GEOTECHNICAL ENGINEERING EXPLORATION
PROPOSED SIX-STORY ADDITION OVER EXISTING FOUR-STORY BUILDING
LOT 1, TRACT 3431
6360 - 6366 WEST HOLLYWOOD BOULEVARD AND 1646 NORTH COSMO STREET
HOLLYWOOD, CALIFORNIA
FOR ARTIST GUILD HOTELS
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 23182
MARCH 11, 2020

INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geotechnical engineering exploration performed on the subject site. The purpose of this study is to evaluate the nature, distribution, engineering properties, and geologic hazards of the earth materials underlying the site with respect to construction of the proposed project. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from consultation with Mr. Daniel Hyde, client, and the as-built Footing Plan provided by Mr. Adam Greco, project structural engineer. The project consists of adding six stories atop the existing four-story concrete-frame building over a basement. Column loads (dead and live) from the additional six floors are expected to be up to 295 kips, which will total up to 590 kips.

RESEARCH

Research of agency records was conducted to locate previous geotechnical reports for the subject site and vicinity. No records were located.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Mr. Daniel Hyde. The as-built Footing Plan prepared by Edward Flaherty Engineer, provided by Mr. Adam Greco, project structural engineer, was a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Site Plan. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on January 21, 2020, with the aid of hand labor. It included excavating three test pits to approximate depths of 5½ to 21 feet below the basement slab-on-grade. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis. The test pits were visually logged by the project soils engineer. Following excavation, logging, and sampling, the test pits were backfilled, mechanically tamped, and patched with concrete.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, review of agency files, preparation of the Site Plan, engineering analysis, and preparation of this report. Earth materials exposed in the test pits are described on the enclosed Log of Test Pits. Appendix I contains a discussion of the laboratory testing procedures and results. The proposed project and the locations of the test pits are shown on the enclosed Site Plan.

SITE DESCRIPTION

The subject property consists of a rectangular-shaped lot that is located just south of the foothills of the Santa Monica Mountains, in the Hollywood section of the city of Los Angeles, California (34.1013° N Latitude, 118.3287° W Longitude). As depicted on the enclosed Aerial Vicinity Map, the property is bounded by Hollywood Boulevard on the north, two-story commercial buildings on the east and south, and Cosmo Street on the west. The property is located approximately one-third of a mile south of the Hollywood (101) Freeway and 200 feet east of Cahuenga Boulevard. A four-story concrete-frame building over a basement currently occupies the subject site. The surrounding area has been developed with single- and multi-family residential buildings, as well as commercial establishments along Hollywood Boulevard.

GROUNDWATER

Groundwater was not encountered in the test pits, which were extended to a depth of 21 feet below the basement level. In *Seismic Hazard Zone Report 026,* the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was greater than 80 feet below ground surface (CGS, 1998), as shown on the enclosed Historic-High Groundwater Map.

Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

METHANE ZONES

The City of Los Angeles Ordinance No. 175790 established methane mitigation requirements and includes construction standards to control methane intrusion into buildings. The subject property is not mapped within either a Methane Zone or Methane Buffer Zone.

EARTH MATERIALS

Fill

Fill, associated with backfilling above the existing column footings, was encountered to a maximum observed depth of 2½ feet in Test Pit 1. Greater depths of fill may occur locally. The fill consists of silty and clayey sand that is brown to dark brown, slightly moist, medium dense, and contains trace amounts of gravel and concrete debris.

Alluvium (Qae)

Natural alluvium typical for this portion of Hollywood underlies the subject site. The alluvium consists of silty sand and sand that are yellowish-brown and light olive-brown, slightly moist to moist, and medium dense to dense with trace amounts of fine to coarse gravel.

GENERAL SEISMIC CONSIDERATIONS

Regional Faulting

The subject property is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey (CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquake-resistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject property, and the property is not located within a currently-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2000). Therefore, the potential for surface rupture onsite is considered to be nil.

The known regional local active and potentially-active faults that could produce the most significant ground shaking on the site include the Santa Monica, Hollywood, and Newport-Inglewood Faults. Fifty faults were found within a 100-kilometer-radius search area from the site using EZ-FRISK V7.65 computer program. The results of seismic-source analysis are listed in Appendix II. The closest mapped "active" fault is the Santa Monica Fault, a Type B fault that is located 0.6 kilometers (0.4 miles) north of the site. The Santa Monica Fault is capable of producing a maximum moment magnitude of 7.4 and an average slip rate of 1.0 ± 0.5 millimeters per year (Cao et al., 2003). The San Andreas Fault, a Type A fault, is located 53.7 kilometers (33.3 miles) northeast of the site. General locations of regional active faults with respect to the subject site are shown on the enclosed Regional Fault Map (Appendix II).

Seismic Design Coefficients

The following table lists the applicable seismic coefficients for the project based on the current California Building Code:

SEISMIC COEFFICIENTS (2020 City of Los Angeles Building Code - Based on ASCE Standard 7-16)		
Latitude = 34.1013° N Longitude = 118.3287° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3.3, ASCE Standard 7-16	Alluvium - D	
Mapped Spectral Accelerations from Figures 22-1 and 22-2 and USGS	S_s = 2.117 (g)	S_1 = 0.750 (g)
Site Coefficients from Tables 11.4-1 and 11.4-2 and USGS	F_A = 1.0	F_V = 1.7 (g)
Maximum Considered Spectral Response Accelerations from Equations 11.4-1 and 11.4-2	S_{MS} = 2.117 (g)	S_{M1} = 1.275 (g)
Design Spectral Response Accelerations from Equations 11.4-3 and 11.4-4	S_{DS} = 1.411 (g)	S_{D1} = 0.850 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.998 (g)	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps,** http://earthquake.usgs.gov/designmaps/us/application.php

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is greater than or equal to 0.75g. Therefore, the project is considered to be in Seismic Design Category E.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Seismic Hazard Deaggregation Analysis

Probabilistic seismic hazard deaggregation analysis was performed on the subject site. Seismic parameters were determined using currently-available earthquake and fault information utilizing data from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project (USGS, 2008). An average shear-wave velocity (Vs30) of 259 meters-per-second (Site Class D) was used in the analysis. Hazard deaggregation indicates a predominant modal earthquake magnitude of 6.9 (Mw) at a modal distance of 5.5 kilometers. The Peak Horizontal Ground Acceleration (PHGA) with a 10-percent probability of exceedance in 50 years is estimated to be 0.53g on the subject site. These ground motions could occur at the site during the life of the project. Results of the analysis are graphically presented in the enclosed "Seismic Hazard Deaggregation Chart" (Appendix II).

Based on a Site Class D, the MCE_G peak ground acceleration adjusted for Site Class effects, PGA_M, is 0.998g. The pseudo-static seismic coefficient (k_h) was derived according to the LADBS memorandum dated July 16, 2014. The horizontal pseudo-static seismic coefficient (k_h) was selected as one-third of the PGA_M (0.33g). These ground motions could occur at the site during the life of the project.

Site-Specific Ground Motion Analysis

Site-specific ground motion analysis was performed in accordance with Chapter 21 of the American Society of Civil Engineers (ASCE) Standard 7-16. The probabilistic and deterministic seismic response spectra, based on maximum rotated component of spectral response at five-percent damping, are enclosed. The analysis is also based on a probability of exceedance of two percent in 50 years (2,475-return period). A computerized program, EZ-FRISK V7.65, was used to generate the seismic response spectra. An averaging of three Next Generation Attenuation relations (Chiou-Youngs 2007 NGA USGS 2008 MRC; Boore-Atkinson 2008 NGA USGS 2008 MRC; and Campbell-Bozorgnia 2008 NGA USGS 2008 MRC) was incorporated in both the probabilistic and deterministic analyses to estimate ground motions at the subject site. The deterministic response spectrum was generated using the 84th percentile of the maximum rotated component of spectral response at five-percent damping. A shear-wave velocity (Vs30) of 259 meters-per-second (Site Class D) was used in the analysis.

The design response spectrum was generated by multiplying the lesser of the deterministic and probabilistic response spectra by two-thirds, according to Sections 21.2.3 and 21.3 of ASCE Standard 7-16. The deterministic lower-limit response spectrum was determined according to Section 21.2.2 of the ASCE Standard 7-16. Spectral response accelerations for selected periods are shown in the following table:

Spectral Response Accelerations (g)*									
Seismic Response Spectra	Fundamental Period (seconds)								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Probabilistic MCE_R	1.9976	2.0606	2.0537	2.0182	1.9038	1.8145	1.6921	1.5624	1.4578
Probabilistic (ASCE 7-16)	1.4113	1.4113	1.4113	1.4113	1.4113	1.4113	1.4113	1.3889	1.2500
Deterministic MCE_R (84th Percentile)	1.4030	1.5760	1.6800	1.7620	1.7330	1.7070	1.6400	1.5430	1.4610
Deterministic Lower Limit on MCE_R Response Spectrum	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000	1.5000
Site Specific MCE_R	1.5000	1.5760	1.6800	1.7620	1.7330	1.7070	1.6400	1.5430	1.4580
80% Design Response Spectrum	1.1290	1.1290	1.1290	1.1290	1.1290	1.1290	1.1290	1.1110	1.0000
Site-Specific Design Response Spectrum	**1.1290**	**1.1290**	**1.1290**	**1.1750**	**1.1550**	**1.1380**	**1.1290**	**1.1110**	**1.0000**

* Reference: *American Society of Civil Engineers (ASCE), Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standard 7-16, 2016.*

The data included in the table above are plotted and presented in the enclosed Site-Specific Seismic Response Spectra figure (see Appendix II). Detailed calculations for fundamental periods up to eight seconds are also included in the "Site-Specific Ground Motion Analysis" table (see Appendix II).

As shown on the enclosed Site-Specific Seismic Response Spectra figure, the site-specific design response spectrum is equal or greater than or equal to 80 percent of the probabilistic response spectrum. According to Section 21.3 of ASCE Standard 7-16, the design response spectrum shall not be less than 80 percent of the probabilistic response spectrum.

Based on Section 21.4 of the ASCE Standard 7-16, the design earthquake spectral response acceleration parameters at short period, $S_{DS,}$ and at one-second period, S_{D1}, derived from the site-specific ground motion analysis, are shown in the following table:

SITE-SPECIFIC SPECTRAL RESPONSE ACCELERATION PARAMETERS (Based on ASCE Standard 7-16 - Chapter 21)		
Latitude = 34.1013° N Longitude = 118.3287° W	Short Period (0.2s)	One-Second Period
Maximum Considered Spectral Response Accelerations Chapter 21 - Section 21.4	S_{MS} = 1.694 (g)	S_{M1} = 1.539 (g)
Design Spectral Response Accelerations Chapter 21 - Section 21.4	S_{DS} = 1.129 (g)	S_{D1} = 1.026 (g)

<u>Liquefaction</u>

The CGS has not mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required, as shown on the enclosed Seismic Hazard Zones Map. Current and historic shallow groundwater levels are not present onsite and, therefore, liquefaction is not considered an issue to the proposed project.

<u>Seiches and Tsunamis</u>

Seiches are large waves generated in enclosed bodies of water, such as lakes and reservoirs, in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. The site is not located near any lake or reservoir. In addition, the site is at an average elevation of 386 feet above mean sea level and is located approximately 11.5 miles from the shoreline. Therefore, the risk to the project from seiches or tsunamis is considered nil.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, three test pits, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geotechnical engineering standpoint, provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is firm undisturbed alluvium, which is expected below the bottom of the existing footings. Conventional foundations may be used to support the additional loads from the proposed six-story addition. In addition, large mat-type foundations may be used to support any new shear walls. Soils to be exposed at finished grade are expected to exhibit a very low expansion potential.

FOUNDATION DESIGN

Spread Footings

Continuous and/or pad footings may be used to support the proposed six-story addition, provided they are founded in firm undisturbed alluvium. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Alluvium	24	6,000	0.30	200	6,000

For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is 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. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

Footings adjacent to retaining walls should be deepened below a 1:1 plane from the bottom of the lower retaining wall, or the footings should be designed as grade beams to bridge from the wall to the l:l plane.

All continuous footings should be reinforced with a minimum of four #4 steel bars: two placed near the top, and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geotechnical engineer prior to placing forms, steel, or concrete.

Mat Foundation

A mat foundation may be used to support new shear walls, provided it is founded in firm undisturbed alluvium. The minimum thickness of the mat should be 12 inches. The structural engineer may require a greater thickness. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Mat (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Alluvium	12	6,000	0.30	200	6,000

For bearing calculations, the weight of the concrete may be neglected. The bearing value shown above is 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. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

The bottom of the mat foundation should be free from loose material and construction debris, and should be approved by the geotechnical engineer prior to placing forms, steel, or concrete.

Modulus of Subgrade Reaction

The allowable modulus of subgrade reaction, k_1, is 250 kips-per-cubic-foot for a 12-inch by 12-inch footing. The modulus should be reduced for larger footings. For rectangular footings of dimensions B x L, the following formula may be used (Bowles, 1996):

$$k_s = k_1 * (m + 0.5) / (1.5 * m)$$

where k_s = Modulus of subgrade reaction for a full-size mat foundation,

m = L / B.

Foundation Settlement

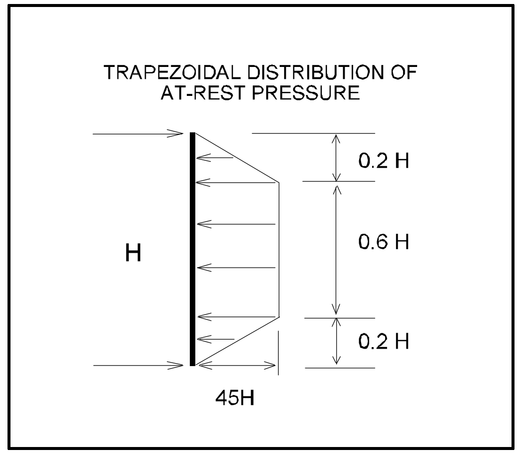
Settlement of the foundation system is expected to occur on initial application of loading. The static settlement analysis of the proposed building is based on a maximum allowable bearing pressure of 6,000 pounds-per-square-foot. Results of static settlement analysis indicate that a total static settlement of 1.07 to 1.33 inches may be anticipated for new footings impacted by additional loads. Differential static settlement should not exceed 0.25 inch across the footprint of the existing building.

It should be emphasized that settlement of the subsurface earth materials has already occurred immediately following the construction of the existing building. Therefore, additional settlement is not anticipated for the existing foundation system, if no additional loads are applied on the existing footings.

RETAINING WALLS

General Design

The following retaining wall recommendations are intended to evaluate the structural adequacy and upgrade the existing basement walls, if deemed necessary by the structural engineer.



Restrained retaining walls, such as the existing basement walls, should be designed for the at-rest lateral earth pressure of 45H, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. The design earth pressures assume that the walls are free draining. Surcharge loads from vehicular traffic and adjacent buildings should be added to the at-rest pressure for restrained retaining walls. Surcharge loads may be calculated using NAVFAC DM-7.02 Design Manual, or an equivalent method.

Basement walls should be provided with a subdrain or gravel pockets covered with a minimum of 12 inches of ¾-inch crushed gravel. A sump pump may be required for basement subdrains.

Foundation Design

Basement walls are currently supported on the existing foundation system. The foundation recommendations included in the "Spread Footings" and "Mat Foundation" sections of this report may be incorporated to upgrade the existing basement wall footings, if deemed necessary by the structural engineer.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

TEMPORARY EXCAVATIONS

Temporary excavations will be required to construct new footings. The excavations are expected to be up to five feet in height and will expose only if over an existing pad fill over alluvium. The fill should be trimmed to 1:1 for wall excavations. The alluvium is capable of maintaining vertical excavations up to five feet (see Calculation Sheet #3). Where vertical excavations in the alluvium exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

The geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

FLOOR SLABS

New sections of floor slabs should be cast over undisturbed firm alluvium or approved compacted fill and reinforced with a minimum of #4 bars on 16-inch centers, each way. Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be sandwiched between the layers of sand, about two inches each, to prevent punctures and aid in the concrete cure. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

CEMENT TYPE AND CORROSION PROTECTION

A representative sample of the near-surface soil was obtained during field exploration for laboratory testing. Corrosion test results are included in Appendix I. The results indicate that concrete structures in contact with the soils onsite will have negligible exposure to water-soluble sulfates in the soil. According to Table 4.3.1 of Section 4.2 of the ACI 318 Code, Type II cement may be used for concrete construction.

The results of the laboratory testing also indicate that the near-surface soil onsite is considered corrosive to ferrous metals. Special mitigation measures for corrosion protection of steel and other metallic elements in contact with the soil may be required. The corrosion information presented in Appendix I of this report should be provided to the underground utility subcontractor.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, temporary excavations, and subdrains. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

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Historic-High Groundwater Map
Site Plan

xc: (4) Addressee (E-mail and Mail)
(1) AMG Structural Engineers, Attention: Adam Greco (E-mail)

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U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps,** http://earthquake.usgs.gov/designmaps/us/application.php.

Software

EZ-FRISK 7.65, Risk Engineering, Inc.

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the alluvium were obtained from the test pits and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-10. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-10. The results are shown on the enclosed Log of Test Pits.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill were determined using the procedures outlined in ASTM D 1557-12, a five-layer standard. The results are shown in the following table.

Test Pit	Depth (Feet)	Earth Material	USCS + Color Soil Type	Maximum Density (pcf)	Optimum Moisture%	Expansion Index
2	0 - 5	Fill/ Alluvium	Silty Clayey Sand Dark Brown	124.0	12.0	9 - Very Low

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D 4829-11. Based upon the testing, soils at the basement grade are expected to exhibit a very low expansion potential.

APPENDIX I (Continued)

Shear Tests

Shear tests were performed on samples of the alluvium using the procedures outlined in ASTM D 3080-11 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the enclosed Shear Test Diagram.

Consolidation

Consolidation tests were performed on *in situ* samples of the alluvium using the procedures outlined in ASTM D 2435-11. Results are graphed on the enclosed Consolidation Curves.

Corrosion

A representative bulk sample of the near-surface soil was transported to Environmental Geotechnology Laboratory for chemical testing. The testing was performed in accordance with Caltrans Standards 643 (pH), 422 (Chloride Content), 417 (Sulfate Content), and 532 (Resistivity). The results of the testing are reported in the following table:

CHEMICAL TEST RESULTS TABLE

Sample	Depth (Feet)	pH	Chloride (PPM)	Sulfate (%)	Resistivity (Ohm-cm)
TP2	0 - 5	9.28	415	0.03	1,200

The chloride and sulfate contents of the soil are negligible and not a factor in corrosion. The pH is near neutral and not a factor. The resistivity indicates that the soil is considered corrosive to ferrous metals.

APPENDIX I

Laboratory Testing and Log of Test Pits

APPENDIX II

Calculations and Figures

March 11, 2020
BG 23182

Artist Guild Hotels
1315 North El Camino Real
San Clemente, California 92672

Attention: Mr. Daniel Hyde

Subject

Transmittal of Geotechnical Engineering Exploration
Proposed Six-Story Addition over Existing Four-Story Building
Lot 1, Tract 3431
6360 - 6366 West Hollywood Boulevard and 1646 North Cosmo Street
Hollywood, California

Dear Mr. Hyde:

Byer Geotechnical has completed our report dated March 11, 2020, which describes the geotechnical engineering conditions with respect to the proposed project. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety (LADBS). The reviewing agency requires three unbound copies, one with a wet signature, a CD (PDF format), an application form, and a filing fee. Copies of the report have been distributed as follows:

(4) Addressee (E-mail and Mail)
(1) AMG Structural Engineers, Attention: Adam Greco (E-mail)

It is our understanding that you or your representative will file the report and CD with the LADBS. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Senior Project Engineer