BYER GEOTECHNICAL, INC.

September 25, 2019
BG 23084

3700 West Riverside Investments, LLC
127 North Madison Avenue, Suite 200
Pasadena, California 91101

Attention: Ms. Zovi Seferian

Subject

Transmittal of Geotechnical Engineering Exploration
Proposed Six-Story with Mezzanine Mixed-Use Building over Subterranean Parking
Assessor's Parcel Nos. 2485-005-004, -014, and -015
3700 West Riverside Drive and 134 North Screenland Drive
Burbank, California

Dear Ms. Seferian:

Byer Geotechnical has completed our report dated September 25, 2019, which describes the geotechnical engineering conditions with respect to the proposed project. The reviewing agency for this document is the City of Burbank, Building Division. The reviewing agency requires two unbound copies, one with wet signature. Four copies of the report are enclosed.

It is our understanding that you or your representative will file the report with the City of Burbank. 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

BYER GEOTECHNICAL, INC.

GEOTECHNICAL ENGINEERING EXPLORATION

PROPOSED SIX-STORY WITH MEZZANINE MIXED-USE BUILDING OVER SUBTERRANEAN PARKING

ASSESSOR'S PARCEL NOS. 2485-005-004, -014, AND -015

3700 WEST RIVERSIDE DRIVE AND 134 NORTH SCREENLAND DRIVE

BURBANK, CALIFORNIA

FOR 3700 WEST RIVERSIDE INVESTMENTS, LLC

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 23084

SEPTEMBER 25, 2019

1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

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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 Ms. Zovi Seferian and the preliminary plans prepared by Struere Advanced Architecture, dated February 15, 2019. Final plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a six-story mixed-use building with a mezzanine level over one subterranean parking level. The ground floor of the proposed building will consist of a concrete-frame retail space and building amenities fronting on Riverside Drive, and parking spaces to the rear. The upper six levels will consist of wood-frame residential units with a mezzanine level above. Retaining walls up to 12 feet high are planned to support the excavation for the subterranean parking level. Foundation loads are expected to be moderate. The existing car wash facility and associated improvements are to be removed.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Ms. Zovi Seferian. The preliminary plans prepared by Struere Advanced Architecture, dated February 15, 2019, were 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 and cross sections. 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 July 17, 2019, with the aid of a hollow-stem-auger drill rig. It included drilling four borings to approximate depths of 36½ to 61½ feet below existing grade. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis. The borings tailings were visually logged by the project soils engineer.

Following drilling, logging, and sampling, the borings were backfilled and mechanically tamped, and patched with asphalt.

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 cross sections, preparation of the Site Plan, engineering analysis, and preparation of this report. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results. Appendix II contains the results of liquefaction analysis.

The proposed project and the locations of the borings are shown on the enclosed Site Plan. Subsurface distribution of the earth materials and the proposed project are shown on Sections A and B.

SITE DESCRIPTION

The subject property consists of a partially-graded, relatively-level parcel located in the southeast portion of the San Fernando Valley in the west portion of the city of Burbank, California (34.1525° N Latitude, 118.3402° W Longitude). As depicted on the enclosed Aerial Vicinity Map, the property is bounded by Riverside Drive on the north, a commercial establishment and a parking lot on the south, Hollywood Way on the east, and Screenland Drive on the west. The property is located approximately 280 feet south of the Ventura (134) Freeway. A car wash facility currently occupies the subject property. The surrounding area has been developed with low- and mid-rise commercial buildings along Riverside Drive and Hollywood Way, as well as single- and multi-family residential buildings behind.

Past grading on the site has consisted of placing minor amounts of fill to create a level pad for the existing car wash facility.

Vegetation on the site consists of hedges and a few trees adjacent to the east and west property lines. Surface drainage is by sheetflow runoff down the contours of the land to the east-southeast.

GROUNDWATER

Groundwater was not encountered in the borings to a maximum depth of 61½ feet below existing grade. In *Seismic Hazard Zone Report 016,* the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was on the order of 10 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.

EARTH MATERIALS

Fill (Afu)

Fill, associated with previous site grading, underlies the northwest portion of the site to a maximum observed depth of 1½ feet in Boring 3. Greater depths of fill may occur locally. The fill consists of silty sand that is olive-brown, moist, and contains concrete debris. Based on the current configuration of the proposed building, any fill will be removed during the excavation for the subterranean parking level.

Alluvium (Qa)

Natural alluvium underlies the subject site and was encountered in the borings. The upper 45 feet of alluvium consists of layers of sand, silty sand, and sandy silt that are light to dark olive-brown and olive-gray, slightly moist to moist, loose in the upper 10 feet becoming medium dense below, and stiff to very stiff. Alluvium below the depth of 45 feet generally consists of gravelly sand that is olive-gray and olive-brown, slightly moist to moist, and medium dense to very dense, with varying amounts of fine- to coarse-grained 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 very low.

The known regional local active and potentially-active faults that could produce the most significant ground shaking on the site include the Hollywood, Santa Monica, and Verdugo Faults. Forty-two 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 Hollywood Fault, a Type B fault that is located 4.8 kilometers (3 miles) south of the site. The Hollywood Fault is capable of producing a maximum moment magnitude of 6.7 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 49 kilometers (30.5 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 California Building Code:

SEISMIC COEFFICIENTS (2019 California Building Code - Based on ASCE Standard 7-16)		
Latitude = 34.1524° N Longitude = 118.3402° 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 = 2.096 (g)	S_1 = 0.699 (g)
Site Coefficients from Tables 11.4-1 and 11.4-2 and USGS	F_A = 1.0	F_V = 1.7
Maximum Considered Spectral Response Accelerations from Equations 11.4-1 and 11.4-2	S_{MS} = 2.096 (g)	S_{M1} = 1.188 (g)
Design Spectral Response Accelerations from Equations 11.4-3 and 11.4-4	S_{DS} = 1.397 (g)	S_{D1} = 0.792 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.984 (g)	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps Web Services,** http://earthquake.usgs.gov/hazards/designmaps/

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 (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)*									
	Fundamental Period (seconds)								
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Probabilistic MCE_R	1.9865	2.0125	1.9644	1.8964	1.7627	1.6416	1.5226	1.4045	1.3101
Probabilistic (ASCE 7-16)	1.3847	1.3847	1.3847	1.3847	1.3847	1.3847	1.3847	1.3556	1.2200
Deterministic MCE_R (84^{th} Percentile)	1.5060	1.6390	1.6970	1.7270	1.6670	1.6140	1.5330	1.4300	1.3440
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
80% Design Response Spectrum	1.1080	1.1080	1.1080	1.1080	1.1080	1.1080	1.1080	1.0840	0.9760
Site-Specific Design Response Spectrum	**1.1080**	**1.1080**	**1.1310**	**1.1510**	**1.1110**	**1.1080**	**1.1080**	**1.0840**	**0.9760**

* 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 graphically 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 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 1.108g and 0.976g, respectively.

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 Design Category

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is less than 0.75g. The design spectral response acceleration parameters for the site for a 1-second period (S_{D1}) is greater than 0.20g, and the short period (S_{DS}) is greater than 0.50g. Therefore, the project is considered to be in Seismic Design Category D.

Liquefaction

The CGS has 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.

Liquefaction is a process that occurs when saturated sediments are subjected to repeated strain reversals during an earthquake. The strain reversals cause increased pore water pressure such that the internal pore pressure approaches the overburden stress and the shear strength approaches zero. Liquefied soils may be subject to flow or excessive strain, which may induce settlement. Liquefaction occurs in soils below the groundwater table. Soils commonly subject to liquefaction include loose to medium-dense sand and silty sand. Predominantly fine-grained soils, such as silts and clay, are less susceptible to liquefaction. Generally, medium dense to dense sand-like soils with fines content (percent passing the No. 200 sieve) greater than 35 percent are not considered susceptible to liquefaction. In addition, cohesive soils with Plasticity Index (PI) values between 12 and 18 and a saturated moisture content less than 80 percent of the Liquid Limit (LL) are not considered susceptible to liquefaction (CGS, 2008, and Bray and Sancio, 2006). Cohesive soils with

PI greater than 18 may be susceptible to liquefaction, if considered sensitive (CGS, 2008). Soil sensitivity is the ratio of the undisturbed shear strength of a cohesive soil to the remolded shear strength at the same water content (Bowles, 1996). Based on the study conducted by Bray and Sancio on soils affected by the 1999 earthquakes in Taiwan and Turkey, soils with a PI greater than 18 tested at low confining effective stresses are not considered susceptible to liquefaction (Bray and Sancio, 2006).

Soils data collected in Boring B1 was utilized to quantify the liquefaction potential of the site. The following input parameters were incorporated in the liquefaction analysis:

Liquefaction Analysis Input Parameters	
Peak Ground Acceleration (g)	0.967 (PGA_M)
Probability of Exceedance in 50 Years	2%
Return Period	2,475 Years
Earthquake Magnitude (Mw)	6.9
Factor of Safety	1.3

For a conservative analysis, it was assumed that groundwater rose to the historic-high groundwater level of 10 feet below the ground surface (see "Groundwater" section of this report).

Laboratory testing consisting of Atterberg Limits (ASTM D 4318-10) and sieve analysis by wash method (ASTM D 1140-14) was performed on representative samples of the earth materials collected in Boring B1. The purpose of these tests was to determine the liquid limit, plasticity index (PI), and fines content (percent passing the No. 200 sieve) and incorporate the results in the liquefaction analysis. The results are shown on the Laboratory Testing program in Appendix I, as well as on the enclosed liquefaction calculations (Appendix II).

A liquefaction potential analysis based upon SPT data from Boring B1 is presented in Appendix II on the plates entitled "Liquefaction Susceptibility Analysis: SPT Method." The column labeled "Factor of Safety" lists the calculated safety factor of each 2½-foot-thick layer of soil encountered in the boring. In addition, a borehole diameter correction factor (C_B) of 1.15 was incorporated in the analysis to account for the stress relief, since the tip of the auger was raised a few inches from the bottom of the hole prior to driving the sampler. The stresses and safety factors for liquefaction were calculated using the methodology of Youd et al. (2001) and Special Publication 117A (CGS, 2008). Soils with a factor of safety less than 1.3 were considered susceptible to liquefaction.

Quantitative evaluation and screening analysis was performed to determine the depths and limits of potentially-liquefiable soil layers encountered in Boring B1 below the historic-high groundwater level. The results are summarized in the following table:

Results of Quantitative Evaluation and Screening Analysis											
Boring No.	Layer Depth (feet)	Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI (%)	Fines Content (%)	Soil Type & Unit	*In-Situ* Moisture Content (%)	Saturated Moisture Content w_c (%)	$(N_1)_{60cs}$	Screening Criteria	Result
B1	17.5	-	-	-	24.6	Sand (SM)	-	-	26.2	CRR < CSR	Liquefiable
B1	20.0	-	-	-	54.1	Silt (ML)	-	-	25.3	CRR < CSR	Liquefiable
B1	22.5	32.2	18.3	13.9	61.2	Clay (CL)	12.6	12.6	29.4	w_c/ LL ≤ 0.8	Non-Liquefiable
B1	25.0	-	-	-	19.1	Sand (SM)	-	-	27.1	CRR < CSR	Liquefiable
B1	27.5	-	-	-	50.4	Sand (SM)	-	-	28.6	CRR < CSR	Liquefiable
B1	30.0	-	-	-	12.9	Sand (SM)	-	-	37.5	CRR > CSR	Non-Liquefiable
B1	32.5	-	-	-	49.5	Sand (SM)	-	-	34.0	CRR > CSR	Non-Liquefiable
B1	35.0	-	-	-	48.3	Sand (SM)	-	-	34.4	CRR > CSR	Non-Liquefiable
B1	37.5	-	-	-	28.7	Sand (SM)	-	-	34.4	CRR > CSR	Non-Liquefiable
B1	40.0	-	-	-	13.9	Sand (SM)	-	-	30.8	CRR > CSR	Non-Liquefiable
B1	42.5	-	-	-	27.1	Sand (SM)	-	-	32.1	CRR > CSR	Non-Liquefiable
B1	57.5	-	-	-	64.6	Silt (ML)	-	-	30.2	CRR > CSR	Non-Liquefiable

It should be noted that the earth materials below the recommended mat foundation will be removed to a depth of 16 feet below ground surface and replaced as future compacted fill. The compacted fill layer is not considered susceptible to liquefaction. Foundation and site preparation recommendations are included in the "Conclusions and Recommendations" section of this report.

The results of liquefaction analysis indicate that there are four, 2½-foot-thick layers of soil, located between the depths of 16 and 27½ feet, that are considered susceptible to liquefaction.

Dynamic Settlement

Earthquake-induced volumetric strain and dissipation of pore pressure in saturated silts and sands after liquefaction can result in settlement. The potential for liquefaction-induced settlement was calculated using the methodology of Tokimatsu and Seed (1987). The seismic settlement potentials were calculated for all granular soil layers at depths below the historic-high groundwater level and with a factor of safety for liquefaction less than 1.3, as described in the "Liquefaction" section above. Based on the results of liquefaction analysis, seismic settlement calculations indicate a total dynamic settlement potential of 2 inches. Differential dynamic settlement potential is expected to be one-half to two-thirds of the total dynamic settlement (1 to 1.3 inches).

Lateral Spreading Hazard

Liquefied soils may be subject to lateral spreading flow failure where adjacent to slopes or "free-faces" such as steep slopes or embankments. The subject property is remote to free-faces, slopes, and canals, and a lateral spreading flow failure is not indicated for the potentially-liquefiable alluvial soils. Therefore, it is the opinion of Byer Geotechnical, Inc., that the lateral spreading hazard at the site is nil, and no mitigation as defined in Public Resources Code Section 2693(c) is required for lateral spreading.

Seiches and Tsunamis

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. Furthermore, the site is at an average elevation of 553 feet above mean sea level and is located approximately 13 miles from the shoreline. Therefore, the risk to the project from seiches or tsunamis is considered very low.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, four borings, 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.

Based on the findings of the field exploration conducted onsite, the upper 10 to 15 feet of the earth materials underlying the subject site are considered loose to medium dense and potentially liquefiable and, therefore, are not suitable to support the proposed building. Remedial grading is required to prepare a firm compacted-fill pad underneath the mat foundation. Recommendations for removal and recompaction are included in the "Site Preparation - Removals" section below.

The recommended bearing material is a future compacted-fill blanket below the subterranean garage level. A mat foundation is recommended to support the proposed building. Soils to be exposed at finished grade are expected to exhibit a low expansion potential.

Geotechnical issues affecting the project include temporary excavations up to 16 feet in height, including the estimated thickness of the mat foundation and a compacted-fill blanket. Temporary shoring, consisting of soldier piles and continuous lagging, is recommended to facilitate the construction of the subterranean retaining walls, the mat foundation subgrade preparation, and to support offsite improvements. Recommendations for temporary shoring are included in the "Temporary Excavations" section of this report.

SITE PREPARATION - REMOVALS

Surficial materials, consisting of loose alluvium, blankets the site. Remedial grading is recommended to improve site conditions. The alluvium below the mat foundation should be removed to three feet below the bottom of the mat and replaced as certified compacted fill. The following general grading specifications may be used in preparation of the grading plan and job specifications. Byer Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The area to receive compacted fill should be prepared by removing all vegetation, demolition debris, existing fill, and upper alluvium. The exposed excavated area should be observed by the soils engineer/geologist prior to placing compacted fill. Removal depths can be found in the "Site Preparation - Removals" section above. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum dry density.

B. The proposed building site shall be excavated to a minimum depth of three feet below the bottom of the mat foundation. The excavated areas shall be observed by the soils engineer/geologist prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts, moistened as required, and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The moisture content of the fill should be near the optimum moisture content. When the moisture content of the fill is too wet or dry, the fill shall be moisture conditioned and mixed until the proper moisture is attained.

E. The fill shall be compacted to at least 90 percent of the maximum laboratory dry density for the material used. The maximum dry density shall be determined by ASTM D 1557-12 or equivalent.

F. Field observation and testing shall be performed by the soils 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 90 percent relative compaction is obtained. A minimum of one compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

FOUNDATION DESIGN

Mat Foundation

A mat foundation is recommended to support the proposed building, provided it is founded in future compacted fill. 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)
Future Compacted Fill	12	3,000	0.36	200	3,000

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 design of the mat foundation should incorporate a theoretical hydrostatic pressure measured from the historic-high groundwater level (10 feet below grade) to the bottom of the mat.

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_l, is 240 kips-per-cubic-foot for a 12-inch by 12-inch footing. The modulus should be reduced for larger footings, such as the proposed mat. For rectangular footings of dimensions B x L, the following formula may be used (Bowles, 1996):

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

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

$sm = L / B$.

Foundation Settlement

Settlement of the mat foundation system is expected to occur on initial application of loading. A total settlement of one inch may be anticipated. Differential settlement should not exceed one-half of an inch across the footprint of the proposed building.

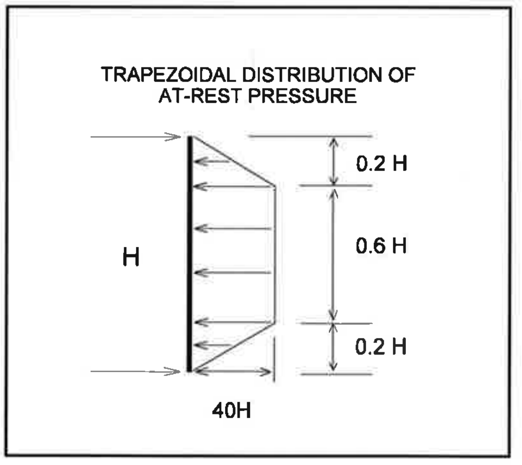
Based on the results of liquefaction analysis performed on the site, a total dynamic settlement of 2 inches and a differential dynamic settlement of 1 to 1.3 inches are possible in the event of a strong earthquake nearby. Therefore, the combined total settlement (static and dynamic) is estimated to be on the order of 3 inches, and the combined differential (static and dynamic) settlement is estimated to range from 1.5 to 1.85 inches.

RETAINING WALLS

General Design

Cantilever retaining walls up to 12 feet high, with a level backslope and uniform vehicular surcharge of 300 pounds, may be designed for an active equivalent fluid pressure of 46 pounds-per-cubic-foot (see Calculation Sheet #1a). Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.

Subterranean retaining walls, which will be restrained, should be designed for an at-rest lateral earth pressure of 40H, where H is the height of the wall (see Calculation Sheet #2a). 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 may be calculated using NAVFAC DM-7.02 Design Manual, or an equivalent method.



Seismic analysis of the cantilever retaining walls indicates that an additional load of 497 pounds is required due to seismic forces for a retained height up to 12 feet (see Calculation Sheet #1Sa). This corresponds to an additional equivalent fluid pressure of 7 pounds-per-cubic-foot. The seismic load should be applied at 0.3H measured from the bottom of the wall.

Seismic analysis of the restrained retaining walls indicates that no additional loading due to seismic forces is required for a retained height up to 12 feet (see Calculation Sheet #2Sa).

Subterranean retaining walls should be provided with a subdrain covered with a minimum of 12 inches of ¾-inch crushed gravel. An alternative subdrain system consisting of Miradrain and gravel pockets connected to a solid pipe outlet may be used behind the subterranean retaining walls. The gravel pockets should be placed at the bottom of the retaining wall, midway between the shoring bays. A sump pump will be required for basement subdrains. The gravel pockets should be excavated to penetrate the slurry backfill behind the lagging to ensure contact with the earth materials behind the lagging.

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557-12, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾-inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Retaining walls may be supported on the mat foundation.

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 the subterranean retaining walls of the proposed building and to support offsite improvements. The excavations are expected to be up to about 16 feet in height, including the estimated thickness of the mat foundation, and will expose minor fill over alluvium. The fill and alluvium are capable of maintaining vertical excavations up to five feet. Where vertical excavations exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

Vertical excavations adjacent to property lines and public right-of-way will require the use of temporary shoring such as soldier piles. Design values can be found in the "Soldier Piles" section below.

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.

Soldier Piles

Drilled, cast-in-place concrete soldier piles may be utilized as temporary shoring to support excavations to construct the subterranean retaining walls of the proposed building and to support offsite improvements. The piles should be a minimum of 18 inches in diameter and a minimum of eight feet into the alluvium below the excavation. Piles may be assumed fixed at three feet into the alluvium below the excavation. The piles may be designed for a skin friction of 500 pounds-per-square-foot for that portion of pile in contact with the alluvium below the excavation. Piles should be spaced a maximum of eight feet on center. Shoring spacing may be increased up to 10 feet on center in local areas such as ramp approaches and corners of shoring.

The soldier piles may be designed for an active equivalent fluid pressure of 41 pounds-per-cubic-foot (see Calculation Sheet #3a). If rakers are incorporated in the temporary shoring system, the soldier piles should be designed for a trapezoidal lateral earth pressure of 26H, where H is the height of shoring.

The equivalent fluid pressure should be multiplied by the pile spacing. The piles may be included in the permanent retaining wall. Where a combination of sloped embankment and shoring is used, the pressure will be greater and must be determined for each combination.

Lateral Design

The friction value 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. Resistance to lateral loading may be provided by passive earth pressure within the alluvium.

Passive earth pressure may be computed as an equivalent fluid having a density of 200 pounds-per-cubic-foot. The maximum allowable earth pressure is 3,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Rakers

Rakers may be used to internally brace the soldier piles. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 3,000 pounds-per-square-foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

Lagging

Continuous lagging is anticipated between the soldier piles. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds-per-square-foot. The space behind lagging should be backfilled with cement slurry.

Lagging should be placed behind the front flange of the shoring steel I-beams. In some cases, the shoring is designed with the lagging behind the rear flange of the shoring steel I-beams. This is to maximize the interior area and position the walls as near the property lines as possible. During the installation of lagging behind the rear flange, the shoring is not supporting the excavation while the lagging is placed and backfilled. This can cause damage to adjacent offsite improvements, such as buildings, site walls, sidewalks, etc. If lagging is to be placed behind the rear flange of the I-beams, the lagging should be installed in slot cuts (ABC method), where lagging is installed and slurry-backfilled in the "A" slots before the "B" and "C" slots are excavated for lagging. Also, the maximum vertical height exposed should be no more than five feet.

Deflection

Some deflection of the shored embankment should be anticipated. Where shoring is planned adjacent to existing structures, it is recommended that lateral deflection not exceed one-half of an inch. For shoring not surcharged by a structure, the allowable deflection is deferred to the structural engineer. If greater deflection occurs during construction, additional bracing or anchors may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

EXTERIOR CONCRETE DECKS

Exterior concrete decking should be cast over 12 inches of approved compacted fill and reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent 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. The subgrade should be moistened prior to placing concrete.

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 copper and severely 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.

Low Impact Development (LID) Requirements

Typically, infiltration systems are utilized in areas underlain by pervious granular earth materials that have high percolation characteristics. In addition, infiltration systems are normally planned at least 10 feet from adjacent property lines or public right-of-way, and 10 feet from a 1:1 plane projected from the bottom of adjacent structural foundations. The subject site is located within a liquefaction zone and the results of liquefaction analysis indicate the presence of potentially liquefiable soil layers underneath the proposed building. Therefore, onsite infiltration is not recommended.

As an alternative, a biofiltration system, a capture-and-reuse system, or equivalent, may be installed on the site. A planter box may be used to capture and treat storm-water runoff through different soil layers before discharging water to the street storm drain. The planter box should be an impermeable rigid structure that is equipped with an underdrain to prevent water infiltration to the underlying subsurface earth materials. Planter boxes may be situated aboveground and placed adjacent to buildings. Planter boxes should be designed as freestanding and for an inward equivalent fluid pressure of 43 pounds-per-cubic-foot. This fluid pressure includes possible vehicular surcharge. Byer Geotechnical, Inc., should be provided with the final plans to verify the location of the planter boxes.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

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 should observe bottoms for fill, compaction of fill, soldier pile excavations, lagging, raker footings, 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. The geologist will prepare reports summarizing pile excavations.

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.

REGISTERED PROFESSIONAL ENGINEER
RAFFI S. BABAYAN
No. 72168
Exp. June 30, 2020
CIVIL
STATE OF CALIFORNIA

Raffi S. Babayan
P. E. 72168

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 06-30-20
GEOTECHNICAL
STATE OF CALIFORNIA

Robert I. Zweigler
G. E. 2120

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Enc: List of References (2 Pages)
- Appendix I - Laboratory Testing and Log of Borings
  - Laboratory Testing (3 Pages)
  - Shear Test Diagrams (2 Pages)
  - Consolidation Curves (8 Pages)
  - Plasticity Chart
  - Log of Borings 1 - 4 (9 Pages)
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  - Seismic Sources (2 Pages)
  - Seismic Hazard Deaggregation Charts (2 Pages)
  - Site-Specific Ground Motion Analysis
  - Site-Specific Seismic Response Spectra
  - Liquefaction Susceptibility Analysis: SPT Method (2 Pages and Sheet)
  - Retaining Wall Calculation Sheets (8 Pages)
  - Shoring Pile Calculation Sheets (2 Pages)
  - Aerial Vicinity Map
  - Regional Topographic Map
  - Historic Topographic Map
  - Regional Geologic Map
  - Regional Fault Map
  - Seismic Hazard Zones Map
  - Historic-High Groundwater Map
  - Site Plan
  - Sections A and B (2 Sheets)

xc: (4) Addressee (Email and Mail)

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Software

EZ-FRISK 7.65, Risk Engineering, Inc.
Settle3D, Rocscience, Inc.

APPENDIX I

Laboratory Testing and Log of Borings

LABORATORY TESTING

Undisturbed and bulk samples of the alluvium were obtained from the borings 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 inch 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 Borings.

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. Remolded samples were prepared at 90 percent of the maximum dry density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Earth Material	Soil Type and Color	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
4	0 - 10	Alluvium	Silty Sand Olive-Brown	120.0	11.0	7 - 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, the soil at the subterranean garage grade is expected to exhibit a very low expansion potential.

Shear Tests

Shear tests were performed on samples of the alluvium and future compacted fill 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 inch 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 Diagrams.

LABORATORY TESTING (Continued)

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.

Atterberg Limits

Atterberg limits were determined on a representative sample of the alluvium obtained from Boring B1 at a depth of 22½ feet using the procedures outlined in ASTM D 4318-10. The tests were performed to assist in the engineering classification of the fine-grained materials and to determine the Liquid Limit (LL) and Plasticity Index (PI). Results of Atterberg Limits are graphed on the enclosed Plasticity Charts and shown in the following table:

Results of Atterberg Limits Laboratory Tests						
Boring No.	Depth (feet)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Soil Type	Reference
B1	22.5	32.2	18.3	13.9	Clay (CL)	Plasticity Chart #1

Fines Content

Sieve analysis (wash method) was performed on representative samples of the alluvium obtained from Boring B1 using the procedures outlined in ASTM D 1140-14. The tests were performed to assist in the classification of the soil and to determine the fines content (percent passing #200 sieve). The results are shown on the enclosed Log of Boring B1 and are summarized in the following table:

Results of Sieve Analysis (Wash Method) Laboratory Tests							
Boring No.	Depth (feet)	Fines Content (%)	Soil Type	Boring No.	Depth (feet)	Fines Content (%)	Soil Type
B1	17.5	24.6	Silty Sand (SM)	B1	32.5	49.5	Silty Sand (SM)
B1	20.0	54.1	Sandy Silt (ML)	B1	35.0	48.3	Silty Sand (SM)
B1	22.5	61.2	Sandy Clay (CL)	B1	37.5	28.7	Silty Sand (SM)
B1	25.0	19.1	Silty Sand (SM)	B1	40.0	13.9	Sand w/Silt (SP-SM)
B1	27.5	50.4	Silty Sand (SM)	B1	42.5	27.1	Silty Sand (SM)
B1	30.0	12.9	Sand w/Silt (SP-SM)	B1	57.5	64.6	Sandy Silt (ML)

LABORATORY TESTING (Continued)

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)
B4	0 - 10	7.33	540	0.013	600

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

BYER
GEOTECHNICAL
INC.
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

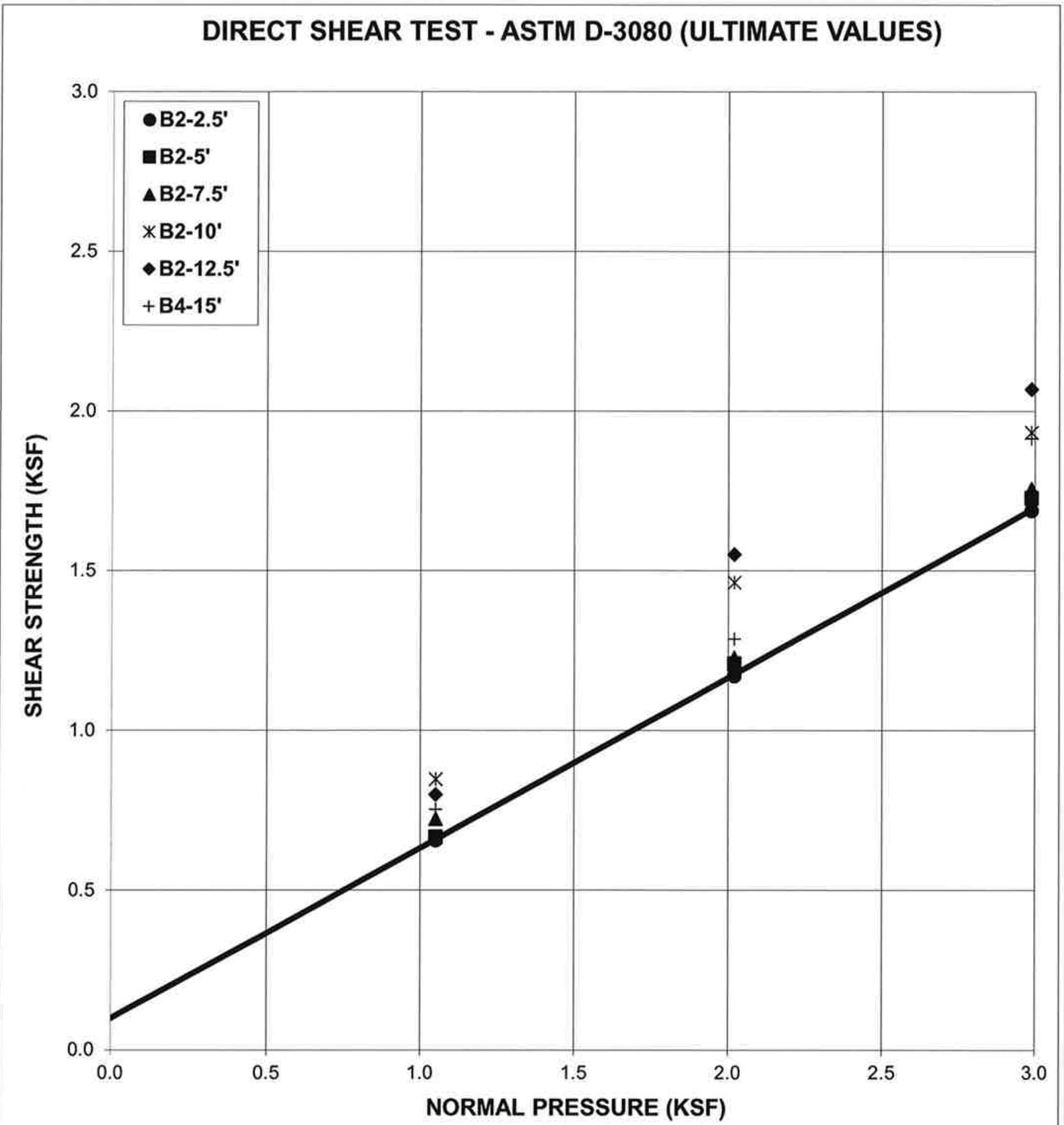
SHEAR TEST DIAGRAM #1

BG: 23084 ENGINEER: RSB
CLIENT: 3700 West Riverside Investments, LLC

EARTH MATERIAL: **Alluvium**

Phi Angle = 28.0 degrees
Cohesion = 100 psf

Average Moisture Content 24.2%
Average Dry Density (pcf) 101.4
Average Saturation 99%



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GEOTECHNICAL
INC.

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tel 818.549.9959 fax 818.543.3747

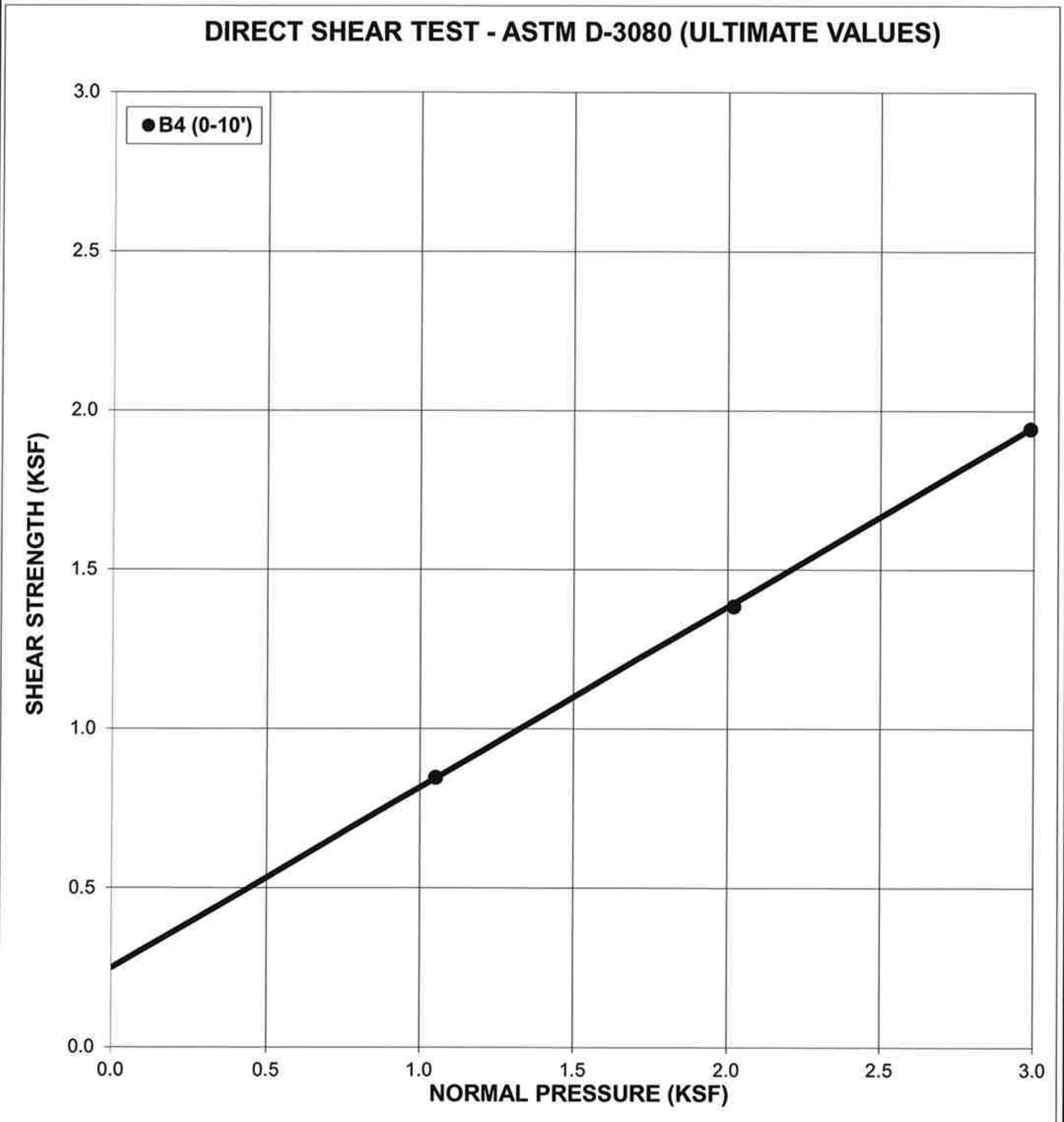
SHEAR TEST DIAGRAM #2

BG: 23084 ENGINEER: RSB
CLIENT: 3700 West Riverside Investments, LLC

EARTH MATERIAL: **Future Compacted Fill (Remolded at 90%)**

Phi Angle =	**29.5**	**degrees**	
Cohesion =	**250**	**psf**	

Moisture Content	**20.3%**
Dry Density (pcf)	**108.0**
Saturation	**99%**



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INC.

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tel 818.549.9959 fax 818.543.3747

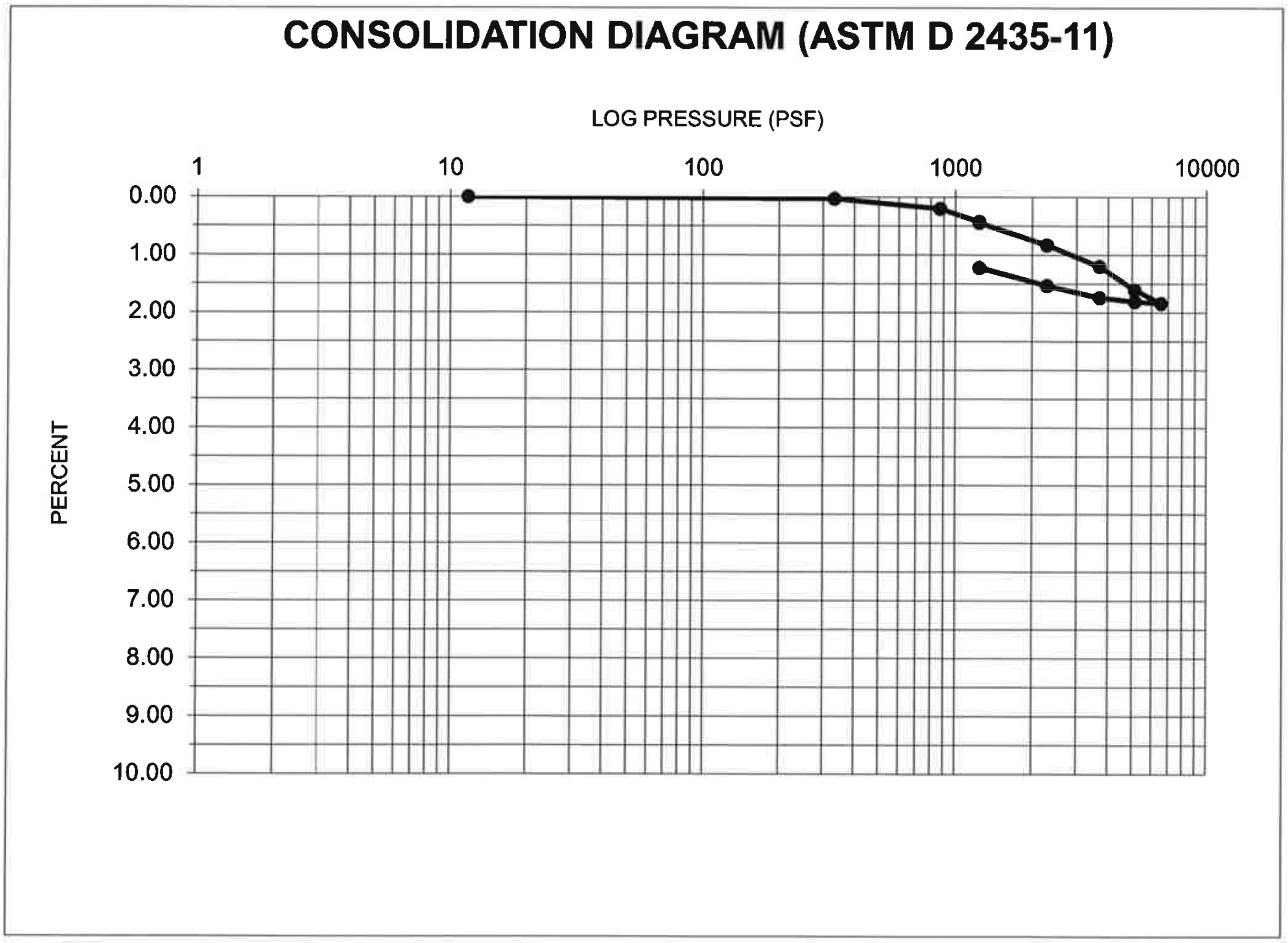
CONSOLIDATION CURVE #1

BG: **23084** ENGINEER: **RSB**

CLIENT: **3700 West Riverside Investments, LLC**

Earth Material:	Alluvium
Sample Location:	B4-12.5'
Dry Weight (pcf):	95.7
Initial Moisture:	1.2%
Initial Saturation:	4.4%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.73
Compression Index (Cc):	0.050
Recompression Index (Cr):	0.020



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1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #2

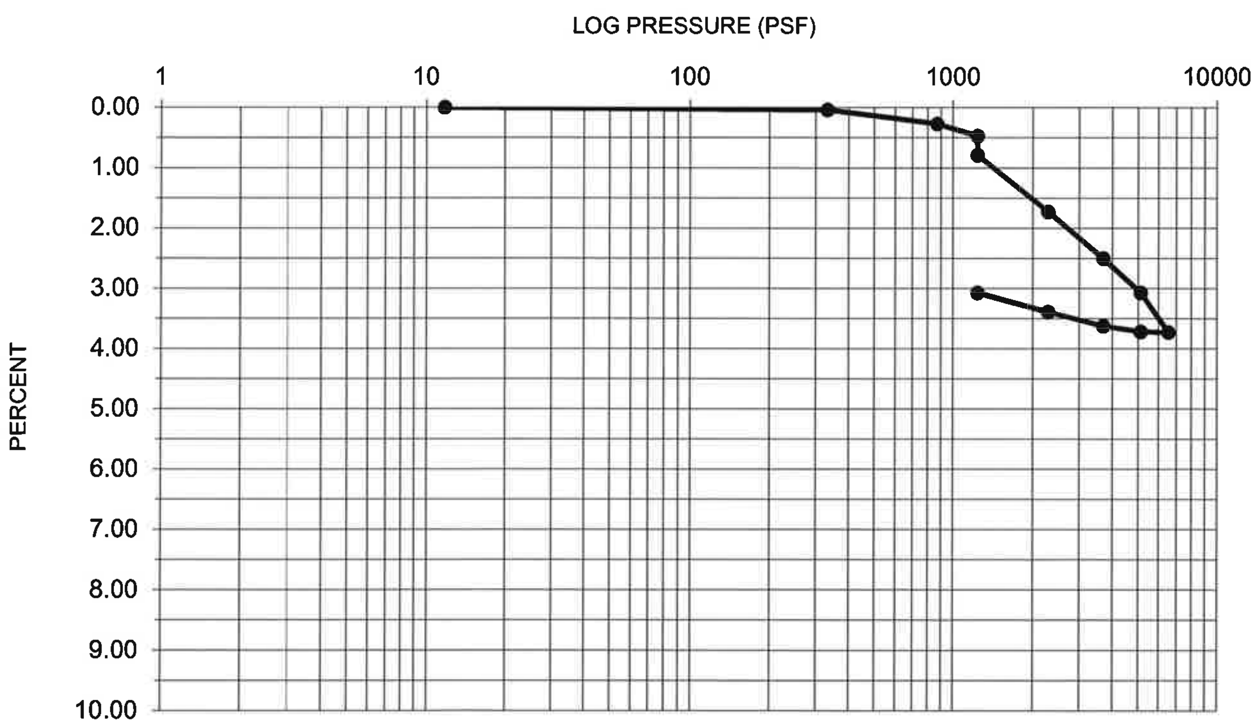
BG: **23084** ENGINEER: **RSB**

CLIENT: **3700 West Riverside Investments, LLC**

Earth Material:	Alluvium
Sample Location:	B2-15'
Dry Weight (pcf):	115.6
Initial Moisture:	3.9%
Initial Saturation:	24.0%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.43
Compression Index (Cc):	0.089
Recompression Index (Cr):	0.017

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #3

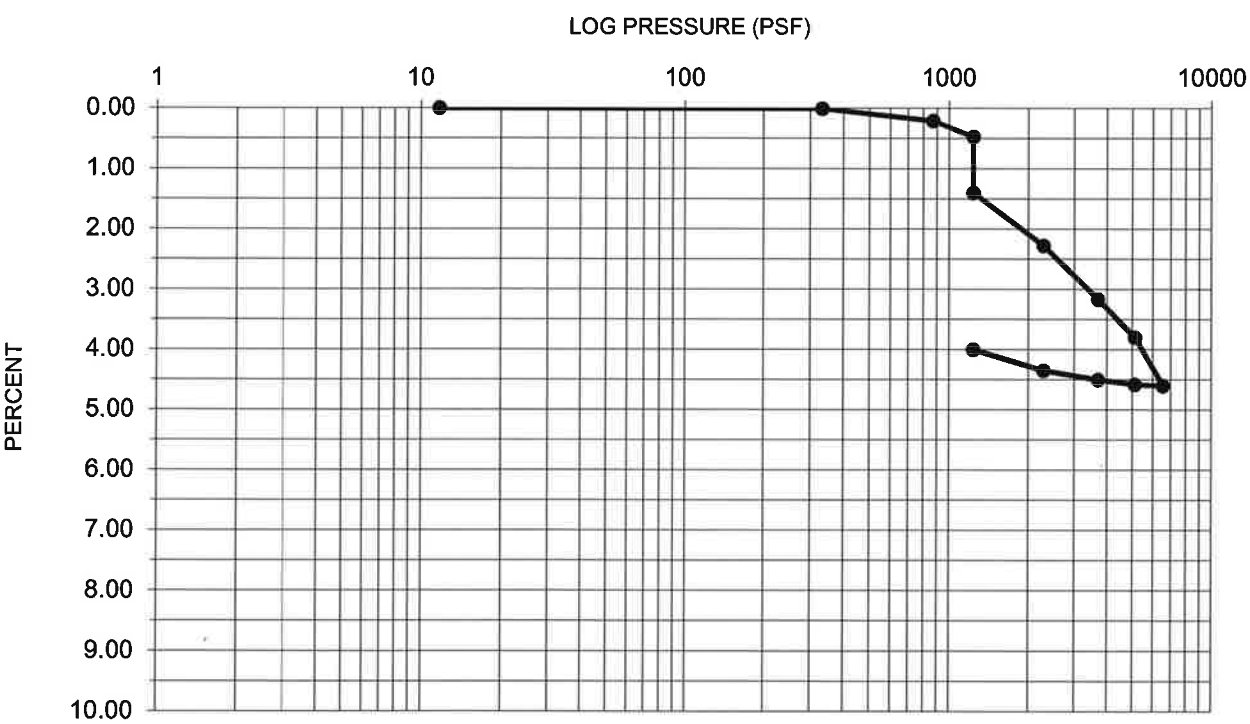
BG: 23084 ENGINEER: RSB

CLIENT: 3700 West Riverside Investments, LLC

Earth Material:	Alluvium
Sample Location:	B4-20'
Dry Weight (pcf):	92.1
Initial Moisture:	18.5%
Initial Saturation:	61.6%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.80
Compression Index (Cc):	0.135
Recompression Index (Cr):	0.024

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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GEOTECHNICAL
INC.

1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #4

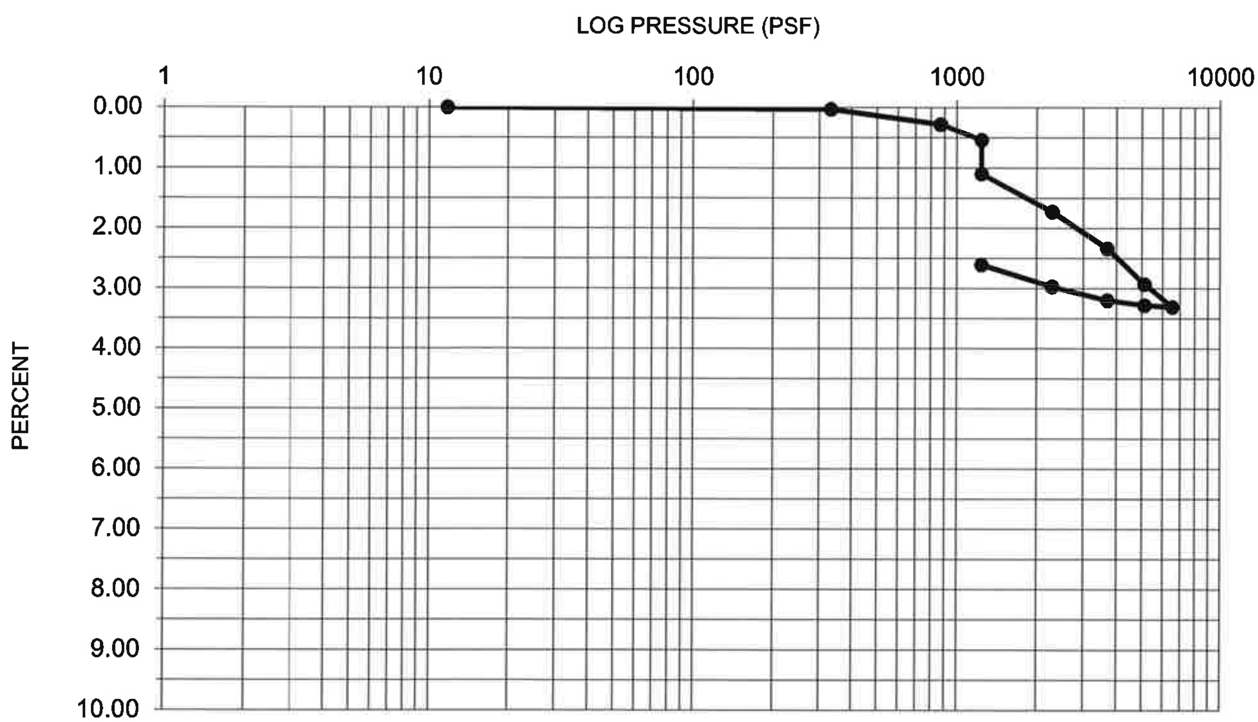
BG: **23084** ENGINEER: **RSB**

CLIENT: **3700 West Riverside Investments, LLC**

Earth Material:	Alluvium
Sample Location:	B2-25'
Dry Weight (pcf):	109.3
Initial Moisture:	4.4%
Initial Saturation:	22.7%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.51
Compression Index (Cc):	0.063
Recompression Index (Cr):	0.020

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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GEOTECHNICAL
INC.

1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #5

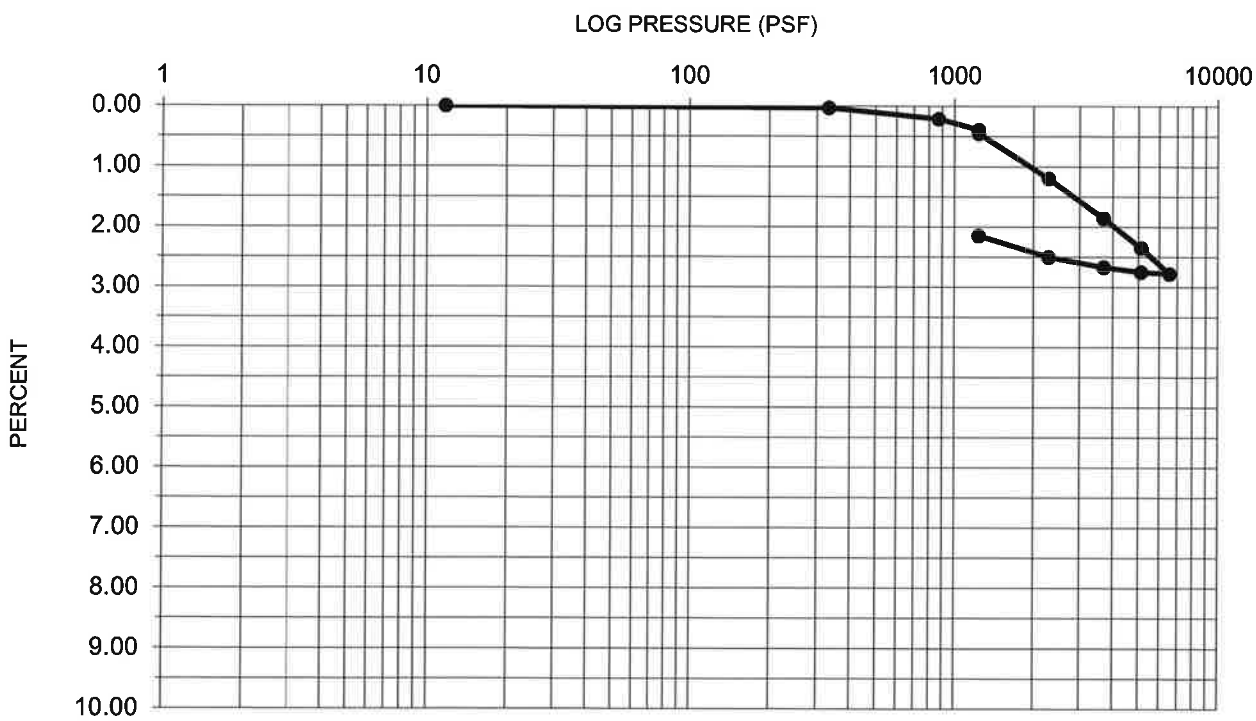
BG: **23084** ENGINEER: **RSB**

CLIENT: **3700 West Riverside Investments, LLC**

Earth Material:	Alluvium
Sample Location:	B4-30'
Dry Weight (pcf):	107.3
Initial Moisture:	16.3%
Initial Saturation:	79.8%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.54
Compression Index (Cc):	0.062
Recompression Index (Cr):	0.020

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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tel 818.549.9959 fax 818.543.3747

CONSOLIDATION CURVE #6

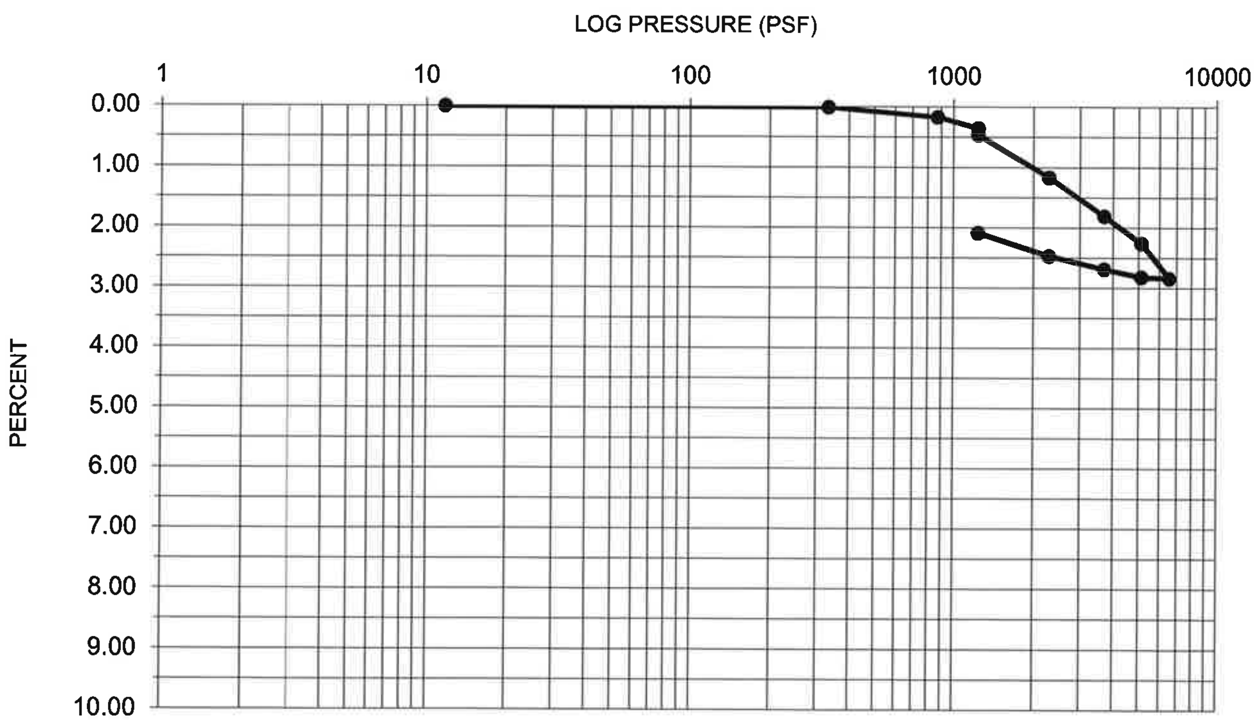
BG: 23084 ENGINEER: RSB

CLIENT: 3700 West Riverside Investments, LLC

Earth Material:	Alluvium
Sample Location:	B2-35'
Dry Weight (pcf):	117.3
Initial Moisture:	16.4%
Initial Saturation:	97.5%
Water Added at (psf)	1237

Specific Gravity:	2.75
Initial Void Ratio:	0.46
Compression Index (Cc):	0.080
Recompression Index (Cr):	0.021

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #7

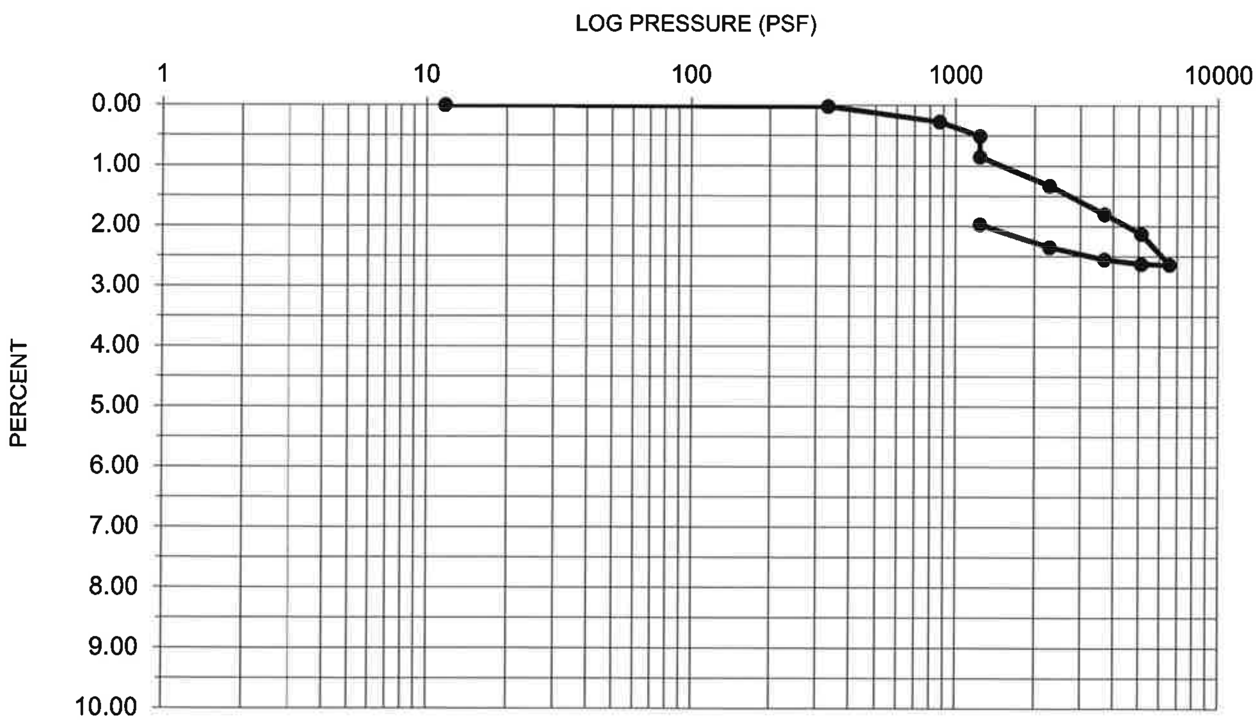
BG: **23084** ENGINEER: **RSB**

CLIENT: **3700 West Riverside Investments, LLC**

Earth Material:	Alluvium
Sample Location:	B2-40'
Dry Weight (pcf):	104.0
Initial Moisture:	12.8%
Initial Saturation:	54.1%
Water Added at (psf)	1237

Specific Gravity:	2.75
Initial Void Ratio:	0.65
Compression Index (Cc):	0.079
Recompression Index (Cr):	0.024

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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CONSOLIDATION CURVE #8

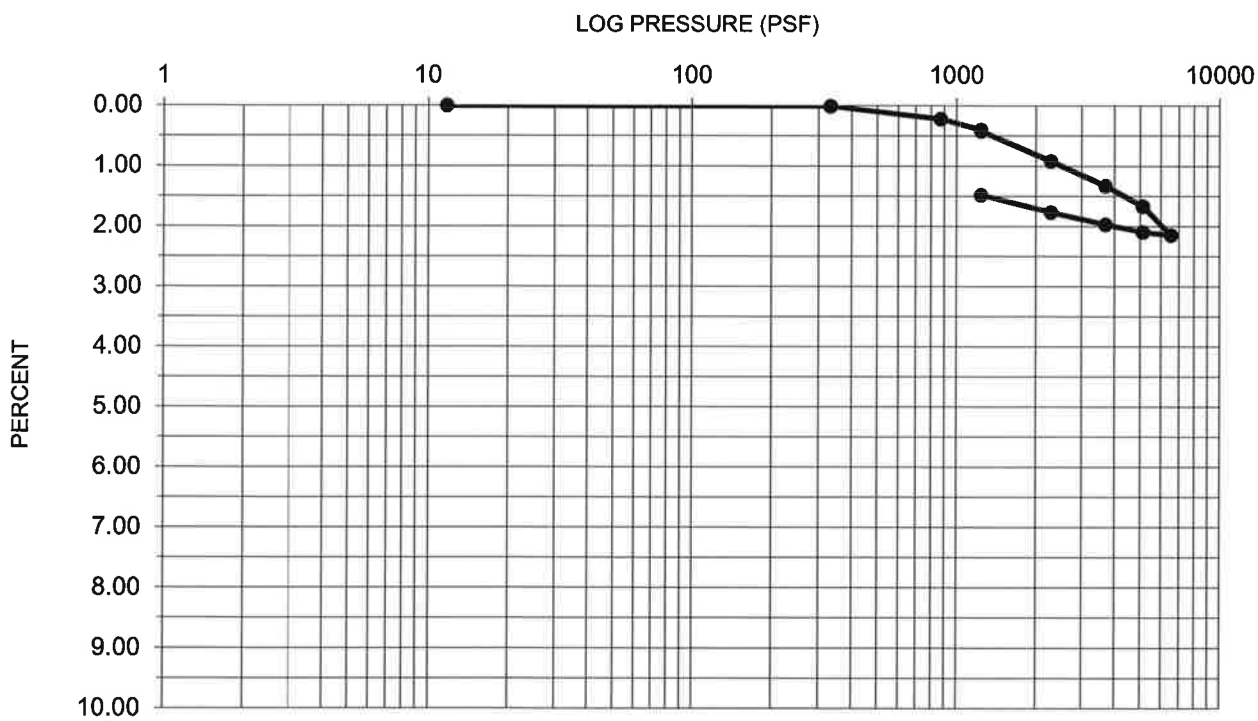
BG: 23084 ENGINEER: RSB

CLIENT: 3700 West Riverside Investments, LLC

Earth Material:	Alluvium
Sample Location:	B4-45'
Dry Weight (pcf):	131.7
Initial Moisture:	1.7%
Initial Saturation:	15.4%
Water Added at (psf)	1237

Specific Gravity:	2.75
Initial Void Ratio:	0.30
Compression Index (Cc):	0.059
Recompression Index (Cr):	0.014

CONSOLIDATION DIAGRAM (ASTM D 2435-11)



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PLASTICITY CHART #1

BG: 23084 ENGINEER: RSB

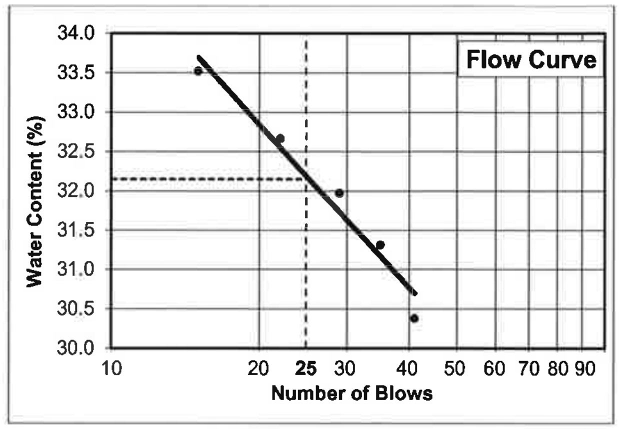
CLIENT: 3700 West Riverside Investments, LLC

Test Pit No.: B1 Sample No.: S9 Depth of Sample: 22.5 Feet Test Date: 8/20/2019

Soil Description: Clay (CL)

Liquid Limit Determination

Can No.	A	B	C	D	E
Soil Wet Wt. + Can (g)	30.83	30.06	30.15	30.49	29.71
Soil Dry Wt. + Can (g)	27.86	27.28	27.41	27.75	27.31
Wt. of Can (g)	19.00	18.77	18.84	19.00	19.41
Wt. of Dry Soil (g)	8.86	8.51	8.57	8.75	7.90
Wt. of Moisture (g)	2.97	2.78	2.74	2.74	2.40
Water Content (%)	**33.5**	**32.7**	**32.0**	**31.3**	**30.4**
Number of Blows	**15**	**22**	**29**	**35**	**41**

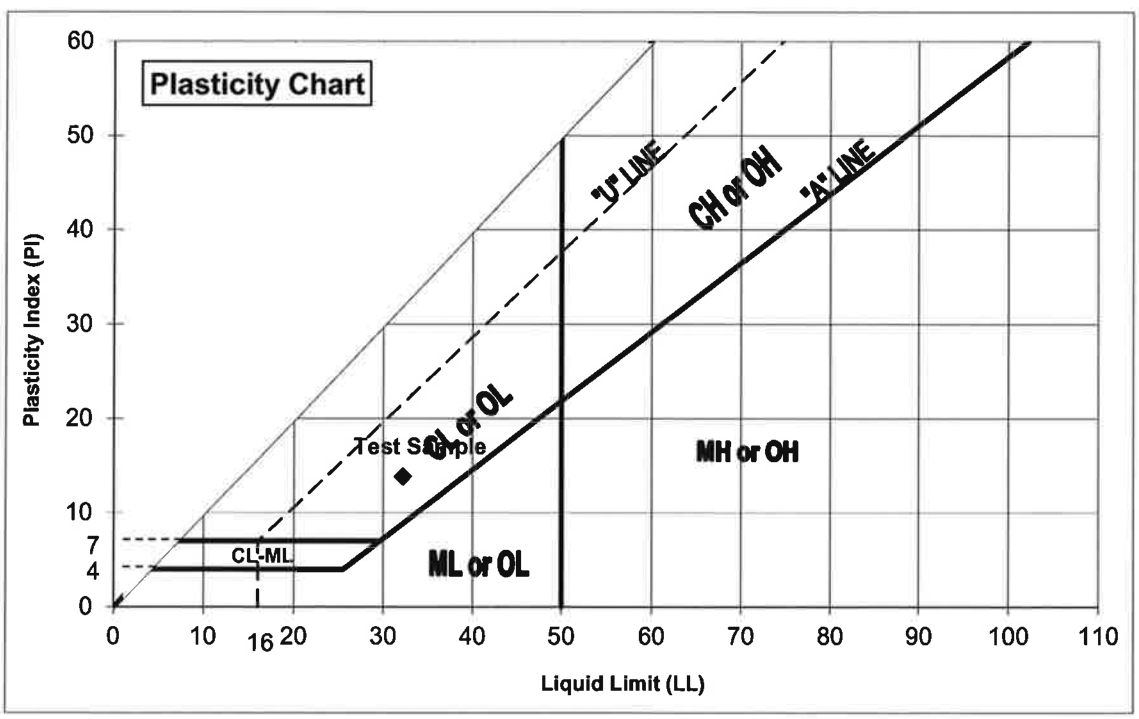


Plastic Limit Determination

Can No.	F				
Soil Wet Wt. + Can (g)	25.91				
Soil Dry Wt. + Can (g)	24.80				
Wt. of Can (g)	18.74				
Wt. of Dry Soil (g)	6.06				
Wt. of Moisture (g)	1.11				
Water Content (%)	**18.3**				

Average Water Content (%) = **18.3**

Liquid Limit, LL = 32.2
Plastic Limit, PL = 18.3
Plasticity Index, PI = 13.9



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818.543.3747 FAX

LOG OF BORING B1

BG No. 23084

PAGE 1 OF 3

CLIENT 3700 West Riverside Investments, LLC **REPORT DATE** 9/25/19 **DRILL DATE** 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 7.5" asphalt, no base.								
		(SM) **ALLUVIUM (Qa):** 0.6' - 2.5': Silty SAND, olive-brown, slightly moist, fine sand.		SM						
550		(SM) 2.5': Silty SAND, light olive-brown, slightly moist, very loose, fine sand.		SM	S1	1 1 2	8.9			
	5	(SM) 5': Top 6": Silty SAND, olive-brown, moist, very loose, fine sand. (SP) Bottom 12": SAND, light olive-brown, slightly moist, very loose, fine sand, trace medium sand.		SM SP	S2	1 1 1	26.9			
545		(SP) 7.5': SAND, light olive-brown, slightly moist, loose, fine sand, trace medium sand, some silt pockets.		SP	S3	1 2 4	15.6			
	10	(SP) 10': SAND, light olive-brown, slightly moist, loose, fine sand, trace medium sand.		SP	S4	2 2 3	8.4			
540		(SP) 12.5': SAND, light olive-gray, slightly moist, medium dense, fine sand, trace medium to coarse sand.		SP	S5	2 5 6	2.8			
	15	(SP) 15': SAND, light olive-gray, slightly moist, medium dense, fine sand, trace medium sand.		SP	S6	3 5 7	2.8			
535		(SM) 17.5': Silty SAND, light olive-gray, slightly moist, medium dense, fine sand, some medium sand, trace fine gravel to 1/2" subangular, 24.6% fines.		SM	S7	4 4 8	6.8			Sieve Wash (-#200)
	20	(ML) 20': Sandy SILT, olive-brown, moist, stiff, fine sand, 54.1% fines.		ML	S8	3 5 6	16			Sieve Wash (-#200)
530		(CL) 22.5': Sandy CLAY, olive-brown, moist, stiff to very stiff, fine sand, 61.2% fines.		CL	S9	5 7 7	12.6			Atterberg Limits, Sieve Wash (-#200)
	25									

Standard Penetration Test

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LOG OF BORING B1

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC REPORT DATE 9/25/19 DRILL DATE 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA LOGGED BY RSB

CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(SM) 25': Silty SAND, olive-gray, slightly moist, medium dense, fine sand, 19.1% fines.		SM	S10	5 8 8	4.8			Sieve Wash (-#200)
525		(SM) 27.5': Silty SAND, dark olive-brown, moist, medium dense, fine sand, 50.4% fines.		SM	S11	4 8 7	19.2			Sieve Wash (-#200)
	30	(SP-SM) 30': SAND with silt, olive-gray, slightly moist, medium dense, fine sand, some medium sand, trace fine gravel to 3/4" subangular, 12.9% fines.		SP-SM	S12	5 12 14	4.1			Sieve Wash (-#200)
520		(SM) 32.5': Silty SAND, olive-gray, slightly moist, medium dense, fine sand, 49.5% fines.		SM	S13	4 9 10	12.3			Sieve Wash (-#200)
	35	(SM) 35': Silty SAND, dark olive-brown, moist, medium dense, fine sand, 48.3% fines.		SM	S14	4 5 7	16.7			Sieve Wash (-#200)
515		(SM) 37.5': Silty SAND, dark olive-brown, moist, medium dense, fine sand, trace medium sand, 28.7% fines.		SM	S15	8 11 11	10.3			Sieve Wash (-#200)
	40	(SP-SM) 40': SAND with silt, olive-gray, slightly moist, medium dense, fine sand, 13.9% fines.		SP-SM	S16	5 11 13	4.5			Sieve Wash (-#200)
510		(SM) 42.5': Silty SAND, dark olive-brown, moist, medium dense, fine sand, some medium to coarse sand, trace fine gravel to 3/4" angular, 27.1% fines.		SM	S17	3 6 11	12			Sieve Wash (-#200)
	45	(SP) 45': Gravelly SAND, olive-gray, slightly moist, dense, fine to medium sand, some coarse sand, fine to coarse gravel to 1.5" subangular.		SP	S18	9 21 25	2.6			
505		(SP) 47.5': Gravelly SAND, light olive-brown, slightly moist, dense, fine to medium sand, some coarse sand, fine to coarse gravel to 1.5" subangular.		SP	S19	16 19 21	2.4			
	50									

Standard Penetration Test

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
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818.543.3747 FAX

LOG OF BORING B1

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC REPORT DATE 9/25/19 DRILL DATE 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA LOGGED BY RSB

CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50	(SP) 50': Gravelly SAND, olive-brown, slightly moist, dense, fine to medium sand, some coarse sand, fine to coarse gravel to 2" subangular.		SP	S20	11 18 12	3.6			
500		(SP) 52.5': Gravelly SAND, olive-brown, slightly moist, very dense, fine to medium sand, some coarse sand, fine to coarse gravel to 2" subangular.		SP	S21	50	1.7			
	55	(SP) 55': Gravelly SAND, olive-brown, slightly moist to moist, medium dense, fine to medium sand, some coarse sand, fine to coarse gravel to 1.5" subangular, some fines.		SP	S22	17 18 9	18.5			
495		(ML) 57.5': Sandy SILT, olive-brown, moist, medium stiff to stiff, fine sand, some medium sand, trace fine gravel, 64.6% fines.		ML	S23	6 3 5	29.2			Sieve Wash (-#200)
	60	(SC) 60': SAND, olive-brown, slightly moist, very dense, fine sand, some medium sand.		SC	S24	9 10 41	4.4			

End at 61.5 Feet; No Groundwater; No Fill.

Standard Penetration Test

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GLENDALE, CA 91206
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818.543.3747 FAX

LOG OF BORING B2

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC REPORT DATE 9/25/19 DRILL DATE 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA LOGGED BY RSB

CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 552 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 6" asphalt, no base.								
550		(SM) **ALLUVIUM (Qa):** 0.5' - 2.5': Silty SAND, olive-brown, moist, fine sand.		SM						
		(SP) 2.5': SAND, light olive-gray, slightly moist, loose, fine sand, trace fines.		SP	R1	4 5 5	5.2	97.1	19.5	Direct Shear
	5	(SP) 5': SAND, light olive-gray, slightly moist, loose, fine sand, trace fines.		SP	R2	2 3 4	4.5	88.4	13.8	Direct Shear
545		(SP) 7.5': SAND, light olive-gray, slightly moist, loose, fine sand.		SP	R3	3 6 6	3.9	97.2	14.7	Direct Shear
	10	(SP) 10': SAND, olive-gray, slightly moist, medium dense, fine sand, trace medium sand.		SP	R4	3 8 10	7.3	101.2	30.5	Direct Shear
540		(SP) 12.5': SAND, olive-gray, slightly moist, medium dense, fine sand, trace medium sand.		SP	R5	6 10 14	4.5	104.1	20.1	Direct Shear
	15	(SP) 15': SAND, olive-gray, slightly moist, medium dense, fine sand, trace medium sand.		SP	R6	6 9 12	3.9	115.5	24.3	Consolidation
535										
	20	(ML) 20': Sandy SILT, olive-brown, moist, stiff, fine sand.		ML	R7	4 6 15	35.5	85.7	100	
530										
	25									

■ Ring Sample

BYER GEOTECHNICAL, INC.

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LOG OF BORING B2

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC **REPORT DATE** 9/25/19 **DRILL DATE** 7/17/19
PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA **LOGGED BY** RSB
CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 552 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(SM) 25': Silty SAND, olive-gray, slightly moist, medium dense, fine sand.		SM	R8	14 25 36	4.4	109.3	22.7	Consolidation
525										
	30	(SP-SM) 30': SAND with silt, olive-gray, slightly moist, dense, fine sand, some medium to coarse sand, some fine to coarse gravel to 1.5" subangular.		SP-SM	R9	12 31 43	2.4	118.3	16.3	
520										
	35	(SM) 35': Silty SAND, dark olive-brown, moist, medium dense, fine sand.		SM	R10	7 13 16	16.4	117.3	100	Consolidation
515										
	40	(SP-SM) 40': SAND with silt, olive-gray, slightly moist, medium dense, fine sand.		SP-SM	R11	6 15 26	12.8	104	57.4	Consolidation

End at 41.5 Feet; No Groundwater; No Fill.

■ Ring Sample

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LOG OF BORING B3

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC REPORT DATE 9/25/19 DRILL DATE 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA LOGGED BY RSB

CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 3.5" asphalt over 5" base.								
		(SM) **FILL (Afu):** 0.7' - 1.5': Silty SAND, olive-brown, moist, concrete debris.		SM						
550		(SM) **ALLUVIUM (Qa):** 1.5'-2.5': Silty SAND, olive-brown, moist, fine sand.		SM						
	5	(SP) 5': SAND, light olive-brown, slightly moist, very loose, fine sand, some fines.		SP	S1	1 1 1	12.1			
545	10	(SP) 10': SAND, light gray, slightly moist, loose, fine sand.		SP	S2	1 2 3	1.9			
540	15	(SM) 15': Silty SAND, light olive-brown, slightly moist, medium dense, fine sand.		SM	S3	4 7 10	7.5			
535	20	(ML) 20': Sandy SILT, light olive-brown, slightly moist, very stiff, fine sand.		ML	S4	5 8 10	3.9			
530	25									

Standard Penetration Test

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LOG OF BORING B3

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC REPORT DATE 9/25/19 DRILL DATE 7/17/19
PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA LOGGED BY RSB
CONTRACTOR Martini Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(SM) 25': Silty SAND, olive-brown, slightly moist, medium dense, fine sand.		SM	S5	3 5 6	17.9			
525										
	30	(SP-SM) 30': SAND with silt, light olive-gray, slightly moist, medium dense, fine sand, trace medium sand.		SP-SM	S6	7 9 8	2			
520										
	35	(SM) 35': Silty SAND, dark olive-brown, moist, medium dense, fine sand, trace medium sand.		SM	S7	3 5 7	16.5			

End at 36.5 Feet; No Groundwater; Fill to 1.5 Feet.

Standard Penetration Test

BYER GEOTECHNICAL, INC.

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GLENDALE, CA 91206
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818.543.3747 FAX

LOG OF BORING B4

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC **REPORT DATE** 9/25/19 **DRILL DATE** 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 6.5" asphalt, no base.								
		(SM) **ALLUVIUM (Qa):** 0.5' - 2.5': Silty SAND, olive-brown, slightly moist, fine sand.		SM						
550		(SM) 2.5': Silty SAND, olive-brown, slightly moist, loose, fine sand.		SM	R1	2 2 3	5.2	95	18.5	
	5	(SM) 5': Silty SAND, olive-brown, slightly moist, loose, fine sand.		SM	Bag1 R2	1 2 3	12.8	84.9	35.8	Max, EI, Remolded Shear, Corrosion Suite
545		(SP) 7.5': SAND, light olive-gray, slightly moist, loose, fine sand.		SP	R3	3 3 4	5.6	87.3	16.5	
	10	(SP) 10': SAND, light olive-gray, slightly moist, loose, fine sand.		SP	R4	4 6 6	2.4	99.9	9.6	
540		(SP) 12.5': SAND, light gray, slightly moist, loose, fine sand, trace medium sand.		SP	R5	5 5 7	1.2	95.7	4.4	
	15	(ML) 15': Sandy SILT, olive-brown, moist, stiff, fine sand.		ML	R6	2 5 13	1.5	120.5	10.4	Direct Shear
535										
	20	(ML) 20': Sandy SILT, olive-brown, slightly moist, very stiff, fine sand.		ML	R7	7 16 19	18.5	92	61.7	
530		(SP) 22.5': SAND, light olive-gray, slightly moist, medium dense, fine sand, some fines.		SP	R8	10 15 18	3.4	104.8	15.5	
	25									

Bulk Sample   Ring Sample

BYER GEOTECHNICAL, INC.

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LOG OF BORING B4

BG No. 23084

CLIENT 3700 West Riverside Investments, LLC **REPORT DATE** 9/25/19 **DRILL DATE** 7/17/19

PROJECT LOCATION 3700 West Riverside Drive, Burbank, CA **LOGGED BY** RSB

CONTRACTOR Martini Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 553 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(SM) 25': Silty SAND, light olive-gray, slightly moist, medium dense, fine sand.		SM	R9	9 13 18	2.9	95.6	10.7	
525		(SM) 27.5': Silty SAND, olive-brown, moist, dense, fine sand.		SM	R10	11 20 37	7.5	102.6	32.5	
	30	(SP-SM) 30': SAND with silt, olive-brown, moist, dense, fine sand, trace medium sand.		SP-SM	R11	6 16 37	16.3	107.3	79.9	
520		(SM) 32.5': Silty SAND, olive-brown, moist, medium dense, fine sand, some medium sand, trace fine to coarse gravel to 1.5" subangular.		SM	R12	7 8 14	26.7	90.2	84.9	
	35	(CL) 35': Sandy CLAY, dark olive-brown, moist, stiff, fine sand, some medium sand, trace fine to coarse gravel to 1.5" subangular.		CL	R13	6 7 11	18.1	116.9	100	
515										
	40	(SP) 40': SAND with silt, dark olive-brown, slightly moist to moist, dense, fine sand, trace medium to coarse sand, trace fine to coarse gravel to 2" subangular.		SP	R14	13 22 37	7.3	117.6	47.8	
510										
	45	(SP) 45': Gravelly SAND, light yellowish-brown, slightly moist, very dense, fine to medium sand, some coarse sand, fine to coarse gravel to 1.5" subangular.		SP	R15	23 50	1.7	131.7	17.9	

End at 46 Feet; No Groundwater; No Fill.

Bulk Sample   Ring Sample

APPENDIX II

Calculations and Figures

SEISMIC SOURCES
EZ-FRISK V7.65

DETERMINISTIC CALCULATION
OF PEAK GROUND ACCELERATION BASED ON DIGITIZED FAULT DATA

BG: 23084
CLIENT: 3700 West Riverside Investments, LLC
ENGINEER: RSB
PROJECT DESCRIPTION: Proposed Multi-Story Building over Subterranean Parking

SITE COORDINATES: LATITUDE: 34.1525
LONGITUDE: -118.3402

SEARCH RADIUS: 100 km

ATTENUATION RELATIONS:
CHIOU-YOUNGS (2007) NGA USGS 2008 MRC
BOORE-ATKINSON (2008) NGA USGS 2008 MRC
CAMPBELL-BOZORGNIA (2008) NGA USGS 2008 MRC

SEISMIC SOURCE SUMMARY
DETERMINISTIC SITE PARAMETERS

FAULT NAME	APPROXIMATE DISTANCE (km)	APPROXIMATE DISTANCE (mi)	MAXIMUM EATHQUAKE MAGNITUDE (Mw)	PEAK GROUND ACCELERATION (g)
Hollywood	4.8	3.0	6.7	0.633
Santa Monica	4.9	3.1	7.4	0.804
Verdugo	6.3	3.9	6.9	0.447
Elysian Park (Upper)	6.7	4.2	6.7	0.481
Puente Hills (LA)	10.4	6.4	7.0	0.451
Raymond	11.3	7.0	6.8	0.341
Puente Hills	11.9	7.4	7.1	0.431
Sierra Madre	12.9	8.0	7.2	0.347
Sierra Madre Connected	12.9	8.0	7.3	0.355
Newport-Inglewood	13.0	8.1	7.5	0.361
Sierra Madre (San Fernando)	13.7	8.5	6.7	0.295
Northridge	17.8	11.1	6.9	0.373
San Gabriel	19.2	11.9	7.3	0.279
Malibu Coast	21.5	13.3	7.0	0.251
Puente Hills (Santa Fe Springs)	21.8	13.6	6.7	0.284
Santa Susana, alt 1	23.8	14.8	6.9	0.221

FAULT NAME	APPROXIMATE DISTANCE		MAXIMUM EATHQUAKE MAGNITUDE	PEAK GROUND ACCELERATION
	(km)	(mi)	(Mw)	(g)
Anacapa-Dume	24.0	14.9	7.2	0.268
Palos Verdes	28.5	17.7	7.3	0.219
Palos Verdes Connected	28.5	17.7	7.7	0.251
Clamshell-Sawpit	29.9	18.6	6.7	0.181
Elsinore	32.1	20.0	7.9	0.246
Holser, alt 1	33.5	20.8	6.8	0.176
Puente Hills (Coyote Hills)	36.1	22.5	6.9	0.188
Simi-Santa Rosa	36.8	22.9	6.9	0.159
Oak Ridge Connected	42.2	26.2	7.4	0.194
San Jose	43.4	26.9	6.7	0.130
Oak Ridge (Onshore)	43.5	27.0	7.2	0.182
Southern San Andreas	49.0	30.5	8.2	0.218
San Cayetano	50.0	31.1	7.2	0.145
Chino	51.5	32.0	6.8	0.114
Cucamonga	53.9	33.5	6.7	0.107
San Joaquin Hills	63.3	39.3	7.1	0.125
Imp Extensional Gridded, Char, Normal	49.8	31.0	7.0	0.116
Imp Extensional Gridded, Char, Strike Slip	49.8	31.0	7.0	0.139
Imp Extensional Gridded, GR, Normal	49.8	31.0	7.0	0.115
Imp Extensional Gridded, GR, Strike Slip	49.8	31.0	7.0	0.221
Santa Ynez (East)	67.7	42.1	7.2	0.111
Santa Ynez Connected	67.9	42.2	7.4	0.122
San Jacinto	73.0	45.4	7.9	0.144
Ventura-Pitas Point	76.6	47.6	7.0	0.098
Pitas Point Connected	76.6	47.6	7.3	0.115
Mission Ridge-Arroyo Parida-Santa Ana	81.7	50.8	6.9	0.079

42 Faults found within a 100 km Search Radius.
Closest Fault to the Site: Hollywood Distance = 4.75 km (2.95mi)
Largest Peak Ground Acceleration: 0.804 g
The San Andreas Fault is Located Aproximately 49 km (30.5 mi) from the Site.

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SEISMIC HAZARD DEAGGREGATION CHART #1
(Probability of Exceedance: 10% in 50 years)

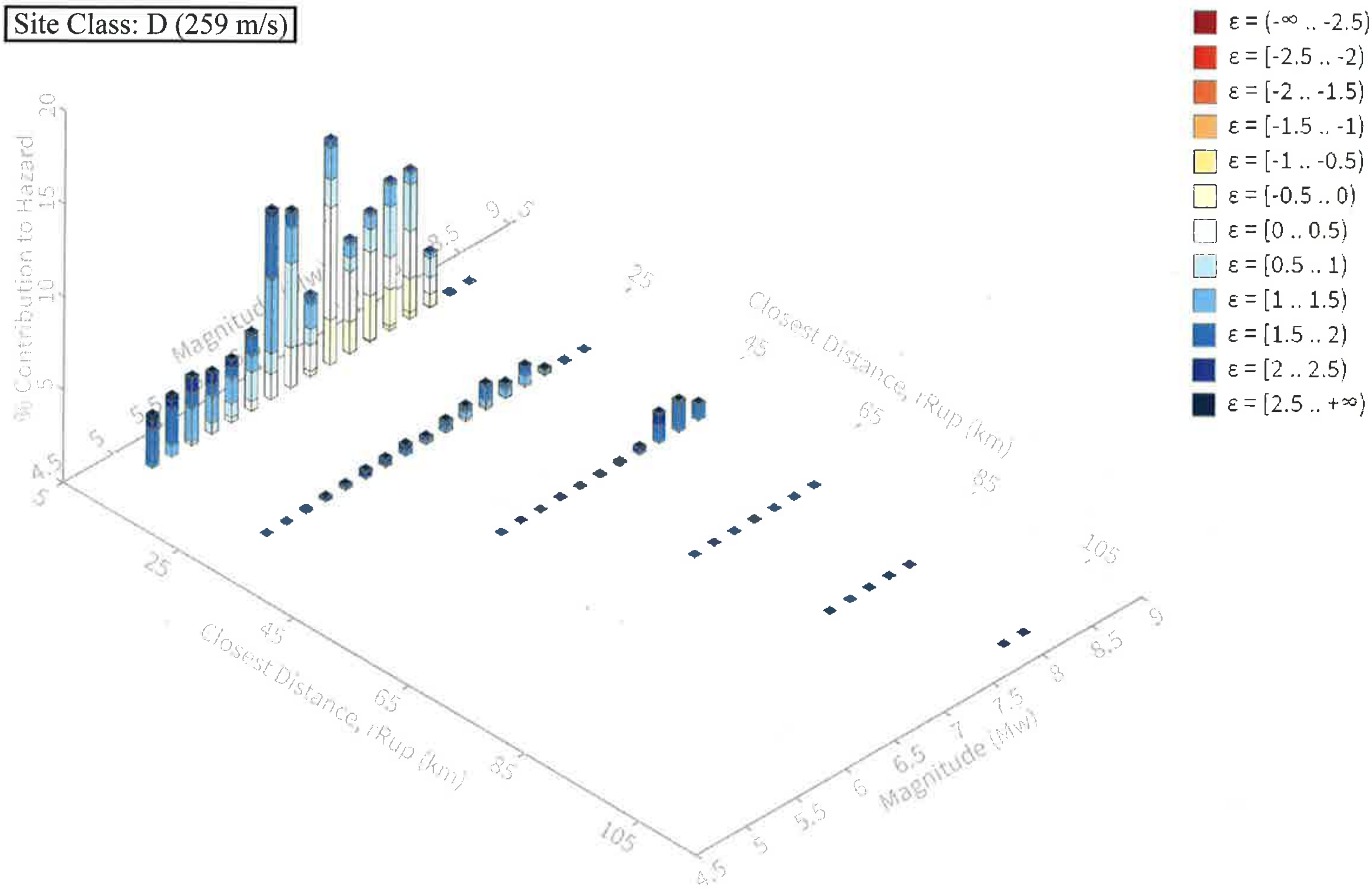
BG: 23084

CLIENT: 3700 WEST RIVERSIDE INVESTMENTS, LLC

ENGINEER: RSB

REFERENCE: USGS, 2019, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2014 (v4.2.0) Edition, https://earthquake.usgs.gov/hazards/interactive/index.php.

Site Class: D (259 m/s)



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.5262084 g

Recovered targets

Return period: 509.20038 yrs
Exceedance rate: 0.0019638634 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.12 %

Mode (largest m-r bin)

m: 6.9
r: 8.33 km
ε_0: 0.41 σ
Contribution: 12.05 %

Mode (largest m-r-ε_0 bin)

m: 6.9
r: 6.73 km
ε_0: 0.22 σ
Contribution: 6.07 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

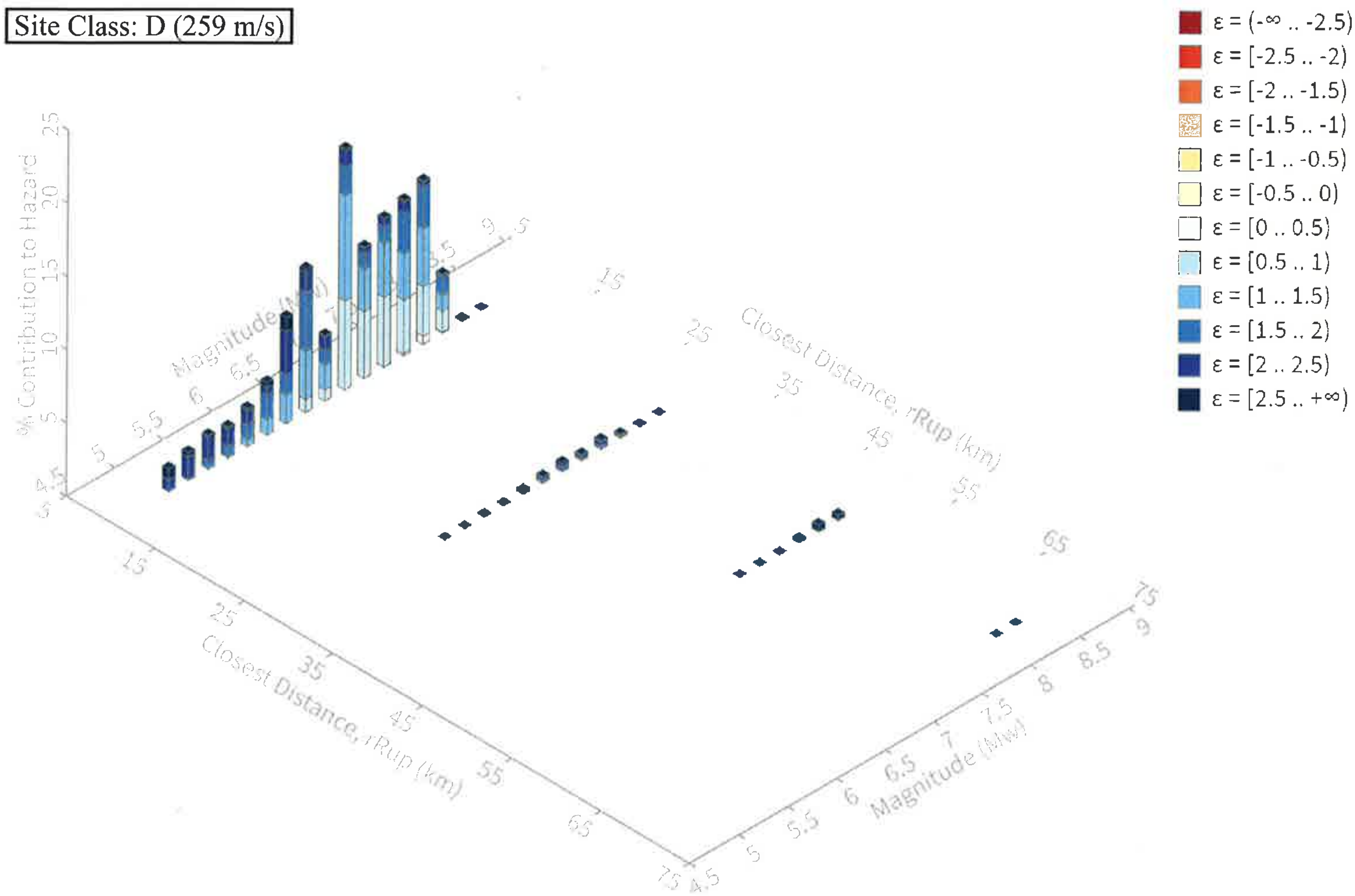
SEISMIC HAZARD DEAGGREGATION CHART #2
(Probability of Exceedance: 2% in 50 years)

BG: 23084

ENGINEER: RSB

CLIENT: 3700 WEST RIVERSIDE INVESTMENTS, LLC

REFERENCE: USGS, 2019, Earthquake Hazards Program, Beta - Unified Hazard Tool, Seismic Hazard Deaggregation, Conterminous U.S. 2014 (v4.2.0) Edition, https://earthquake.usgs.gov/hazards/interactive/index.php.



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.8895057 g

Recovered targets

Return period: 3021.515 yrs
Exceedance rate: 0.0003309598 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mode (largest m-r bin)

m: 6.9
r: 7.19 km
ε₀: 1.26 σ
Contribution: 16.4 %

Mode (largest m-r-ε₀ bin)

m: 6.9
r: 7.17 km
ε₀: 1.25 σ
Contribution: 7.22 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Site-Specific Ground Motion Analysis (Based on ASCE 7-16 Standard)

BG: 23084 Client: 3700 West Riverside Investments, LLC
Project Description: Proposed Multi-Story Mixed-Use Building Engineer: RSB

Ss (0.2s) =	2.077	Latitude:	34.1525			Periods (seconds):	80% of Sections. 11.4.3 & 11.4.4 of ASCE 7-16	**RESULTS** Design Values ASCE 7-16 (Section 21.4)
S1 (1s) =	0.732	Longitude:	-118.3402		T_o =	0.176		
Fa =	1.00	Site Class:	D		T_s =	0.881		
Fv =	2.50				T_L =	8		
SMs =	2.077		**Fig. 22-18A**	S_{MS}=	**1.554**	<	**1.662**	**1.662**
SM1 =	1.830	C_{RS}:	0.905	S_{M1}=	**1.464**	=	**1.464**	**1.464**
SDs =	1.385		**Fig. 22-19A**	S_{DS}=	**1.036**	<	**1.108**	**1.108**
SD1 =	1.220	C_{R1}:	0.901	S_{D1}=	**0.976**	=	**0.976**	**0.976**

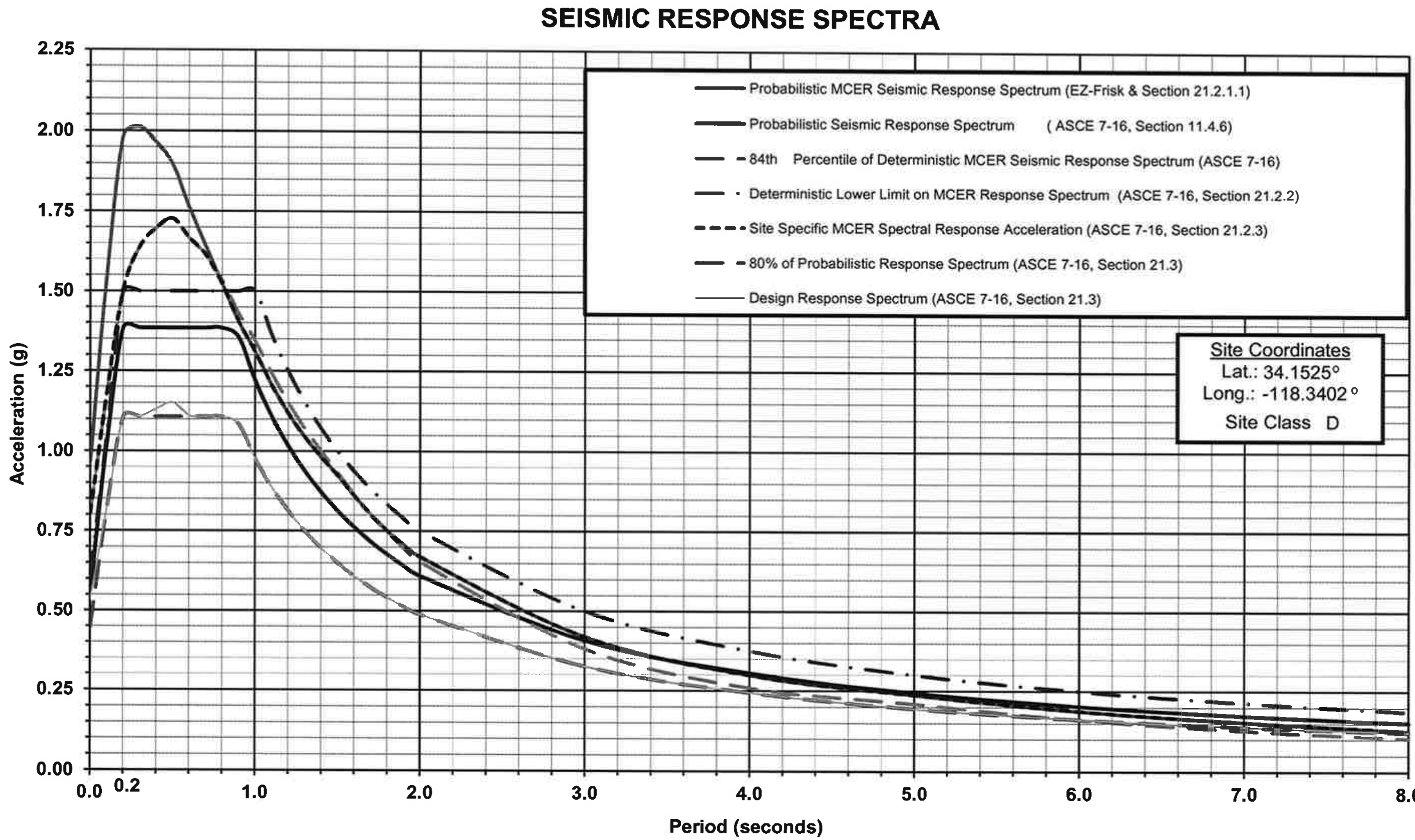
Fundamental Period	Risk Coefficient C_R (Method 1, Section 21.2.1.1, ASCE 7-16)	Probabilistic MCE_R Seismic Response Spectrum (EZ-Frisk & Section 21.2.1.1)	Probabilistic Seismic Response Spectrum (ASCE 7-16, Section 11.4.6)	84th Percentile of Deterministic MCE_R Seismic Response Spectrum (ASCE 7-16)	Deterministic Lower Limit on MCE_R Response Spectrum (ASCE 7-16, Section 21.2.2)	Site Specific MCE_R Spectral Response Acceleration (ASCE 7-16, Section 21.2.3)	80% of Probabilistic Response Spectrum (ASCE 7-16, Section 21.3)	Design Response Spectrum (ASCE 7-16, Section 21.3)
T (sec)		Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
0.0	0.905	0.9674	0.5539	0.8036	0.600	0.804	0.443	0.536
0.1	0.905	1.5476	1.0259	1.1790	1.050	1.179	0.821	0.821
0.2	0.905	1.9865	1.3847	1.5060	1.500	1.506	1.108	1.108
0.3	0.905	2.0125	1.3847	1.6390	1.500	1.639	1.108	1.108
0.4	0.904	1.9644	1.3847	1.6970	1.500	1.697	1.108	1.131
0.5	0.904	1.8964	1.3847	1.7270	1.500	1.727	1.108	1.151
0.6	0.903	1.7627	1.3847	1.6670	1.500	1.667	1.108	1.111
0.7	0.903	1.6416	1.3847	1.6140	1.500	1.614	1.108	1.108
0.8	0.902	1.5226	1.3847	1.5330	1.500	1.523	1.108	1.108
0.9	0.902	1.4045	1.3556	1.4300	1.500	1.405	1.084	1.084
1.0	0.901	1.3101	1.2200	1.3440	1.500	1.310	0.976	0.976
1.1	0.901	1.2064	1.1091	1.2430	1.364	1.206	0.887	0.887
1.2	0.901	1.1190	1.0167	1.1520	1.250	1.119	0.813	0.813
1.3	0.901	1.0452	0.9385	1.0720	1.154	1.045	0.751	0.751
1.4	0.901	0.9812	0.8714	0.9997	1.071	0.981	0.697	0.697
1.5	0.901	0.9244	0.8133	0.9340	1.000	0.924	0.651	0.651
1.6	0.901	0.8609	0.7625	0.8638	0.938	0.861	0.610	0.610
1.7	0.901	0.8019	0.7176	0.8016	0.882	0.802	0.574	0.574
1.8	0.901	0.7513	0.6778	0.7467	0.833	0.751	0.542	0.542
1.9	0.901	0.7076	0.6421	0.6981	0.789	0.708	0.514	0.514
2.0	0.901	0.6697	0.6100	0.6547	0.750	0.670	0.488	0.488
3.0	0.901	0.4183	0.4067	0.3807	0.500	0.418	0.325	0.325
4.0	0.901	0.2984	0.3050	0.2591	0.375	0.298	0.244	0.244
5.0	0.901	0.2376	0.2440	0.2101	0.300	0.238	0.195	0.195
6.0	0.901	0.1888	0.2033	0.1615	0.250	0.189	0.163	0.163
7.0	0.901	0.1540	0.1743	0.1283	0.214	0.154	0.139	0.139
8.0	0.901	0.1268	0.1525	0.1040	0.188	0.127	0.122	0.122

* The Probabilistic and Deterministic Seismic Response Spectra are Based on the Maximum Rotated Component (MRC) of Ground Motion.

References:

- American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standard ASCE/SEI 7-16, Chapter 21.

- Division of the State Architect (DSA), 2009, Use of the Next Generation Attenuation (NGA) Relations, State of California, Department of General Services, DSA Bulletin 09-01, Effective March 1, 2009.



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1461 E. CHEVY CHASE DRIVE, #200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

SITE-SPECIFIC SEISMIC RESPONSE SPECTRA

Proposed Multi-Story Mixed-Use Building

BG: 23084	Client: 3700 West Riverside Investments, LLC
Engineer: RSB	Date: September 18, 2019

P:\23000 - 23999\23084 3700 Riverside Investments\Engineering\23084 Ground Motion Analysis 0.84 Percentile_ASCE 7-16

Liquefaction Susceptibility Analysis: SPT Method (2475-Yr Return)
(Input Data)

Project No.: 23084 Client: 3700 West Riverside Investments, LLC

Project Description.: Proposed Multi-Story Mixed-Use Building

Engineer: RSB

Boring No.	Top Elevation (ft)	Total Depth (ft)	Existing GW Depth (ft)	Design GW Depth (ft)	Recommended Fill Depth (ft)
B1	553	60	100	10	0

Peak Ground Acceleration:	0.967
Earthquake Magnitude:	6.9
Probability of Exceedance in 50 Years:	2%
Borehole Diamter (inches):	8
Delivered Energy Ratio, ERm (%):	75
Energy Ratio Correction Factor, $C_{E:}$	1.25
Borehole Diameter Correction Factor, $C_{B:}$	1.15
Rod Length Correction Factor, $C_{R:}$	1
Sampler Correction with or without Liners, $C_{S:}$	1.2
Minimum Factor of Safety, FS_{liq}:	1.3

References: - *Youd, T. L., et. al. (2001), Liquefaction Resistane of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.*

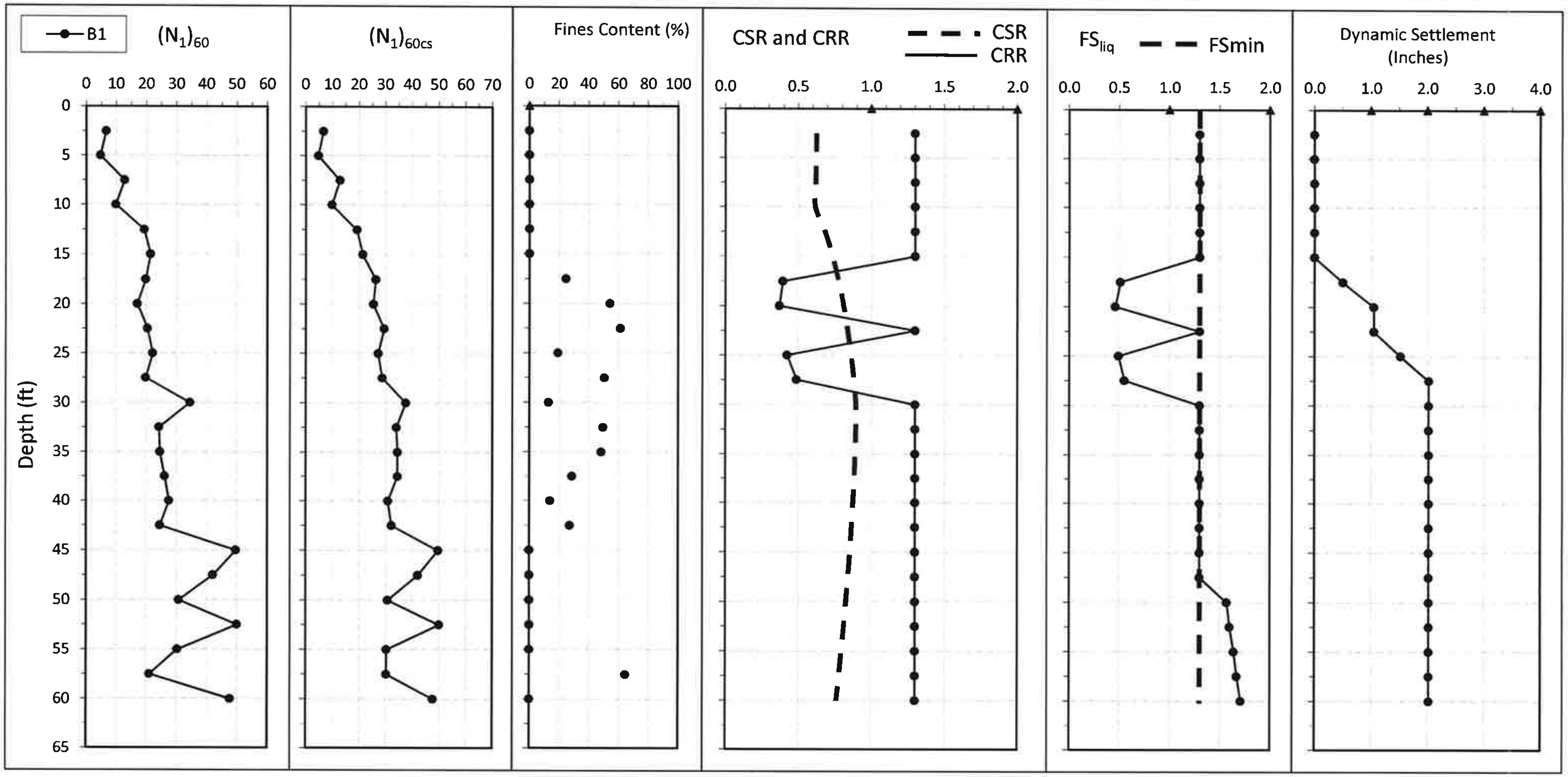
- *Tokimatsu and Seed (1987), Evaluation of Settlements in Sands due to Earthquake Shaking, American Society for Civil Engineers, Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.*

- *California Geological Survey (2008), Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.*

- *County of Los Angeles, Department of Public Works (2009), Liquefaction/Lateral Spreading, Administrative Manual, Publication No. GS 045-0, May 28, 2009.*

Liquefaction Susceptibility Analysis: SPT Method (2475-Yr Return)

Project No.: 23084
Client: 3700 West Riverside Investments, LLC
Project Description.: Proposed Multi-Story Mixed-Use Building
Engineer: RSB



Graphical Representation of the SPT Liquefaction Susceptibility Analysis - Boring B1
(Based on the Methodology of Youd et. al. (2001) and Tokimatsu (1987))

Liquefaction Susceptibility Analysis: SPT Method
(2475-Yr Return)

Project No.: 23084 Client: 3700 West Riverside Investments
Project Description.: Proposed Multi-Story Mixed-Use Building Engineer: RSB

Energy Ratio Correction Factor, C_E = 1.25
Borehole Diameter Correction Factor, C_B = 1.15
Sampler Correction with or without Liners, C_S = 1.2 (With Liners)

Boring No.	SPT Depth (ft)	Elev. (ft)	Approximate Layer Depth (ft)	Approx. Layer Thick. (ft)	Soil Type (USCS)	Screening Analysis & Behavior (Based on Laboratory Testing, Existing Conditions, and Project Configuration)	Fines Content FC (%)	Plasticity Index PI (%)	Liquid Limit LL (%)	Saturated Moisture Content w_c (%)	w_c / LL	Unit Weight γ_t (pcf)	SPT Blow Count Nm (blows/ft)
B1	2.5	550.5	0 to 3.8	3.8	SM	Basement						120	3
B1	5	548.0	3.8 to 6.3	2.5	SM	Basement						120	2
B1	7.5	545.5	6.3 to 8.8	2.5	SP	Basement						120	6
B1	10	543.0	8.8 to 11.3	2.5	SP	Basement						120	5
B1	12.5	540.5	11.3 to 13.8	2.5	SP	Basement						120	11
B1	15	538.0	13.8 to 16.3	2.5	SP	Future Compacted Fill						120	12
B1	17.5	535.5	16.3 to 18.8	2.5	SM		24.6					120	12
B1	20	533.0	18.8 to 21.3	2.5	ML		54.1					120	11
B1	22.5	530.5	21.3 to 23.8	2.5	CL	Wc/LL <= 0.8	61.2	13.9	32.2	12.6	0.39	120	14
B1	25	528.0	23.8 to 26.3	2.5	SM		19.1					120	16
B1	27.5	525.5	26.3 to 28.8	2.5	SM		50.4					120	15
B1	30	523.0	28.8 to 31.3	2.5	SP-SM		12.9					120	26
B1	32.5	520.5	31.3 to 33.8	2.5	SM		49.5					120	19
B1	35	518.0	33.8 to 36.3	2.5	SM		48.3					120	20
B1	37.5	515.5	36.3 to 38.8	2.5	SM		28.7					120	22
B1	40	513.0	38.8 to 41.3	2.5	SP-SM		13.9					120	24
B1	42.5	510.5	41.3 to 43.8	2.5	SM		27.1					120	22
B1	45	508.0	43.8 to 46.3	2.5	SP							120	46
B1	47.5	505.5	46.3 to 48.8	2.5	SP							120	40
B1	50	503.0	48.8 to 51.3	2.5	SP							120	30
B1	52.5	500.5	51.3 to 53.8	2.5	SP							120	50
B1	55	498.0	53.8 to 56.3	2.5	SP							120	31
B1	57.5	495.5	56.3 to 58.8	2.5	ML		64.6					120	22
B1	60	493.0	58.8 to 61.5	2.7	SP							120	51

SPT Depth (ft)	C_R	N_{60}	σ_{vc} (psf)	σ'_{vc} (psf) (Current) (GW)	σ'_v (psf) (Hist.) (GW)	C_N (Youd) (2001)	$(N_1)_{60}$	α	β	$(N_1)_{60cs}$ for Clean Sand	Stress Red. Coef. r_d	CSR	MSF	$CRR_{7.5}$	CRR Adjusted with MSF	Factor of Safety FS_{liq} (Liquefiable/ Non Liquefiable) (Min FS = 1.3)		Post-Liquefaction Reconsolidation Settlement: Vol. Strain ε_v	Seismic Settle. (in)	Cum. Settle. (in)
2.5	0.75	3.9	300.0	300.0	300.0	1.70	6.6	0.00	1.00	6.6	0.994	0.625	1.24	N/A	N/A	N/A	**Non Liq**	0.0000	0.00	0.00
5	0.80	2.8	600.0	600.0	600.0	1.70	4.7	0.00	1.00	4.7	0.988	0.621	1.24	N/A	N/A	N/A	**Non Liq**	0.0000	0.00	0.00
7.5	0.80	8.3	900.0	900.0	900.0	1.53	12.7	0.00	1.00	12.7	0.983	0.618	1.24	N/A	N/A	N/A	**Non Liq**	0.0000	0.00	0.00
10	0.85	7.3	1200.0	1200.0	1200.0	1.33	9.7	0.00	1.00	9.7	0.977	0.614	1.24	N/A	N/A	N/A	**Non Liq**	0.0000	0.00	0.00
12.5	0.85	16.1	1500.0	1500.0	1344.0	1.19	19.2	0.00	1.00	19.2	0.971	0.681	1.24	N/A	N/A	N/A	**Non Liq**	0.0000	0.00	0.00
15	0.95	19.7	1800.0	1800.0	1488.0	1.08	21.3	0.00	1.00	21.3	0.965	0.734	1.24	0.233	Non Liq	N/A	**Non Liq**	0.0000	0.00	0.00
17.5	0.95	19.7	2100.0	2100.0	1632.0	1.00	19.7	4.25	1.11	26.2	0.959	0.776	1.24	0.318	0.393	0.51	**Liq**	0.0167	0.50	0.50
20	0.95	18.0	2400.0	2400.0	1776.0	0.94	16.9	5.00	1.20	25.3	0.953	0.809	1.24	0.298	0.369	0.46	**Liq**	0.0183	0.55	1.05
22.5	0.95	22.9	2700.0	2700.0	1920.0	0.89	20.3	5.00	1.20	29.4	0.948	0.838	1.24	0.429	Non Liq	N/A	**Non Liq**	0.0000	0.00	1.05
25	0.95	26.2	3000.0	3000.0	2064.0	0.84	22.0	3.45	1.07	27.1	0.942	0.861	1.24	0.341	0.422	0.49	**Liq**	0.0156	0.47	1.52
27.5	0.95	24.6	3300.0	3300.0	2208.0	0.80	19.7	5.00	1.20	28.6	0.936	0.879	1.24	0.393	0.487	0.55	**Liq**	0.0167	0.50	2.02
30	1.00	44.9	3600.0	3600.0	2352.0	0.77	34.4	1.86	1.04	37.5	0.93	0.895	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
32.5	1.00	32.8	3900.0	3900.0	2496.0	0.74	24.1	5.00	1.20	34.0	0.91	0.894	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
35	1.00	34.5	4200.0	4200.0	2640.0	0.71	24.5	5.00	1.20	34.4	0.889	0.889	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
37.5	1.00	38.0	4500.0	4500.0	2784.0	0.69	26.0	4.62	1.14	34.4	0.869	0.883	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
40	1.00	41.4	4800.0	4800.0	2928.0	0.66	27.5	2.17	1.04	30.8	0.848	0.874	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
42.5	1.00	38.0	5100.0	5100.0	3072.0	0.64	24.4	4.49	1.13	32.1	0.828	0.864	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
45	1.00	79.4	5400.0	5400.0	3216.0	0.63	49.7	0.00	1.00	49.7	0.808	0.853	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
47.5	1.00	69.0	5700.0	5700.0	3360.0	0.61	42.0	0.00	1.00	42.0	0.787	0.839	1.24	(N1)60cs >= 30	1.300	N/A	**Non Liq**	0.0000	0.00	2.02
50	1.00	51.8	6000.0	6000.0	3504.0	0.59	30.7	0.00	1.00	30.7	0.767	0.826	1.24	(N1)60cs >= 30	1.300	1.57	**Non Liq**	0.0000	0.00	2.02
52.5	1.00	86.3	6300.0	6300.0	3648.0	0.58	50.0	0.00	1.00	50.0	0.747	0.811	1.24	(N1)60cs >= 30	1.300	1.60	**Non Liq**	0.0000	0.00	2.02
55	1.00	53.5	6600.0	6600.0	3792.0	0.57	30.3	0.00	1.00	30.3	0.726	0.794	1.24	(N1)60cs >= 30	1.300	1.64	**Non Liq**	0.0000	0.00	2.02
57.5	1.00	38.0	6900.0	6900.0	3936.0	0.55	21.0	5.00	1.20	30.2	0.706	0.778	1.24	(N1)60cs >= 30	1.300	1.67	**Non Liq**	0.0000	0.00	2.02
60	1.00	88.0	7200.0	7200.0	4080.0	0.54	47.7	0.00	1.00	47.7	0.686	0.761	1.24	(N1)60cs >= 30	1.300	1.71	**Non Liq**	0.0000	0.00	2.02

BYER GEOTECHNICAL INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#1a**
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	100.0 psf
Phi Angle, φ	28.0 degrees
Density, γ	120.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	CANTILEVERED
Retained Height, H	12 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1190 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	54.8 degrees
Area of Critical Wedge	50.0 square feet
Length of Critical Failure Plane, L	12.8 feet
Depth of Critical Tension Crack	1.6 feet
Horizontal Upslope Distance to Critical Tension Crack	7.4 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	19.5 degrees
Factored Cohesion on Critical Slip Plane, C'	66.7 psf
Weight of Critical Wedge, W	6,000 pounds
External Surcharge on Critical Wedge, V	0 pounds
Static Gravitational Driving Force, W'	6,000 pounds
Mobilized Cohesive Force, C'L	853 pounds
Mobilized Frictional Force, R	6,494 pounds
Calculated Unbalanced Force, P	3,256 pounds
Calculated Horizontal Unbalanced Force, P_h	3,256 pounds
Calculated Equivalent Fluid Pressure	45.2 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	46.0 pcf
Design Horizontal Force	3,312 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,541)	(0,0)	12		
(0,553)	(0,12)			
(15,553)	(15,12)			Uniform Load: 300 psf
(25,553)	(25,12)			
(26,553)	(26,12)			
(27,553)	(27,12)			
(30,553)	(30,12)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 12 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 46 POUNDS PER CUBIC FOOT.

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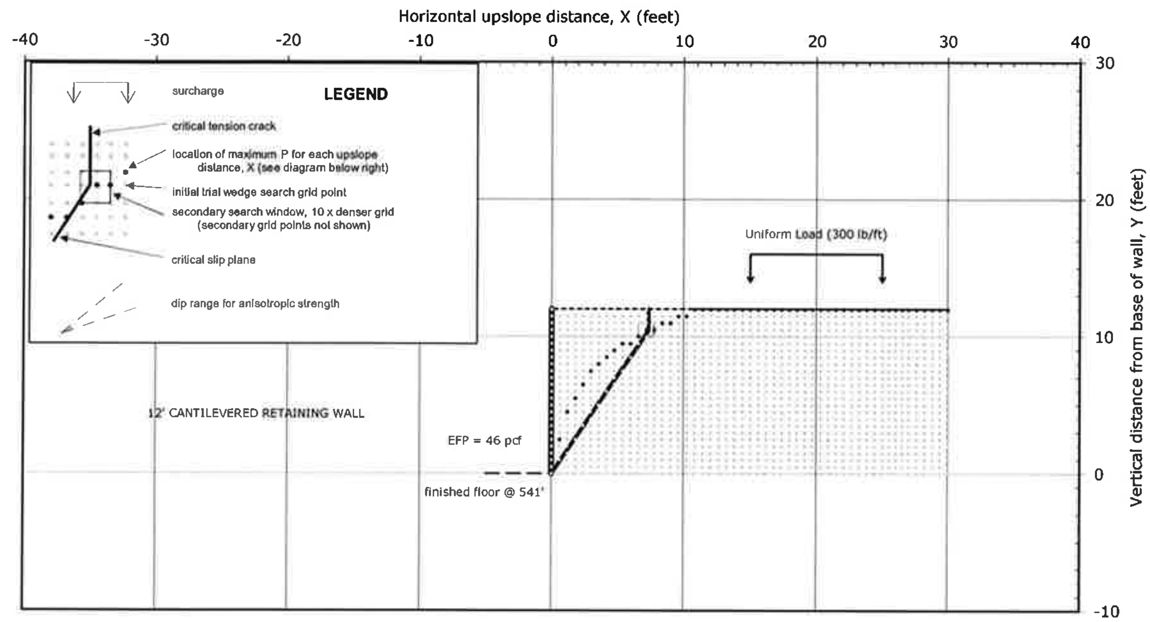
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

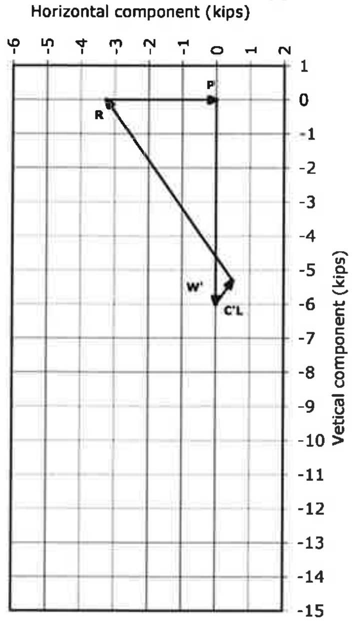
BG: **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#1b**
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



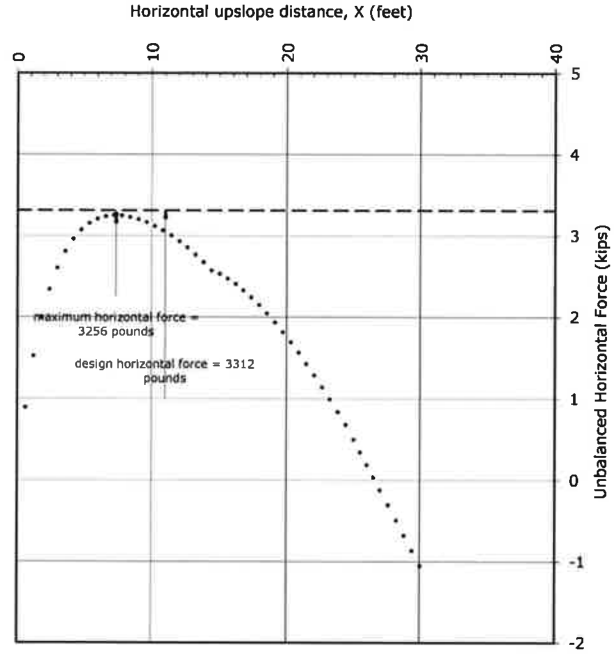
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG 23084 CLIENT: 3700 West Riverside Investments, LLC
CONSULTANT: RSB
SHEET: #1Sa
Cantilevered Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBE-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	100.0 psf
Phi Angle, φ	28.0 degrees
Density, γ	120.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	CANTILEVERED
Retained Height, H	12 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	SEISMIC
PGA_M	0.97 g

Pseudostatic Coefficients:

horizontal, K_h***	0.32 g
vertical, K_v****	0.00 g
Calculation Safety Factor, FS	1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)
**** Kv > 0 indicates downward acceleration and upward inertial force

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist., elev)	(X, Y)	H (ft)	β (deg)	surcharge
(0,541)	(0,0)	12		
(0,553)	(0,12)			
(15,553)	(15,12)			Uniform Load: 300 psf
(25,553)	(25,12)			
(26,553)	(26,12)			
(27,553)	(27,12)			
(30,553)	(30,12)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case

Critical Failure Angle, α	54.8 degrees
Area of Critical Wedge	50.0 square feet
Length of Critical Failure Plane, L	12.8 feet
Depth of Critical Tension Crack	1.6 feet
Horizontal Upslope Distance to Critical Tension Crack	7.4 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	28.0 degrees
Factored Cohesion on Critical Slip Plane, C'	100.0 psf
Weight of Critical Wedge, W	6,000 pounds
External Surcharge on Critical Wedge, V	0 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	6,304 pounds
Mobilized Cohesive Force, C'L	1,279 pounds
Mobilized Frictional Force, R	5,550 pounds
Calculated Unbalanced Force, P	3,809 pounds
Calculated Horizontal Unbalanced Force, P_h	3,809 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	3,809 pounds
Recommended Static Horizontal Force from sheet 1a	3,312 pounds
Calculated Seismic Force ***	497 pounds

*** the seismic force should be applied at 0.6H, where H is the retained height

CONCLUSIONS

THE CALCULATED SEISMIC FORCE ON THE WALL IS THE DIFFERENCE BETWEEN THE PSEUDO-STATIC AND STATIC FORCE, AND IS 497 POUNDS. THE WALL SHOULD BE DESIGNED FOR THIS FORCE IN ADDITION TO THE RECOMMENDED DESIGN PARAMETERS ON SHEET 1A. THE SEISMIC FORCE MAY BE APPLIED AT 0.6H ABOVE THE BASE, WHERE H IS THE RETAINED HEIGHT.

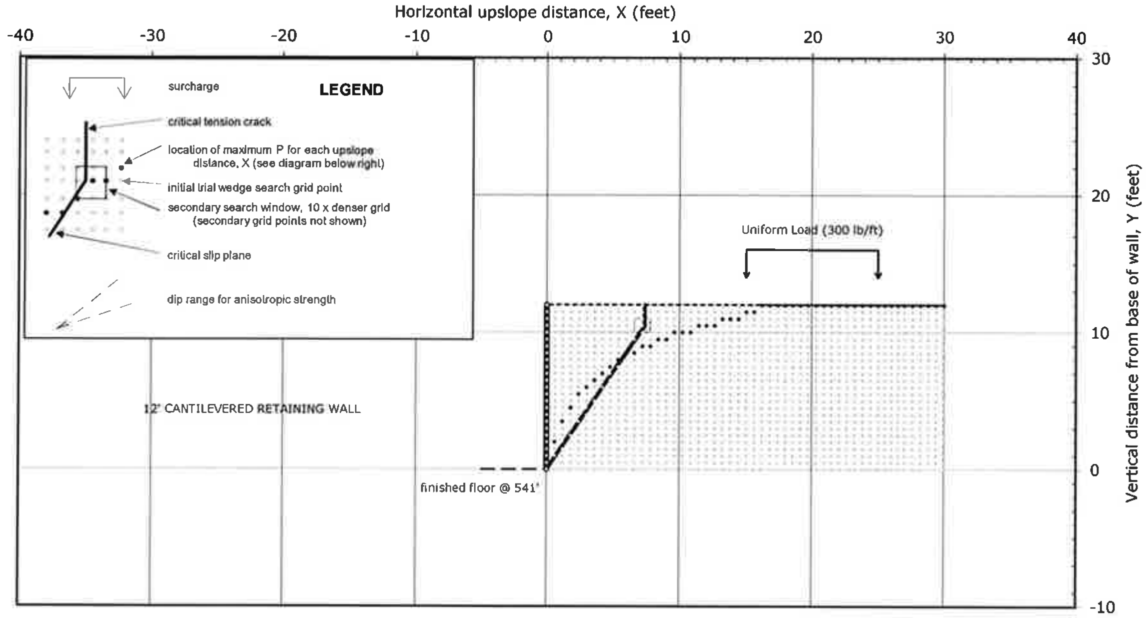
BYER
GEOTECHNICAL
INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

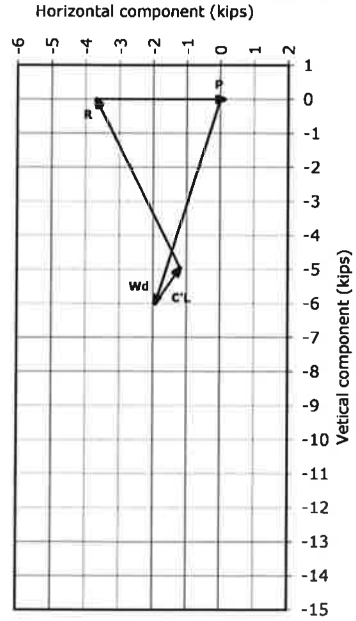
BG: **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#1Sb**
Cantilevered Retaining Wall, basement

Cross Section and Critical Active Wedge



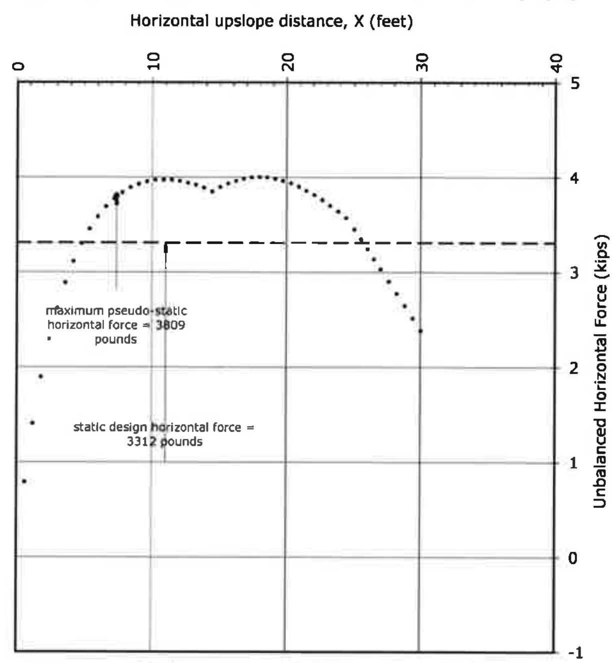
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

BYER GEOTECHNICAL INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#2a**
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	100.0 psf
Phi Angle, φ	28.0 degrees
Density, γ	120.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	RESTRAINED
Retained Height, H	12 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.5

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1190 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	54.8 degrees
Area of Critical Wedge	50.0 square feet
Length of Critical Failure Plane, L	12.8 feet
Depth of Critical Tension Crack	1.6 feet
Horizontal Upslope Distance to Critical Tension Crack	7.4 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	19.5 degrees
Factored Cohesion on Critical Slip Plane, C'	66.7 psf
Weight of Critical Wedge, W	6,000 pounds
External Surcharge on Critical Wedge, V	0 pounds
Static Gravitational Driving Force, W'	6,000 pounds
Mobilized Cohesive Force, C'L	853 pounds
Mobilized Frictional Force, R	6,494 pounds
Calculated Unbalanced Force, P	3,256 pounds
Calculated Horizontal Unbalanced Force, P_h	3,256 pounds
Calculated Trapezoidal Design Pressure *	28.3 H psf
Calculated At-Rest Equivalent Fluid Pressure **	63.7 pcf
Calculated At-Rest Trapezoidal Earth Pressure *	39.8 H psf

RECOMMENDED DESIGN PARAMETERS

Trapezoidal Design Pressure, TDP*	40 H psf
Design Horizontal Force	4,608 pounds

* H is restrained height, see report for diagram of trapezoidal pressure distribution
** at-rest equivalent fluid pressure is calculated as: γ (1- sin(φ))

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,541)	(0,0)	12		
(0,553)	(0,12)			
(15,553)	(15,12)			Uniform Load: 300 psf
(25,553)	(25,12)			
(26,553)	(26,12)			
(27,553)	(27,12)			
(30,553)	(30,12)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED RESTRAINED RETAINING WALL, WITH A RETAINED HEIGHT OF UP TO 12 FEET, MAY BE DESIGNED FOR A TRAPEZOIDAL DESIGN PRESSURE (TDP) OF 40 H POUNDS PER SQUARE FOOT, WHERE H IS THE RETAINED HEIGHT. SEE REPORT FOR DIAGRAM OF TRAPEZOIDAL PRESSURE DISTRIBUTION.

THE STATIC DESIGN IS GOVERNED BY THE AT-REST CONDITION.

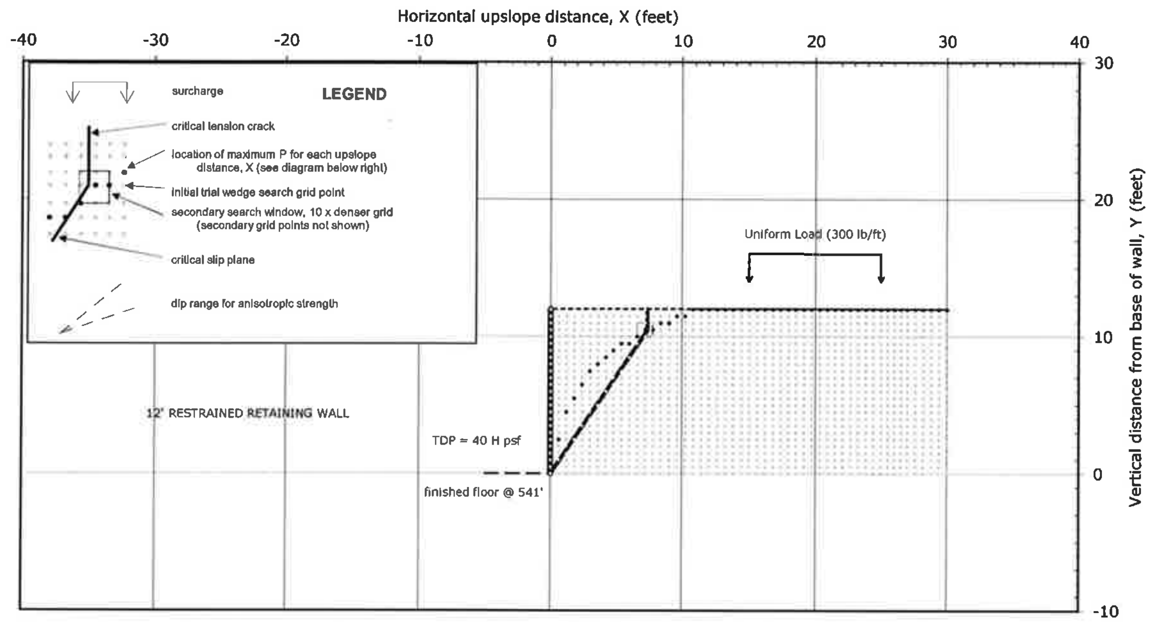
BYER
GEOTECHNICAL
INC.

1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

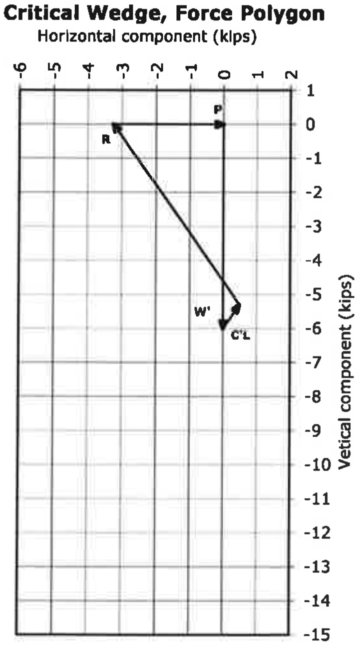
BG: **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#2b**
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



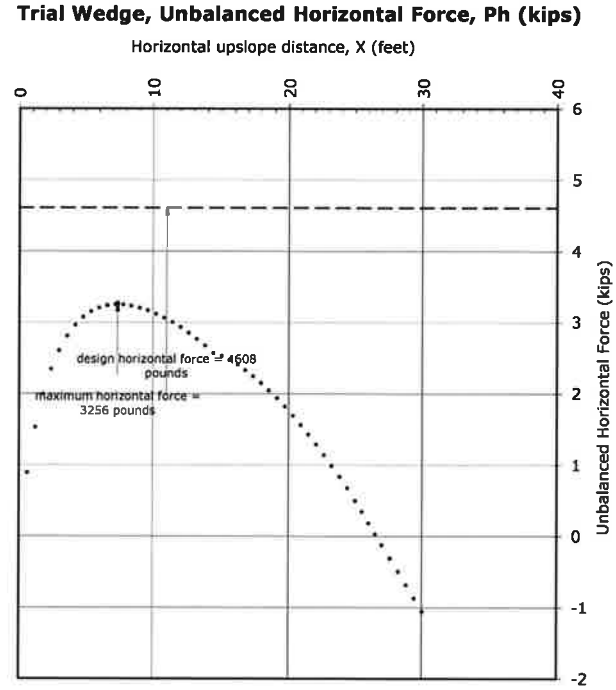
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

BG 23084 CLIENT: 3700 West Riverside Investments, LLC
CONSULTANT: RSB
SHEET: #2Sa
Restrained Retaining Wall, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED RESTRAINED RETAINING WALL. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE PSEUDO-STATIC (MONONOBE-OKABE) METHOD FOR SEISMIC LOADING.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	100.0 psf
Phi Angle, φ	28.0 degrees
Density, γ	120.0 pcf
Anisotropic Strength Function	NO
Restraining Device	RETAINING WALL
Type	RESTRAINED
Retained Height, H	12 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	SEISMIC
PGA_M	0.97 g

Pseudostatic Coefficients:

horizontal , K_h***	0.32 g
vertical, K_v****	0.00 g
Calculation Safety Factor, FS	1

* Critical wedge 'sees' only portion of regional backslope

*** Calculated using methodology of Abrahamson and Silva (1986)

**** Kv > 0 indicates downward acceleration and upward inertial force

CALCULATION OUTPUT

Use Critical Trial Wedge From Static Case

Critical Failure Angle, α	54.8 degrees
Area of Critical Wedge	50.0 square feet
Length of Critical Failure Plane, L	12.8 feet
Depth of Critical Tension Crack	1.6 feet
Horizontal Upslope Distance to Critical Tension Crack	7.4 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	28.0 degrees
Factored Cohesion on Critical Slip Plane, C'	100.0 psf
Weight of Critical Wedge, W	6,000 pounds
External Surcharge on Critical Wedge, V	0 pounds
Pseudo-Static (Gravitational + Dynamic) Driving Force, Wd	6,304 pounds
Mobilized Cohesive Force, C'L	1,279 pounds
Mobilized Frictional Force, R	5,550 pounds
Calculated Unbalanced Force, P	3,809 pounds
Calculated Horizontal Unbalanced Force, P_h	3,809 pounds

RECOMMENDED DESIGN PARAMETERS

Calculated Pseudo-Static Horizontal Force	3,809 pounds
Recommended Static Horizontal Force from sheet 2a	4,608 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,541)	(0,0)	12		
(0,553)	(0,12)			
(15,553)	(15,12)			Uniform Load: 300 psf
(25,553)	(25,12)			
(26,553)	(26,12)			
(27,553)	(27,12)			
(30,553)	(30,12)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

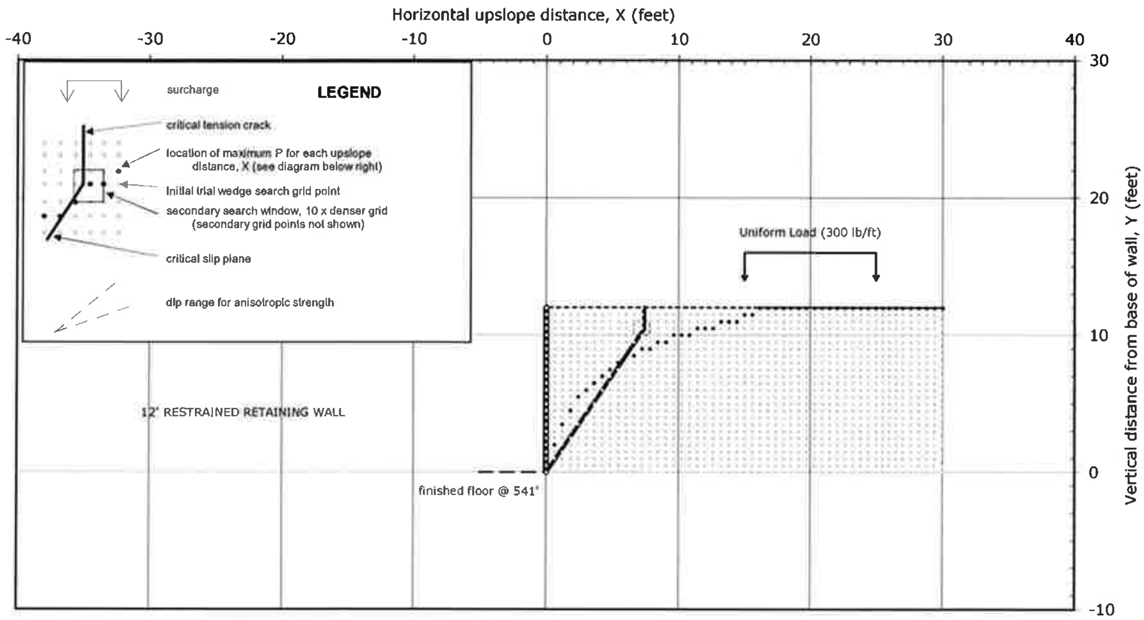
THE CALCULATED STATIC FORCE EXCEEDS THE CALCULATED PSEUDO-STATIC FORCE. THEREFORE, THE RECOMMENDED DESIGN PARAMETERS ON SHEET 2A ARE SUFFICIENT.

BYER GEOTECHNICAL INC.
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

RETAINING WALL CALCULATION

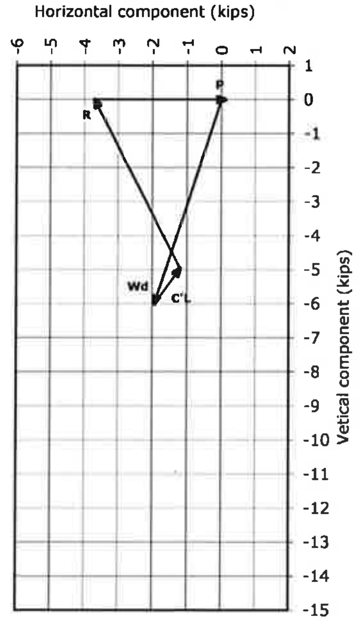
BG: **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#2Sb**
Restrained Retaining Wall, basement

Cross Section and Critical Active Wedge



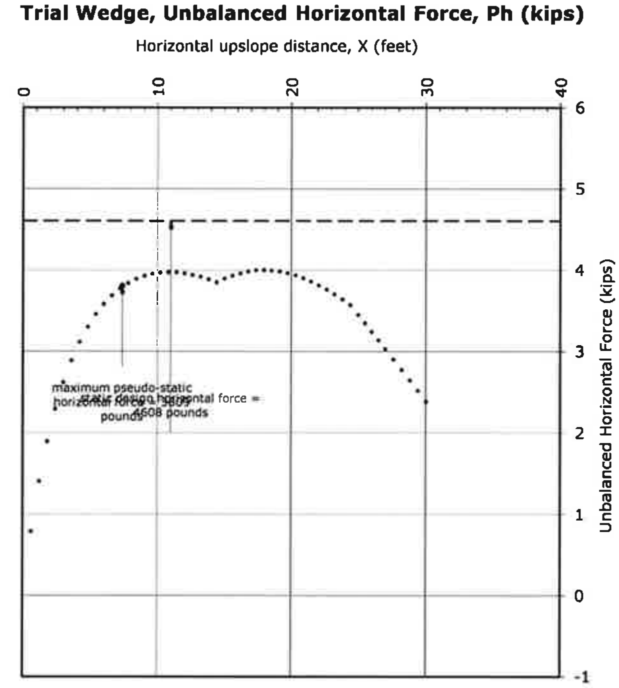
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the pseudo-static (gravitational and dynamic) driving force, Wd; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

BG 23084 CLIENT: 3700 West Riverside Investments, LLC
CONSULTANT: RSB
SHEET: #3a
Cantilevered Shoring Pile, basement

CALCULATE THE DESIGN PRESSURE FOR PROPOSED CANTILEVERED SHORING PILE. USE THE GENERAL TRIAL WEDGE METHOD*. APPLY THE SAFETY FACTOR TO THE COHESION AND PHI ANGLE. THE RETAINED HEIGHT, BACKSLOPE GEOMETRY, AND SURCHARGE CONDITIONS, ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

* FIND THE WEDGE, CHARACTERIZED BY A SINGLE STRAIGHT SLIP PLANE AND A VERTICAL TENSION CRACK, THAT MAXIMIZES THE UNBALANCED PRESSURE. MAKE NO ASSUMPTION ABOUT TENSION CRACK DEPTH. ALLOW ANY BACKSLOPE GEOMETRY AND SURCHARGE CONDITION. VARY X- AND Y-COORDINATES OF BOTTOM OF TENSION CRACK. USE PRIMARY GRID AND SECONDARY SEARCH WINDOW TO FOCUS SEARCH. USE METHODOLOGY DESCRIBED IN NAVFAC DESIGN MANUAL 7.02, 1986, PP. 59-70, AND US ARMY TECHNICAL REPORT ITL-92-11 (1992), P. 79 AND APPENDIX A.

CALCULATION INPUT

Earth Material	Alluvium
Shear Diagram	#1
Cohesion, Coh	100.0 psf
Phi Angle, φ	28.0 degrees
Density, γ	120.0 pcf
Anisotropic Strength Function	NO
Restraining Device	SHORING PILE
Type	CANTILEVERED
Retained Height, H	16 feet
Wall Friction Angle, δ	0 degrees
External Surcharge	see below
General Backslope Condition*	level
Loading	STATIC
Calculation Safety Factor, FS	1.25

* Critical wedge 'sees' only portion of regional backslope

CALCULATION OUTPUT

Trial Wedges Analyzed, Initial Search Grid	1606 trials
Trial Wedges Analyzed, Secondary Search Window	324 trials
Critical Failure Angle, α	56.6 degrees
Area of Critical Wedge	83.2 square feet
Length of Critical Failure Plane, L	16.8 feet
Depth of Critical Tension Crack	2.0 feet
Horizontal Upslope Distance to Critical Tension Crack	9.2 feet
Effective Backslope on Critical Wedge, β_{eff}	0.0 degrees
Factored Phi Angle on Slip Plane, φ'	23.0 degrees
Factored Cohesion on Critical Slip Plane, C'	80.0 psf
Weight of Critical Wedge, W	9,979 pounds
External Surcharge on Critical Wedge, V	0 pounds
Static Gravitational Driving Force, W'	9,979 pounds
Mobilized Cohesive Force, C'L	1,342 pounds
Mobilized Frictional Force, R	10,628 pounds
Calculated Unbalanced Force, P	5,132 pounds
Calculated Horizontal Unbalanced Force, P_h	5,132 pounds
Calculated Equivalent Fluid Pressure	40.1 pcf

RECOMMENDED DESIGN PARAMETERS

Design Equivalent Fluid Pressure, EFP	41.0 pcf
Design Horizontal Force	5,248 pounds

BACKSLOPE GEOMETRY AND SURCHARGE CONDITIONS*

(dist , elev)	(X , Y)	H (ft)	β (deg)	surcharge
(0,537)	(0,0)	16		
(0,553)	(0,16)			
(15,553)	(15,16)			Uniform Load: 300 psf
(25,553)	(25,16)			
(26,553)	(26,16)			
(27,553)	(27,16)			
(30,553)	(30,16)			

* X is the upslope distance from the wall; Y is the vertical distance above the base of the wall; H is wall height; β is backslope. H, β, and surcharge apply to section between two coordinates. Only first 20 coordinates are shown.

CONCLUSIONS

THE CALCULATION INDICATES THAT THE PROPOSED CANTILEVERED SHORING PILE, WITH A RETAINED HEIGHT OF UP TO 16 FEET, MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE (EFP) OF 41 POUNDS PER CUBIC FOOT. FOR PILES, THE PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

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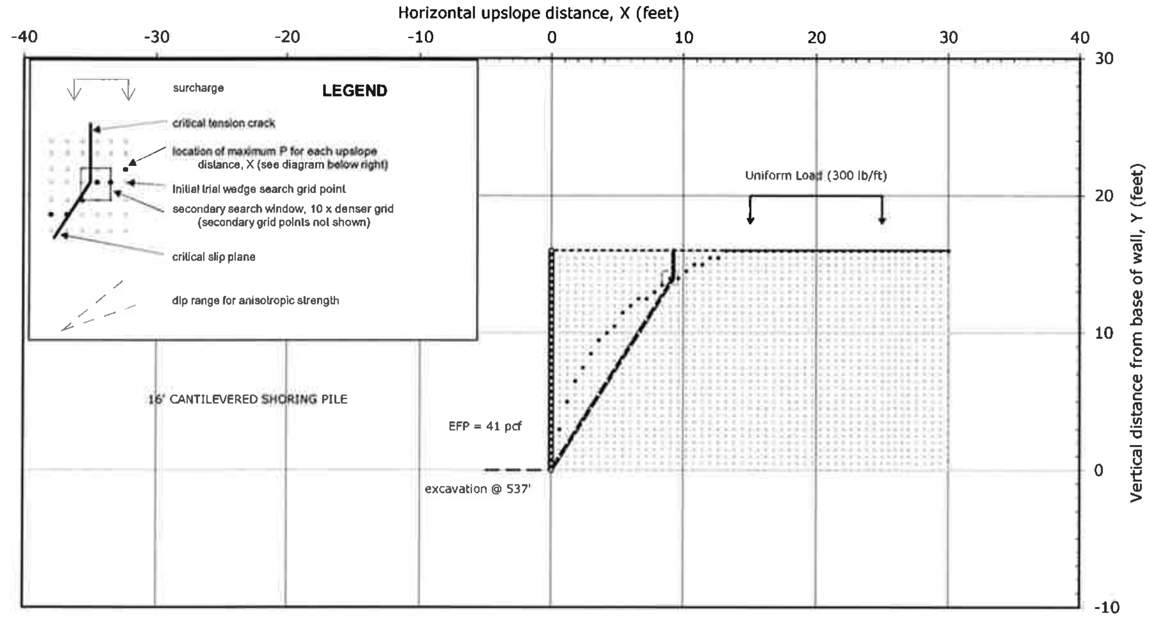
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHORING PILE CALCULATION

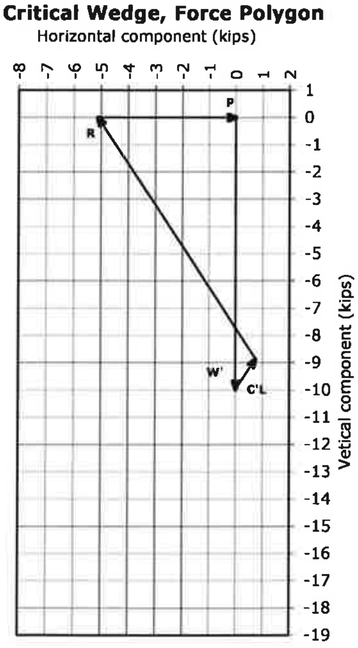
BG: **23084** CLIENT: **3700 West Riverside Investments, LLC**
CONSULTANT: **RSB**
SHEET: **#3b**
Cantilevered Shoring Pile, basement

Cross Section and Critical Active Wedge



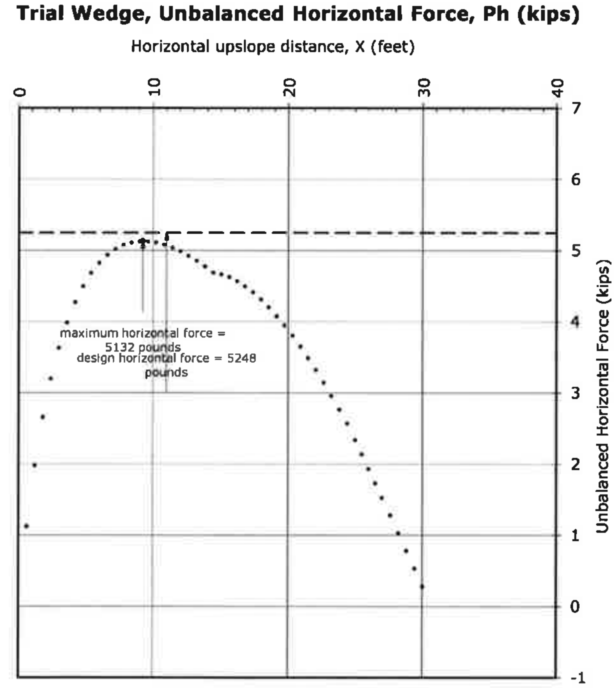
The cross section shows the surface geometry; surcharges; the range of dip for any defined anisotropic strength function; the critical trial wedge; the initial search grid; and the secondary search window. Each grid point defines the upslope coordinate of the slip plane and bottom coordinate of tension crack for a trial wedge. For each for upslope distance, X, the grid point for which the horizontal unbalanced pressure, Ph, is maximum is shown in black. The critical wedge has the maximum horizontal unbalanced pressure of all trial wedges.

Critical Wedge, Force Polygon



The polygon shows the static (gravitational) driving force, W'; the mobilized cohesive force, C'L; the mobilized frictional force, R; and the unbalanced pressure, P, for the critical wedge.

Trial Wedge, Unbalanced Horizontal Force, Ph (kips)



The maximum calculated horizontal unbalanced pressure, Ph, is plotted for each upslope distance, X. The location of the maximum Ph for each X is indicated in the cross section, above. All points from initial search grid and maximum from secondary search window are plotted.

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

AERIAL VICINITY MAP

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 100'

REFERENCE: LOS ANGELES COUNTY DEPARTMENT OF REGIONAL PLANNING, GIS-NET, 2013, http://gis.planning.lacounty.gov/GIS-NET_Public/Viewer.html

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818.543.3747 FAX

REGIONAL TOPOGRAPHIC MAP

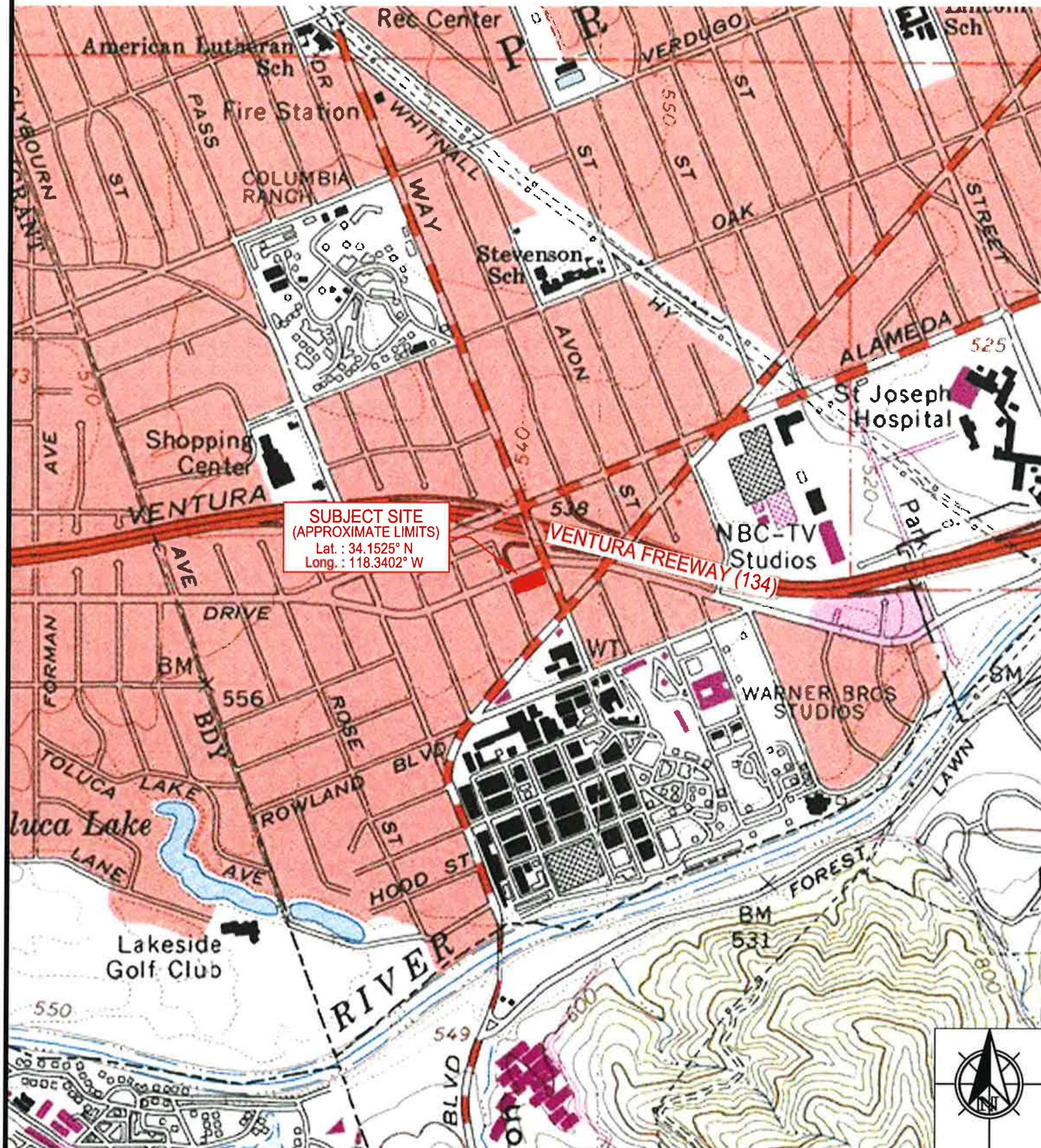
BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, BURBANK 7.5-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1981.



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818.549.9959 TEL
818.543.3747 FAX

HISTORIC TOPOGRAPHIC MAP

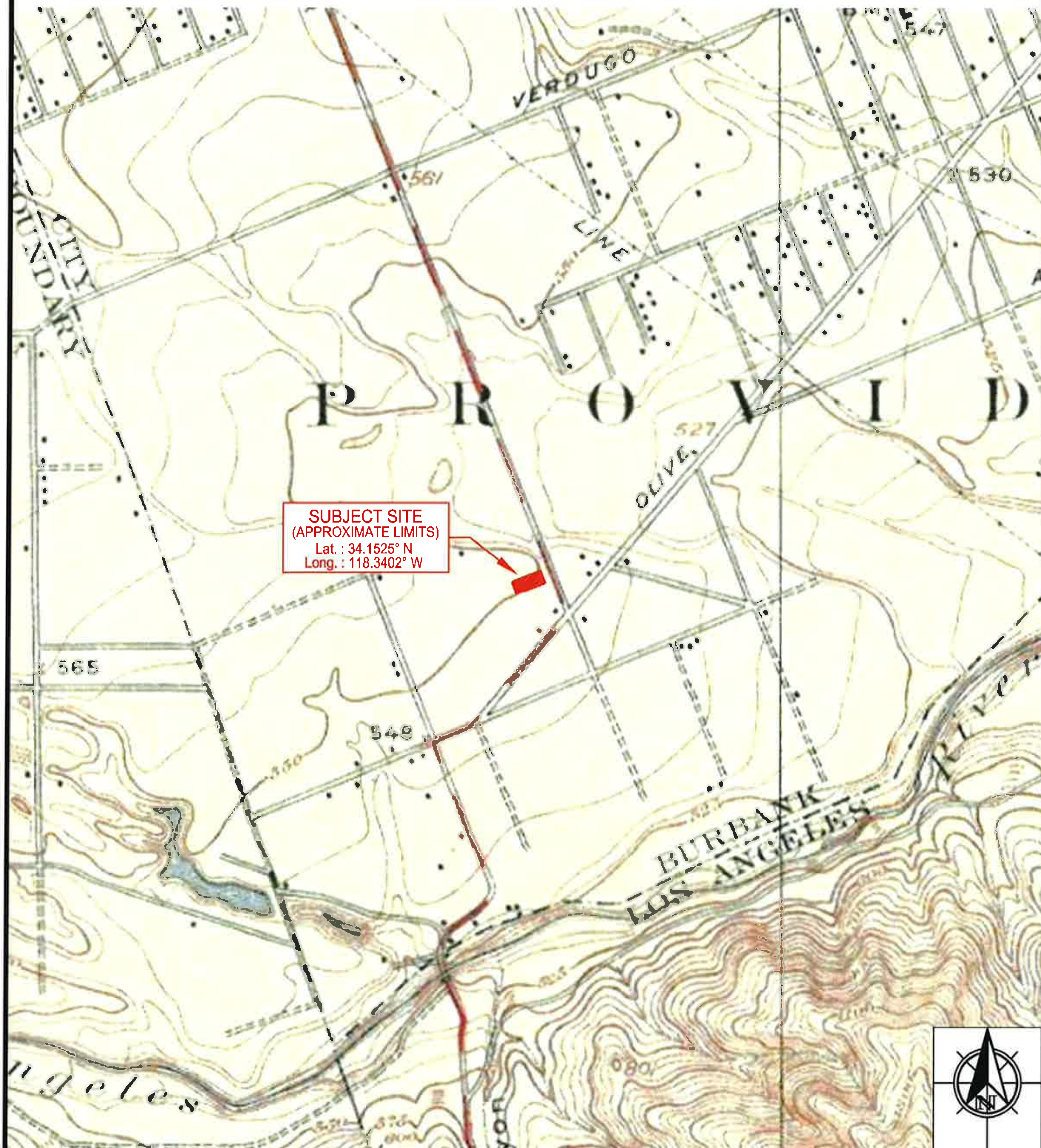
BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 1000'

REFERENCE: USGS TOPOGRAPHIC MAP, BURBANK 6-MINUTE SERIES QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA CREATED 1926.



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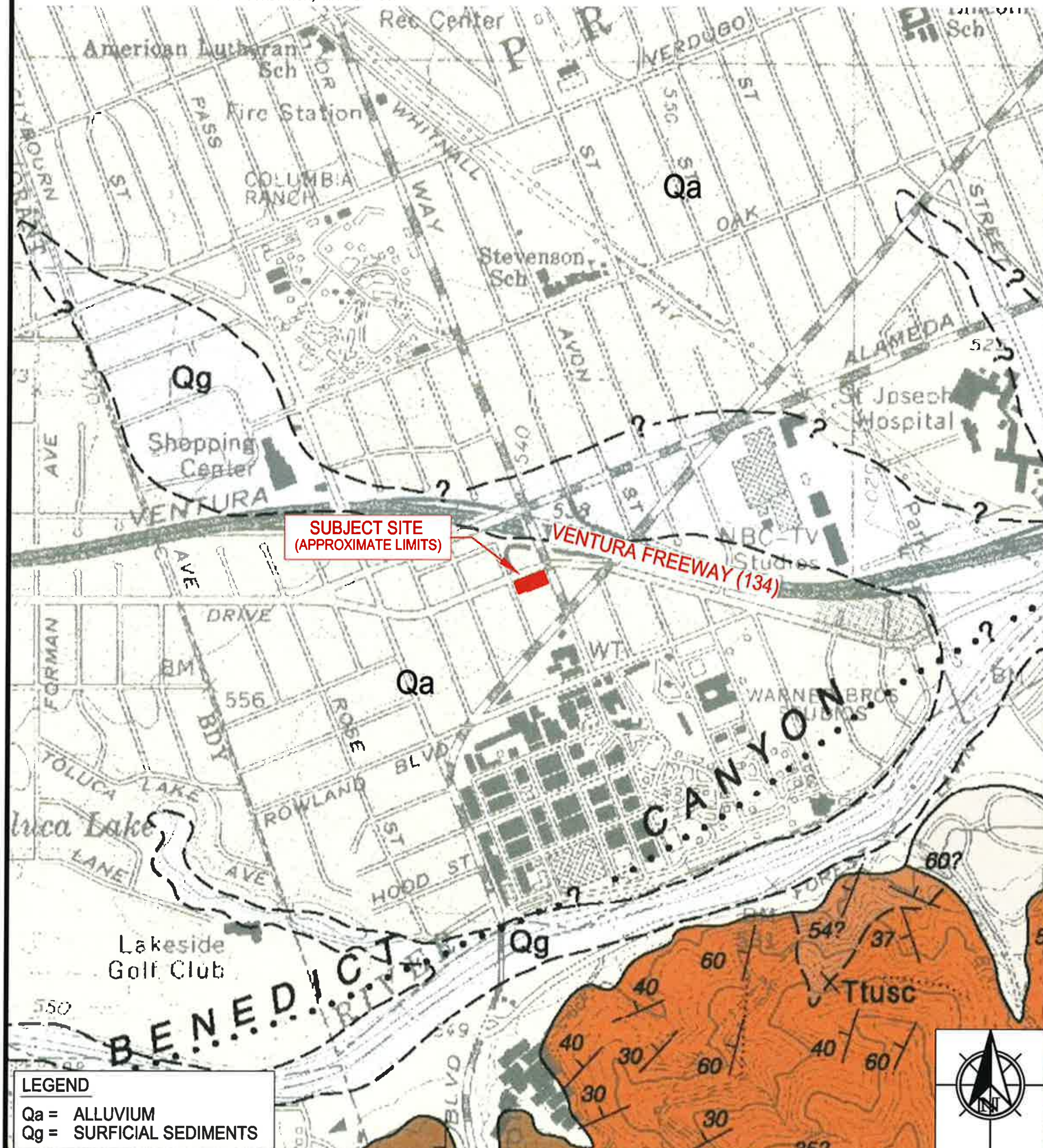
REGIONAL GEOLOGIC MAP

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB	SCALE: 1" = 1000'
DRAWN BY : AS	

REFERENCE: DIBBLEE, T.W. (1991), GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES, LOS ANGELES, CALIFORNIA DIBBLEE GEOLOGICAL FOUNDATION, MAP DF-30.



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REGIONAL FAULT MAP

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 12 MILES

REFERENCE: JENNINGS, C.W., AND BRYANT, W.A.,2010, FAULT ACTIVITY MAP OF CALIFORNIA GEOLOGICAL SURVEY, 150th ANNIVERSARY, MAP No 6.

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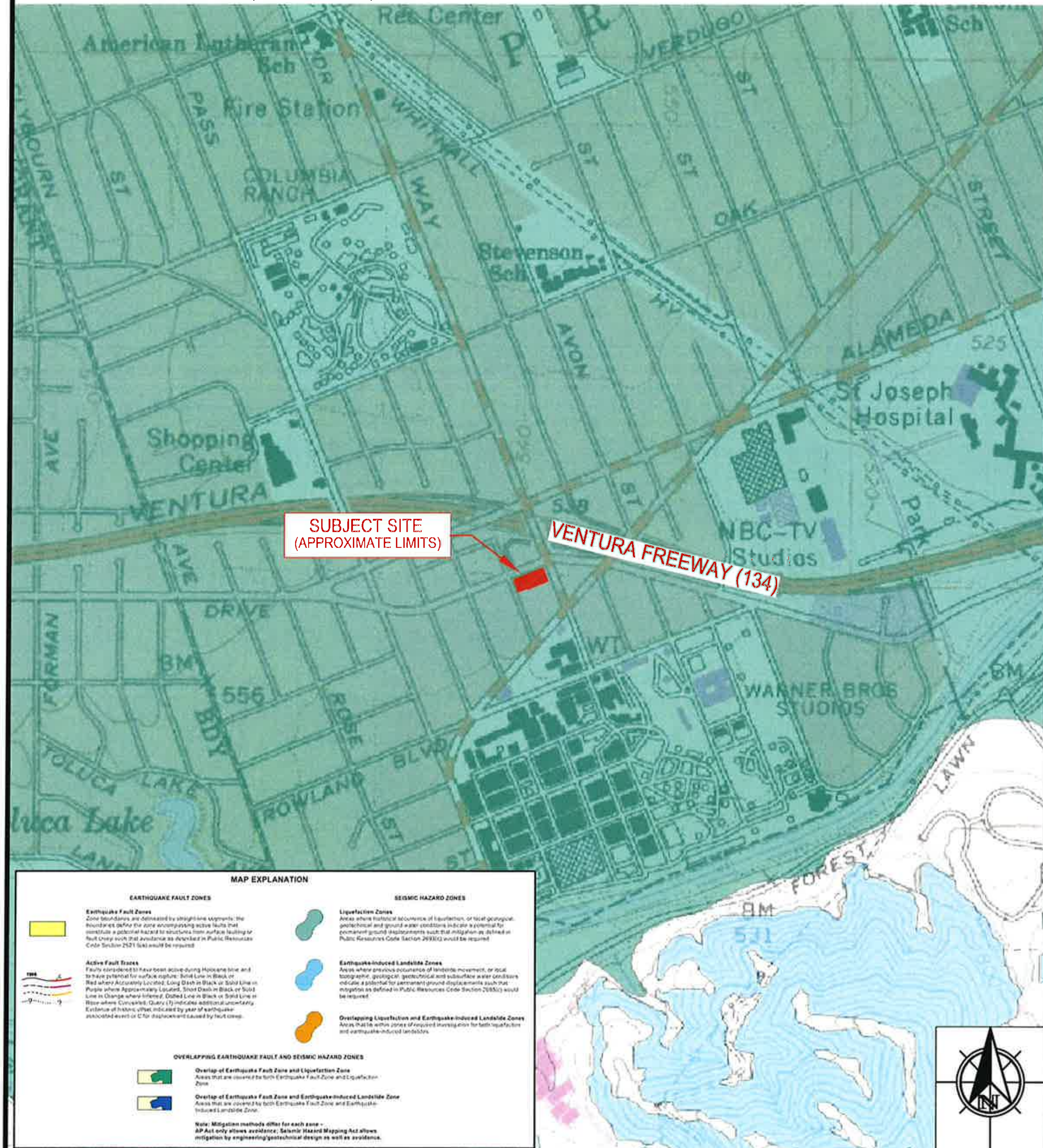
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SEISMIC HAZARD ZONES MAP

BG: 23084	3700 WEST RIVERSIDE INVESTMENTS, LLC
CONSULTANT : RSB	SCALE: 1" = 1000'
DRAWN BY : AS	

REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION BURBANK QUADRANGLE; EARTHQUAKE FAULT ZONES, DATED JANUARY 1, 1979 AND SEISMIC HAZARD ZONES, DATED MARCH 25, 1999.



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818.549.9959 TEL
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HISTORIC-HIGH GROUNDWATER MAP

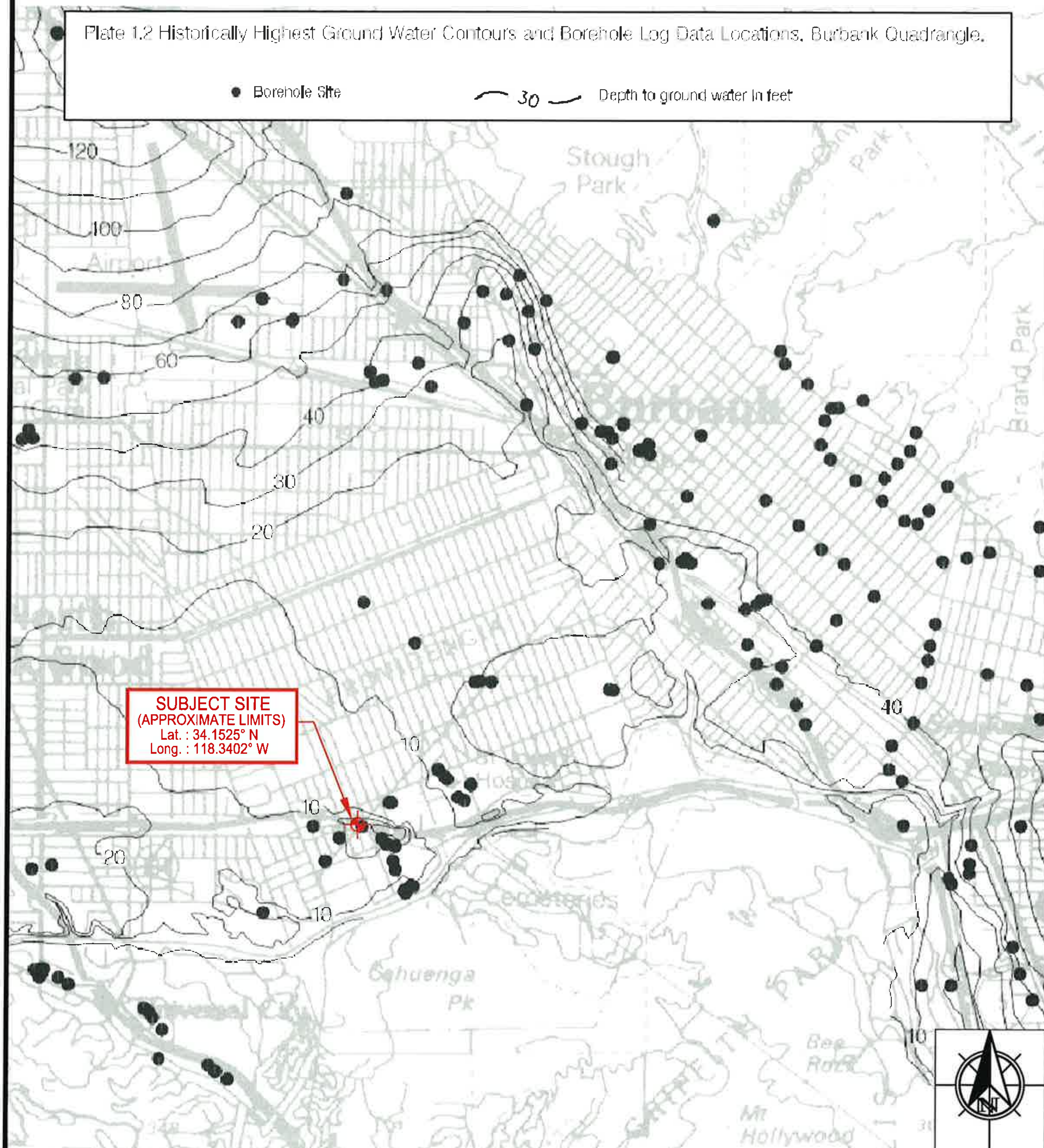
BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

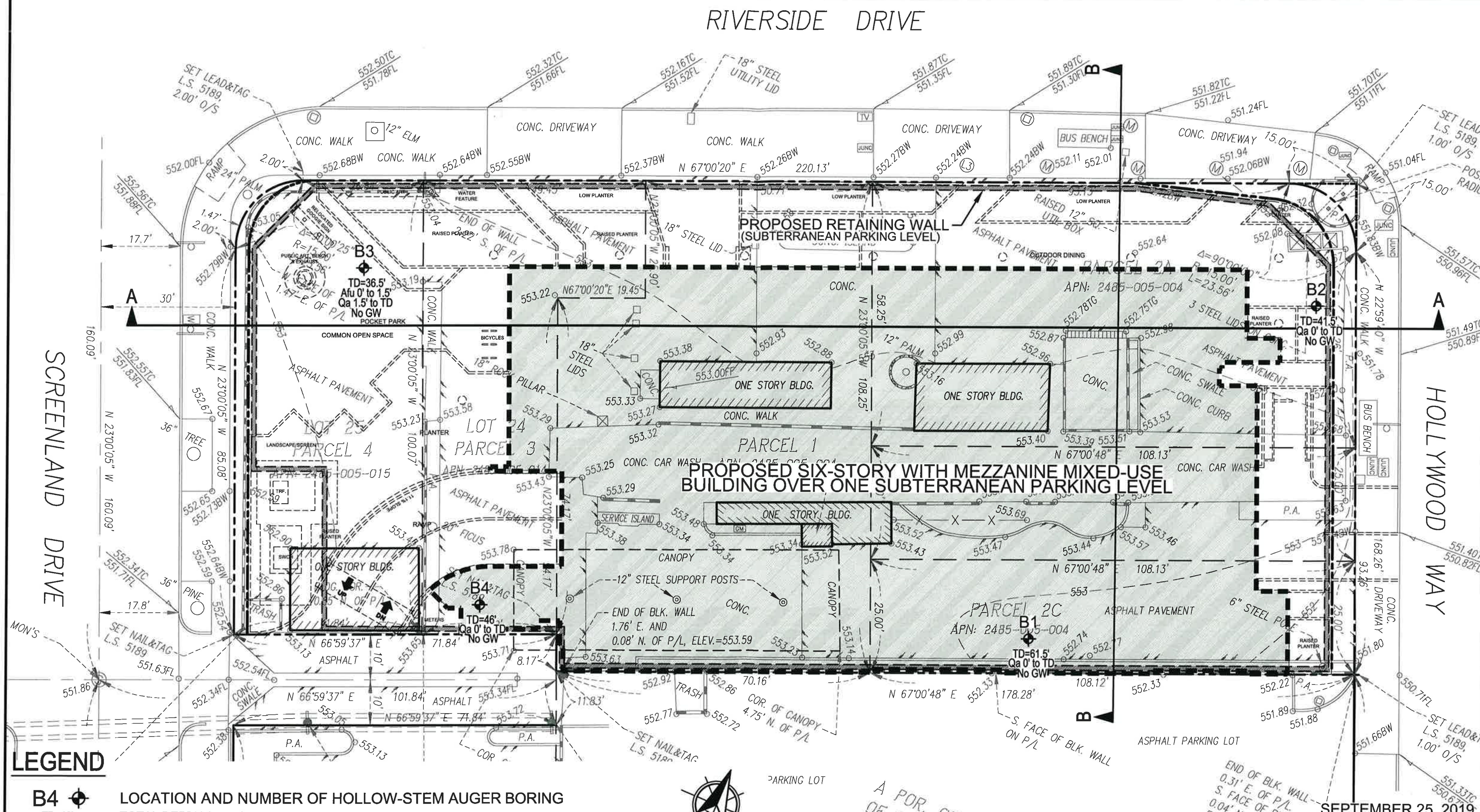
CONSULTANT : RIZ

DRAWN BY : AS

SCALE: 1" = 4000'

REFERENCE: CGS, 1998, Seismic Hazard Zone Report for the Burbank 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 016.





LEGEND

B4 ⌖	LOCATION AND NUMBER OF HOLLOW-STEM AUGER BORING
TD=46'	TOTAL DEPTH (FEET)
Qa 0' to TD	DEPTH OF ALLUVIUM (FEET)
No GW	NO GROUNDWATER ENCOUNTERED
B B	LINE OF CROSS SECTION

GRAPHIC SCALE

20 15 10 5 0 10 20 40

(IN FEET)
1 inch = 20 ft.

REFERENCE: TOPOGRAPHIC SURVEY PREPARED BY CALIFORNIA SURVEYING SERVICE, DATED 02/12/2018 AND PLANS BY STRUERE ADVANCED ARCHITECTURE, DATED 02/15/2019.

SEPTEMBER 25, 2019

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

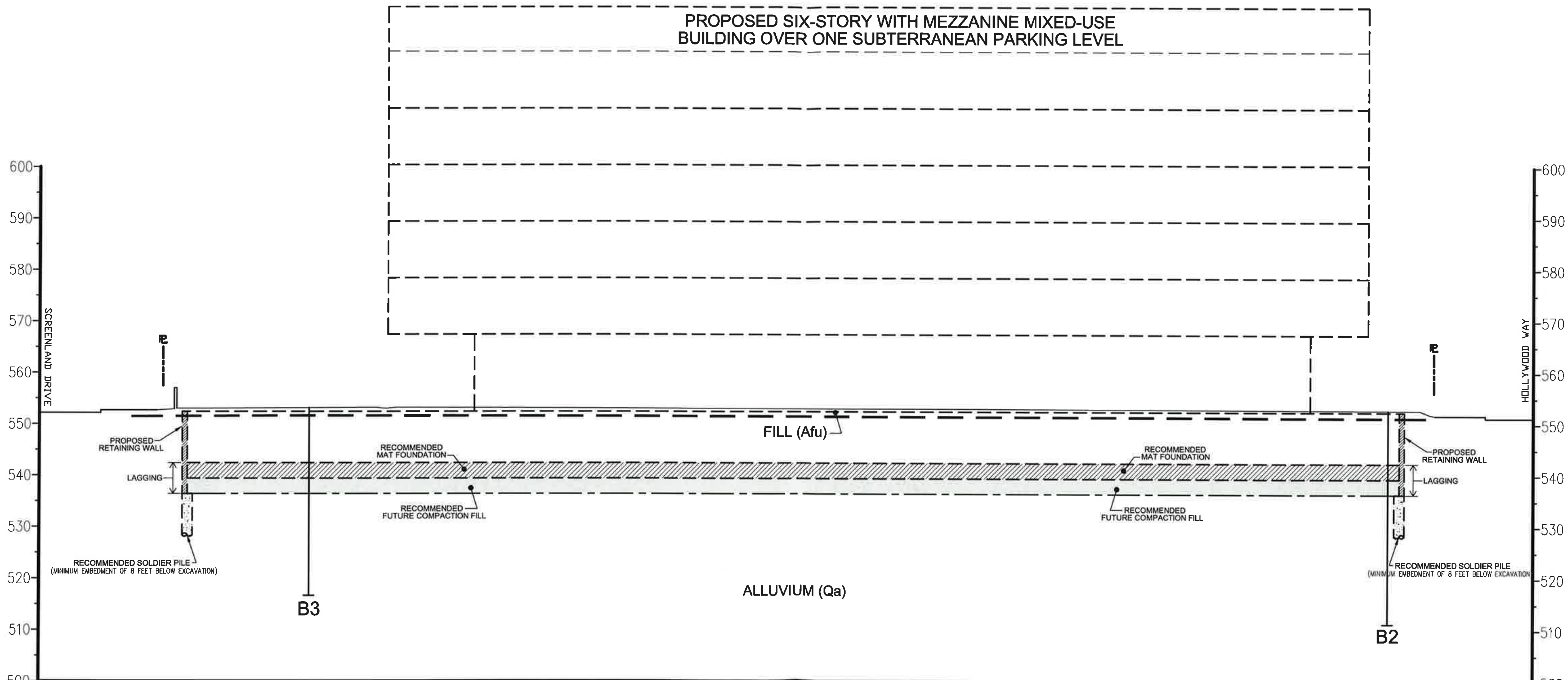
SITE PLAN

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 20'



SECTION A-A

SEPTEMBER 25, 2019

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1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

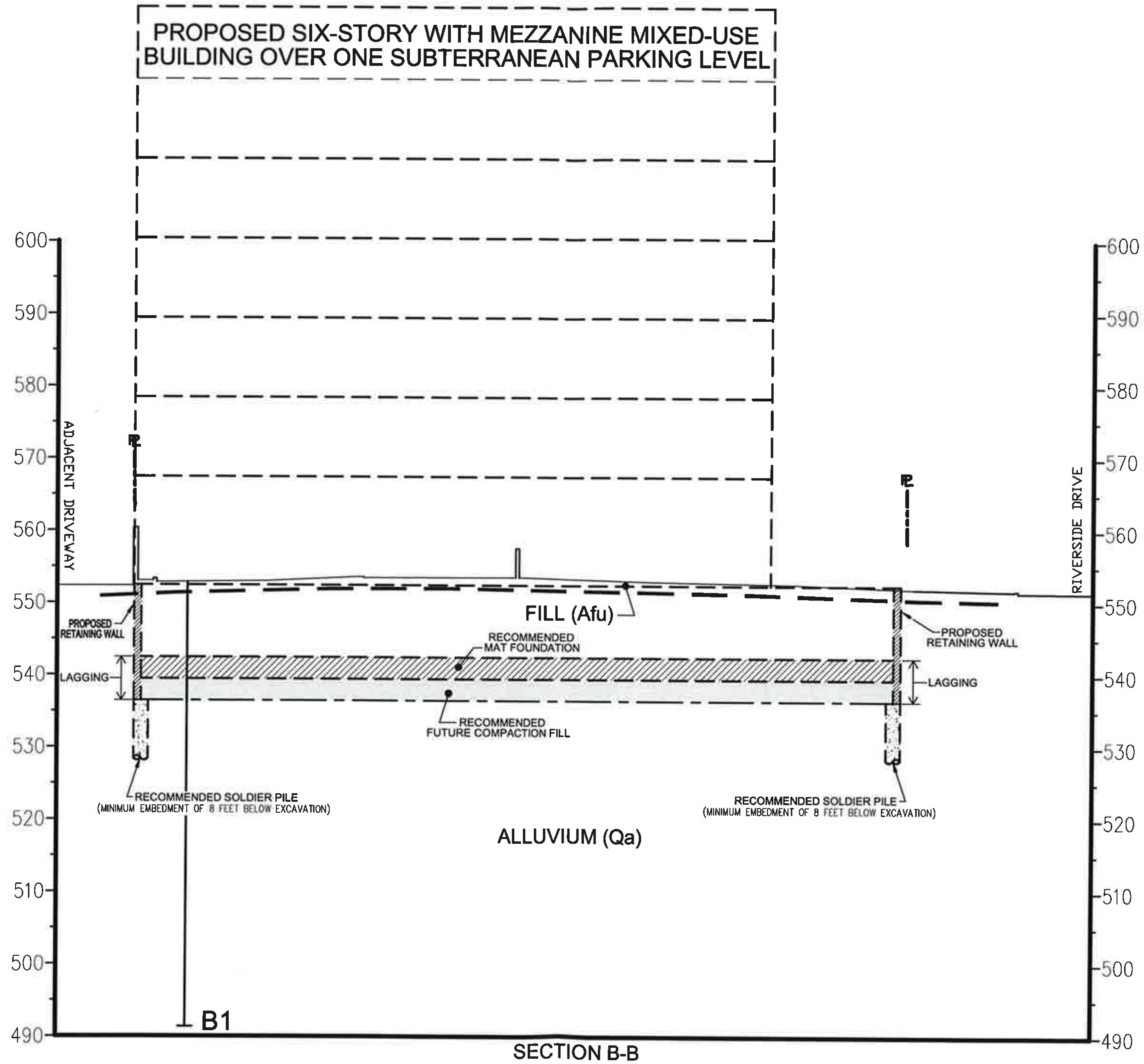
SECTION A

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 20'



SEPTEMBER 25, 2019

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818.549.9959 TEL
818.543.3747 FAX

SECTION B

BG: 23084 3700 WEST RIVERSIDE INVESTMENTS, LLC

CONSULTANT : RSB

DRAWN BY : AS

SCALE: 1" = 20'

CAD FILE: S:\LandProjects\23084_3700 Riverside Investments\DWG\SITE PLAN 07262019.dwg   PLOT DATE/TIME: 9/25/2019 - 10:10am