Appendix F3

Geotechnical Investigation

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.

Review of the Earthquake Zones of Required Investigation Map of the Hollywood Quadrangle (CGS, 2014) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The closest zone is the Hollywood Fault Zone, which is located approximately one mile to the north of the subject site. A copy of this map is enclosed herein.

The City of West Hollywood General Plan established "Fault Precaution Zones" for the Hollywood Fault. These zones are illustrated in the City Fault Location and Precaution Zone Map, contained in the Geologic and Seismic Technical Background Report for the City of West Hollywood General Update (KFM Geoscience, 2011). A copy of the City of West Hollywood Fault Location and Precaution Map has been enclosed. The Fault Precaution Zone, FP-1, includes all areas immediately adjacent to the approximate location of surface trace of the Hollywood Fault. Developments within Fault Precaution Zone, FP-1, requires site specific fault rupture evaluation by a California Certified Engineering Geologist. The Fault Precaution Zone, FP-2, includes an area between 100 and 500 feet wide south of the Fault Precaution Zone, FP-1. Developments within the Fault Precaution Zone, FP-2, requires a fault rupture evaluation by a California Certified Engineering Geologist and/or strengthening of foundations to provide for estimated ground displacement of 1 to 2 inches. The subject site is not located within the Fault Precaution Zones, FP-1 or FP-2.

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

Liquefaction

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

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

Two site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration, at depths ranging between 18½ and 19 feet below the existing site grade. According to the Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle (CDMG, 2006), the historically highest groundwater level for the site was approximately 10 feet below the existing ground surface. The enclosed liquefaction analysis takes into consideration the historically highest and current groundwater levels.

Section 11.8.3 of ASCE 7-10 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the USGS U.S. Seismic Design Maps tool, this corresponds to a PGA_M of 0.99g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.90g (2 percent in 50 years ground motion) and a modal magnitude of 6.9 for the site. The liquefaction potential evaluation was performed by utilizing a magnitude 6.9 earthquake, and a peak horizontal acceleration of 0.99g.

The enclosed "Empirical Estimations of Liquefaction Potential" are based on the results obtained from Borings B1 and B2, which were prosecuted to depths of 130 and 180 feet below grade, respectively. Standard Penetration Test (SPT) data were collected at 5 and 10-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E-Plates and F-Plates.

Based on CGS Special Publication 117A (CDMG, 2008) and (Bray and Sancio, 2006), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, soils having a PI greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. The results of Atterberg Limits testing (shown on Plate F) indicate that some of soil layers below the subject site have PI greater than 18. Therefore, these soils are not considered prone to liquefaction, and the analysis of these soil layers was turned off in the liquefaction susceptibility columns.

The site-specific liquefaction analyses included in the Appendix indicates that the site soils would not be prone to liquefaction during the ground motion expected during the design-based seismic event.

Dynamic Settlement

As explained in the previous section, the site soils are not considered prone to liquefaction. Therefore, it is the opinion of this firm that the anticipated liquefaction settlement at the site may be considered to be negligible.

The proposed structure will extend below the current and historically highest groundwater levels. Therefore, dynamic dry-sand settlement is not expected below the proposed structure.

Tsunamis, Seiches and Flooding

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

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site is located within mapped inundation boundaries if the Mulholland Reservoir should breach. However, review of the applicable Flood Insurance Rate Map (06037C1605F) indicates the site lies within an area of minimal flood hazard.

A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

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

CONCLUSIONS AND RECOMMENDATIONS

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

This report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should re-evaluate the recommendations presented herein, to ensure they are suitable for the proposed development. A final geotechnical engineering investigation, suitable for submission to the building official for building permit purposes, will be prepared at that time.

Due to the preliminary nature of the project, the structural loads are currently not available. Detailed structural loads shall be provided to this firm for analyses when the project achieves more definition. Foundation recommendations presented herein shall be considered preliminary and are subject to be confirmed or modified subsequent to review of foundation loads.

Between 5 and 8 feet of existing fill materials was encountered during exploration at the site. The existing fill materials are considered to be unsuitable for support of new foundations, floor slabs, or additional fill. It is however anticipated that the existing fill materials will be removed during excavation of the proposed subterranean levels.

The lowest subterranean level of the proposed structure is expected to extend to an approximate depth of 33 feet below the existing ground surface, with foundations expected to extend between 5 and 10 feet below this depth. Preliminarily, it is anticipated that the proposed structure may be supported on a mat foundation bearing in the older alluvial soils present near the subterranean subgrade. Detailed structural loads, size and dimensions of the mat footing shall be provided to this firm for analyses when the project achieves more definition. The design of the foundation system for support of the tower will be an iterative process between the structural engineer and the geotechnical engineer.

Groundwater was encountered at depths ranging between 18½ and 19 feet below the existing site grade during exploration. Therefore, excavation of the proposed subterranean levels will require dewatering measures to provide a dry excavation. It is expected that a formal pre-construction temporary dewatering program consisting of wells or well-points will be required to lower the groundwater table prior to excavation of the subterranean levels. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

Once the temporary construction dewatering is discontinued, the water table will likely return to its current elevation. Since the elevation of the water table is higher than the proposed bottom of structure, hydrostatic forces on the walls and floor will result. It is recommended the proposed development be designed to resist hydrostatic forces in lieu of installation of a permanent dewatering system. This will eliminate the need for maintenance of a permanent dewatering system and continuous handling of waters pumped from the system. Hydrostatic forces are addressed in the “Retaining Wall Design” and “Foundation Design” sections of this report.

It is recommended that the mat foundation system and retaining walls be completely watertight in order to prevent water seepage through normal shrinkage cracks or construction joints. It is recommended care be taken in the design and installation of waterproofing to avoid moisture problems, and to prevent water seepage into the structure. The design and inspection of waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floors, and foundations.

Although temporary dewatering will lower the groundwater elevation prior to construction, the soils at the proposed subgrade level should be expected to be well above their optimum moisture level. These soils could be wet and soft. The placement of a mat of gravel over the bottom excavation will most likely be necessary to protect the subgrade soils from disturbance, create a firm working surface, and provide a firm bottom that is suitable for support of the proposed mat foundation. Placement of gravel and wet subgrade soils are discussed in a following section.

Due to the depth of the proposed subterranean levels, and the proximity of the property lines, excavations around the perimeter of the proposed structure will require shoring in order to provide a stable excavation. Shoring recommendations are provided in the "Excavations" section of this report.

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

SEISMIC DESIGN CONSIDERATIONS

Based on the height of the proposed structure, it is anticipated that the structure will be designed following a performance-based design. Due to the preliminary stage of the project, a site-specific response spectrum and time histories analyses have not been developed for the preparation of this report. It is anticipated that these analyses will be developed once a structural engineer has been selected for the project.

Seismic Shearwave Velocity Measurements

Downhole seismic velocity measurements were performed by GeoPentech within Boring Number 2, which was excavated to a depth of 180 feet below the existing site grade. However, the survey was conducted to a maximum depth of 149 feet. Results of the seismic velocity measurements are presented in the enclosed Downhole Seismic Tests Results report by GeoPentech, dated October 22, 2019. The following table presents the average shear wave velocities of the underlying earth materials measured within Boring Number 2. A copy of the GeoPentech's report is enclosed at the end of the Appendix.

Depth Range (feet)	Average Shear Wave Velocity (feet/second)
0 to 5	863
5 to 35	1,166
35 to 50	934
50 to 75	1,256
75 to 95	1,010
95 to 149	1,459

2019 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 20.3-1 of ASCE 7-10, and ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program at https://seismicmaps.org in order to calculate ground motion parameters for the site.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.092g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.092g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.395g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.750g
Site Coefficient (F_v)	1.7
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.275g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.850g

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

EXPANSIVE SOILS

The onsite geologic materials are in the Moderate to High expansion range. The Expansion Index was found to be 82 and 94 for representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slabs on Grade" sections of this report.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on four samples representative of the onsite soils by Project X Corrosion Engineering indicate that the electrical resistivities of the soils are severely corrosive to general metals when saturated. The soil pH value of the samples was between 7.8 and 8.2. The pH was determined to be at levels not detrimental to copper or aluminum alloys, but can allow corrosion of steel and iron in moist environments. Chloride levels in the samples are low and may cause insignificant corrosion of metals. Ammonia and Nitrates concentrations are not high enough to cause accelerated corrosion of copper and copper alloys.

Sulfate content in the samples are considered negligible for corrosion of metals and cement. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible.

Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Corrosion Evaluation Report prepared by Project X Corrosion Engineering, dated October 9, 2019.

TEMPORARY DEWATERING

Groundwater was encountered during exploration, to depths ranging between 18½ and 19 feet below grade. It is anticipated that the lowest subterranean level will extend to a depth of 33 feet below grade, and the mat foundation system may extend 5 to 10 feet below that depth.

Since the proposed subterranean level will extend well below the current groundwater level, it is recommended that a qualified dewatering consultant should be retained during the design phase of the project. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water, however, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected within the excavation through the use of French drains and sump pumps.

Wet Subgrade Soils

Soils at the proposed subgrade level should be expected to be well above their optimum moisture level. A representative of this office should observe the subgrade as it becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary. At this time, pumping, rutting, and disturbance of the high-moisture content soils should be expected to occur during operation of heavy equipment. In order to minimize disturbance of the subgrade bearing soils, provide a firm working surface, and provide a subgrade suitable for support of the proposed mat foundation, it is recommended the subgrade be protected and/or stabilized as it becomes exposed.

Protection or stabilization of the subgrade may be accomplished by placement of a layer of angular ¾-inch gravel. This layer should be a minimum of 1 foot in thickness; however, the exact thickness of the gravel would be a trial and error procedure, and would be determined in the field. The gravel should be placed and vibrated to a dense state as the subgrade becomes exposed. The elevation at the bottom of excavation will require adjustment to provide space for the gravel mat. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on soft subgrade soils will likely result in excessive disturbance to the soils, which in turn could result in a delay to the construction schedule. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane Zone and Methane Buffer Zones map. Based on this review it appears that the subject site is not located within a Methane Zone or a Buffer Zone as designated by the City.

GRADING GUIDELINES

The following recommendations are provided for any miscellaneous grading that may be required, such as 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 should be mechanically compacted in layers not more than 8 inches thick. Based on the moderate to high expansion index of the site soils, it is recommended that fill materials are moisture conditioned to approximately 3 to 5 percent over optimum moisture content before recompaction. Any fill to be placed below the proposed mat foundation shall be compacted to at least 95 percent of the maximum laboratory density for the materials used. All other fill 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 and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

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

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Shrinkage

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

Weather Related Grading Considerations

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

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

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

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

Geotechnical Observations and Testing During Grading

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

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

FOUNDATION DESIGN

MAT FOUNDATION

Due to the preliminary nature of the project, the structural loads are currently not available. Foundation recommendations presented herein shall be considered preliminary and are subject to be confirmed or modified subsequent to review of foundation loads. Detailed structural loads, size and dimensions of the mat footing shall be provided to this firm for analyses when the project achieves more definition. The design of the foundation system for support of the tower will be an iterative process between the structural engineer and the geotechnical engineer.

Preliminarily, it is anticipated that the proposed structure may be supported on a mat foundation system, bearing in the dense older alluvial soils expected at the subterranean subgrade. It is estimated that the proposed structure will have an average bearing pressure between 7,000 and 10,000 pounds per square foot. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

The proposed mat foundation shall bear in the older alluvial soils expected at the subterranean subgrade. For preliminary design purposes, an allowable bearing pressure of up to 10,000 pounds per square foot, with locally higher pressures up to 12,000 pounds per square foot may be utilized in the mat foundation design.

The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

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

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

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

Hydrostatic Considerations for Mat Foundations

Where constructed below the groundwater level, mat foundations shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the historically highest groundwater level. As discussed in the "Groundwater" Section of this report, the historically highest groundwater level on the site may be considered to correspond to a depth of 10 feet below grade. The uplift pressure to be used in design should be 62.4(H) pounds per square foot, where "H" is the height of the height of the historically highest groundwater level above the bottom of the mat foundation in feet.

If necessary, uplift anchors may be designed to provide resistance against the anticipated hydrostatic uplift pressures acting on the recommended mat foundations. Uplift anchors should be a minimum of 12 inches in diameter, and should be embedded a minimum of 20 feet into the underlying native soils. Preliminarily, it is assumed that pressure grouted anchors will be utilized. Uplift anchors may be designed using a frictional capacity of 3 kips per lineal foot. Based on communication with the structural engineer, uplift anchors are not anticipated. In the event that uplift anchors will be required, please contact this office so installation and testing guidelines are provided.

Lateral Mat Foundation Design

Resistance to lateral loading may be provided by friction acting at the base of the mat and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Mat Foundation Settlement

Settlement of the mat foundation will be analyzed once structural loads are available.

RETAINING WALL DESIGN

Retaining walls on the order of 33 feet in height are anticipated for the proposed subterranean parking levels. As a precautionary measure, recommendations for retaining walls up to a height of 40 feet are provided herein. It is anticipated these walls will be restrained. Foundations for these walls may be designed in accordance with the previous "Foundation Design" section.

As previously discussed, it is recommended that the proposed structure be designed to resist hydrostatic forces in lieu of installing a permanent dewatering system at the base of the structure. Wall pressures are provided below for hydrostatic design.

Additional active pressure should be added to the retaining wall design for any additional surcharge conditions, such as adjacent traffic and structures. Additional surcharge pressures should be considered for all adjacent foundations located within a 1:1 (45 degree) surcharge plane. Based on review of the enclosed Plot Plan and Cross Section A-A', it is anticipated that some of the northern and eastern retaining walls will be surcharged by existing structures. Information regarding the loading of the adjacent foundations will be required to analyze the anticipated surcharge pressure.

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

Restrained Retaining Walls

Restrained subterranean retaining walls 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. The pressures provided in the following table are based on a full hydrostatic design. These pressures shall be applied over the entire length of the wall.

RESTRAINED BASEMENT WALLS ***(HYDROSTATIC DESIGN)***		
	AT-REST EARTH PRESSURE (Pounds per Cubic Foot) Includes Hydrostatic Pressure of 62.4 pcf	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure) Includes Hydrostatic Pressure of 62.4 pcf**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 40 feet	94	93*

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. In accordance with the Geologic and Seismic Technical Background Report for the City of West Hollywood General Plan Update, published by KFM Geoscience (2010), a peak ground acceleration of 0.52g was utilized to determine the seismic wall pressure. The procedure prescribed by Mikola and Sitar (2013), was utilized to determine the mean seismic wall pressure. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 36 pounds per cubic foot. The point of application should be at 1/3(H) from the base of the retaining wall, where H is the height of the retaining wall. The seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Miscellaneous Drained Cantilever Retaining Walls

Miscellaneous cantilever retaining walls to be built above the historically highest groundwater levels may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 45 pounds per cubic foot for walls retaining up to 10 feet of earth. This pressure assumes that the wall will be built above the historically highest groundwater level, and that a subdrain system will be installed behind the wall. In addition to this pressure, cantilever walls greater than 6 feet in height shall be designed to resist seismic earth pressure indicated in the “Dynamic (Seismic) Earth Pressure” section above.

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 active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Waterproofing

Moisture affecting 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.

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

Retaining Wall Drainage

This section is intended for miscellaneous drained retaining walls, to be built above the historically highest groundwater level. A drainage system is not anticipated for the subterranean parking garage retaining walls, because these will be designed to fully resist hydrostatic forces.

Where miscellaneous retaining walls are designed for a drained condition, these walls should be provided with a subdrain consisting of a perforated pipe, placed with perforations facing down, covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. 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 water 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.

It is recommended a qualified dewatering consultant be retained in order to establish design flow rates and ensure adequate sizing of subdrainage pipes and systems. Subdrainage pipes should outlet to an acceptable location.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and 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

It is anticipated that excavations between 38 and 43 feet in vertical height will be required for construction of the proposed subterranean levels and mat foundation. The excavations are expected to expose fill and dense native alluvial soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures.

Due to the presence of groundwater, the depth of the excavation, and the proximity of adjacent structures and public ways, excavation of the proposed subterranean levels will require shoring and dewatering measures to provide a stable and dry excavation. Shoring recommendations are provided in the following section.

All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation nor to flow towards it. No vehicular surcharge should be allowed within 5 feet of the top of an unshored cut.

SHORING DESIGN

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

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. Due to the depth of the soldier piles, it is anticipated they will be laterally braced utilizing drilled tie-back anchors.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2½ diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 5,000 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 earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 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.

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

Groundwater was encountered during exploration at depths between 18½ and 19 feet below the existing site grade. Depending on the draw down level associated with the future dewatering program, it is anticipated that the proposed piles will likely encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

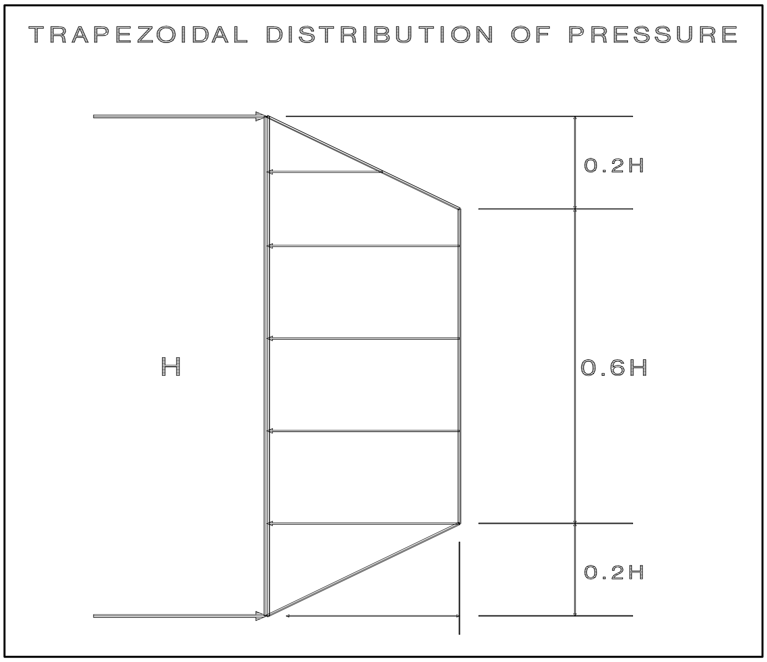
It is recommended that lagging be installed throughout the entire depth of the excavation. Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be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

Lateral Pressures

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The trapezoidal distribution of pressure is shown in the diagram below. The shoring wall pressure for design of restrained shoring is presented in the following table:

Height of Shoring (feet)	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 45 feet	25H psf

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



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

Tieback Anchor Design and Installation

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

Tieback anchors may be installed between 20 and 45 degrees below the horizontal. Caving may occur within granular materials. Where caving occurs 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.

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 500 pounds per square foot. 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 should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for “Quick”, 200 percent tests. 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.

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

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

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

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 24 inches in width and length as well as 24 inches in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

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 the shoring be designed for a maximum deflection of 1-inch at the top of the shored embankment. Embankments which are surcharged by adjacent structures should be designed for a maximum deflection of ½-inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and streets. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

Monitoring

Because of the depth of the excavations, 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.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many local agencies require that shoring installation be performed under the continuous observation of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also, the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

SLABS ON GRADE

Outdoor Concrete Slabs

Outdoor concrete flatwork should be a minimum of 5 inches in thickness. Outdoor concrete flatwork should be cast over 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.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

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

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

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

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

Concrete Crack Control

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

For standard control of concrete cracking, a maximum crack control joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 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 90 percent relative compaction 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	4	6
Moderate Truck	5	8

Concrete paving may also be used on the project. For passenger cars and moderate truck traffic, concrete paving should be 6 inches of concrete over 4 inches of compacted base. For standard crack control maximum expansion joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

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

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

STORMWATER DISPOSAL

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

Groundwater was encountered below the subject site at depths ranging between 18½ and 19 feet below the ground surface during exploration, and the historically highest groundwater level was 10 feet below the ground surface. Based on the anticipated finished floor elevation of the proposed subterranean parking levels, there is no potential for filtration of stormwater prior to its interaction with groundwater. Based on this consideration, stormwater infiltration is not recommended for the subject site.

Where infiltration of stormwater into the subgrade soils is not advisable, most Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

DESIGN REVIEW

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

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

CONSTRUCTION MONITORING

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

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

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

EXCAVATION CHARACTERISTICS

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

CLOSURE AND LIMITATIONS

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

The 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 is 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 an automatic-trip 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.

Expansion Index Testing

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

Laboratory Compaction Characteristics

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

Grain Size Distribution

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

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plates.

REFERENCES

Blake, T.F., 1994-1996, LIQ2_30.WQ1 - A QUATTRO-PRO Spreadsheet Computer Program for the determination of liquefaction potential using the NCEER (1996) semi-empirical method.

California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2006), Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 026, map scale 1:24,000.

California Department of Conservation, Division of Mines and Geology, 1999, Seismic Hazard Zones Map, Hollywood 7½-minute Quadrangle, CDMG Seismic Hazard Zone Mapping Act of 1990.

California Geological Survey, 2008, Guidelines for Evaluation and Mitigation of Seismic Hazards in California, Special Publication 117A.

California Geological Survey, 2014, Earthquake Fault Zones, Hollywood 7½-minute Quadrangle.

Department of Water Resources, 1961, Planned Utilization of the Ground Water Basins of the Coastal Plain of Los Angeles County, Bulletin 104, Appendix A, 181 pages.

Dibblee, T.W. Jr. 1991, Geologic Map of the Hollywood and Burbank (South ½) Quadrangles, DMG Map #DF-30, map scale 1: 24,000.

FEMA, Flood Insurance Rate Map 06037C1605F, dated September 26, 2008.

KFM Geoscience, March 15, 2010, Geologic and Seismic Technical Background Report, City of West Hollywood General Plan Update, West Hollywood, Los Angeles County, California, Project No. CWH 08-14E.

Leighton and Associates, Inc., 1990, Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

Mikola, R.G., and Sitar, N, 2013, Seismic Earth Pressures on Retaining Structures in Cohesionless Soils, Caltrans Contract No. 65A0367 and NSF-NEES-CR Grant No. CMMI-0936376, Report No. UCB GT-13-01.

SEAOC/OSHPD U.S. Seismic Design Maps tool.

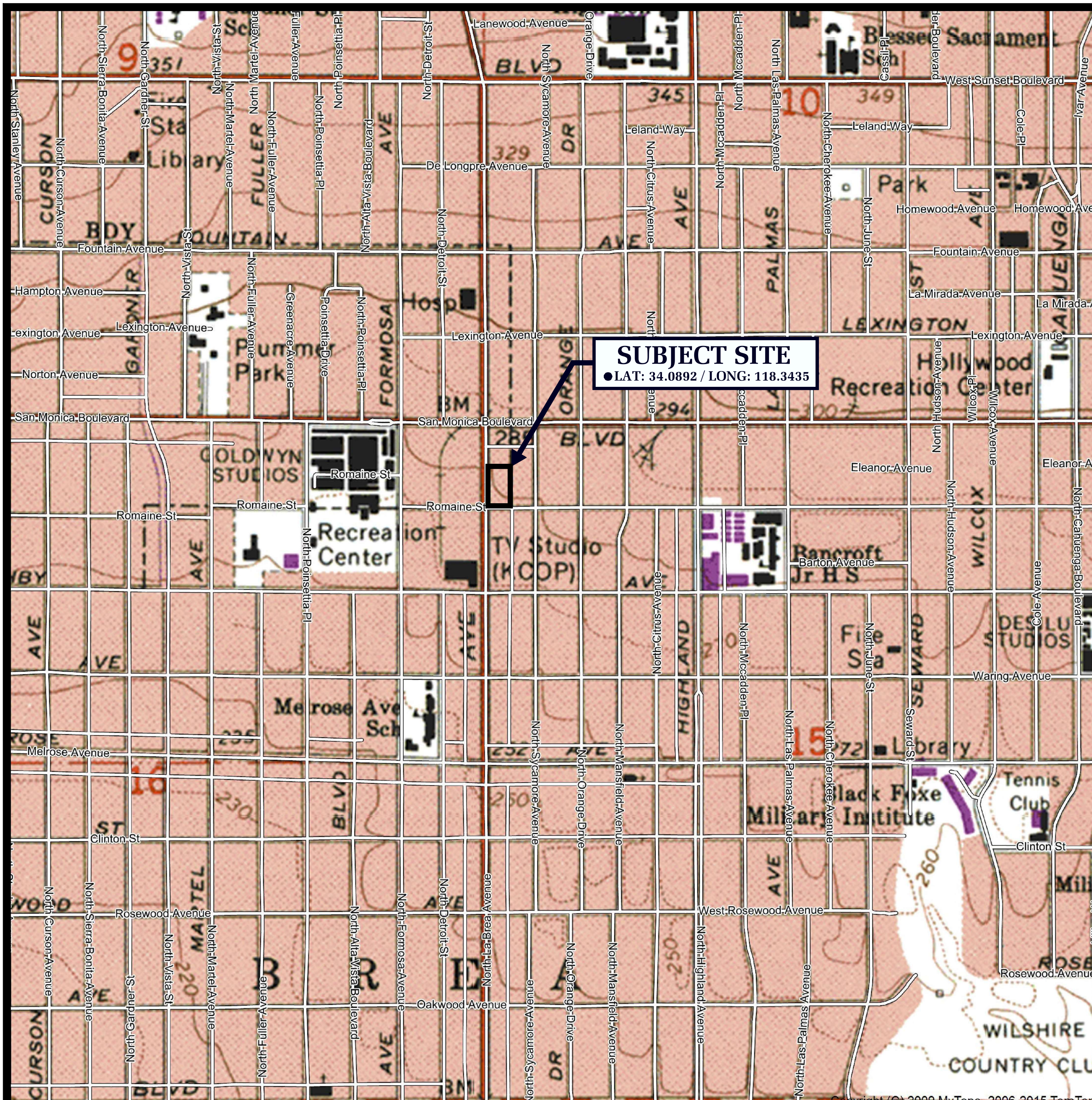
Tinsley, J.C., and Fumal, T.E., 1985, Mapping Quaternary Sedimentary Deposits for Areal Variations in Shaking Response, in Evaluation Earthquake Hazards in the Los Angeles Region-An Earth Science Perspective, U.S. Geological Survey Professional Paper 1360, Ziony, J.I. ed., pp 101-125.

REFERENCES - continued

Tokimatsu, K. and Seed, H. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.

United States Geological Survey, 2008, U.S.G.S. Interactive Deaggregation Program. http://eqint.cr.usgs.gov/deaggint/2008/index.php.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., Vedder, J.G., 1965, Geology of the Los Angeles Basin, Southern California-An Introduction, U.S. Geological Professional Paper 420-A.



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MILES

0 1000 2000 3000
FEET

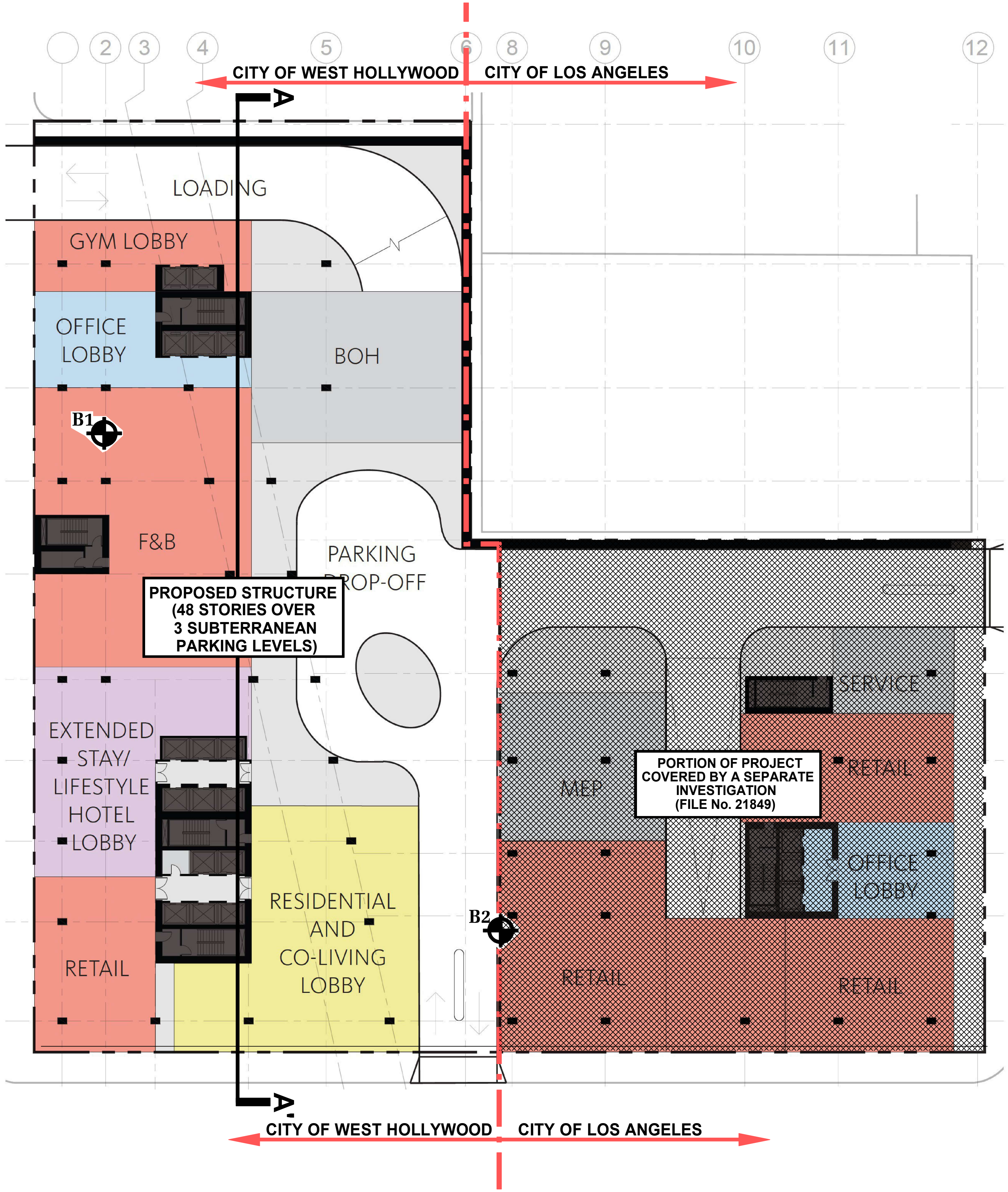
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

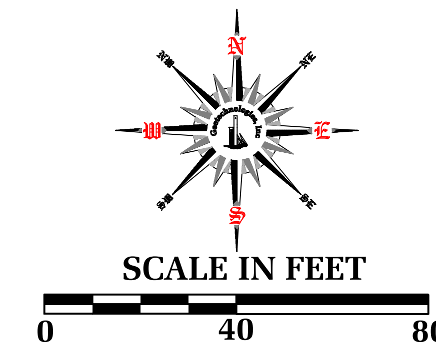
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FARING
1010, 1014 & 1020 N. LA BREA AVE., WEST HOLLYWOOD

FILE NO. 21848





LEGEND

B2 — LOCATION & NUMBER OF PROPOSED BORING

A — A' — CROSS SECTION

REFERENCE: GROUND FLOOR PLANS BY GENSLER DATED 9/19/19

Geotechnologies, Inc.
Consulting Geotechnical Engineers

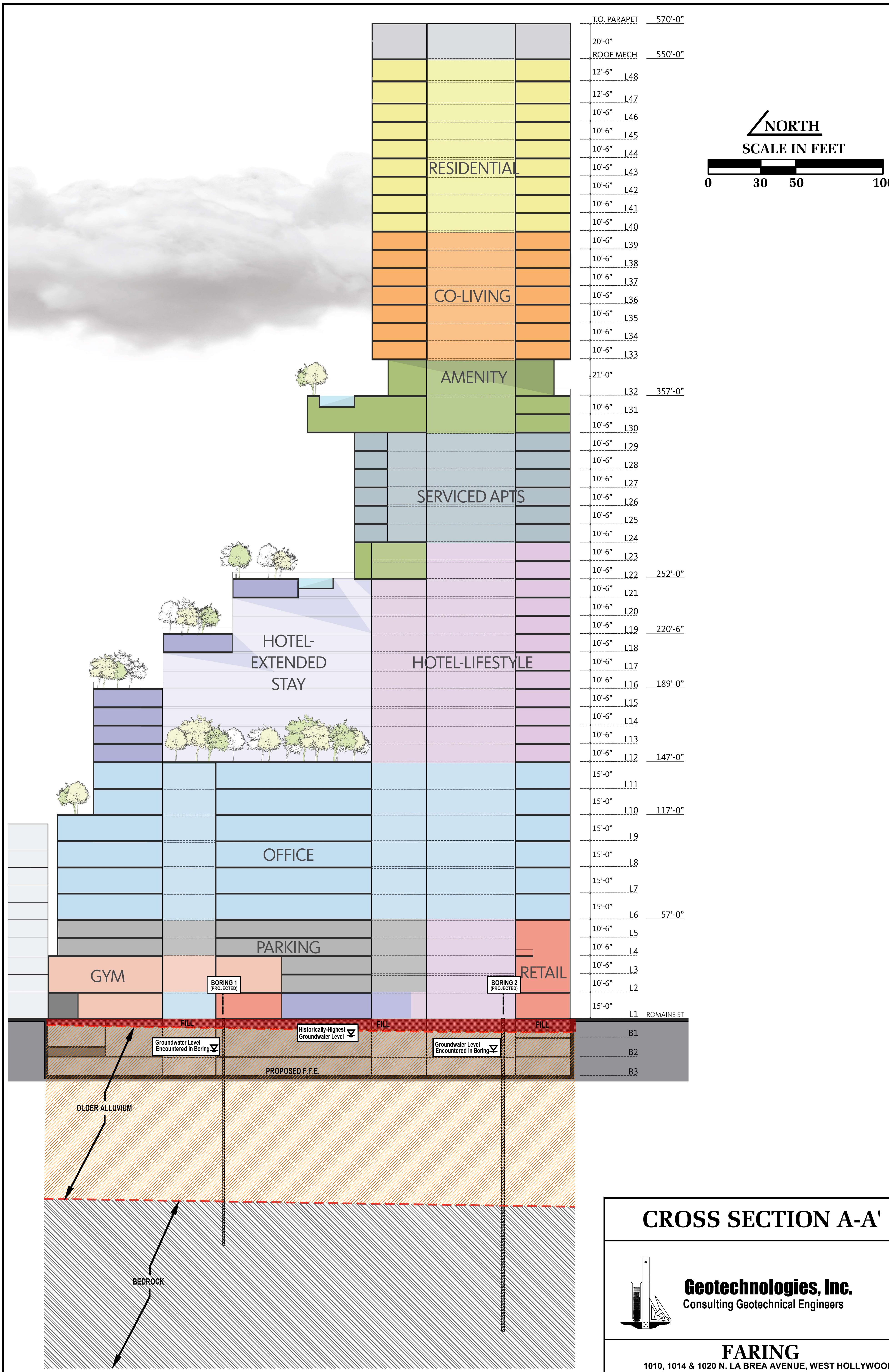
PLOT PLAN

FARING

1010, 1014 & 1020 N. LA BREA AVENUE, WEST HOLLYWOOD

FILE No. 21848

Date: October '19



CROSS SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

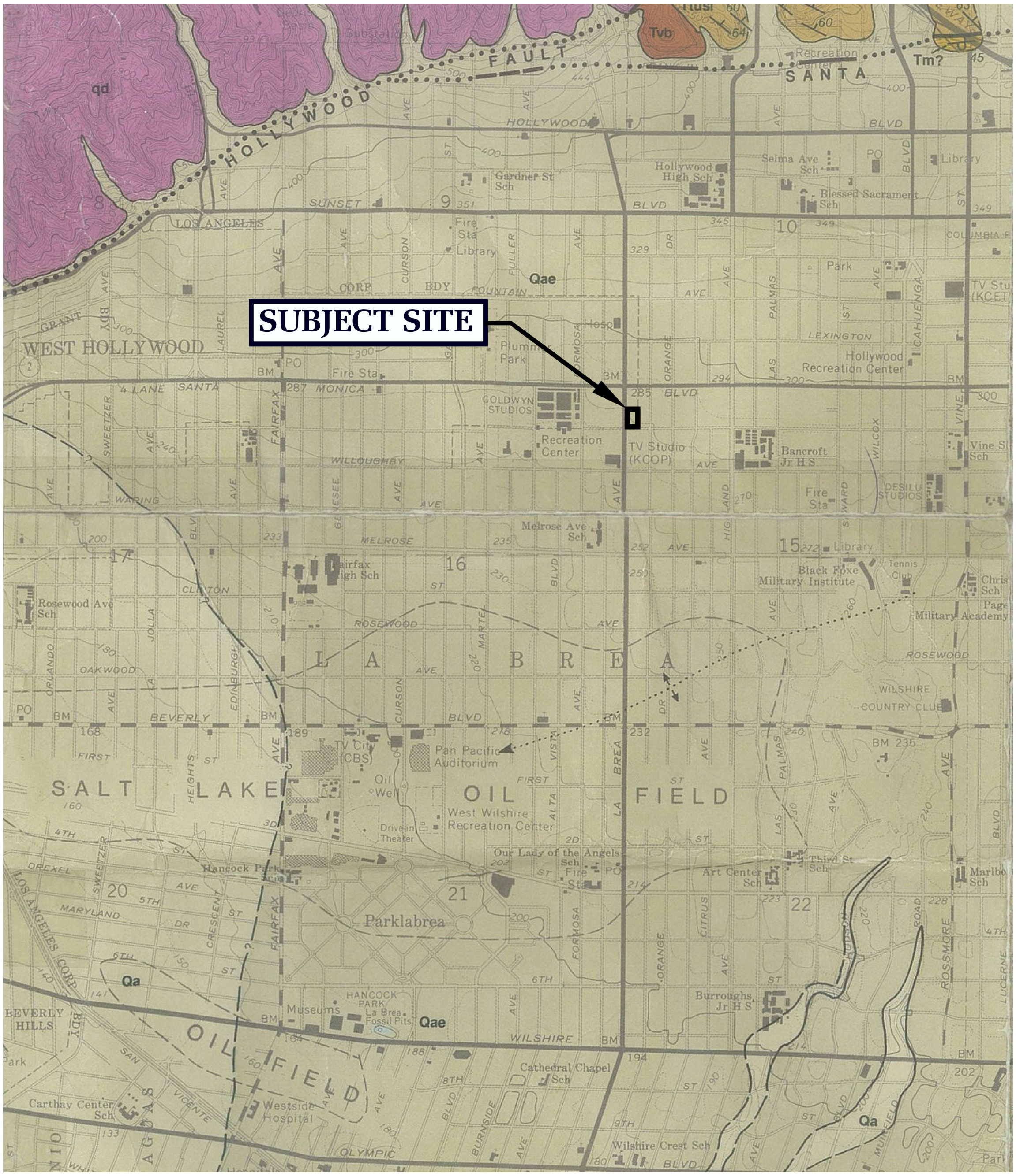
FARING

1010, 1014 & 1020 N. LA BREA AVENUE, WEST HOLLYWOOD

FILE No. 21848

DATE: October '19

REFERENCE: NORTH-SOUTH SECTION BY GENSLER DATED 9/19/19



SCALE IN FEET
0 500 1000 2000

LEGEND

Qa: Surficial Sediments: alluvium - gravel, sand and clay
Qae: Older Surficial Sediments: similar to QA, slightly elevated and dissected
Tm: Monterey Formation: platy siliceous shale
Ttusi: Upper Topanga Formation: micaceous clay shale or claystone
Tvb: Middle Topanga Formation and Volcanic Rocks: basaltic volcanic rocks
Qd: Granitic Rocks: quartz diorite, mostly of plagioclase feldspar
Folds - arrow on axial trace of fold indicates direction of plunge
Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE HOLLYWOOD-BURBANK (SOUTH 1/2) QUADRANGLES (#DF-30)

LOCAL GEOLOGIC MAP - DIBBLEE

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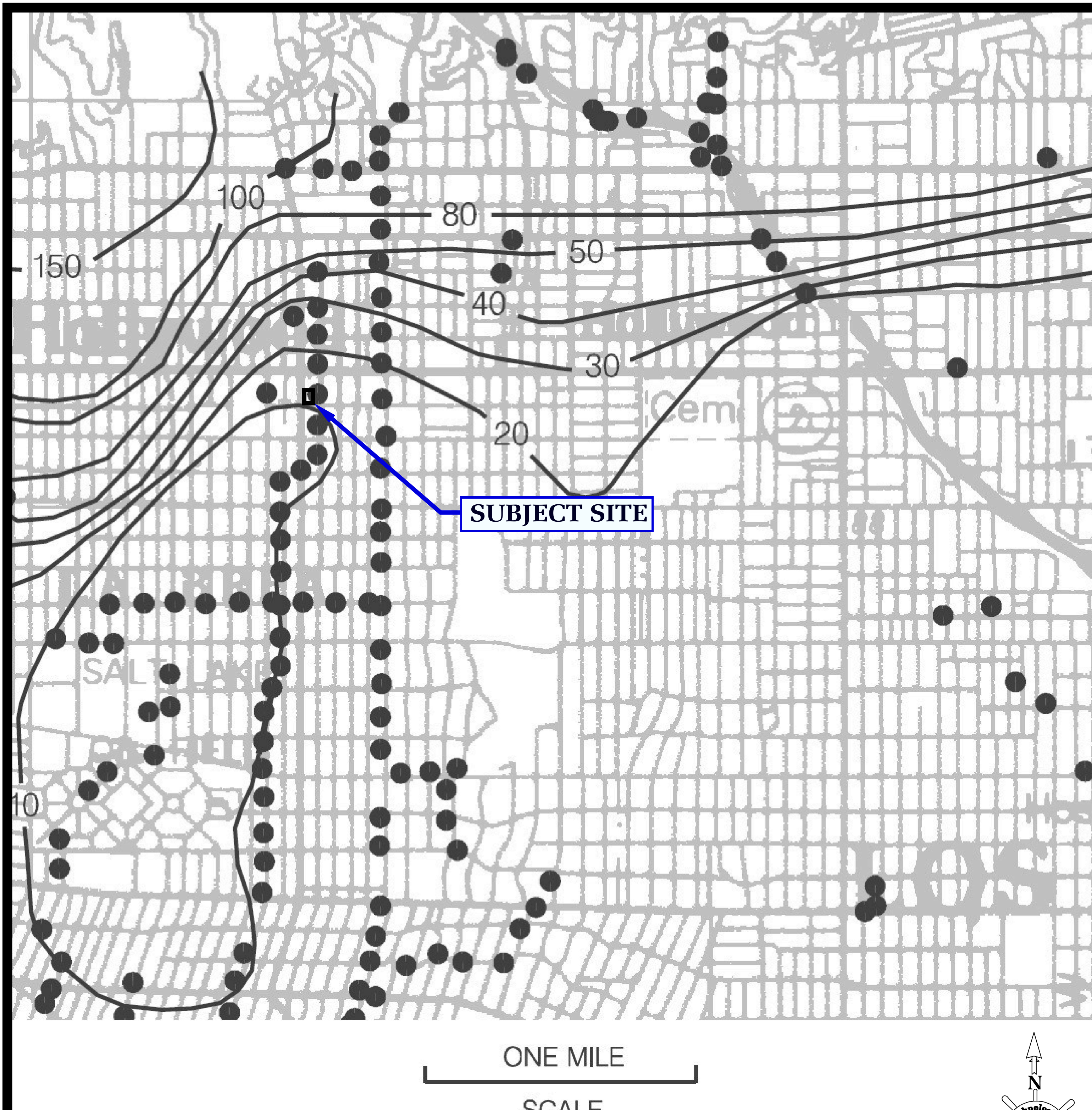
FARING
1010, 1014 & 1020 N. LA BREA AVE., WEST HOLLYWOOD

FILE NO. 21848



20 Depth to groundwater in feet

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

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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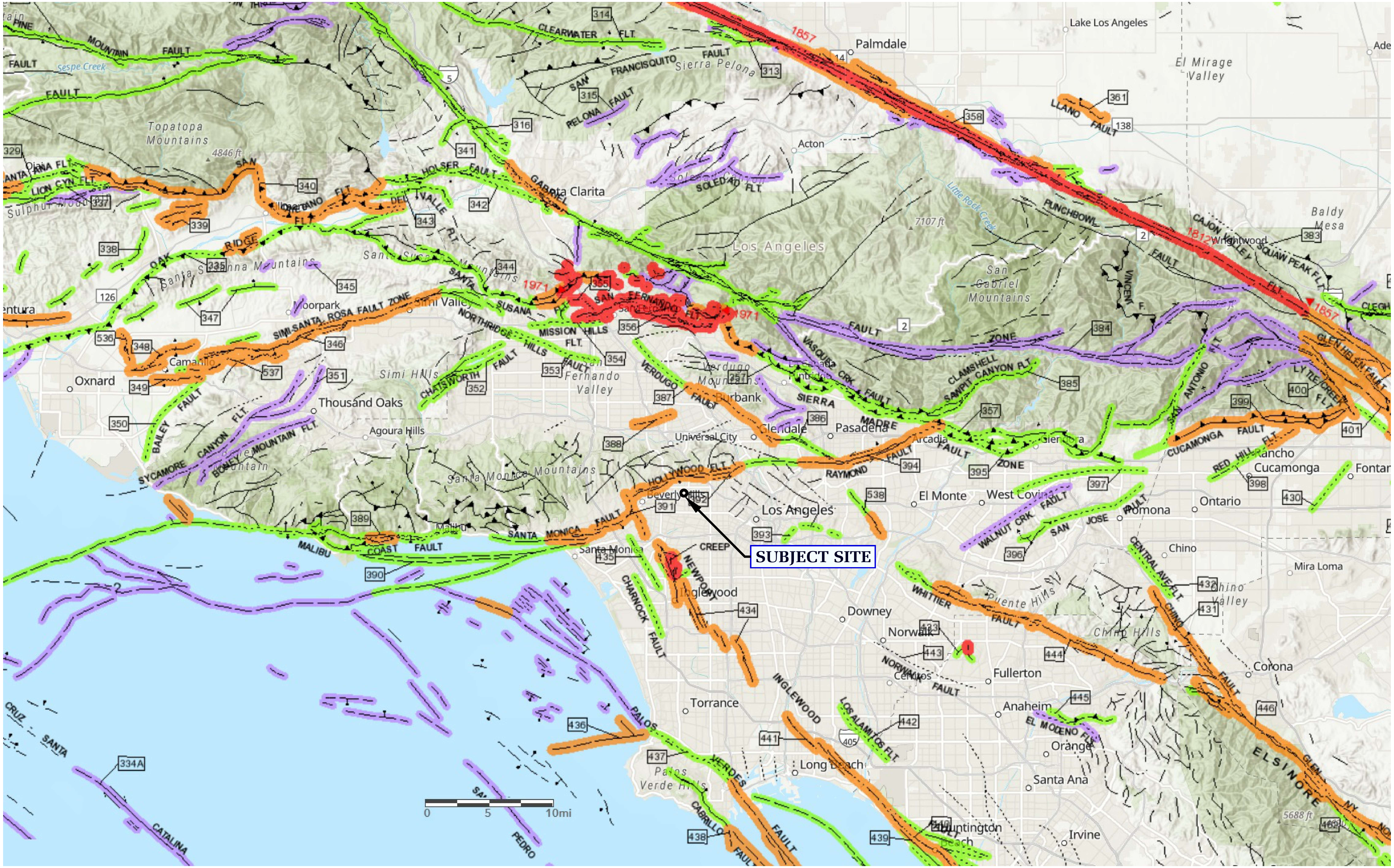

REGIONAL FAULT LOCATION MAP

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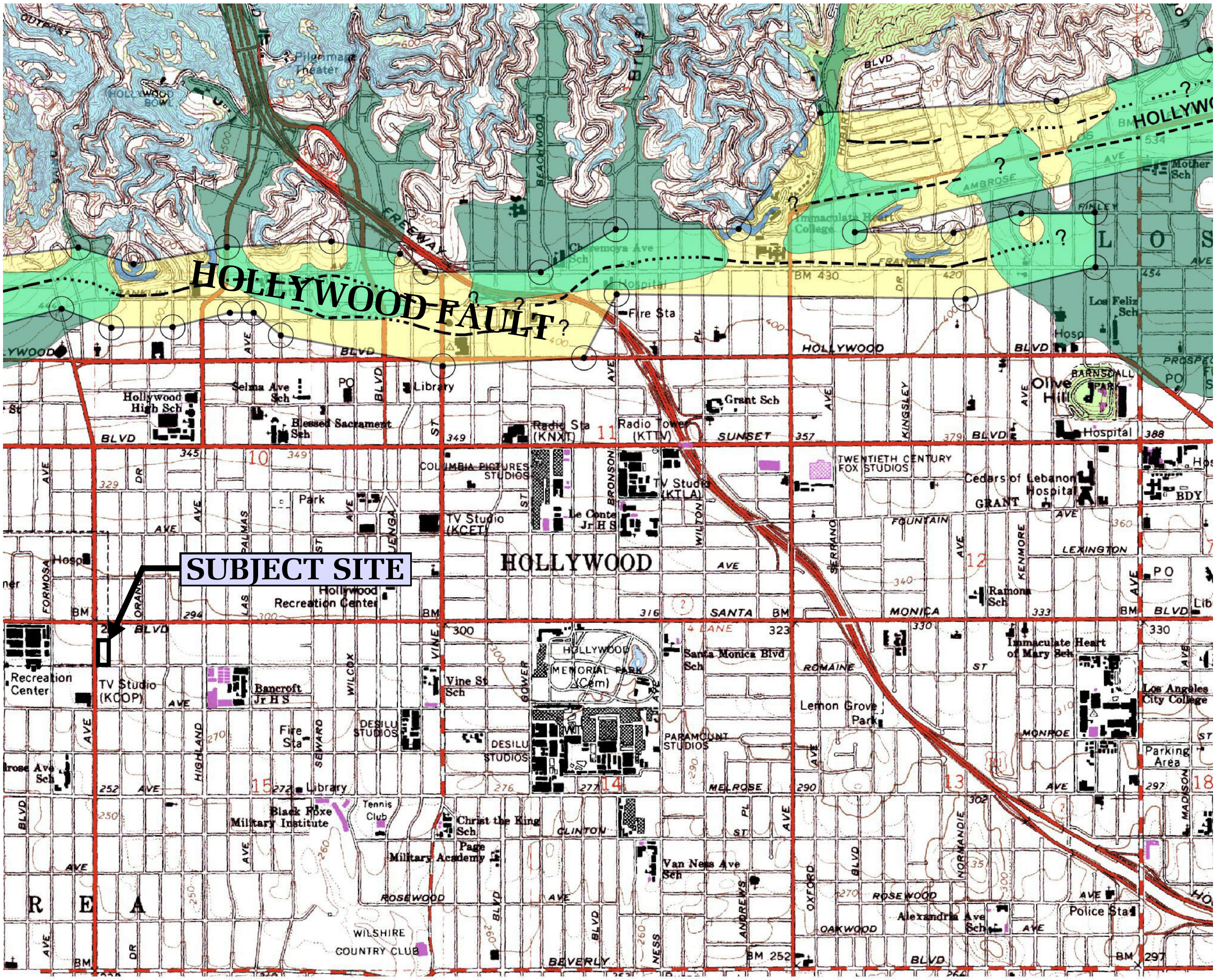
REFERENCE: California Department of Conservation, 2010 Fault Activity Map



Scale 1: 24000

1 0.5 0 1 2 MILES

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000 FEET

1 0.5 0 1 2 KILOMETERS

1000 500 0 1000 2000 METERS

Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

REFERENCE: EARTHQUAKE FAULT ZONES, HOLLYWOOD QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, NOVEMBER 2014

EARTHQUAKE FAULT ZONE

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