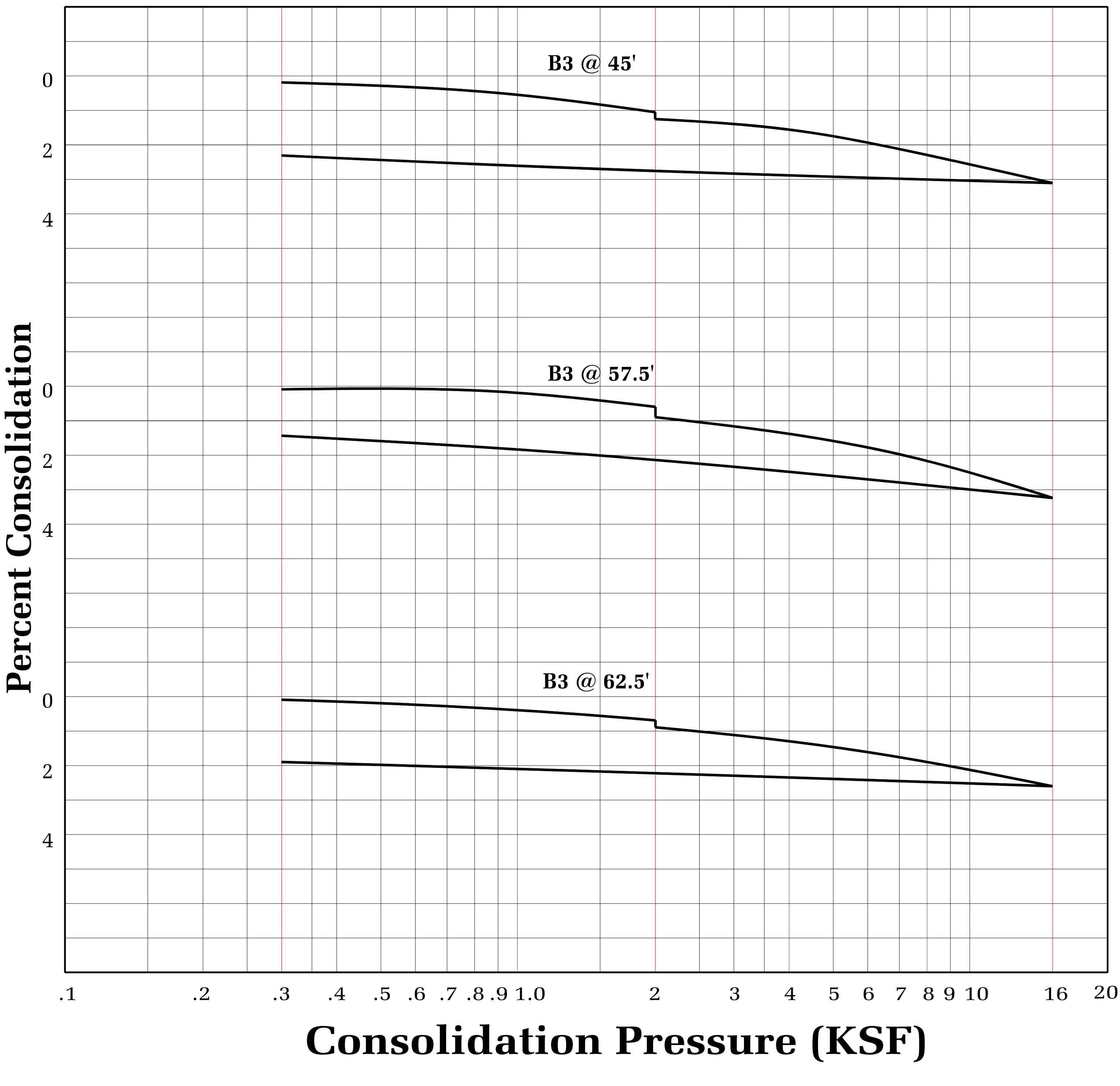
Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG - 900, 910 & 925 E. 4TH ST.,
405-411 S. HEWITT LLC

FILE NO. 21324

PLATE: C-1

WATER ADDED AT 2 KSF



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG - 900, 910 & 925 E. 4TH ST.,
405-411 S. HEWITT LLC

FILE NO. 21324

PLATE: C-2

ASTM D-1557

SAMPLE	B2 @ 1-5'	B5 @ 1-5'
SOIL TYPE:	SM	SM
MAXIMUM DENSITY pcf.	131.1	125.9
OPTIMUM MOISTURE %	9.3	9.5

ASTM D 4829

SAMPLE	B2 @ 1-5'	B5 @ 1-5'
SOIL TYPE:	SM	SM
EXPANSION INDEX UBC STANDARD 18-2	3	4
EXPANSION CHARACTER	VERY LOW	VERY LOW

SULFATE CONTENT

SAMPLE	B2 @ 1-5'	B5 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10%	< 0.10%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

LIG - 900, 910 & 925 E. 4TH ST.,
405-411 S. HEWITT LLC

FILE NO. 21324 | PLATE: D

Geotechnologies, Inc.

Project: LIG 900, 910 & 925 E. 4th Street

File No.: 21324

Settlement Calculation - Column Footing

Soil Unit Weight	120.0 pcf	Column Footing
Bearing Value	10000.0 psf	2560 kips
Depth of Footing	32.0 feet	
Width of Footing	16.00 feet	

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

Depth Below Basement Surbgrade (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)
32.0													
	36.0	4.0	4.0	70%	6971	4320	11291	B6 @ 35'	2.25	1.45	0.80	8.0	0.77
40.0													
	42.5	10.5	1.5	35%	3542.5	5100	8642.5	B1 @ 40'	3.10	2.35	0.75	5.0	0.45
45.0													
	50.0	18.0	0.9	16%	1560	6000	7560	B3 @ 45'	2.20	1.95	0.25	10.0	0.30
55.0													
	58.8	26.8	0.6	7%	738	7050	7788	B3 @ 57.5'	2.10	2.00	0.10	7.5	0.09
62.5													

Settlement: 1.61

Reduction: 0.67

Total Settlement in inches: ***1.07***

Geotechnologies, Inc.

Project: LIG 900, 910 & 925 E. 4th Street

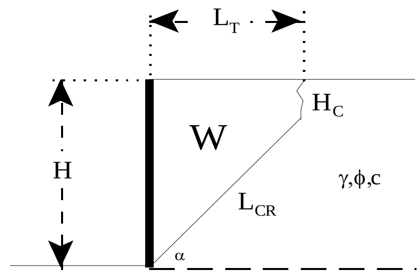
File No.: 21324

Description: Cantilever Retaining Walls (up to 8 feet high)

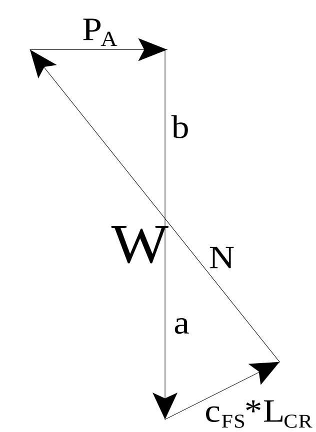
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	8.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	24.2	degrees
	(c_{FS})	86.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.0	33	4081.4	7.7	2244.5	1836.9	519.4
41	2.9	32	3996.4	7.8	2127.0	1869.4	564.0
42	2.8	31	3903.9	7.8	2016.4	1887.5	605.6
43	2.7	30	3806.4	7.8	1912.8	1893.6	644.2
44	2.6	30	3705.7	7.8	1816.1	1889.6	679.8
45	2.5	29	3603.2	7.8	1726.1	1877.1	712.6
46	2.5	28	3499.7	7.7	1642.3	1857.4	742.5
47	2.4	27	3396.1	7.7	1564.3	1831.7	769.5
48	2.3	26	3292.7	7.6	1491.8	1800.9	793.9
49	2.3	26	3190.0	7.6	1424.1	1765.9	815.5
50	2.3	25	3088.2	7.5	1361.1	1727.1	834.5
51	2.2	24	2987.6	7.4	1302.3	1685.3	850.9
52	2.2	23	2888.1	7.4	1247.2	1640.9	864.7
53	2.2	22	2790.0	7.3	1195.7	1594.3	876.0
54	2.2	22	2693.2	7.2	1147.5	1545.8	884.8
55	2.2	21	2597.8	7.1	1102.1	1495.7	891.2
56	2.1	20	2503.8	7.1	1059.4	1444.4	895.1
57	2.1	19	2411.1	7.0	1019.2	1391.9	896.6
58	2.1	19	2319.7	6.9	981.2	1338.5	895.6
59	2.2	18	2229.5	6.8	945.2	1284.3	892.2
60	2.2	17	2140.6	6.7	911.1	1229.6	886.4
61	2.2	16	2052.9	6.7	878.6	1174.3	878.1
62	2.2	16	1966.3	6.6	847.6	1118.6	867.3
63	2.2	15	1880.7	6.5	818.0	1062.6	854.0
64	2.3	14	1796.1	6.4	789.6	1006.4	838.2
65	2.3	14	1712.3	6.3	762.3	950.1	819.7



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 896.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 28.0 pcf

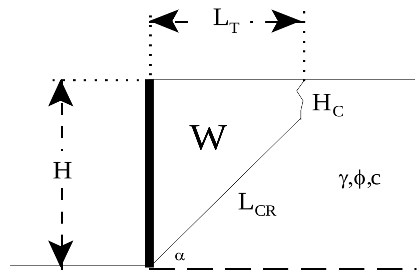
Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.
Project: LIG 900, 910 & 925 E. 4th Street
File No.: 21324
Description: Cantilever Retaining Walls (8 to 15 feet high)

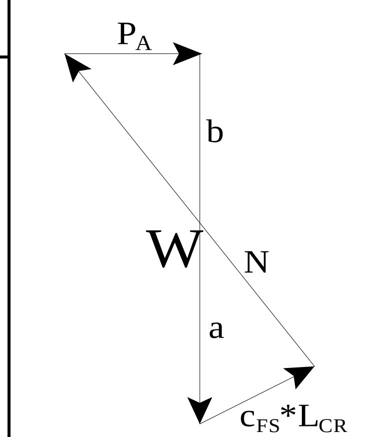
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	24.2	degrees
	(c_{FS})	86.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.0	129	16073.4	18.6	5408.2	10665.2	3015.5
41	2.9	125	15572.0	18.4	5047.0	10525.0	3175.3
42	2.8	121	15079.5	18.3	4723.1	10356.3	3322.6
43	2.7	117	14597.1	18.1	4431.8	10165.3	3458.2
44	2.6	113	14125.8	17.9	4168.9	9956.9	3582.3
45	2.5	109	13665.7	17.6	3930.8	9734.9	3695.6
46	2.5	106	13217.0	17.4	3714.6	9502.4	3798.3
47	2.4	102	12779.5	17.2	3517.6	9261.9	3891.0
48	2.3	99	12353.0	17.0	3337.6	9015.4	3974.0
49	2.3	95	11937.2	16.8	3172.8	8764.4	4047.5
50	2.3	92	11531.7	16.6	3021.4	8510.3	4111.8
51	2.2	89	11136.0	16.4	2882.0	8254.0	4167.2
52	2.2	86	10749.8	16.2	2753.4	7996.5	4213.9
53	2.2	83	10372.6	16.1	2634.4	7738.2	4252.0
54	2.2	80	10004.1	15.9	2524.1	7479.9	4281.7
55	2.2	77	9643.7	15.7	2421.7	7222.0	4303.1
56	2.1	74	9291.0	15.5	2326.4	6964.6	4316.2
57	2.1	72	8945.7	15.3	2237.5	6708.3	4321.2
58	2.1	69	8607.4	15.2	2154.4	6453.0	4318.0
59	2.2	66	8275.7	15.0	2076.6	6199.1	4306.6
60	2.2	64	7950.2	14.8	2003.6	5946.6	4286.9
61	2.2	61	7630.6	14.7	1935.0	5695.6	4259.0
62	2.2	59	7316.6	14.5	1870.3	5446.2	4222.7
63	2.2	56	7007.8	14.3	1809.3	5198.4	4177.8
64	2.3	54	6703.9	14.2	1751.6	4952.3	4124.3
65	2.3	51	6404.6	14.0	1696.8	4707.8	4061.9



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 4321.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 38.4 pcf

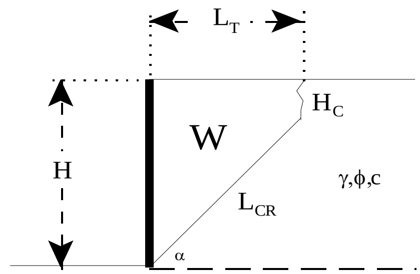
Design Wall for an Equivalent Fluid Pressure: 39 pcf

Geotechnologies, Inc.
Project: LIG 900, 910 & 925 E. 4th Street
File No.: 21324
Description: Cantilever Retaining Walls (15 to 32 feet high)

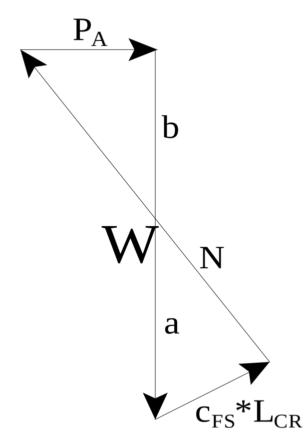
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	32.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	24.2	degrees
	(c_{FS})	86.7	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.0	605	75586.6	45.1	13091.6	62495.0	17670.0
41	2.9	584	73018.5	44.4	12138.3	60880.2	18366.8
42	2.8	564	70540.7	43.7	11296.7	59244.0	19007.3
43	2.7	545	68148.6	43.0	10549.5	57599.1	19594.7
44	2.6	527	65837.6	42.3	9882.8	55954.8	20131.6
45	2.5	509	63603.2	41.7	9285.2	54318.0	20620.3
46	2.5	492	61441.1	41.1	8747.3	52693.7	21063.0
47	2.4	475	59347.0	40.5	8261.3	51085.7	21461.7
48	2.3	459	57316.9	39.9	7820.5	49496.5	21818.0
49	2.3	443	55347.2	39.4	7419.4	47927.8	22133.4
50	2.3	427	53434.2	38.8	7053.4	46380.8	22409.2
51	2.2	413	51574.6	38.3	6718.5	44856.1	22646.5
52	2.2	398	49765.3	37.8	6411.1	43354.2	22846.4
53	2.2	384	48003.2	37.3	6128.3	41875.0	23009.4
54	2.2	370	46285.8	36.9	5867.5	40418.3	23136.4
55	2.2	357	44610.3	36.4	5626.5	38983.8	23227.8
56	2.1	344	42974.3	36.0	5403.3	37571.0	23284.0
57	2.1	331	41375.5	35.6	5196.1	36179.4	23305.2
58	2.1	318	39811.8	35.2	5003.6	34808.3	23291.4
59	2.2	306	38281.2	34.8	4824.2	33457.0	23242.7
60	2.2	294	36781.6	34.5	4656.9	32124.8	23158.8
61	2.2	282	35311.4	34.1	4500.5	30811.0	23039.4
62	2.2	271	33868.8	33.8	4354.1	29514.8	22884.0
63	2.2	260	32452.2	33.4	4216.7	28235.4	22692.0
64	2.3	248	31060.0	33.1	4087.8	26972.2	22462.7
65	2.3	238	29690.8	32.8	3966.4	25724.4	22195.2



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 23305.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 45.5 pcf

Design Wall for an Equivalent Fluid Pressure: 46 pcf

Geotechnologies, Inc.

Project: LIG 900, 910 & 925 E. 4th Street

File No.: 21324

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	34	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	32	feet

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.441	
$\sigma'_v = \gamma H$	4000.0	psf

$\sigma'_h =$	1763.2	psf	
EFP =	55	pcf	
$P_o =$	28211.7	lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 55 pcf

Geotechnologies, Inc.
Project: LIG 900, 910 & 925 E. 4th Street
File No.: 21324

Seismically Induced Lateral Soil Pressure on Retaining Wall

Input:

Height of Retaining Wall:	(H)	32.0	feet
Retained Soil Unit Weight:	(γ)	125.0	pcf
Horizontal Ground Acceleration:	(k_h)	0.29	g

Seismic Increment (ΔP_{AE}):

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$
ΔP_{AE} = 13920.0 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$
T = 12528.0 lbs/ft

$EFP = 2*T/H^2$
EFP = 24 pcf
triangular distribution of pressure

Geotechnologies, Inc.

Project: LIG 900, 910 & 925 E. 4th Street

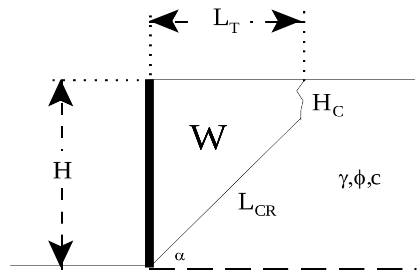
File No.: 21324

Description: Temporary Shopring Walls (up to 12 feet high)

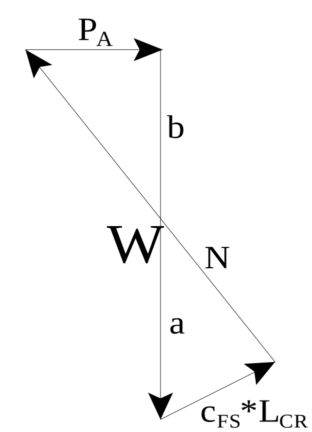
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	12.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	28.4	degrees
	(c_{FS})	104.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.7	72	9056.5	11.3	5124.1	3932.4	810.7
41	4.4	72	8941.9	11.5	4822.5	4119.4	924.4
42	4.2	70	8785.3	11.7	4535.7	4249.6	1031.9
43	4.0	69	8600.9	11.8	4267.2	4333.6	1132.7
44	3.8	67	8398.1	11.8	4018.3	4379.8	1226.9
45	3.6	65	8183.6	11.9	3788.6	4395.0	1314.2
46	3.5	64	7961.7	11.8	3577.0	4384.6	1395.0
47	3.4	62	7735.6	11.8	3382.4	4353.2	1469.1
48	3.3	60	7507.6	11.8	3203.3	4304.3	1536.8
49	3.2	58	7279.4	11.7	3038.4	4240.9	1598.2
50	3.1	56	7051.9	11.6	2886.4	4165.5	1653.3
51	3.0	55	6826.0	11.6	2746.0	4080.0	1702.4
52	3.0	53	6602.3	11.5	2616.2	3986.2	1745.5
53	2.9	51	6381.2	11.4	2495.9	3885.3	1782.8
54	2.9	49	6162.8	11.3	2384.2	3778.6	1814.3
55	2.8	48	5947.4	11.2	2280.4	3667.0	1840.2
56	2.8	46	5734.9	11.1	2183.6	3551.3	1860.4
57	2.8	44	5525.5	11.0	2093.2	3432.3	1875.1
58	2.8	43	5319.1	10.9	2008.7	3310.5	1884.3
59	2.8	41	5115.7	10.7	1929.4	3186.3	1888.0
60	2.8	39	4915.1	10.6	1854.9	3060.2	1886.2
61	2.8	38	4717.3	10.5	1784.7	2932.6	1879.0
62	2.8	36	4522.1	10.4	1718.4	2803.7	1866.2
63	2.8	35	4329.5	10.3	1655.6	2673.9	1847.9
64	2.9	33	4139.2	10.2	1595.9	2543.3	1824.1
65	2.9	32	3951.2	10.0	1539.2	2412.1	1794.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1888.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 26.2 pcf

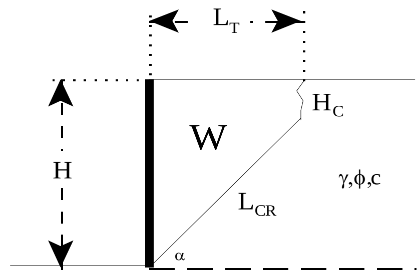
Design Shoring for an Equivalent Fluid Pressure: 28 pcf

Geotechnologies, Inc.
Project: LIG 900, 910 & 925 E. 4th Street
File No.: 21324
Description: Temporary Shopring Walls (12 to 20 feet high)

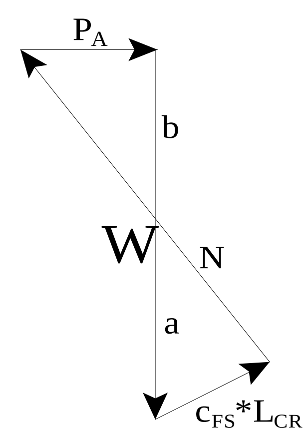
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	28.4	degrees
	(c_{FS})	104.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.7	225	28124.6	23.7	10765.9	17358.7	3578.5
41	4.4	219	27347.8	23.7	9919.4	17428.3	3911.2
42	4.2	212	26555.1	23.6	9173.1	17382.0	4220.7
43	4.0	206	25758.8	23.5	8512.7	17246.1	4507.8
44	3.8	200	24966.6	23.4	7926.0	17040.6	4773.3
45	3.6	193	24183.6	23.2	7402.9	16780.7	5018.0
46	3.5	187	23412.7	23.0	6934.4	16478.3	5242.5
47	3.4	181	22655.9	22.8	6513.4	16142.5	5447.7
48	3.3	175	21914.1	22.5	6133.5	15780.6	5634.2
49	3.2	170	21187.9	22.3	5789.7	15398.3	5802.7
50	3.1	164	20477.5	22.1	5477.3	15000.2	5953.6
51	3.0	158	19782.6	21.8	5192.7	14589.9	6087.6
52	3.0	153	19102.9	21.6	4932.6	14170.3	6205.1
53	2.9	148	18438.0	21.4	4694.2	13743.8	6306.5
54	2.9	142	17787.5	21.2	4475.1	13312.3	6392.0
55	2.8	137	17150.7	20.9	4273.3	12877.4	6462.1
56	2.8	132	16527.1	20.7	4086.8	12440.2	6517.0
57	2.8	127	15916.0	20.5	3914.2	12001.8	6556.8
58	2.8	123	15317.0	20.3	3754.0	11563.0	6581.6
59	2.8	118	14729.4	20.1	3605.1	11124.4	6591.6
60	2.8	113	14152.7	19.9	3466.2	10686.5	6586.8
61	2.8	109	13586.2	19.7	3336.5	10249.8	6567.2
62	2.8	104	13029.4	19.5	3215.0	9814.5	6532.7
63	2.8	100	12481.9	19.3	3101.0	9380.9	6483.1
64	2.9	96	11942.9	19.1	2993.7	8949.2	6418.4
65	2.9	91	11412.2	18.9	2892.6	8519.5	6338.3



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\,max}$ 6591.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 33.0 pcf

Design Shoring for an Equivalent Fluid Pressure: 33 pcf

Geotechnologies, Inc.

Project: LIG 900, 910 & 925 E. 4th Street

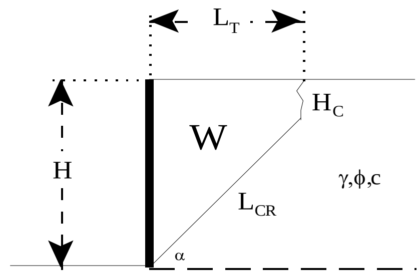
File No.: 21324

Description: Temporary Shopring Walls (20 to 35 feet high)

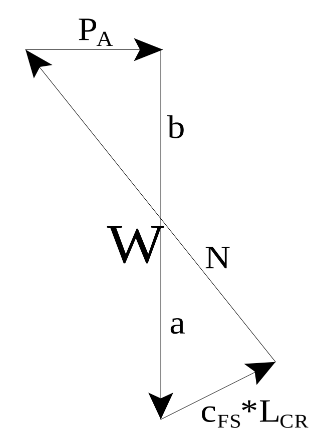
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	34.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	28.4	degrees
	(c_{FS})	104.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.7	717	89574.4	47.1	21344.2	68230.1	14065.7
41	4.4	693	86663.7	46.6	19476.2	67187.5	15077.8
42	4.2	671	83821.0	46.1	17868.2	65952.8	16014.7
43	4.0	648	81052.8	45.5	16472.8	64579.9	16880.1
44	3.8	627	78361.1	45.0	15253.0	63108.1	17677.6
45	3.6	606	75746.1	44.4	14179.7	61566.4	18410.4
46	3.5	586	73206.0	43.8	13229.6	59976.5	19081.4
47	3.4	566	70738.7	43.3	12384.0	58354.7	19693.4
48	3.3	547	68341.2	42.7	11627.7	56713.5	20248.8
49	3.2	528	66010.5	42.2	10948.2	55062.3	20749.7
50	3.1	510	63743.6	41.7	10335.3	53408.2	21198.0
51	3.0	492	61537.1	41.1	9780.3	51756.8	21595.6
52	3.0	475	59387.9	40.7	9275.9	50112.0	21943.8
53	2.9	458	57293.2	40.2	8816.1	48477.1	22244.1
54	2.9	442	55249.8	39.7	8395.6	46854.2	22497.5
55	2.8	426	53255.1	39.3	8010.0	45245.1	22704.9
56	2.8	410	51306.4	38.8	7655.4	43651.0	22867.1
57	2.8	395	49401.1	38.4	7328.6	42072.5	22984.8
58	2.8	380	47536.9	38.0	7026.6	40510.2	23058.3
59	2.8	366	45711.3	37.6	6747.0	38964.3	23087.9
60	2.8	351	43922.3	37.2	6487.4	37434.9	23073.7
61	2.8	337	42167.8	36.8	6246.1	35921.7	23015.7
62	2.8	324	40445.7	36.5	6021.1	34424.6	22913.6
63	2.8	310	38754.3	36.1	5811.1	32943.2	22767.1
64	2.9	297	37091.7	35.8	5614.6	31477.0	22575.6
65	2.9	284	35456.1	35.4	5430.4	30025.7	22338.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 23087.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 37.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 38 pcf