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December 5, 2018
File Number 21491

Jamison Properties
3470 Wilshire Boulevard, Suite 700
Los Angeles, California 90010

Attention: Garrett Lee

Subject: Preliminary Geotechnical Engineering Investigation for Entitlement Process
Proposed "Central Plaza" Residential Development
3440 Wilshire Boulevard, Los Angeles, California

Ladies and Gentlemen:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides preliminary geotechnical recommendations for entitlement of the proposed development, 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.

This report is considered preliminary since the development is in the initial conceptual design phase. Limited exploration and research were performed as part of this preliminary report. Additional exploration will be necessary in order to achieve a thorough investigation of the site. A comprehensive report shall be prepared when the site is available for additional exploration and the development plan achieves more refinement.

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
FOR ENTITLEMENT PROCESS
PROPOSED "CENTRAL PLAZA" RESIDENTIAL DEVELOPMENT
3440 WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

INTRODUCTION

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

This report is considered preliminary since the development is in the initial conceptual design phase. Limited exploration and research were performed as part of this preliminary report. Additional exploration will be necessary in order to achieve a thorough investigation of the site. A comprehensive report shall be prepared when the site is available for additional exploration and the development plan achieves more refinement.

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The site is proposed to be developed with a new residential development, which will consist of two residential towers. The proposed towers will be 23 and 28 stories in height, with roof top amenities. The entire development will be constructed over 1 to 2 subterranean parking levels and 4 podium parking levels. Due to the gently sloping nature of the site, the proposed subterranean levels will extend between 5 to 20 feet below the existing site grade. Based on the preliminary plans, the lowest subterranean B2 Level will have a finished floor varying between 195.0 feet and 202.5 feet above Mean Sea Level (MSL).

Preliminarily, column loads are estimated to be between 2,500 and 3,500 kips. Grading will consist of excavations between 15 and 30 feet in depth for the proposed subterranean parking levels and foundation elements.

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

SITE CONDITIONS

The property is located at 3440 Wilshire Boulevard, in the City of Los Angeles, California. The area of the proposed development is currently developed with an existing two-story parking structure, which will be demolished prior to construction of the proposed development. The neighboring development consists primarily of residential and commercial structures. Three 12-story office buildings (3440-3460 Wilshire Boulevard) exist north of the proposed development. Another 12-story office building (3470 Wilshire Boulevard) is located west of the northwest portion of the proposed development. An existing 3-story parking structure is located immediately west of the proposed development. The project site is also bounded by Mariposa Avenue to the east, by 7th Street to the south.

The overall topography surrounding the project site slopes very gently to the southwest, with approximately 27 feet of elevation change from the corner of Mariposa Avenue and 7th Street (with an approximate high elevation of 225.0 feet above MSL) to the corner of Irolo Street and 7th Street (with an approximate low elevation of 198.0 feet above MSL). Drainage across the site is by sheetflow to the city streets. The vegetation on the site consists of isolated trees.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between September 29, 2015, and September 30, 2015, by excavating 4 exploratory borings. The exploratory borings varied between 40 to 60 feet in depth below the existing site grade. The borings were excavated with the aid of a truck-mounted drilling machine, equipped with an automatic hammer, and using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-4.

Geologic Materials

Fill materials underlying the subject site consist of silty sands and sandy clays, which are yellowish to dark brown in color, slightly moist to moist, medium dense to dense, stiff, fine to coarse grained, with occasional gravel, and brick and concrete fragments. Fill thickness ranging from 5½ and 7½ feet was encountered during exploration.

Native soils consist of Older Alluvium, comprising of sandy to silty clays, and silty sands, which are olive brown to dark grayish brown in color, slightly moist to moist, stiff to very stiff, dense, fine grained.

The native soils are underlain by bedrock materials at depths between 27½ and 45 feet below site grades. The bedrock underlying the subject site consists of sandstone and siltstone of the Upper-Miocene Puente Formation. The rock is moderately bedded, gray to dark olive gray in color, moist, moderately hard to hard. More detailed soil profiles may be obtained from individual boring logs.

Groundwater

Seepage of water was encountered at depths between 22 and 26½ feet in Boring Number 1 and 2, respectively. However, groundwater was not encountered in Boring Number 3 and 4, which were excavated to depths of 40 feet below the existing site grade.

The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 20 feet below the existing site grade.

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

Caving

Caving could not be directly observed during exploration due to the type of excavation equipment utilized. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

Research

Available geotechnical reports for the site were reviewed during the preparation of this investigation. The following reports were obtained from the City of Los Angeles Records Division for review:

1. Preliminary Report, Foundation investigation, Proposed Commercial Buildings, Wilshire Boulevard and Mariposa Avenue, Los Angeles, California, for the Tishman Realty & Construction Company, by Dames and Moore Soil Engineers (DM), dated December 5, 1950;
2. Additional Foundation Exploration, Proposed Additional Parking Facilities, 3440 Wilshire Boulevard, for Tishman Realty and Construction Company, by Dames and Moore Soil Engineers, dated May 11, 1954;
3. Soil Engineering Report for Proposed Canopies and Sign Monuments, at 3450 Wilshire Boulevard, Los Angeles, California, by Earth Systems Consultants (ESC), dated March 13, 1992.

A preliminary report (Reference #1) was prepared by DM in 1950 for the development of a total of three 12-story buildings (office buildings north of the proposed development), two single-story buildings, and a two-level garage facility (existing parking structure). A total of eight boring logs were presented as part of the preliminary report. However, a plan showing the location of the borings could not be found during research of the City records. According to the boring logs, bedrock (shale) was encountered at depths approximately between 24 and 44 feet below the ground surface. Groundwater seepage was generally encountered above the native soils/bedrock contact. According to the report, drilled and belled caissons were recommended for the development. A bearing pressure of 12,000 psf was recommended for caissons embedded a minimum of 3 feet into the shale bedrock, and 16,000 psf was recommended for caissons embedded a minimum of 5 feet into the shale bedrock.

Reference #2 presented the results of additional borings drilled by DM for the development of the Additional Parking Facilities. Three additional 18-inch diameter bucket-auger borings were drilled as part of this additional investigation to determine the depths to the shale bedrock, and to confirm the strength of the bedrock. DM provided the same caisson design parameters for the parking structure as Reference #1.

Reference #3 was prepared by ESC in 1992, for the development of canopies and sign monuments along the north side of the property at Wilshire Boulevard. A total of five borings were excavated by ESC as part of their geotechnical investigation. The borings encountered bedrock at depths of 24 to 32 feet below the ground surface in Test Hole No. 1 through 3, but were not encountered in Test Hole No. 4 or 5, which were only excavated to a depth of 10 feet bgs. Groundwater was encountered at depths of 28 and 23 feet bgs, in Test Hole No. 2 and 3, respectively, but were not encountered in Test Hole No. 1 (which was excavated to a maximum depth of 30 feet), or Test Hole No. 4, or 5 (which were excavated to a maximum depth of 10 feet bgs).

The locations of the borings referenced in Reference No. 2 and 3 above are plotted on the enclosed Plot Plan. The borings logs for these prior borings are provided following the Exploration Logs (Plates A-1 through A-4) prepared by this firm. The referenced reports are presented at the end of this report for reference.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

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

REGIONAL FAULTING

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

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

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

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

Surface Rupture

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

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

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

Liquefaction

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

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

The project site is underlain by Older Alluvium and bedrock. Based on the dense nature of the underlying Older Alluvium and bedrock, the potential for liquefaction occurring at the site is considered to be remote.

Dynamic Dry Settlement

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

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

Tsunamis, Seiches and Flooding

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

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

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a breached upgradient reservoir.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

This report is considered preliminary since the development is in the initial schematic design phase. Additional exploration will be necessary in order to achieve a thorough investigation of the site. A comprehensive report should be prepared when the site is available for additional exploration and the development plan achieves more refinement.

Between 5½ and 7½ feet of existing fill materials was encountered during exploration at the site. Due to the variable nature and the varying depths of the existing fill materials, the existing fill materials are considered to be unsuitable for support of the proposed foundations, floor slabs, or additional fill.

Due to the gently sloping nature of the site, the proposed subterranean levels will extend between 5 to 20 feet below the existing site grade. Excavations between 15 and 30 feet in depth will be required for the proposed subterranean parking levels and foundation elements. Preliminarily, it is anticipated that excavation of the proposed subterranean levels will remove the existing fill materials and expose the underlying dense native soils and/or bedrock. The proposed towers may be supported on a mat foundation bearing in the underlying bedrock, and the podium parking structure may be supported on conventional foundations bearing in the underlying dense Older Alluvium.

Excavation of the proposed subterranean level will require shoring and dewatering measures to provide a stable and dry excavation due to the depth of the excavation, the presence of water seepage, and the proximity of adjacent structures.

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

SEISMIC DESIGN CONSIDERATIONS

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)	2.343g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.343g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.562g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.831g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.246g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.831g

FILL SOILS

The maximum depth of fill encountered on the site was 7½ feet. This material and any fill generated during demolition should be removed during the excavation of the subterranean levels and wasted from the site.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to (low, moderate, high) expansion range. The Expansion Index was found to be 54 for bulk samples remolded to 90 percent of the laboratory maximum density. Recommended reinforcing is noted in the "Foundation Design" and "Slabs-on-Grade" sections of this report.

WATER-SOLUBLE SULFATES

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

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

DEWATERING

Seepage of water was encountered at depths between 22 and 26½ feet in Boring Number 1 and 2, respectively. However, groundwater was not encountered in Boring Number 3 and 4, which were excavated to depths of 40 feet below the existing site grade.

The historically highest groundwater level was established by review of California Geological Survey Seismic Hazard Zone Report of the Hollywood Quadrangle. Review of this report indicates that the historically highest groundwater level is on the order of 20 feet below the existing site grade.

Due to the gently sloping nature of the site, the proposed subterranean levels will extend between 5 to 20 feet below the existing site grade. Based on the preliminary plans, the lowest subterranean B2 Level will have a finished floor varying between 195.0 feet and 202.5 feet above Mean Sea Level (MSL). Currently, the lowest finished floor level will not extend below the historically highest groundwater level, and therefore, a permanent dewatering system below the structure is not required.

The subterranean walls of the building should be designed with subdrainage devices to relieve hydrostatic pressure as recommended in the "Retaining Wall Drainage" section below.

METHANE ZONES

Based on review of the Navigate LA (*http://navigatela.lacity.org/NavigateLA/*) website, maintained by the City of Los Angeles, the subject property is located within a Methane Buffer Zone as designated by the City. A qualified methane consultant should be retained to consider the requirements and implications of the City's Methane (Buffer) Zone designation. A copy of the portion of the map covering the Project Site is included herein.

GRADING GUIDELINES

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

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.
- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

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

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

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

Compaction

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

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

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

Acceptable Materials

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

Imported materials should be free from chemical or organic substances which could 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 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Wet Soils

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

Pumping (yielding or vertical deflection) of the high-moisture content soils at the bottom of the excavation may occur during operation of heavy equipment. Where pumping is encountered, angular minimum ¾-inch gravel should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. It would likely be on the order of 1 to 2 feet thick.

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

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

It is recommended that the proposed towers be supported on a mat foundation bearing in the underlying bedrock, and the podium parking structure be supported on conventional foundations bearing in the underlying dense Older Alluvium.

Mat Foundation

The proposed towers will be constructed over 2 subterranean parking levels extending up to 20 feet below grade. Preliminarily, it is anticipated that the proposed towers will have an average bearing pressure between 5,000 and 6,000 pounds per square foot. 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 foundations, the average bearing pressure of 6,000 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 6,000 pounds per square foot, with locally higher pressures up to 10,000 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 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

where K = Reduced Subgrade Modulus
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

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

Conventional

The podium parking structure may be supported on conventional foundations bearing in the underlying dense Older Alluvium.

Continuous foundations may be designed for a bearing capacity of 2,500 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.

Column foundations may be designed for a bearing capacity of 3,000 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 200 per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

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

Miscellaneous Foundations

Foundations for small miscellaneous outlying structures, such as property line fence walls, planters, exterior canopies, and trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing in properly compacted fill and/or the native soils. Wall footings may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended.

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

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

Foundation Settlement

The majority of the foundation settlement is expected to occur on initial application of loading. Preliminarily, it is anticipated that total settlement between 2½ to 3 inches will occur below the more heavily loaded central core portions of the mat foundation beneath the tower. Settlement along the edges of the mat footing is estimated to be on the order of 1½ inches.

Total settlement of conventional spread footings bearing in the native soils is estimated to be on the order of 1 inch. Differential settlement of conventional spread footings is estimated to be ½ inch.

Additional exploration will be necessary in order to achieve a thorough investigation of the site. Additional settlement analyses will need to be performed when the project achieves more definition and structural loading conditions are available.

Foundation Observations

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

RETAINING WALL DESIGN

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

Height of Retaining Wall (feet)	Cantilever Retaining Wall Triangular Distribution of Active Earth Pressure (pcf)	Restrained Retaining Wall Triangular Distribution of At-Rest Earth Pressure (pcf)
10 feet	35 pcf	67 pcf
15 feet	42 pcf	67 pcf
20 feet	47 pcf	67 pcf

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

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

Dynamic (Seismic) Earth Pressure

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

Surcharge from Adjacent Structures

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

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

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

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

where:

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

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

Waterproofing

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

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

Retaining Wall Drainage

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

A compacted fill blanket or other seal shall be provided at the surface. Retaining walls may be backfilled with gravel adjacent to the wall to within 2 feet of the ground surface. The onsite earth materials are acceptable for use as retaining wall backfill as long as they are compacted to a minimum of 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 the most recent revision of ASTM D 1557.

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

Where retaining walls are to be constructed adjacent to property lines, there is usually not enough space for placement of a standard perforated pipe and gravel drainage system. Under these circumstances, 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.

<u>Retaining Wall Backfill</u>

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density obtainable by the latest revision of ASTM D1557. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

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

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Seepage of water was encountered at depths between 22 and 26½ feet in Boring Number 1 and 2, respectively. However, groundwater was not encountered in Boring Number 3 and 4, which were excavated to depths of 40 feet below the existing site grade.

The historically highest groundwater level is on the order of 20 feet below the existing site grade. Preliminarily, the proposed lowest subterranean level is to be serviced by the backdrainage system will extend to a maximum depth of 20 feet below the existing site grade. It is anticipated that the only water which could affect the proposed retaining walls would be irrigation waters and precipitation. Additionally the 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 20 gallons per minute may be assumed.

TEMPORARY EXCAVATIONS

It is anticipated that excavations on the order of 15 to 30 feet in vertical height will be required for the proposed subterranean levels and foundation elements. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged by adjacent traffic, public way, properties, or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be sloped back without shoring. Excavations over 5 feet in height should may be excavated at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 20 feet. A uniform sloped excavation does not have a vertical component.

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

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

Temporary Dewatering

Seepage of water was encountered at depths between 22 and 26½ feet in Boring Number 1 and 2, respectively. However, groundwater was not encountered in Boring Number 3 and 4, which were excavated to depths of 40 feet below the existing site grade.

Preliminarily, it is anticipated that the proposed subterranean levels and foundation elements will extend on the order of 15 to 30 feet below existing site grades. Continuous groundwater is not expected, however, finite zones of perched groundwater could be encountered locally.

Temporary dewatering should be installed as necessary. Temporary dewatering should consist of gravel-filled drainage trenches leading to a sump area. The collected water should be pumped to an acceptable disposal area. Where the exposed subgrade is wet pumping may be encountered. Under these conditions please refer to the “Wet Soils” section of this report.

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

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

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

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

Lagging

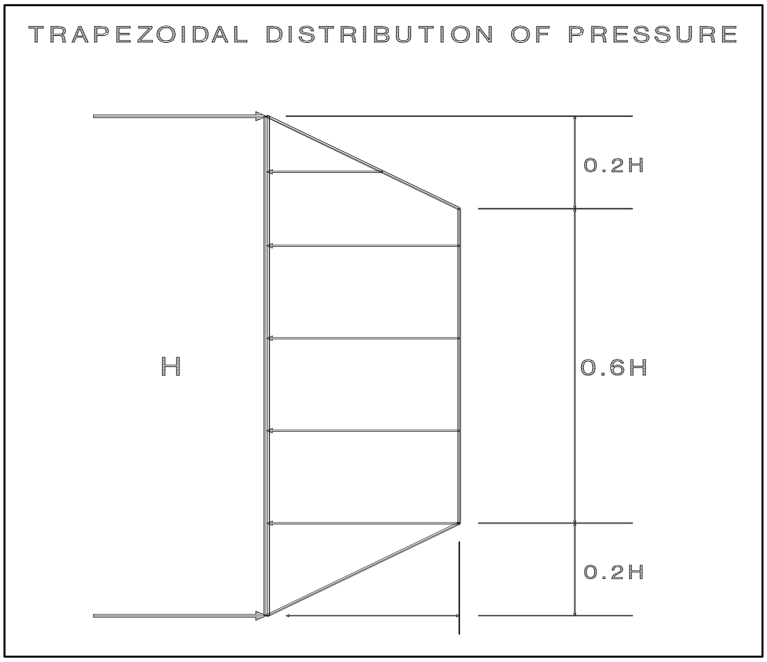
Soldier piles and anchors should be designed for the full anticipated pressures. Due to the cohesionless nature of the underlying earth materials, lagging will be required throughout the entire depth of the excavation. Due to arching in the geologic materials, the pressure on the lagging will be less.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

Lateral Pressures

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

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
20 feet	36 pcf	24H psf
25 feet	40 pcf	26H psf
30 feet	44 pcf	28H psf

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



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

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

Tied-Back Anchors

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

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

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

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

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

Anchor Installation

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

Deflection

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

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

Monitoring

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

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

Shoring Observations

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

SLABS ON GRADE

Concrete Slabs-on Grade

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

Outdoor concrete flatwork should be a minimum of 4 inches in thickness, and should be reinforced with a minimum of #3 steel bars on 18-inch centers each way. Outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

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

Concrete Crack Control

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

For standard control of concrete cracking, a maximum crack control joint spacing of 10 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.

PAVEMENTS

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

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

A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving. Concrete paving for passenger cars and moderate truck traffic shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. Concrete paving for heavy truck traffic shall be a minimum of 7½ inches in thickness, and shall be underlain by 6 inches of aggregate base. For standard crack control maximum expansion joint spacing of 10 feet should not be exceeded. Lesser spacing would provide greater crack control. Joints at curves and angle points are recommended.

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

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

Preliminarily, it is anticipated that the proposed subterranean levels and foundation elements will extend between 15 and 30 feet below the existing site grade.

Bedrock is anticipated between 22 and 32 feet below the existing site grade. By nature, the underlying bedrock is relatively impermeable, and therefore, it is the opinion of this firm that stormwater infiltration is not feasible for the project site.

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

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

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

GEOTECHNICAL TESTING

Classification and Sampling

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

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

Moisture and Density Relationships

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

Direct Shear Testing

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

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

Consolidation Testing

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

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

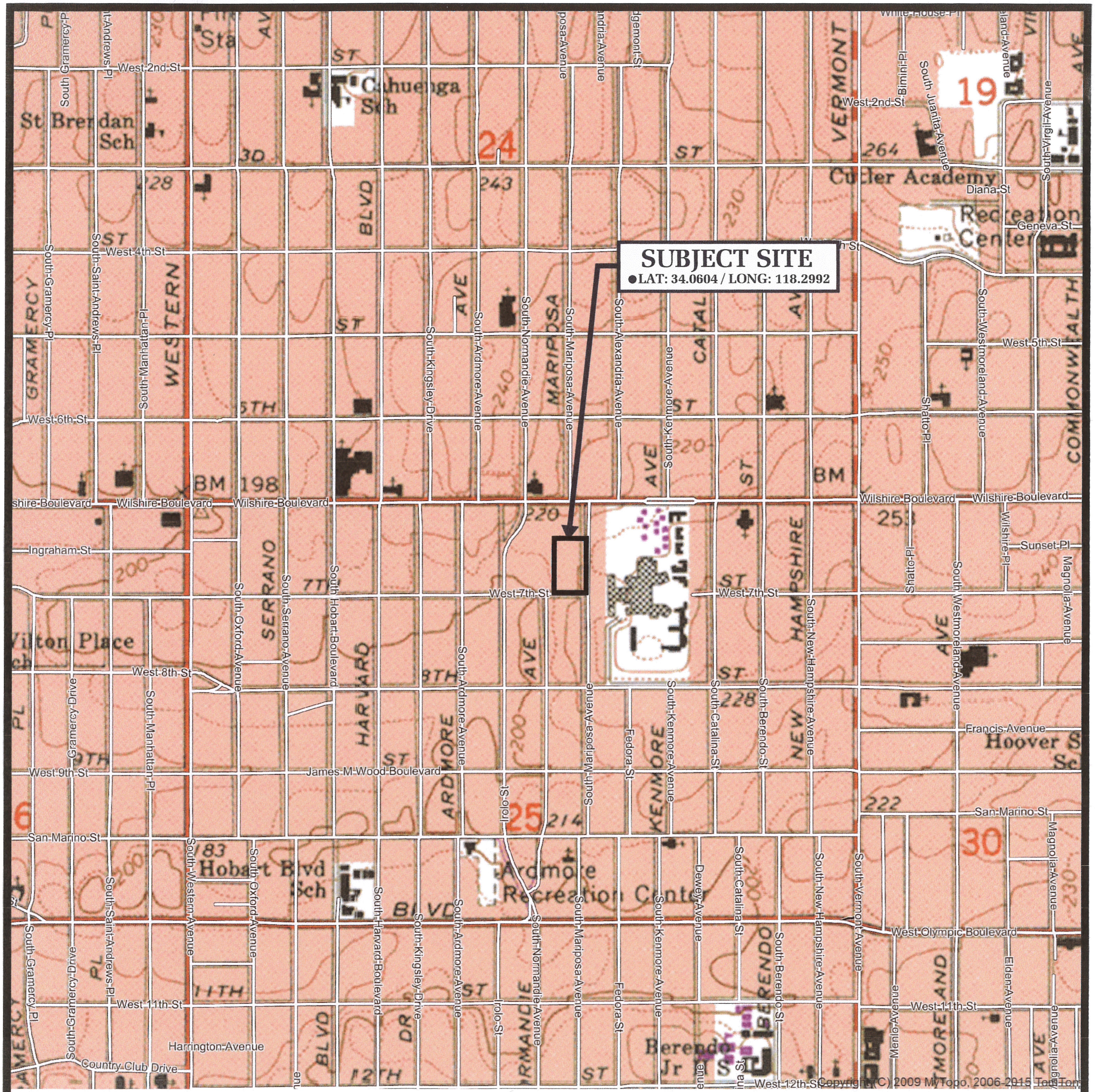
These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number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

REFERENCES

1. American Society of Civil Engineers, 1994, "Settlement Analysis," Technical Engineering and Design Guides, as adapted from the U.S. Army Corps of Engineers, No. 9.
2. Bartlett, S.F. and Youd, T.L., 1992, "Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads," Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, SUNY-Buffalo, Buffalo, NY.
3. Bartlett, S.F. and Youd, T.L., 1995, "Empirical Prediction of Liquefaction-Induced lateral Spread," Journal of Geotechnical Engineering, Vol. 121, No.4, April.
4. Bowles, Joseph E., 1977, "Foundation Analysis and Design," 2nd Edition, McGraw-Hill, New York.
5. California Division of Mines and Geology, 1997, Seismic Hazard Zone Map, map scale 1:24,000.
6. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Hollywood 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 026.
7. California Geological Survey, 2008, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California," CDMG Special Publication 117A.
8. City of Los Angeles, Department of Public Works, 2003, Methane and Methane Buffer Zones Map, Map Number A-20960.
9. Department of the Navy, NAVFAC Design Manual 7.1, 1982, "Soil Mechanics," Naval Facilities Engineering Command, May.
10. Department of the Navy, NAVFAC Design Manual 7.02, 1986, "Foundations and Earth Structures," Naval Facilities Engineering Command, September.
11. Dibblee, T.W., 1991, Geologic Map of the Hollywood and Burbank (South ½) 7.5-Minute Quadrangles, Map No DF-30, map scale 1: 24,000.
12. Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.
13. Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.
14. Southern California Earthquake Center, 1999, "Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Liquefaction in California," March.

REFERENCES - continued

15. Tinsley, J.C., Youd, T.L, Perkins, D.M., and Chen, A.T.F., 1985, Evaluating Liquefaction Potential: in Evaluating Earthquake Hazards in the Los Angeles Region-An earth Science Perspective, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.
16. Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.
17. Tokimatsu, K. and Seed, H. B., 1987, “Evaluation of Settlements in Sands Due to Earthquake Shaking,” Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August.
18. United States Geological Survey, 2011, U.S.G.S. Ground Motion Parameter Calculator (Version 5.0.9a). http://earthquake.usgs.gov/hazards/designmaps/.
19. Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, “Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement”, Journal of Geotechnical Engineering, Vol. 128, No. 12, December.



SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6

MILES

0 1000 2000 3000

FFFT

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

VICINITY MAP

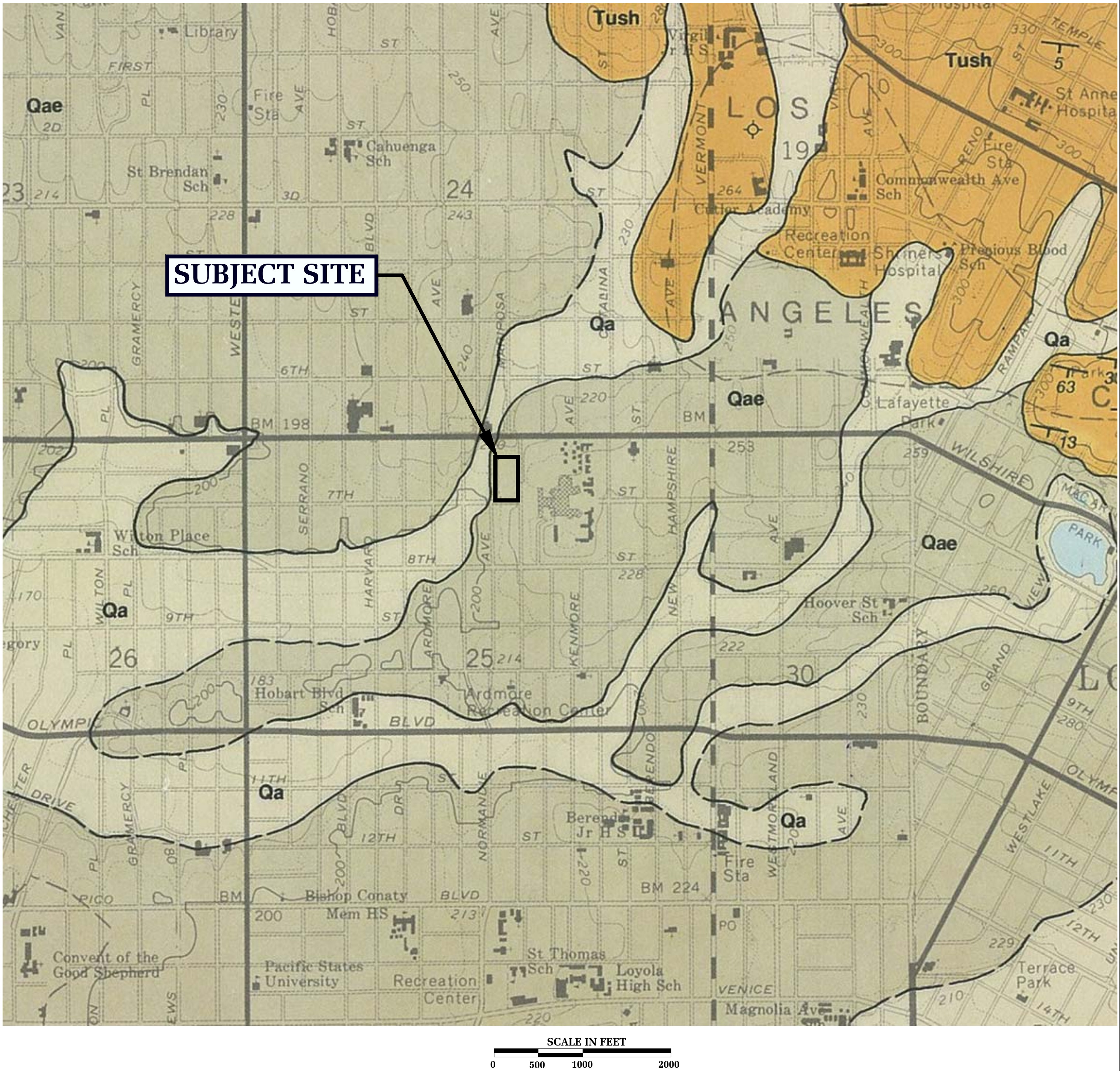
Geotechnologies, Inc.

Consulting Geotechnical Engineers

JAMISON PROPERTIES

3440 WILSHIRE BLVD., LOS ANGELES

FILE NO. 21491



LEGEND

Qa: Surficial Sediments - alluvium; gravel, sand and clay
Qae: Older Surficial Sediments - similiar to Qa but slightly elevated and dissected
Tush: Unnamed Shale - gray to light brown, thin-bedded silty clay shale, soft and crumbly

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

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE HOLLYWOOD & BURBANK (SOUTH HALF) QUADRANGLES (#DF-30)

LOCAL GEOLOGIC MAP - DIBBLEE

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JAMISON PROPERTIES

3440 WILSHIRE BLVD., LOS ANGELES

FILE NO. 21491

LIQUEFACTION AREA

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

SEISMIC HAZARD ZONE MAP

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JAMISON PROPERTIES
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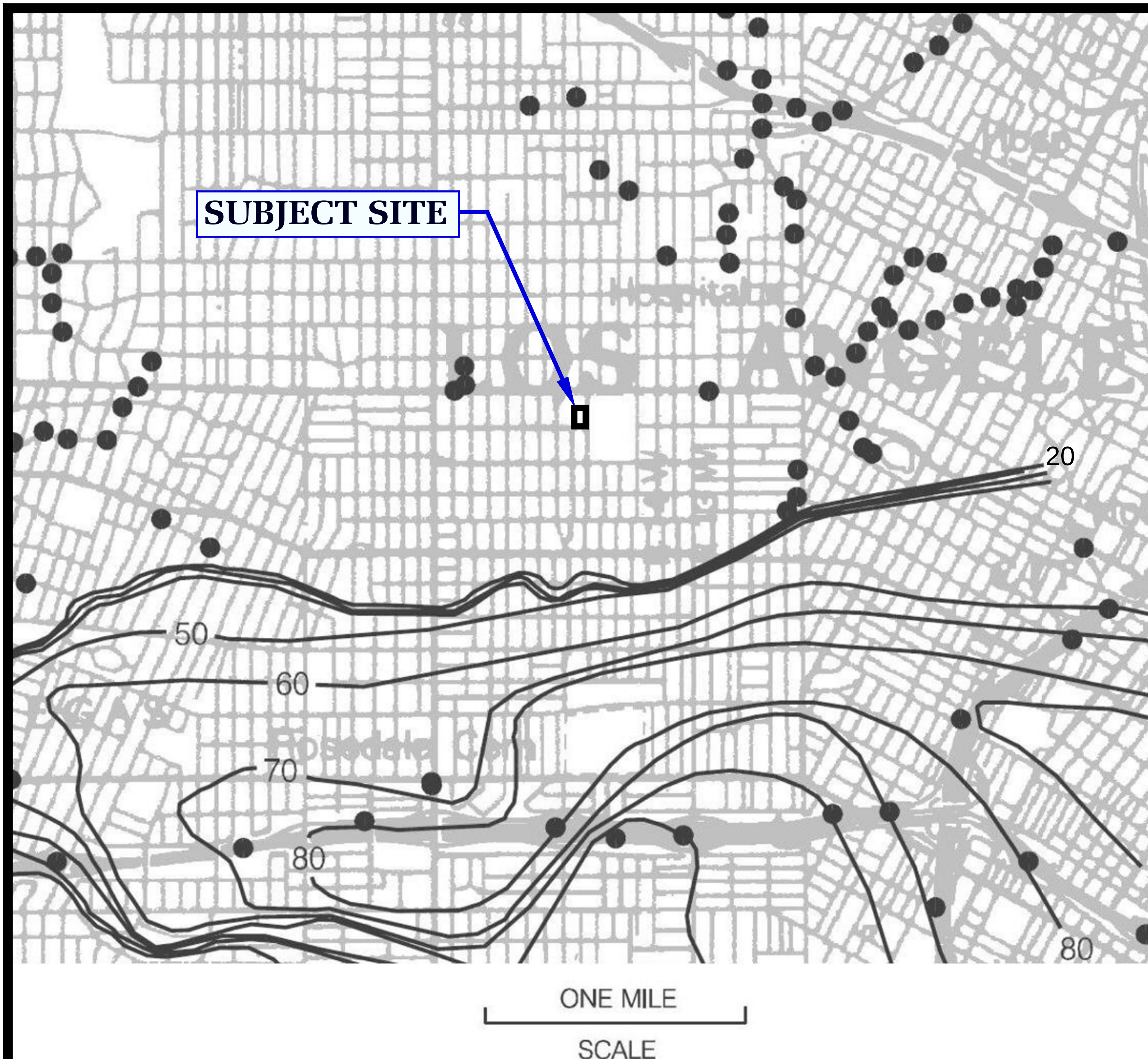
FILE NO. 21491



20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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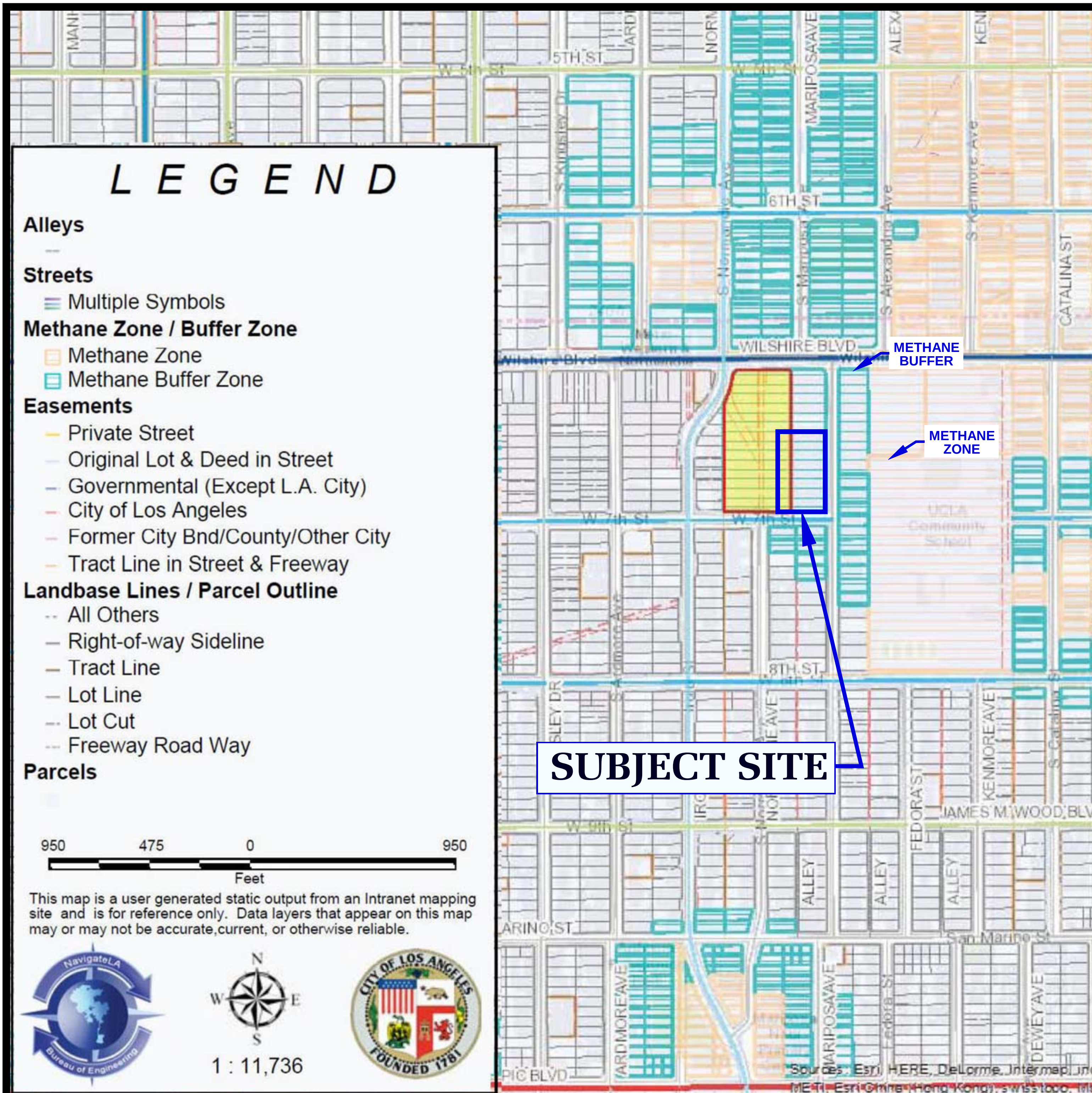
JAMISON PROPERTIES
3440 WILSHIRE BLVD., LOS ANGELES

FILE NO. 21491



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

METHANE ZONE RISK MAP

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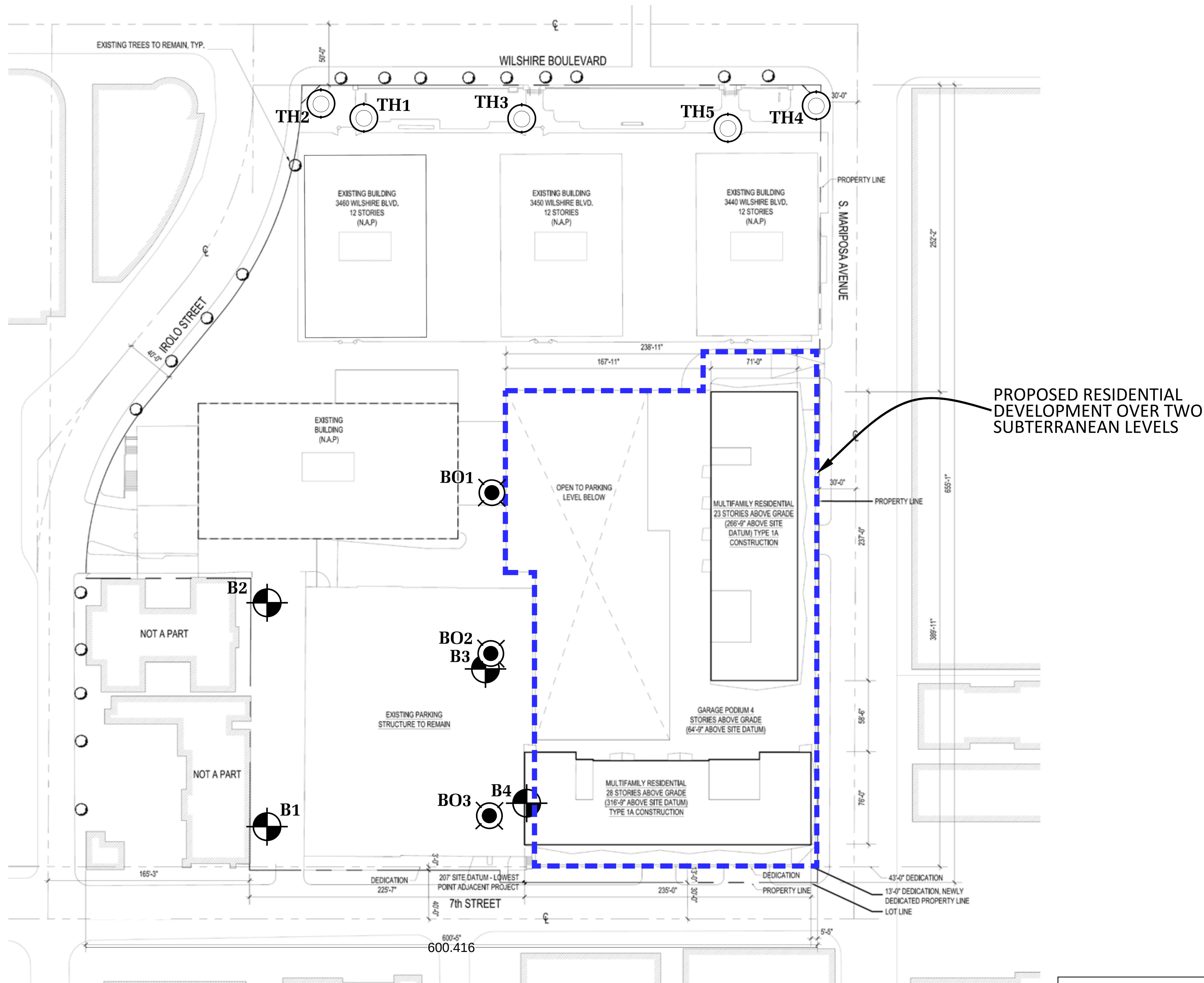
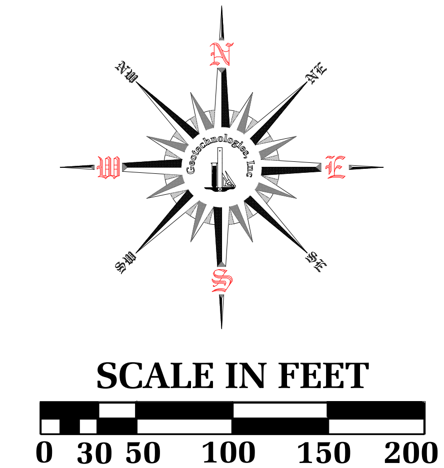
JAMISON PROPERTIES
3440 WILSHIRE BLVD., LOS ANGELES

FILE NO. 21491





LEGEND

B4 — LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION, FILE NO. 21051)

SUBJECT SITE

BO3 — LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION BY DAMES & MOORE, JOB NO. 979-0, DATED MAY 7, 1954)

TH5 — LOCATION & NUMBER OF TEST HOLES (PREVIOUS INVESTIGATION BY EARTH SYSTEMS CONSULTANTS, DATED MARCH 13, 1992)

REFERENCE: PLOT PLAN PROVIDED BY CALLISON RTKL, INC. DATED AUGUST 11, 2017

PLOT PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

JAMISON PROPERTIES
3440 WILSHIRE BLVD., LOS ANGELES

FILE No. 21491	DRAWN BY: TC
DATE: October 2017	

BORING LOG NUMBER 1

Jamison Properties **Date: 09/29/15**

File No. 21051 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 3-inch Base
				1 --		FILL: Silty Sand, dark brown to yellowish brown, slightly moist, medium dense to dense, fine to medium grained, occasional gravel, brick and concrete fragments
				2 --		
				3 --		
				4 --		
5	8	12.2	SPT	5 --		
				6 --		Clayey Sand to Sandy Clay, brown, slightly moist, loose to medium dense, medium firm, fine to coarse grained, slightly porous, with occasional gravel
				7 --		
7.5	20	12.1	107.6			
				8 --	SC	Clayey Sand, dark grayish brown, moist, dense, fine grained
				9 --		
10	14	15.7	SPT	10 --		
				11 --	CL	Sandy Clay, dark grayish brown, moist, medium firm to stiff, fine grained
				12 --		
12.5	15	23.0	96.0			
				13 --		
				14 --		
15	10	17.8	SPT	15 --		
				16 --		
				17 --		
17.5	11	24.3	95.3			
				18 --		
				19 --		
20	18	27.2	SPT	20 --		
				21 --		
				22 --		
22.5	22	23.3	98.1			
				23 --	SC	Clayey Sand, olive brown, wet, medium dense to dense, fine grained
				24 --		
25	20	24.5	SPT	25 --		
					CL	Sandy Clay, olive brown, wet, medium firm, fine grained

BORING LOG NUMBER 1

Jamison Properties

File No. 21051

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	39	17.5	107.6	-		
				28 --	SM	Silty Sand, olive brown, wet, dense, fine grained
				-		
				29 --		
				-		
30	23	19.1	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	22	30.4	95.9	-		
				33 --	CL	Sandy Clay, olive, very moist, stiff, fine grained
				-		
				34 --		
				-		
35	17	22.7	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	50	20.2	103.7	-		
	50/6"			38 --	SM	Silty Sand, dark olive brown, wet, very dense, fine to medium grained
				-		
				39 --		
				-		
40	34	20.3	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42.5	41	17.3	110.4	-		
	50/6"			43 --		
				-		
				44 --		
				-		
45	34	25.9	SPT	45 --		
				-		BEDROCK: Siltstone to Shale, olive gray to dark gray, moist,
				46 --		moderately hard
				-		
				47 --		
47.5	61	21.6	106.4	-		
				48 --		
				-		
				49 --		
				-		
50	35	21.8	SPT	50 --		
				-		

BORING LOG NUMBER 1

Jamison Properties

File No. 21051

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	48	17.2	111.2	-		
	50/3"			53 --		Sandstone to Siltstone, gray, moist, hard
				-		
				54 --		
				-		
55	47	20.8	SPT	55 --		
	50/3"			-		
				56 --		
				-		
				57 --		
57.5	100/7"	15.7	113.1	-		
				58 --		
				-		
				59 --		
				-		
60	49	21.7	SPT	60 --		
	50/5"			-		Total Depth 60 feet
				61 --		Water at 22 feet
				-		Fill to 7½ feet
				62 --		
				-		
				63 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				64 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				65 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				66 --		
				-		SPT=Standard Penetration Test
				67 --		
				-		
				68 --		
				-		
				69 --		
				-		
				70 --		
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 2

Jamison Properties **Date: 09/29/15**

File No. 21051 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Silty Sand, dark to yellowish brown, slightly moist, medium dense, fine to medium grained, with occasional gravel, brick and concrete fragments
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	22	13.8	111.0	5 --		
				-		
				6 --	SM	Silty Sand, dark brown, moist, medium dense to dense, fine to medium grained
				-		
				7 --		
7.5	11	10.8	103.5	-		
				8 --		
				-		
				9 --		
				-		
10	12	13.3	101.5	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	34	14.5	112.4	15 --		
				-	CL	Sandy Clay, dark brown, moist, very stiff, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	34	18.2	106.4	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	33	17.1	105.8	25 --		
				-	SC	Clayey Sand, dark brown, moist, dense, fine to medium grained

BORING LOG NUMBER 2

Jamison Properties

File No. 21051

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	18	35.7	91.0	30 --		
				-	CL	Sandy Clay, dark olive brown, very moist, very stiff, fine grained
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	53	20.3	104.9	35 --		
				-	SP	Sand, dark to yellowish brown, wet, very dense, fine to coarse
				36 --		grained
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	35	33.8	85.9	40 --		
				-		BEDROCK: Siltstone, olive gray, moist, moderately hard to hard
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	25	30.0	91.0	45 --		
	50/6"			-		
				46 --		
				-		NOTE: The stratification lines represent the approximate
				47 --		boundary between earth types; the transition may be gradual.
				-		
				48 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				49 --		Modified California Sampler used unless otherwise noted
				-		
50	20	32.1	89.8	50 --		
	50/5"			-		Total Depth 50 feet
						Water at 26½ feet
						Fill to 5½ feet

BORING LOG NUMBER 3

Jamison Properties **Date: 09/30/15**

File No. 21051 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Silty Sand, dark brown, slightly moist, medium dense, fine to medium grained, occasional gravel, slightly porous
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	11	14.1	110.5	5 --		
				-		
				6 --	CL	Sandy Clay, dark brown, moist, medium firm to stiff, fine to medium grained
				-		
				7 --		
7.5	14	12.9	114.0	-		
				8 --		
				-		
				9 --		
				-		
10	25	14.0	115.4	10 --		
				-		stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	58	14.5	114.2	15 --		
				-	SM	Silty Sand, dark to yellowish brown, moist, very dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	15	43.5	75.3	20 --		
				-	CL	Sandy Clay, grayish brown, moist to very moist, medium stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	15	37.0	77.7	25 --		
				-		

BORING LOG NUMBER 3

Jamison Properties

File No. 21051

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	60	34.0	85.5	30		
				31		BEDROCK: Shale, dark olive gray, moist, moderately hard to hard
				32		
				33		
				34		
35	29 50/5"	34.1	87.8	35		
				36		
				37		
				38		
				39		
40	31 50/6"	37.7	80.3	40		
				41		Total Depth 40 feet Water Seepage at 35 feet Fill to 5½ feet
				42		
				43		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				44		
				45		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 4

Jamison Properties | **Date: 09/30/15**

File No. 21051 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay, dark brown, moist, medium firm to stiff, fine to medium grained
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	7	10.5	SPT	5 --		
				-		
				6 --		
				-		
				7 --	SC	Clayey Sand, dark brown, moist, medium dense to dense, fine grained
7.5	16	23.8	95.9	-		
				8 --		
				-		
				9 --		
				-		
10	12	20.0	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	37	19.0	109.6	-		
				13 --		
				-		
				14 --		
				-		
15	11	22.1	SPT	15 --		
				-	CL	Sandy Clay, dark to yellowish brown, moist, stiff, fine grained
				16 --		
				-		
				17 --		
17.5	45	4.6	113.0	-		
				18 --	SM/SP	Silty Sand to Sand, yellowish brown, slightly moist, fine to coarse grained, with gravel
				-		
				19 --		
				-		
20	22	16.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	57	28.3	94.2	-		
				23 --	CL	Sandy Clay, olive brown to orange brown, moist, very stiff, fine grained
				-		
				24 --		
				-		
25	18	37.6	SPT	25 --		
				-		

BORING LOG NUMBER 4

Jamison Properties

File No. 21051

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
27.5	58	34.2	85.0	-		
				28 --		BEDROCK: Shale to Siltstone, olive gray, moist, moderately hard to hard
				-		
				29 --		
				-		
30	32	34.4	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	33	35.6	81.7	-		
	50/3"			33 --		
				-		
				34 --		
				-		
35	42	42.7	77.7	35 --		
	50/4"			-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	23	37.6	82.0	40 --		
	50/5"			-		Total Depth 40 feet
				41 --		No Water
				-		Fill to 6½ feet
				42 --		
				-		
				43 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual.
				44 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				45 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				46 --		
				-		SPT=Standard Penetration Test
				47 --		
				-		
				48 --		
				-		
				49 --		
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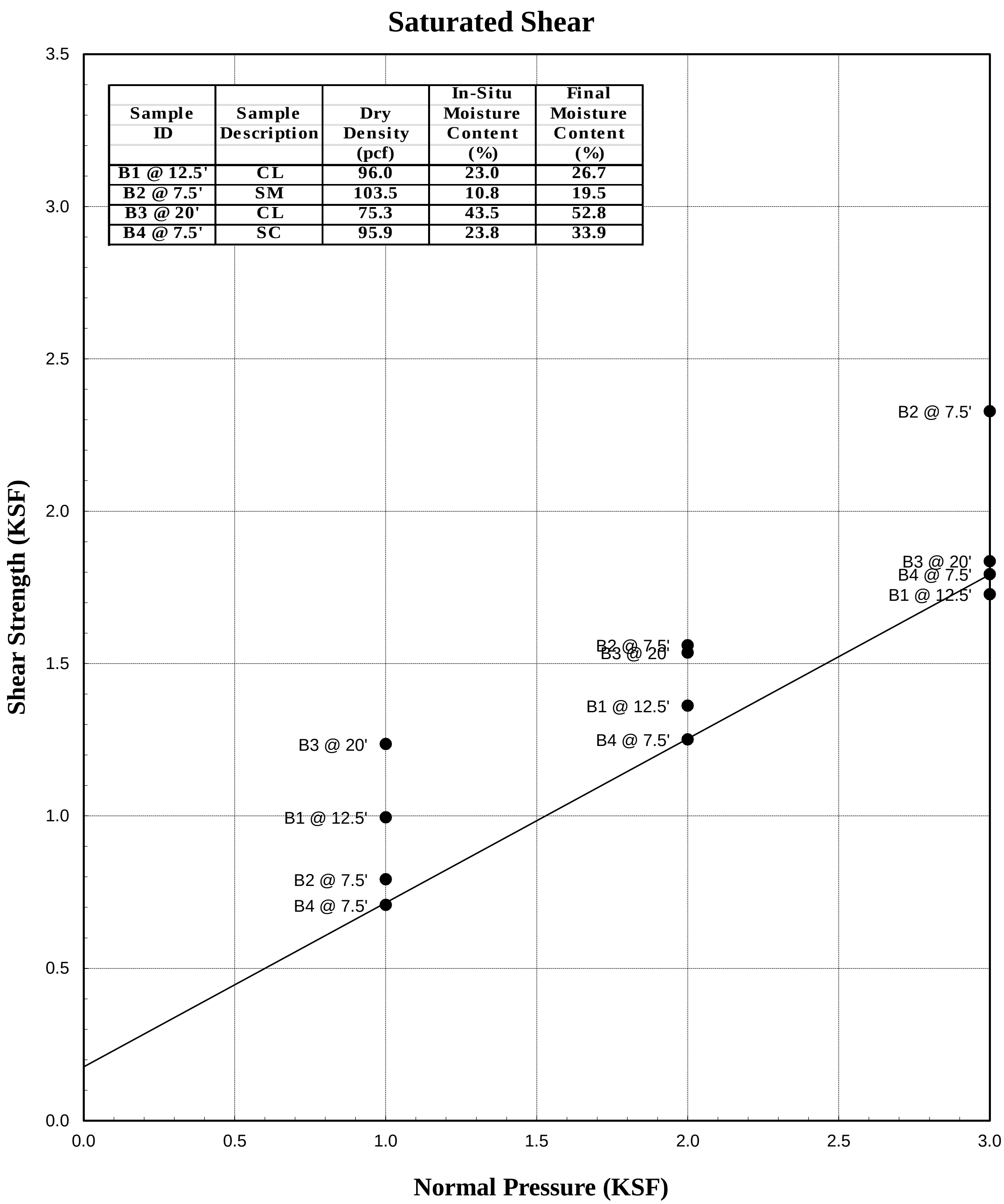
				50 --		
				-		



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 12.5'	CL	96.0	23.0	26.7
B2 @ 7.5'	SM	103.5	10.8	19.5
B3 @ 20'	CL	75.3	43.5	52.8
B4 @ 7.5'	SC	95.9	23.8	33.9

ϕ: 28.5 degrees
c: 185.0 psf

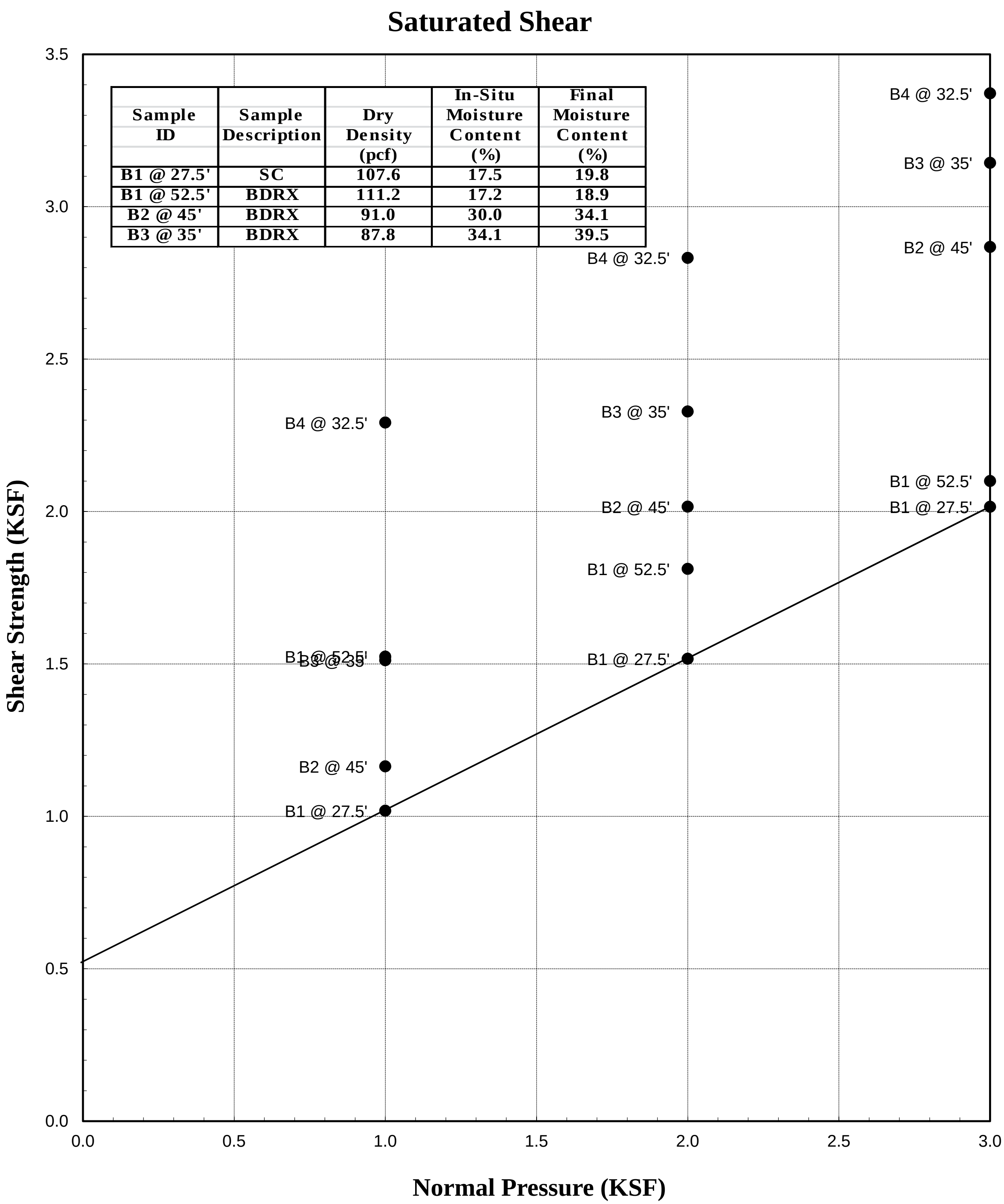
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JAMISON PROPERTIES

FILE NO.: 21491

PLATE: B-1



Sample ID	Sample Description	Dry Density (pcf)	In-Situ Moisture Content (%)	Final Moisture Content (%)
B1 @ 27.5'	SC	107.6	17.5	19.8
B1 @ 52.5'	BDRX	111.2	17.2	18.9
B2 @ 45'	BDRX	91.0	30.0	34.1
B3 @ 35'	BDRX	87.8	34.1	39.5

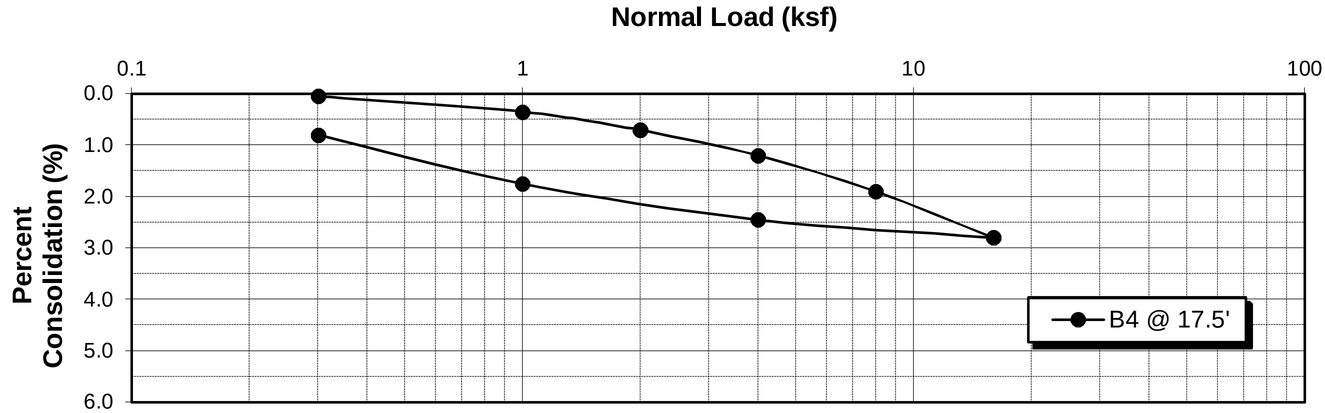
ϕ: 26.5 degrees
c: 520.0 psf

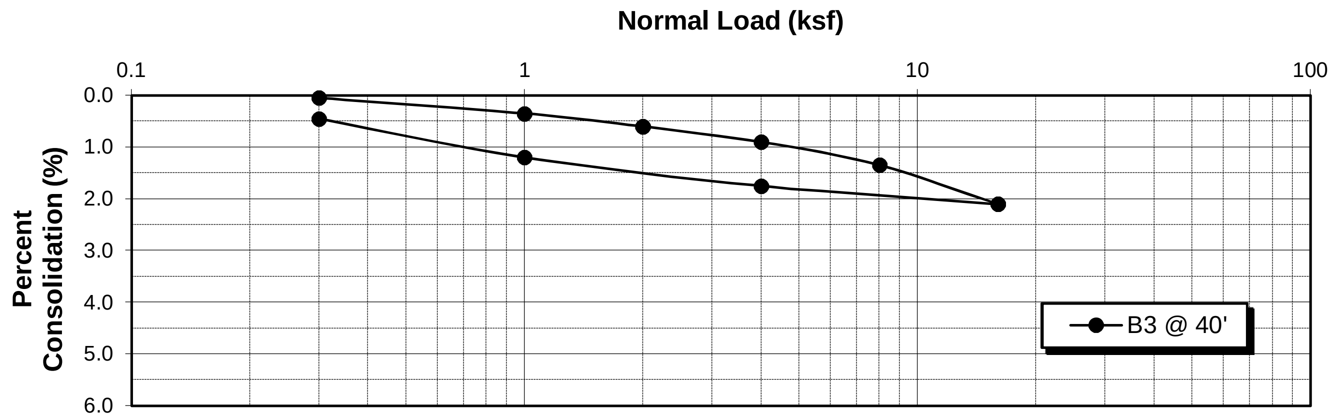
SHEAR TEST DIAGRAM

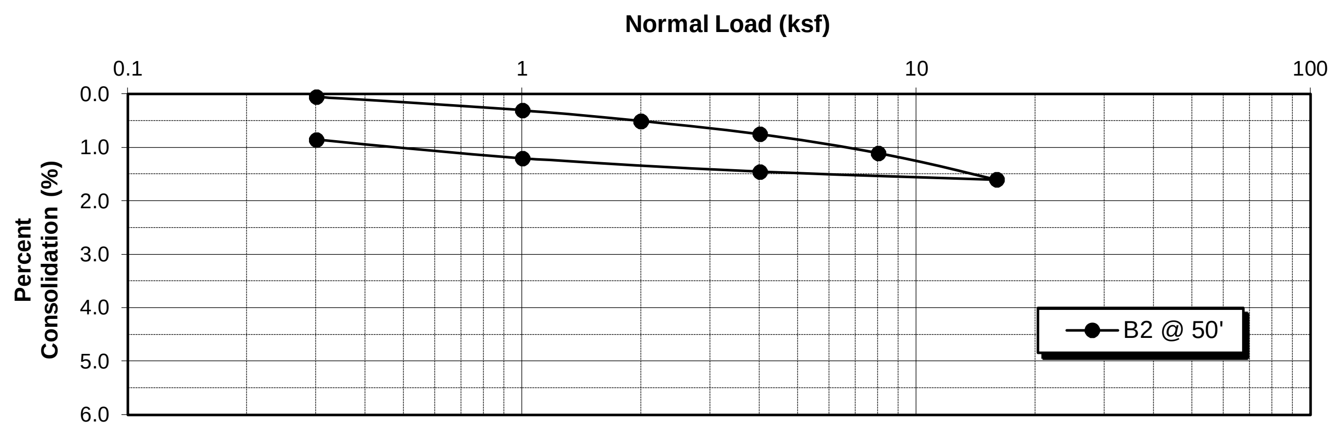
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: JAMISON PROPERTIES

FILE NO.: 21491 | PLATE: B-2







Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JAMISON PROPERTIES

FILE NO. 21491

PLATE: C-1

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

Jamison Properties
File No. 21491

COMPACTION/EXPANSION/SULFATE DATA SHEET

ASTM D-1557

Sample	B1 @ 1' – 5'
Soil Type	SM
Maximum Density (pcf)	131.0
Optimum Moisture Content (percent)	10.0
Percent finer than 0.005mm (percent)	<15%

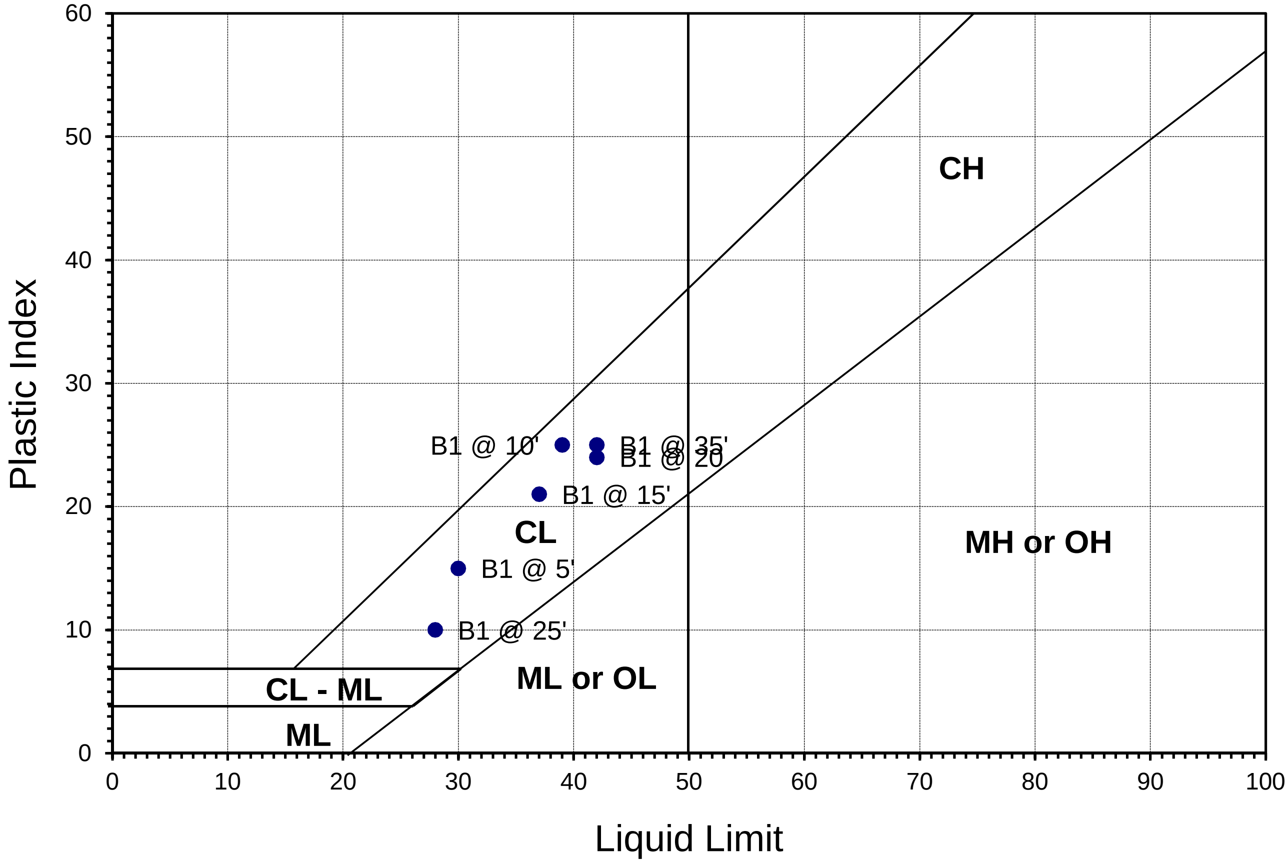
EXPANSION INDEX

Sample	B1 @ 1' – 5'
Soil Type	SM
Expansion Index – UBC Standard 18-2	54
Expansion Characteristic	Moderate

SULFATE CONTENT

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

PLATE D



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B1 @ 5'	CL	51.5	30.0	15.0	15.0
B1 @ 10'	CL	64.3	39.0	14.0	25.0
B1 @ 15'	CL	54.2	37.0	16.0	21.0
B1 @ 20	CL	80.0	42.0	18.0	24.0
B1 @ 25'	CL	57.6	28.0	18.0	10.0
B1 @ 30'	SM	24.4			
B1 @ 35'	CL	84.5	42.0	17.0	25.0

ATTERBERG LIMITS

Geotechnologies, Inc.
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: JAMISON PROPERTIES

FILE NO. 21491 | PLATE: F

[illegible] Civil Engineers
[illegible]
LOS ANGELES 17, CALIF.
L. LEROY CRANDALL

[illegible]

Brandow & Johnston, Structural Engineers
[illegible] South [illegible] Street
Los Angeles 17, California

Attention: Mr. Roy Johnston

Gentlemen:

Preliminary Report
Foundation Investigation
Proposed Commercial Buildings
Wilshire Boulevard and Mariposa Avenue
Los Angeles, California
For the Tishman Realty & Construction Company

3470

SCOPE

In accordance with your recent request, foundation recommendations resulting from our investigation of the site of the Proposed Commercial Buildings that are to be constructed for the Tishman Realty & Construction Company on Wilshire Boulevard near Mariposa Avenue are submitted herewith. The recommendations and data contained herein will be substantiated and presented in more complete form in our forthcoming formal report.

The ultimate development of the site will occupy an area of approximately 445 by 465 feet in plan, and will consist of three 12-story buildings, two single-story buildings and two-level garage facilities. We have previously investigated the sites of one of the 12-story buildings, and the adjoining single-story building and garage structures.* The development with which this investigation is concerned will consist of two 12-story buildings and one single-story structure, the locations of which are shown on Plate 1, Plot Plan. On the basis of our previous investigation, the structures then planned were designed to be supported on drilled-and-belled caissons that penetrate to firm shale that underlies the site. The present investigation was undertaken to determine whether

*See our "Report of Foundation Investigation Proposed Commercial Development, Wilshire Boulevard and Mariposa Avenue, Los Angeles, California, for the Tishman Realty & Construction Company" dated September 22, 195[illegible].

0330160017

[illegible] and a similar type of foundation would be employed for the structures presently planned.

[illegible] AND [illegible]

The site was explored by drilling a total of eight exploration borings with rotary bucket-type drilling equipment to depths varying from [illegible] to [illegible] feet below the ground surface. The locations at which the borings were drilled are shown on the Plot Plan. Graphical representations of the soils encountered in the exploration borings and the results of the shear tests and moisture and density determinations performed on selected samples are presented on Plates A-[illegible] through A-[illegible], Log of Borings. The nomenclature employed is described by the Soil Classification Chart and Key to Test Data on Plate A-[illegible].

The exploration borings indicated surface soils consisting of fill, alluvium, and weathered residual material. At depths varying from [illegible] to [illegible] feet below the ground surface (between Elevations [illegible] and [illegible]), a firm shale deposit was encountered. For convenience in reference, the upper surface of the firm shale is defined on the log of each boring and is also shown at the location of each boring on the Plot Plan. Perched water in appreciable quantity was observed to overlie the shale. In some of the borings, caving occurred in the soils overlying the shale. Notations regarding ground water and the caving of the borings are entered on the logs.

RECOMMENDATIONS

Drilled-and-belled caissons can be effectively employed at the site. However, because of the perched water and tendency of the soils to cave, it will be necessary to employ casing and to unwater some of the excavations before the installation of the concrete. These minor construction difficulties were discussed with Brandow & Johnston — it was concluded that these construction factors would not sufficiently hinder installation of foundations to justify the use of other foundation types.

While the shale is capable of providing high bearing values, the perched water has caused the upper few feet of shale to become somewhat weaker than the underlying materials. Accordingly, a minimum penetration of three feet into the shale is advised for all caissons. At this minimum depth, a bearing value of 12,000 pounds per square foot may be employed. If the penetration into the shale is increased to a depth of five feet, a bearing value of 16,000 pounds per square foot may be employed. These bearing values apply to the total of all design loads, dead, live, and seismic.

Caissons penetrating the minimum depth of three feet into the shale would be capable of resisting the 250,000-pound upward load, provided

03301600179

that the bottom of the column is at least twelve feet below the bottom of the floor slab. A minimum total penetration of eight feet below the bottom of the floor slab be required for columns penetrating five feet into the shale.

The ultimate settlements of the 13-story buildings supported on the shale will be approximately one-half inch. Ultimate settlements of the foundations of the eight-story structures will be approximately one-fourth inch. It is anticipated that the greater portion of the settlements will occur within six months after structural loads are applied.

Yours very truly,

DAMES & MOORE

By L. LeRoy Crandall
L. LeRoy Crandall

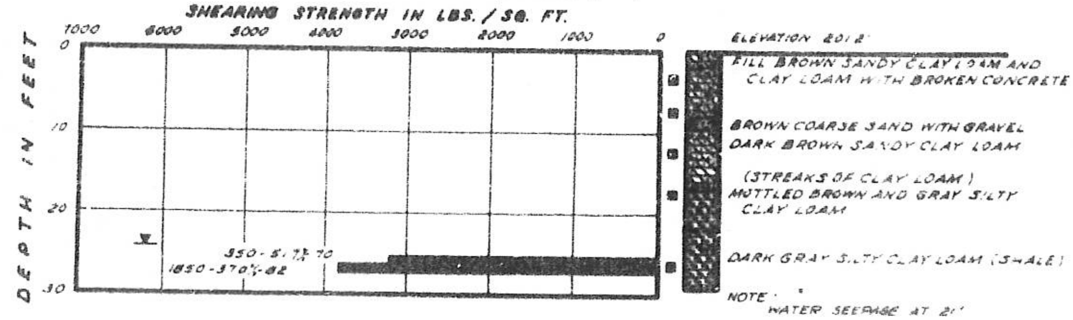
LLC-WDC RB

(3 copies submitted)
Attachments (4)
Via Messenger

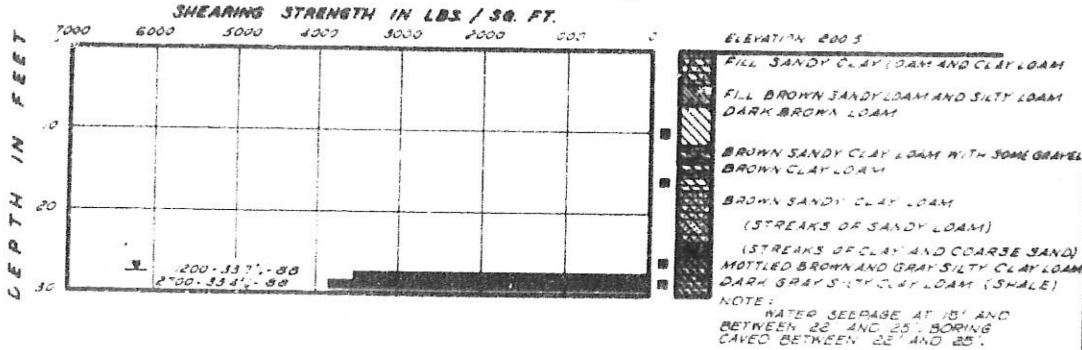
cc Claud Beelman, Architect (2)

03301600180

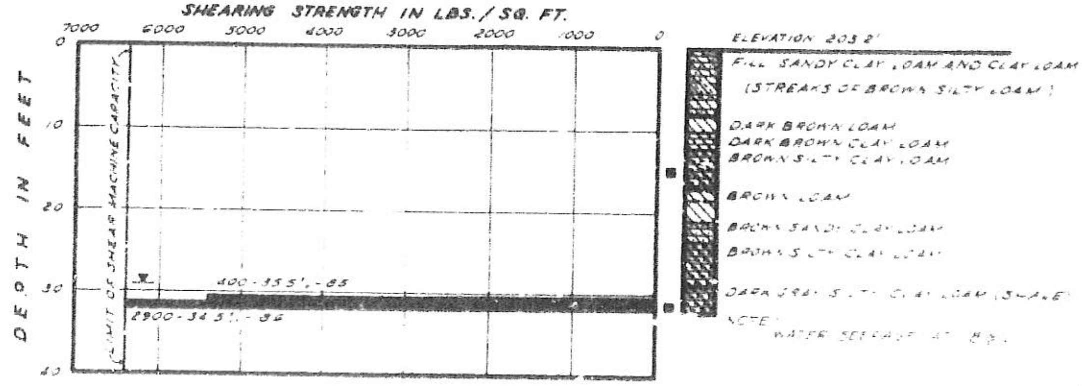
BORING 1



BORING 2



BORING 3

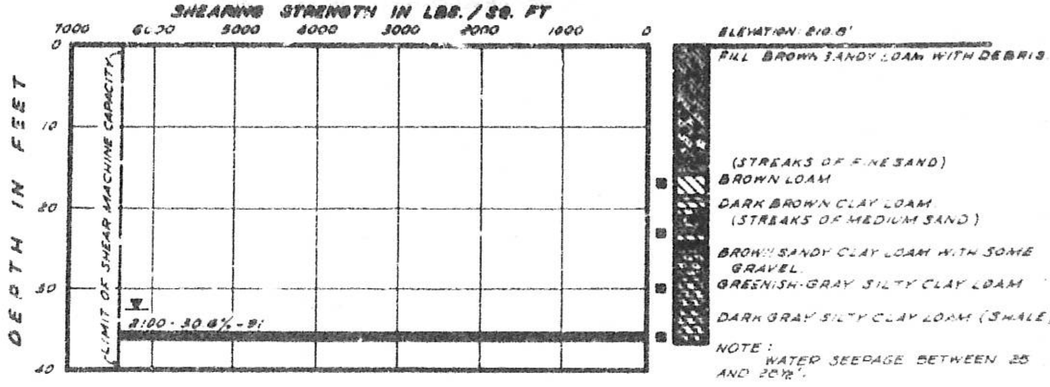


LOG OF BORINGS

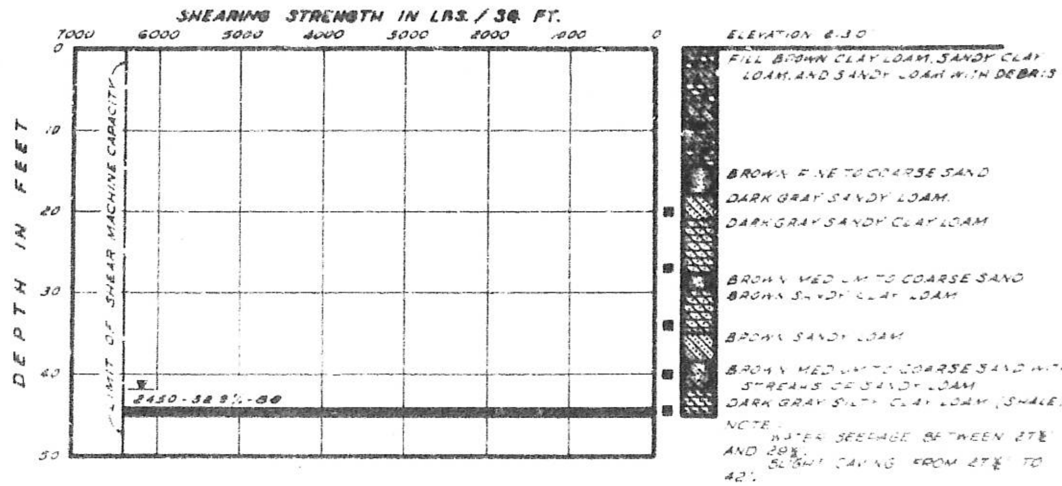
DAMES & MOORE
FOUNDATION ENGINEERS

PLATE A-1A

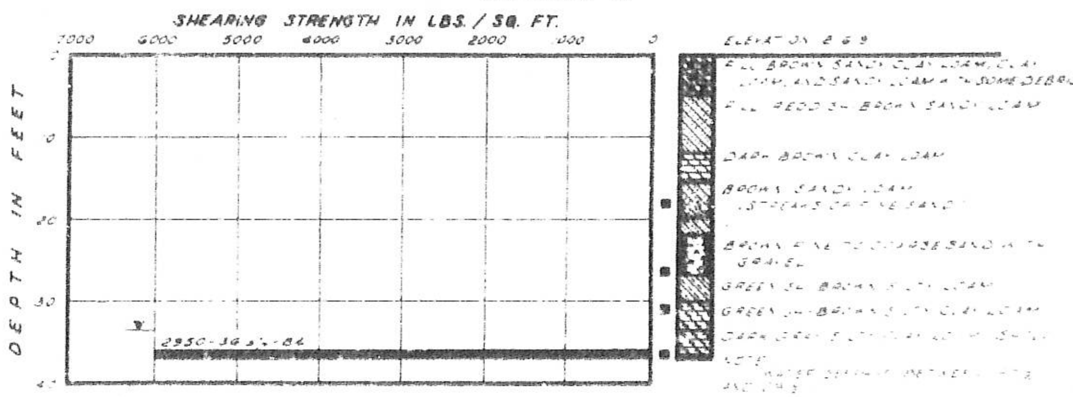
BORING 4



BORING 5



BORING 6

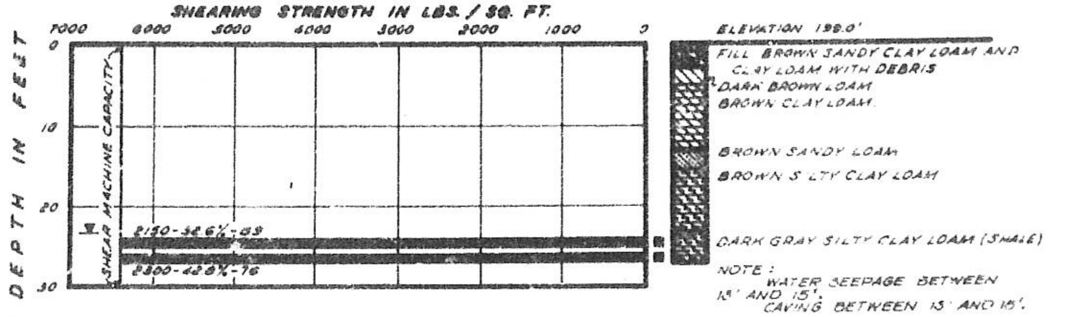


LOG OF BORINGS

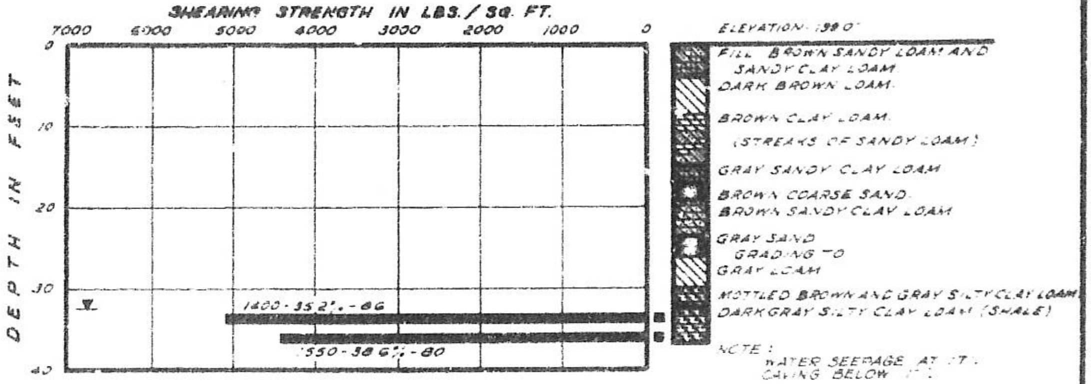
DAMES & MOORE
FOUNDATION ENGINEERS

PLATE A-1B

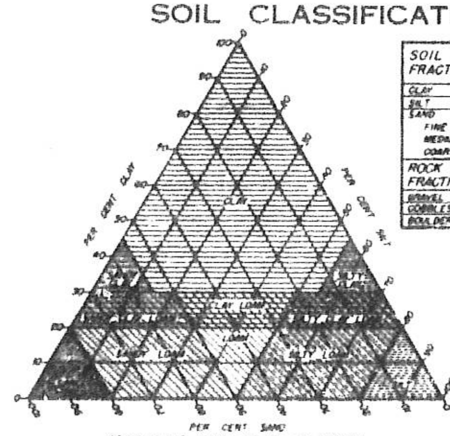
BORING 7



BORING 8



SOIL CLASSIFICATION CHART



SOIL FRACTION	PARTICLE SIZE			
	LOWER LIMIT		UPPER LIMIT	
	MM.	INCHES	MM.	INCHES
CLAY			.005	.0002
SILT	.005	.0002	.05	.002
SAND FINE	.05	.002	.25	.01
MEDIUM	.25	.01	.5	.02
COARSE	.5	.02	2.0	.08
ROCK FRACTION	THESE FRACTIONS ARE NOT CONSIDERED IN DETERMINING SOIL CLASSIFICATION.			
GRAVEL	2.0	.08	64	2.5
COBBLES	64	2.5	256	10
BOULDERS	256	10		

KEY TO TEST DATA

TESTS AT FIELD MOISTURE

2500 — 20.4% — 104 — TEST SURCHARGE PRESSURE IN POUNDS PER SQUARE FOOT; PER CENT FIELD MOISTURE EXPRESSED AS A PERCENTAGE OF THE DRY WEIGHT OF SOIL; DRY DENSITY EXPRESSED IN POUNDS PER CUBIC FOOT

— SHEARING STRENGTH IN POUNDS PER SQUARE FOOT

■ INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED

▼ INDICATES UPPER SURFACE OF FIRM SHALE

LOG OF BORINGS

DAMES & MOORE
FOUNDATION ENGINEERS

PLATE A-10

[illegible]

SOIL MECHANICS ENGINEERS

[illegible] WEST THIRD STREET · LOS ANGELES [illegible] CALIFORNIA · [illegible]

RESIDENT PARTNER, VERNON A. [illegible]

[illegible]

A. C. Martin & Associates
333 South Beaudry
Los Angeles 13, California

Attention: Mr. Norman Patten, Structural Engineer

Gentlemen:

Additional Foundation Explorations
Proposed Additional Parking Facilities
3440 Wilshire Boulevard
For Tishman Realty and Construction Company

This letter presents the results of exploration borings drilled for the Proposed Additional Parking Facilities to be constructed for the Tishman Realty and Construction Company at 3440 Wilshire Boulevard, Los Angeles, California. Several borings were previously drilled in a portion of this site during our foundation investigations for the Tishman Realty and Construction Company in 1950 and 1951*. These additional explorations consisted of three borings drilled in a portion of the site not previously explored. The purpose of these borings was to determine the depths to shale, and to establish whether the shale is similar in strength to the shale encountered in the previous investigations.

The borings were drilled with 18-inch-diameter bucket-type drilling equipment at the locations shown on Plate 1, Plot Plan. Continuous observations of the soils encountered in the borings were recorded by our job engineer. Logs showing the soils encountered in the exploration borings are presented on Plates 2-A through 2-C. The method used in classifying the soils is shown on the Soil Classification Chart on Plate 3. Undisturbed core samples were obtained from the borings for examination in the laboratory.

---

*See our reports dated September [illegible], 1950, and February 9, [illegible], entitled "Report of Foundation Investigation, Proposed [illegible] [illegible], Wilshire Boulevard and Mariposa Avenue, [illegible] [illegible]

03301500185

A. G. Smoots & Associates
May 11, 1954
Page Two

The area in which the borings were drilled was at one time crossed in an approximate north-south direction by a drainage channel. As discussed in our report dated February 8, 1951, alluvial soils have been deposited in the channel, and fills have been placed over the alluvium. The shale was encountered at elevations varying from 168.4 to 169.8 feet. In the old channel, perched water overlies the shale. The perched water has caused the upper few feet of shale to become somewhat weaker than the underlying shale and also weaker than the upper shales encountered in the exploration borings away from the old channel.

The caissons should be founded at least three feet into the shale. The caissons may be designed to impose bearing pressures of 12,000 pounds per square foot. If the caissons are drilled five feet into shale, a bearing value of 16,000 pounds per square foot may be used. These bearing values apply to the total of all design loads, dead, live, and seismic. These recommendations are similar to those given in our report dated February 8, 1951.

Yours very truly,

DAMES & MOORE

Vernon A Smoots

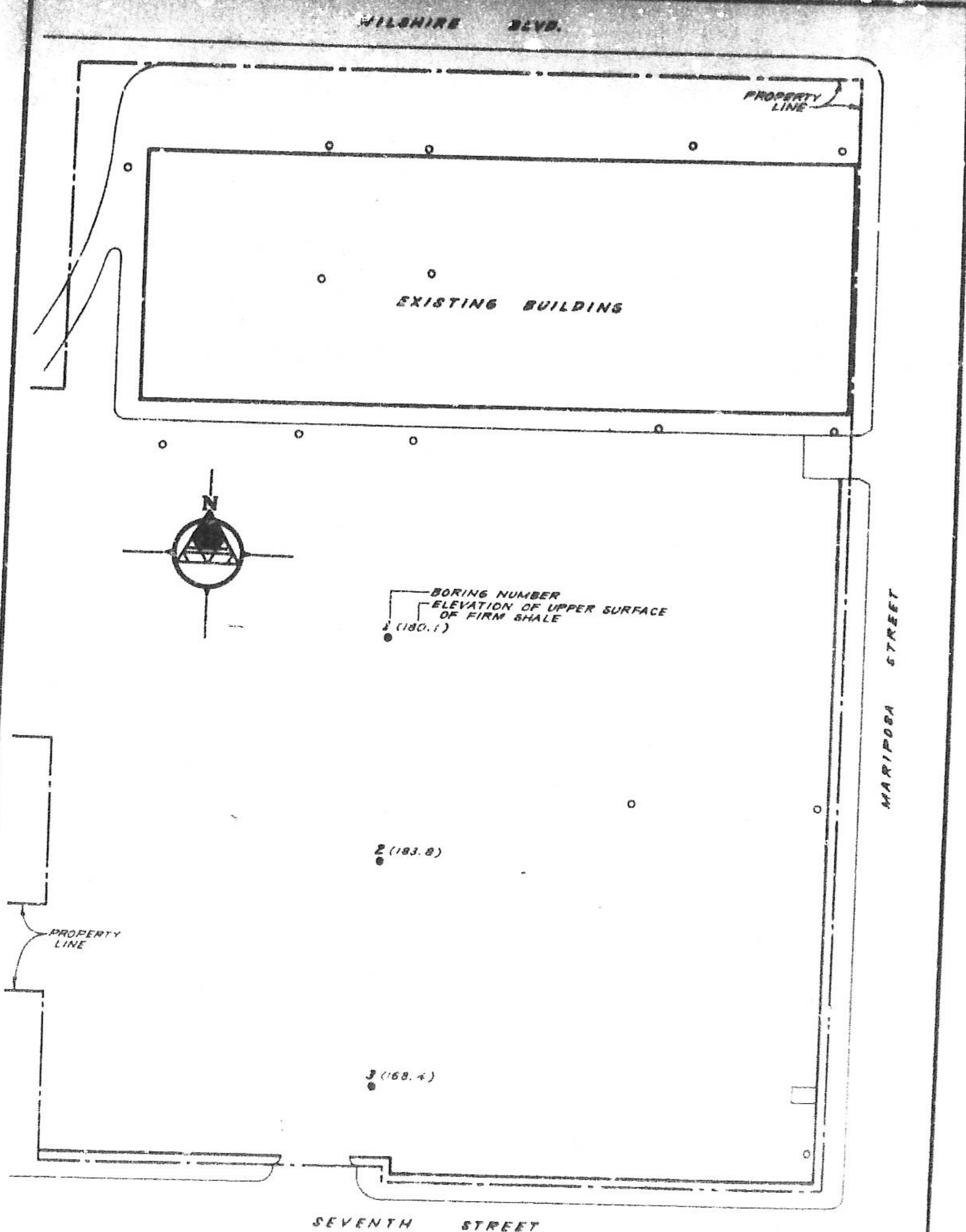
Vernon A. Smoots

VAS-LH MA

Attachments (5)

(3 copies submitted)

03301500186



KEY:
● BORINGS DRILLED IN CONJUNCTION WITH PRESENT REPORT.
○ BORINGS DRILLED IN CONJUNCTION WITH DAMES & MOORE INVESTIGATIONS, DATED SEPT. 22, 1950 AND FEB. 8, 1950.

REFERENCE:
DRAWING TITLED "PARKING EXPANSION STUDIES" BY ALBERT C. MARTIN & ASSOCIATES, ARCHITECTS AND ENGINEERS.

PLOT PLAN

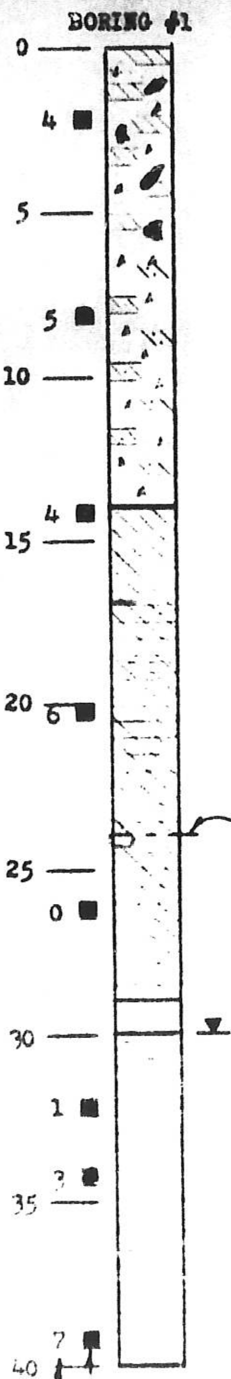
FEET
50 0 50 100

DAMES & MOORE
FOUNDATION INVESTIGATIONS · SOIL MECHANICS ENGINEERS

PLATE 1

DAMES & MOORE
JOB No. 979-0
LOCATION Los Angeles, Calif.
CLIENT Tishman Realty
DRAWN BY A.R. DATE 5-7-54

LOG OF BORINGS



ELEVATION 210.1'

Fill- Brown sand and gravel with patches of clay loam and pieces of concrete

Patches of brown sandy loam and sandy clay loam

Dark brown loam

Grading to clay loam

Grading to mottled brown silty clay loam

Water seepage

Grey & reddish brown sand

Greenish grey & grey stratifications of decomposed shale (Silty clay)

Grading to bluish grey & dark brown

Indicates depth at which undisturbed sample was extracted

Blows per foot to drive sampler

Depth in feet below ground surface

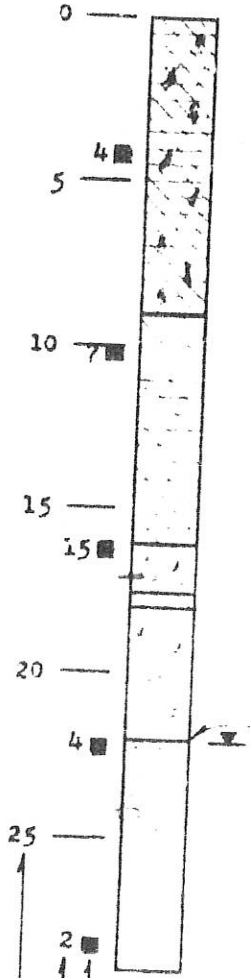
DAMES & MOORE

PLATE 2-A

03301600188

LOG OF BORINGS

BORING #2 ELEVATION 205.8'



Fill- Dark brown sandy clay loam with pieces of concrete and rock

Dark brown loam

Grading to sandy clay loam

Brown medium to coarse sand and gravel

Brown clay loam

Brown medium to coarse sand and gravel

Water seepage

Brown and grey stratifications of decomposed shale (Silty clay)

Indicates depth at which undisturbed sample was extracted

Blows per foot to drive sampler

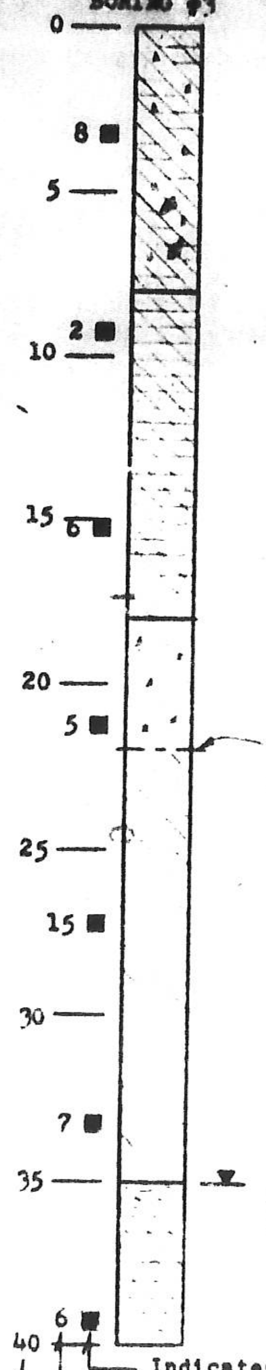
Depth in feet below ground surface

DAMES & MOORE

0330150018

LOG OF BORINGS

BORING #3 ELEVATION 203.4'



Fill- Patches of brown sandy loam and sandy clay loam with gravel

Concrete fragments

Dark brown sandy clay loam

(shale fragments)

Grading to mottled brown clay loam and sandy loam

Brown medium to coarse sand with gravel

water seepage

Grading to light brown sandy loam

Dark greyish brown silty clay loam (shale)

Indicates depth at which undisturbed sample was extracted

Blows per foot to drive sampler

Depth in feet below ground surface

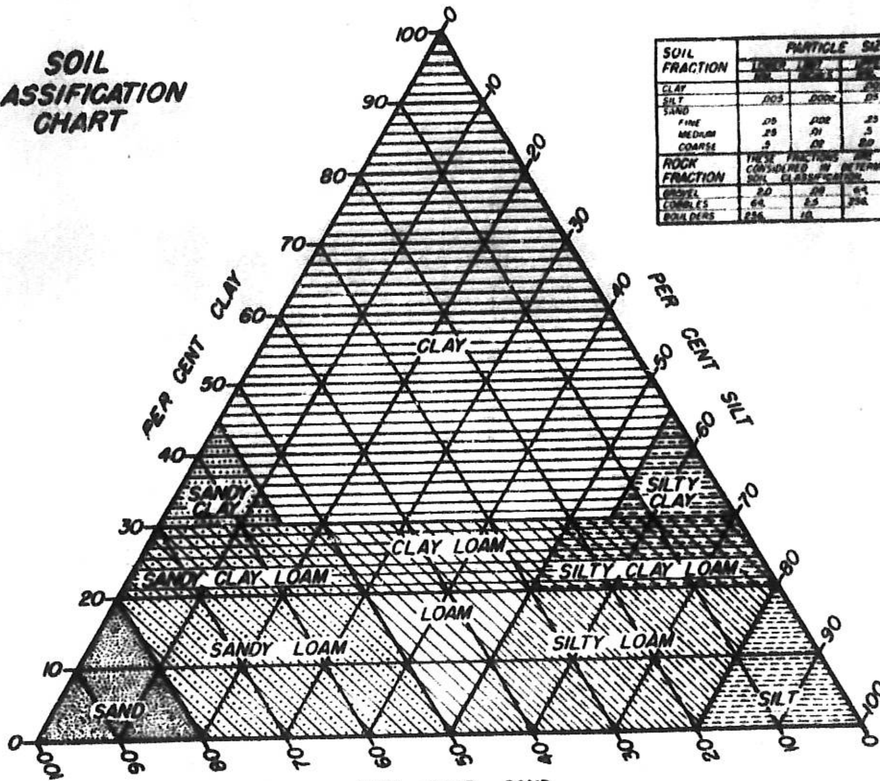
DAMES & MOORE

PLATE 2-C

03301600190

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

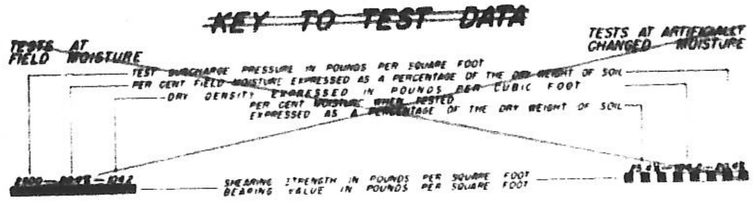
SOIL CLASSIFICATION CHART



(PERCENTAGES GIVEN ARE BY DRY WEIGHT)

SOIL FRACTION	PARTICLE SIZE			
	[illegible]	[illegible]	[illegible]	[illegible]
CLAY			.002	[illegible]
SILT	.005	.0002	.05	.002
SAND				
FINE	.05	.002	.25	.01
MEDIUM	.25	.01	.5	.02
COARSE	.5	.02	2.0	.08
ROCK FRACTION	THESE FRACTIONS ARE NOT CONSIDERED IN DETERMINING SOIL CLASSIFICATION			
GRAVEL	2.0	.08	64	2.5
COBBLES	64	2.5	256	10.
BOULDERS	256	10.		

KEY TO TEST DATA



■ INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED

0330160019 1

28161

Earth Systems Consultants
Southern California

Foundation Engineering Division
18344 Oxnard Street
Los Angeles, CA 91356
(818) 996-1600
(213) 873-5032
FAX (818) 996-8025

SOIL ENGINEERING REPORT

for

PROPOSED CANOPIES AND SIGN MONUMENTS

at

3450 Wilshire Boulevard
Los Angeles, California

REQUESTED BY

MR. DAVID WILLIS
ZUFU INVESTMENTS, INC.
dba
TOTAL PROPERTIES MANAGEMENT CO.
3450 WILSHIRE BOULEVARD, SUITE 400
LOS ANGELES, CALIFORNIA 90010

STRUCTURAL ENGINEER

MR. DON STRAND
BRANDOW & JOHNSTON ASSOCIATES
1660 West Third Street
Los Angeles, California 90017
(213) 484-8950

March 13, 1992

F-1537-

Earth Systems Consultants
Southern California

Foundation Engineering Division

18344 Oxnard Street
Los Angeles, CA 91356
(818) 996-1600
(213) 873-5032
FAX (818) 996-8025

SOIL ENGINEERING REPORT

Introduction

A soil engineering study has been conducted for three proposed canopies and two sign monuments to be located at 3450 Wilshire Boulevard, Los Angeles, California. The proposed canopies will be constructed in front of three high-rise structures which are addressed at 3440, 3450, and 3460 Wilshire Boulevard. The site is located at the southeast corner of the intersection of Wilshire Boulevard and Normandie Avenue.

The intent of this report is only to provide recommendations for foundation system of the proposed structures.

Surface Conditions

The vacant site, which is located between Wilshire Boulevard and the high-rise buildings, is approximately 50 feet wide and 420 feet long. Normandie and Mariposa Avenues border the site on the west and east, respectively.

Ground surface elevation of the site almost matches the sidewalk elevation along the adjoining street.

Apparently, the site was previously used as a planting area which has recently been cleared.

3450 Wilshire Boulevard
Los Angeles, California

The surficial soils consist of sandy clay and silty sand and are considered to be soft or loose. Apparently, most of the onsite soils consist of basement wall backfill of the high-rise structures. It is understood that the basement floors are at a depth of 16 feet.

Exploration

Five test holes were drilled to depths ranging between 10 and 35 feet near the locations of the proposed signs and canopies. All test holes were backfilled on completion of sampling. The approximate locations of the test holes are shown on the attached "Location of Test Holes" plan. The plan was prepared from available information, and our observations at the site. The plan should be considered as an approximate representation and is not intended to be used for construction.

Foundation Conditions

Our exploration revealed that the westerly and middle portions of the site are covered with deep (25± feet) deposits of soft, very moist fill. The fill was generally classified as sandy clay and clayey sand. The easterly part of the site, where observed, is covered by 1 to 2 feet of fill.

The fill is underlain by various types of natural soils. The natural soils were classified as sandy clay, clayey sand, sandy silt, silty sand, and sand. These soils are generally medium stiff or medium dense to stiff or dense.

Bedrock consisting of weathered mudstone and shale was encountered below the soils. The bedrock is considered to be dense.

As fill was encountered on a considerable portion of the site, it may be found at other locations in variable quantity and quality.

3450 Wilshire Boulevard
Los Angeles, California

Groundwater or perched water was encountered at depths of 28 and 23 feet in Test Holes Nos. 2 and 3, respectively. The groundwater may rise above during periods of prolonged rainfall, or from other causes not evident during the exploration. Although the proposed structures will not be directly affected by the groundwater, consideration should be given to the effect of groundwater during construction.

Description of the Proposed Construction

The proposed canopies to be placed in the middle and westerly portions of the site will be supported by friction piles. The easterly canopy will be supported by spread footings.

In addition to the canopies, two sign monuments will be placed at the easterly and westerly ends of the site. The westerly sign will also be supported by friction piles.

According to the project structural engineer, the load on isolated foundations is approximately 3 kips. Also, the proposed foundations will not be used to resist lateral loads as the structures will be tied to the existing foundations of the buildings.

The approximate locations of the proposed structures are shown on the attached plan.

This report is intended only for construction described above. Changes must be reviewed for additional recommendations as recommendations contained in this report will not be vali. .or other uses.

Testing

Classification. Grading analysis, density and moisture content determinations were made on representative samples. The results of these tests are shown on the attached

7 3 0 1 0 3 2

logs. The logs are based on laboratory tests and visual observation by the engineer in the field. A legend describing the classification tests precedes the logs.

Penetrometer. Field tests were made to measure the consistency of the soils. The results of these tests are shown on the attached logs.

Expansion. An expansion index test was conducted on a sample of the near surface soil consisting of sandy clay. The test results indicate that the soil has an expansion index of 50. The soil is considered to have a medium high potential for expansion. Requirements concerning expansive soil are included in the **Recommendations** section of this report.

Shear Tests. These tests were conducted on representative samples of the foundation soils. The samples that were selected were considered to have the least strength for each of the types of soil. In general, these were the samples with the lowest density, the highest degree of plasticity, and the highest percentage of fines passing the No. 200 sieve. The results of the shear tests appear to be consistent with the conditions found in the field and with results of tests on similar soils. The test results of each direct shear test have been plotted graphically and are attached.

Descriptions of the test methods are attached.

Design Calculations

Bearing Piles. The capacity of friction piles was determined by calculating the skin friction along the surface of the pile for earth pressure at rest and a safety factor of 2. An additional safety factor is present in end bearing which is neglected in the calculations. Graphs of the allowable bearing capacity versus depth for 1.5 and 2 foot diameter piles are attached.

7 3 - 0 1 0 0 3 2 1

3450 Wilshire Boulevard
Los Angeles, California

The bearing capacity of piles presented in the report was based, in part, by calculating the frictional resistance exerted by a 3 to 5 foot thick layer of soil underlain by bedrock. The weight of the fill (20 and 25 feet thick) overlying the soil was also considered in the calculations.

The settlement of piles is expected to be less than 1 inch. Differential settlement between any two piles is expected to be less than 1/4 of an inch.

Discussion

Exploration Limits. The recommendations in this report are based on the surface reconnaissance, lim ted exploration, and sampling described in the previous paragraphs. Geologic conditions and soil deposits may vary in type, consistency, and other properties across the site. Therefore, this report should be considered to be preliminary; its purpose is to provide recommendations for the grading and the general foundation system of the proposed structures described in the report. Earth Systems Consultants should be retained during construction to continue observations of the subsurface conditions and, if needed, provide additional recommendations for changes in design. Provision should be made for possible design and construction and construction changes during construction.

Variation in Pile Length. The length of piles may vary as a result of variations in the depth of fill deposits, natural soils, and bedrock. The length of piles should be given on the pla as estimated lengths. Provisions should be made for payment for shorter lengths and for longer lengths.

Fill. Our exploration was performed using small diameter equipment. It is difficult to detect depth of fill or thin soil layers in small diameter equipment. Therefore, the depth of fill or demarcation of soil profile presented on the attached logs should be considered approximate. Also, provisions should be made for possible variations in soil conditions.

Seismic Conditions. It is generally acknowledged that, with its present tectonic setting and abundant active faults, California is one of the most seismically "alive" portions of the United States. The property lies within a seismically active portion of Southern California and should be expected to experience occasional shaking from moderate to large magnitude earthquakes during the life of the proposed structures. The subsurface conditions at the site are comparable to those within the nearby vicinity. Therefore, we believe that if the seismic conditions are recognized and provided for in the project's engineering design and construction, structural behavior will be comparable to similar structures within the vicinity which have undergone past, seismically-induced ground shaking.

Limitations. This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team for the project and that the applicable information is incorporated into the plans, and that the necessary steps are taken to see that the contractors and subcontractors carry out such recommendations. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this ort may be invalidated, wholly or in part, by changes outside of our control. The validity of the recommendations of this report assumes that Earth Systems Consultants will be retained to provide these services. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around this site.

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Construction Responsibility. Representatives of Earth Systems Consultants will observe the work in progress, make tests of the soil, and examine the excavations. It should be understood that the contractor or others shall supervise and direct the work and that they shall be solely responsible for all construction means, methods, techniques, sequences and procedures, and they shall be solely and completely responsible for conditions of the job site, including safety of all persons and property during the performance of the work. Periodic or continuous observation by Earth Systems Consultants will not, nor is intended to include verification of dimensions or review of the adequacy of the contractor's safety measures in, on, or near the construction site.

General Recommendations

It is recommended that the proposed middle canopy, westerly canopy, and westerly sign be supported on friction piles extending through the fill and embedded into firm natural material.

The easterly canopy and sign may be supported on spread footings embedded at least 18 inches into firm natural soil.

The recommendations contained in this report are intended to augment or to supersede certain sections of the Building Code. This report should be reviewed by the governing authorities as some of the recommendations exceed the allowable values provided in the Code or may not conform to the reviewing agency's regulations.

It is recommended that the completed plans and specifications be submitted to us for review of the geotechnical aspects. The review would not include checking calculations by the structural engineer.

All work should be completed in accordance with the approved plans and specifications and the applicable provisions of the Building Code, safety ordinances, and the regulations of the Building Department. Periodic inspection during construction may be required by personnel from these agencies.

Additional recommendations for foundation design and construction are given in subsequent sections of the report.

Recommendations for Design

Spread Footings. Footings shall have a minimum width of 12 inches and the base of the footing shall be placed at a minimum depth of 18 inches below the lowest adjacent finished grade.

Footings having a width of 12 inches and a depth of 12 inches may be designed for a foundation pressure of 1000 pounds per square foot. An increase of 200 pounds per square foot for each additional foot of depth may be used. The foundation pressure should not exceed 3000 pounds per square foot.

The weight of the footing below the lowest adjacent grade can be neglected. The allowable foundation pressure may be increased by 1/3 of the given value for earthquakes or other temporary forces.

Piles. Pile capacity may be selected from the attached graphs. However, the actual length of pile may vary during placement due to changes in soil and bedrock elevations. Provision should be made for such variations. Without regard to the capacity selected from the attached chart, the piles supporting the westerly canopy and sign shall penetrate at least 5 feet into firm natural material.

Piles supporting the middle canopy should penetrate at least 10 feet into firm natural materials.

The load on the pile is the load at the top; the weight of the pile may be neglected.

Piles should be spaced at least 2-1/2 diameters apart, center to center, with a minimum spacing of 3 feet.

Piles may be cast-in-drilled-holes.

Site Drainage. The site should be graded to slope away from the structures. Areas where water could pond adjacent to the structures in planter areas, or where walks and drives would create depressed areas, should be eliminated by use of area drains. Area drains should not be placed next to or in contact with the structures.

Recommendations for Construction

A meeting between representatives of the contractor, the governmental agencies, the soil engineer, and the owner should be held at the job site at the time equipment is at the site and work is about to commence. The purpose of the meeting is to review the responsibilities of each member of the team.

Caving of the pile shafts may occur below groundwater. In that case, casing may be necessary to maintain the pile shafts open. After drilling the holes and placing the reinforcement, concrete may be tremied in the holes. The tremie shall be placed to within one foot of the bottom of the shaft. The tremie shall be kept filled with concrete while placing concrete in the bottom of the shaft and below the water. The tremie tip should be kept at least 3 feet into the concrete during placement.

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All excavations for friction piles must be observed by Earth Systems Consultants during drilling. Friction piles shall be inspected during placing of concrete by a deputy concrete inspector approved by governmental agency having jurisdiction over the project.

Footing excavations should be examined by Earth Systems Consultants before the forms are set. A note regarding this should be placed on the plans.

Conclusions

We conclude that the site will be suitable for the proposed construction described in this report, providing the design and construction are properly executed. Our recommendations are based on site conditions encountered during exploration, laboratory tests, and experience with similar sites, and are in accordance with generally accepted procedures of geotechnical engineering.

EARTH SYSTEMS CONSULTANTS
FOUNDATION ENGINEERING

Shaine Shahidi
Shaine Shahidi
Project Engineer

Jack W. Rolston
Jack W. Rolston
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JACK W. ROLSTON
734
EXP. 12/31/92
GEOTECHNICAL
STATE OF CALIFORNIA

SS:sp

March 13, 1992

3450 Wilshire Boulevard
Los Angeles, California

List of Attachments

1 - Location of Test Holes
1 - Legend for Log of Test Holes
1 - Standard Penetrometer
4 pgs - Logs of Test Holes
1 pg - Description of Test Methods
3 pgs - Direct Shear Tests
2 pgs - Pile Capacity

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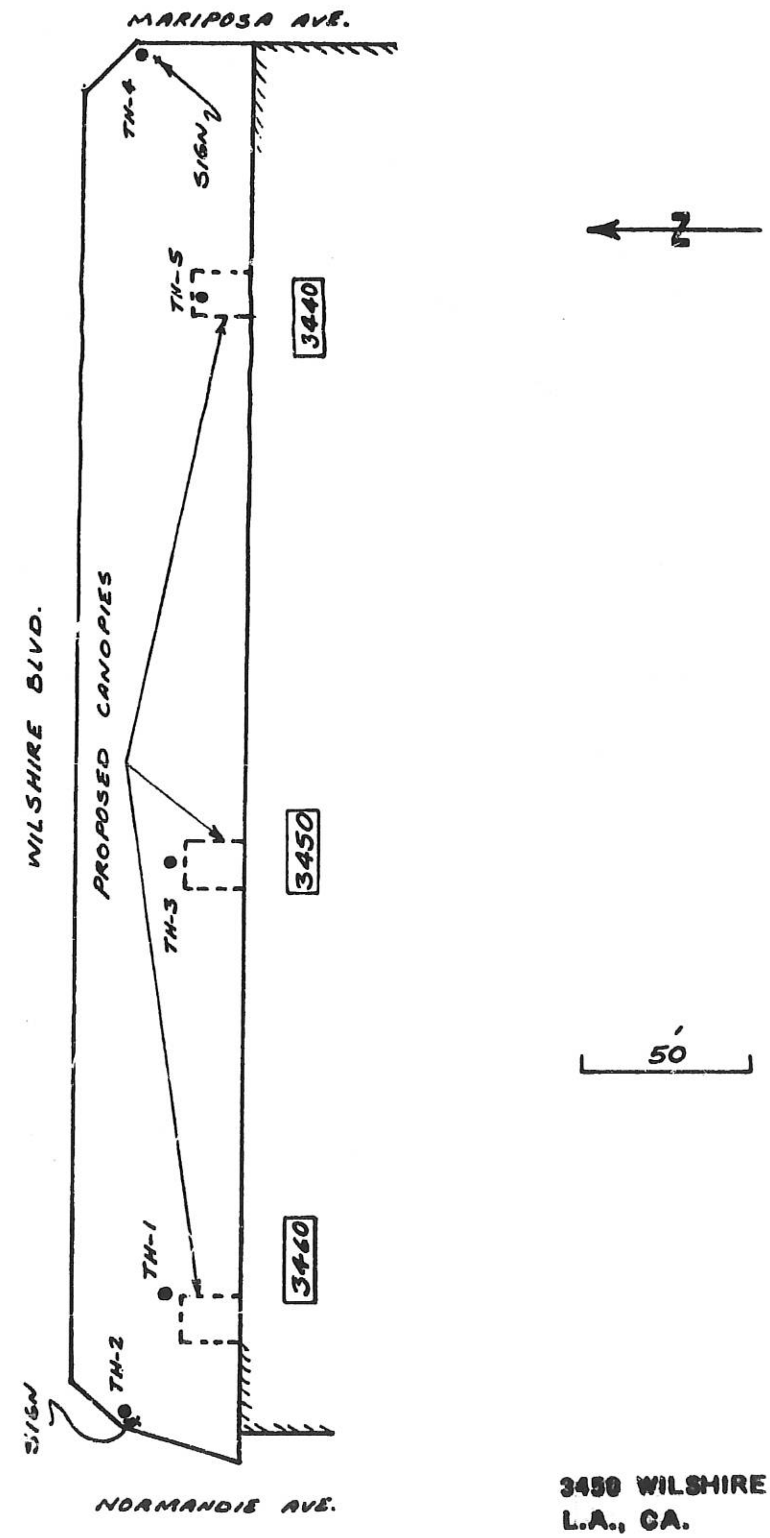
LOCATION OF TEST HOLES



3450 WILSHIRE BL.
L.A., CA.

E A R T H   S Y S T E M S   C O N S U L T A N T S

7 3 0 0 1 0 0 3 3 0

LEGEND FOR

LOG OF TEST HOLE

The Log of Test Hole indicates the geotechnical conditions at that location at that particular time. Soil and groundwater conditions can change with time. The stratification lines shown on the logs represent the approximate boundary between soil and rock types and the transition may be gradual or abrupt.

Relatively undisturbed samples are usually obtained by a thin wall sampler in accordance with the American Society for Testing and Materials (ASTM) Test Method D 1587 "Thin-walled Tube Sampling of Soils." Samples are sometimes obtained by ASTM D 3550 "Ring-lined Barrel Sampling of Soils." These samples are sealed and delivered to the laboratory for testing.

Numerous samples are selected primarily for classification tests. These samples are obtained in accordance with ASTM D 1412 "Soil Investigation and Sampling by Auger Borings." In some cases samples are obtained by ASTM Method D 1587 "Penetration Test and Split-Barrel Sampling of Soils." The soil is classified in the field by the engineer in accordance with ASTM D 2488 "Description of Soils (Visual-Manual Procedure)." While at the site, the engineer obtains other relevant information in general accordance with ASTM D-420 "Investigating and Sampling Soil and Rock for Engineering Purposes."

Classification tests are made in the laboratory on both undisturbed samples and disturbed samples in accordance with ASTM D 2487 "Classification of Soils for Engineering Purposes" and ASTM 2488. Only a broad classification of soils is given on the logs because the soils are variable. Therefore, the group symbol is not used. Fine grained soil is differentiated on the basis of ASTM D 2488. The logs are intended to portray easily identifiable changes in strata. When two soils are used as a soil classification, the last named in the predominate soil and the preceding soil type is the second most predominate soil. The shaded symbols are standard symbols and are not intended to indicate relative importance of soil components within a given classification.

An explanation of the symbols and values shown on the logs are as follows:

w	Moisture content, percent of Dry Density
D	Dry Density, pounds per cubic foot.
4	The ~cent of the material that will pass a no. 4 sieve (3/16"). The material larg.. than the no. 4 sieve and smaller than 3 inches would be designated as a gravel, and the material smaller than the no. 4 and larger than the no. 200 would be termed a sand. Cobbles range in size from 3 to 12 inches. Boulders are greater than 12 inches.
200	The percent of the material that will pass a no. 200 sieve (about the smallest that can be seen with the unaided eye.) The fraction finer than the 200 sieve is classed as a clay or silt.

STANDARD PENETROMETER

The standard penetrometer is used to provide a measure of the consistency of soil and to approximate the strength properties. With knowledge of the soil classification and penetration resistance, estimates of performance of the soil under influence of loads can be made. The oldest form of penetrometer testing, and the one most widely used, is called the "standard penetration test." This is performed in accordance with American Society for Testing and Materials (ASTM) test method D1586. It is performed by noting the number of blows (N) of a 140 lb. hammer falling 30 inches on a 2 inch (O.D.) sampler necessary to drive the sampler 12 inches, after seating the sampler 6 inches.

Gibbs & Holtz (1957) established that overburden pressure affected the blow count in cohesionless soils when tests were made in holes slightly larger than the penetrometer. Where penetrometer tests are made in the bottom of a test pit the values for zero depth should be used. Where penetrometer tests are made in a bucket auger hole a value between 0 and 15 feet should be used.

The following general relationships exist:

COHESIONLESS SOIL

Consistency	Depth 0'	15'	30'	60'	Relative Density %	ϕ (Deg.)
	Penetration Resistance N					
Loose						
	3	4	6	10	35	28-38
Medium Dense						
	6	12	20	30	65	30-42
Dense						
	12	24	30	50	85	32-45
Very Dense						

COHESIVE SOIL

Overburden does not significantly affect the penetration resistance according to Terzaghi (1948).

Consistency	N	Unconfined Compression TSF
Soft		
	4	0.5
Medium Stiff		
	8	1.0
Stiff		
	15	2.0
Very Stiff		

791111

Soil Engineering Report

3450 Wilshire Boulevard
Los Angeles, California

LOG OF TEST HOLE NO. 1

Depth	N	w	D	4	200	DESCRIPTION
0						FILL; Silty Sand, gravelly, light brown. Sandy Clay, brown, moist. Sandy Silt, brown, moist.
	5	20		100	76	Sandy Clay, mottled brown, soft.
	4	25	94	100	78	
	2	16		100	48	Clayey Sand, very loose, moist, organic, orange veins.
19′	17	17	108	100	24	SILTY SAND; gray, shale pieces, medium coarse grained, medium dense.
24′	30	34	85	100	97	MUDSTONE; highly weathered, green-gray, dense.
	29	31		100	75	
30′						

Drilled 2/25/92
No groundwater

Soil Engineering Report

3450 Wilshire Boulevard
Los Angeles, California

LOG OF TEST HOLE NO. 2

Depth	N	w	D	4	200	DESCRIPTION
0						FILL; Sandy Clay, brown, moist, soft.
	4	17		99	42	Clayey Sand, brown, loose.
	3	19		100	44	
	4	18		100	45	Few gravel, brown.
19′	47	5	112	100	8	SAND; light brown, coarse grained, dense, few gravel.
23′	39	29	89	100	58	SANDY SILT; mottled brown, very moist, stiff.
28′	41	34	86	100	98	MUDSTONE; light brown, highly weathered, blue gray.
30′						

Drilled 2/25/92
Groundwater 28 feet

Soil Engineering Report

3450 Wilshire Boulevard
Los Angeles, California

LOG OF TEST HOLE NO. 3

0

23'

24'

28'

32'

35'

N	w	D	4	200	DESCRIPTION
					FILL; Sandy Clay, brown, soft, moist. Clayey Sand, dark gray, organic odor.
					Sandy Clay, brick pieces.
	25	99	100	55	AC pieces.
5	21		100	51	
8					No recovery.
					Sandy Clay, brown.
10	21	105	100	55	SANDY CLAY; brown, medium stiff. Thin layer of Silty Sand.
20	40	80	100	88	SANDY SILT; green-gray, clayey, stiff.
44	35		100	65	SILTSTONE; mottled gray, stiff, organic odor.

Drilled 2/25/92
Groundwater 23 feet

Soil Engineering Report

3450 Wilshire Boulevard
Los Angeles, California

LOG OF TEST HOLE NO. 4

Depth	N	w	D	4	200	DESCRIPTION
0						FILL; Sandy Clay
1'	25	16	110	100	65	SANDY CLAY; brown, stiff, damp.
3'	17	16		100	42	CLAYEY SAND; brown, fine grained, dense.
9' – 10'	22	20	104	100	61	SANDY CLAY; brown, stiff.

LOG OF TEST HOLE NO. 5

Depth	N	w	D	4	200	DESCRIPTION
0						FILL; Sandy Clay, brown, soft.
2'	18	18	105	100	48	CLAYEY SAND; brown, damp, fine grained, dense.
	17	20		100	47	
9' – 10'	32	29		100	85	SANDY CLAY; brown, stiff.

Drilled 2/25/92
No groundwater

73300100333

DIRECT SHEAR TESTS

The shear test is to determine the strength of soil under various confining loads and moisture conditions. The direct shear test is in accordance with the American Society for Testing and Materials (ASTM) Test Method D 3080, Direct Shear Test of Soils under Consolidated Drained Conditions. The specimens were 2-1/2 inches in diameter and 1 inch in height. Specimens were flooded and allowed to consolidate under the normal load for 24 hours. The confining loads range from 500 to 4000 pounds per square foot. Shear loads were applied at the rate of 0.01 inch per minute while the specimen was immersed in water. The shear load was applied continuously until the end of the test in accordance with the generally accepted test procedure for the consolidated-undrained shear test (R). On completion of the test, the final moisture content was obtained to determine if approximately 100 percent saturation was attained.

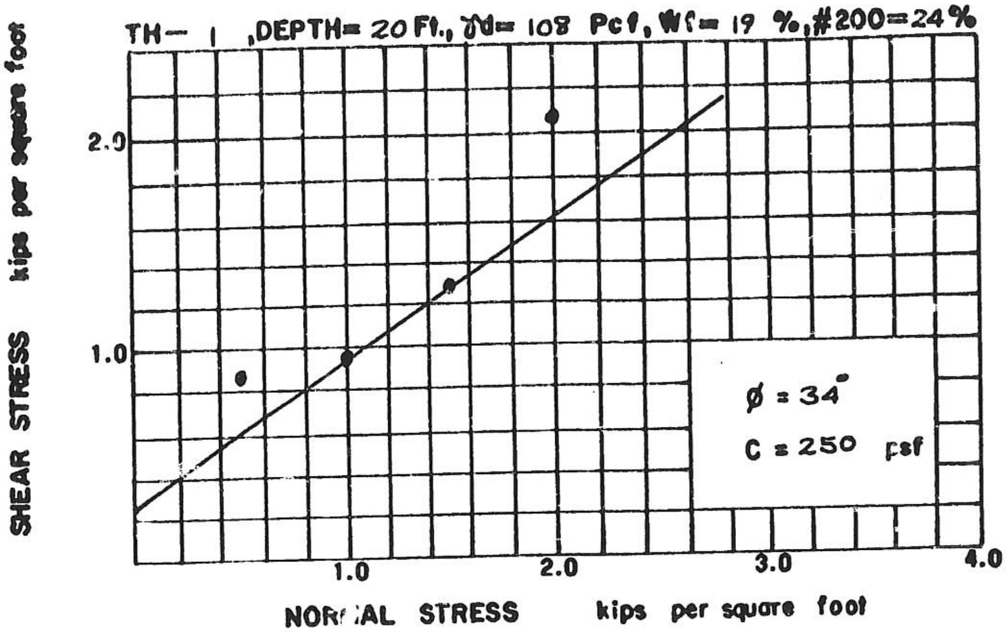
EXPANSION TEST

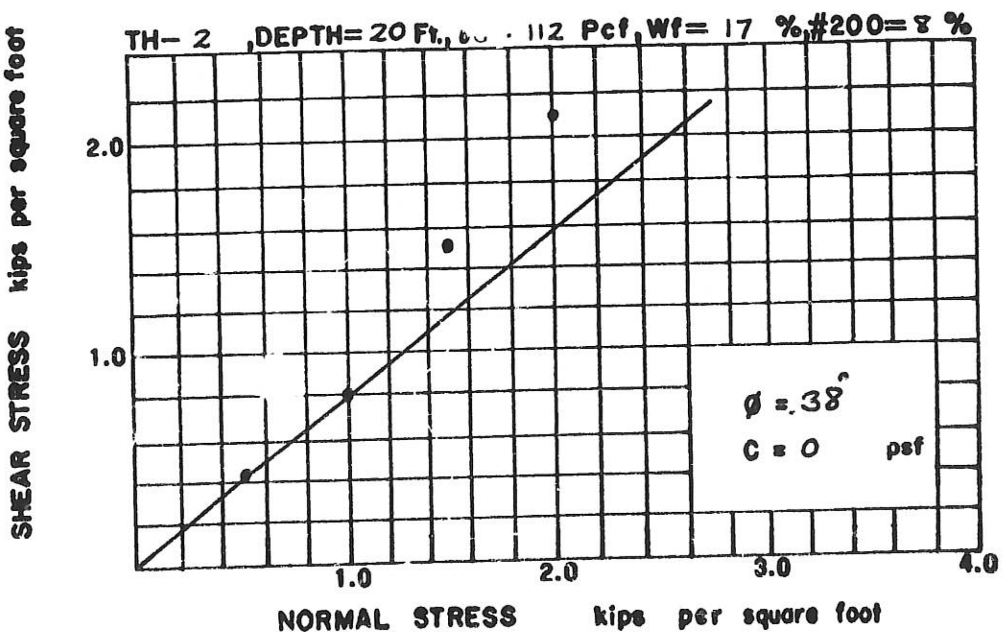
The expansion test is an index test to classify the expansive characteristics of a soil. The test is conducted in accordance with the Uniform Building Code Standard No. 29-2, Expansion Index Test. A moistened sample was compacted in 2 layers in a four inch diameter ring. Each layer is about 1 inch in height and compacted with 15 blows of a 5.5 pound hammer having a fall of 12 inches. If the degree of saturation ranged between 49 and 51 percent for an assumed specific gravity of 2.7, the specimen was loaded with 144 pounds per square foot and flooded. After 24 hours the expansion was noted and the expansion indices calculated. The expansion characteristics are given in the following table.

Expansion Index	Potential for Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
Above 130	Very High

73500100337

DIRECT SHEAR TESTS





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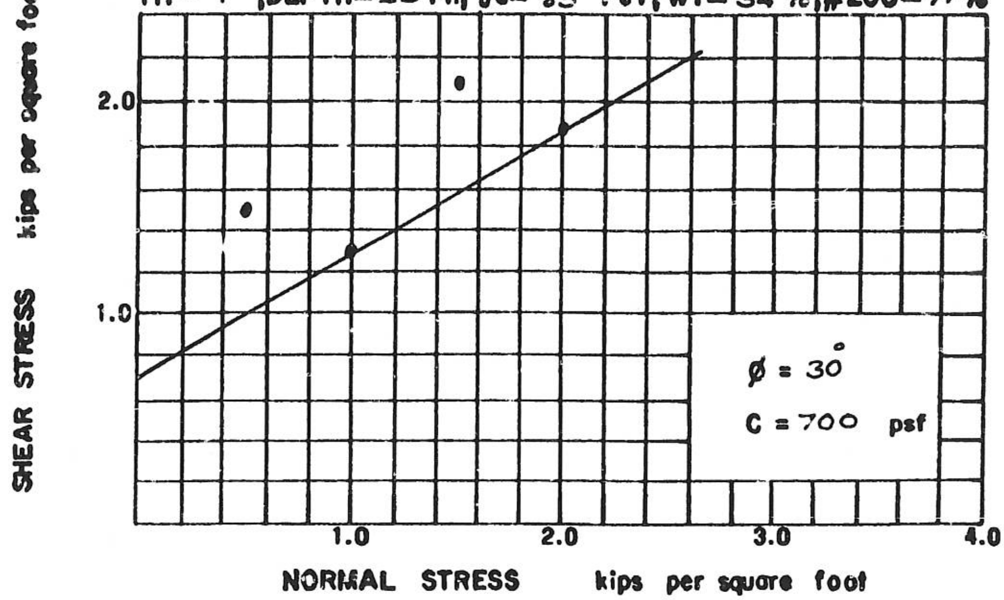
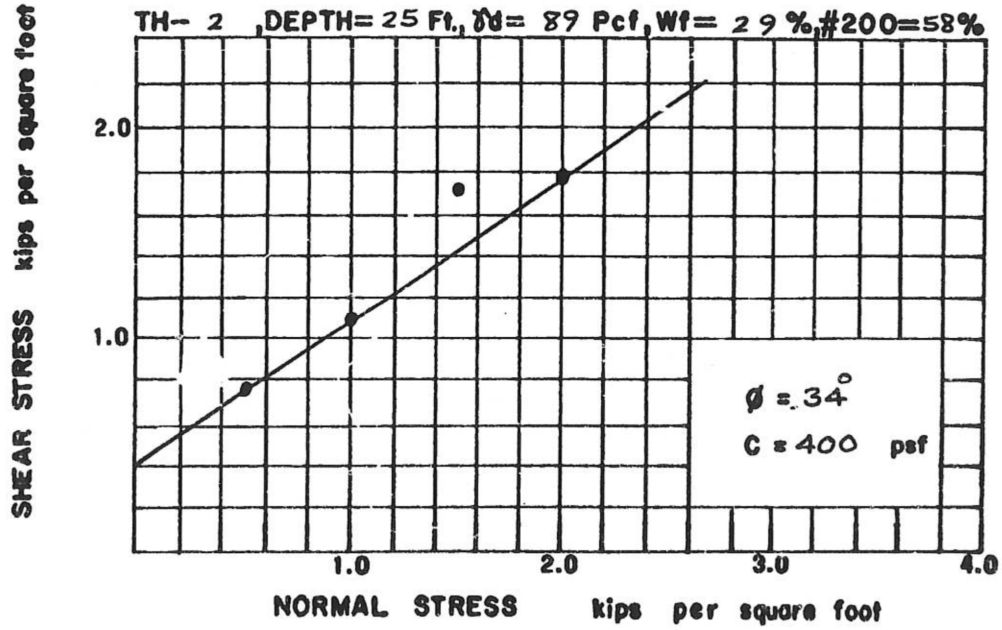
DIRECT SHEAR TESTS

TH- 1 ,DEPTH= 25 Ft., γ_d= 85 Pcf, Wf= 34 %, #200= 97%



TH- 2 ,DEPTH= 25 Ft., γ_d= 89 Pcf, Wf= 29 %, #200= 58%



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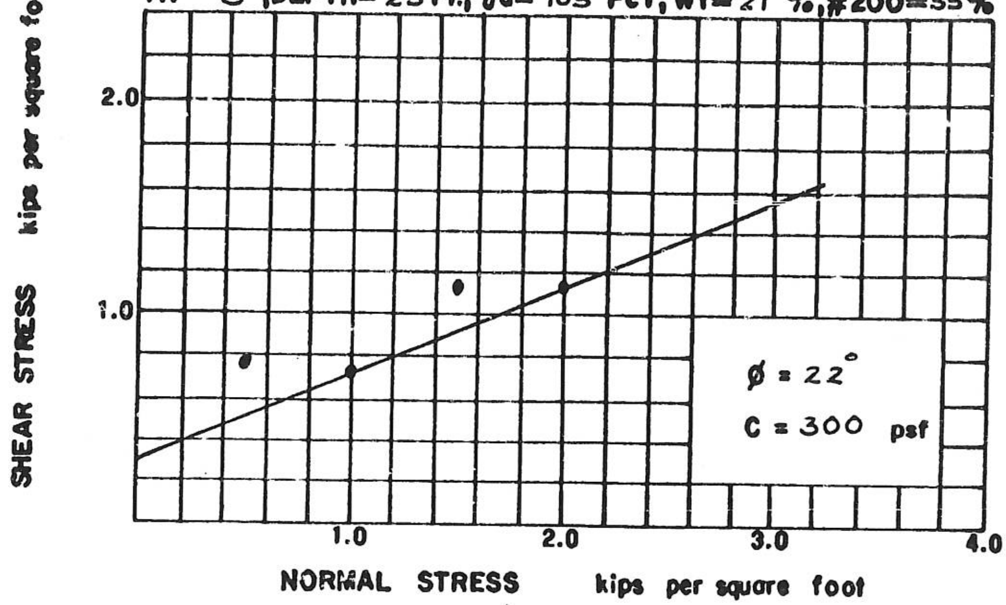
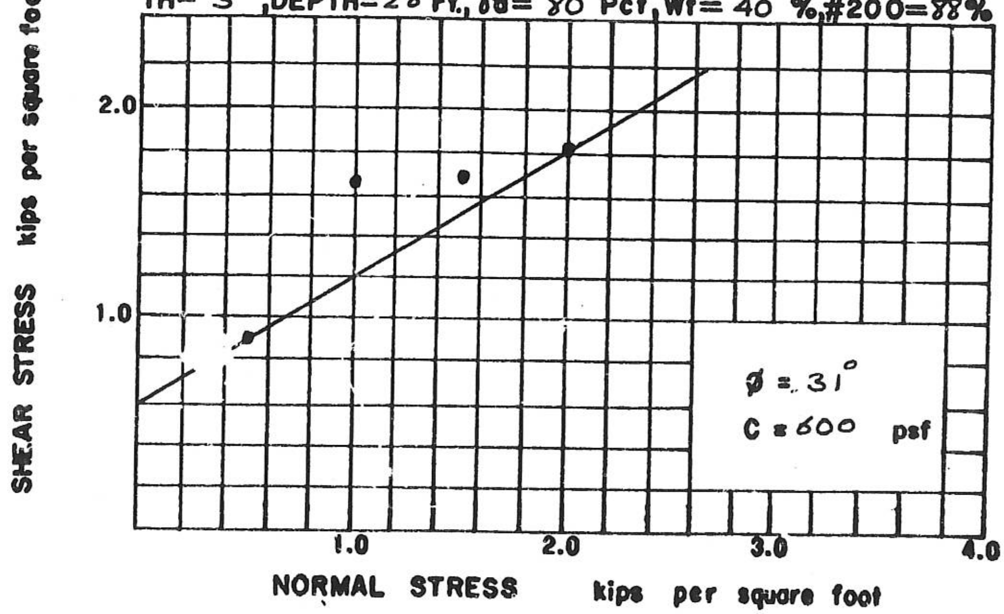
DIRECT SHEAR TESTS

TH- 3 ,DEPTH= 25 Ft., γd= 105 Pcf, Wf= 21 %, #200=55%



TH- 3 ,DEPTH= 28 Ft., γd= 80 Pcf, Wf= 40 %, #200=88%



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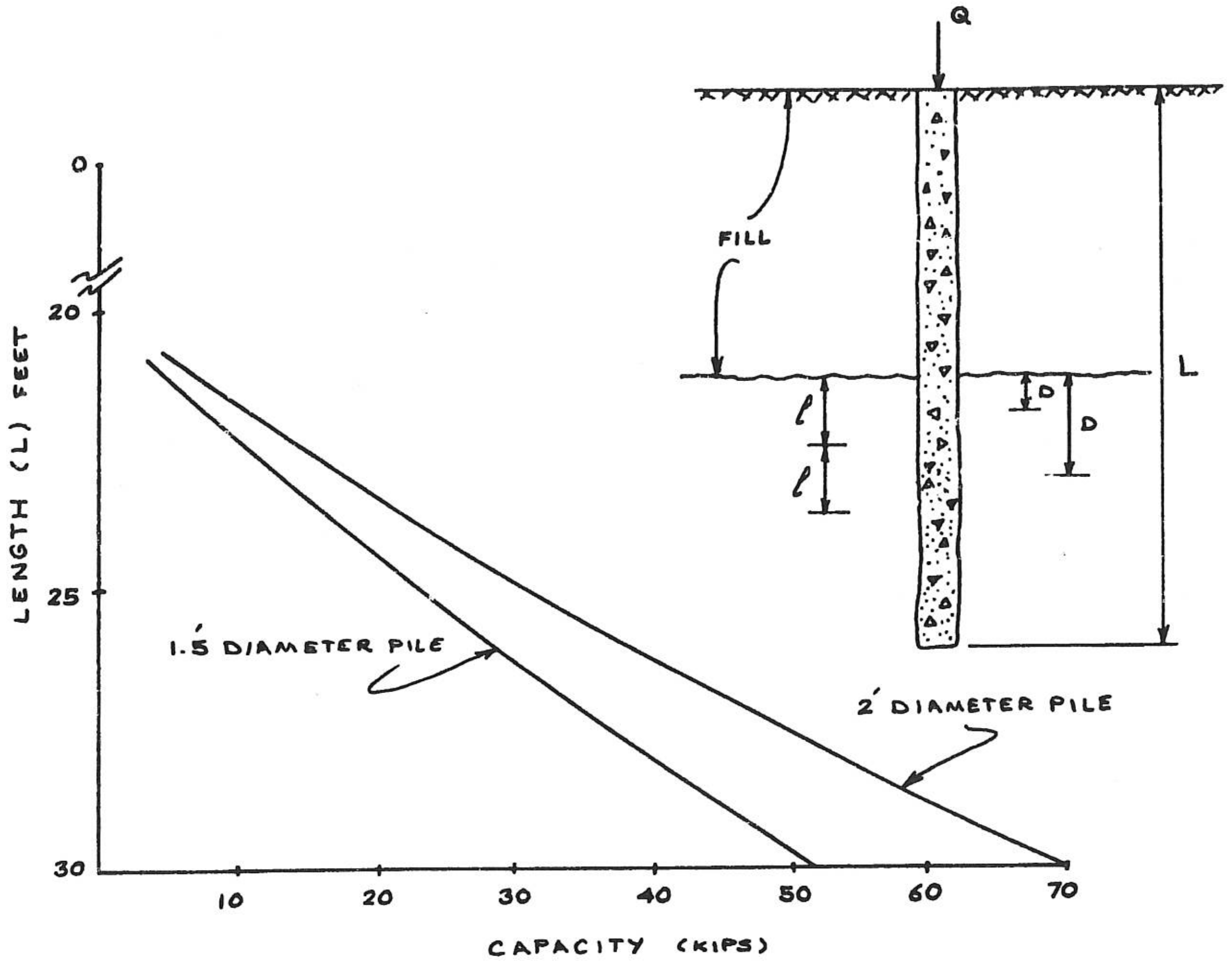
PILE CAPACITY

WESTERLY CANOPY & SIGN



$$Q = \sum \frac{\pi d \ell}{SF} (C + \gamma D \tan \phi)$$

DESIG VALUES

	0-20'	20'-25'	25'-30'	
γ =	110	120	115	(PCF)
ϕ =	IGNORED	34	34	DEGREES
C =	IGNORED	250	400	(PSF)

GROUNDWATER DEPTH = 28'

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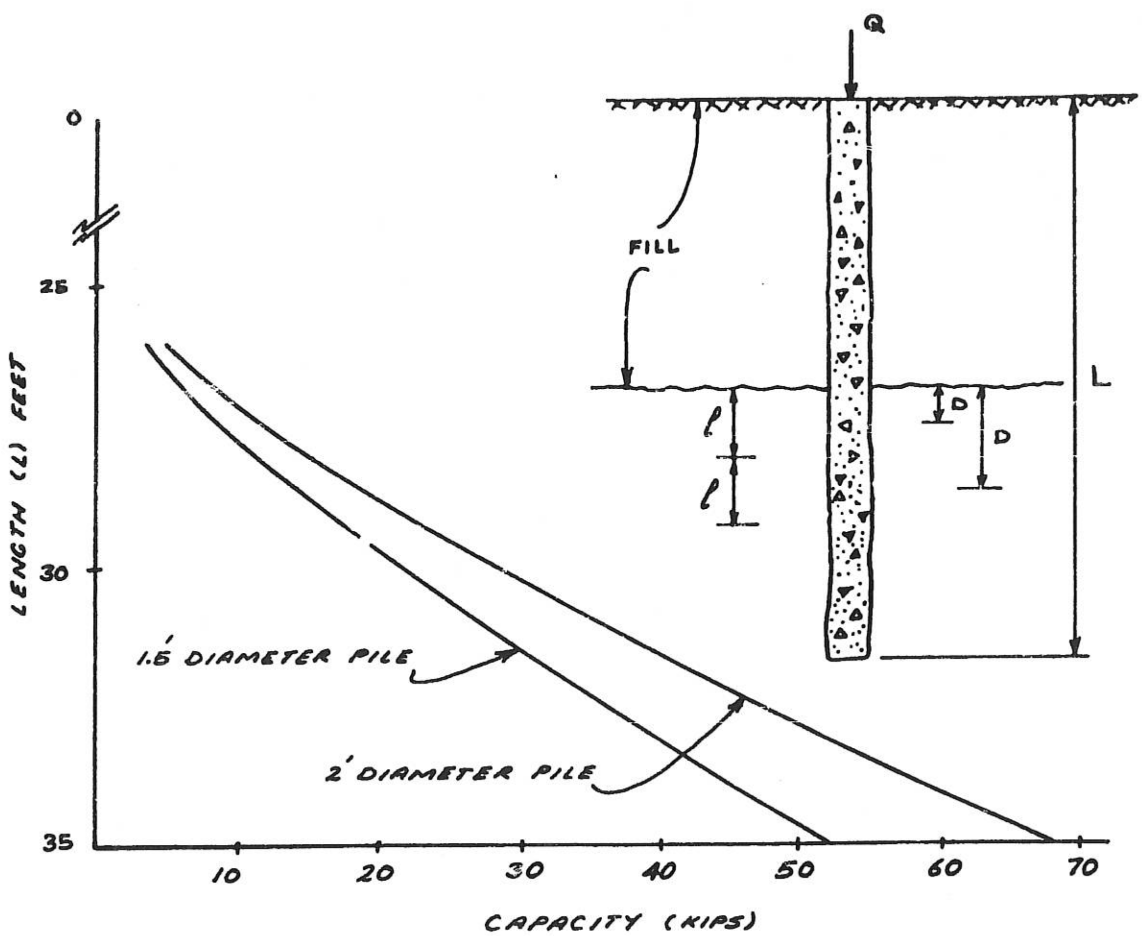
EARTH SYSTEMS CONSULTANTS

PILE CAPACITY

MIDDLE CANOPY



$$Q = \sum \frac{\pi d \ell}{SF}(c + \gamma D \tan \phi)$$

DESIGN VALUES

	0-25'	25'-28'	28'-36'	
γ =	110	125	115	(PCF)
ϕ =	IGNORED	22	31	(DEG.)
C =	IGNORED	300	600	(PSF)

GROUNDWATER DEPTH = 23'

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L.A., CA.

EARTH SYSTEMS CONSULTANTS

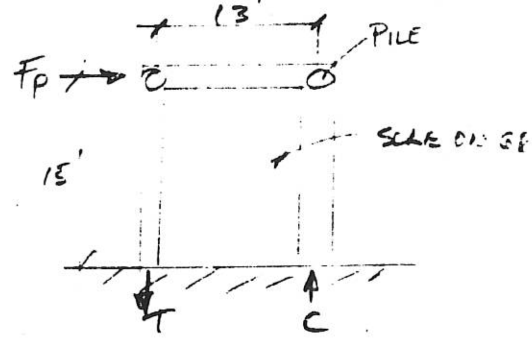
PROJECT CENTRAL PLAZA	BRANDOW & JOHNSTON ASSOCIATES	SHEET NO. 5A
FOR LUCKMAN PARTNERSHIP	CONSULTING STRUCTURAL ENGINEERS	JOB NO. 91036
LOCATION 3440, 50 & 60 WILSHIRE	1660 W. THIRD ST., LOS ANGELES, CALIFORNIA 90017	DATE 3/92
ITEM CANOPY FOUNDATIONS	TELEPHONE 484-8950 AREA CODE 213	ENGINEER DRS

SEE ATTACHED ORIG. CALCS 4 & 5

FOR NEW PILE FNDN - PER SOIL REPORT EARTH SYSTEMS 3/3?

$P_{MAX} = 3.6^K$

SEIS. $F_P = 1.88^K$



PLAN

TAKE SEISMIC INTO FTG

$$T = C = \frac{1.88 \times 15}{13} = 2.17^K$$

$$A_S = \frac{2.17 \times 1.4}{.9 \times 40} = 0.084\,\square''$$

NOM. 2-#5 INTO EXIST'G CONC. WALL OK

$$V_u\ wall = \frac{1.88 \times 1.4}{13'} = 202\ PLF$$

$$v_u = \frac{202}{.85 \times 4 \times 12} = 5\ psi \quad \text{MINI MESH OK}$$

NEW 18" ϕ PILE -

$$P/A = \frac{3.6}{\frac{\pi}{4} \times 18^2} = 14\ psi < .45\sqrt{3000} = 24.6\ psi$$

$$0.5\%\ STL = .005 \times \frac{\pi}{4} \times 18^2 = 1.27\,\square'' < 4 \times .44 = 1.76$$

PIL CAPAC. FOR 5' EMBED $> 15^K$

18" ϕ PILE w/ 4-#6 VERT

CANOPY LOAD = 3.6^K
PILE x 12' = 3.2 $= \frac{\pi}{4} \times 1.5^2 \times 12 \times .15$
GR. BM = 4.7 $= 2 \times 1.12 \times 14' \times .15$

$11.5^K < 15$ OK

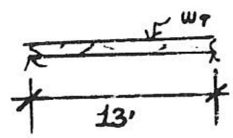
REGISTERED PROFESSIONAL ENGINEER DONALD R. STRAND No. 1162 EXP. 12-31-92 STRUCTURAL STATE OF CALIFORNIA

7 3 6 0 0 1 0 0 3 4 3

LOADING

DL 10 psf } SKYLIGHT 20 psf { STUDS + 3/4 STONE ~~CEMENT~~
LL 12 psf }

Check BEAM W10x22



Span = 13'

$w_T = (10psf_{DL} + 12psf_{LL}) \times 9' + 20psf \times 5' \times 2 = .4$ k/ft

$M_{ax} = (.4 k/ft \times 13^2)/8 = 8.5$ k-ft

$l/r_t = (13' \times 12)/1.51 = 103.3 < \sqrt{\frac{510 \times 10^3 \times 1.0}{36}} = 119$

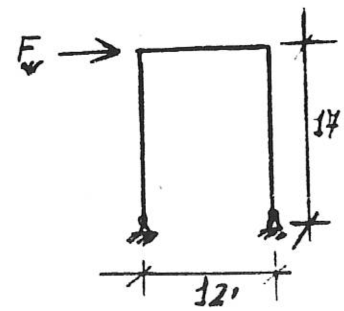
$> \sqrt{\frac{102 \times 10^3}{36}} = 53.23$

$F_b = \left[\frac{2}{3} - \frac{36(103.3)^2}{1530 \times 10^3}\right] \times 36$ ksi $= 15$ ksi

$S_{req} = (8.5$ k-ft $\times 12)/15$ ksi $= 6.8$ in³ < 23.2 in³ o.k.

$\boxed{\text{Use W10x22}}$

Check COLUMN



SEISMIC $F_p = .3\,(\overset{RF+BM}{15' \times 12' \times 7'} + \overset{WALL}{20 \times 25' \times 5' \times 2}) = 1.88^k$

WIND PRES. $E = C_e C_q q_s I$

$P = .7 \times 1.3 \times 13psf \times 1.0 = 11.83$ psf

$F_w = 11.83$ psf $\times 13' \times 8' \times 1/2 = 620^{\#}$

$P = .4$ k/ft $\times 13'/2 = 2.6^k$

$\theta_{col} = \frac{1.88^k/2 \times (17' \times 12)^2}{3 \times 29 \times 10^3 \times 170 \text{ in}^4} = .0026$ $.0026 + .00064 = .0024$

$\theta_{BEAM} = \frac{1.82^k \times (12' \times 12)^2}{12 \times 29 \times 10^3 \times 118 \text{ in}^4} = 0.00064$

$\Delta = 17' \times 12 \times (.0026 \times 10^{-3} + 6.4 \times 10^{-4}) = .6''$

7 3 0 0 1 0 0 3 4

PROJECT central plaza	BRANDOW & JOHNSTON ASSOCIATES	SHEET NO. 5
FOR	CONSULTING STRUCTURAL ENGINEERS	JOB NO. 91036
LOCATION	1660 W. THIRD ST. LOS ANGELES, CALIFORNIA 90017	DATE 2/91
ITEM. CANOPY SUPPORT	TELEPHONE 484-8950 AREA CODE 213	ENGINEER A.M.

$K\ell = 2.0 \times 13' = 26'$

TRY W10x?? $P_{all} = 56^k$

$M_{xx} = 1.88^k/2 \times 17' = 16^{k'}$ $F_b = \left[\frac{2}{3} - \frac{36(131.6)^2}{1530 \times 10^3 \times 1.75}\right] \times 36 = 15.6$ ksi

$2.6^k/56^k + \frac{(16^{k'} \times 12)/35 in^3}{15.6 ksi} = .39 < 1.0$ o.k.

$\boxed{\text{USE W10x33}}$

Footing

$P_{max} = 2.7^k + .62^k \times 17/12' = 3.6^k$

$\text{width} = \frac{3600^{\#}}{3' \times 1000 psf} = 1.2'$ or 14.5"

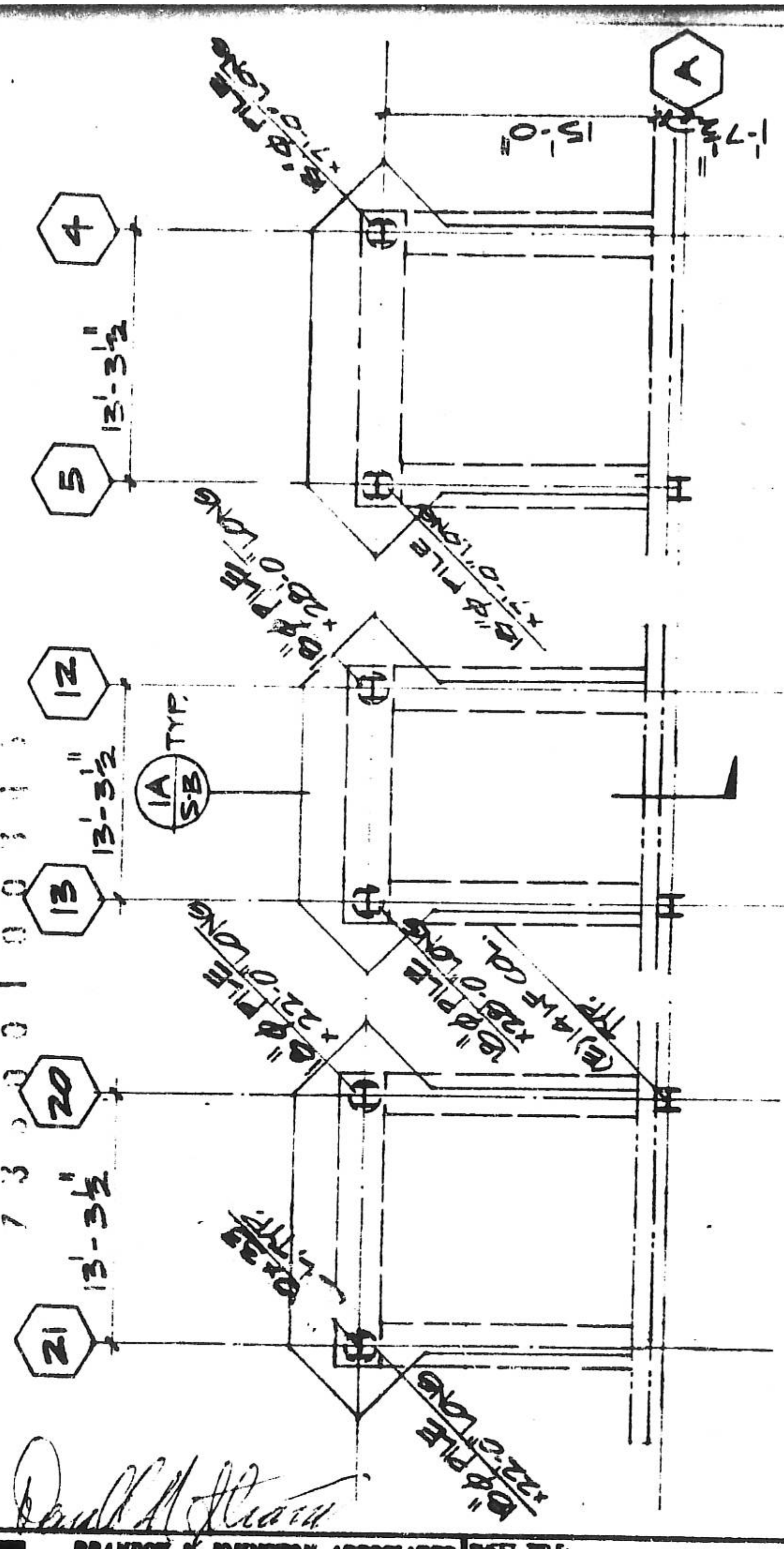
13" conc. column Design @ PARKING

column length = 19' $k = 1.25$

$P_{ax} = \overbrace{28' \times \pi \times 1.083^2/4 \times 150^{\#}/ft^3}^{\text{column weight}} + 2.2^k \times 2 + \overbrace{1.43^k \times 7.5' \times 1/9'}^{\text{seismic}} = 9.5^k$ see page 5A

use a 13" COLUMN
w/ 5 #5 bars & #3 spiral

Footing $A = 9.5^k/1 ksf = 9.5 ft^2 \Rightarrow W = D = 3 ft$ use 3 ft SQ-FTG
w/ 3 #5 bars E.W.



PARTIAL FIRST FLOOR AND FOUNDATION PLAN

SCALE: 1/8" = 1'-0"   SEE SHT. S-1

REGISTERED PROFESSIONAL ENGINEER
DONALD R. STRAND
No. 1182
EXP. 12-31-92
STRUCTURAL
STATE OF CALIFORNIA

THIS DRAWING SHALL AMEND AND SUPPLEMENT ORIGINAL DRAWINGS, SPECIFICATIONS AND CONTRACT DOCUMENTS ONLY TO THE EXTENT SPECIFICALLY SHOWN OR NOTED

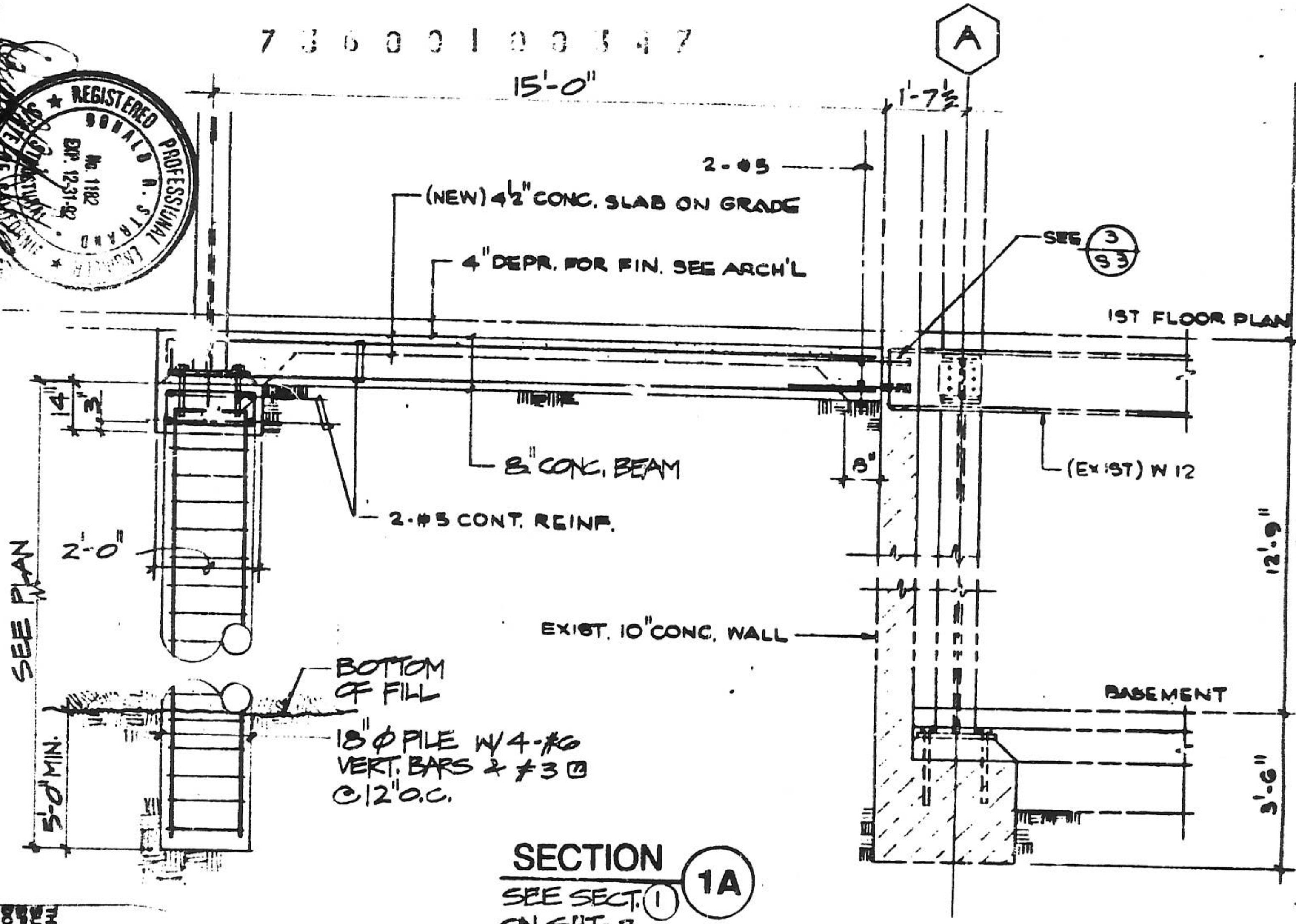
BRANDOW & JOHNSTON ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
1660 W. THIRD ST., LOS ANGELES, CALIFORNIA 90017
TELEPHONE (213)-[illegible]

SHEET TITLE:
PARTIAL FIRST FLOOR & FOUNDATION PLAN

THE LUCKMAN PARTNERSHIP, INC
ARCHITECTURE/PLANNING
9220 SUNSET BOULEVARD
LOS ANGELES, CA. 90069

PROJECT:
CENTRAL PLAZA REMODEL
3440, 3456, & 3460 WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

PROJECT NO.:
DATE: 3-18-92
DRAWN:
SHEET: S-A



SECTION 1A
SEE SECT. 1 ON SHT-2

NOTE–

1. PILE INSTALLATION SHALL CONFORM TO SOILS ENG'G. REPORT BY EARTH SYSTEMS CONSULTANTS DATED MARCH 13, 1992.
2. ALL PILE WORK SHALL BE UNDER DIRECTION OF A REPRESENTATIVE OF EARTH SYSTEMS CONSULTANTS.
3. CONCRETE SHALL HAVE A 28 DAY STRENGTH OF 3000 PSI. & REINF. SHALL CONFORM TO ASTM A615-60 GR. WORK SHALL BE APPROVED BY A CITY OF LOS ANGELES BUILDING INSPECTOR.

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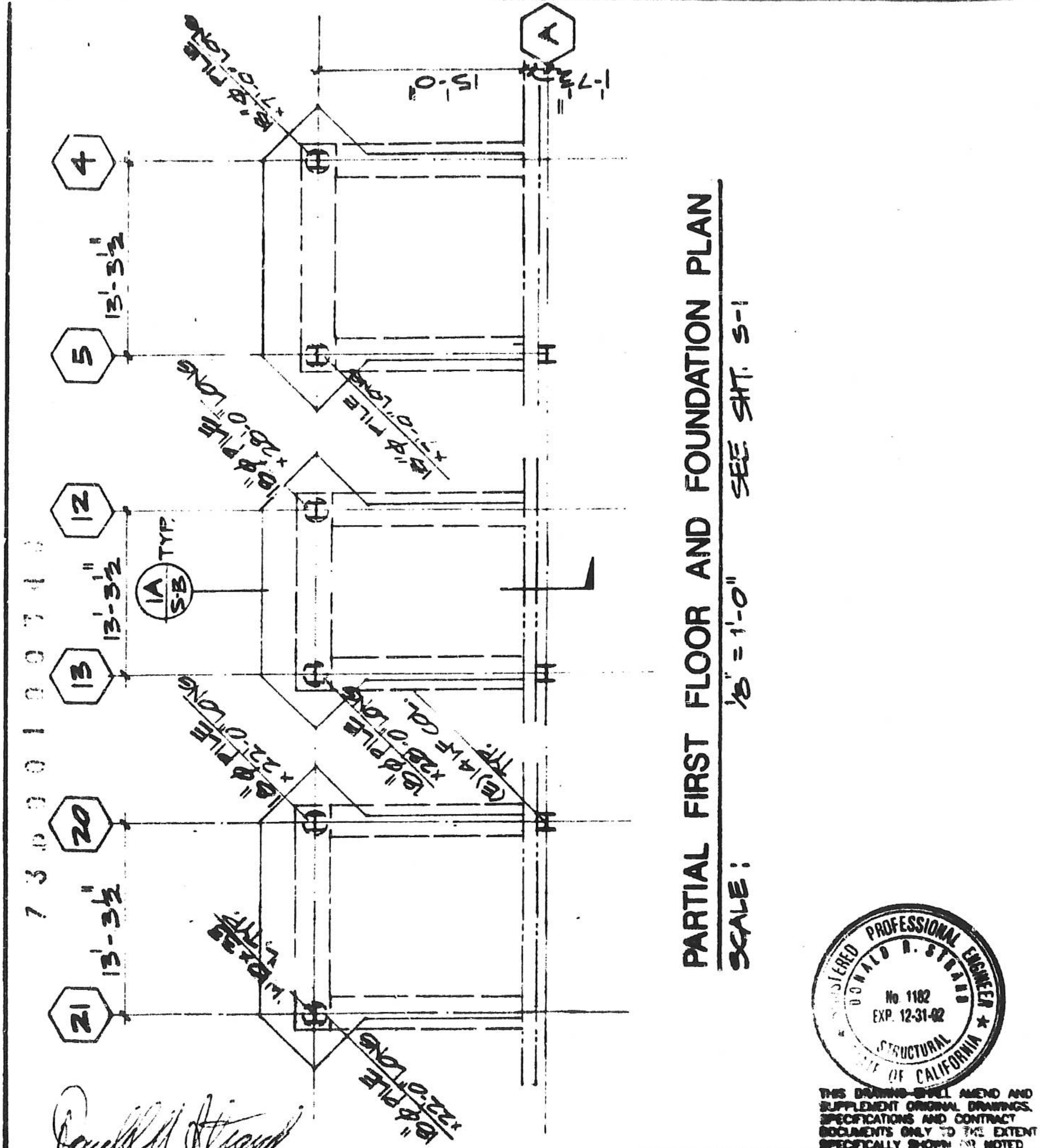
BRANDOW & JOHNSTON ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
1660 W. THIRD ST., LOS ANGELES, CALIFORNIA 90017

THE LUCKMAN PARTNERSHIP, INC
ARCHITECTURE/PLANNING
9220 SUNSET BOULEVARD
LOS ANGELES, CA. 90069

SHEET TITLE: SECTION

PROJECT: CENTRAL PLAZA REMODEL
3440, 3460, & 3450 WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

PROJECT No.:
DATE: 3-18-92
DRAWN:
SHEET: S-B



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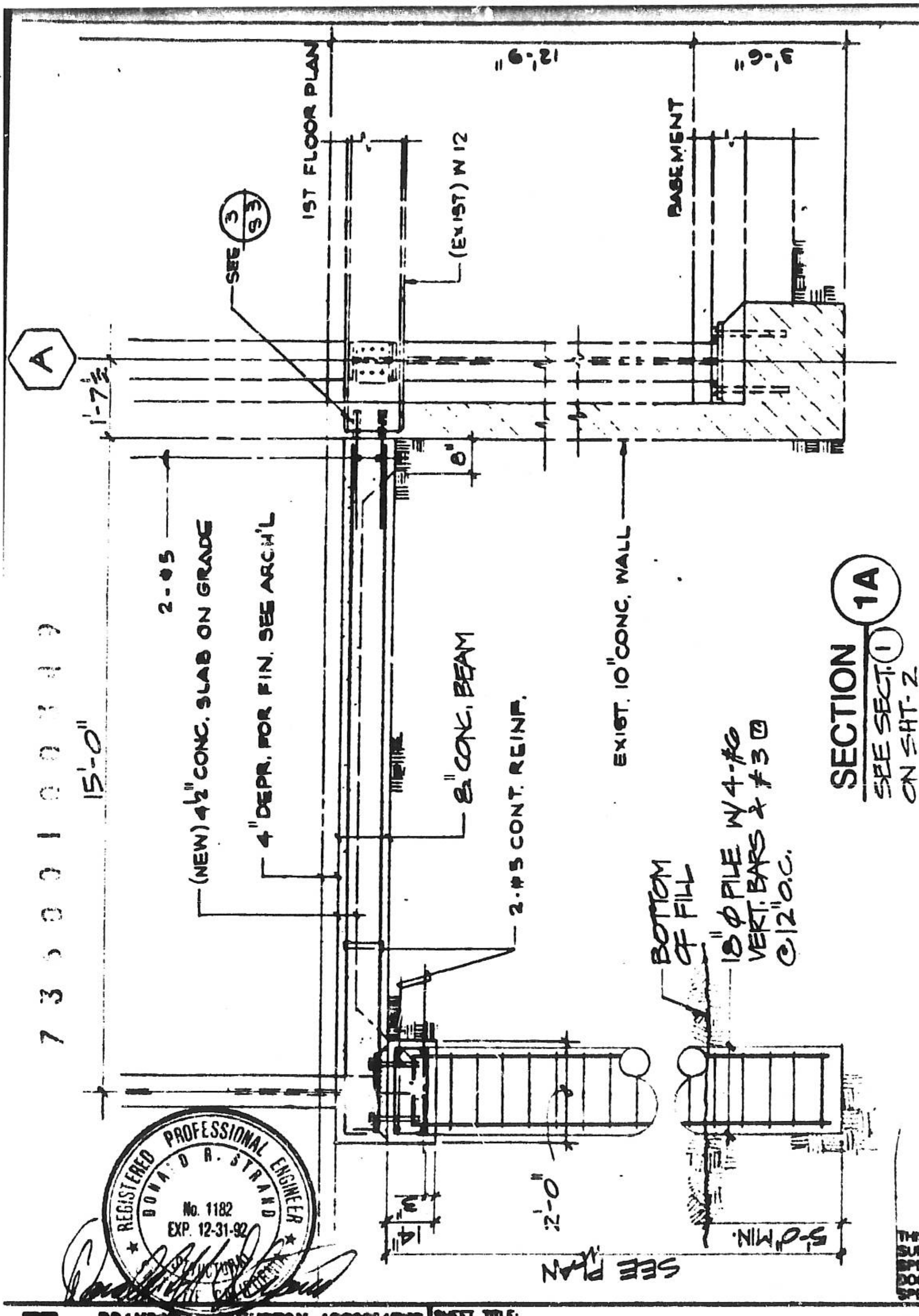
BRANDOW & JOHNSTON ASSOCIATES CONSULTING STRUCTURAL ENGINEERS	SHEET TITLE: PARTIAL FIRST FLOOR & FOUNDATION PLAN
THE LUCKMAN PARTNERSHIP, INC ARCHITECTURE/PLANNING 9220 SUNSET BOULEVARD LOS ANGELES, CA. 90069	PROJECT: CENTRAL PLAZA REMODEL 3440, 3450, & 3460 WILSHIRE BLVD. LOS ANGELES, CALIFORNIA
PROJECT No.: DATE: 3-12-98 DRAWN:	SHEET: S-A



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BRANDOW & JOHNSTON ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS

SHEET TITLE:

SECTION

THE LUCKMAN PARTNERSHIP, INC
ARCHITECTURE/PLANNING
9220 SUNSET BOULEVARD
LOS ANGELES, CA. 90069

PROJECT:

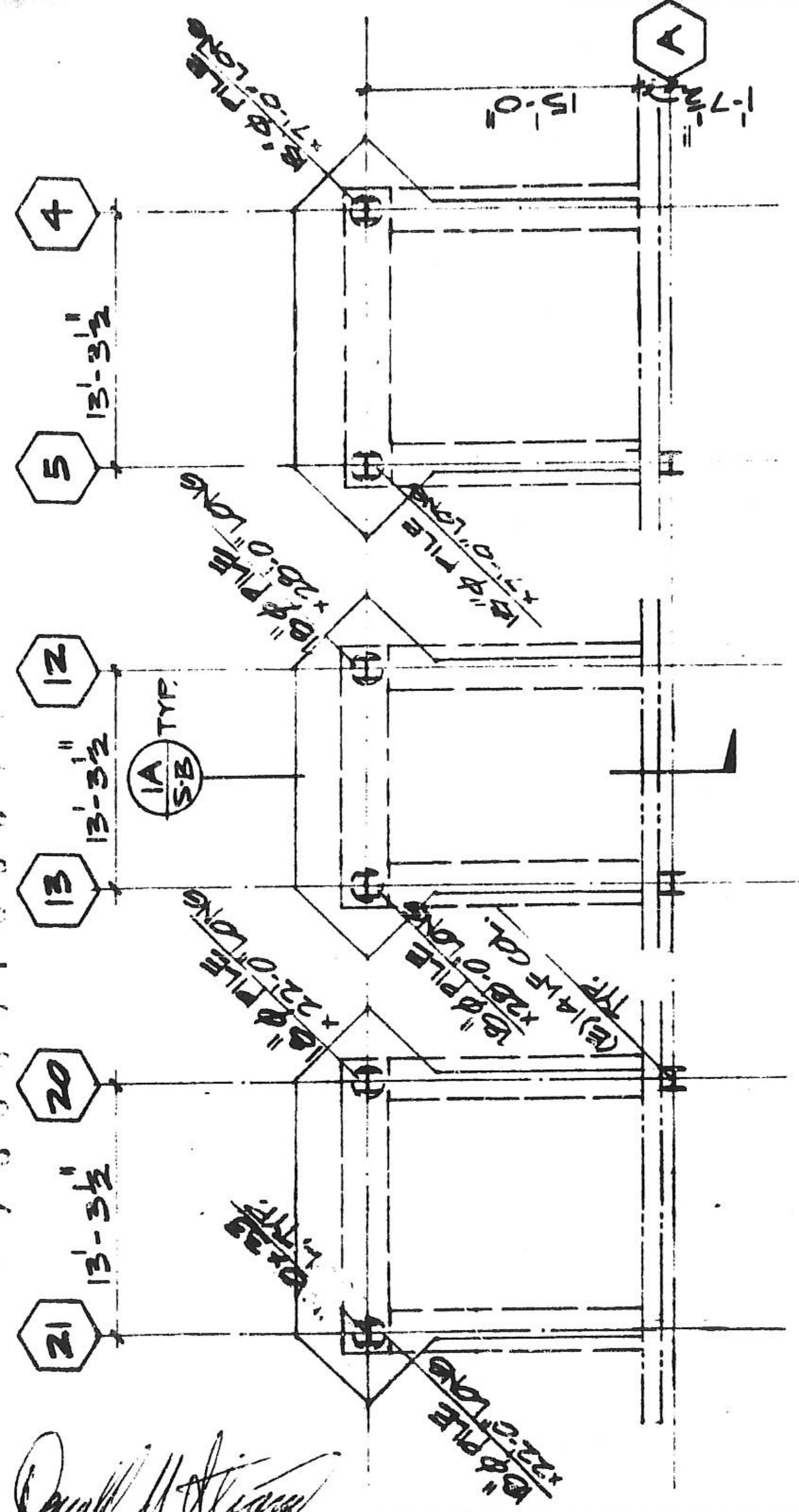
CENTRAL PLAZA REMODEL
3440, 3450, & 3460 WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

PROJECT No.:

DATE: 3-18-92

DRAWN:

SHEET: S-B



PARTIAL FIRST FLOOR AND FOUNDATION PLAN

SCALE: 1/8" = 1'-0"   SEE SHT. S-1

REGISTERED PROFESSIONAL ENGINEER
DONALD R. STRAND
No. 1182
EXP. 12-31-92
STRUCTURAL
STATE OF CALIFORNIA

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BRANDOW & JOHNSTON ASSOCIATES CONSULTING STRUCTURAL ENGINEERS 1660 W. THIRD ST., LOS ANGELES, CALIFORNIA 90017 TELEPHONE (213)-484-6950	SHEET TITLE: PARTIAL FIRST FLOOR & FOUNDATION PLAN		
THE LUCKMAN PARTNERSHIP, INC ARCHITECTURE/PLANNING 9220 SUNSET BOULEVARD LOS ANGELES, CA. 90069	PROJECT: CENTRAL PLAZA REMODEL 3440, 3450, & 3460 WILSHIRE BLVD. LOS ANGELES, CALIFORNIA	PROJECT No. DATE: 3-18-92 DRAWN:	SHEET: S-A

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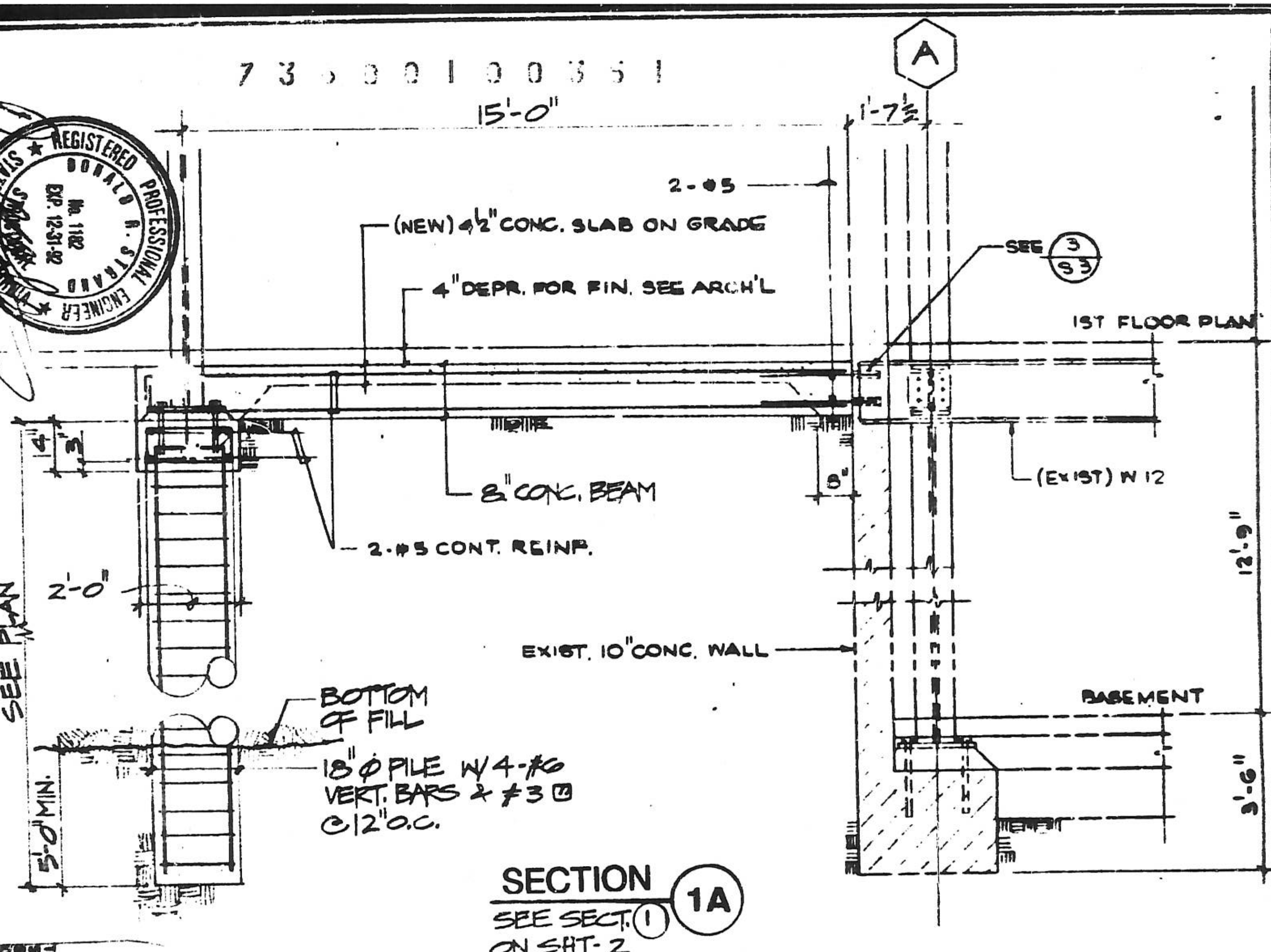

SECTION 1A

SEE SECT. 1 ON SHT-2

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REGISTERED PROFESSIONAL ENGINEER DONALD R. STRAND No. 1182 EXP. 12-31-92 STRUCTURAL STATE OF CALIFORNIA

BRANDOW & JOHNSTON ASSOCIATES
CONSULTING STRUCTURAL ENGINEERS
1660 W. THIRD ST., LOS ANGELES, CALIFORNIA 90017
TELEPHONE: (213)-484-8950

SHEET TITLE: SECTION

THE LUCKMAN PARTNERSHIP, INC
ARCHITECTURE/PLANNING
9220 SUNSET BOULEVARD
LOS ANGELES, CA. 90069

PROJECT: CENTRAL PLAZA REMODEL
3440, 3450, & 3460 WILSHIRE BLVD.
LOS ANGELES, CALIFORNIA

PROJECT No.:
DATE: 5-18-92
DRAWN:
SHEET: S-B

Earth Systems Consultants
Southern California

Foundation Engineering Division

18344 Oxnard Street
Los Angeles, CA 91356
(818) 996-1600
(213) 873-5032
FAX (818) 996-8025

March 13, 1992

F-1537-AP

Mr. David Willis
ZuFu Investments, Inc., dba
Total Properties Management Co.
3450 Wilshire Blvd., Suite 400
Los Angeles, CA 90010

PROPOSED CANOPIES/SIGN MONUMENTS
3450 WILSHIRE BOULEVARD
LOS ANGELES, CALIFORNIA

We are pleased to submit the enclosed Soil Engineering Report for the proposed canopies and sign monuments.

The proposed foundations for the westerly sign, westerly canopy, and middle canopy may consist of friction piles. The foundations for the easterly sign and canopy may be spread footings.

Caving of the pile shafts may occur during drilling. In that case, casing should be installed to maintain the holes open. Concrete may be tremied below water in the shaft.

Provision should be made for equitable payment to the contractor for variation in the depth of footings and length of piles because of expected variation in the character of the soil.

All excavations for piles must be inspected by a representative of Earth Systems Consultants during drilling. Piles shall be inspected during placing of concrete by a deputy concrete inspector.

Copies of the report and a copy of this letter with the related information have been distributed as indicated below.

Mr. David Willis
ZuFu Investments, Inc., dba
Total Properties Management Co.

Re: 3450 Wilshire Boulevard
Los Angeles, California

It has been a pleasure to work with you and the design team on this project. If there are any questions, please call.

Shaine Shahidi
Project Engineer

SS:sp

2 Copies of the Report Enclosed
4 Copies - Don Strand

7 3 0 0 1 0 0 3 7

LEGAL DESCRIPTION AND CORRESPONDING ADDRESS(ES)

Write a number in the first blank of the Legal Description and corresponding Address line to link up the legal(s) with the address(es) listed below. For example, if one address has two legal descriptions, write the number 1 in each of the first two legal description lines and the address line:

____	Tract	MP-8-113	Blk	____	Lot	Frac 93 arb 2 Frac 94-97 98 arb 1
____	Tract	____	Blk	____	Lot	____
____	Address	3450 Wilshire Blvd			Unit	____
____	Address	____			Unit	____
____	Address	____			Unit	____

DOCUMENT TYPE (Use a separate sheet for each doc type & include dates)

	Name	Document Date(s)	
____	Admin. Approval (11)	Dt ____	ENTER DATA FROM ADMN APPRVL FORM
____	Grdg Affidavit (14)	Dt ____	ENTER DATA FROM AFFIDAVIT FORM
____	Grdg Compaction File (5)	Ltr ____	Rpts ____
	Grdg Dept. Letter (7)		
____	Methane Control File	Ltr ____	Rpts ____
____	Contaminated Soil File	Ltr ____	Rpts ____
____	E.I.R. File	Ltr ____	Rpts ____
____	Other ____	Ltr ____	Rpts ____
____	Grdg Foundation File (1)	Ltr ____	Rpts ____
____	Grdg Responsibility Ltr (18)	____	
____	Grdg Seismology File (6)	Ltr ____	Rpts ____
____	Grdg Soils/Geo File (2)	Ltr 3-25-92	Rpts 3-13-92
		Rpts ____	
____	Grdg Storm Damage File (17)	Ltr ____	Rpts ____
____	Grdg Tent. Tract Doc. (51)	____	
____	Grdg Information Only (9)	____	
____	Grdg Oversized Doc (92)	X-Ref ____ Dt ____	;X-Ref ____ Dt ____

DAFS DATABASE COMMENT

Any comment printed on the line below will be typed in the DAFS Comment field:

____

DOCUMENT PREPARATION AND RETENTION

Reviewed by Cm   Document(s) Stored in Retention Box No. 1080

(PWP041591GIS/SWP) RS.30.91

11

B&S B-17 (R 6/86)

CO REF #: MP8-113

City of Los Angeles

132-B-197

DEPARTMENT OF BUILDING AND SAFETY

Grading Division CD# 4 / LA

District MAP# | Log No. 28161

ADDRESS APPROVED | [signature] 3/23/92 Signature/Date

APPLICATION FOR REVIEW OF TECHNICAL REPORTS AND IMPORT-EXPORT ROUTES

INSTRUCTIONS

A. Address all communications to the Grading Division, Department of Building and Safety, Room 460A, City Hall, Los Angeles, California 90012-4869. Phone (Area Code 213) 485-3435.

B. Obtain address approval from the Department of Public Works prior to submittal.

C. Submit 2 copies (4 for fault study zone) of reports and 3 copies of application with items (1) through (10) completed.

D. Check should be made to the Department of Building and Safety. Note: Please Print

(1) LOT: Frac 97 & 6 & 98 & Frac of 38 to 92 | BLK.: — | TRACT: Wilshire Harvard Heights | COUNTY REF. NO.: MP8-113

A.L.M. ... NE ... SW ... T1S R14W

PROJECT ADDRESS 3450 Wilshire Bl

APPLICANT Swinerton & Walberg Co.

Address ... Wilshire Blvd. Suite 301

City Los Angeles

Phone (Daytime) 351-3470 Zip ...

(3) OWNER Zen Investment Inc, Total Properties Mgt Co.

Address 3450 Wilshire Blvd Suite 400

City Los Angeles Zip 90005

Phone (Daytime) 386-6336

(5) Report(s) Prepared by Earth Systems Consultants

(6) Report Date(s) March 13, 1992

(7) Status of project: ☐ Proposed ☒ Under Construction ☐ Storm Damage

(8) Previous site reports? ____ If yes, give date(s) of report(s) and name of company(s) who prepared report(s).

(9) Previous Department actions? ____ If yes, please give dates and attach a copy to expedite processing.

Dates

(10) Signature of applicant Paul Terwey Position Project Engineer

(DEPARTMENT USE ONLY)

REVIEW REQUESTED & PROCESSING	FEES	REVIEW REQUESTED & PROCESSING	FEES
☐ Foundation Investigation		☐ Seismology report per 91.2305(d)	
☒ Soils Engineering	200 —	☐ Environmental Assessment	
☐ Geology		☐ Import-Export Route	
☐ Combined Soils Engr. & Geol.		☐ Division of Land	
☐ Supplemental		Sub-total	200
☐ Combined Supplemental		SYS 12 One-Stop Surcharge	4

THE REPORT IS ☒ APPROVED WITH CONDITIONS ☐ NOT APPROVED TOTAL FEE 216

DEPARTMENT ACTION BY: ____ For Geology ____ Date Burr E. Peck For Soils & Foundation 3-25-92 Date

☒ Conditions of Approval ☐ Reasons for Non-Approval ☐ See Attached letter ☒ Supplemental Sheet 1 Attached

① Existing fill shall not be used for vertical or lateral support.

② Conditions Nos. 2, 9, 20, 22, 24, 26, 27, 30 on the attached "Supplemental Conditions for Foundation Investigation Reports".

(Continued Over)

03/23/92 ... GRADING REPORT SYS DEV ONE STOP TOTAL CHECK 216.00 (Cashier Use Only)

DEPARTMENT USE ONLY

Fee Due 216.00

Fee Verified [initials]

Date 3-23-92

DISTRIBUTION

☒ Owner ☒ Applicant ☐ ____

☒ Soil Engineer ☐ Geologist ☐ Board files ☒ Tract file

☒ LA Plan Check ☐ VN ☐ WLA ☐ SP/WLA

☒ LA Inspection ☐ VN ☐ WLA ☐ SP/WLA

☐ BMI ☐ BI

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY

Log No. 28161
3450 WILSHIRE BL
3-25-92

SUPPLEMENTAL CONDITIONS FOR FOUNDATION INVESTIGATION REPORTS

1. A grading permit shall be obtained.

2. Existing uncertified fill shall not be used for support of footings, floor slab, or proposed fill.

3. No fill shall be placed until the City Grading Inspector has inspected and approved the bottom excavations.

4. The fill shall be placed under the inspection and approval of the responsible Engineer. A compaction report shall be submitted to the Department upon completion of the compaction.

5. If import soils are used, no footings shall be poured until the responsible Engineer has submitted a compaction report and in place shear test data and settlement data to the Department and obtained approval.

6. Compacted fill shall extend beyond the footings a minimum distance equal to the depth of the fill below the footings.

7. Prior to the issuance of any permit, the owner shall file a notarized Covenant and Agreement with the Office of the Los Angeles County Recorder acknowledging the proposed pavement will be constructed on uncertified fill and future settlement may occur.

8. The building design shall incorporate provision for anticipated differential settlements in excess of one-fourth inch.

9. The responsible Engineer shall review and approve the foundation plan and/or the Excavation/Shoring plan prior to the issuance of any permits.

10. A supplemental report shall be submitted to the Grading Division containing recommendations for shoring, underpinning and sequence of construction if any excavation would remove the lateral support of the public way or adjacent structures.

11. Prior to issuance of any permit, the owner of the subject site shall record a notarized affidavit with the office of the Los Angeles County Recorder which will inform future owners of the subject site that the lateral support of a portion of the building footings on the adjoining property is provided by the subterranean walls of the building on the subject site.

12. Approval from the Department of Public Works shall be obtained for any excavation that would remove the lateral support of the public way.

13. All roof and pad drainage shall be conducted to the street by gravity.

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

15. The design of the subdrainage system required to prevent possible hydrostatic pressure behind retaining/basement walls shall be approved by the responsible Engineer prior to issuance of the building permit. Installation of the subdrainage system shall be inspected and approved by the Soil Engineer.

16. Basement excavations shall be performed under the continuous inspection and approval of the responsible Engineer.

17. Installation of shoring, underpinning, and/or slot cutting excavations shall be performed under the continuous inspection and approval of the responsible Engineer.

September 1989

18. Basement walls and slab shall be waterproofed with an L.A. City approved "Below-grade waterproofing" material with a research report number.

19. If the actual foundation design loads do not conform to the foundation loads assumed in the report, the responsible Engineer shall submit a supplementary report containing specific design recommendations for the heavier loads to the Department for review and approval prior to issuance of a permit.

20. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety.

21. Prior to the issuance of any permit which 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.

22. A copy of the foundation report and/or supplements and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above foundation report and/or supplements to the Building Department Plan Checker prior to issuance of the permit.

23. All pile driving shall be performed under continuous inspection and approval of the responsible Engineer. A log of pile driving shall be kept and a copy submitted to the Department along with written certification that the work supervised meets the conditions of the report. Such supervision does not waive the required inspection by the City Building Inspector.

24. All friction pile drilling and installation shall be performed under the continuous inspection and approval of the responsible Engineer.

25. Spread footings and slab-on-grade shall be designed for expansive soil conditions.

26. Pile and/or caisson foundation ties are required by Code Section, 91.2312(j)3B. Exceptions and modification to this requirement are provided in Rule of General Application 662.

27. When water over 3 inches in depth is present in drilled pile holes, a concrete mix with a strength p.s.i. of 1000 over the design p.s.i. shall be tremied from the bottom up; an admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included.

28. The installation and testing of tie-back anchors shall comply with the attached sheets titled "Requirements for Tie-back Earth Anchors".

29. Provide a notarized letter from adjoining property owners allowing tie-back anchors on their property.

30. Prior to the pouring of concrete, a representative of the consulting Foundation Engineer shall inspect and approve the footing excavations. He shall post a notice on the job site for the City Building Inspector and the contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City building Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Department upon completion of the work.

31. Prior to excavation, an initial inspection shall be called at where time sequence of shoring, protection fences and dust and traffic control will be scheduled.