Appendix G
Geotechnical Report

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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May 13, 2024
File Number 22517

Burbank Housing Corporation
1819 Grismer Avenue
Burbank, California 91504

Attention: Sylvia Moreno

Subject: Geotechnical Engineering Investigation
Proposed Affordable Residential Development
2321 through 2335 North Fairview Street, Burbank, California

Dear Ms. Moreno:

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

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

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED AFFORDABLE RESIDENTIAL DEVELOPMENT
2321 THROUGH 2335 NORTH FAIRVIEW STREET
BURBANK, CALIFORNIA

INTRODUCTION

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

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

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. In addition, the architectural drawings prepared by Y&M Architects, dated January 1, 2024, were reviewed for the preparation of this investigation. The proposed project consists of construction of a 60-unit affordable residential development. The proposed structure will be three to four stories in height, built over one subterranean parking level. It is anticipated that the finished grade of the subterranean parking level will extend 12 feet below the proposed ground level. The anticipated location, alignment and depth of the proposed structure is illustrated in the enclosed Plot Plan, Site Plan and Cross Section A-A'.

Structural information is not available at this time. Column loads are estimated to be between 300 and 600 kips. Wall loads are estimated to be between 5 and 20 kips per lineal foot. Grading will consist of excavations to an estimated depth of 15 feet below the existing grade for construction of the proposed subterranean parking level, including 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 site is located at 2321 through 2335 North Fairview Street in the City of Burbank, California. The site is rectangular in shape and approximately 0.62 acre in area. The Project Site is bounded by a two-story residential building built over a partial basement level to the north, North Fairview Street to the east, a two-story residential building to the south, and three two-story residential buildings and a one-story residential building to the west. The site is shown relative to nearby topographic features on the attached Vicinity Map.

The site is relatively level. According to the Survey Plan prepared by KPFF dated March 31, 2023, site elevations range from 670.8 feet on the east to 673.4 feet on the west. The site is currently developed with seven 1 to 2-story residential buildings and at-grade parking lots.

The vegetation on the site consists of several mature trees, shrubs, and grasses. Drainage across the site appears to be by sheetflow to the city streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on March 22, 2024, by excavating three borings. The exploratory borings varied in depth from 30 to 60 feet below the existing grade. The exploration was prosecuted with the aid of a truck-mounted drilling machine using 8-inch diameter hollow-stem augers. The exploration locations are shown on the Plot Plan and Site Plan and the geologic materials encountered are logged on Plates A-1 through A-3.

The location and elevation of the exploratory borings was approximated from information included in the Survey Plan prepared by KPFF dated March 31, 2023. The location and elevation of the borings should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered in all three exploratory borings, to an approximate depth of 3 feet below the existing grade. The fill consist of sand, which is dark, yellowish, and grayish brown in color, moist, medium dense, fine grained.

Native alluvial soils consist primarily of sand, with some silty sand layers. The native soils are dark, yellowish, and grayish brown in color, moist, medium dense to very dense, and fine to coarse grained, with few cobbles. The geologic materials consist of sediments deposited by river and stream action typical to this area of Los Angeles County. More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during site exploration to maximum depth of 60 feet. The historic high groundwater level was established by review of California Geological Survey Seismic Hazard Evaluation Report, Plate 1.2 entitled “Historically Highest Ground Water Contours”. Review of this plate indicates that the historically highest groundwater level is on the order of 58 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 that encounter granular, cohesionless soils will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

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 Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

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. As revised in 2018, The Act defines

"Holocene-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 11,700 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 Holocene-Active 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 Holocene-active or Pre-Holocene faults underlie the Project Site. In addition, the Project Site is not located within the Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the Project 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 Earthquake Zones of Required Investigation Map (CGS, 2016) 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.

Based on the density of the soils underlying the site, absence of groundwater within the upper 60 feet strata, and the mapped depth to the historically highest groundwater level, the soils underlying the site are not considered capable of liquefaction during the ground motion expected during the design-based earthquake. Furthermore, the site soils are not considered susceptible to liquefaction-related issues, such as lateral spreading and surface manifestation.

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 lies within the inundation boundaries of the Hansen Dam. It should be noted, however, that Hansen Dam is primarily a flood control basin, and is rarely full. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

Approximately 3 feet of fill materials were encountered during exploration. The existing fill materials are considered unsuitable for support of the foundations, floor slabs, or additional fill. However, it is anticipated that the fill materials will be removed during excavation of the proposed subterranean parking level. The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subterranean subgrade.

Groundwater was not encountered in the exploratory excavations to a maximum depth of 60 feet below existing site grade. Based on review of Seismic Hazard Evaluation Report 016 (CDMG, 1998, revised 2006), the historically highest groundwater level for the site corresponds to a depth of 58 feet below the existing grade. It is anticipated that the finished floor elevation of the subterranean level will extend to a depth of 12 feet below the existing grade. Therefore, the finished floor elevation of the proposed structure is expected to be located above the historically highest groundwater level and the current groundwater level.

Grading is expected to consist of excavations in the order of 15 feet below the existing site grade for the construction of the proposed subterranean parking level and foundation elements. Due to close proximity to property lines and adjacent properties, it is anticipated that the installation of a temporary shoring system will be required in order to maintain a stable excavation. Shoring requirements are described in the “Temporary Excavations” section of this report.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from 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

2022 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the Project Site is classified as Site Class D, which corresponds to a “Stiff Soil” Profile, according to ASCE 7-16 standard. This information and the site coordinates were input into the OSHPD seismic utility program in order to calculate ground motion parameters for the site.

2022 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2022
ASCE Design Standard	7-16
Risk Category	II
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.990g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.990g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.327g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.677g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.151g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.767g*

** These values are determined based on the tabulated values shown on ASCE 7-16, Table 11.4-2 for the Long-Period Site Coefficient, F_v. It should be noted that the exception to performing a site-specific hazard analysis for structures on Site Class D for values of S_1 greater than or equal to 0.2 presented by ASCE 7-16 may be followed, provided the conditions outlined in the ASCE 7-16 Section 11.4.8 (including Supplement 3 of the ASCE 7 Standards) are implemented in the structural design and analyses. This exception recommends that, where a site-specific ground motion hazard analysis is not required, the S_{M1} determined by eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this standard. This would require that the S_{M1} and S_{D1} values provided in the above table are increased by 50%.*

EXPANSIVE SOILS

The onsite geologic materials are in the very low expansion range. The Expansion Index was found to be 1 and 2 for bulk samples representative of the on-site soils.

WATER-SOLUBLE SULFATES

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

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

The water-soluble sulfate content of the onsite geologic materials was tested by California Test 417. The water-soluble sulfate content was determined to be less than 0.1% percentage by weight for the soils tested. Based on the most recent revision to American Concrete Institute (ACI) Standard 318, the sulfate exposure is considered to be negligible for geologic materials with less than 0.1%. Therefore, there are no restrictions on the cement types utilized for concrete foundations in contact with the site soils.

GRADING GUIDELINES

The following guidelines are provided for any miscellaneous grading that may be required for the project, such as footing or trench backfill, or subgrade preparation.

Site Preparation

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

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

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

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

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

Compaction

All fill materials should be mechanically compacted in layers not more than 8 inches thick (uncompacted thickness). The materials placed should be in moisture conditions to within three percent of the optimum moisture content of the particular material placed. All fill materials shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with 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 compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris, oversized materials 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 20. 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 of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

FOUNDATION DESIGN

Conventional Foundations

The proposed structure may be supported by conventional foundations bearing in the native alluvial soils expected at the subterranean subgrade. Continuous foundations may be designed for a bearing capacity of 3,000 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

Column foundations may be designed for a bearing capacity of 3,500 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 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 6,000 pounds per square foot.

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

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structure may bear in native soils, or properly recompacted fill materials. Continuous footings may be designed for a bearing capacity of 2,000 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

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

Foundation Reinforcement

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

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.40 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 1,500 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

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

Foundation Observations

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

RETAINING WALL DESIGN

Based on the estimated depth of the proposed subterranean level, it is anticipated that retaining walls up to 12 feet in height will be required for the project. As a precautionary measure, recommendations for the design of underground retaining walls up to a height of 15 feet have been provided herein. Retaining walls may be designed as indicated below, depending on whether the walls will be restrained or cantilevered. Retaining wall foundations may be designed in accordance with the provisions of the "Foundation Design" section of this report.

Additional lateral pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures. Depending on the final depth of the proposed retaining walls, the residential structure located to the south may impose a surcharge load on the proposed retaining walls. Information regarding the foundation loading of this neighboring structure will be necessary to analyze the anticipated surcharge.

Vehicular traffic is expected in the vicinity of the proposed structure. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to the 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 the assumed 300 pounds per square foot traffic surcharge. If traffic surcharge is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Restrained Retaining Walls

Restrained subterranean retaining walls up to 15 feet in height and supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below.

RESTRAINED BASEMENT WALLS		
	AT-REST EARTH PRESSURE	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure)**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 12 feet	59	35*
12 to 15 feet	59	39*

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 lateral pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures.

Dynamic (Seismic) Earth Pressure

The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. 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 load combination equations from the building code, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition. If retaining walls are less than 6 feet in height, the dynamic (seismic) earth pressure may be omitted.

Cantilever Retaining Walls

Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 35 pounds per cubic foot for walls retaining up to 12 feet of earth. This lateral pressure should be combined with dynamic (seismic) earth pressure if retaining wall exceeds 6 feet in height.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional lateral pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures.

Retaining Wall Drainage

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

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

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

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 the retaining walls be waterproofed.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 60 feet. Therefore, the only water which could affect the proposed retaining walls would be irrigation water and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

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

Waterproofing

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

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

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

TEMPORARY EXCAVATIONS

Excavations on the order of 15 feet in depth below the existing site grade are anticipated for the construction of the proposed subterranean level. This includes the depth of the proposed foundation elements. The excavations are expected to expose fill and medium dense to dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations over 5 feet or which will be surcharged by adjacent traffic or structures should be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 15 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

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

Excavation Observations

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

SHORING DESIGN

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

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

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than two diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation may be assumed to be 600 pounds per square foot per foot of depth, up to a maximum of 6,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Caving should be anticipated during drilling. Casing will be required should caving be experienced. If a 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.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 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 below the plane of excavation may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Lagging

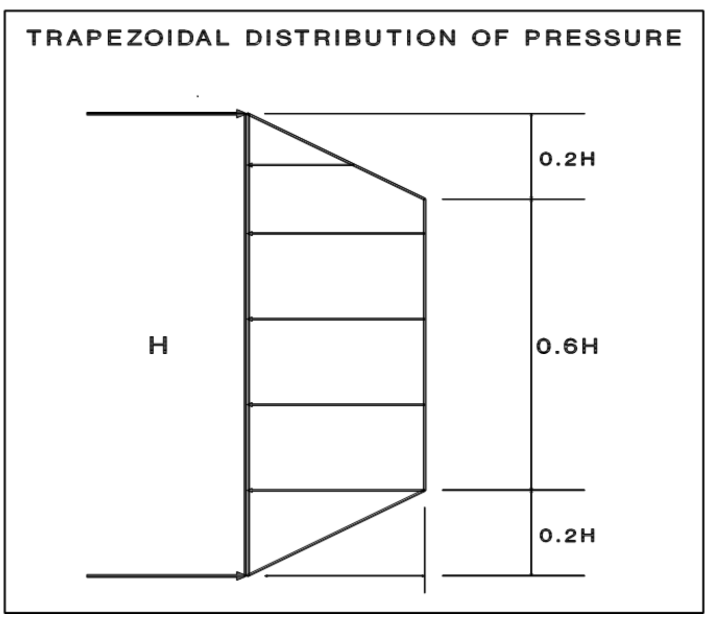
Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but may be limited to a maximum of 400 pounds per square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

Lateral Pressure

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

HEIGHT OF SHORING "H" (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 15	30
15 to 20	34

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



Shoring restrained by bracings or tiebacks and supporting a level backslope may be designed utilizing a trapezoidal distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 15	20H
15 to 20	22H

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

Monitoring

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

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 30 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge. Anchors should be placed at least 6 feet on center to be considered isolated.

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors, provided the design does not rely on end-bearing plates to provide the necessary capacity. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Where caving of the anchor shafts is experienced, the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Tieback Anchor Testing

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

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

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

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

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

Raker Foundations

Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 5 feet in width and length as well as 5 feet in depth into native alluvial soils. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

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

Monitoring

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

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

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations ensure 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 and outdoor flatwork should be a minimum of 4 inches in thickness and should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. Slabs-on-grade and outdoor 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 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, where necessary, it is recommended that a qualified consultant should 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 on various components of the structure.

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

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

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-06, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Where a vapor retarder/barrier is used, it should be placed on a level and compact subgrade. Precautions should be taken to protect the vapor retarder/barrier from damage during installation of reinforcing, utilities, and concrete. The use of stakes driven thought the vapor retarder/barrier should be avoided. Repair any damaged areas of the vapor retarder/barrier prior to concrete placement.

Concrete Crack Control

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

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

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent 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 laboratory maximum dry 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

For concrete paving, the following sections are recommended:

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

Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way. For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacing would provide greater crack control. Joints at curves and angle points are recommended.

Aggregate base should be compacted to a minimum of 95 percent of the laboratory maximum dry density according to the most recent revision of ASTM D 1557. 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.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress.

SITE DRAINAGE

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

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

STORMWATER DISPOSAL

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

Percolation Testing

Percolation testing was conducted in Boring 1, which was drilled to a depth of 60 feet below the existing grade. At the completion of drilling, a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a slotted PVC pipe within the lower 30 feet of the borehole, and solid PVC pipe to the top of the borehole. A sand pack consisting of #3 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

After the casing was installed, the borehole was filled with water for the purpose of pre-soaking for a minimum of 2 hours. After presoaking, the borehole was refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first. The percolation testing was performed within the native alluvial soils encountered between depths of 30 and 60 feet.

Based on results of the percolation testing and following the LA County method described in Guidelines for Low Impact Development Stormwater Infiltration (GS200.1 dated June 20, 2021), a percolation rate of 33.1 inches per hour was obtained. This percolation rete may be utilized for design of the proposed deep infiltration system (drywell).

The Proposed System

A specific stormwater infiltration system has not been discussed for the project. Preliminarily, it is anticipated that a suitable infiltration system may consist of a drywell system. The final location and design of the proposed infiltration system shall be reviewed and approved by this office prior to construction to evaluate whether the intent of the recommendations provided by this firm are satisfied.

Recommendations

Based on the results of the exploration, testing and research, it is the finding of this firm that on-site stormwater infiltration is feasible for the site. A suitable stormwater infiltration system may consist of a drywell system. The potential stormwater infiltration system is not expected to impact the proposed development, or existing neighboring development, provided the advice and recommendations presented herein are implemented during design and construction.

Because the proposed structure will occupy the majority of the site, it is anticipated that any potential infiltration drywells would be installed within the footprint of the proposed structure, below the subterranean level. But where sufficient space is available, the drywell may also be installed outside the proposed structure. It is recommended that the edge of any potential drywell system should maintain a minimum horizontal setback of 15 feet away from private property lines.

Stormwater infiltration shall only occur in the soils located below the primary zone of foundation influence. Based on anticipated size, depth, and loading distribution of the proposed column foundations, it is the opinion of this firm that the primary zone of foundation influence for the proposed structure would extend to a depth of 15 feet below the bottom of the proposed foundations. Therefore, it is recommended that stormwater infiltration should only occur in the native alluvial soils located at, or deeper, than 15 feet below the bottom of the deepest foundation adjacent to the potential drywell.

Soils located within the primary zone of foundation influence should not become wet or saturated as a result of a drywell. It is anticipated that a settling chamber will be installed within this primary zone of foundation influence; therefore, the seams and bottom of the settling chamber should be adequately sealed to prevent infiltration at this zone.

State regulations require that the bottom of infiltration units maintain a minimum vertical distance of 10 feet above the groundwater level. Groundwater was not encountered at the site during exploration, conducted to a depth of 60 feet below grade. Therefore, it is recommended that the drywell system does not extend deeper than 50 feet below the existing grade.

Any potential drywells should be installed centered in between surrounding foundations. Depending on their final location, it is anticipated that the settling chamber of the drywell may be surcharged by proposed adjacent foundations, in which case the chamber should be designed to withstand this additional surcharge load. The final location of the proposed drywells shall be reviewed and approved by this office prior to construction.

The Project Site is not located in an area considered susceptible to liquefaction. The proposed stormwater infiltration system will not be located in hillside area, and no slopes are nearby. The onsite soils are in the very low expansion range and are not susceptible to significant hydroconsolidation.

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

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

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

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

- Caving should be anticipated during drilling of the drywell. Where caving occurs during drilling of the drywell, it will be necessary to utilize casing to maintain an open shaft.

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 engineering 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 recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or 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 report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

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

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with 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.

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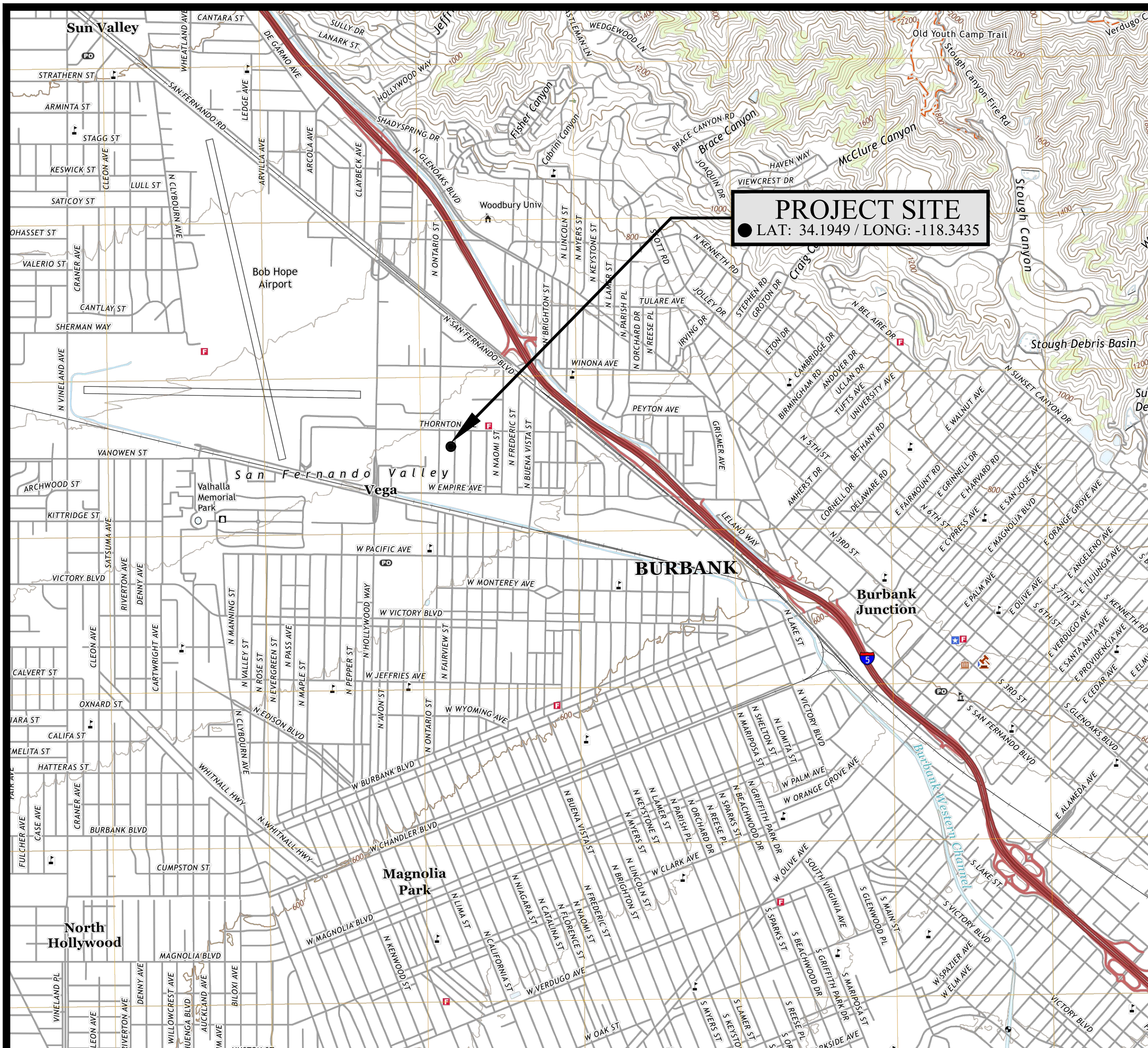
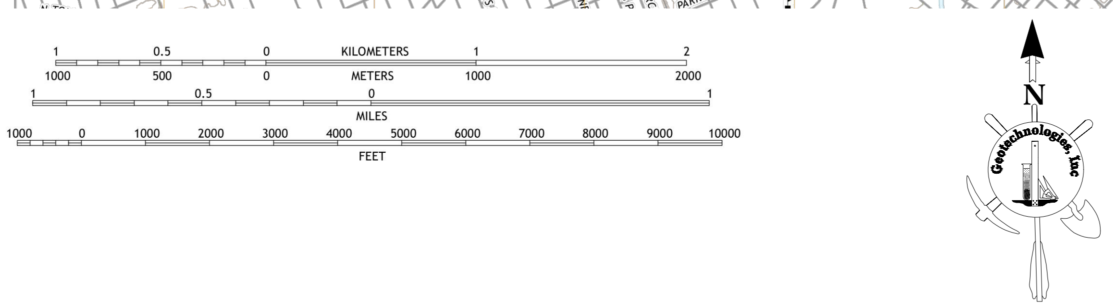
United States Geological Survey, 2014, Unified Hazard Tool (Online Application), https://earthquake.usgs.gov/hazards/interactive/.





REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 - MINUTE SERIES BURBANK, CA QUADRANGLE 2022

VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION

FILE NO: 22517

Plot Date: Apr. 30, 2024

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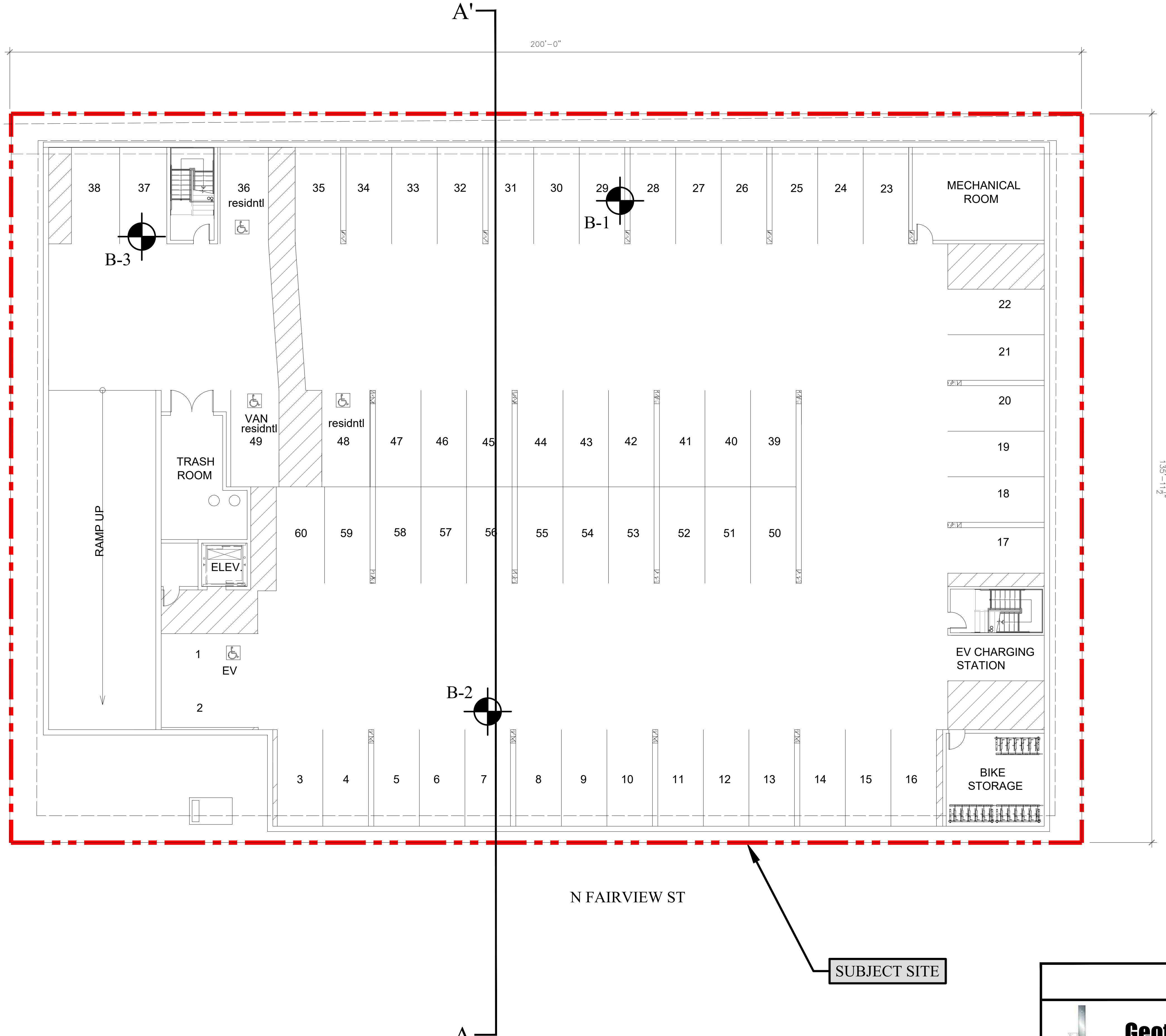


SCALE IN FEET

0 10 20 40

LEGEND

B-3 LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)

A A' CROSS SECTION

LIMITS OF PROJECT SITE

PLOT PLAN

Geotechnologies, Inc.

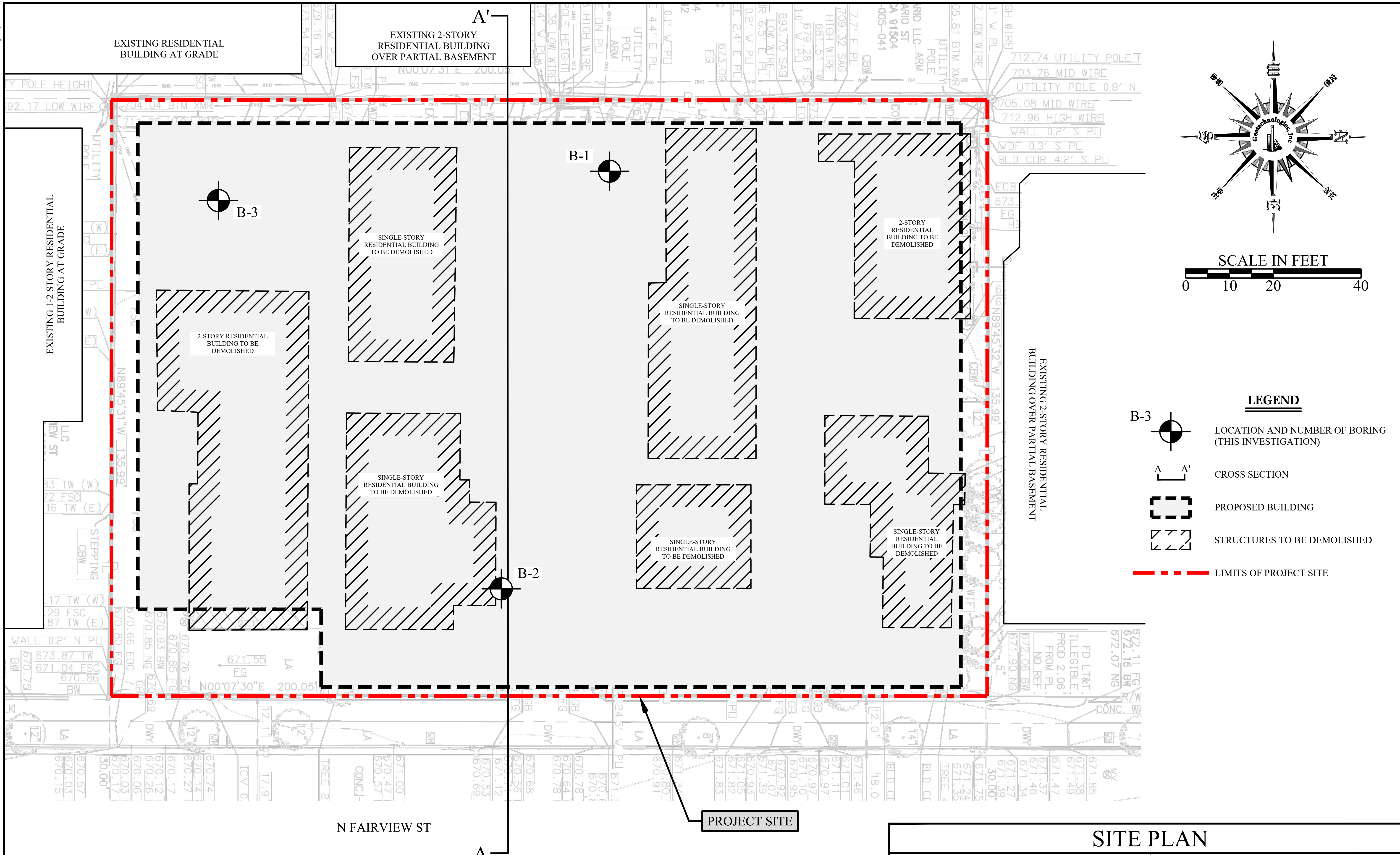
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION

2321-2335 N. FAIRVIEW ST., BURBANK

Drawn by: JD	File No.: 22517
Date: April 2024	

REFERENCE: SUBTERRANEAN GARAGE (SHEET A-003) BY Y&M ARCHITECTS DATED 01/01/2024



SITE PLAN

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION
2321-2335 N. FAIRVIEW ST., BURBANK

Drawn by: JD	File No.: 22517
Date: May 2024	

REFERENCE: ALTA/NSPS LAND TITLE SURVEY BY kpff DATED 03/31/2023 &
SITE PLAN (SHEET A-002) BY Y&M ARCHITECTS DATED 01/01/2024

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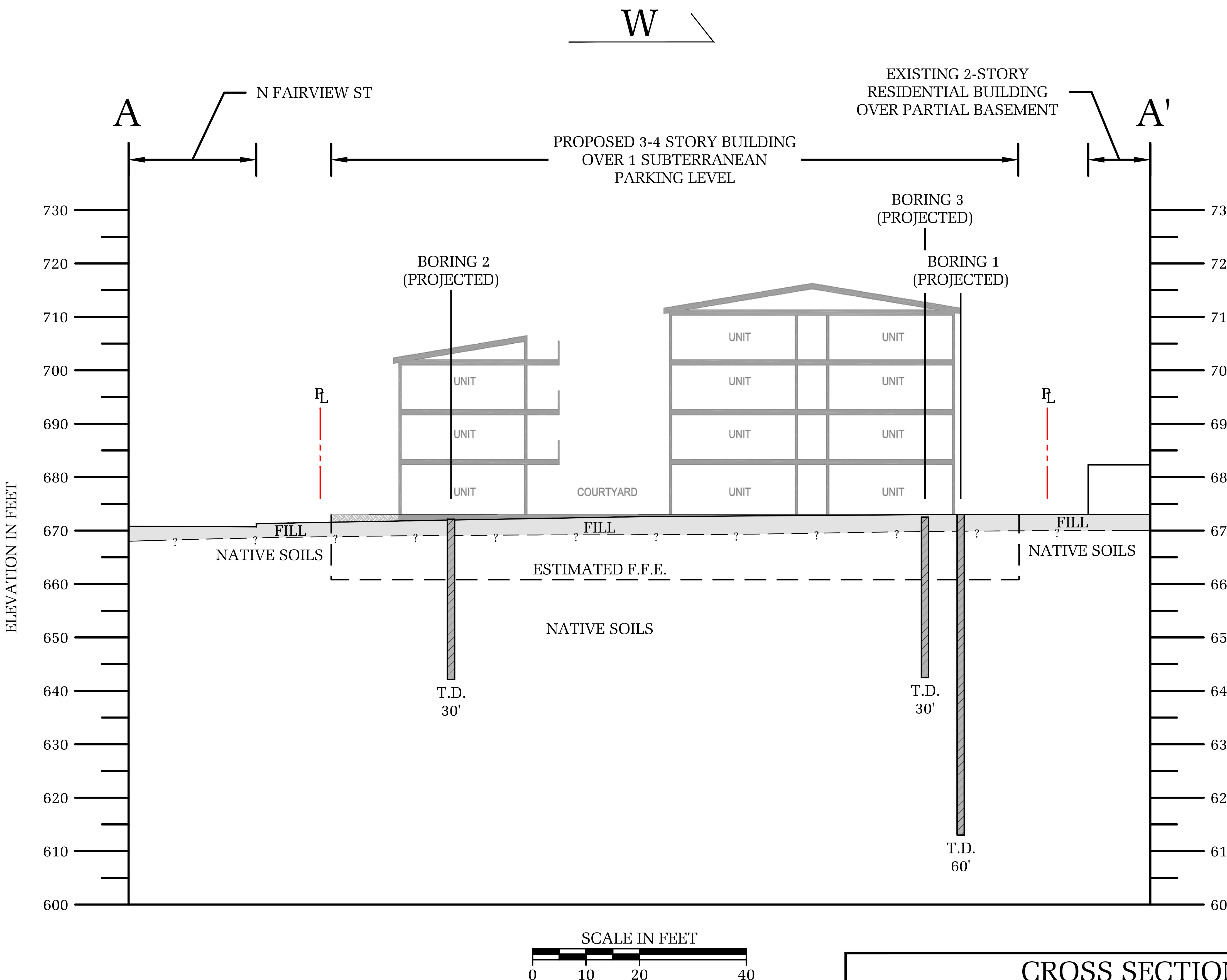


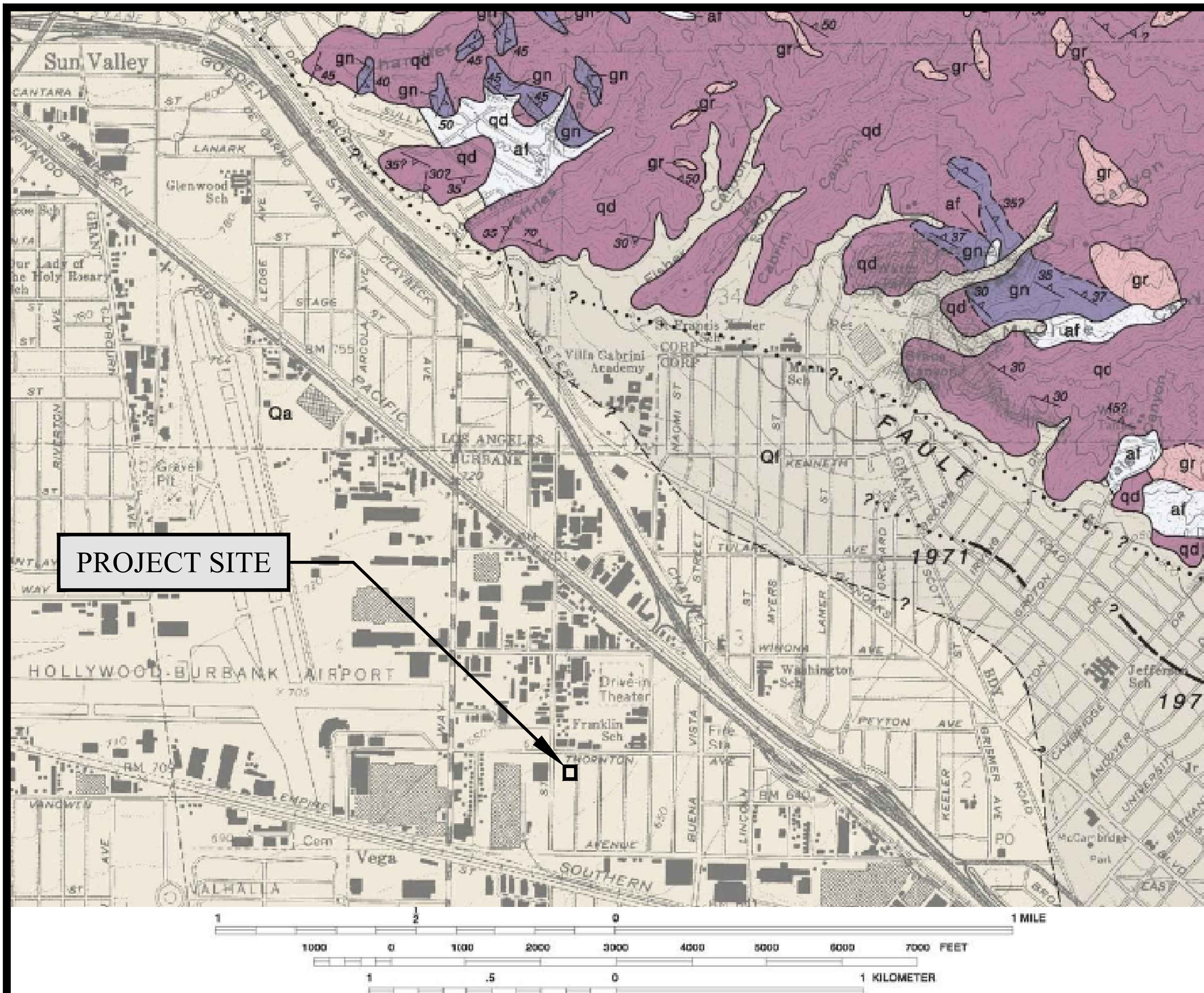
REFERENCE: SUBTERRANEAN GARAGE (SHEET A-003) BY Y&M ARCHITECTS DATED 01/01/2024

CROSS SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION
2321-2335 N. FAIRVIEW ST., BURBANK

Drawn by: JD	File No.: 22517
Date: May 2024	



LEGEND

af: Surficial Sediments - Artificial cut and fill

Qa: Alluvium: gravel, sand and clay of valley areas

Qf: Alluvial fan gravel derived from Verdugo Mountains; may be in par equivalent to Qof

gr: Leucocratic Granitic Rocks - Nearly white, massive, medium to fine-grained granitic rocks mostly of quartz monzonite-granodiorite composition; essentially of quartz, potassic feldspar and sodic plagioclase feldspar, sparse biotite; complexly intruded into quartz diorite (qd) and gneiss (gn), moderately coherent; not everywhere differentiated from qd

qd: Quartz Diorite - Gray, medium-grained quartz diorite with variations to diorite; composed of sodic plagioclase feldspar, biotite, hornblende, and minor quartz; massive to vaguely gneissoid; somewhat incoherent where weathered, complexly intruded by gr

gn: Gneissic Rocks - Medium to dark gray biotite-quartz-feldspar gneiss; hard coherent, but much fractured; ranges from thin-layered gneiss to somewhat incoherent gneissoid quartz diorite or biotite diorite, includes small remnants of metaquartzite, biotit schist-gneiss locally containing graphite and norhfelsic gneiss, containing calc-silicate minerals (garnet, epidote, diopside, and sillimanite)

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

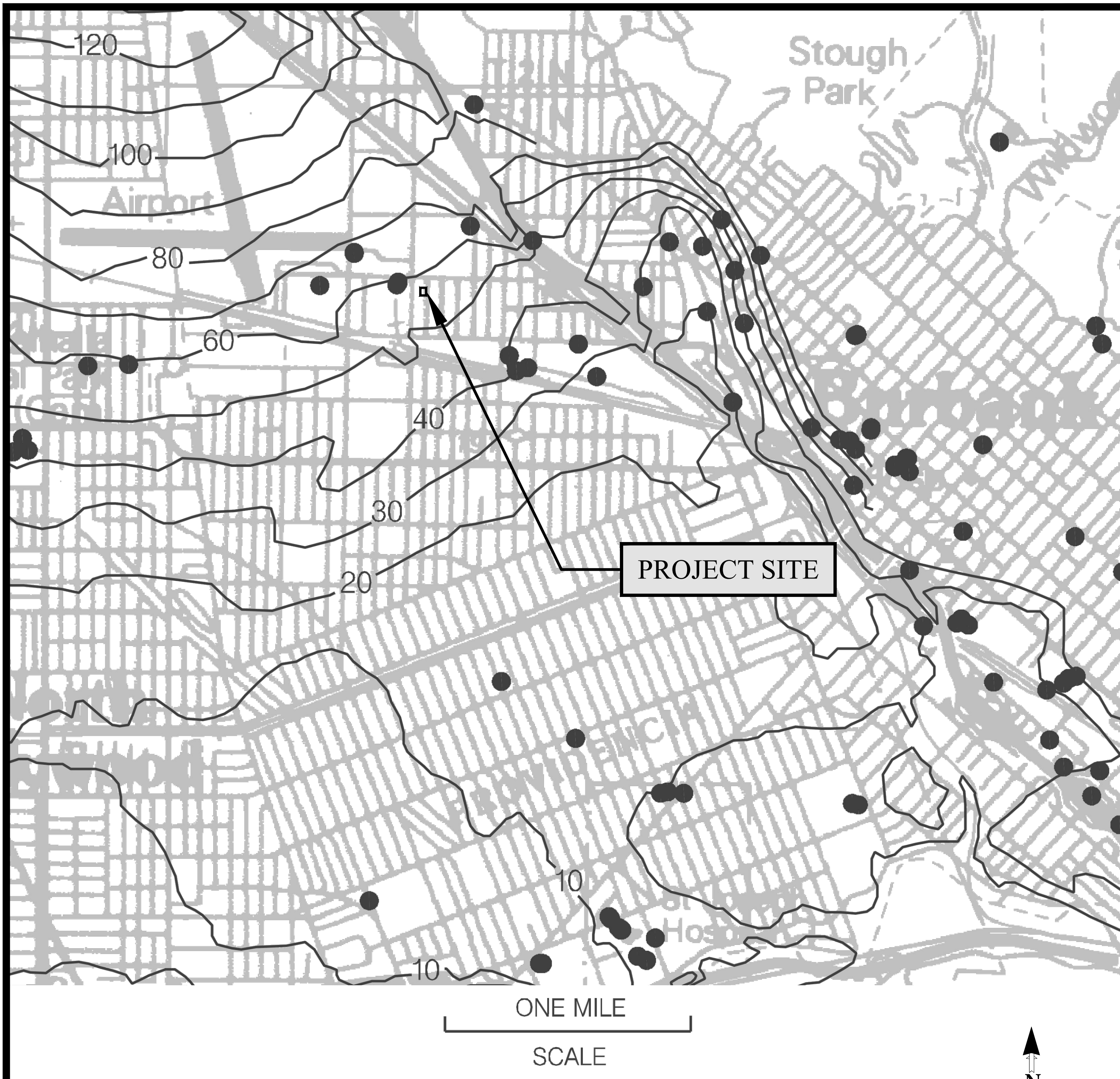
REFERENCE: T.W. DIBBLEE (EDITED 2010) GEOLOGIC MAP OF SUNLAND AND NORTH ½ BURBANK QUADRANGLES (#DF-32)

GEOLOGIC MAP (DIBBLEE)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION

FILE NO: 22517



LEGEND

● Borehole Site

30 Depth to ground water in feet

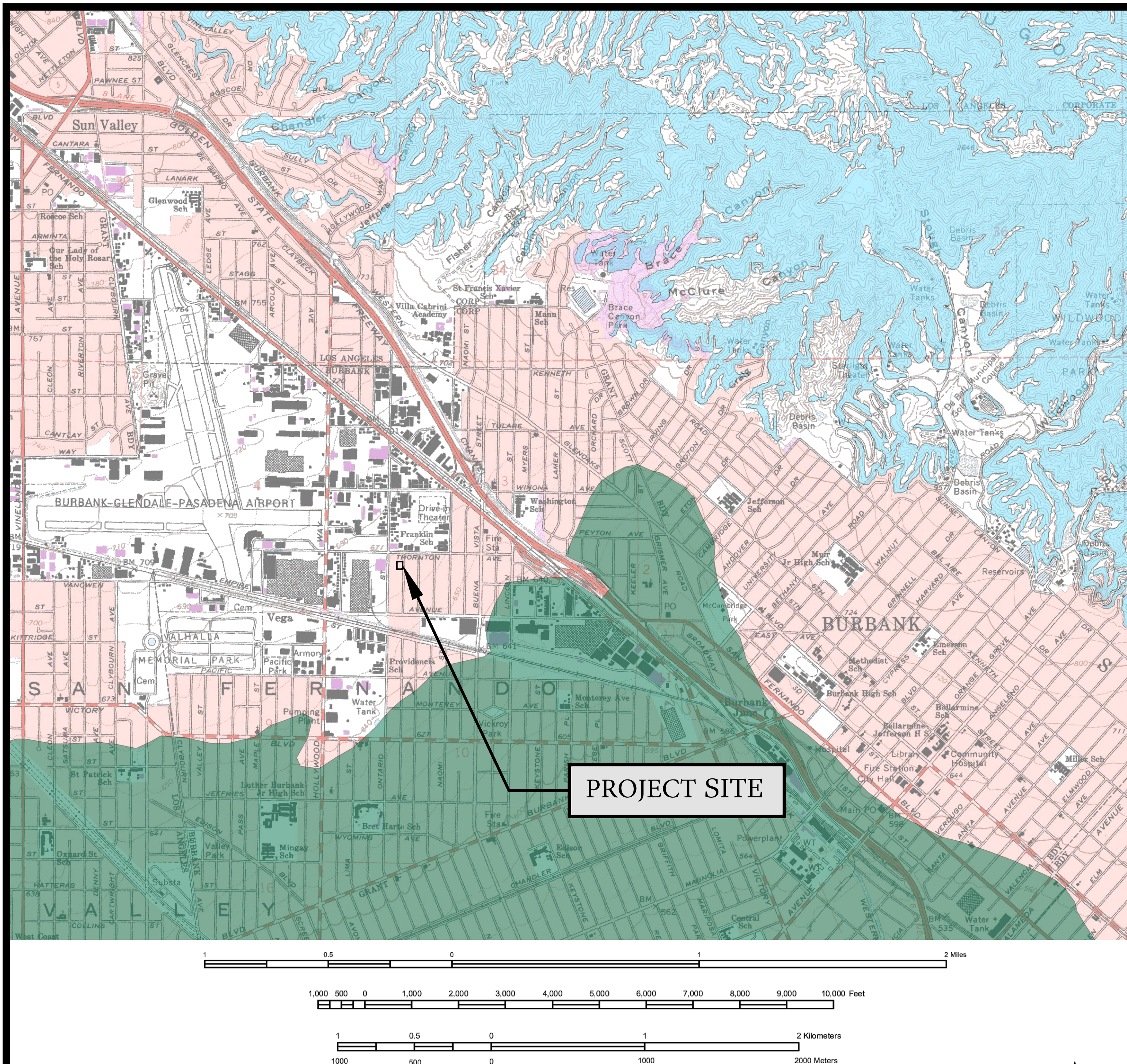
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 016 BURBANK, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998, REV. 2006

HISTORICALLY HIGHEST GROUNDWATER LEVELS MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION

FILE NO: 22517



LEGEND

LIQUEFACTION ZONES

EARTHQUAKE-INDUCED LANDSLIDE ZONES

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, BURBANK QUADRANGLE (CGS, 2016)

N

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION

FILE NO: 22517

BORING LOG NUMBER 1

Burbank Housing Corporation | **Date: 03/22/24** | **Elevation: 673.0' ***

File No. 22517 | **Method: 8-inch diameter Hollow Stem Auger**

kk | *Based on Survey Plan by KPFF dated March 29, 2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions:
				0 --		5.5" Concrete, 5" Base
				-		
				1 --		
				-		FILL: Sand, yellowish brown, moist, medium dense, fine grained
				2 --		
2.5	30	4.2	114.2	-		
				3 --		
				-	SP	NATIVE: Sand, yellow and grayish brown, moist, medium dense, fine grained
				4 --		
				-		
5	14	3.7	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	45	4.4	114.5	-		
				8 --		Sand, grayish brown, moist, medium dense, fine to medium grained, minor cobbles
				-		
				9 --		
				-		
10	24	5.7	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	38	5.4	116.2	-		
				13 --		
				-		
				14 --		
				-		
15	27	4.7	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	73	3.0	121.2	-		
				18 --		Sand, grayish brown, moist, dense, fine to coarse grained
				-		
				19 --		
				-		
20	27	6.2	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	45	4.7	122.0	-		
	50/5"			23 --		Sand, grayish brown, moist, very dense, fine to medium grained
				-		
				24 --		
				-		
25	44	3.4	SPT	25 --		
				-		dense

BORING LOG NUMBER 1

Burbank Housing Corporation

File No. 22517

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	50	4.3	118.6			- - - - - - - - - - - - -
				28		medium dense
				29		
30	28	4.1	SPT	30		
				31		
				32		
32.5	56	4.4	119.6			
				33		
				34		
35	36	3.6	SPT	35		
				36		
				37		
37.5	72	3.8	119.6			- - - - - - - - - - - - -
				38		Sand, grayish brown, moist, dense, fine to medium grained
				39		
40	49	5.0	SPT	40		
				41		
				42		
42.5	65	4.0	120.2			
				43	SM/SP	Silty Sand to Sand, dark and grayish brown, moist, dense, fine to medium grained
				44		
45	45	5.5	SPT	45		
					SP	Sand, yellowish brown, moist, dense, fine grained
				46		
				47		
47.5	73	3.1	115.8			
				48		
				49		
50	51	4.3	SPT	50		

BORING LOG NUMBER 1

Burbank Housing Corporation

File No. 22517

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				51 --		
				-		
				52 --		
52.5	67	4.3	114.1	-		
				53 --		
				-		
				54 --		
				-		
55	56	5.1	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57.5	79	6.0	117.5	-		
				58 --		
				-		
				59 --		
				-		
60	48	3.2	SPT	60 --		
				-		Total Depth 60 feet
				61 --		No Water
				-		Fill to 3 feet
				62 --		
				-		
				63 --		
				-		
				64 --		
				-		
				65 --		
				-		
				66 --		
				-		NOTE: The stratification lines represent the approximate
				67 --		boundary between earth types; the transition may be gradual.
				-		
				68 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				69 --		Modified California Sampler used unless otherwise noted
				-		
				70 --		SPT=Standard Penetration Test
				-		
				71 --		
				-		
				72 --		
				-		
				73 --		
				-		
				74 --		
				-		
				75 --		
				-		

BORING LOG NUMBER 2

Burbank Housing Corporation | **Date: 03/22/24** | **Elevation: 672.0' ***

File No. 22517 | **Method: 8-inch diameter Hollow Stem Auger**

kk | *Based on Survey Plan by KPFF dated March 29, 2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete Slab for Drive Way
				0 --		4" Concrete, No Base
				-		
				1 --		FILL: Sand, dark and grayish brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	26	7.1	109.2	-		
				3 --		
				-	SP	NATIVE: Sand, yellow and grayish brown, moist, medium dense, fine to medium grained
				4 --		
				-		
5	26	3.2	112.4	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	41	3.7	116.0	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	58	4.2	117.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	42 50/5"	3.9	127.5	20 --		
				-		Sand, yellow and grayish brown, moist, very dense, fine to medium grained, minor cobbles
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	25 50/4"	2.5	Disturbed	25 --		
				-		

BORING LOG NUMBER 2

Burbank Housing Corporation

File No. 22517

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		Sand, yellow and grayish brown, moist, medium dense, fine to medium grained
30	37	4.9	118.0	30		
				31		Total Depth 30 feet No Water Fill to 3 feet
				32		
				33		
				34		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				35		
				36		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 3

Burbank Housing Corporation | **Date: 03/22/24** | **Elevation: 672.6' ***

File No. 22517 | **Method: 8-inch Hollow Stem Auger**

kk | *Based on Survey Plan by KPFF dated March 29, 2023

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete for Parking
				0 --		5" Concrete, No Base
				-		
				1 --		FILL: Sand, yellow and grayish brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	37	2.7	117.5	-		
				3 --		
				-	SP	NATIVE: Sand, yellowish brown, moist, medium dense, fine to medium grained
				4 --		
				-		
5	34	2.5	112.7	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	55	3.1	115.2	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	38	2.7	116.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	63	2.6	116.1	20 --		
				-		Sand, yellow and grayish brown, moist, dense, fine to coarse grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	78	3.1	112.4	25 --		
				-		

BORING LOG NUMBER 3

Burbank Housing Corporation

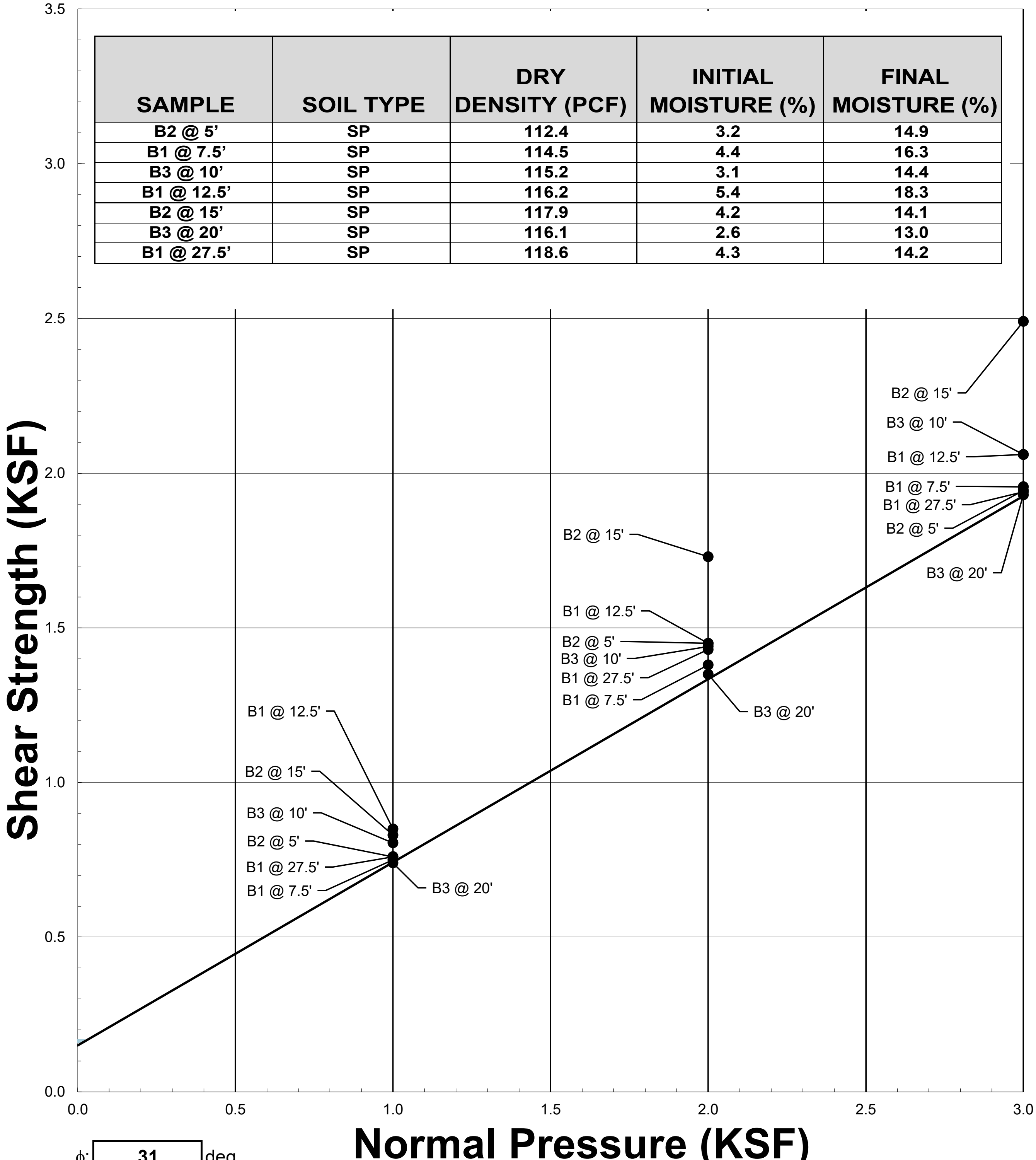
File No. 22517

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	38	4.1	113.1	30 --		
				-		Total Depth 30 feet
				31 --		No Water
				-		Fill to 3 feet
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
				35 --		
				-		NOTE: The stratification lines represent the approximate
				36 --		boundary between earth types; the transition may be gradual.
				-		
				37 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				38 --		Modified California Sampler used unless otherwise noted
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

NATIVE SOIL

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 5'	SP	112.4	3.2	14.9
B1 @ 7.5'	SP	114.5	4.4	16.3
B3 @ 10'	SP	115.2	3.1	14.4
B1 @ 12.5'	SP	116.2	5.4	18.3
B2 @ 15'	SP	117.9	4.2	14.1
B3 @ 20'	SP	116.1	2.6	13.0
B1 @ 27.5'	SP	118.6	4.3	14.2



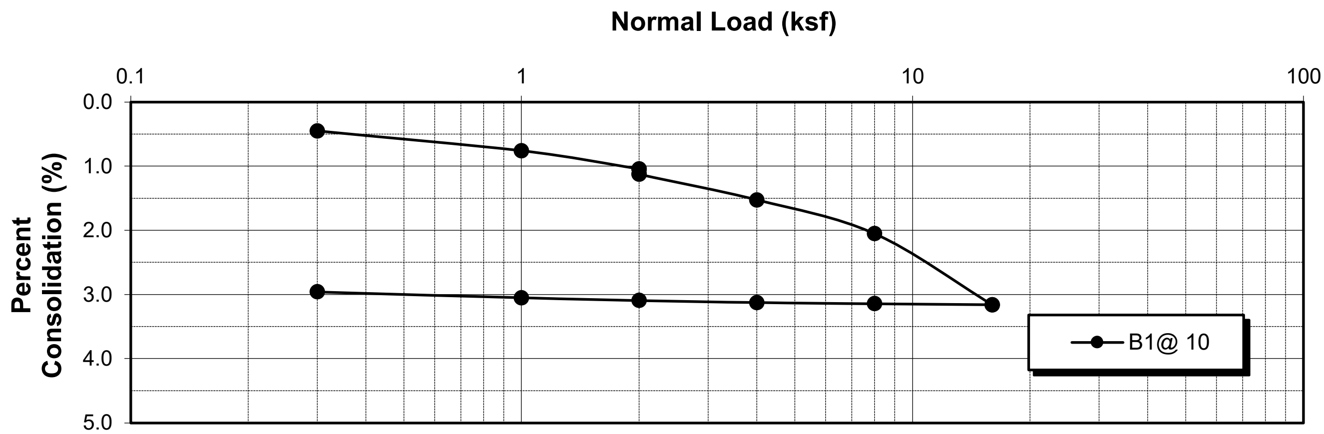
ϕ:	31	deg
c:	150	psf

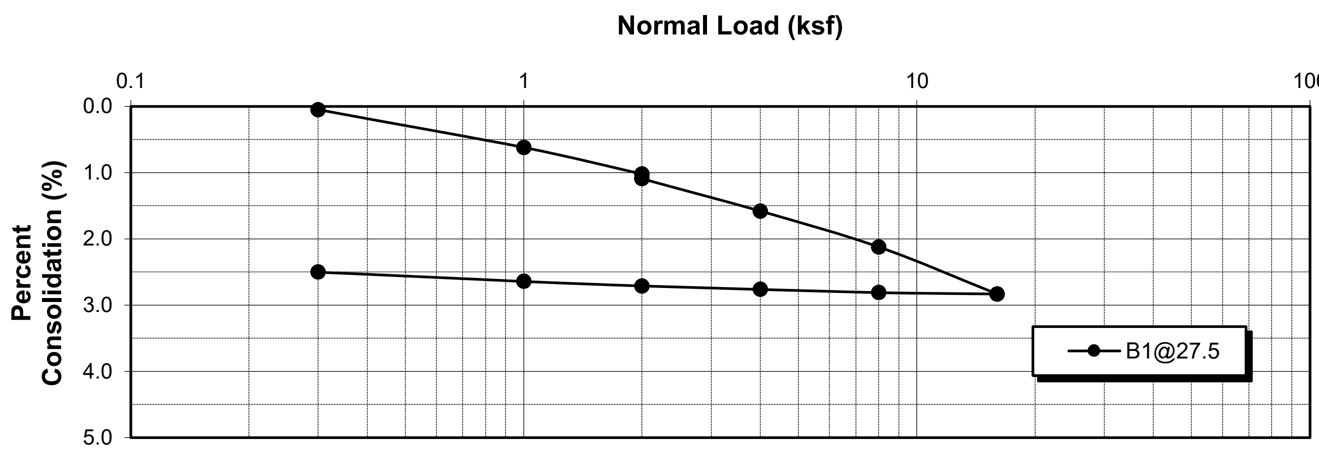
SHEAR TEST DIAGRAM

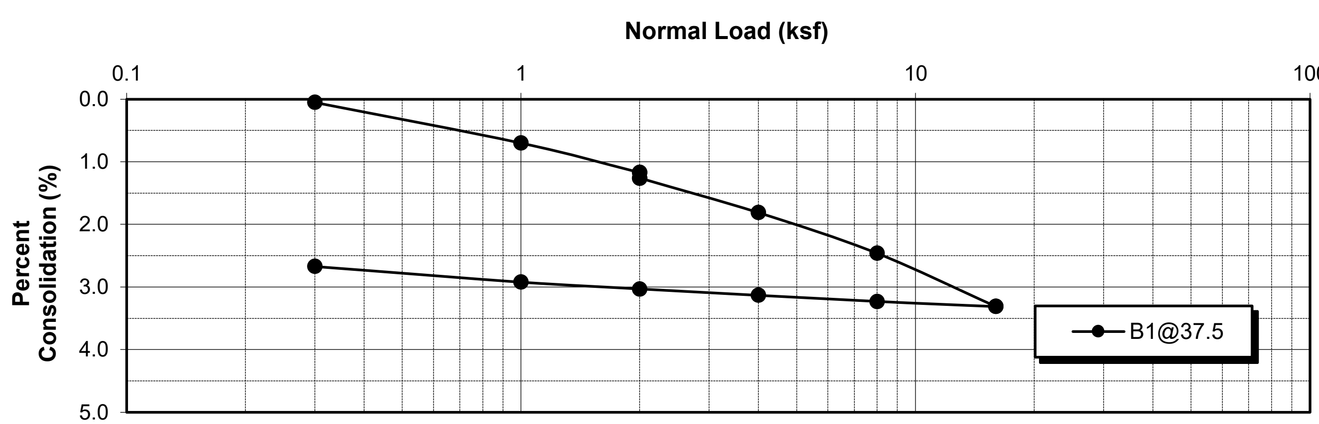
Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION
2321-2335 N. Fairview Street, Burbank

FILE NO.: 22517 | **PLATE: B**





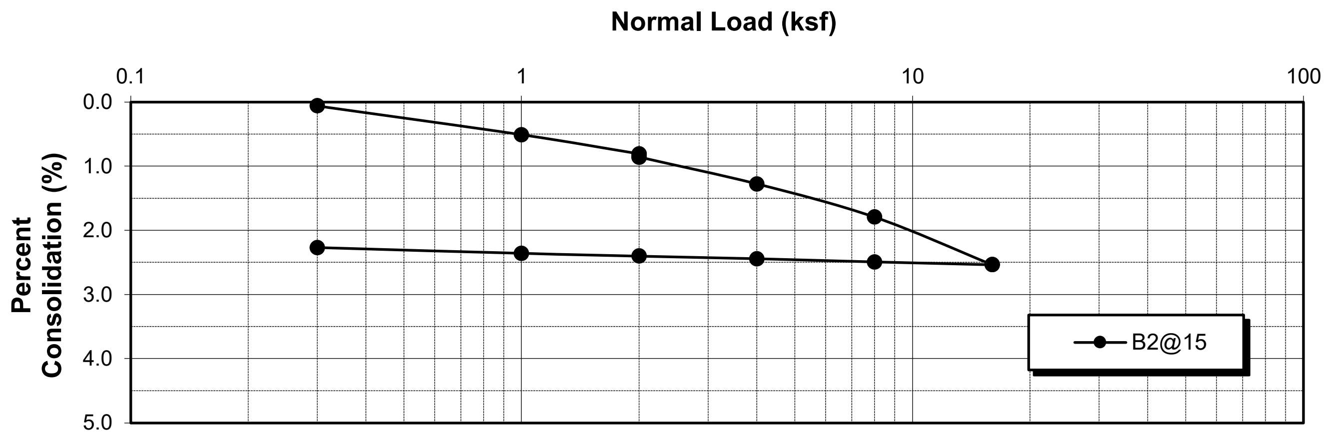


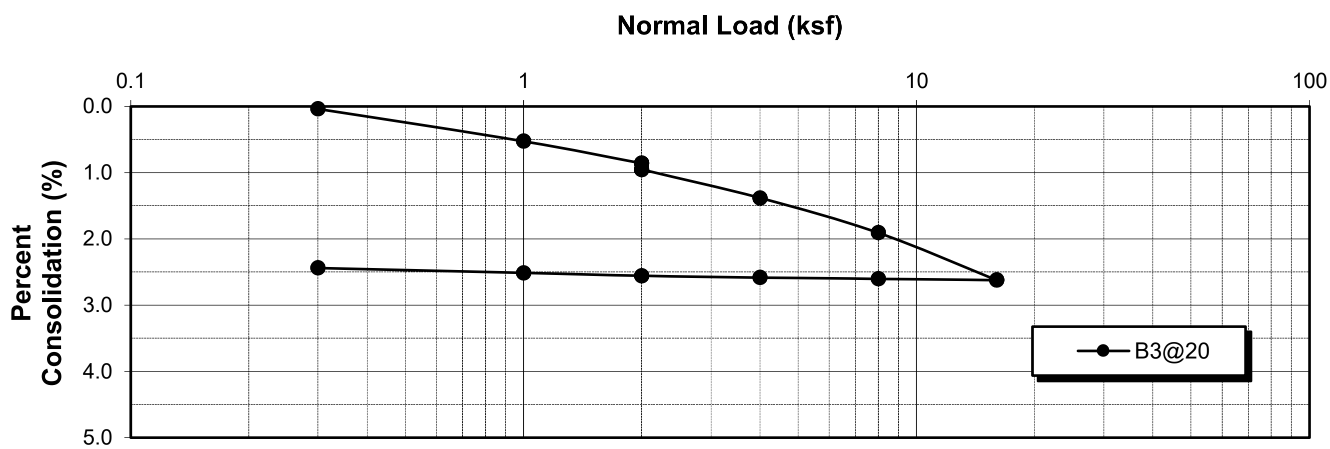
Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: Burbank Housing Corporation	
FILE NO.: 22517	PLATE: C-1





Water added at 2 KSF

CONSOLIDATION

Geotechnologies, Inc.

Consulting Geotechnical Engineers

PROJECT: Burbank Housing Corporation

FILE NO.: 22517

PLATE: C-2

LABORATORY COMPACTION CHARACTERISTICS
(ASTM D1557)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	SP	SP
MAXIMUM DRY DENSITY (PCF)	126.5	124.1
OPTIMUM MOISTURE %	9.8	11.7

EXPANSION INDEX
(ASTM D4829)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SOIL TYPE	SP	SP
EXPANSION INDEX UBC STANDARD 18-2	1	2
EXPANSION CHARACTER	VERY LOW	VERY LOW

SULFATE CONTENT
(CALIFORNIA TEST 417)

SAMPLE	B1 @ 1'-5'	B2 @ 1'-5'
SULFATE CONTENT: (Percentage by Weight)	<0.1%	<0.1%

SULFATE CONTENT
(CALIFORNIA TEST 417)

SAMPLE	B1 @ 12.5'	B2 @ 10'
SULFATE CONTENT: (Percentage by Weight)	<0.1%	<0.1%

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

BURBANK HOUSING CORPORATION
2321-2335 N. FAIRVIEW ST., BURBANK

FILE NO: 22517 PLATE: D

Date: 22-Mar-24
File No. 22517
File Name : Burbank Housing Corporation

Percolation Rate Calculation for Small Diameter Boring

Testing Well Number	1
Boring Diameter (DIA)	8 inches
Depth of Boring	60 feet
Pre-soak Time	2 hours
Measured By	H.C.

Reading Number	Clock Time	Elapsed Time	Water Measurement (d_i) and (d_f)	Water Level Drop	Rate of Drop Variation		Flow Rate	Wet Surface Area	Pre-Adjusted Infiltration Rate
		Min	feet	in	%		in^3/hr	in^2	in/hr
1	1:03		30.00						
	1:18	15	59.30	351.60			70693.5	261.4	270.5
2	1:25		30.00						
	1:40	15	59.20	350.40	-0.34		70452.3	291.5	241.7
3	1:49		30.00						
	2:04	15	59.10	349.20	-0.34		70211.0	321.7	218.3
4	2:08		30.00						
	2:23	15	59.00	348.00	-0.34		69969.7	351.9	198.9
5	2:32		30.00						
	2:47	15	59.00	348.00	0.00		69969.7	351.9	198.9

Note: Calculation based on County of Los Angeles, Administrative Manual, Low Impact Development Best Management PracticeGuideline for Design, Investigation, and Reporting, dated 6/30/21.
LA County Minimum 0.3 Inches per hour

Raw Percolation Rate= 198.9 in/hr
RF_t= 2
RF_v= 2
RF_s= 2

Design Infiltration Rate = 33.1 in/hr