Appendix D

1. Geotechnical Report

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September 15, 2017
File Number 21472

Mayer Brown
350 South Grand Avenue, 25th Floor
Los Angeles, California 90071

Attention: Mr. Edgar Khalatian

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
676 Mateo Street, Los Angeles, California

Dear Mr. Khalatian:

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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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
676 MATEO 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 drilling two borings, collection of representative samples, laboratory testing, engineering analysis, review of available geotechnical engineering information and the preparation of this report. The boring locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The site is proposed to be developed with an eight-story mixed-use development over three subterranean parking levels. A concrete podium with steel framing above ground is anticipated. It is anticipated that the lowest finish floor elevation will be approximately 30 feet below the existing site grade (approximate elevation 220 feet).

Structural loads were not available at this time; however, based on our experience on similar projects, we have estimated that for this type of structure, column loads may be on the order of 300 and 1,500 kips and wall loads are estimated to be between 7 and 12 kips per lineal foot.

Structural loads should be provided to this office as soon as they become available to verify that our results are still appropriate, in particularly our settlement analyses.

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

SITE CONDITIONS

The site is located at 676 Mateo Street in the Arts District of the City of Los Angeles, California. The site is bounded by an asphalt pavement parking lot to the north, by Imperial Street to the east, by a one-story warehouse structure to the south, and by Mateo Street to the west. The site is shown relative to topographic features on the attached Vicinity Map.

Rectangular in shape, the site is approximately 1.03 acres in area. The topography is relatively uniform with elevations ranging from 250 feet above mean sea level at the northeast corner to 249 feet at the southwest corner. The site gradient is approximately 300 to 1 (horizontal to vertical) descending to the southwest.

The site is developed with a one-story, at-grade warehouse building and a concrete parking lot. The neighboring development consists of one to two-story, at-grade, commercial and residential structures.

Drainage across the site appears to be by sheetflow to the city streets. The site is fully developed with a structure and parking, as such the site is barren of vegetation

LOCAL GEOLOGY

The site is located in the Los Angeles Basin which is bordered to the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains, and the west by the Pacific Ocean. Over 22 million years ago the Los Angeles basin was a deep marine basin. 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.

Located approximately 0.4 miles from the Los Angeles River, the site is underlain by a thick accumulation of recent alluvium and old alluvium that extends to a depth of approximately 220 feet below the ground surface (Yerkes, R.F. and Others, 1977). Underlying the alluvium is siltstone bedrock of the Fernando Formation (Dibblee, T.W., 1989). The bedrock is relatively impermeable and forms a barrier to vertical migration of groundwater. The distribution of the geologic units in the site vicinity is shown on the attached Local Geologic Map.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on July 20 and July 21, 2017, by drilling two exploratory borings. The depths of the exploratory borings varied between 52 and 61 feet below the existing site grade. The borings were drilled with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and by using 8-inch diameter hollow-stem augers. The boring locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 and A-2.

The locations of the exploratory borings were determined by measurement from hardscape features and existing structures shown on the attached Plot Plan. Elevations of the exploratory borings were interpolated from features shown on the Plot Plan. The location and elevation of the exploratory borings should be considered accurate only to the degree implied by the method used.

Percolation Testing was performed in Boring 2. A detailed discussion of the percolation test and its findings are presented later in this report.

Geologic Materials

The ground surface was paved with concrete that ranged in between 4 and 5 inches thick. Fill soil was encountered in all the exploratory borings to a depth of 3 feet. Fill soil underlying the site consists of silty sand, which is yellowish brown and dark brown, moist, and fine grained.

Underlying the fill is natural alluvium consisting of poorly- to well-graded sand, and silty sand which is yellowish brown and grayish brown and dark brown in color, moist to very moist, medium dense to very dense. The alluvium appears to coarsen with depth with increasing frequency and size and of gravel below a depth of 20 feet. The alluvium is consistent with the materials indicated on the attached Local Geologic Map.

Although not identified in the borings, siltstone bedrock of the Fernando Formation underlies the alluvium near a depth of 220 feet below the ground surface (Yerkes, R.F. and Others, 1977). More detailed soil profiles may be obtained from individual boring logs.

Groundwater

Groundwater was not encountered during exploration to a maximum depth of 61 feet below ground surface (bgs).

Based on Review of the Seismic Hazard Zone Report of the Los Angeles Quadrangle (CGS, 1999) the historically highest groundwater level is greater than 150 feet below the existing ground surface. A copy of the Historically Highest Groundwater Levels Map is attached.

It should be noted that water was not encountered nor is anticipated within the excavation depth or the anticipated shoring piles. 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 hollowstem auger. However, based on the experience of this firm, large diameter excavations that encounter granular, cohesionless soils will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

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

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.

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

Groundwater was not encountered during exploration, conducted to a maximum depth of 61 feet below the ground surface. In addition, the historic-high groundwater level for the site was deeper than 150 feet below the ground surface. Therefore, based on the dense to very dense consistency of the alluvium, depth to groundwater, and the depth to historic highest groundwater level, the potential for liquefaction occurring at the site is considered to be remote.

Dynamic Dry Settlement

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

Due to the dense consistency of the natural alluvium that the proposed structure will bear on, seismic settlement is not anticipated.

Tsunamis, Seiches and Flooding

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

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site lies within mapped inundation boundaries of the Sepulveda and Hansen Dams. 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 lack of slope across the site, the probability of a seismically-induced landslide is considered to be remote.

CONCLUSIONS AND RECOMMENDATIONS

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

Up to three feet of fill material was encountered during exploration at the site. The existing fill is considered to be unsuitable for support of new foundations, floor slabs, or additional fill. It is anticipated that the existing fill will be removed during excavation for the proposed subterranean levels. Natural alluvial soils at the level of the proposed foundation consist of dense to very dense sand.

Groundwater was not encountered within the 61-foot depth explored. Based on the California Geological Survey Seismic Hazard Zone Report of the Los Angeles 7.5-Minute Quadrangle (SHZR 029), the historically highest groundwater level is deeper than 150 feet bgs. As a result, the site is not considered liquefiable during the design earthquake.

The site is not located within an earthquake fault zone. The site is located within a flood area caused by an earthquake-induced seiche located in the Hansen Dam.

It is anticipated that excavations will be approximately 32 feet in depth will be required. The proposed structure may be supported by conventional foundations bearing in the undisturbed natural alluvial soils anticipated at the subgrade elevation. As an alternative option, a mat foundation may be used to distribute the building loads more uniformly.

The excavations will require temporary shoring measurements to provide a stable condition during the excavation process. Since water was note encountered within the 61-foot depth explored, water is not anticipated.

The site is suitable for infiltration of storm water into the dense alluvial soils.

SEISMIC DESIGN CONSIDERATIONS

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)	2.324g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.324g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.549g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.814g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.220g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.814g

EXPANSIVE SOILS

The onsite geologic materials are in the very low expansion range. The Expansion Index was found to be 7 for a representative bulk sample remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs on Grade" sections of this report. Reinforcing beyond the minimum required by the City of Los Angeles Department of Building and Safety is not required.

WATER-SOLUBLE SULFATES

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

METHANE ZONES

According to the City of Los Angeles Methane and Methane Buffer Zones map (City of Los Angeles, 2003), it appears that the subject site is located within a Methane Buffer Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane Buffer Zone designation. A copy of the portion of the map covering the Project Site is included herein.

GRADING GUIDELINES

The following guidelines are provided for any miscellaneous compaction that may be required, such as retaining wall backfill or sub-grade 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 95 percent of the maximum density in accordance with the most recent revision of the Los Angeles Building Code. Based on the laboratory test results performed by this firm, the granular soils encountered at the site would require the 95 percent compaction requirement.

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

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

Acceptable Materials

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

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

Utility Trench Backfill

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

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor of 5 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

It is recommended that the proposed mixed-use structure be supported on a system of conventional spread footings bearing in the competent native soils expected to be exposed at the subterranean level. As an alternative option, a mat foundation may be used to distribute the building loads more uniformly. Recommendations for both options are provided in the sections below.

Conventional Footings

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 bearing material.

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 bearing material.

The bearing capacity increase for each additional foot of width is 500 pounds per square foot. The bearing capacity increase for each additional foot of depth is 1,500 pounds per square foot. The maximum recommended bearing capacity is 8,000 pounds per square foot.

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

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing in the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. The client should be aware that miscellaneous structures constructed in this manner may potentially be damaged and will require replacement should liquefaction occurs during a major seismic event.

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

Based on City of Los Angeles minimum requirements, 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 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be ¾ inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ¼ 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.

Mat Foundation

The proposed mixed-use building may be constructed over three subterranean parking levels, extending on the order of approximately 30 feet below grade. Structural loads were not available at this time; however, based on experience with similar projects, it is the opinion of this firm that an average bearing pressure of 2,000 pounds per square foot may be used. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

Given the size of the proposed mat foundation, the average bearing pressure of 2,000 pounds per square foot is well below the allowable bearing pressures, with factor of safety well exceeding 3. For design purposes, an average bearing pressure of 2,500 pounds per square foot, with locally higher pressures up to 5,000 pounds per square foot may be utilized in the mat foundation design. The mat foundation may be designed utilizing a modulus of subgrade reaction of 300 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

Where:

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

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

Lateral Design for Mat Foundation

Resistance to lateral loading may be provided by soil friction, and by the passive resistance of the soils. A coefficient of friction of 0.45 may be used with the dead load forces between footings and the underlying supporting soils.

Passive earth pressure for the sides of footings poured against undisturbed soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot, with a maximum earth pressure of 3,000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. A one-third increase in the passive value may be used for wind or seismic loads. A minimum safety factor of 2 has been utilized in determining the allowable passive pressure.

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. It is anticipated that total settlement of 2 inches will occur below the more heavily loaded central core portions of the mat foundation beneath the tower. Settlement on the edges of the mat foundation is expected to be between ¾ to 1 inch.

RETAINING WALL DESIGN

Retaining walls on the order of 30 feet in height are anticipated for the proposed subterranean levels. It is anticipated these walls will be restrained.

Cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of active earth pressure. Restrained retaining walls may be designed utilizing a triangular distribution of at-rest earth pressure. Retaining walls may be designed utilizing the following table:

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure (pcf)
Up to 10	30	55
10 to 20	36	55
20 to 30	41	55

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

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

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 24 pounds per cubic foot. When using the load combination equation from the building code, the seismic earth pressure should be combined with the lateral active pressure for analyses of restrained basement walls under seismic loading condition. The comparison is made in the following table:

Use of Seismic Wall pressure (All Pressure Distributions are Triangular)			
Wall Height (feet)	**Active pressure (pcf)**	**Active +Seismic (pcf)**	**At-Rest (pcf)**
Up to 10	30	54	55
10 to 20	36	60	55
30 to 40	41	65	55

Surcharge from Adjacent Structures

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

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

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

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

Where:

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

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

Waterproofing

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

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

Retaining Wall Drainage

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

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by ASTM D 1557.

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

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

Where retaining walls are to be constructed adjacent to property lines there is usually not enough space for emplacement of a standard pipe and gravel drainage system. Under these circumstances, the use of a flat drainage produce is acceptable.

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

Retaining Wall Backfill

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

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Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration, drilled to a maximum depth of 61 feet which corresponds to 31 feet below the lowest proposed finished floor. Based the Seismic

Hazard Zone Report of the Los Angeles 7.5 Minutes Quadrangle (SHZR 029), the historic-groundwater level at the site was established deeper than 150 feet bgs. Therefore, the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 32 in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 4 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, or adjacent structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 (h:v) slope gradient to a maximum height of 32 feet. A uniform sloped excavation does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles

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

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.45 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 275 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 underlying the site. Where caving occurs, it will be necessary to utilize casing or polymer drilling fluid to maintain open pile shaft. 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. Larger sized materials should be anticipated during drilling (i.e. gravels and cobbles).

Lagging

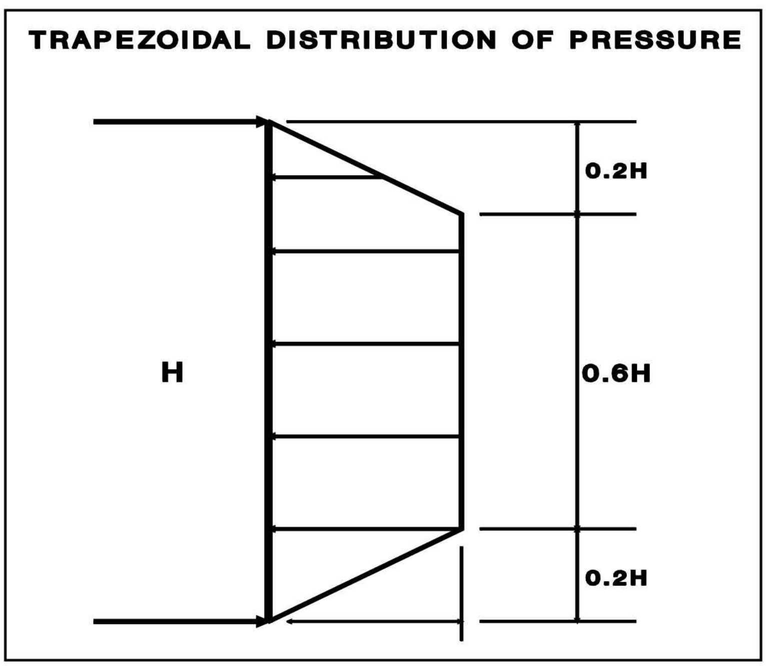
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less.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

Lateral Pressures

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

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure Triangular Distribution of Pressure Active Earth Pressure (pcf)	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 10	25	18H
10 to 20	27	18H
20 to 30	33	21H
30 to 40	36	23H

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



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

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

Tied-Back Anchors

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

Drilled friction anchors may be designed for a skin friction of 600 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

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

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

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

Anchor Installation

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

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment.

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment, where a structure is within a 1:1 (h;v) plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 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. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

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

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

Shoring Observations

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

Raker Brace Foundations

An allowable bearing pressure of 4,000 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 4 feet in width and length as well as 4 feet in depth. 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.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 4 inches in thickness. Slabs-on-grade should be cast over undisturbed natural earth material or properly controlled fill material. Any earth material loosened or over-excavated should be wasted from the site or properly compacted to 95 percent of the maximum dry density.

Outdoor Concrete Slabs

Outdoor concrete flatwork should be a minimum of 3 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed natural earth material or properly controlled fill material. Any earth material loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

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

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

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

Slab Reinforcing

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Introduction

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.

Percolation Testing

In order to establish a percolation rate for the onsite soils, percolation testing was performed in Boring 2, which was drilled to a depth of 61 feet below the existing grade. After completion of drilling, a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a slotted PVC pipe within the lower 10 feet of the borehole, and solid PVC pipe to the top of the borehole. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

Prior to testing, the borehole was filled with water for the purpose of pre-soaking for 4 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first. At the completion of the percolation testing, the PVC casing was removed from the percolation testing well, and the resulting hole was backfilled with on-site soils to the ground surface.

Recommendations

Based on results of the percolation tests, a field percolation rate of 60 inches per hour was obtained. An infiltration rate of 20 inches per hour may be utilized for design purposes when considering a factor of safety of 3. Based on the project design, it is proposed to construct up to three subterranean parking levels, approximately 30 feet below the existing grade. Therefore, we recommend infiltrating the onsite water in the ground by using a deep infiltration system such as a dry well to infiltrate at depths below the building foundation. Generally, the native site soils encountered during our geotechnical explorations consist mainly of granular sandy soils suitable for stormwater infiltration. The potential for creating a perched water condition by infiltrating stormwater at the anticipated depth is considered remote.

It is anticipated that the potential drywell system would be installed below the lowest subterranean level. Stormwater infiltration shall only occur below the primary zone of foundation influence. Based on the estimated structure loads, it is the determination of this firm that the primary zone of foundation influence extends to a depth of 15 feet below the bottom of the foundation. Therefore, it is recommended that stormwater infiltration should occur in the native alluvial soils located deeper than 15 feet below the bottom of the deepest adjacent foundation.

It is recommended that any potential drywell is installed at least 20 feet away from a private property line. The drywells should be installed centered in between surrounding foundations. Depending on its final location, it is anticipated that the settling chamber of the drywell may be surcharged by proposed adjacent foundations, in which case the chamber should be designed to withstand this additional surcharge load. The final location of the proposed drywells shall be reviewed and approved by this office prior to construction.

Drilling for the proposed drywells will most likely encounter gravelly material and possible large sized materials (i.e. cobbles). Due to the granular nature of the site soils, caving may occur in the drilled shafts. The use of casing to maintain open shafts for installation of the drywells should be anticipated.

It is recommended that the design team, including the structural engineer, waterproofing consultant, plumbing engineer, environmental engineer and landscape architect be consulted in regards to the design and construction of filtration systems. The design and construction of stormwater infiltration facilities is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

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

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

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

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

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. 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.

The City of Los Angeles does not require corrosion testing. However, 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 boring 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 "Boring 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-Plate.

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

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

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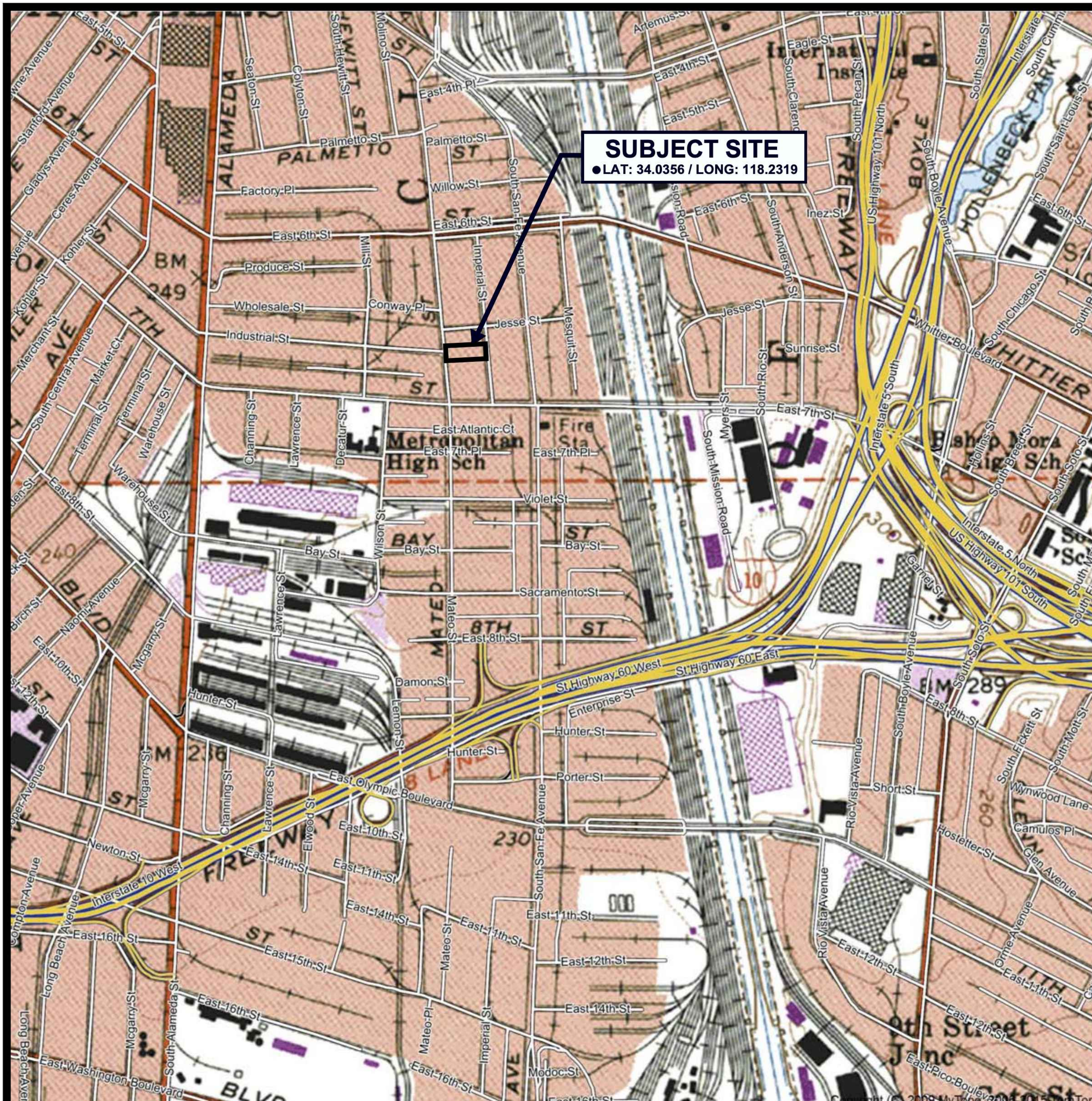
Yerkes, R.F., Tinsley, J.C. and Williams, K.M., Geologic Aspects of Tunneling in the Los Angeles Area, United State Geological Survey, Miscellaneous Field Studies Map MF-866, Map Scale 1:12,000.



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6

MILES

0 1000 2000 3000

FEET

N

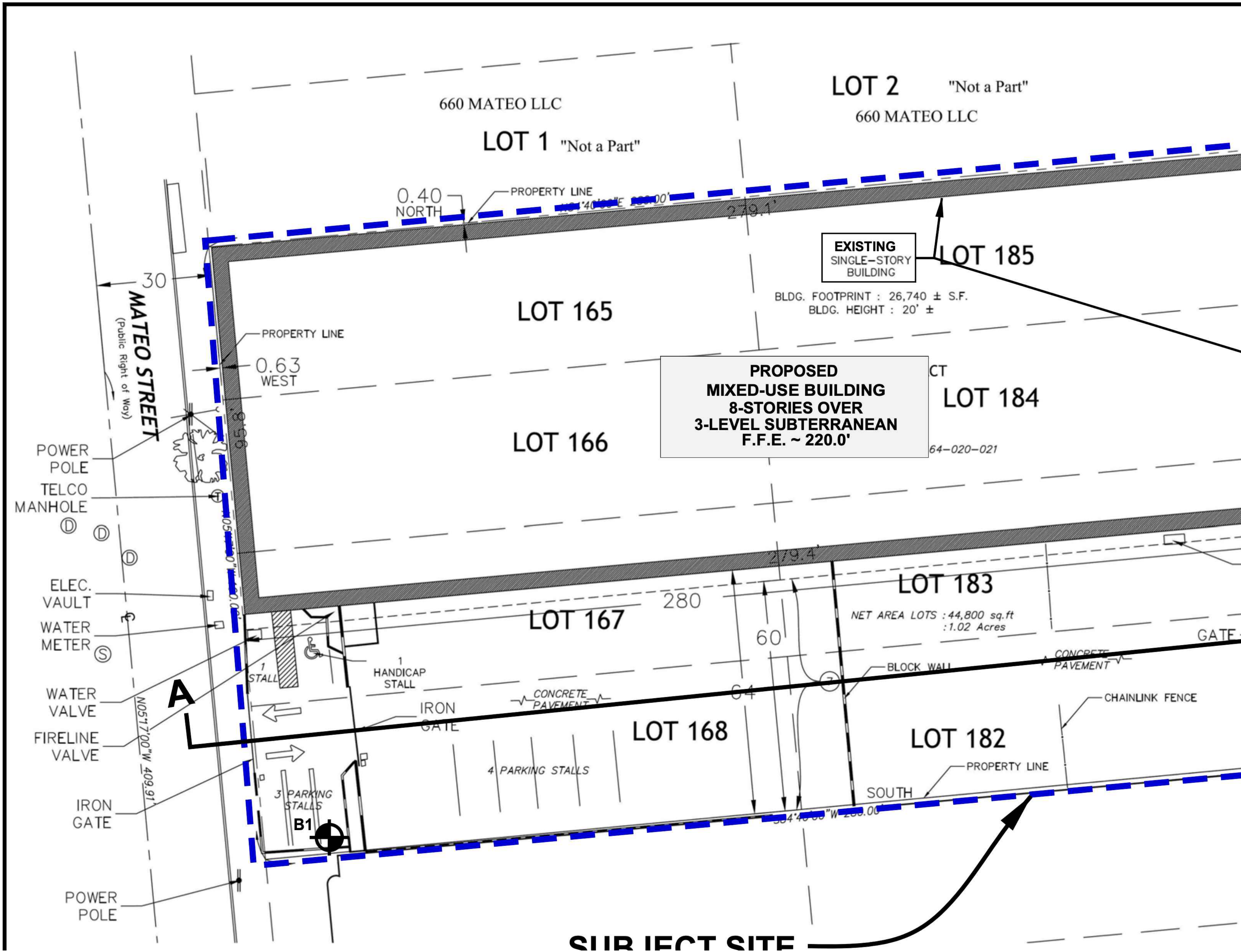
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES, CA QUADRANGLE

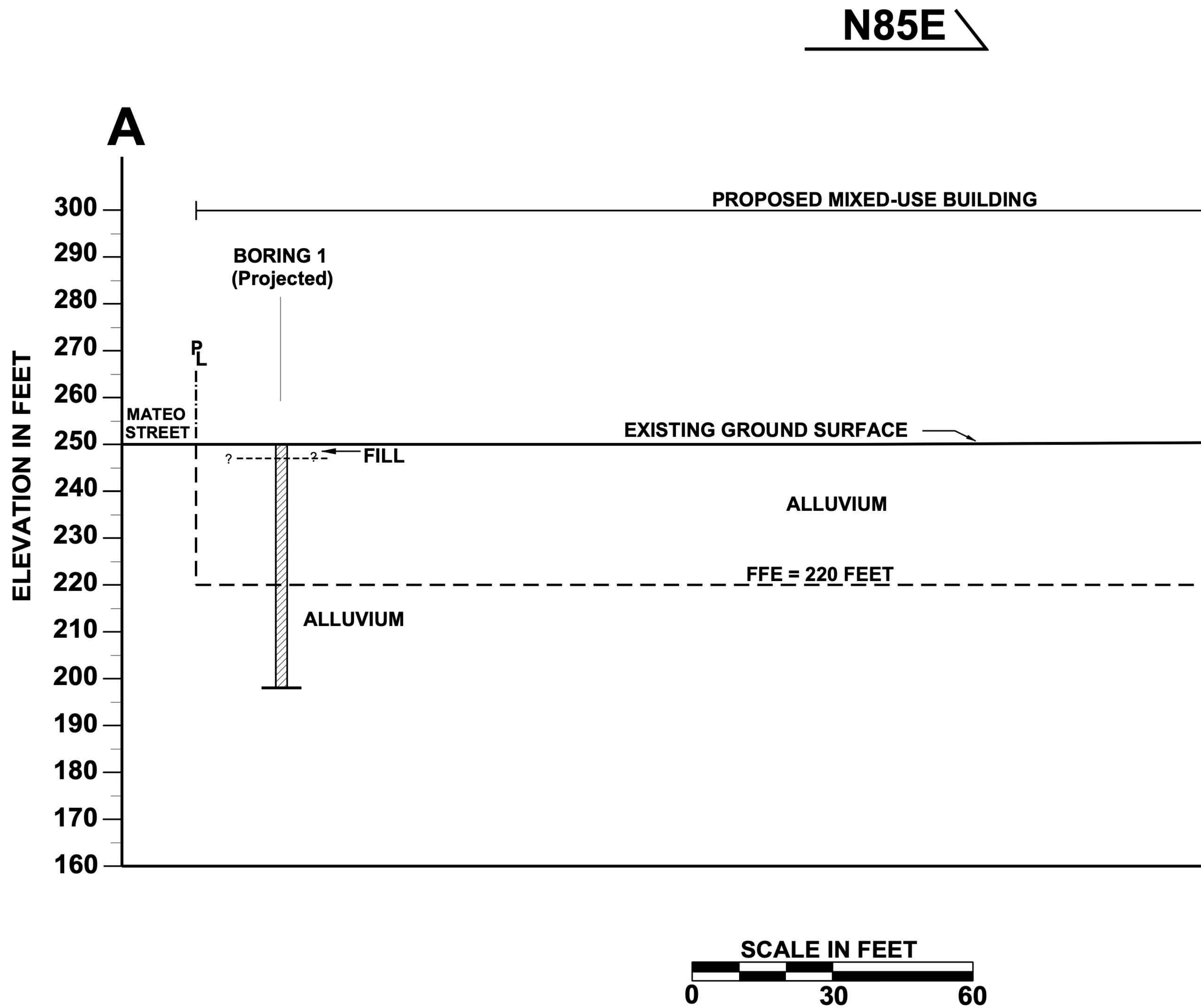
VICINITY MAP

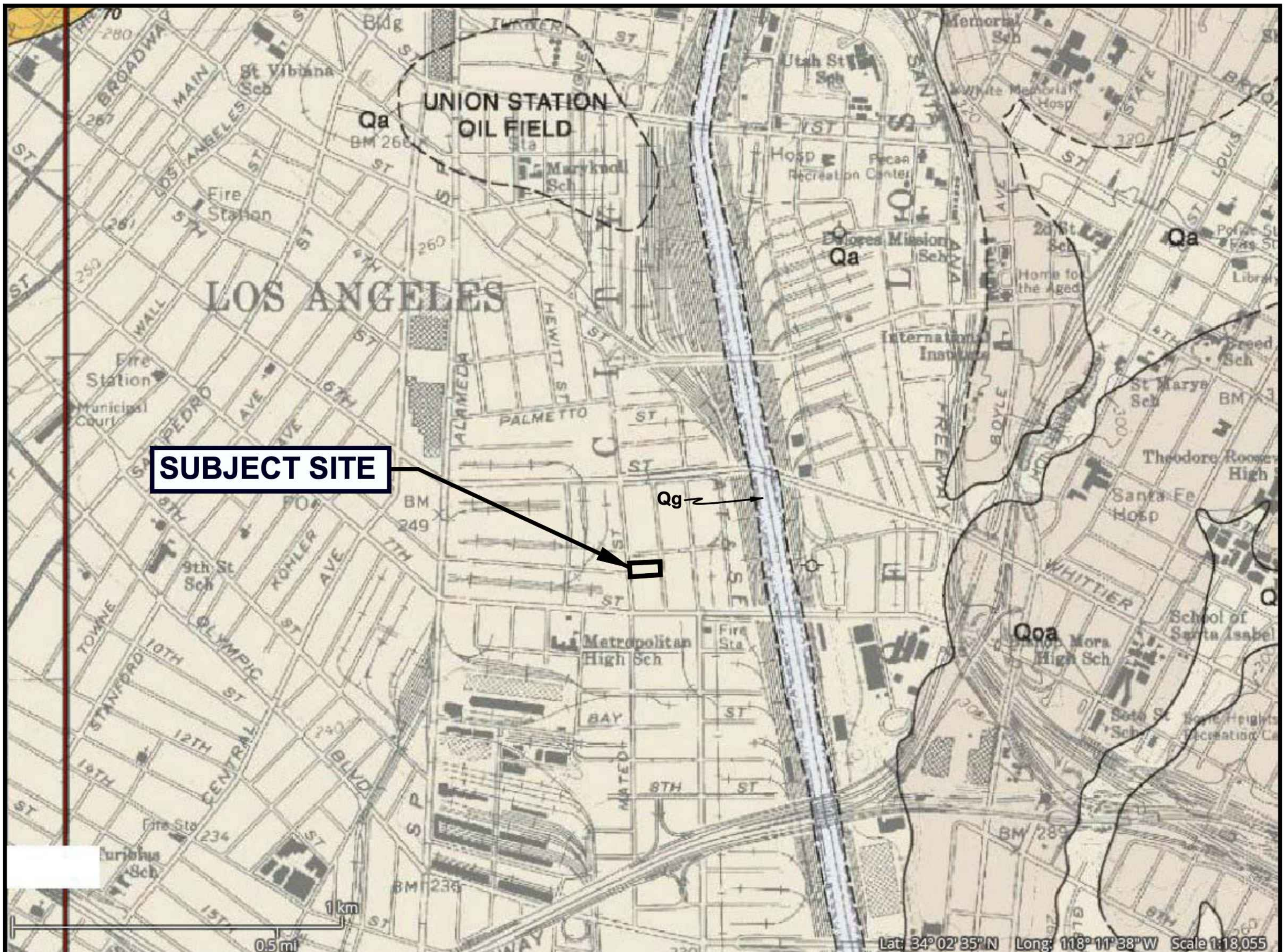
Geotechnologies, Inc.
Consulting Geotechnical Engineers

MAYER BROWN
676 MATEO STREET, LOS ANGELES

FILE NO. 21472


LOT 2
"Not a Part"
660 MATEO LLC
660 MATEO LLC
LOT 1 "Not a Part"
0.40
NORTH
PROPERTY LINE
279.1'
EXISTING
SINGLE-STORY
BUILDING
LOT 185
BLDG. FOOTPRINT : 26,740 ± S.F.
BLDG. HEIGHT : 20' ±
30
MATEO STREET
(Public Right of Way)
LOT 165
PROPERTY LINE
0.63
WEST
PROPOSED
MIXED-USE BUILDING
8-STORIES OVER
3-LEVEL SUBTERRANEAN
F.F.E. ~ 220.0'
LOT 184
LOT 166
POWER
POLE
TELCO
MANHOLE
ELEC.
VAULT
WATER
METER
WATER
VALVE
FIRELINE
VALVE
IRON
GATE
POWER
POLE
N05°17'00"W 409.91'
279.4'
LOT 183
LOT 167
280
60
64
NET AREA LOTS : 44,800 sq.ft
: 1.02 Acres
1
HANDICAP
STALL
1
STALL
BLOCK WALL
CONCRETE
PAVEMENT
CHAINLINK FENCE
IRON
GATE
A
LOT 168
LOT 182
4 PARKING STALLS
3 PARKING
STALLS
B1
PROPERTY LINE
SOUTH
SUBJECT SITE


N85E
A
ELEVATION IN FEET
300
290
280
270
260
250
240
230
220
210
200
190
180
170
160
PROPOSED MIXED-USE BUILDING
BORING 1
(Projected)
PL
MATEO
STREET
EXISTING GROUND SURFACE
FILL
ALLUVIUM
FFE = 220 FEET
ALLUVIUM
SCALE IN FEET
0
30
60



LEGEND

Qg: Surficial Sediments - stream channel deposits
Qa: Surficial Sediments - alluvium: unconsolidated floodplain deposits of silt, sand and gravel
Qoa: Older Surficial Sediments - remnants of older weakly consolidated alluvial deposits of gravel, sand & silt

Folds - arrow on axial trace of fold indicates direction of plunge
Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (1989) GEOLOGIC MAP OF THE LOS ANGELES QUADRANGLE (#DF-22)

N

LOCAL GEOLOGIC MAP - DIBBLEE

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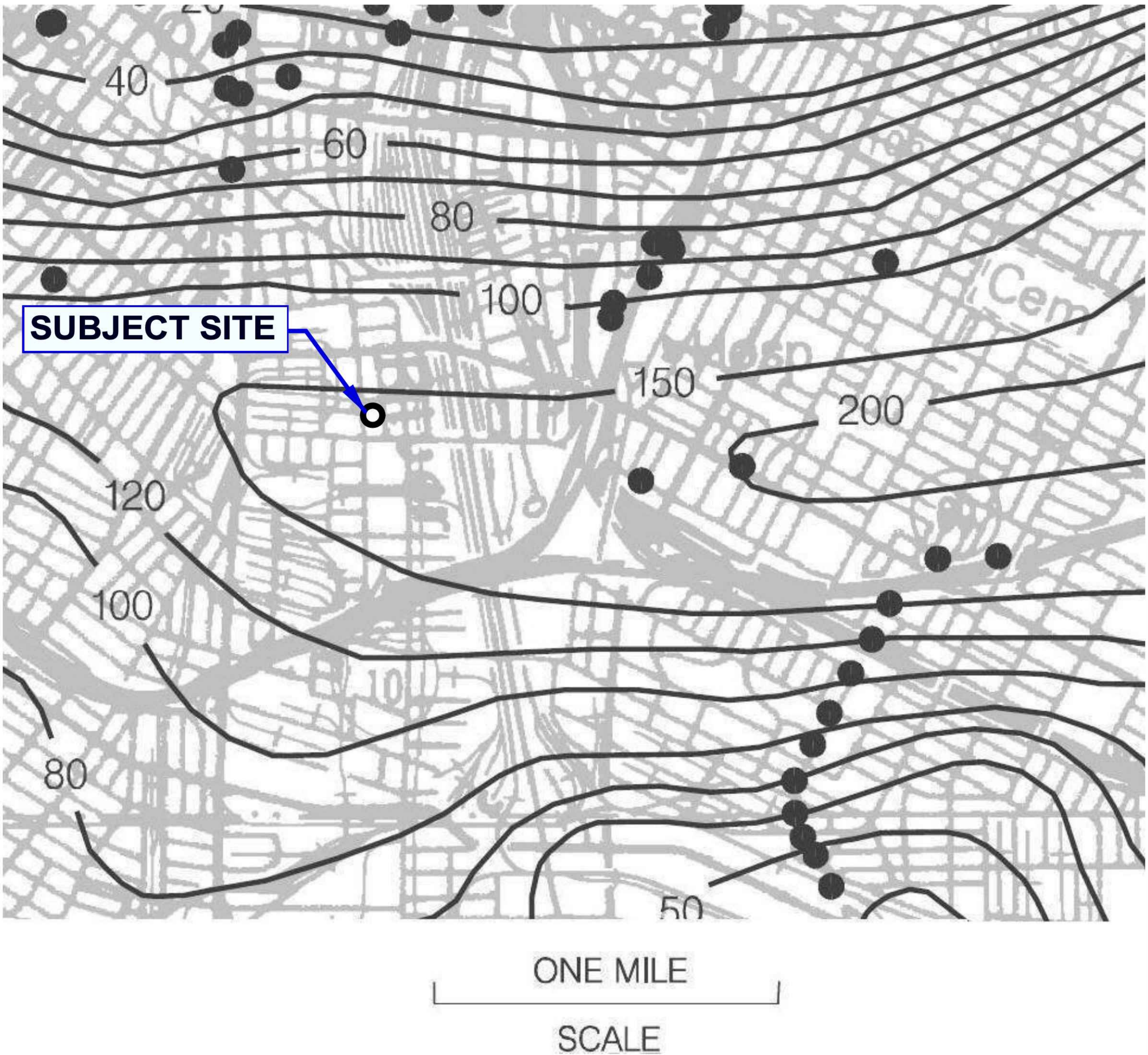
Consulting Geotechnical Engineers

MAYER BROWN
676 MATEO STREET, LOS ANGELES

FILE NO. 21472



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

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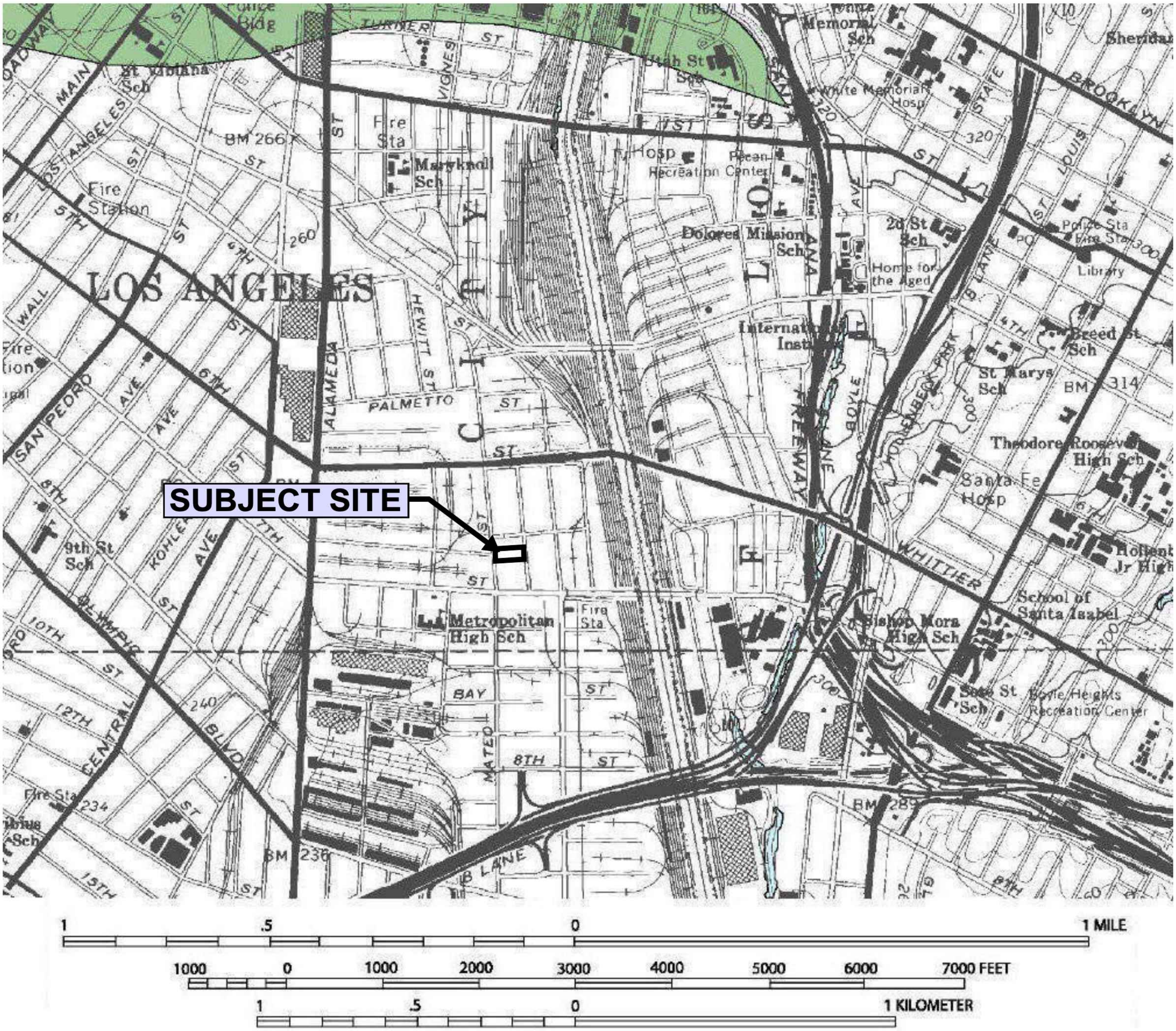
MAYER BROWN
676 MATEO STREET, LOS ANGELES

FILE NO. 21472



LIQUEFACTION AREA

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

N

SEISMIC HAZARD ZONE MAP

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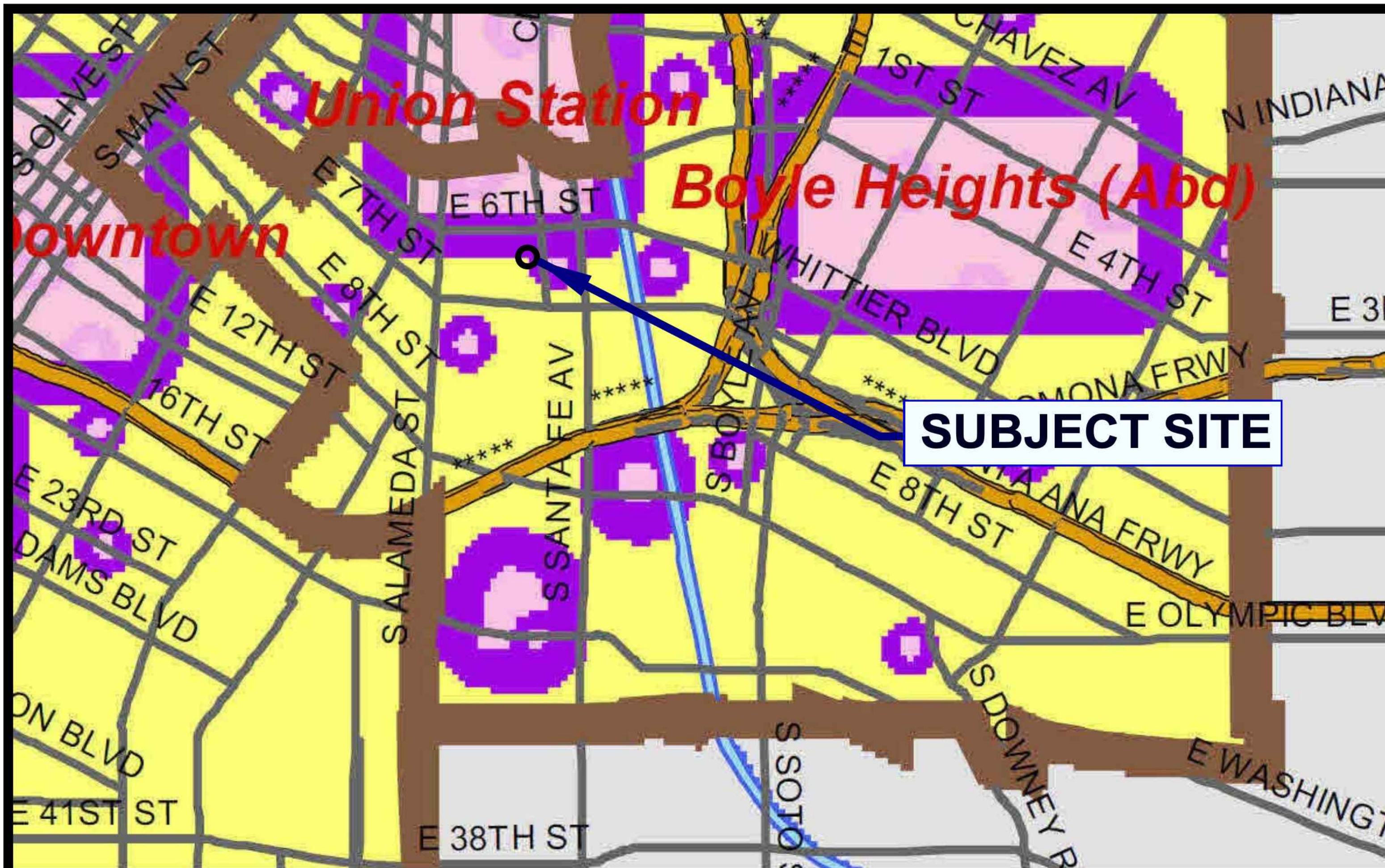
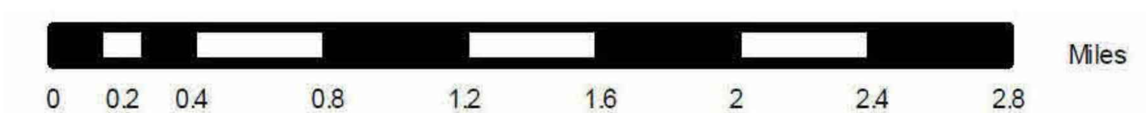
MAYER BROWN
676 MATEO STREET, LOS ANGELES

FILE NO. 21472





LEGEND

- Methane Zone
- Methane Buffer Zone
- Council District Boundary

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

METHANE ZONE RISK MAP

Geotechnologies, Inc.
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MAYER BROWN
676 MATEO STREET, LOS ANGELES

FILE NO. 21472

BORING LOG NUMBER 1

Mayer Brown - 676 Mateo St. **Date: 07/21/17** **Elevation: 250'***

File No. 21472 **Method: 8-inch diameter Hollow Stem Auger**

km

*Reference: Based on Google Earth Pro Website, 2016 image

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete for Parking
				0 --		5-inch Thick Concrete, No Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, fine grained
				-		
				2 --		
2.5	19	5.5	104.1	-		
				3 --		
				-	SP	ALLUVIUM: Sand, dark gray, moist, loose, fine grained, some Silt, poorly graded
				4 --		
				-		
5	7	2.9	SPT	5 --		
				-		gray, less Silt
				6 --		
				-		
				7 --		
7.5	15	6.7	110.0	-		
				8 --		thin layer of Sandy Silt, olive gray, moist, fine Sand
				-		
				9 --		
				-		
10	17	1.7	SPT	10 --		
				-		yellowish gray, medium dense, fine grained, some medium
				11 --		
				-		
				12 --		
12.5	23	4.7	107.6	-		
				13 --		light gray, fine to medium grained
				-		
				14 --		
				-		
15	10	2.6	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	43/6" 50/3"	0.4	124.7	-		
				18 --		some Silt
				-		
				19 --	SW	Sand, gray, moist, very dense, fine to coarse grained, some Gravel, well graded
				-		
20	22	3.8	SPT	20 --		
				-	SP	Sand, gray, moist, medium dense, fine to medium grained, poorly graded
				21 --		
				-		
				22 --		
22.5	75	2.7	110.7	-		
				23 --		light gray, fine to medium grained
				-		
				24 --		
				-		
25	43	2.2	SPT	25 --		
				-		dense, minor fine gravel

BORING LOG NUMBER 1

Mayer Brown - 676 Mateo St.

File No. 21472

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	76	1.6	114.7	-		
				28 --		light gray, moist, dense, fine to medium grained, some coarse
				-		
				29 --		
				-		
30	60	2.2	SPT	30 --		
				-		yellowish brown, moist, very dense, fine to medium grained, minor Gravel
				31 --		
				-		
				32 --		
32.5	100/9"	0.4	127.3	-		
				33 --		fine to coarse grained, some Gravel
				-		
				34 --		
				-		
35	58	1.8	SPT	35 --		
				-		yellowish brown, moist, very dense, fine to medium grained, minor Gravel
				36 --		
				-		
				37 --		
37.5	100/9"	1.4	132.6	-		
				38 --		fine to coarse grained, some Gravel
				-		
				39 --		
				-		
40	50/6"	1.3	SPT	40 --		
				-		dark gray, very dense
				41 --		
				-		
				42 --		
42.5	100/9"	0.5	121.7	-		
				43 --	SW	Sand, gray, moist, very dense, fine to coarse grained, some Gravel, well graded
				-		
				44 --		
				-		
45	50/5"	1.6	SPT	45 --		
				-	SP	Sand, gray, moist, very dense, fine grained, some medium, few Gravel, poorly graded
				46 --		
				-		
				47 --		
47.5	100/8.5"	No Recovery		-		
				48 --		
				-		
				49 --		
				-		
50	36/6"	0.9	SPT	50 --		
	50/2"			-		gray, moist, very dense, fine grained, more Gravel

BORING LOG NUMBER 1

Mayer Brown - 676 Mateo St.

File No. 21472

km

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

BORING LOG NUMBER 2

Mayer Brown - 676 Mateo St. **Date: 07/20/17** **Elevation: 251'***

File No. 21472 **Method: 8-inch diameter Hollow Stem Auger**

km *Reference: Based on Google Earth Pro Website, 2016 image

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete for Parking
				0 --		5-inch Thick Concrete, No Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, fine grained
				-		
				2 --		
2.5	14	3.8	100.6	-		
				3 --		
				-	SP	ALLUVIUM: Sand, yellowish brown, moist, loose, fine grained, trace fine Gravel, poorly graded
				4 --		
				-		
				5 --		
				-		
6	28	2.5	109.1	6 --		
				-		medium dense, some Silt, few Gravel
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	37	1.9	107.1	10 --		
				-		yellowish brown and gray, moist, medium dense, fine to medium grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	74	2.6	115.1	15 --		
				-		dense, fine to medium grained, more Gravel
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	38/6" 50/5"	2.4	118.7	20 --		
				-		very dense, some coarse grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	30/6" 50/3"	0.9	112.5	25 --		
				-		yellowish brown, fine grained

BORING LOG NUMBER 2

Mayer Brown - 676 Mateo St.

File No. 21472

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	35/6" 50/3"	12.3	113.0	30 --		gray, fine to medium grained, some coarse
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	100/8"	1.5	119.9	35 --		fine to coarse grained, some Gravel (up to 2" in size)
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	100/8"	1.5	122.7	40 --		gray, fine to medium grained, some coarse
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	100/7"	1.7	124.9	45 --		light yellowish brown, moist, very dense, fine to medium grained
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	100/6"	7.3	133.1	50 --		dark olive brown, some Silt, few Gravel
				-		

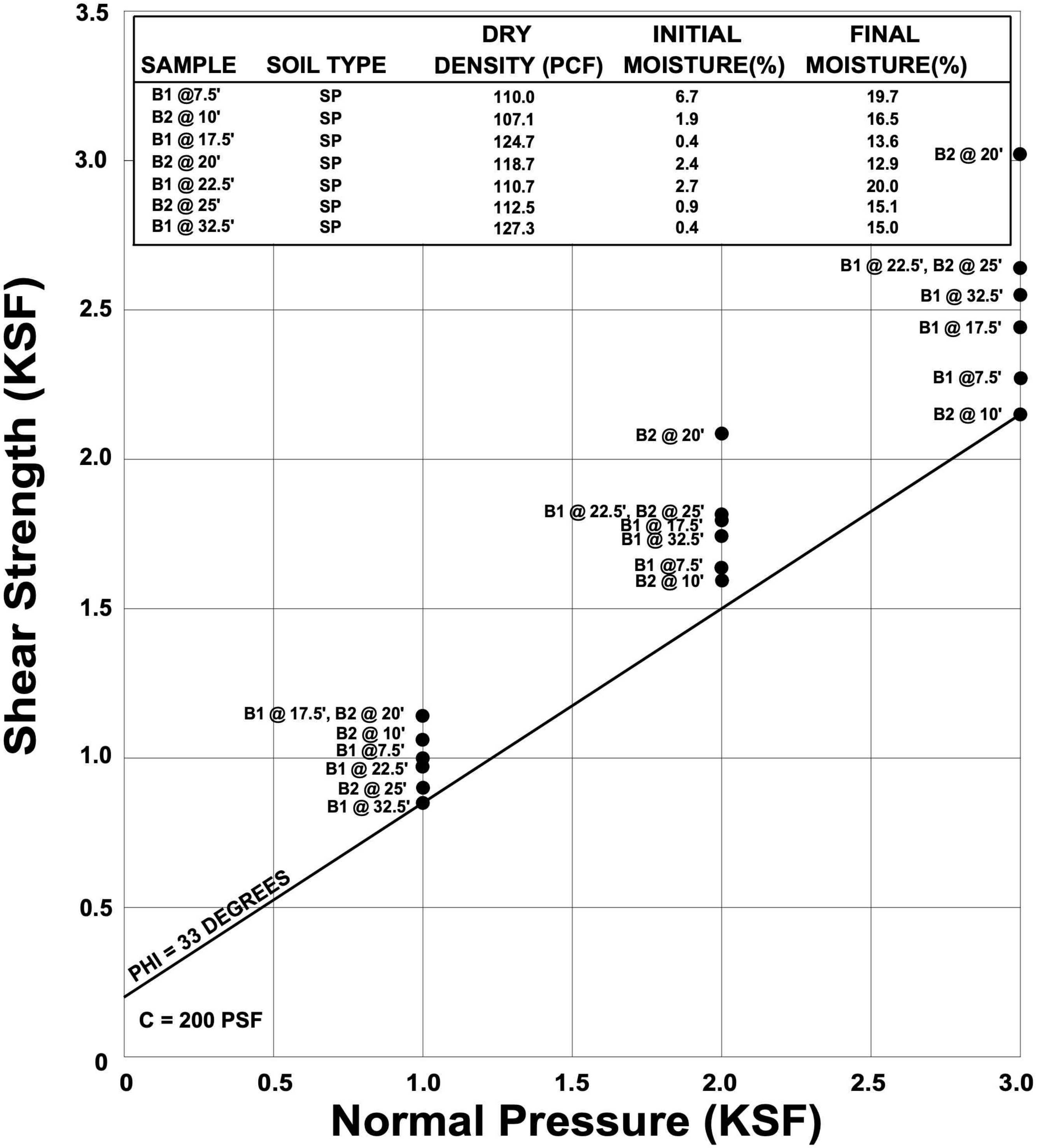
BORING LOG NUMBER 2

Mayer Brown - 676 Mateo St.

File No. 21472

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	90	12.9	103.4	55 --		
				-		grayish brown, fine to medium grained
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	100/8"	4.5	100.3	60 --		light olive brown, fine grained, some medium
				-		
				61 --		
				-		Total Depth 61 feet
				62 --		No Water
				-		Fill to 3 feet
				63 --		
				-		
				64 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				65 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				66 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				67 --		
				-		Performed Percolation Test in Boring
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @7.5'	SP	110.0	6.7	19.7
B2 @ 10'	SP	107.1	1.9	16.5
B1 @ 17.5'	SP	124.7	0.4	13.6
B2 @ 20'	SP	118.7	2.4	12.9
B1 @ 22.5'	SP	110.7	2.7	20.0
B2 @ 25'	SP	112.5	0.9	15.1
B1 @ 32.5'	SP	127.3	0.4	15.0

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
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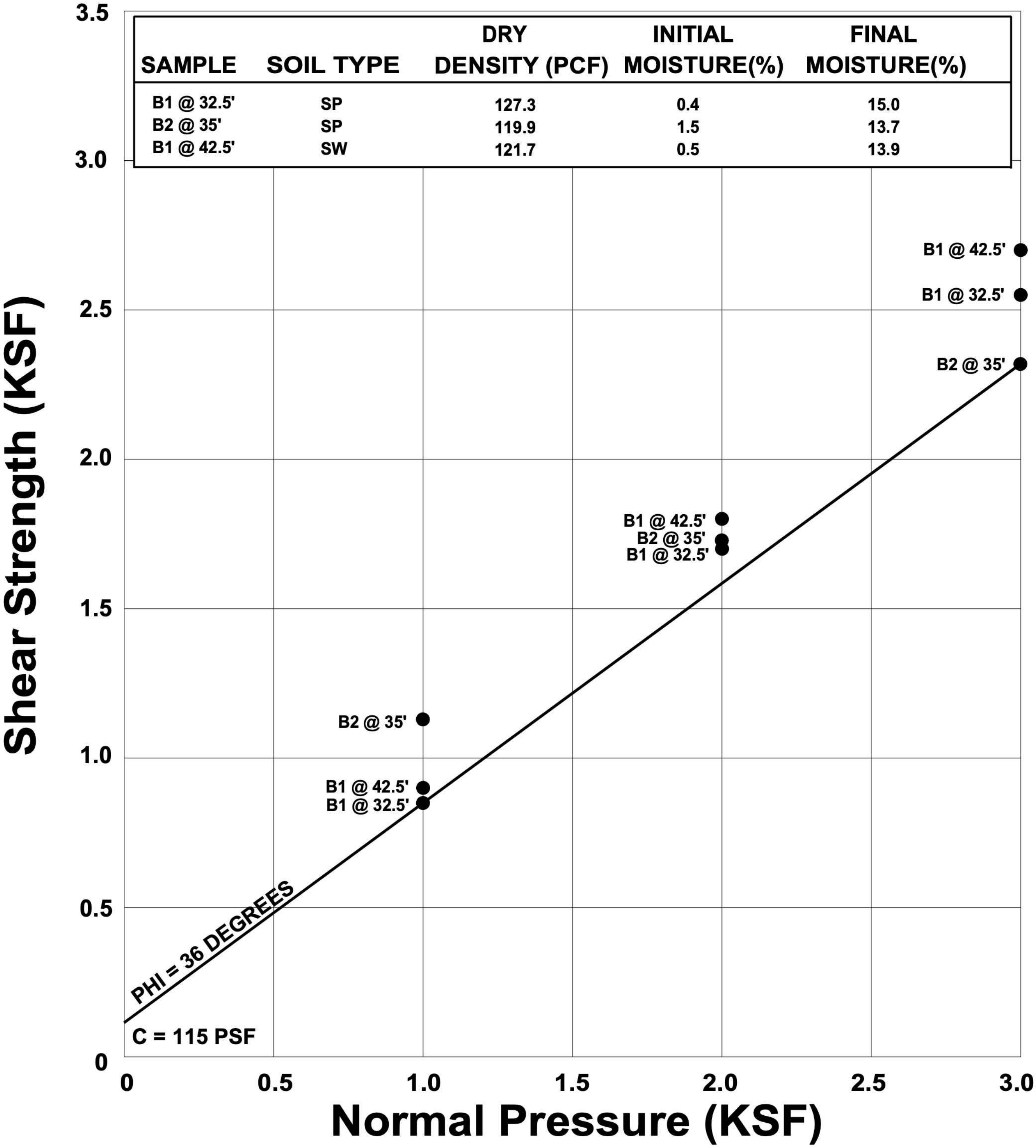
MAYER BROWN
676 Mateo Street, Los Angeles

FILE NO. 21472 | PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 32.5'	SP	127.3	0.4	15.0
B2 @ 35'	SP	119.9	1.5	13.7
B1 @ 42.5'	SW	121.7	0.5	13.9



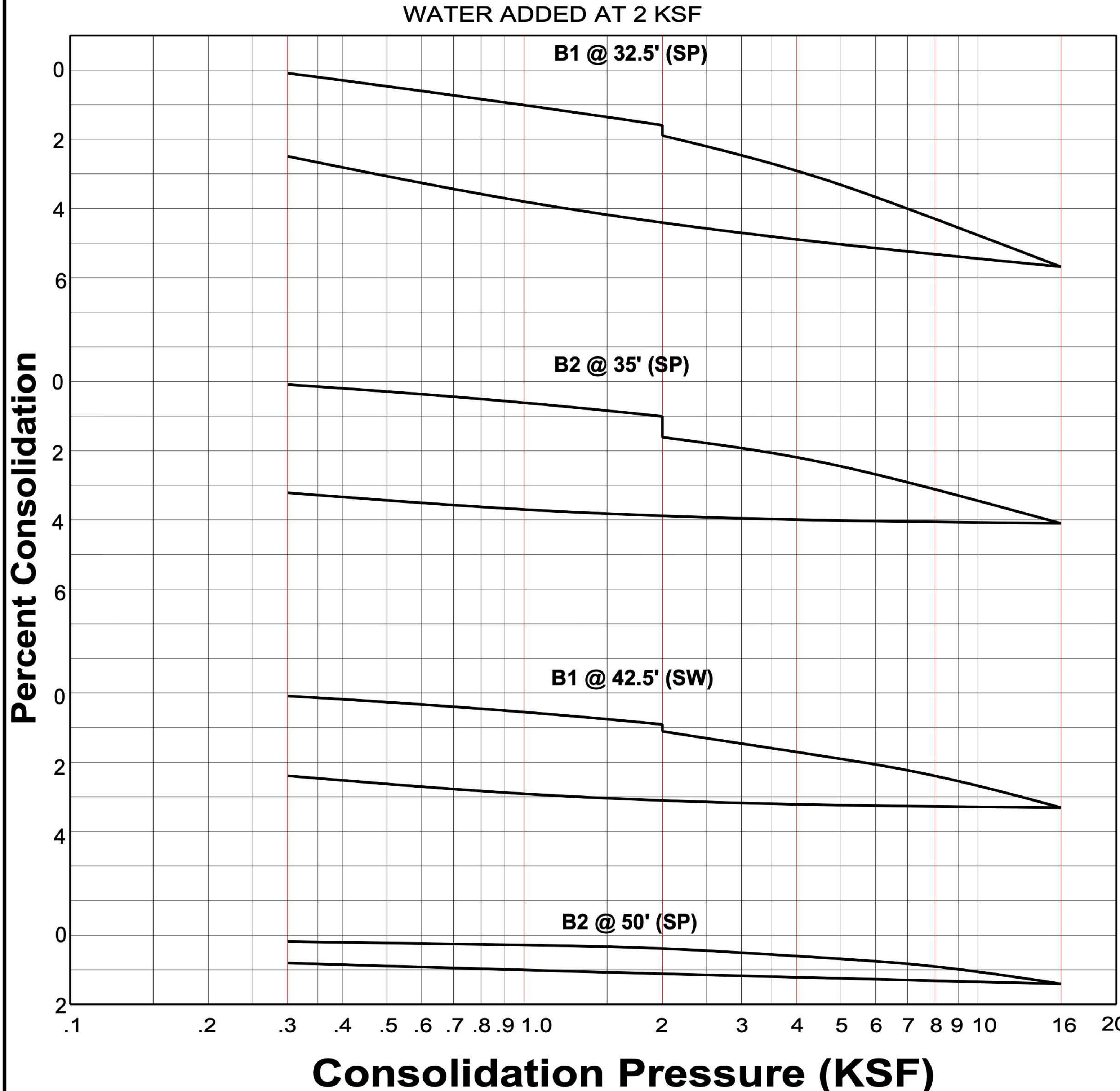
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MAYER BROWN
676 Mateo Street, Los Angeles

FILE NO. 21472 | PLATE: B-2



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MAYER BROWN 676 Mateo Street, Los Angeles	
FILE NO. 21472	PLATE: C

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM/SP
MAXIMUM DENSITY pcf.	128.4
OPTIMUM MOISTURE %	9.0

ASTM D 4829

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

SULFATE CONTENT

SAMPLE	B1 @ 1-5'	B2 @ 1-5'	B1 @ 35'	B2 @ 35'
SULFATE CONTENT: (percentage by weight)	< 0.1 %	< 0.1 %	< 0.1 %	< 0.1 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

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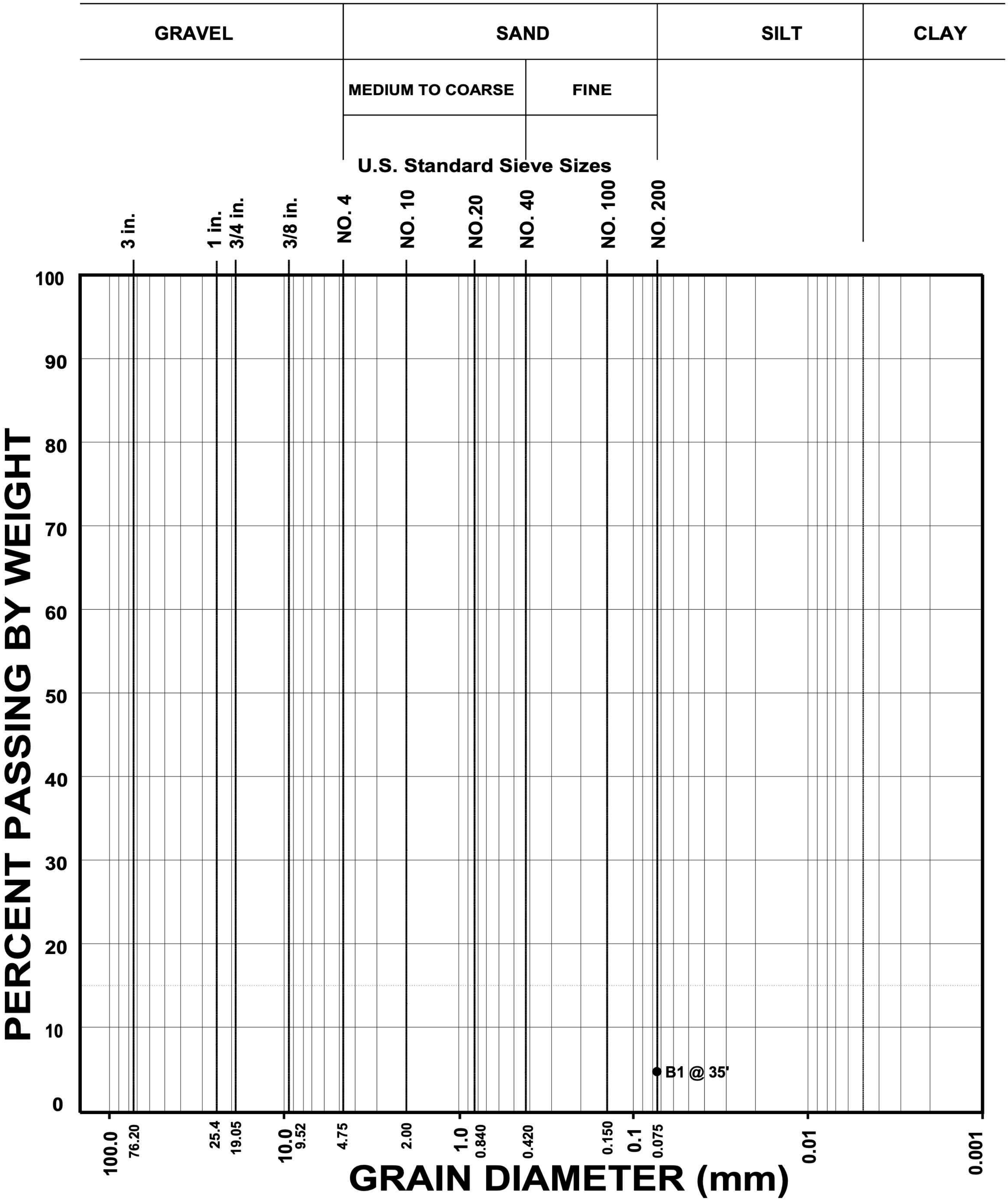
Consulting Geotechnical Engineers

MAYER BROWN
676 Mateo Street, Los Angeles

FILE NO. 21472 | PLATE: D



GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MAYER BROWN
676 Mateo Street, Los Angeles

FILE NO. 21472

PLATE: E

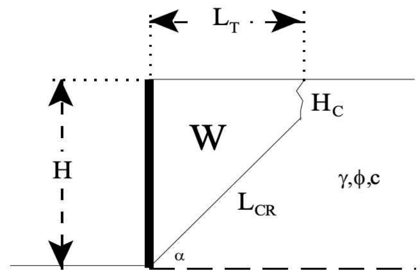
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Retaining Walls up to 10 feet

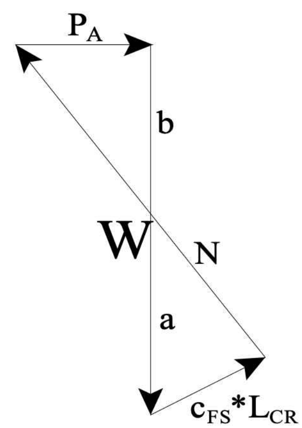
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	33.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	23.4 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.9	42	5078.6	8.6	2859.7	2218.8	878.1
46	3.8	41	4948.1	8.6	2736.0	2212.1	920.4
47	3.7	40	4814.2	8.6	2618.7	2195.4	958.7
48	3.7	39	4678.0	8.5	2507.8	2170.2	993.1
49	3.6	38	4540.4	8.5	2402.9	2137.5	1023.7
50	3.5	37	4402.3	8.4	2303.8	2098.5	1050.4
51	3.5	36	4264.1	8.4	2210.2	2053.9	1073.3
52	3.5	34	4126.2	8.3	2121.8	2004.5	1092.4
53	3.4	33	3989.0	8.2	2038.1	1950.9	1107.8
54	3.4	32	3852.7	8.1	1958.9	1893.8	1119.6
55	3.4	31	3717.4	8.1	1883.8	1833.6	1127.6
56	3.4	30	3583.2	8.0	1812.5	1770.7	1132.0
57	3.4	29	3450.3	7.9	1744.7	1705.6	1132.8
58	3.4	28	3318.5	7.8	1680.1	1638.4	1129.9
59	3.4	27	3188.0	7.7	1618.4	1569.6	1123.3
60	3.4	25	3058.7	7.6	1559.3	1499.3	1113.1
61	3.4	24	2930.5	7.5	1502.7	1427.8	1099.2
62	3.5	23	2803.5	7.4	1448.1	1355.3	1081.6
63	3.5	22	2677.4	7.3	1395.4	1282.0	1060.2
64	3.6	21	2552.4	7.1	1344.3	1208.1	1035.1
65	3.6	20	2428.2	7.0	1294.6	1133.6	1006.1
66	3.7	19	2304.8	6.9	1246.0	1058.8	973.3
67	3.8	18	2182.0	6.8	1198.2	983.8	936.6
68	3.9	17	2059.7	6.6	1151.0	908.8	895.9
69	4.0	16	1937.8	6.4	1103.9	833.9	851.2
70	4.1	15	1816.0	6.3	1056.8	759.2	802.6



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1132.76 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$
EFP 22.7 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf (Recommended)

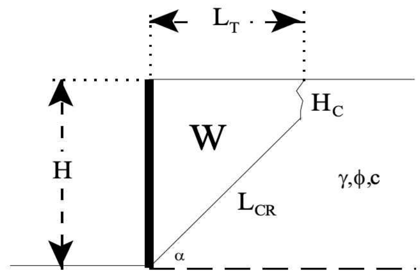
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Retaining Walls up to 20 feet

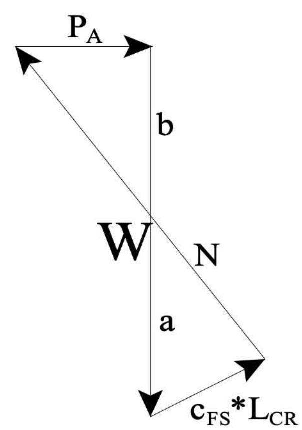
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	23.4	degrees
	(c_{FS})	133.3	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.9	192	23078.6	22.7	7562.3	15516.2	6140.3
46	3.8	186	22330.5	22.5	7164.0	15166.5	6310.2
47	3.7	180	21599.4	22.2	6799.3	14800.1	6463.1
48	3.7	174	20885.2	22.0	6464.5	14420.8	6599.4
49	3.6	168	20187.6	21.7	6156.4	14031.2	6719.7
50	3.5	163	19506.1	21.5	5872.3	13633.8	6824.4
51	3.5	157	18840.2	21.2	5609.7	13230.5	6913.9
52	3.5	152	18189.4	21.0	5366.5	12822.9	6988.5
53	3.4	146	17553.0	20.7	5140.8	12412.2	7048.4
54	3.4	141	16930.5	20.5	4930.9	11999.6	7093.8
55	3.4	136	16321.1	20.3	4735.3	11585.9	7125.0
56	3.4	131	15724.4	20.0	4552.6	11171.8	7142.0
57	3.4	126	15139.6	19.8	4381.8	10757.8	7144.9
58	3.4	121	14566.2	19.6	4221.6	10344.6	7133.7
59	3.4	117	14003.5	19.4	4071.2	9932.3	7108.3
60	3.4	112	13451.0	19.1	3929.6	9521.4	7068.7
61	3.4	108	12908.1	18.9	3796.0	9112.0	7014.8
62	3.5	103	12374.2	18.7	3669.8	8704.4	6946.2
63	3.5	99	11848.9	18.5	3550.2	8298.7	6862.9
64	3.6	94	11331.6	18.3	3436.7	7894.9	6764.5
65	3.6	90	10821.8	18.1	3328.5	7493.3	6650.6
66	3.7	86	10318.9	17.8	3225.1	7093.8	6520.9
67	3.8	82	9822.6	17.6	3126.1	6696.5	6374.9
68	3.9	78	9332.2	17.4	3030.8	6301.5	6212.0
69	4.0	74	8847.4	17.2	2938.7	5908.7	6031.7
70	4.1	70	8367.5	16.9	2849.2	5518.3	5833.4



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

$P_{A,\,max}$ 7144.89 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 35.7 pcf

Design Wall for an Equivalent Fluid Pressure: 36 pcf

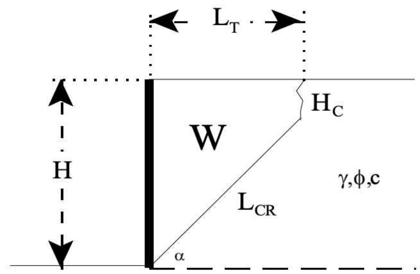
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Retaining Walls up to 30 feet

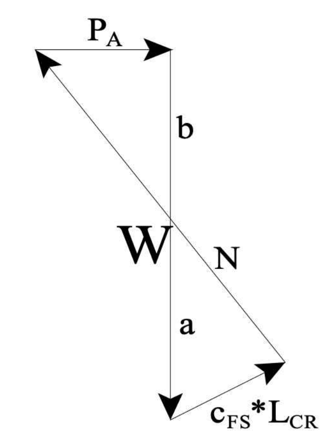
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	23.4	degrees
	(c_{FS})	133.3	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.9	442	53078.6	36.9	12264.9	40813.6	16151.3
46	3.8	428	51301.2	36.4	11592.1	39709.1	16521.5
47	3.7	413	49574.9	35.9	10979.9	38595.0	16854.0
48	3.7	399	47897.4	35.4	10421.2	37476.2	17150.3
49	3.6	386	46266.2	35.0	9909.9	36356.3	17411.5
50	3.5	372	44679.1	34.5	9440.7	35238.3	17638.6
51	3.5	359	43133.7	34.1	9009.2	34124.5	17832.6
52	3.5	347	41627.9	33.7	8611.2	33016.7	17994.1
53	3.4	335	40159.6	33.3	8243.5	31916.2	18123.8
54	3.4	323	38726.7	32.9	7902.9	30823.9	18222.2
55	3.4	311	37327.4	32.5	7586.7	29740.6	18289.7
56	3.4	300	35959.6	32.1	7292.7	28666.9	18326.5
57	3.4	289	34621.8	31.7	7018.8	27603.0	18332.7
58	3.4	278	33312.2	31.4	6763.1	26549.1	18308.4
59	3.4	267	32029.3	31.0	6523.9	25505.4	18253.6
60	3.4	256	30771.5	30.7	6299.8	24471.7	18167.9
61	3.4	246	29537.3	30.4	6089.4	23447.9	18051.0
62	3.5	236	28325.5	30.0	5891.6	22434.0	17902.6
63	3.5	226	27134.7	29.7	5705.1	21429.6	17722.0
64	3.6	216	25963.6	29.4	5529.0	20434.6	17508.6
65	3.6	207	24811.0	29.1	5362.3	19448.7	17261.5
66	3.7	197	23675.8	28.8	5204.3	18471.5	16979.7
67	3.8	188	22556.8	28.5	5053.9	17502.9	16662.2
68	3.9	179	21453.0	28.2	4910.5	16542.5	16307.6
69	4.0	170	20363.3	27.9	4773.4	15589.9	15914.5
70	4.1	161	19286.6	27.6	4641.6	14645.0	15481.4



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

$P_{A,\,max}$ 18332.71 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$
EFP 40.7 pcf

Design Wall for an Equivalent Fluid Pressure: 41 pcf

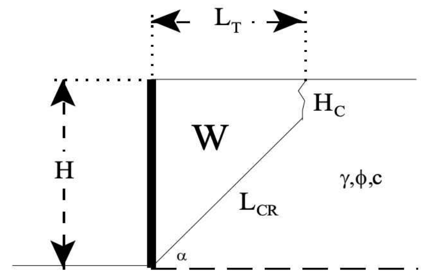
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Shoring Wall up to 10 ft

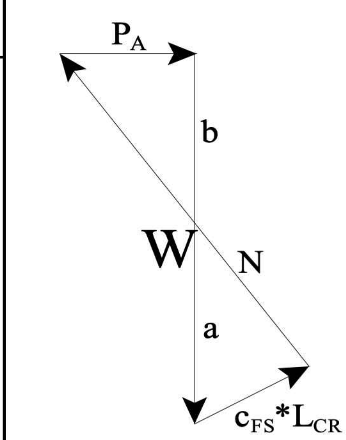
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	27.5	degrees
	(c_{FS})	160.0	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
45	5.6	35	4151.8	6.3	2963.7	1188.1	375.7
46	5.4	34	4132.7	6.5	2882.5	1250.3	419.5
47	5.2	34	4090.7	6.6	2793.7	1297.0	460.5
48	5.0	34	4031.2	6.7	2701.1	1330.1	498.5
49	4.9	33	3958.0	6.7	2607.0	1351.0	533.4
50	4.8	32	3874.4	6.8	2513.3	1361.1	565.1
51	4.7	32	3782.6	6.8	2421.0	1361.7	593.4
52	4.6	31	3684.6	6.8	2330.7	1353.8	618.3
53	4.6	30	3581.6	6.8	2243.1	1338.6	639.8
54	4.5	29	3475.0	6.8	2158.2	1316.8	657.9
55	4.5	28	3365.4	6.8	2076.2	1289.2	672.5
56	4.4	27	3253.7	6.7	1997.0	1256.7	683.6
57	4.4	26	3140.3	6.7	1920.7	1219.6	691.3
58	4.4	25	3025.6	6.6	1847.0	1178.7	695.6
59	4.4	24	2910.1	6.5	1775.8	1134.3	696.4
60	4.4	23	2793.9	6.5	1707.0	1086.9	693.7
61	4.4	22	2677.2	6.4	1640.3	1036.9	687.6
62	4.4	21	2560.1	6.3	1575.4	984.7	678.0
63	4.5	20	2442.8	6.2	1512.2	930.6	664.9
64	4.5	19	2325.2	6.1	1450.4	874.8	648.4
65	4.6	18	2207.4	6.0	1389.7	817.7	628.5
66	4.7	17	2089.2	5.8	1329.8	759.4	605.1
67	4.8	16	1970.8	5.7	1270.4	700.4	578.3
68	4.9	15	1851.9	5.5	1211.1	640.8	548.2
69	5.0	14	1732.4	5.4	1151.6	580.9	514.8
70	5.1	13	1612.2	5.2	1091.3	520.9	478.1



Design Equations (Vector Analysis):

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

Maximum Active Pressure Resultant

$P_{A,\,max}$ 696.37 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 13.9 pcf

Design Shoring for an Equivalent Fluid Pressure: 25 pcf (Recommended)

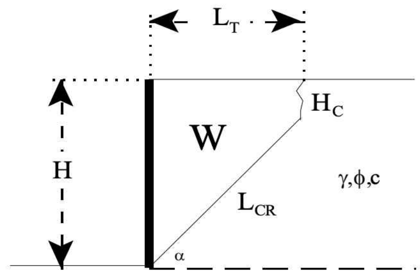
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Shoring Wall up to 20 ft

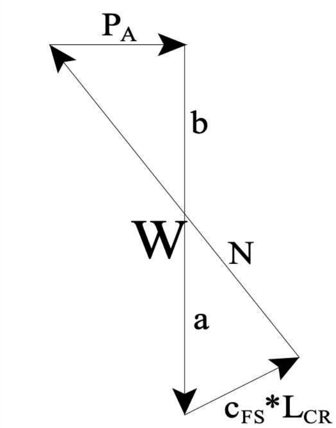
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	27.5	degrees
	(c_{FS})	160.0	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
45	5.6	185	22151.8	20.4	9623.8	12528.0	3961.4
46	5.4	179	21515.1	20.4	9087.8	12427.4	4169.5
47	5.2	174	20876.0	20.3	8596.1	12279.9	4359.9
48	5.0	169	20238.4	20.1	8144.6	12093.8	4533.0
49	4.9	163	19605.2	20.0	7729.5	11875.7	4689.2
50	4.8	158	18978.2	19.8	7347.0	11631.2	4829.0
51	4.7	153	18358.7	19.7	6994.1	11364.6	4952.6
52	4.6	148	17747.7	19.5	6667.8	11079.9	5060.4
53	4.6	143	17145.6	19.3	6365.5	10780.1	5152.7
54	4.5	138	16552.7	19.2	6084.9	10467.8	5229.8
55	4.5	133	15969.1	19.0	5824.0	10145.1	5291.8
56	4.4	128	15394.8	18.8	5580.8	9814.0	5339.0
57	4.4	124	14829.6	18.6	5353.7	9475.9	5371.5
58	4.4	119	14273.3	18.4	5141.1	9132.2	5389.3
59	4.4	114	13725.6	18.2	4941.8	8783.8	5392.6
60	4.4	110	13186.2	18.0	4754.4	8431.8	5381.4
61	4.4	105	12654.8	17.8	4577.8	8076.9	5355.5
62	4.4	101	12130.9	17.6	4411.1	7719.8	5315.0
63	4.5	97	11614.2	17.4	4253.2	7361.1	5259.7
64	4.5	93	11104.4	17.2	4103.2	7001.2	5189.5
65	4.6	88	10600.9	17.0	3960.4	6640.5	5104.1
66	4.7	84	10103.4	16.8	3823.9	6279.5	5003.3
67	4.8	80	9611.3	16.6	3692.9	5918.4	4886.9
68	4.9	76	9124.4	16.3	3566.7	5557.6	4754.5
69	5.0	72	8642.0	16.1	3444.6	5197.4	4605.8
70	5.1	68	8163.7	15.8	3325.8	4837.9	4440.4



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

$P_{A,\ max}$ 5392.64 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$
EFP 27.0 pcf

Design Shoring for an Equivalent Fluid Pressure: 27 pcf

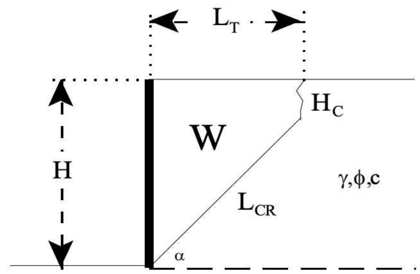
Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street
File No.: 21472
Description: Shoring Wall up to 30 ft

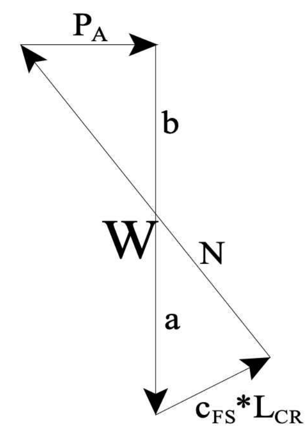
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	30.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	27.5	degrees
	(c_{FS})	160.0	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
45	5.6	435	52151.8	34.6	16283.9	35867.9	11341.4
46	5.4	421	50485.8	34.3	15293.0	35192.8	11807.4
47	5.2	407	48851.5	33.9	14398.5	34453.0	12232.2
48	5.0	394	47250.6	33.6	13588.2	33662.3	12617.3
49	4.9	381	45683.8	33.2	12851.9	32831.9	12963.9
50	4.8	368	44151.2	32.9	12180.7	31970.4	13273.3
51	4.7	355	42652.2	32.5	11567.2	31085.0	13546.4
52	4.6	343	41186.3	32.2	11004.9	30181.4	13784.3
53	4.6	331	39752.2	31.9	10488.0	29264.3	13987.8
54	4.5	320	38349.0	31.5	10011.7	28337.3	14157.4
55	4.5	308	36975.4	31.2	9571.8	27403.5	14293.9
56	4.4	297	35630.1	30.8	9164.6	26465.5	14397.7
57	4.4	286	34311.8	30.5	8786.7	25525.1	14469.1
58	4.4	275	33019.4	30.2	8435.3	24584.1	14508.3
59	4.4	265	31751.4	29.9	8107.7	23643.7	14515.5
60	4.4	254	30506.7	29.6	7801.8	22704.9	14490.8
61	4.4	244	29284.0	29.3	7515.4	21768.6	14434.0
62	4.4	234	28082.2	28.9	7246.7	20835.5	14345.0
63	4.5	224	26900.0	28.6	6994.1	19905.9	14223.4
64	4.5	214	25736.4	28.3	6756.0	18980.3	14068.8
65	4.6	205	24590.1	28.0	6531.1	18059.1	13880.7
66	4.7	196	23460.2	27.7	6317.9	17142.3	13658.5
67	4.8	186	22345.6	27.4	6115.4	16230.2	13401.5
68	4.9	177	21245.2	27.1	5922.4	15322.8	13108.6
69	5.0	168	20157.9	26.8	5737.7	14420.2	12779.0
70	5.1	159	19082.8	26.5	5560.3	13522.5	12411.5



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

$P_{A,\ max}$ 14515.54 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 32.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 33 pcf

Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo Street

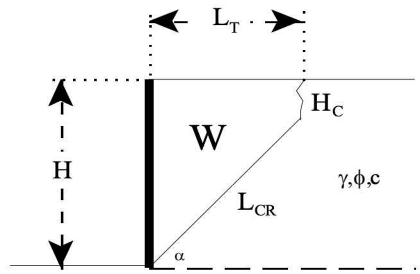
File No.: 21472

Description: Shoring Wall up to 40 ft

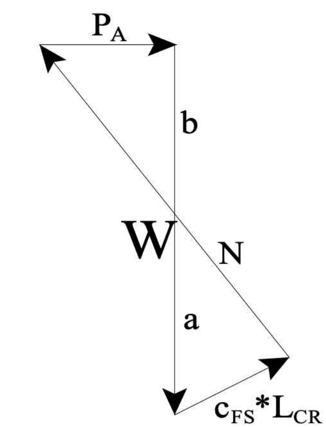
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	40.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	27.5	degrees
	(c_{FS})	160.0	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
45	5.6	785	94151.8	48.7	22944.0	71207.8	22515.9
46	5.4	759	91044.7	48.2	21498.3	69546.4	23333.3
47	5.2	733	88017.1	47.6	20200.9	67816.2	24077.5
48	5.0	709	85067.5	47.0	19031.8	66035.7	24751.4
49	4.9	685	82193.8	46.5	17974.3	64219.5	25357.5
50	4.8	662	79393.3	45.9	17014.5	62378.9	25898.1
51	4.7	639	76663.2	45.4	16140.4	60522.8	26375.1
52	4.6	617	74000.3	44.9	15341.9	58658.4	26790.2
53	4.6	595	71401.5	44.4	14610.4	56791.1	27145.1
54	4.5	574	68863.8	43.9	13938.5	54925.3	27441.0
55	4.5	553	66384.1	43.4	13319.7	53064.4	27678.9
56	4.4	533	63959.4	42.9	12748.4	51211.1	27859.7
57	4.4	513	61587.0	42.4	12219.7	49367.3	27984.1
58	4.4	494	59263.9	42.0	11729.4	47534.5	28052.4
59	4.4	475	56987.6	41.5	11273.7	45713.9	28065.1
60	4.4	456	54755.4	41.1	10849.2	43906.2	28022.0
61	4.4	438	52565.0	40.7	10453.0	42112.0	27923.0
62	4.4	420	50414.0	40.3	10082.4	40331.6	27767.8
63	4.5	403	48300.1	39.9	9735.1	38565.0	27555.9
64	4.5	385	46221.1	39.5	9408.8	36812.3	27286.4
65	4.6	368	44175.0	39.1	9101.7	35073.3	26958.4
66	4.7	351	42159.8	38.7	8812.0	33347.8	26570.7
67	4.8	335	40173.5	38.3	8537.9	31635.6	26121.9
68	4.9	318	38214.3	37.9	8278.0	29936.3	25610.4
69	5.0	302	36280.2	37.5	8030.8	28249.4	25034.3
70	5.1	286	34369.6	37.1	7794.8	26574.8	24391.4



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

$P_{A, max}$ 28065.07 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 35.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 36 pcf

Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo St.
File No.: 21472

Soil Weight	γ	120	pcf
Internal Friction Angle	ϕ	33	degrees
Cohesion	c	0	psf
Height of Retaining Wall	H	30	feet

NON-HYDROSTATIC (DRAINED) DESIGN

Restrained Retaining Wall Design based on At Rest Earth Pressure

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

$K_o = 1 - \sin\phi$	0.455	
$\sigma'_v = \gamma H$	3600.0	psf

$\sigma'_h =$	1639.3	psf	
EFP =	54.6	pcf	
$P_o =$	24589.5	lbs/ft	(based on a triangular distribution of pressure)

Design wall for an EFP of 55 pcf

Geotechnologies, Inc.

Project: Mayer Brown - 676 Mateo St.

File No.: 21472

Seismically Induced Lateral Soil Pressure on Retaining Wall

<u>Input:</u>

Height of Retaining Wall:	(H)	30.0	feet
Retained Soil Unit Weight:	(γ)	120.0	pcf
Peak Ground Acceleraction:	(PGA_M)	0.87	g
Horizontal Ground Acceleration:	(k_h)	0.29	g

Seismic Increment (ΔP_{AE}):

$k_h = 0.5*0.67*PGA_M$

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.75*k_h)$

$\Delta P_{AE} =$ 11844.4 lbs/ft

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

T = 10660.0 lbs/ft

$EFP = 2*T/H^2$

EFP = 23.7 pcf Recommended: **24 pcf**

triangular distribution of pressure, applied to the proposed retaining wall.

Geotechnologies, Inc.

Tiebacks Calculations

(Ref: Bowles, 1982)

Project: Mayrr Brown - 676 Mateo St.

File No. 21472

Soil Parameters:

Weight of Soil	γ	120.00	*lbs/ft³*
Friction Angle	ϕ	33.00	*degrees*
Cohesion	c	200.00	*lbs/ft²*
Tieback Angle	α	35.00	*degrees*

Design Assumptions:

Diameter of Grout	d	1.00	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	12.00	*feet*
Earth Pressure Coefficient	K	0.65	
Factor of Safety Applied	F.S.	1.50	

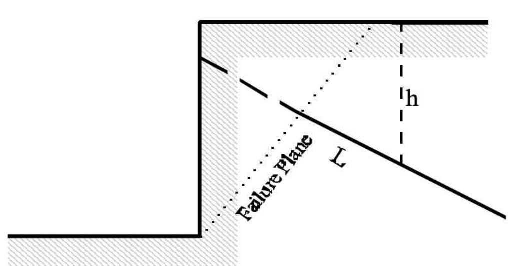
Ultimate Resistance: R_{ult} 56.93 *kips*

Eq: pi*d*γ*L*h*cos(a)*tan(ϕ)+c*pi*d*L

Allowable Resistance: $R_{allow} = R_{ult}$/F.S. 37.95 *kips*

Allowable Skin Friction: R_{allow}/2/pi/r/L 604.02 *psf*

Allowable Skin Friction Design Value **600** ***psf***



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Project: Mayer Brown - 676 Mateo St.
File No.: 21472

Settlement Calculation - Column Footing

Description: 18.5' x 18.5' Square Footing at Basement Level
Gridline:

Soil Unit Weight	120.0 pcf	Column Footing
Bearing Value	5000.0 psf	1,650 kips
Depth of Footing	32.0 feet	
Width of Footing	18.5 feet	

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

Depth Below Ground Surface (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
32.0													
	33.0	1.0	18.5	93%	4651	3960	8611	B1 @ 32.5'	3.15	1.60	1.55	2.0	0.37
34.0													
	35.0	3.0	6.2	79%	3963	4200	8163	B1 @ 35'	2.55	1.70	0.85	2.0	0.20
36.0													
	37.0	5.0	3.7	67%	3330	4440	7770	B1 @ 35'	2.45	1.75	0.70	2.0	0.17
38.0													
	39.0	7.0	2.6	56%	2798	4680	7478	B1 @ 35'	2.40	1.85	0.55	2.0	0.13
40.0													
	41.0	9.0	2.1	46%	2324	4920	7244	B1 @ 35'	2.35	1.90	0.45	2.0	0.11
42.0													
	43.0	11.0	1.7	38%	1897	5160	7057	B1 @ 42.5'	2.05	1.80	0.25	2.0	0.06
44.0													
	45.0	13.0	1.4	33%	1646	5400	7046	B1 @ 42.5'	2.00	1.85	0.15	2.0	0.04
46.0													
	47.0	15.0	1.2	29%	1429	5640	7069	B1 @ 42.5'	2.05	1.90	0.15	2.0	0.04
48.0													
	49.0	17.0	1.1	22%	1083	5880	6963	B1 @ 42.5'	2.00	1.95	0.05	2.0	0.01
50.0													
	51.0	19.0	1.0	19%	931.3	6120	7051	B2 @ 50'	0.90	0.85	0.05	2.0	0.01
52.0													
	58.0	26.0	0.7	13%	639.8	6960	7600	B2 @ 50'	0.92	0.90	0.02	14.0	0.03
54.0													
	57.0	25.0	0.7	13%	639.8	6840	7480	B2 @ 50'	0.95	0.90	0.05	6.0	0.04
60.0													
	63.0	31.0	0.6	7%	369	7560	7929	B2 @ 50'	0.98	0.96	0.02	6.0	0.01
66.0													
	69.0	37.0	0.5	7%	369	8280	8649	B2 @ 50'	1.01	1.00	0.01	6.0	0.01
72.0													

Settlement: 1.23
Reduction: 0.67

Total Settlement in inches: ***0.82***

CITY OF LOS ANGELES

CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK M. BUSH
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

March 26, 2019

LOG # 107422
SOILS/GEOLOGY FILE - 2

District Centre LP
350 S. Grand Ave, 25th flr.
Los Angeles, Ca 90071

TRACT:	WINGERTER TRACT (M R 15-52)
BLOCK:	---
LOT(S):	165-168, 182-185
LOCATION:	676 S. Mateo Street

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	21472	09/15/2017	Geotechnologies, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provide recommendations for the proposed 8-story mixed-use building over 3 levels of subterranean parking (11 levels total). The site is currently developed with a 1-story commercial building and a paved parking lot. Subsurface exploration performed by the consultant consisted of two hollow-stem auger borings to a maximum depth of 61 feet. The earth materials at the subsurface exploration locations consist of up to 3 feet of uncertified fill underlain by alluvium.

The consultants recommend to support the proposed structure(s) on conventional or mat-type foundations bearing on native undisturbed soils.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2017 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. The soils engineer shall review and approve the detailed plans prior to issuance of any permit. This approval shall be by signature on the plans that clearly indicates the soils engineer has reviewed the plans prepared by the design engineer; and, that the plans included the recommendations contained in their reports (7006.1).

2. All recommendations of the report(s) that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

3. This approval does not extend to the use of an on-site infiltration systems. If an on-site infiltration system is proposed, the consultant shall provide an evaluation on the items discussed in Information

Bulletin P/BC 2017-118 in a supplemental report with plans drawn to scale and suitable for reproduction and archiving purposes that clearly shows the location of the infiltration facility, all property lines, proposed and existing grades and structures, and the location of the proposed infiltration system. The plan shall be provided on the soils consultant's stationary or shall be signed and stamped by the soils engineer. Note: On-site infiltration systems are required to be a minimum of 10 feet (in any direction) from any foundation, and a minimum of 10 feet horizontally from private property lines.

4. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans (7006.1). Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit.

5. A grading permit shall be obtained for all structural fill and retaining wall backfill (106.1.2).

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density. Placement of gravel in lieu of compacted fill is only allowed if complying with LAMC Section 91.7011.3.

7. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill (1809.2, 7011.3).

8. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction (7013.12).

9. Grading shall be scheduled for completion prior to the start of the rainy season, or detailed temporary erosion control plans shall be filed in a manner satisfactory to the Grading Division of the Department and the Department of Public Works, Bureau of Engineering, B-Permit Section, for any grading work in excess of 200 cubic yards (7007.1).

   201 N. Figueroa Street 3rd Floor, LA (213) 482-7045

10. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the General Safety Orders of the California Department of Industrial Relations (3301.1).

11. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

12. Prior to the issuance of any permit that authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation (3307.1).

13. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit (3307.3.2).

14. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil

engineer shall submit a supplementary report with revised recommendations to the Department for approval.

15. Unsurcharged temporary excavation may be cut vertical up to 4 feet. Excavations over 4 feet shall be trimmed back at a uniform gradient not exceeding 1:1, from top to bottom of excavation, as recommended.

16. Shoring shall be designed for the lateral earth pressures specified in the section titled "Shoring Design" starting on page 25 of the 09/15/2017 report; all surcharge loads shall be included into the design.

17. Shoring shall be designed for a maximum lateral deflection of 1 inch, provided there are no structures within a 1:1 plane projected up from the base of the excavation. Where a structure is within a 1:1 plane projected up from the base of the excavation, shoring shall be designed for a maximum lateral deflection of ½ inch, or to a lower deflection determined by the consultant that does not present any potential hazard to the adjacent structure.

18. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

19. All foundations shall derive entire support from native undisturbed soils, as recommended and approved by the soils engineer by inspection.

20. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4), ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top of the footing.

21. The seismic design shall be based on a Site Class D as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

22. Basement/Retaining walls shall be designed for the lateral earth pressures specified in the section titled "Retaining Walls" starting on page 19 of the 09/15/2017 report. Note: All surcharge loads shall be included into the design.

23. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions as specified on page 20 of the 09/15/2017 report (1803.5.12).

24. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted in a non-erosive device to the street in an acceptable manner (7013.11).

25. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

26. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector (108.9).

27. Basement walls and floors shall be waterproofed/damp-proofed with an LA City approved "Below-grade" waterproofing/damp-proofing material with a research report number (104.2.6).

28. Prefabricated drainage composites (Miradrain, Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

29. All roof, pad and deck drainage shall be conducted to the street in an acceptable manner in non-erosive devices or other approved location in a manner that is acceptable to the LADBS and the Department of Public Works[; water shall not be dispersed on to descending slopes without specific approval from the Grading Division and the consulting geologist and soils engineer] (7013.10).

30. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS (7013.10).

31. The soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading (7008 & 1705.6).

32. Prior to pouring concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the work inspected meets the conditions of the report. No concrete shall be poured until the LADBS Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

33. Prior to excavation an initial inspection shall be called with the LADBS Inspector. During the initial inspection, the sequence of construction; shoring; pile installation; protection fences; and, dust and traffic control will be scheduled (108.9.1).

34. Installation of shoring and/or pile excavations shall be performed under the inspection and approval of the soils engineer and deputy grading inspector (1705.6, 1705.8).

35. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Inspector and the Contractor stating that the soil inspected meets the conditions of the report. No fill shall be placed until the LADBS Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included (7011.3).

36. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whichever is more restrictive.

DAN RYAN EVANGELISTA
Structural Engineering Associate II

DRE/dre
Log No. 107422
213-482-0480

cc: Geotechnologies, Inc., Project Consultant
LA District Office