Appendix H:
Geotechnical Engineering Study

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GEO-ENGINEERING SOLUTIONS, INC.
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Geotechnical Engineering • Engineering Geology • Materials Testing

UPDATED DESIGN LEVEL GEOTECHNICAL ENGINEERING STUDY

Residential and Retail Development
Cleaveland Road and Crescent Plaza
Pleasant Hill, California 94523

Prepared for:

Blake/Griggs Properties
550 Hartz Avenue, Suite 200
Danville, California 94526

Prepared by:

Geo-Engineering Solutions, Inc.
2570 San Ramon Valley Boulevard, Suite A102
San Ramon, California 94583
Geo-Eng Project No. 06-1023-B

GEO-ENGINEERING SOLUTIONS, INC.
2570 San Ramon Valley Blvd., Suite A102
San Ramon, CA | 925-433-0450

May 28, 2019

Blake/Griggs Properties
550 Hartz Avenue, Suite 200
Danville, California 94526

Subject: **Updated Design Level Geotechnical Engineering Study**
Residential and Retail Development
Cleaveland Road and Crescent Plaza
Pleasant Hill, California 94523
Geo-Eng Project No. 06-1023-B

Reference:

1) *Design Level Geotechnical Engineering Study (DRAFT), Residential and Retail Development, Cleaveland Road and Crescent Plaza, prepared by Geo-Engineering Solutions, Inc., dated October 26, 2017*

Dear Mr. Sudbury:

Geo-Engineering Solutions, Inc. (Geo-Eng) has prepared an Updated Design Level Geotechnical Engineering Study for the proposed Residential and Retail Development located at Cleaveland Road and Crescent Plaza in Pleasant Hill, California. We previously prepared a draft design level geotechnical engineering study for the subject project, referenced above. It is our understanding that since the completion of the original draft report, the proposed scope of work for the subject project has changed; the proposed development now consists of the construction of a new five-story above ground residential and retail structure, with a two-story subterranean parking structure.

Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed foundations, interior and exterior concrete slabs, site grading and drainage, utility trench backfilling, and pavements. In general, the proposed improvements at the site are considered to be geotechnically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned at (925) 433-0450; or by e-mail at eswenson@geo-eng.net.

We greatly appreciate the opportunity to be of service to Blake/Griggs Properties and to be involved in the design of this project.

Sincerely,

GEO-ENGINEERING SOLUTIONS, INC.

Colin Frost, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER COLIN J. FROST C 87575 Exp. 9.2019 CIVIL STATE OF CALIFORNIA

Eric J. Swenson, G.E., C.E.G.
President

CERTIFIED ENGINEERING GEOLOGIST ERIC J. SWENSON No. 1855 Exp. 9.2019 REGISTERED STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER ERIC J. SWENSON No. 2474 Exp. 12/2020 GEOTECHNICAL STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of our work is to prepare a Geotechnical Engineering Study, evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed development. We have provided specific recommendations regarding suitability and geotechnical concerns relative to the proposed structural design.

The scope of this study includes conducting field exploration and laboratory testing programs, engineering analysis of the collected samples and test results, and preparation of this report. The conclusions and recommendations presented in this report are based on the limited samples collected and analyzed during this study, and on prudent engineering judgment and experience. This study does not include an in-depth assessment of potentially toxic or hazardous materials that may currently be present on or beneath the site.

1.2 Site Description

The proposed project includes improvements on a parcel located southeast of the intersection of Cleaveland Road and Crescent Plaza at 85 Cleaveland Road, in Pleasant Hill, California, as shown on *Figure 1 – Site Vicinity Map*. The project site is currently occupied by a tilt up concrete structure and associated parking lot, with some landscape areas. It is bordered by Cleaveland Road to the west, residential developments and the Century Movie Theater to the north, the Century Movie Theater and Crescent Drive to the east, and residential developments and Boyd Drive to the south. The site has generally flat topography with approximate elevations on the order of +52 to +54 feet, based off Google Earth elevations.

1.3 Proposed Development

The proposed development is shown on *Figure 2 – Developmental Site Plan,* which will consist of the construction of a new five-story above ground residential and retail structure, with the excavation of a two-story subterranean parking structure. The building will have one and two bedroom units, amenity areas, a parking garage, and landscape areas. The proposed gross floor area of the structure is roughly 228,870 square feet. Associated improvements will include grading, parking, landscaping, and utilities. It should be noted that the current project scope is omitting the originally proposed Crescent Drive Building, originally located southwest of the main proposed development.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time, Geo-Eng should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geo-Eng should be notified to determine if additional recommendations are required. Additionally, if Geo-Eng is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid, since Geo-Eng's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are consistent with the subsurface conditions revealed during construction. Geo-Eng's involvement should include foundation and grading plan review; observation of foundation excavations; grading observation and testing; testing of utility trench backfill; and testing of subgrade preparation in new hardscape and pavement areas, and pavement baserock and asphalt concrete sections.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various United States Geological Survey (USGS), California Geological Survey (CGS) and online resources, and other applicable government publications and maps, as included in the References section.

2.2 Field Exploration

A total of eight borings were drilled at the site on September 28 to 29, 2017, at the locations shown on *Figure 2 – Site Development Plan* and *Figure 3 – Site Plan and Site Geology Map*. The borings were drilled to depths between 15 feet and 50 feet. The borings were drilled using a truck mounted B-53 drill rig equipped with an 8-inch diameter, hollow stem auger.

A Geo-Eng field geologist visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a 140-pound wireline hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded in the field as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. Following the completion of drilling, the boreholes were backfilled using a cement grout.

All of the field blow counts recorded using the Modified California (MC) split spoon sampler were converted in the final logs to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 with inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts.

The boring logs with descriptions of the various materials encountered in each boring, pocket penetration test resistance values, and some of the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs are approximate (i.e., rounded to the nearest foot) and were estimated using Google Earth. Actual surface elevations at the boring locations could differ slightly. Boring locations were determined in the field by visual orientation in relation to both on-site and off-site physical landmarks. The locations and elevations of the borings should only be considered accurate to the degree implied by the means and methods used to define them.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and/or included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and D2937) – In-situ density and moisture tests were conducted to determine the in-place dry density and moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soil. Test results are presented on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) – Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, helps evaluate the expansive characteristics of the soil, and for determining the soil type according to the USCS. Several tests were performed, and the test results are presented in Section 4.1, in Appendix B, and on the applicable boring log.

Particle Size Analysis (Wet and Dry Sieve) and Fines Content (ASTM D422 and D1140) - Sieve analysis or fines content (minus No. 200 sieve) tests were conducted on several selected samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented on the boring logs or in Appendix B.

Unconfined Compressive Strength (ASTM D2166) – Unconfined compressive strength tests were run on several liner samples to obtain strength parameters for use in foundation and retaining wall design.

Soil Corrosivity, Redox (ASTM D1498), pH (ASTM D4972), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) – Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the CBC and IBC. Test results are presented and discussed in Section 4.3.

3.0 GEOLOGY AND SEISMICITY

3.1 Geologic Setting

The site is located within the central portion of the Coast Ranges geomorphic province of California. The Coast Ranges geomorphic province consists of numerous small to moderate linear mountain ranges trending north to south and northwest to southeast. The Coast Ranges lies between the Pacific Ocean to the west and the Great Valley Geomorphic Province to the east. This province is approximately 400 miles long and extends from the Klamath Mountains in the north to the Santa Ynez River within Santa Barbara County in the south. It generally consists of marine sedimentary rocks and volcanic rocks. The province is characterized by northwest-trending faults and folds, as well as erosion and deposition within the broad transform boundary between the North American and Pacific plates. Translational motion along the plate boundary occurs across a distributed zone of right-lateral shear expressed as a nearly 50-mile-wide zone of northwest-trending, near-vertical active strike-slip faults. This motion occurs primarily along the active San Andreas, Hayward, Calaveras and San Gregorio faults.

Locally, the site is located northwest of Mount Diablo, south of Susuin Bay, and east of the Briones Hills. The underlying sediment within the vicinity of the subject site consists of Holocene alluvium. Pleistocene alluvial gravel and sand is located toward the Briones Hills to the west. Eocene Meganos Formation, which consists of clay shale with thin sandstone, and Martinez Formation, which consists of marine clay shale/siltstone and sandstone is located approximately 1.5 miles west of the subject property (Dibblee, 2005). The beds of these formations are folded and generally northwest trending. Miocene Monterey Formation, consisting of folded sandstone and clay shale/siltstone with generally northeast trending beds, exists within 2.5 miles southwest. About three miles east of the subject site on the other side of the Concord fault is Eocene Kreyenhagen Formation consisting of shale. See *Figure 4 – Site Vicinity Geologic Map.*

3.2 Seismic Setting

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The Bay Area of Northern California is a seismically active region dominated by four major northwest trending right lateral strike slip faults that include the San Andreas Fault, the Hayward Fault, the Calaveras Fault, and the Greenville Fault.

Faults near the subject property include the Contra Costa (Southampton) fault located approximately 1.5 miles to the west, the Contra Costa (Larkey) fault located approximately 1.3 miles southwest, the Franklin Fault located approximately 2.2 miles southwest, and the Concord Fault located about 2.7 miles east.

According to the State of California Special Studies Zones map for Walnut Creek, which shows active faults that have a relatively high potential for surface rupture, the inferred location of the Concord Fault zone is about 2.4 miles east of the subject site, see *Figure 5 – Regional Fault Map*. Other major faults within the general region of the subject property include the Hayward Fault approximately 25 miles to the west and the Clayton Fault about 15 miles to the east.

4.0 FIELD AND LABORATORY FINDINGS

4.1 Subsurface Soil Conditions

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to a maximum depth of about 50 feet bgs in eight borings performed for this study. From the data collected we conclude that the proposed project site is generally underlain by approximately 0 to 3 feet of fill consisting of clayey silt, silty sand, and silts. Beneath the fill is stiff to hard silty clay, sandy clay, and sandy silt down to about 20 feet bgs. Fat clay was encountered at about 15 feet in borings six, seven, and eight. Silty clay, clayey silt, and fat clay are located from 20 feet to the maximum depth explored of 50 feet bgs.

Unconfined compressive strength tests were run on three samples. The unconfined compressive strength was 40.6 psi in B-3 at 2 feet bgs, 9.6 psi in B-5 at 4.5 feet bgs, and 15.8 psi in B-7 at 5.5 feet bgs, with corresponding undrained shear strengths of 2,925 psf, 1,141 psf and 692 psf, respectively.

Atterberg limits were analyzed in four samples. In B-1 at a depth of 4.5 feet bgs, the Liquid Limit (LL) was 40 and the corresponding Plasticity Index (PI) was 26; B-1 at 9.5 feet bgs had a LL of 48 and a PI of 34; B-5 at 2 feet bgs had a LL of 34 and a PI of 19; and B-8 at 9.5 feet bgs had a LL of 39 and a PI of 24. Based on these measurements, the near surface native clayey soil would be considered to have a moderate to high plasticity and a moderate to high expansion potential.

A subsurface profile below the proposed building site is presented in *Figure 6 – Schematic Geologic Cross Section A-A'*. Additional details of soils encountered in the exploratory borings are included in the boring logs provided in Appendix A.

4.2 Groundwater

Groundwater was encountered at 13 to 16 feet bgs in all borings except in B-4. Groundwater was not encountered in B-4, which was drilled to a depth of 20 feet bgs. For design purposes we are assuming a historic high groundwater table of 10 feet bgs. Note that the borings may not have been left open for a sufficient period to establish equilibrium groundwater conditions. Groundwater levels can also vary in response to adjacent water sources, time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage.

4.3 Corrosion Testing

A composite sample taken from the upper one to three feet, and a bulk sample collected from the upper one to three feet in boring B-2 were tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following table.

Table 1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Dark brown silty clay with sand (B-2)	1-3	77	11	490	1,036	Negative	6.8
Dark reddish brown sandy clay (composite)	1-3	218	16	502	682	Negative	7.9

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content in B-2 was measured to be about 77 mg/kg (ppm) or 0.0077% by dry weight in the soil sample, suggesting the soil within the vicinity of B-2 should have negligible impact on buried concrete structures at the site. The water-soluble sulfate content in the composite sample was measured to be about 218 mg/kg (ppm) or 0.0218% dry weight in the soil sample, suggesting the site soil should have a negligible impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06% by dry weight. The chloride content in B-2 was measured to be 11 mg/kg (ppm) or 0.0011% by dry weight in the soil sample. In addition, the chloride content in the composite sample was measured to be 16 mg/kg (ppm) or 0.0016% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105, and shown on Table 3.

Table 3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming fair site drainage, the sample for B-2 had a total score of 6 points, indicating a moderate corrosive rating. The composite sample had a total score of 10 due to the resistivity result of 682 ohm-cm, indicating a corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe, and use of cathodic corrosion protection is often recommended. The results from the composite sample indicate corrosive soils.

These results are preliminary, and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils are not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement (densification), fault ground rupture and fault creep, seismic slope failure, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Applicable potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site may experience strong ground shaking from a major earthquake originating from a number of significant faults in the general Bay Area. Earthquake intensities vary throughout the region depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site and other factors.

In addition to shaking of the structure, strong ground shaking can induce other related phenomena that may have an effect on structures, such as liquefaction and dynamic densification settlement.

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher inter-granular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances, generally when the liquefied layer is in relatively close proximity to an open, free slope face such as the bank of a creek channel. Lateral spreading can cause surficial ground tension cracking (i.e., lurch cracking) and settlement.

However, based on the stiff to very stiff clays under the site, we judge the potential for liquefaction settlement and resulting impact to the proposed development to be low.

5.1.3 Dynamic Densification (Settlement)

Dynamic densification or settlement is a process in which unsaturated, loose, relatively clean sands and silts are densified by the vibratory motion of a strong seismic event. No loose native sandy soils were encountered onsite during our exploration, and any loose surficial sand fills encountered during site grading are recommended to be removed and replaced as engineered fill. Therefore, we judge the potential for dynamic settlement at the site to occur to an extent to significantly impact the proposed development to be very low.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones or Earthquake Fault Zones surrounding faults or fault segments judged to be sufficiently active, well-defined and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site. The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). In our opinion, the potential for fault ground rupture and surface creep at the site is low.

5.2 Expansive Soils

Moderately expansive fine-grained soils were encountered in the upper five feet during our subsurface exploration. The results of the laboratory testing performed on a representative sample of the most expansive near-surface soils indicated a measured Plasticity Index of 26, indicative of a moderate plasticity and moderate expansion potential. It is our understanding that the surficial soil will be excavated for the basement level. Therefore, the impact of expansive soils on the project site will be minimal and special measures to mitigate the potential effects of expansive soils are not expected to be required for the main structures. However, ancillary structures and pavement areas may be subject to expansive pressures from these surficial soils and will require mitigation.

5.3 Consolidation Settlement

Consolidation occurs as a result of water being squeezed out from a saturated soil as internal pore water pressures induced by an external load are dissipated over time. As the water moves out from the soil, the solid particles re-align into a more-dense configuration with settlement resulting. Consolidation typically occurs as a result of new buildings or fills being placed over them, but consolidation can also occur from groundwater withdrawal. Consolidation of clayey soils is usually a long-term process, where-by the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load.

It is our understanding that a basement level is planned to be excavated. This will result in a zero-net load condition, where the weight of the building is offset by the weight of the soil excavated for the basement, and the underlying bearing soil will not be subject to a large increase in vertical effective stress. Additionally, the soils we encountered, very stiff sandy clay and hard lean clay, typically do not experience consolidation settlement. Therefore, consolidation settlement is expected to be negligible underneath the proposed new structure.

5.4 Flood Hazard

Based on our review of the Federal FIRM maps, the project site is located within an Unshaded Zone X Flood Hazard Zone, indicating a low risk area, outside the 0.2% annual chance floodplain. The project site is not within a Special Flood Hazard Area.

6.0 CONCLUSIONS AND ENGINEERING RECOMMENDATIONS

The following conclusions and engineering recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that could affect design or construction at this site are summarized below and addressed in the following sections.

Seismic Ground Shaking: The site is located within a seismically active region and expected to be subjected to moderately strong to very strong ground shaking during the life of the new structures. As a minimum, the building designs should consider the effects of seismic activity in accordance with the latest edition of the California Building Code (CBC).

Surface Fill – In general, our borings encountered a 0 to 3 foot thick layer of silty to sandy clay and sandy to clayey silt. This layer was stiff to very stiff and moist. It appears that this fill layer was placed with compaction effort and is most likely an engineered fill. However, due to the presence of an existing building at the site of the proposed new building, undocumented fills associated with the demolition of the building and removal of associated foundations and utilities may be present. Undocumented onsite fill soils if encountered in the new building pad and loose or debris laden soils if encountered in other areas, should be completely removed and replaced by engineered compacted fill. The portion of over-excavated material not consisting of debris or organic topsoil may be reused as fill material upon approval of the geotechnical engineer.

Basement Excavation and Groundwater – It is our understanding that a single story below grade basement level is planned to be excavated. The design ground water level can be assumed to 10 feet below existing grade, which may have an impact on the excavation and construction of the basement and underlying foundation. Appropriate dewatering measurements should be considered both during and after construction. In addition, the subgrade at the excavation level may be soft and pumping. We recommend stabilization of the basement level with a geogrid and 18-inch gravel layer below the building pad. The gravel layer can be incorporated into the dewatering system. Below grade structures should be properly waterproofed. As a minimum, all below grade cold-joints should be water stopped and penetrations sealed. Appropriate waterproofing membranes should be installed, and consideration should be given to utilization of Xypex as a water-proofing additive.

Winter Construction: If grading occurs in the winter rainy season, appropriate erosion control measures may be required and weatherproofing of the building pad and/or hardscape areas may need to be considered. Winter rains may also impact foundation excavations and underground utilities.

6.1 Seismic Coefficients: Site Specific Ground Motion Analysis

The proposed building should be designed in accordance with local design practice to resist the lateral forces generated by ground shaking associated with a major earthquake occurring within the greater Bay Area. Based on the subsurface conditions encountered in our borings and our evaluation of the geology of the site, Site Class "D", representative of stiff soils averaged over the uppermost 100 feet of the subsurface profile would be appropriate for this site. We estimated the shear wave velocity in the upper 100 feet of the subsurface profile using published data from the California Department of Conservation and the California Geologic Survey.

We performed a site-specific ground motion analysis for the subject site per ASCE 7-16 Chapter 21, as required by ASCE 7-16 Section 11.4.8 for structures on Site Class D sites with S_1 greater than or equal to 0.2. The procedures and relevant analysis data and seismic source summary data obtained from the site-specific ground motion analysis using the "EZ-FRISK 7.65" software are presented in Appendix C. Table 4 below contains the site-specific seismic ground motion parameters to be used for design based off 2019 CBC. If the project is designed under a different code than CBC 2019, we should be consulted to provide updated seismic parameter recommendations.

Table 4: Seismic Design Parameters Based on 2019 CBC (ASCE 7-16)

Parameter	Value
Site Class	D
Seismic Design Category Based on (for S_1>0.75 g)	
Occupancy Categories I, II & III	D
Occupancy Category IV	D
Mapped Spectral Response Accelerations	
Short Period, S_S	1.772 g
1-second Period, S_1	0.600 g
Adjusted Maximum Spectral Response Accelerations	
Short Period, S_{MS}	2.046 g
1-second Period, S_{M1}	1.515 g
Design Spectral Response Accelerations	
Short Period, S_{DS}	1.364 g
1-second Period, S_{D1}	0.010 g
Peak Ground Acceleration (PGA_M)	0.793 g

6.2 Site Grading

6.2.1 General Grading and Material Requirements

Site grading is generally anticipated to consist of finish grading to establish site grades, or additional mass grading for improved foundation bearing capacities if desired; utility trench excavation and backfills, preparation of supporting subgrades for site pavements and hardscape; and placement of aggregate base (baserock) sections for hardscape and pavements.

On-site soils having an organic content of less than three percent by weight can be reused as fill as approved by the Geotechnical Engineer. Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use on site.

6.2.2 Project Compaction Recommendations

Table 6 provides the recommended compaction requirements for this project. Some items listed below may not apply to this project. Specific moisture conditioning and relative compaction recommendations will be discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Percent Relative Compaction	Minimum Percent Above Optimum Moisture Content
Building Pad, Onsite Soil	90	3 to 5
Building Pad, Subgrade Soil	90	3 to 5
Building Pad, Imported Select Fill	90	2
Building Pad, Treated Soil	90	2
Structures, Upper 12" below footings	95	3 to 5
AC or Concrete Pavement, Subgrade, Upper 6"	95	3 to 5
AC or Concrete Pavement, Onsite Soil or Fill	90	3 to 5
AC or Concrete Pavement, Class 2 Baserock	95	2
AC or Concrete Pavement, Treated Soil, Subgrade	93	2
Concrete Flatwork, Class 2 Baserock	90	2
Concrete Flatwork, Subgrade Soil	90	3 to 5
Underground Utility Trench Backfill	90	2
Underground Utility Trench Backfill - Landscape Areas (not including areas below flatwork)	85	2
Underground Utility Trench Backfill, Clean Sand	95	4
Underground Utility Trench Backfill, Upper 3' Feet below Existing Pavement Sections or 6" below New Pavement Sections	95	2

Fill materials should be properly moisture conditioned in accordance with Table 6 as determined using ASTM D-1557 and placed in uniform loose lifts not to exceed eight inches. Smaller lifts may be necessary to achieve the minimum required compaction using lighter weight compaction equipment. It should be noted that the use of on-site soils for fill will require moisture conditioning (drying or wetting). Moisture conditioning may be difficult to achieve during cold, wet periods of the year, or during extreme temperatures and after precipitation events.

6.2.3 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geo-Eng prior to starting the stripping and demolition operations at the site.

The site should be cleared of existing pavements (if any), vegetation, organic topsoil, debris, existing undocumented loose or soft fill, and other deleterious materials within the proposed development area. Removed fill soil may be evaluated by the Geotechnical Engineer for possible reuse and placement as engineered fill. The grading contractor should be aware of the possibility of buried objects and underground utilities at the site which are to be removed or abandoned appropriately. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with properly compacted engineered fill or other material approved by the Geotechnical Engineer. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

It is possible that existing underground utilities exist and if so, may impact the project construction. If encountered, the utilities will need to be properly abandoned and/or entirely removed from proposed building area. In general, utility pipelines less than four inches in diameter to be abandoned may be left in place provided they will not be in close proximity to new foundation elements or interfere with new utilities. Such pipes should be plugged at the ends with concrete or sand-cement slurry. Larger utility pipelines or pipelines that underlie new foundations should be removed and replaced with engineered fill, or left in place and completely grouted with flowable sand-cement slurry or other approved Controlled Density Fill (CDF; also, known as Controlled Low Strength Material, or CLSM).

6.2.4 Building Subgrade Preparation

Following excavation to the required grades, subgrades in areas to receive engineered fill, slabs-on-grade or hardscape should be scarified to a depth of at least eight inches; moisture conditioned and compacted to the requirements for engineered fill presented in Table 5.

Due to the presence of relatively shallow ground water we recommend that the building pad at the bottom of the basement excavation be underlain by an 18-inch minimum thick layer of ¾-inch drain rock gravel, over a ground stabilization fabric such as Mirafi 500X or equivalent. This will aid in the constructability of the building pad and provide a stable working surface at the bottom of the excavation. The gravel layer should be vibrated following placement by a smooth drum vibratory roller. If the gravel layer is installed per recommendation, the scarification and re-compaction specification can be waived for the mat foundation.

The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. To achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. Fill material should be evenly spread and compacted in lifts not exceeding eight inches in pre-compacted thickness.

Unstable subgrades in smaller, isolated areas can be stabilized by over excavating to a minimum of 18-inch depth below finished subgrade elevation where competent, stable soils are not encountered. The bottom of the excavation should then be completely covered with a ground stabilization geotextile fabric such as Mirafi 500X or equivalent, and typically backfilled with Class 2 aggregate base. Alternatively, with the approval of the Geotechnical Engineer, such areas can be stabilized by over-excavating at least one foot, placing Tensar TriAx TX-140 or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock in either case should be compacted to at least 90 percent relative compaction.

Final grading should be designed to provide positive drainage away from the building. We suggest exposed soil/landscape areas, if any, within 10 feet of the proposed building be sloped at a minimum of three percent away from the building. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the building or into a closed pipe system channeled away from the building to an approved collector or outfall.

6.2.5 Flatwork Areas

Areas to receive concrete flatwork, excluding the basement foundation, should be scarified to a depth of eight inches below existing grade or final subgrade, whichever is lower. Scarified areas should be moisture conditioned and compacted per the recommendations presented in section 6.2.2. Once the compacted subgrade has been reached, it is recommended that baserock in paved areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until the baserock is placed. Rubber-tired heavy equipment, such as a full water truck, should be used to proof roll exposed pavement subgrade areas where pumping is suspected. Proof rolling will determine if the subgrade soil is capable of supporting construction paving equipment without excessive pumping or rutting.

6.2.6 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present, and compaction of on-site soils may not be feasible. These conditions may be remedied using appropriate soil admixtures, such as lime or other admixtures. More detailed recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or TriAx TX-140 geogrid or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 90 percent relative compaction. Alternatively, a non-woven stabilization geotextile such as Mirafi 500X overlain by a minimum 18 inches of baserock may be substituted for geogrid and baserock.

6.3 Utility Trench Construction

6.3.1 Trench Backfilling

Utility trenches may be backfilled with onsite soil or import soil pre-approved by the Geotechnical Engineer above the utility bedding and shading materials. If cobbles, rocks or concrete larger than four inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches.

Pipeline trenches should be backfilled with fill placed in lifts of approximately eight inches in pre-compacted thickness, and compacted to the requirements presented in Section 6.2.2. However, thicker lifts can be used, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved.

6.3.2 Utility Penetrations at Building Perimeter

Flexible connections at building perimeters should be considered for utility lines going through perimeter foundations. This would provide flexibility during a seismic event. This could be provided by special flexible connections, pipe sleeving with appropriate waterproofing, or other methods.

6.4 Temporary Excavation Slopes

Below-grade construction, if any is ultimately proposed for the project, may require temporary excavation slopes if more than a few feet below existing grade. The Contractor should incorporate all appropriate requirements of

OSHA/ Cal OSHA into the design of the temporary construction slopes and shoring system, whichever is used. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the on-site near-surface materials may be assumed to be moderately strong and cohesive materials, and categorized as OSHA Type B with temporary slope inclination of no steeper than 1:1 (horizontal: vertical) for excavations less than 20 feet deep. Excavations below groundwater will automatically be classified as Type C, with a required 1-1/2:1 inclination. This should be field verified by the contractor's responsible person. If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations.

We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

6.5 Temporary Shoring Recommendations

Based on our review of the planned development, it appears that shoring will be required along the eastern, southern and western portions of the property lines. For the soil conditions encountered, we believe that a soldier beam and lagging shoring system is the most appropriate system. Based on our understanding of the design concept, the excavation would extend approximately 10 feet below existing grade. The geotechnical design conditions for shoring are assumed to consist of very stiff cohesive soil with groundwater at approximately 12 feet. The lateral earth pressures as well as tieback design parameters are summarized in the attached Figure 7.

Our design recommendations assume that there will not be any surcharging of the adjacent buildings onto the temporary shoring. An additional surcharge load may be required should any adjacent foundation element be located within a 1:1 plane from the base of new excavation. We should review the shoring plans for this condition. We assume that there will be no underpinning of adjacent buildings required. Supplemental underpinning recommendations can be provided if needed. We do recommend that monitoring and a pre-construction survey of adjacent improvements be performed prior to excavation on the site. Monitoring should be installed by the project surveyor and as a minimum weekly survey reading of the adjacent structures should be performed.

Soldier piles should be placed in predrilled holes and a combination of structural concrete and lean concrete placed in the piles. The installation of the soldier beams should be observed by the Geotechnical Engineer of Record to confirm the design assumptions prior to excavation. During excavation, intermittent observation of the lagging operation should also be performed. We anticipate that primarily cohesive materials will be encountered and the potential for sloughing during the lagging operation is low. Care should be taken to avoid over-cutting the lagging excavation to maintain good positive contact with the lagging. Excessive voids should be grouted to maintain positive contact with lagging.

Tieback testing should consist of both performance testing and proof testing. Performance testing is a more rigorous cyclic loading test which is intended to evaluate the design assumptions as well as the quality of the materials. A minimum of 2 tiebacks should be tested at the beginning of the project under the longer and more rigorous performance testing. During production, the less rigorous proof loading of all tiebacks will be required.

6.5.1 Performance Tests:

A minimum of 2 tiebacks shall be tested in accordance with the procedures described below. The Geotechnical Engineer shall select the tiebacks to be performance tested. The remaining tiebacks shall be tested in accordance with the proof test procedures.

The performance test shall be made by incrementally loading and unloading the tieback in accordance with the schedule provided. The load shall be raised from one increment to another immediately after recording the tieback movement. The tieback movement shall be measured and recorded to the nearest 0.001 inch with respect to an independent fixed point at the alignment load and at each increment of load. The performance testing shall be done by apply load in accordance with the following schedule. The load should be applied long enough to take readings and then move to the next loading increment. The final load should be held for 10 minutes prior to lock off.

PERFORMANCE TEST SCHEDULE

Step	Load
1	AL
2	0.25DL
3	AL
4	0.25DL
5	0.50DL
6	AL
7	0.25DL
8	0.50DL
9	0.75DL
10	AL
11	0.25DL
12	0.50DL
13	0.75DL
14	1.00DL
15	AL
16	0.25DL
17	0.50DL
18	0.75DL
19	1.00DL
20	1.20DL
21	AL
22	0.25DL
23	0.50DL
24	0.75DL
25	1.00DL
26	1.20DL
27	1.33DL
28	HOLD FOR 10 MINUTES
29	AL
30	Adjust to lock-off load

The jack shall be adjusted as necessary in order to maintain a constant load. The load-hold period shall start as soon as the maximum test load is applied and the ground anchor movement, with respect to a fixed reference, shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. If the ground anchor movement between one (1) and ten (10) minutes exceeds 0.04 inch, the maximum test load shall be held for an additional 50 minutes. If the load-hold is extended, the ground anchor movement shall be recorded at 15 minutes, 20, 30, 40, 50 and 60 minutes.

6.5.2 Proof Tests:

The proof test shall be performed by incrementally loading the ground anchor in accordance with the following schedule. The load shall be raised from one increment to another immediately after recording the ground anchor movement. The ground anchor movement shall be measured and recorded to the nearest 0.001 inch with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with the primary pressure gauge. At load increments, other than the maximum test load, the load shall be held just load enough to obtain the movement reading.

PROOF TEST SCHEDULE

Step	Load
1	AL
2	0.25DL
3	0.50DL
4	0.75DL
5	1.00DL
6	1.20DL
7	1.33DL
8	Reduce to lock-off load
9	AL (optional)
10	Adjust to lock-off load

The maximum test load in a proof test shall be held for ten (10) minutes. The jack shall be adjusted as necessary in order to maintain a constant load. The load-hold period shall start as soon as the maximum test load is applied and the ground anchor movement with respect to a fixed reference shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6 and 10 minutes. If the ground anchor movement between one (1) and ten (10) minutes exceeds 0.04 inch, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the ground anchor movements shall be recorded at 15 minutes, 20, 30, 40, 50 and 60 minutes.

6.6 Foundation Recommendations

6.6.1 Structural Mat Foundation

The proposed building can be supported on a structural mat foundation. Where over excavations below design footing depth is required, the over excavated portion of footing excavation should be backfilled with structural or lean concrete or a Controlled Low Strength Material (CLSM). The structural mat slab should have a minimum thickness of 24 inches with a minimum embedment of 12 inches. Reinforcement and final slab thickness should be determined by the project Structural Engineer. A modulus of subgrade reaction of 100 pci can be used for the native soil on site. The mat should be supported on an 18-inch layer of gravel with an underlying subgrade stabilization fabric. Hydrostatic loading of the slab should be based on a groundwater elevation of 10-feet below the existing surface.

For the design of the structural mat slab bearing on native soil, we recommend an average allowable bearing pressure of 1,500 psf. A one-third increase in the allowable bearing pressure may be used for loads that include wind and seismic. The allowable pressures provided are net values, as the weight of the slab itself has already been accounted for and can be neglected as a load for design purposes.

6.6.2 Lateral Resistance

Shallow foundations can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an *ultimate* passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. The top foot of passive resistance at foundations not adjacent to pavement or hardscape should be neglected. To fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. The friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.6.3 Construction Considerations

Geo-Eng personnel should be retained to observe and confirm that footing excavations prior to formwork and reinforcing steel placement bear in soils suitable for the recommended maximum design bearing pressure. If unsuitable soil is encountered, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using engineered soil or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing material and facilitate removal of water and slough if rainwater fills the excavations.

6.7 Retaining/Basement Walls

6.7.1 Lateral Earth Pressures

The following recommended lateral earth design pressures are based on the assumption that on-site soils will be used as wall backfill. For a level backfill condition, unrestrained walls (i.e., walls that are free to deflect or rotate) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. Restrained walls for a level backfill condition should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot, plus an additional uniform lateral pressure of 5H pounds per square foot, where H = height of backfill above the top of the wall footing, in feet. For seismic design of walls greater than six feet in retained height, unrestrained and restrained walls with level backfill should be designed to resist an additional uniform load equal to 15H psf, added to the *unrestrained* condition in either case. A seismic increment is not required for site walls retaining less than six feet.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every two degrees of slope inclination from horizontal. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load for unrestrained walls, and 0.50 times the anticipated surcharge load for restrained walls.

Retaining wall foundation elements can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an *ultimate* passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of the footing perpendicular to the direction of loading where the footing is poured neat against undisturbed material (i.e., native soils or engineered fills). The top foot of passive resistance at foundations not adjacent to and confined by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied.

The lateral earth pressures herein do not include any factor-of-safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

6.7.2 Retaining Wall Foundations

Retaining and below-grade walls may be founded on spread footing foundations bearing on undisturbed stiff to very stiff, onsite native clay soil. Where over excavations below design footing depth is required, the over excavated portion of footing excavation should be backfilled with structural or lean concrete or a Controlled Low Strength Material (CLSM). Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 18 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

Footings should be designed for the following allowable bearing pressures, assuming design Factors-of-Safety of 3.0, 2.0 and 1.5 for dead loads, dead plus live loads and total loads, respectively, from the calculated ultimate bearing pressure.

Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	2,000
Dead plus Live Loads	3,000
Total Loads (including wind or seismic)	4,000

These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes. Footings should be designed with sufficient reinforcing to provide structural continuity and permit spanning of local irregularities. These pressures assume a uniform embedment into vert stiff native soil or engineered fill. Footings may need to be over-excavated during construction to achieve this requirement and all footings shall be observed by a Geo-Eng Engineer to confirm this.

6.7.3 Retaining Wall Drainage

The aforementioned recommended lateral pressures assume that walls are fully back drained to prevent the build-up of hydrostatic pressures. To reduce the potential for hydrostatic loading on retaining and below-grade walls due to possible seasonal subsurface groundwater seepage, a subsurface drain system may be considered for construction behind below-grade walls. Alternatively, below-grade walls can be designed to accommodate an additional hydrostatic pressure increment. Design groundwater elevation can be assumed to be 10 feet below existing ground surface.

The drain system should consist of free-draining granular soils containing less than five percent fines passing a No. 200 sieve, placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines, or else should be encapsulated in a suitable filter fabric. A drainage system consisting of perforated drain lines (minimum 4" diameter placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. Sub drains constructed to protect interior spaces should have the invert elevation of the sub drain a minimum of six-inches below the interior finished floor elevation. Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance. An impervious soil should be used in the upper one-foot layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure, such as geo-composite, may be used as a substitute for the granular backfill adjacent to the wall.

The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility. The foundation of the retaining wall should be protected and prevented from any erosion of the surroundings.

We recommend that an appropriate water proofing consultant be retained to consult on the design of the project. As a minimum, all below grade cold-joints should be water stopped and penetrations sealed. Appropriate waterproofing membranes should be installed, and consideration should be given to utilization of Xypex as a water-proofing additive.

6.7.4 Retaining Wall Backfill Compaction

Retaining wall backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than a depth of five feet should be compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced. Over compaction or surcharge from heavy equipment too close to the wall may cause excessive lateral earth pressures which could result in excessive outward wall movement.

6.8 Concrete Slabs-on-Grade

6.8.1 Interior Concrete Slabs

Non-structural concrete interior slab-on-grade floors should be a minimum of five inches in thickness. As a minimum, slab reinforcing should consist of No. 4 steel reinforcement spaced at 18-inch centers each way, and in any case, be sufficient to satisfy the anticipated use and loading of the slab. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

Care should be taken to maintain the minimum recommended moisture content in the subgrade until floor slabs and/or engineered fills are constructed. Positive drainage should also be developed away from the building to prevent water from ponding along the perimeter and affecting future floor slab performance. We recommend a positive cutoff in utility trenches at the structure/building lines to reduce the potential for water migrating through the utility trench backfill to areas under the building.

Slab-on-grade concrete floors with moisture sensitive floor coverings should be underlain by a moisture retarder system constructed between the slab and subgrade. Such a system could consist of four inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick “Stego Wrap Class A”), or to Class B (Griffolyn Type 85, Moistop Ultra B, or equivalent) may be used in place of a Class C retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if a Class A barrier is used beneath the floor slab, and a sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater. Sand and crushed rock layers may be considered to comprise part of the thickness of the recommended non-expansive fill underlying the interior slab.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer’s specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

6.8.2 Exterior Concrete Flatwork

Exterior concrete flatwork with pedestrian traffic should be at least four inches thick and should be underlain by at least six-inches of aggregate baserock. The subgrade beneath the flatwork should be moisture conditioned and compacted as specified in the grading section of this report.

Flatwork can be reinforced to reduce potential tripping hazards, but welded wire mesh should not be utilized. Where critical, the flatwork can be doweled into the building foundation adjacent to doorways and into curbs to prevent possible tripping hazards. We also recommend that control joints be designed and constructed in accordance with the American Concrete Institute (ACI) recommendations. In general, this would require control joints on a maximum spacing of approximately 10 feet by 10 feet, with a closer spacing depending on the shape of the concrete slab.

6.9 Plan Review

It is recommended that Geo-Eng be provided the opportunity to review the foundation, grading, and drainage plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and the incorporation of these recommendations into the project plans and specifications.

6.10 Observation and Testing During Construction

It is recommended that Geo-Eng be retained to provide observation and testing services during site preparation, site grading, utility construction, and foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

6.11 Validity of Report

This report is valid for three years after publication. If construction begins after this time period, Geo-Eng should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geo-Eng should be notified to determine if additional recommendations are required. Additionally, if Geo-Eng is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid since Geo-Eng's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the subsurface conditions revealed during construction. Geo-Eng's involvement should include foundation and grading plan review;

observation of foundation excavations; grading observation and testing; testing of utility trench backfills and retaining wall backfill as applicable to the project; and subgrade preparation in flatwork areas.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, Geo-Eng should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly, the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geo-Eng after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geo-Eng should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geo-Eng be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geo-Eng will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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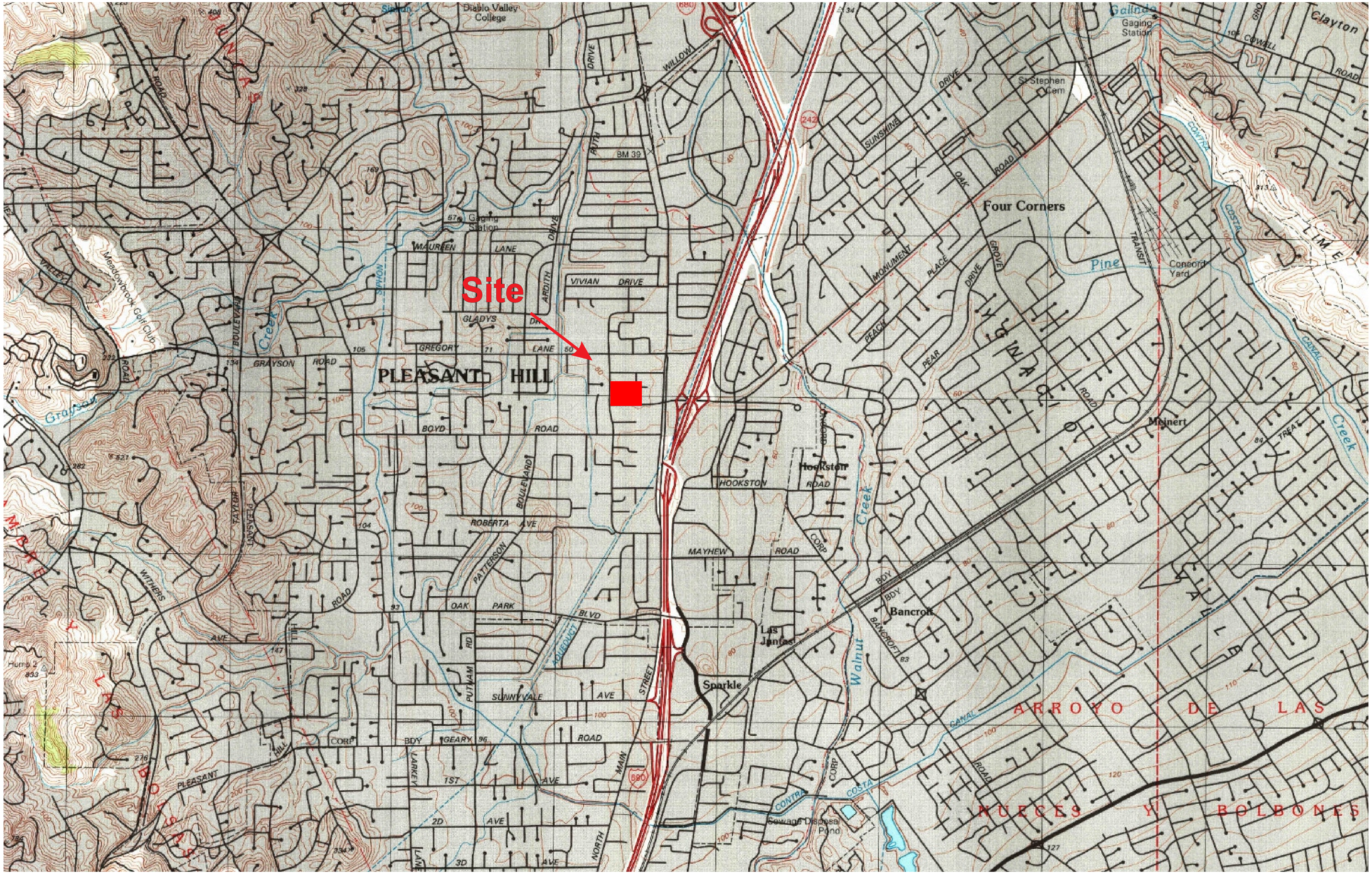
FIGURES



Map Scale

KILOMETERS 1 0.5 0 1 2

METERS 1000 500 0 1000 2000

MILES 1 0.5 0 1

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

Source: Walnut Creek Quadrangle, California, US Topographic Map 1:24,000, United States Geological Survey (1995)

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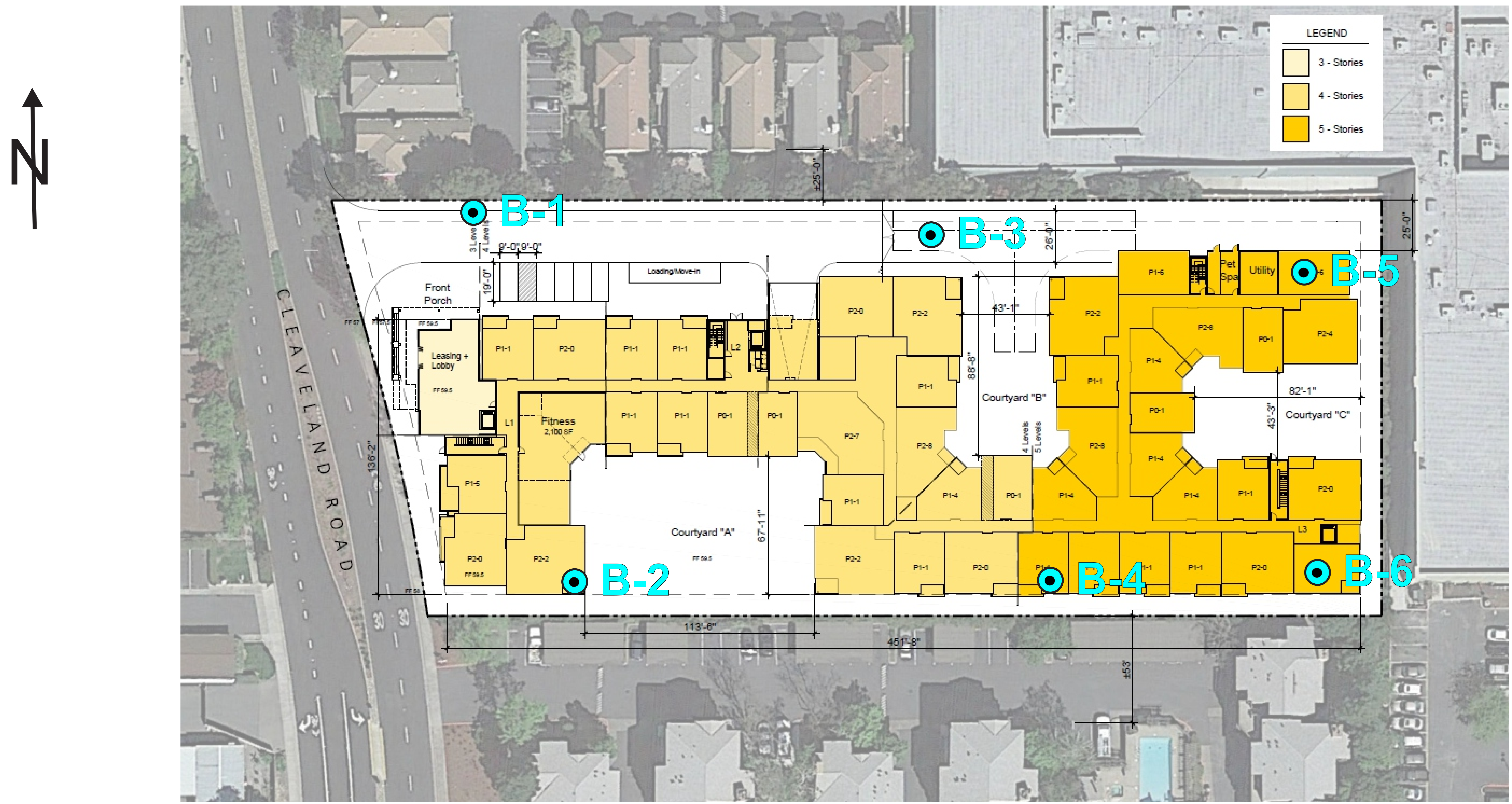
Residential and Retail Development Cleaveland Road and Crescent Plaza Pleasant Hill, California	06-1023	May 2019
	Site Vicinity Map	Figure 1



- Approximate Boring Location

Source: Revised Conceptual Site Plan, Cleaveland and Crescent, prepared by KTGY Architecture and Planning

GEO-ENGINEERING SOLUTIONS, INC. A PEI COMPANY Geotechnical Engineering • Engineering Geology • Materials Testing	Residential and Retail Development Cleaveland Road and Crescent Plaza Pleasant Hill, California	06-1023	May 2019
		Developmental Site Plan	Figure 2

Source: USGS w/ California Geologic Survey Scientific Investigations Map 2918 Geologic Map of the San Francisco Bay Region

Base Map Reference: Google Earth

Geologic Contacts

Geologic Faults

Qpa - Alluvium, Pleistocene

Qha- Alluvium, Holocene

- Approximate Boring Location

- Geologic Cross Section

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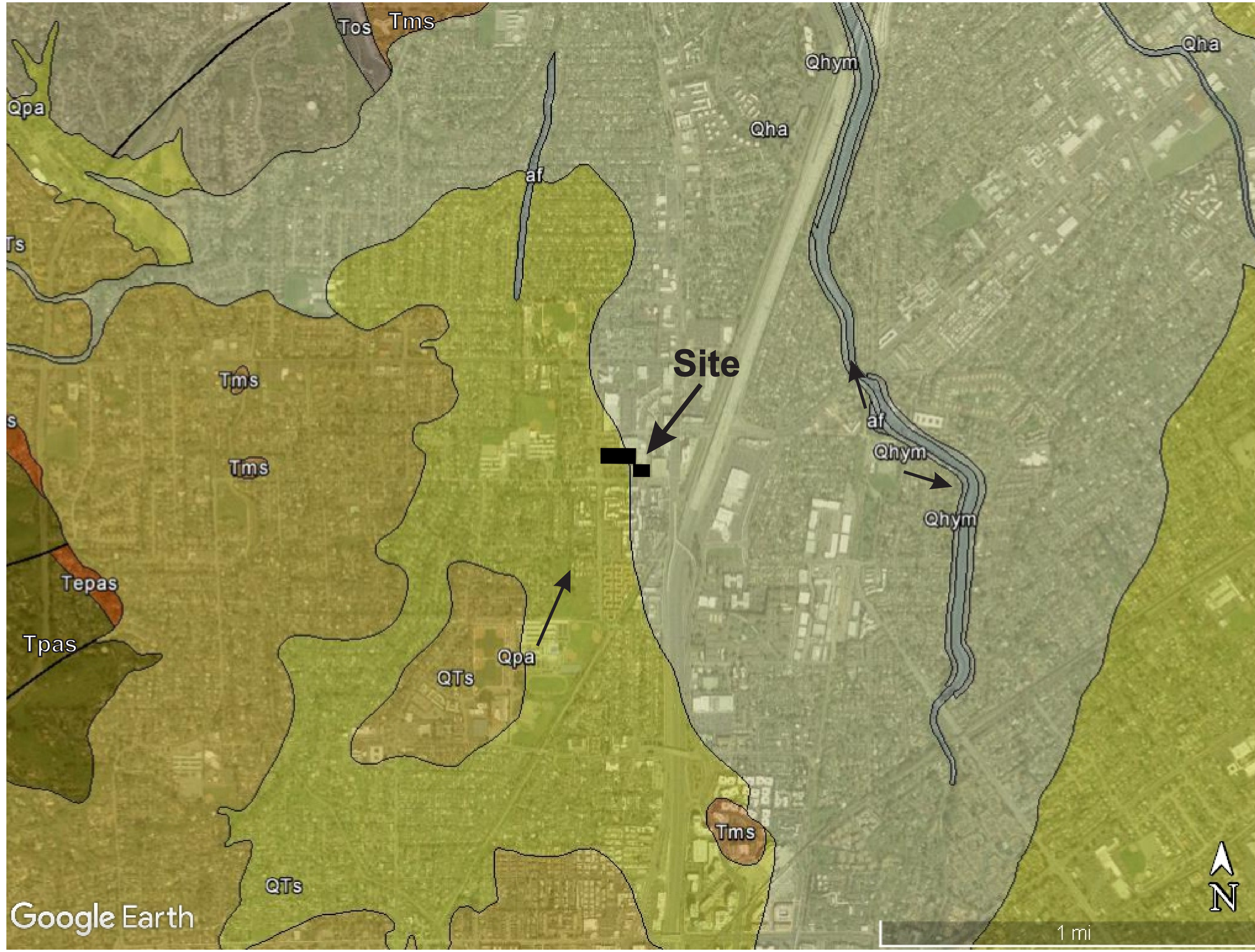
Residential and Retail Development Cleaveland Road and Crescent Plaza Pleasant Hill, California	06-1023	May 2019
	Site Plan and Site Geologic Map	Figure 3

Source: USGS w/ California Geologic Survey Scientific Investigations Map 2918 Geologic Map of the San Francisco Bay Region



af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qpa	Alluvium (Pleistocene)
Qha	Alluvium (Holocene)
QTs	Sediments (early Pleistocene and (or) Pliocene)
Tms	Sedimentary rocks (Miocene)
Tos	Sedimentary rocks (Oligocene)
Tepas	Sedimentary rocks (Eocene and (or) Paleocene)
Tpas	Sedimentary rocks (Paleocene)

Geologic Faults

Geologic Contacts

Base Map Reference: Google Earth

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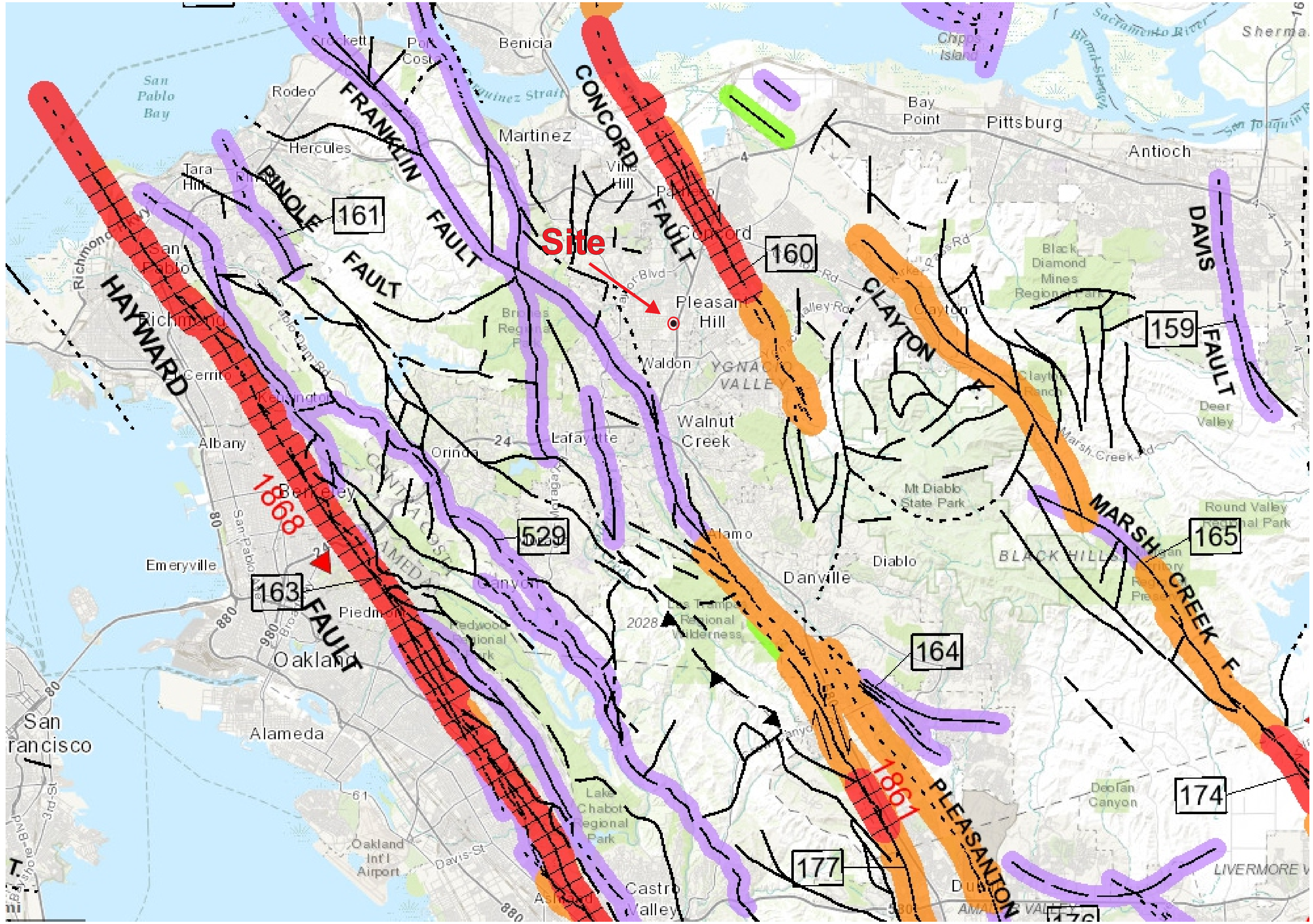
Residential and Retail Development Cleaveland Road and Crescent Plaza Pleasant Hill, California	06-1023	May 2019
	Site Vicinity Geologic Map	Figure 4

DESCRIPTION	
ON LAND	OFFSHORE
Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement
Quaternary	Late Quaternary	Historic	200		
		Holocene	11,700		
		Pleistocene	700,000		
	Early Quaternary		1,600,000		
Pre-Quaternary			4.5 billion (Age of Earth)		



Base Map Reference: California Geological Survey - 2010 Fault Activity Map of California

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Residential and Retail Development Cleaveland Road and Crescent Drive Pleasant Hill, California	06-1023	May 2019
	Regional Fault Map	Figure 5

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SUBSURFACE DIAGRAM

CLIENT Blake/Griggs Properties

PROJECT NUMBER 06-1023

PROJECT NAME Cleaveland and Crescent Development

PROJECT LOCATION Cleaveland Road and Crescent Plaza

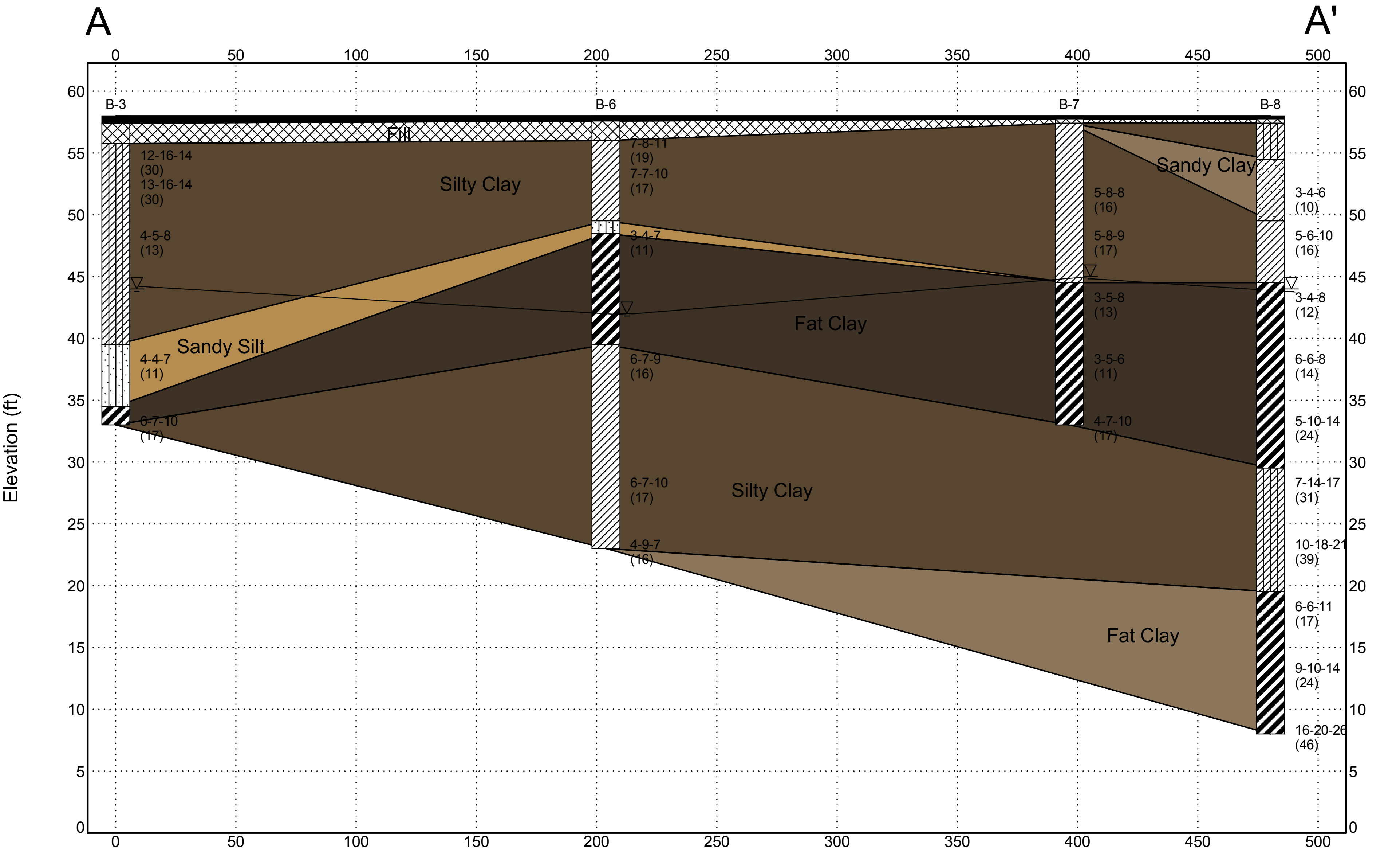


Figure 6 - Schematic Geologic Cross Section: A-A'

Geo-Engineering Solutions, Inc.

SUBSURFACE DIAGRAM

CLIENT Blake/Griggs Properties

PROJECT NUMBER 06-1023

PROJECT NAME Cleaveland and Crescent Development

PROJECT LOCATION Cleaveland Road and Crescent Plaza

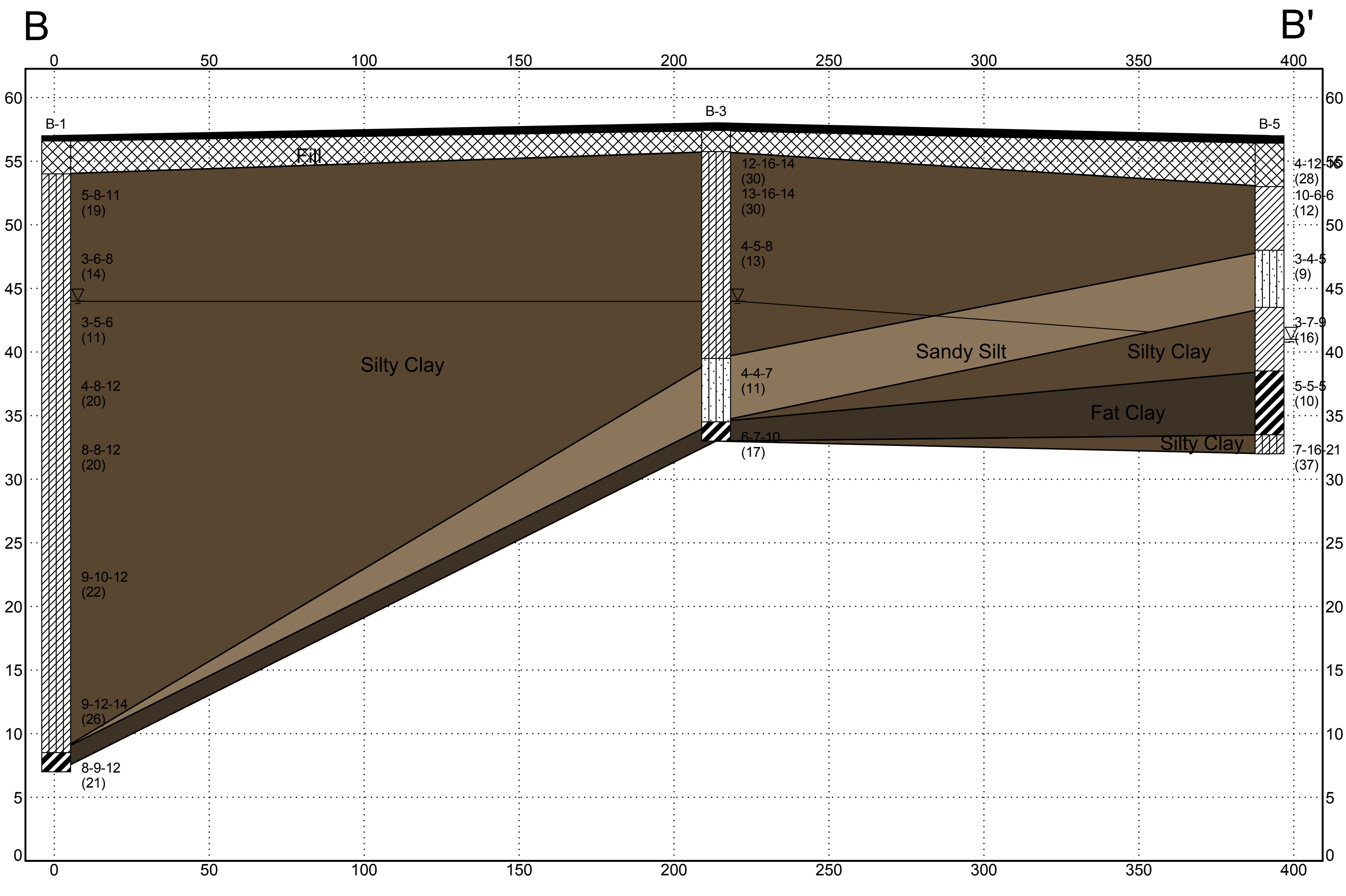
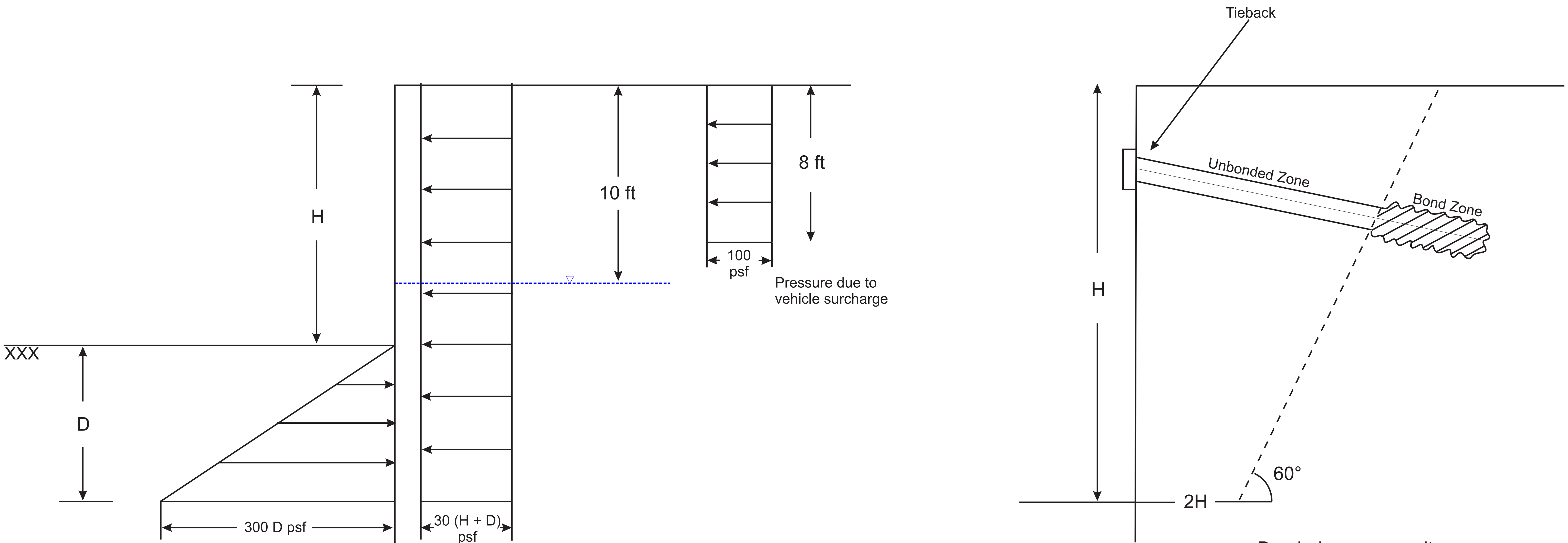


Figure 6 - Schematic Geologic Cross Section: B-B'



Bonded zone capacity

Gravity Grouted = 1,000 psf

Pressure Grouted = 2,500 psf

Notes: 1) Passive pressure taken over 3 pier diameters
2) Assume hyrdostatic load from groundwater at 10 feet below existing ground surface elevation
3) All shoring to be observed by Geotechnical Engineer of Record
4) All lagging to be placed in positive contact with cut faces.
ALL EXCESSIVE VOIDS shall be grouted as directed by the Geotechnical Engineer.

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Residential and Retail Development Cleaveland Road and Crescent Plaza Pleasant Hill, California	06-1023	May 2019
	Soldier Beam, Lagging and Shoring Recommendations	Figure 7

APPENDIX A

Key to Exploratory Boring Logs
Boring Logs

Unified Soil Classification (USC) System (from ASTM D 2487)

Major Divisions				Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)

SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

Particle Sizes

Components		*Size or Sieve Number*
Boulders		Over 12 inches
Cobbles		3 to 12 inches
Gravels	Coarse	3/4 to 3 inches
	Fine	Number 4 to 3/4 inch
Sand	Coarse	Number 10 to Number 4
	Medium	Number 40 to Number 10
	Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

Soil Moisture

Descriptor	*Description*
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Standard Proctor Optimum
Saturated	Free Water in Sample

Bulk Sample

Standard Penetration Test

2.5 Inch Modified California Sampler

Shelby Tube

First Water Level Reading

Final Water Level Reading

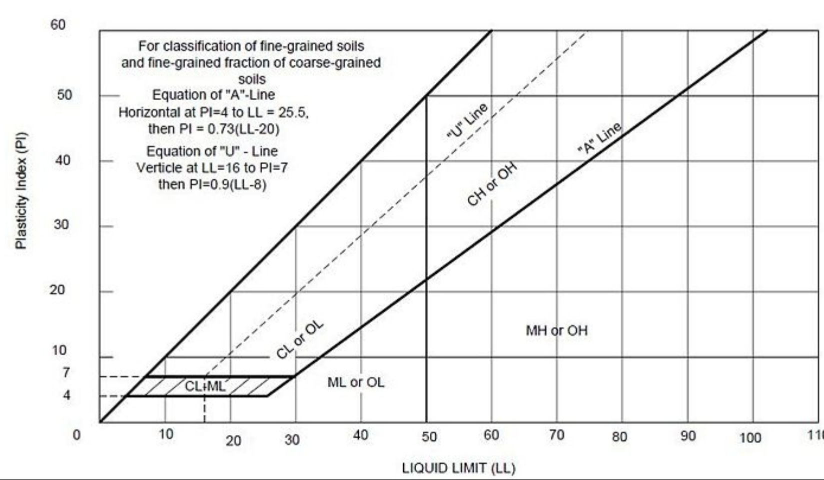
Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5” indicates 50 blows recorded for 5 inches of penetration. Note all of the field blow counts recorded using a Modified California sampler were converted to equivalent SPT blow counts.

N-Value
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 innch I.D.) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test).

General Notes:
1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source identified in the report. The location and elevation of borings should be considered accurate only to the degree implied by the method.

2.The stratification lines represent the approximate boundary between soil types. The transition may be gradual.

3. Water level readings in the drill holes were recorded at the time and under the conditions stated on the boring logs. It should be noted that flucuations in the level of groundwater may occur due to variations in rainfall, tides and other factors at the time measurements were made



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Key to Exploratory Boring Logs

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San Ramon, CA 94583
Telephone: 925433450

BORING NUMBER B-1

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023
PROJECT LOCATION Cleaveland Road and Crescent Plaza
DATE STARTED 9/28/17 **COMPLETED** 9/28/17
GROUND ELEVATION 57 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING 13.00 ft / Elev 44.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**5" ASPHALT CONCRETE**										
		(ML) **CLAYEY SILT (FILL)** : Light reddish brown, moist, stiff to very stiff										
		(CL) **SILTY CLAY** : With fine sand, dark brown, moist, very stiff										
5			MC 1-1		5-8-11 (19)	3.0	99	22	40	14	26	
10		With few fine sand, moist, stiff	MC 1-2		3-6-8 (14)	1.3	94	28	48	14	34	
	▽											
15		With fine sand, very moist to wet, stiff	MC 1-3		3-5-6 (11)	0.50						
20		Very stiff	MC 1-4		4-8-12 (20)	0.25						
25		With silt and small thin shell fragments, very stiff	SPT 1-5		8-8-12 (20)							

(Continued Next Page)

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BORING NUMBER B-1

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023
PROJECT LOCATION Cleaveland Road and Crescent Plaza

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
25		(CL) **SILTY CLAY** : With fine sand, dark brown, moist, very stiff *(continued)*										
30												
35		With fine sand, moist, very stiff	SPT 1-6		9-10-12 (22)							
40												
45		With light brown mottling	SPT 1-7		9-12-14 (26)							
50		(CH) **FAT CLAY** : With fine sand and silt, light brown with light brown to white mottling, moist, very stiff, plastic	SPT 1-8		8-9-12 (21)							

Bottom of borehole at 50.0 feet.

GEOTECH BH COLUMNS - GINT STD US LAB.GDT - 10/26/17 09:14 - Z:\SHARED\GEO-ENGINEERING SOLUTIONS\ACTIVE PROJECTS\06-1023 CLEAVELAND AND CRESCENT RESIDENTIAL AND RETAIL DEVELOPMENT\BORING LO

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Telephone: 925433450

BORING NUMBER B-2

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE STARTED** 9/29/17 **COMPLETED** 9/29/17
PROJECT LOCATION Cleaveland Road and Crescent Plaza
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING 13.00 ft / Elev 45.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**7" ASPHLAT**										
		(ML) **CLAYEY SILT (FILL)** : Light reddish brown, moist										
		(CL) **SILTY CLAY** : Dark brown with trace sand, dry, hard, non plastic	MC 2-1		14-13-18 (31)	>4.5						
5												
10												
15		(SM) **SILTY SAND** : Dark brown, fine to coarse in size, with subround to subangular gravels up to 3/4 inch, wet, medium dense	MC 2-2		7-7-7 (14)		102	25				59
		(CL) **SILTY CLAY** : Dark brown, with sand, moist										

Bottom of borehole at 16.5 feet.

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BORING NUMBER B-3

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE STARTED** 9/29/17 **COMPLETED** 9/29/17
PROJECT LOCATION Cleaveland Road and Crescent Plaza
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
▽ **AT TIME OF DRILLING** 14.00 ft / Elev 44.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**7" ASPHALT**										
		(CL) **SANDY CLAY (FILL)** : With fine gravels, light yellowish/orange brown with grayish brown mottling, moist, hard										
		Undrained Shear Strength = 2,925 psf	MC 3-1		12-16-14 (30)	>4.5						
		(CL) **SILTY CLAY** : Dark brown, moist, hard										
		With fine sand, moist	MC 3-2		13-16-14 (30)	>4.5	99	15				
5												
		With trace fine sand, moist to very moist, stiff	MC 3-3		4-5-8 (13)	2.3	102	25				
10												
	▽											
15												
		(ML) **SANDY SILT** : Yellowish light brown, very moist, stiff	MC 3-4		4-4-7 (11)		102	23				60
20												
		(CH) **FAT CLAY** : With fine sand and silt, dark brown, moist, very stiff, plastic	SPT 3-5		6-7-10 (17)							
25												

Bottom of borehole at 25.0 feet.

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BORING NUMBER B-4

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE STARTED** 9/29/17 **COMPLETED** 9/29/17
PROJECT LOCATION Cleaveland Road and Crescent Plaza
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	**4.5" ASPHALT**										
	(CL) **SANDY CLAY (FILL)** : With gravel, light brown										
	(CL) **SILTY CLAY** : Dark brown with orange/red brown mottling, moist, stiff	MC 4-1		8-7-8 (15)	2.0						
	Very stiff	MC 4-2		8-13-16 (29)	3.3	102	20				
5											
	Coarsening downward										
	With fine sand, moist, very stiff	MC 4-3		3-6-10 (16)	2.0						
10											
	(CL-ML) **SILTY CLAY** : With fine sand, dark brown, moist, medium stiff	SPT 4-4		4-2-5 (7)							
15											
	(CH) **FAT CLAY** : With fine sand and silt, light brown, moist, very stiff, plastic	MC 4-5		4-10-14 (24)							
20											

Bottom of borehole at 20.0 feet.

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BORING NUMBER B-5

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE**
PROJECT LOCATION Cleaveland Road and Crescent Plaza
STARTED 9/29/17 **COMPLETED** 9/29/17
GROUND ELEVATION 57 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING 16.00 ft / Elev 41.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**7" ASPHALT**										
		(CL) **SANDY CLAY (FILL)** : Fine to medium grained sand, light greenish gray and reddish gray, moist, very stiff	MC 5-1		4-12-16 (28)		106	17	34	15	19	
		Silty fine sand, light yellow										
5		(CL) **SILTY CLAY** : Dark brown, moist, stiff Undrained Shear Strength = 1,141 psf	MC 5-2		10-6-6 (12)	2.0						
		Dark brown, moist, stiff										
10		(ML) **SANDY SILT** : With clay, yellowish brown, moist, stiff	MC 5-3		3-4-5 (9)	1.8						
15		(CL) **SILTY CLAY** : Dark brown, very moist, very stiff	MC 5-4		3-7-9 (16)	1.3	92	30				
20		(CH) **FAT CLAY** : With silt, dark brown, moist, stiff, plastic	SPT 5-5		5-5-5 (10)							
25		(CL-ML) **SILTY CLAY** : With fine sand, dark brown, moist, hard	SPT 5-6		7-16-21 (37)							

Bottom of borehole at 25.0 feet.

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Telephone: 925433450

BORING NUMBER B-6

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE**
PROJECT LOCATION Cleaveland Road and Crescent Plaza
STARTED 9/29/17 **COMPLETED** 9/29/17
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
∇ **AT TIME OF DRILLING** 16.00 ft / Elev 42.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	**5" ASPHALT**										
	(ML) **SILT** : With sand and clay, light red-orangeish brown, moist, very stiff	MC 6-1		7-8-11 (19)		100	17				
	(CL) **SILTY CLAY** : Dark brown, very stiff, moist										
5		MC 6-2		7-7-10 (17)							
	(ML) **SANDY SILT** : With clay, light brown, stiff	MC 6-3		3-4-7 (11)	1.3						
10	(CH) **FAT CLAY** : With silt, dark brown, stiff, moist, plastic										
15	∇										
	(CL) **SILTY CLAY** : Dark brown, very stiff, moist	SPT 6-4		6-7-9 (16)							
20											
25											

(Continued Next Page)

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San Ramon, CA 94583
Telephone: 925433450

BORING NUMBER B-6

CLIENT Gemdale 85 Cleaveland Road LLC

PROJECT NAME Cleaveland and Crescent Development

PROJECT NUMBER 06-1023

PROJECT LOCATION Cleaveland Road and Crescent Plaza

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
25		(CL) **SILTY CLAY** : Dark brown, very stiff, moist *(continued)*										
30			SPT 6-5		6-7-10 (17)							
35			SPT 6-6		4-9-7 (16)							

Bottom of borehole at 35.0 feet.

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BORING NUMBER B-7

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE**
PROJECT LOCATION Cleaveland Road and Crescent Plaza
STARTED 9/29/17 **COMPLETED** 9/29/17
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING 13.00 ft / Elev 45.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**3" ASPHALT**										
		4" BASEROCK										
		(CL) **SILTY CLAY** : With trace sand, dark brown, moist	GB 7-1									
		Light brown, moist	GB 7-2									
5		With silt, brown, moist, very stiff Undrained Shear Strength = 692 psf	MC 7-3		5-8-8 (16)	1.8						
10		With trace fine gravels, dark brown, moist, very stiff	MC 7-4		5-8-9 (17)	2.5	97	27				
	▽											
15		(CH) **FAT CLAY** : With trace fine to medium grained sand and silt, dark brown, wet, stiff, plastic	MC 7-5		3-5-8 (13)	0.50						
20		Dark brown, moist, stiff, plastic	SPT 7-6		3-5-6 (11)							
25		Very stiff	SPT 7-7		4-7-10 (17)							

Bottom of borehole at 25.0 feet.

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San Ramon, CA 94583
Telephone: 925433450

BORING NUMBER B-8

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023 **DATE**
PROJECT LOCATION Cleaveland Road and Crescent Plaza
STARTED 9/28/17 **COMPLETED** 9/28/17
GROUND ELEVATION 58 ft **HOLE SIZE** 8" inches
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
▽ **AT TIME OF DRILLING** 14.00 ft / Elev 44.00 ft
LOGGED BY EP **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**3" ASPHALT**										
		4" BASEROCK										
		(CL) **SILTY CLAY** : Dark brown, moist, stiff										
		(CL) **SANDY CLAY** : Light brown, moist, stiff	GB									
5			MC		3-4-6 (10)	1.5						
		(CL) **SILTY CLAY** : Light brown, moist, very stiff	MC		5-6-10 (16)	1.3	97	26	39	15	24	
10												
		▽ (CH) **FAT CLAY** : With silt, light brown, moist, stiff	MC		3-4-8 (12)	1.5						
15												
			MC		6-6-8 (14)	1.0						
20		Light brown, wet, stiff										
		Very stiff	MC		5-10-14 (24)	1.0						
25												

(Continued Next Page)

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BORING NUMBER B-8

CLIENT Gemdale 85 Cleaveland Road LLC
PROJECT NAME Cleaveland and Crescent Development
PROJECT NUMBER 06-1023
PROJECT LOCATION Cleaveland Road and Crescent Plaza

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
25		(CH) **FAT CLAY** : With silt, light brown, moist, stiff *(continued)* With trace fine sand, hard										
30		(CL) **SILTY CLAY** : Dark brown, moist, hard	MC		7-14-17 (31)	2.0						
35			MC		10-18-21 (39)	2.3						
40		(CH) **FAT CLAY** : With silt, dark brown, wet, very stiff, plastic	SPT		6-6-11 (17)							
45		With fine sand at 43.5' to 44.5', light brown, moist, very stiff, plastic	SPT		9-10-14 (24)							
50		Light brown, wet, hard, plastic	SPT		16-20-26 (46)							

Bottom of borehole at 50.0 feet.

APPENDIX B

LABORATORY TEST RESULTS
Liquid and Plastic Limits Test
Particle Size Distribution Report
Unconfined Compressive Strength Results

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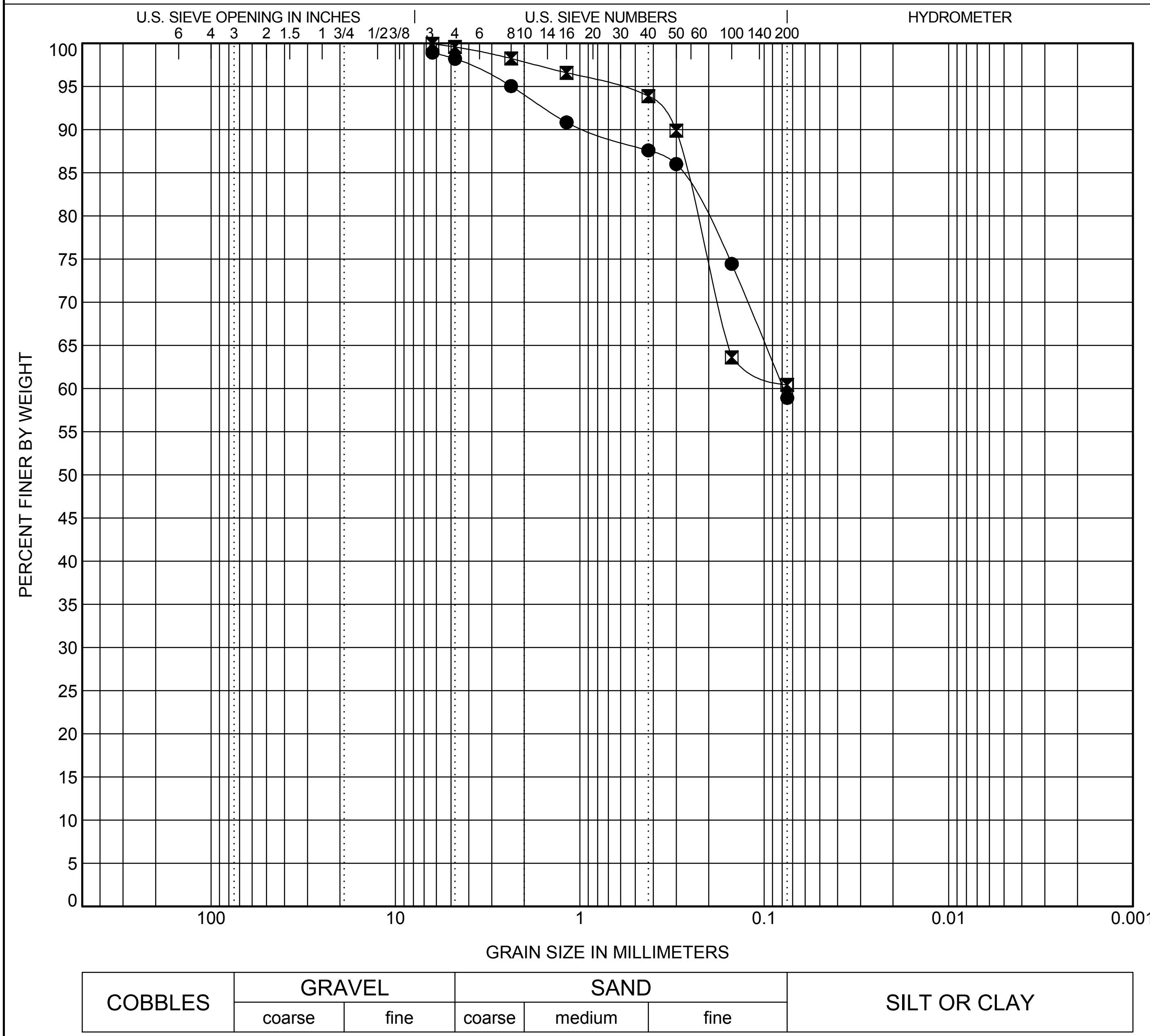
San Ramon, CA 94583
Telephone: 925433450

GRAIN SIZE DISTRIBUTION

CLIENT Gemdale 85 Cleaveland Road LLC **PROJECT NAME** Cleaveland and Crescent Development

PROJECT NUMBER 06-1023 **PROJECT LOCATION** Cleaveland Road and Crescent Plaza



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-2**	**16.0'**	**SILTY SAND (SM)**					
⊠	**B-3**	**19.5'**	**SANDY SILT (ML)**					

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2**	**16.0'**	**6.3**	**0.079**			**0.7**	**39.3**	**58.9**	
⊠	**B-3**	**19.5'**	**6.3**				**0.4**	**39.2**	**60.4**	

ATTERBERG LIMITS - GINT STD US LAB.GDT - 10/26/17 11:53 - Z:\SHARED\GEO-ENGINEERING SOLUTIONS\ACTIVE PROJECTS\06 BLAKE GRIGGS PROPERTIES\06-1023 CLEAVELAND AND CRESCENT RESIDENTIAL AND RETAIL DEVELOPMENT\BORING LOGS\G

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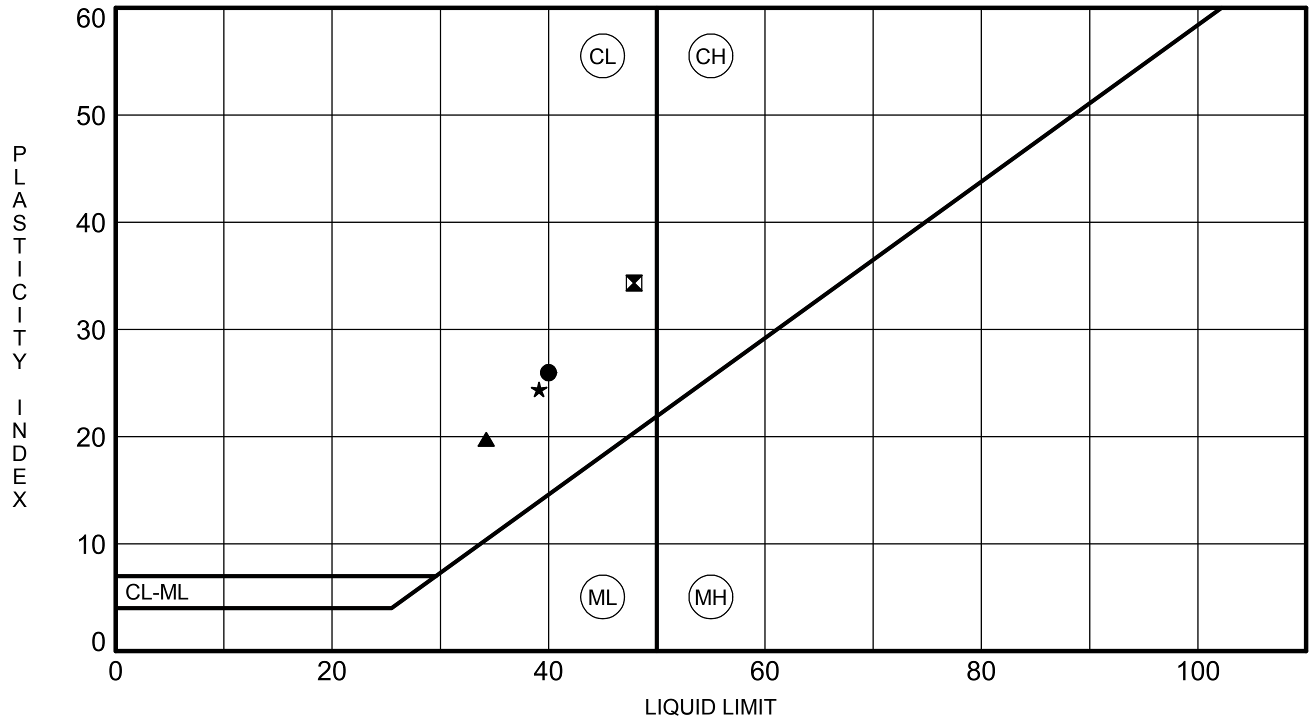
ATTERBERG LIMITS' RESULTS

CLIENT Gemdale 85 Cleaveland Road LLC

PROJECT NUMBER 06-1023

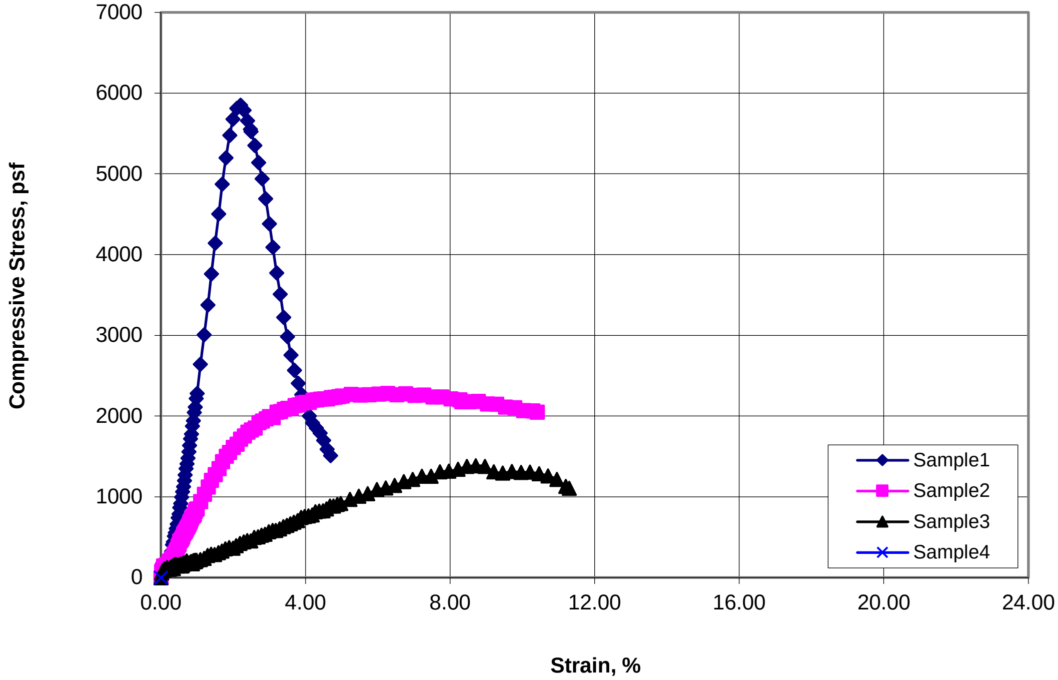
PROJECT NAME Cleaveland and Crescent Development

PROJECT LOCATION Cleaveland Road and Crescent Plaza



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**4.5'**	**40**	**14**	**26**		**SILTY CLAY (CL)**
⧗	**B-1**	**9.5'**	**48**	**14**	**34**		**SILTY CLAY (CL)**
▲	**B-5**	**2.0'**	**34**	**15**	**19**		**SANDY CLAY (CL)**
★	**B-8**	**9.5'**	**39**	**15**	**24**		**SILTY CLAY (CL)**

Unconfined Compressive Strength
ASTM D2166



Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	5851	2281	1384		
Unconfined Compressive Strength, psi	40.6	15.8	9.6		
Undrained Shear Strength, psf	2925	1141	692		
Failure Strain, %	2.2	6.3	8.7		
Strain Rate, % per minute	1.0	1.0	1.0		
Strain Rate, inches/minute	0.05	0.05	0.05		
Moisture Content, %	20.4	23.4	28.2		
Dry Density, pcf	97.4	102.8	95.2		
Saturation, %	75.6	98.7	98.7		
Void Ratio	0.730	0.640	0.771		
Specimen Diameter, inches	2.395	2.382	2.390		
Specimen Height, inches	5.09	4.96	4.97		
Height to Diameter Ratio	2.1	2.1	2.1		
Assumed Specific Gravity	2.70	2.70	2.70		

Sample Location				
	Boring	Sample	Depth, ft.	Soil Description
1	B3		2	Olive Brown Clayey SAND grading to Dark Olive Brown CLAY w/ Sand
2	B7		5.5	Olive Brown Clayey SAND
3	B5		4.5	Dark Olive Brown CLAY w/ Sand
4				

Job No.:	1003-008	
Client:	Geo-Engineering Solutions	
Project:	06-1023	
Date:	10/12/2017	By: MD/RU

Type of Sample: Undisturbed

Remarks:

COOPER TESTING LABORATORY

APPENDIX C

SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

Procedure and Summary of Results

Site Specific Seismic Ground Motion Analysis

The site is located within a seismically active region and should be designed to account for earthquake ground motions as described in this report. The SDC for the proposed development and subject site is D based on CBC 1613.3.5. A site-specific ground motion analysis was completed for the subject site per ASCE 7-16 Chapter 21. The procedure and assumptions for the site-specific ground motion analysis are summarized in the following:

Probabilistic (MCE_R) Ground Motions

A site-specific ground motion analysis in accordance to 2019 CBC and ASCE 7-16, Section 21.2, was performed for the site. The site specific probabilistic ground motion response spectrum is the product of the risk coefficient (C_R) and the spectral response acceleration from a 5% damped acceleration response spectrum for a 2% exceedance in 50 years (return period of 2475 years) and was performed as specified in ASCE 7-16, Section 21.2.1.1, Method 1 using EZ-FRISK 7.65 Build 004. The value of C_R was determined using values of C_{RS} and C_{R1} from Figures 22-18 and 22-19. The attenuation relationships of Abrahamson-et al (2008) NGA, Boore-et al (2008) NGA, and Campbell-Bozorgnia (2008) NGA were used, and the mean spectral acceleration for each period was taken. The results are presented below.

Deterministic (MCE_R) Ground Motions

A site-specific deterministic analysis was performed for all known influential seismic sources (within 100 km) in the region (as per ASCE 7-16, Section 21.2.2), using EZ-FRISK 7.65 Build 004. The attenuation relationships of Abrahamson-et al (2008) NGA, Boore-et al (2008) NGA, and Campbell-Bozorgnia (2008) NGA were utilized, and the mean acceleration for each period was taken for each of the faults analyzed. The highest acceleration for each period, comparing the different faults, was used and compared to the deterministic lower limit as shown in Figure 21.2.1 (ASCE 7-16). The results are presented below.

Site-Specific MCE_R

The site-specific Risk Targeted Maximum Considered Earthquake ground motion was then determined as per ASCE 7-16, Section 21.2.3 by taking the lower of the spectral accelerations taken from the probabilistic and deterministic analysis performed per ASCE 7-16, Sections 21.2.1 and 21.2.2. The results are presented below.

Design Response Spectrum

The design response spectral acceleration was calculated per ASCE 7-16, Section 21.3 and compared to the design response spectrum from ASCE 7-16, Section 11.4.6 to verify that the values from the site-specific analysis meet the requirement of not less than 80% of the accelerations obtained from section 11.4.6. If the values were less than the 80% requirement, they were then raised to the 80% value to obtain the final Design Response Spectrum S_a (g). The results are presented below.

The site classification and seismic coefficients are presented in Chapter 6 of this report. The adjusted maximum spectral response accelerations and designed spectral response accelerations values were determined from the site-specific analysis as per ASCE 7-16, Section 21.4 and were confirmed that the values are not less than 80% of the values obtained from, ASCE 7-16, Section 11.4.6 The results are presented below.

Design Acceleration Parameters

The design acceleration parameters were calculated per ASCE 7-16, Section 21.4 and the values were compared to verify that the values meet the requirement of not less than 80% of the values determined in accordance with Section 11.4.5. The results are presented below.

Peak Ground Acceleration (PGA)

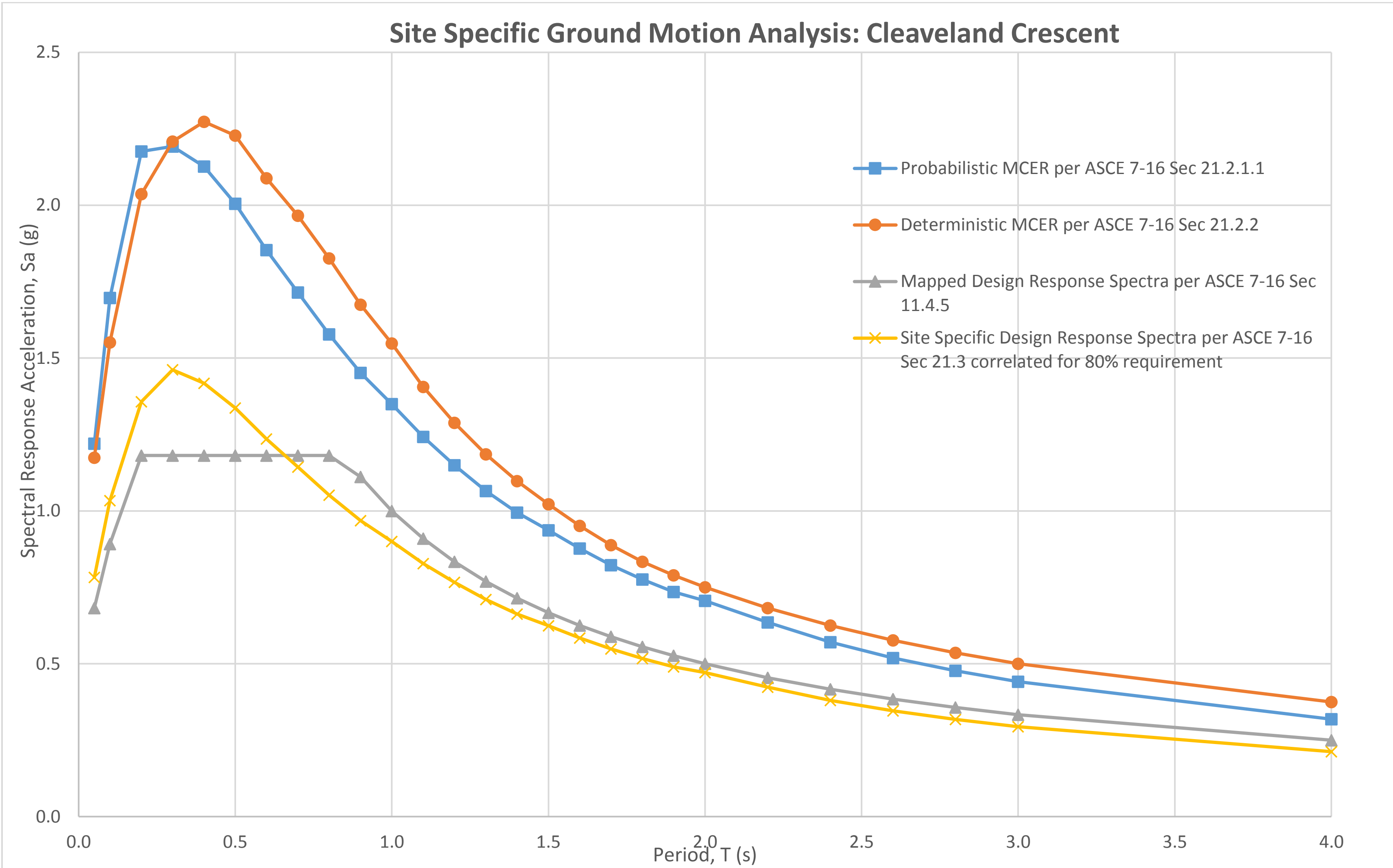
Peak Ground Acceleration (PGA) was determined per ASCE 7-16, Section 21.5 by taking the lower of the PGA determined by the probabilistic ground motions, and deterministic ground motions, not less than 80% of PGA_M determined from ASCE 7-16, equation 11.8.1. The results are presented below.

Cleaveland and Crescent – Seismic Parameter Summary Table

Parameter	Value per ASCE 7-16 Chap 11	Value per Site Specific Analysis ASCE 7-16 Chap 21
S_S (g)	1.772	-
S_1 (g)	0.6	-
S_{MS} (g)	1.772	2.036
S_{M1} (g)	1.5	1.412
S_{DS} (g)	1.181	1.357
S_{D1} (g)	1	0.941
PGA_M (g)	0.793	1.013
C_{RS}	0.917	-
C_{R1}	0.904	-
F_a	1	-
F_V	2.5	-
F_{PGA}	1.1	-
T_0 (s)	0.17	0.14
T_S (s)	0.85	0.69
T_L (s)	8	-

Cleaveland and Crescent – Site Specific Ground Motion Analysis Data Table

Period (s)	5% Damped Acceleration Response for 2% in 50 yrs (g)	Probabilistic MCER Sa (g)	Deterministic 84th-percentile 5% Damped Spectral Response Acceleration (g)	Deterministic Lower Limit on MCER Response Spectrum (g)	Deterministic MCER Sa (g)	Site Specific MCER Response Acceleration S_{aM} (g)	Site Specific Design Response Spectrum Sa, per ASCE 7-16 Sec 21.3 (g)	Mapped Design Response Spectrum Sa, per ASCE 7-16 Sec 11.4.5 (g)	80% of Mapped Design Response Spectrum per ASCE 7-16 Sec 11.4.6 (g)	Site Specific Design Response Spectrum Sa, correlated for 80% requirement (g)
PGA	1.142	1.142	1.013	0.550	1.013	1.013	-	0.793	-	-
0.100	1.850	1.696	1.551	1.050	1.551	1.551	1.034	0.891	0.713	1.034
0.200	**2.373**	**2.176**	**2.036**	**1.500**	**2.036**	**2.036**	**1.357**	**1.181**	**0.945**	**1.357**
0.300	2.395	2.192	2.208	1.500	2.208	2.192	1.462	1.181	0.945	1.462
0.400	2.327	2.126	2.273	1.500	2.273	2.126	1.418	1.181	0.945	1.418
0.500	2.198	2.005	2.228	1.500	2.228	2.005	1.337	1.181	0.945	1.337
0.600	2.036	1.854	2.088	1.500	2.088	1.854	1.236	1.181	0.945	1.236
0.700	1.887	1.715	1.966	1.500	1.966	1.715	1.143	1.181	0.945	1.143
0.800	1.739	1.578	1.826	1.500	1.826	1.578	1.052	1.181	0.945	1.052
0.900	1.603	1.452	1.675	1.500	1.675	1.452	0.968	1.111	0.889	0.968
1.000	**1.493**	**1.350**	**1.548**	**1.500**	**1.548**	**1.350**	**0.900**	**1.000**	**0.800**	**0.900**
1.100	1.374	1.242	1.406	1.364	1.406	1.242	0.828	0.909	0.727	0.828
1.200	1.272	1.150	1.288	1.250	1.288	1.150	0.767	0.833	0.667	0.767
1.300	1.178	1.065	1.185	1.154	1.185	1.065	0.710	0.769	0.615	0.710
1.400	1.100	0.994	1.098	1.071	1.098	0.994	0.663	0.714	0.571	0.663
1.500	1.036	0.937	1.022	1.000	1.022	0.937	0.624	0.667	0.533	0.624
1.600	0.970	0.877	0.951	0.938	0.951	0.877	0.585	0.625	0.500	0.585
1.700	0.910	0.822	0.889	0.882	0.889	0.822	0.548	0.588	0.471	0.548
1.800	0.858	0.776	0.833	0.833	0.833	0.776	0.517	0.556	0.444	0.517
1.900	0.813	0.735	0.784	0.789	0.789	0.735	0.490	0.526	0.421	0.490
2.000	0.781	0.706	0.747	0.750	0.750	0.706	0.471	0.500	0.400	0.471
2.200	0.703	0.635	0.670	0.682	0.682	0.635	0.423	0.455	0.364	0.423
2.400	0.631	0.570	0.606	0.625	0.625	0.570	0.380	0.417	0.333	0.380
2.600	0.574	0.519	0.552	0.577	0.577	0.519	0.346	0.385	0.308	0.346
2.800	0.527	0.477	0.506	0.536	0.536	0.477	0.318	0.357	0.286	0.318
3.000	0.488	0.441	0.467	0.500	0.500	0.441	0.294	0.333	0.267	0.294
4.000	0.353	0.319	0.337	0.375	0.375	0.319	0.213	0.250	0.200	0.213


Site Specific Ground Motion Analysis: Cleaveland Crescent
Spectral Response Acceleration, Sa (g)
Period, T (s)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Probabilistic MCER per ASCE 7-16 Sec 21.2.1.1
Deterministic MCER per ASCE 7-16 Sec 21.2.2
Mapped Design Response Spectra per ASCE 7-16 Sec 11.4.5
Site Specific Design Response Spectra per ASCE 7-16 Sec 21.3 correlated for 80% requirement

Latitude, Longitude: 37.9444, -122.063

Date	5/24/2019, 1:26:44 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.772	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.772	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.181	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.721	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.793	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.538	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.768	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.772	Factored deterministic acceleration value. (0.2 second)
S1RT	0.882	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.975	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.721	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.917	Mapped value of the risk coefficient at short periods
C_{R1}	0.904	Mapped value of the risk coefficient at a period of 1 s

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Analysis Data
And
Seismic Source Summary Data

```
*********************************************
*****             EZ-FRISK              *****
***** SEISMIC HAZARD ANALYSIS DEFINITION *****
*****       FUGRO CONSULTANTS, INC.      *****
*****        WALNUT CREEK, CA  USA       *****
*********************************************
```

PROGRAM VERSION
  EZ-FRISK 7.65 Build 004

ANALYSIS TITLE:
  Cleaveland and Crescent

ANALYSIS TYPE:
  Single Site Analysis

SITE COORDINATES
  Latitude 37.9444
  Longitude -122.063

INTENSITY TYPE: Maximum Rotated Component of Spectral Response @ 5% Damping

HAZARD DEAGGREGATION
  Status: OFF

SOIL AMPLIFICATION
  Method: Do not use soil amplification

ATTENUATION EQUATION SITE PARAMETERS
  Depth[Vs=1000m/s] (m): 50
  Estimate Z1 from Vs30 for AS NGA: 1
  Vs30 (m/s): 293.5
  Vs30 Is Measured: 1
  Z25 (km): 3

AMPLITUDES - Acceleration (g)
  0.0001
  0.001
  0.01
  0.02
  0.05
  0.07
  0.1
  0.2
  0.3
  0.4
  0.5
  0.7
  1
  1.23008
  1.37133
  1.42428
  2
  3
  3.54071

```
PERIODS (s)
  PGA
  0.05
  0.1
  0.2
  0.3
  0.4
  0.5
  0.6
  0.7
  0.8
  0.9
  1
  1.1
  1.2
  1.3
  1.4
  1.5
  1.6
  1.7
  1.8
  1.9
  2
  2.2
  2.4
  2.6
  2.8
  3
  4

DETERMINISTIC FRACTILES
  0.5
  0.84

PLOTTING PARAMETERS
  Period at which to plot PGA: 0.005

CALCULATIONAL PARAMETERS
  Fault Seismic Sources -
    Maximum inclusion distance          :   200 km
    Down dip integration increment      :   1 km
    Horizontal integration increment    :   1 km
    Number rupture length per earthquake :   1
  Subduction Interface Seismic Sources -
    Maximum inclusion distance          :   1000 km
    Down dip integration increment      :   5 km
    Horizontal integration increment    :   20 km
    Number rupture length per earthquake :   1
  Subduction Slab Seismic Sources -
    Maximum inclusion distance          :   1000 km
    Down dip integration increment      :   5 km
    Horizontal integration increment    :   20 km
    Number rupture length per earthquake :   1
  Area Seismic Sources -
    Maximum inclusion distance          :   200 km
    Vertical integration increment      :   3 km
    Number of rupture azimuths          :   3
```

```
    Minimum epicentral distance step     :   0.5 km
    Maximum epicentral distance step     :   10 km
  Gridded Seismic Sources -
    Maximum inclusion distance            :   200 km
    Default number of rupture azimuths    :   20
    Maximum distance for default azimuths :   40 km
    Minimum distance for one azimuth      :   150
    Use binned calcuations if possible    :   true
    Bins per decade in distance (km)      :   20
  All Seismic Sources -
    Magnitude integration step           :   0.1 M
    Apply magnitude scaling              :   NO
    Include near-source directivity      :   NO
```

ATTENUATION EQUATIONS

Name: Abrahamson-Silva (2008) NGA MRC
Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
Base: FEMA P-750 Table C21.2-1
Truncation Type: No Truncation
Truncation Value: 0
Magnitude Scale: Moment Magnitude
Distance Type: Distance To Rupture

Name: Boore-Atkinson (2008) NGA USGS 2008 MRC
Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
Base: FEMA P-750 Table C21.2-1
Truncation Type: No Truncation
Truncation Value: 0
Magnitude Scale: Moment Magnitude
Distance Type: Distance To Rupture

Name: Campbell-Bozorgnia (2008) NGA USGS 2008 MRC
Database: C:\Program Files (x86)\EZ-FRISK 7.65\Files\standard.bin-attendb
Base: FEMA P-750 Table C21.2-1
Truncation Type: No Truncation
Truncation Value: 0
Magnitude Scale: Moment Magnitude
Distance Type: Distance To Rupture

SEISMIC SOURCE SUMMARY TABLE

Source	Region	Closest Distance	Deterministic Magnitude	Fault Mechanism	Dip Angle	Dips To	Site Lies
San Andreas Creeping Section Gridded	USGS 2008 California	99.15	6.0000	Strike Slip	90.0000	--	N
Shear 1 Gridded	USGS 2008 California	109.80	7.6000	Strike Slip	90.0000	--	SW
Bartlett Springs	USGS 2008 California	116.49	7.3000	Strike Slip	90.0000	--	S
Collayomi	USGS 2008 California	107.27	6.7000	Strike Slip	90.0000	--	SE
Great Valley 1	USGS 2008 California	150.43	6.8000	Reverse	15.0000	W	S
Great Valley 10	USGS 2008 California	169.87	6.5010	Reverse	15.0000	SW	NW
Great Valley 11	USGS 2008 California	191.23	6.6000	Reverse	15.0000	SW	NW
Great Valley 2	USGS 2008 California	128.96	6.5010	Reverse	15.0000	W	S
Great Valley 3, Mysterious Ridge	USGS 2008 California	79.61	7.1000	Reverse	20.0000	SW	S
Great Valley 4a, Trout Creek	USGS 2008 California	61.40	6.6000	Reverse	20.0000	SW	S
Great Valley 4b, Gordon Valley	USGS 2008 California	35.57	6.8000	Reverse	20.0000	W	S
Great Valley 5, Pittsburg Kirby Hills	USGS 2008 California	22.99	6.7000	Strike Slip	90.0000	--	SW
Great Valley 7	USGS 2008 California	51.54	6.9000	Reverse	15.0000	SW	NW
Great Valley 8	USGS 2008 California	93.36	6.8000	Reverse	15.0000	W	NW
Great Valley 9	USGS 2008 California	131.83	6.8000	Reverse	15.0000	SW	NW
Green Valley Connected	USGS 2008 California	5.15	6.8000	Strike Slip	90.0000	--	SW
Greenville Connected	USGS 2008 California	21.28	7.0000	Strike Slip	90.0000	--	W
Greenville Connected U	USGS 2008 California	21.28	7.0000	Strike Slip	90.0000	--	W
Hosgri	USGS 2008 California	201.59	7.3000	Strike Slip	80.0000	NE	N
Hunting Creek-Berryessa	USGS 2008 California	57.72	7.1000	Strike Slip	90.0000	--	S
Maacama-Garberville	USGS 2008 California	89.15	7.4000	Strike Slip	90.0000	--	SE
Monte Vista-Shannon	USGS 2008 California	58.69	6.5010	Reverse	45.0000	SW	N
Monterey Bay-Tularcitos	USGS 2008 California	113.83	7.3000	Strike Slip	90.0000	--	N
Mount Diablo Thrust	USGS 2008 California	11.15	6.7000	Reverse	38.0000	NE	N
Ortigalita	USGS 2008 California	101.75	7.1000	Strike Slip	90.0000	--	NW
Point Reyes	USGS 2008 California	66.62	6.9000	Reverse	50.0000	NE	E
Quien Sabe	USGS 2008 California	128.91	6.6000	Strike Slip	90.0000	--	NW
Rinconada	USGS 2008 California	143.63	7.5000	Strike Slip	90.0000	--	N
SAF - creeping segment	USGS 2008 California	136.55	6.7000	Strike Slip	90.0000	--	N
San Gregorio Connected	USGS 2008 California	52.39	7.5000	Strike Slip	90.0000	--	E
West Napa	USGS 2008 California	29.16	6.7000	Strike Slip	90.0000	--	SE
Zayante-Vergeles	USGS 2008 California	95.42	7.0000	Strike Slip	90.0000	--	N
California Gridded	USGS 2008 California	0.00	7.0000	SS|R	90.0000	--	Above
Calaveras	USGS 2008 California	14.94	7.0250	Strike Slip	90.0000	--	N
Hayward-Rodgers Creek	USGS 2008 California	18.18	7.3340	Strike Slip	90.0000	--	NE
Northern San Andreas	USGS 2008 California	47.14	8.0500	Strike Slip	90.0000	--	NE

Probabilistic Response Spectra

Probabilistic Spectra results for EZ-FRISK 7.65 Build 004

ANNUAL FREQUENCY OF EXCEEDANCE: 2.107e-003
RETURN PERIOD: 474.6
PROBABILITY OF EXCEEDENCE: 10.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008 MRC
Column 4: Acceleration (g) for: Abrahamson-Silva (2008) NGA MRC
Column 5: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008 MRC

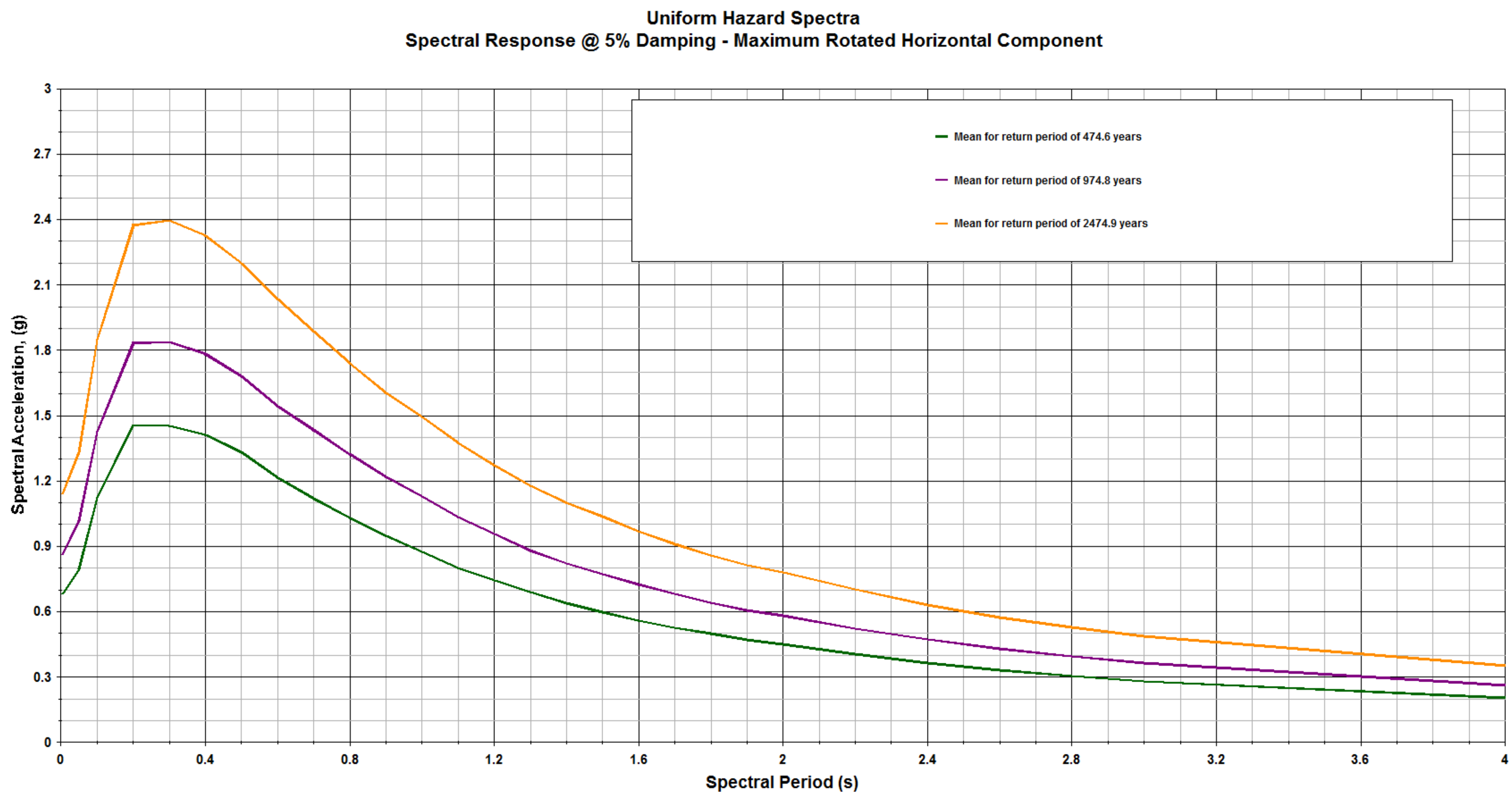
1	2	3	4	5
PGA	6.853e-001	6.717e-001	7.536e-001	6.204e-001
0.05	7.948e-001	7.946e-001	8.396e-001	7.508e-001
0.1	1.123e+000	1.147e+000	1.149e+000	1.075e+000
0.2	1.455e+000	1.466e+000	1.532e+000	1.362e+000
0.3	1.453e+000	1.457e+000	1.569e+000	1.313e+000
0.4	1.412e+000	1.451e+000	1.499e+000	1.262e+000
0.5	1.332e+000	1.405e+000	1.382e+000	1.204e+000
0.6	1.216e+000	1.287e+000	1.258e+000	1.094e+000
0.7	1.119e+000	1.196e+000	1.146e+000	1.015e+000
0.8	1.032e+000	1.101e+000	1.050e+000	9.364e-001
0.9	9.477e-001	1.010e+000	9.670e-001	8.652e-001
1	8.757e-001	9.286e-001	8.895e-001	8.091e-001
1.1	8.014e-001	8.528e-001	8.080e-001	7.440e-001
1.2	7.431e-001	7.918e-001	7.464e-001	6.903e-001
1.3	6.897e-001	7.417e-001	6.822e-001	6.391e-001
1.4	6.393e-001	6.995e-001	6.235e-001	5.972e-001
1.5	5.989e-001	6.574e-001	5.793e-001	5.622e-001
1.6	5.600e-001	6.094e-001	5.440e-001	5.285e-001
1.7	5.271e-001	5.690e-001	5.140e-001	4.999e-001
1.8	4.986e-001	5.346e-001	4.860e-001	4.712e-001
1.9	4.703e-001	5.049e-001	4.598e-001	4.465e-001
2	4.503e-001	4.762e-001	4.503e-001	4.250e-001
2.2	4.045e-001	4.233e-001	4.052e-001	3.833e-001
2.4	3.642e-001	3.806e-001	3.640e-001	3.481e-001
2.6	3.316e-001	3.449e-001	3.301e-001	3.200e-001
2.8	3.055e-001	3.165e-001	3.030e-001	2.966e-001
3	2.810e-001	2.922e-001	2.770e-001	2.738e-001
4	2.044e-001	2.073e-001	1.971e-001	2.082e-001

ANNUAL FREQUENCY OF EXCEEDANCE: 1.026e-003
RETURN PERIOD: 974.8
PROBABILITY OF EXCEEDENCE: 5.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008 MRC
Column 4: Acceleration (g) for: Abrahamson-Silva (2008) NGA MRC
Column 5: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008 MRC

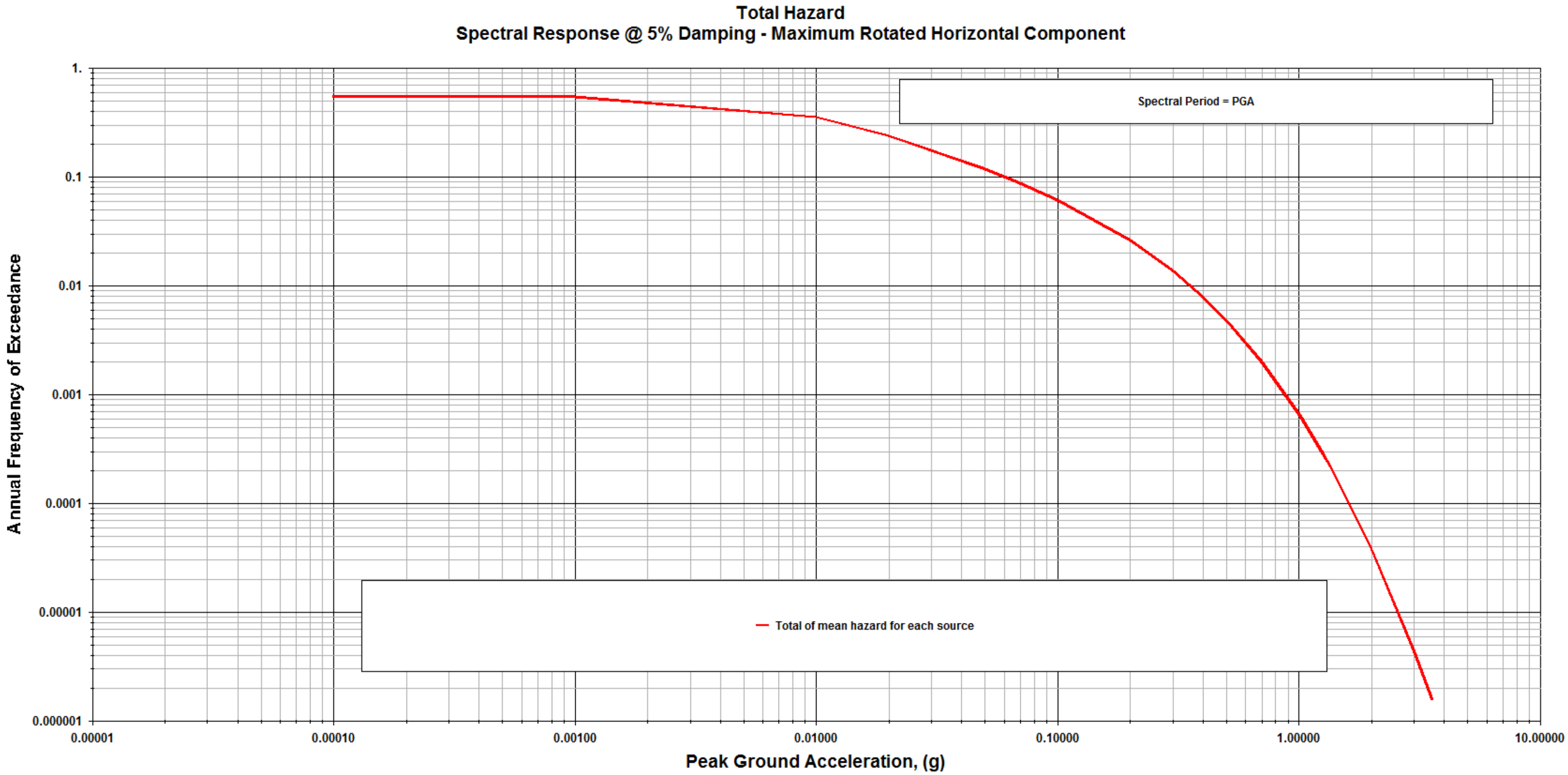
1	2	3	4	5
PGA	8.662e-001	8.393e-001	9.709e-001	7.841e-001
0.05	1.017e+000	1.012e+000	1.080e+000	9.485e-001
0.1	1.425e+000	1.460e+000	1.462e+000	1.346e+000
0.2	1.831e+000	1.855e+000	1.946e+000	1.688e+000
0.3	1.838e+000	1.854e+000	2.003e+000	1.643e+000
0.4	1.783e+000	1.857e+000	1.893e+000	1.585e+000
0.5	1.680e+000	1.803e+000	1.721e+000	1.513e+000
0.6	1.543e+000	1.656e+000	1.574e+000	1.390e+000
0.7	1.435e+000	1.548e+000	1.449e+000	1.290e+000
0.8	1.323e+000	1.436e+000	1.330e+000	1.197e+000
0.9	1.219e+000	1.314e+000	1.226e+000	1.110e+000
1	1.129e+000	1.214e+000	1.131e+000	1.041e+000
1.1	1.035e+000	1.117e+000	1.030e+000	9.528e-001
1.2	9.558e-001	1.039e+000	9.450e-001	8.778e-001
1.3	8.815e-001	9.709e-001	8.572e-001	8.171e-001
1.4	8.213e-001	9.096e-001	7.886e-001	7.669e-001
1.5	7.729e-001	8.578e-001	7.373e-001	7.247e-001
1.6	7.256e-001	7.978e-001	6.954e-001	6.811e-001
1.7	6.825e-001	7.470e-001	6.532e-001	6.400e-001
1.8	6.418e-001	7.035e-001	6.171e-001	6.048e-001
1.9	6.067e-001	6.602e-001	5.859e-001	5.744e-001
2	5.817e-001	6.223e-001	5.753e-001	5.477e-001
2.2	5.227e-001	5.523e-001	5.186e-001	4.970e-001
2.4	4.726e-001	4.987e-001	4.679e-001	4.515e-001
2.6	4.302e-001	4.512e-001	4.243e-001	4.152e-001
2.8	3.955e-001	4.134e-001	3.879e-001	3.835e-001
3	3.639e-001	3.806e-001	3.556e-001	3.557e-001
4	2.626e-001	2.685e-001	2.506e-001	2.688e-001

ANNUAL FREQUENCY OF EXCEEDANCE: 4.041e-004
RETURN PERIOD: 2474.9
PROBABILITY OF EXCEEDENCE: 2.0% IN 50.0 YEARS
Column 1: Spectral Period
Column 2: Acceleration (g) for: Mean
Column 3: Acceleration (g) for: Boore-Atkinson (2008) NGA USGS 2008 MRC
Column 4: Acceleration (g) for: Abrahamson-Silva (2008) NGA MRC
Column 5: Acceleration (g) for: Campbell-Bozorgnia (2008) NGA USGS 2008 MRC

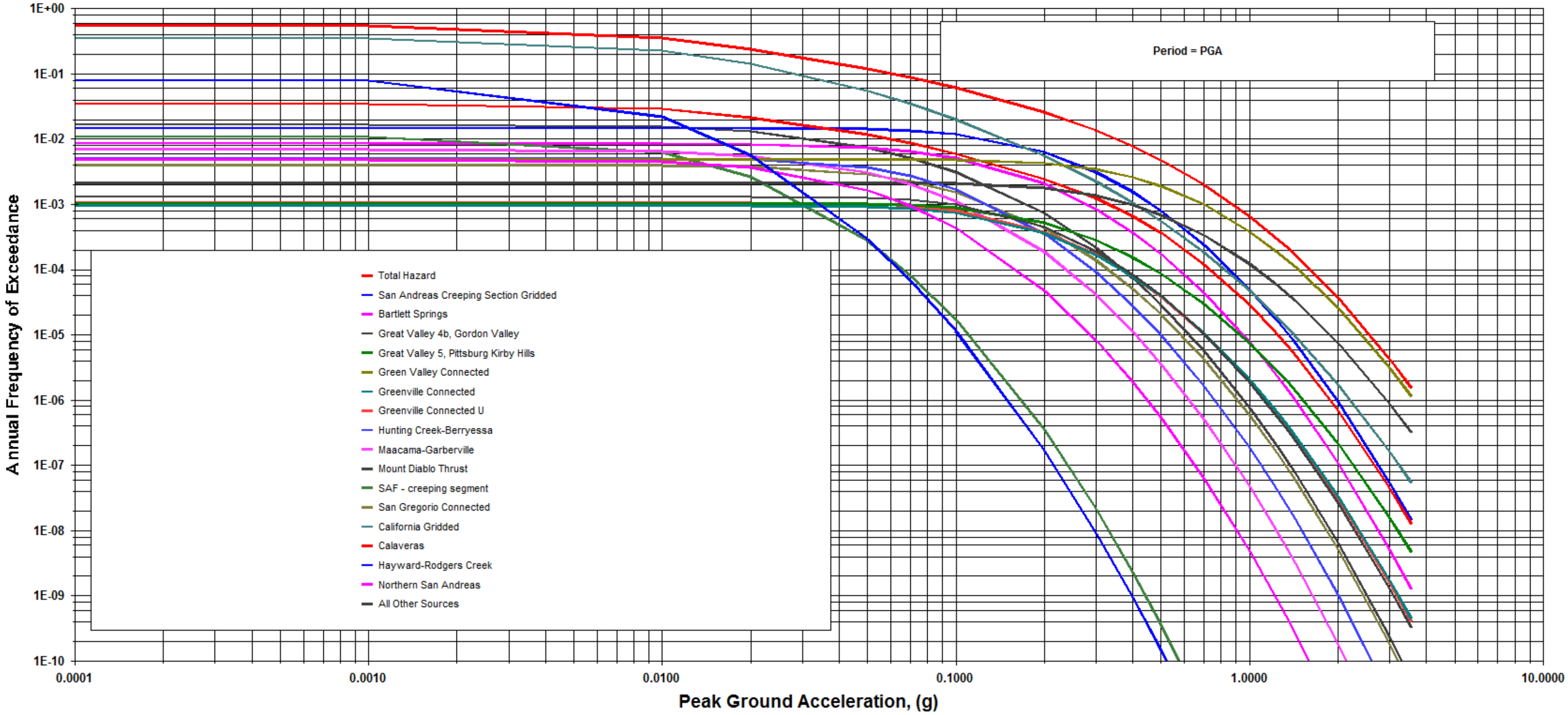
1	2	3	4	5
PGA	1.142e+000	1.094e+000	1.283e+000	1.028e+000
0.05	1.331e+000	1.321e+000	1.424e+000	1.237e+000
0.1	1.850e+000	1.916e+000	1.911e+000	1.719e+000
0.2	2.373e+000	2.413e+000	2.514e+000	2.176e+000
0.3	2.395e+000	2.426e+000	2.589e+000	2.135e+000
0.4	2.327e+000	2.442e+000	2.441e+000	2.068e+000
0.5	2.198e+000	2.388e+000	2.217e+000	1.965e+000
0.6	2.036e+000	2.214e+000	2.048e+000	1.795e+000
0.7	1.887e+000	2.081e+000	1.876e+000	1.672e+000
0.8	1.739e+000	1.927e+000	1.717e+000	1.560e+000
0.9	1.603e+000	1.757e+000	1.585e+000	1.459e+000
1	1.493e+000	1.623e+000	1.472e+000	1.372e+000
1.1	1.374e+000	1.504e+000	1.342e+000	1.260e+000
1.2	1.272e+000	1.405e+000	1.236e+000	1.165e+000
1.3	1.178e+000	1.316e+000	1.123e+000	1.086e+000
1.4	1.100e+000	1.239e+000	1.034e+000	1.020e+000
1.5	1.036e+000	1.170e+000	9.648e-001	9.593e-001
1.6	9.700e-001	1.088e+000	9.056e-001	8.993e-001
1.7	9.096e-001	1.019e+000	8.553e-001	8.484e-001
1.8	8.580e-001	9.540e-001	8.121e-001	8.046e-001
1.9	8.132e-001	8.963e-001	7.744e-001	7.664e-001
2	7.811e-001	8.462e-001	7.626e-001	7.328e-001
2.2	7.026e-001	7.508e-001	6.875e-001	6.613e-001
2.4	6.309e-001	6.742e-001	6.169e-001	6.009e-001
2.6	5.736e-001	6.083e-001	5.593e-001	5.525e-001
2.8	5.274e-001	5.562e-001	5.131e-001	5.129e-001
3	4.882e-001	5.139e-001	4.717e-001	4.770e-001
4	3.526e-001	3.631e-001	3.335e-001	3.609e-001



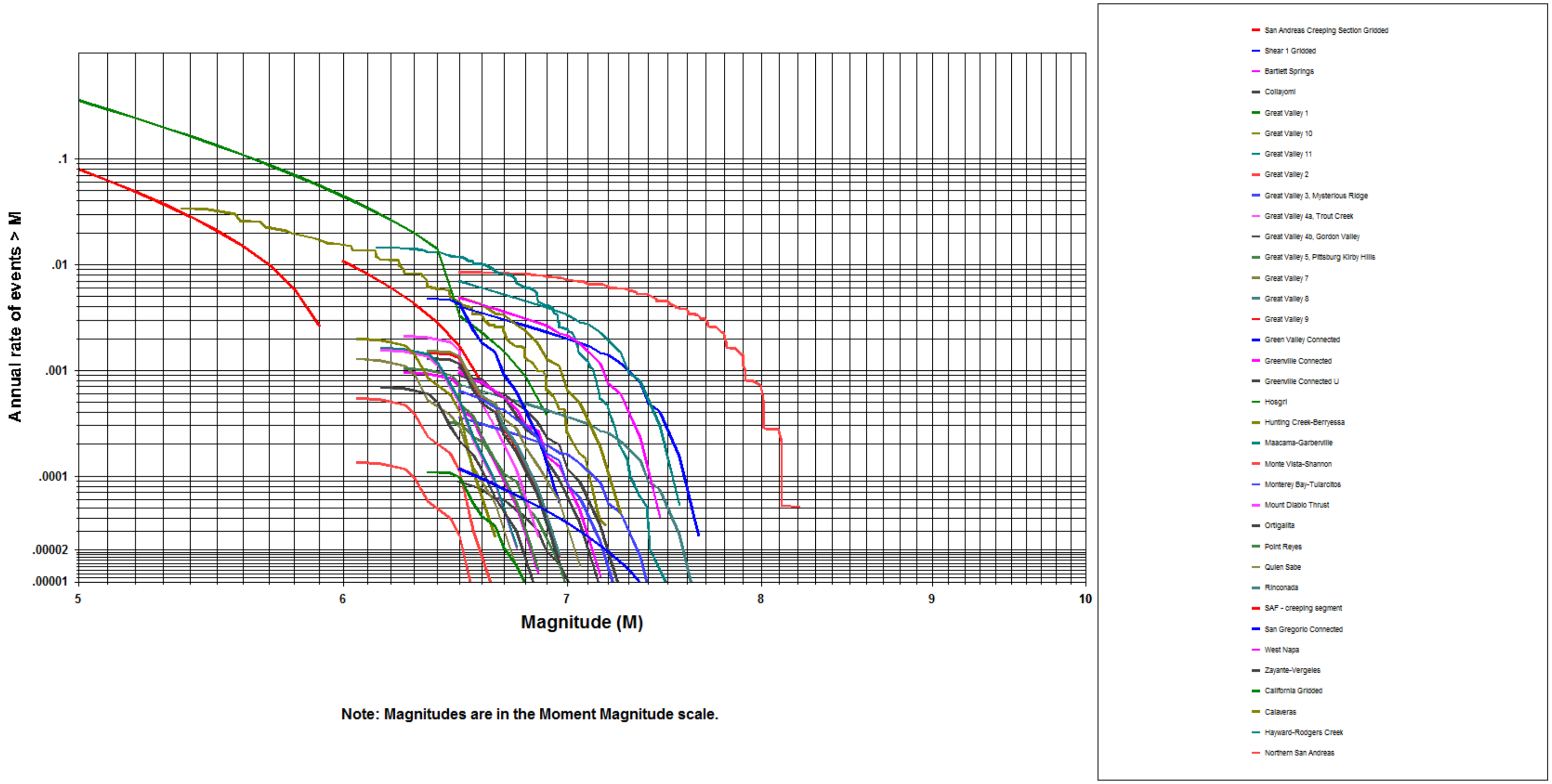
Uniform Hazard Spectra
Spectral Response @ 5% Damping - Maximum Rotated Horizontal Component
Mean for return period of 474.6 years
Mean for return period of 974.8 years
Mean for return period of 2474.9 years
Spectral Acceleration, (g)
Spectral Period (s)
0
0.3
0.6
0.9
1.2
1.5
1.8
2.1
2.4
2.7
3
0.4
0.8
1.2
1.6
2
2.4
2.8
3.2
3.6
4


Total Hazard
Spectral Response @ 5% Damping - Maximum Rotated Horizontal Component
Spectral Period = PGA
Total of mean hazard for each source
Annual Frequency of Exceedance
1.
0.1
0.01
0.001
0.0001
0.00001
0.000001
0.00001
0.00010
0.00100
0.01000
0.10000
1.00000
10.00000
Peak Ground Acceleration, (g)

Hazard by Seismic Source
Spectral Response @ 5% Damping - Maximum Rotated Horizontal Component



Activity Rate by Seismic Source



Note: Magnitudes are in the Moment Magnitude scale.

Deterministic Response Spectra

Deterministic Spectra Results using EZ-FRISK 7.65 Build 004

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations

Amplitude Units: Acceleration (g)

Fractile: 0.5

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	5.592e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	6.477e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	8.521e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	1.123e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	1.213e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	1.266e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	1.249e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	1.159e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	1.082e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	9.978e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	9.113e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	8.395e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	7.580e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	6.915e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	6.324e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	5.822e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	5.390e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	4.985e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	4.631e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	4.320e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	4.044e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	3.834e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	3.431e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	3.106e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	2.830e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	2.595e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	2.395e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	1.726e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Fractile: 0.84

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	1.013e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	1.174e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	1.551e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	2.036e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	2.208e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	2.273e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	2.228e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	2.088e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	1.966e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	1.826e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	1.675e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	1.548e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	1.406e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	1.288e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	1.185e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	1.098e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	1.022e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	9.510e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	8.885e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	8.332e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	7.841e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	7.471e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	6.698e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	6.062e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	5.521e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	5.062e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	4.670e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	3.372e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Largest Amplitudes of Ground Motions Considering Sources Calculated with Boore-Atkinson (2008) NGA USGS 2008 MRC

Amplitude Units: Acceleration (g)

Fractile: 0.5

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	6.308e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	7.907e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	1.076e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	1.380e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	1.491e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	1.553e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	1.518e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	1.411e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	1.326e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	1.213e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	1.086e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	9.832e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	8.784e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	8.048e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	7.506e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	7.036e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	6.625e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	6.135e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	5.707e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	5.331e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	4.998e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	4.702e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	4.201e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	3.791e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	3.450e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	3.161e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	2.914e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	2.084e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Fractile: 0.84

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	1.143e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	1.433e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	1.969e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	2.502e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	2.730e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	2.829e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	2.799e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	2.636e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	2.505e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	2.306e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	2.065e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	1.871e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	1.684e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	1.554e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	1.458e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	1.375e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	1.302e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	1.211e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	1.131e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	1.061e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	9.988e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	9.431e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	8.418e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	7.588e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	6.897e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	6.314e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	5.815e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	4.173e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Largest Amplitudes of Ground Motions Considering Sources Calculated with Abrahamson-Silva (2008) NGA MRC
Amplitude Units: Acceleration (g)

Fractile: 0.5

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	5.744e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	6.101e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	7.871e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	1.087e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	1.195e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	1.227e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	1.167e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	1.074e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	9.824e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	9.013e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	8.309e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	7.694e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	7.003e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	6.456e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	5.896e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	5.420e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	5.010e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	4.654e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	4.339e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	4.059e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	3.808e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	3.692e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	3.319e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	2.985e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	2.690e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	2.443e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	2.233e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	1.534e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Fractile: 0.84

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	1.041e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	1.106e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	1.426e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	1.969e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	2.165e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	2.181e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	2.036e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	1.879e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	1.725e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	1.595e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	1.479e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	1.376e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	1.256e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	1.161e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	1.067e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	9.890e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	9.217e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	8.633e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	8.115e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	7.653e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	7.239e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	7.073e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	6.362e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	5.724e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	5.160e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	4.686e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	4.285e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	2.948e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Largest Amplitudes of Ground Motions Considering Sources Calculated with Campbell-Bozorgnia (2008) NGA USGS 2008 MRC

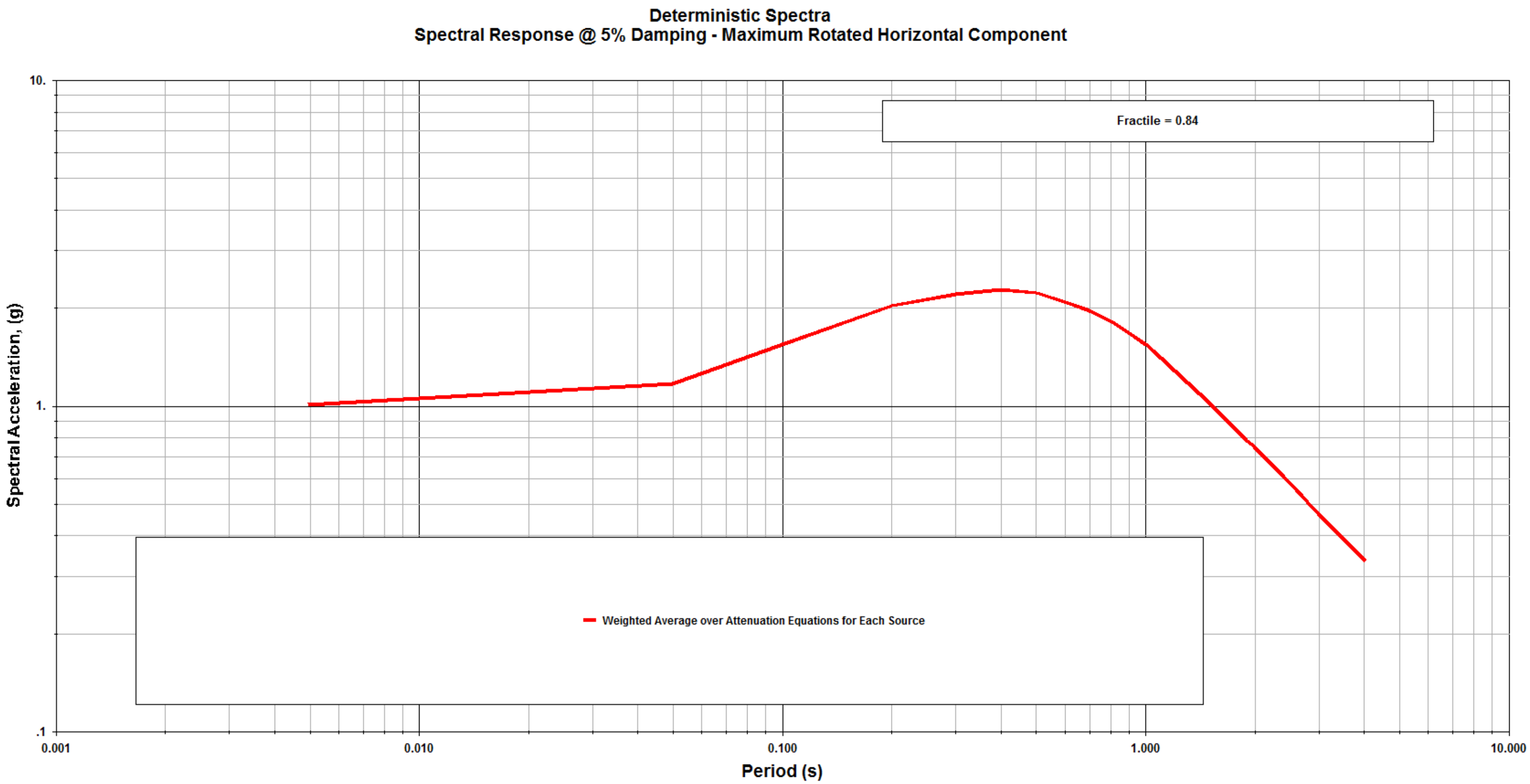
Amplitude Units: Acceleration (g)

Fractile: 0.5

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	4.758e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	5.499e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	7.022e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	9.028e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	9.544e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	1.018e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	1.060e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	9.920e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	9.376e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	8.786e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	8.171e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	7.658e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	6.952e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	6.365e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	5.868e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	5.443e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	5.075e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	4.712e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	4.396e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	4.116e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	3.869e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	3.647e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	3.242e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	2.911e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	2.637e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	2.406e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	2.210e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	1.677e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded

Fractile: 0.84

Period	Amplitude	Magnitude	Closest Distance(km)	Region	Controlling Source
PGA	8.622e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.05	9.965e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.1	1.273e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.2	1.636e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.3	1.730e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.4	1.809e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.5	1.850e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.6	1.749e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.7	1.669e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.8	1.578e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
0.9	1.480e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1	1.398e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.1	1.276e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.2	1.175e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.3	1.088e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.4	1.014e+000	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.5	9.495e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.6	8.864e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.7	8.310e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.8	7.819e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
1.9	7.381e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2	6.989e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.2	6.214e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.4	5.582e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.6	5.058e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
2.8	4.616e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
3	4.240e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded
4	3.221e-001	7.00 Mw	5.01	USGS 2008 California	California Gridded


Deterministic Spectra
Spectral Response @ 5% Damping - Maximum Rotated Horizontal Component
Fractile = 0.84
Weighted Average over Attenuation Equations for Each Source
Spectral Acceleration, (g)
10.
1.
.1
0.001
0.010
0.100
1.000
10.000
Period (s)

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