REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED SINGLE FAMILY RESIDENCE
LOT 7, TRACT 10036
589 ARCH PLACE
GLENDALE, CALIFORNIA

FOR

MR. ARSEN AGHAJANIAN

PROJECT NO. 24-422

JUNE 21, 2024

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June 21, 2024 24-422

Mr. Arsen Aghajanian
589 Arch Place
Glendale, California 91206

Subject: Geotechnical Investigation
Proposed Single Family Residence
Lot 7, Tract 10036
589 Arch Place
Glendale, California

Dear Mr. Aghajanian:

INTRODUCTION

This report presents the results of a geotechnical investigation for the subject project. The purpose of our investigation was evaluate the engineering properties of the subsurface materials in order to evaluate slope stability and to provide recommendations for design and construction of foundations. The investigation included geologic mapping, subsurface exploration, soil and bedrock sampling, laboratory testing, engineering and geological evaluation and analysis, consultation and preparation of this report.

During this investigation, a topographic survey map of the site prepared by California Engineering and Surveying, dated October 12, 2020, was used as reference. In addition, a site plan showing the proposed single family residence was provided by V J & Associates, dated August 15, 2023, was used as reference. The site map showed the approximate location of the proposed single family residence

The enclosed Drawing No. 1 shows surface geology and approximate locations of the exploratory test pits in relation to the site boundaries and the proposed building. This drawing also shows the approximate location of the Geologic Cross Sections A-A'. Drawing No. 2 shows the profile of the Geologic Cross Sections A-A'.

The attached Appendix I, describes the method of field exploration. Plate Nos. I-1 presents summaries of the materials found in our exploratory borings.

PROJECT CONSIDERATIONS

It is our understanding that the subject project will consist of construction of a single family residence. The building is expected to be a 2-story high reinforced concrete structure over a basement garage. The approximate location of the proposed residence with respect to the site boundaries are shown on the enclosed Drawing No. 1.

The floors of the proposed building will be created by creation of a terrace, through cutting operations. The resulting vertical cuts will then be supported by retaining walls having vertical heights of less than 10 feet. The approximate location of the proposed building and the associated improvements are shown on the enclosed Geologic Map and Site Plan.

Structural loading data was not available during the course of preparation of this report. For the purpose of this investigation, however, it is assumed that maximum concentrated loads will be on the order of 80 kips, combined dead plus frequently applied live loads. The retaining wall footings are expected to have loads of on the order of 6 kips per lineal foot.

SITE GRADING

Site grading for the proposed project is expected to involve cutting operations in order to create finished grades. As part of the site grading work, wall backfilling will also be made in the areas where temporary excavations have been made using unsupported, sloped cuts and behind the proposed basement and retaining walls. Only non-expansive, granular soils should be used for wall backfilling.

It is expected that the maximum excavation height (including footing/grade beam depths) would be less than 12 feet. The planned excavation will remove the existing, potentially unstable materials (fill, native soils) from the site. See the enclosed Geologic Cross Sections A-A’ (Drawing No. 2) for the proposed Structures at the subject property.

As part of the site grading work, basement walls will be constructed. The basement walls will then be integrated into the proposed building. The basement walls will be part of the proposed structure. The vertical height of the walls are expected to be less than 10 feet.

SITE CONDITIONS

SITE LOCATION AND SURFACE CONDITIONS

The subject site is located on a west -facing slope in the Verdugo Mountains. The lot is irregular in shape. The lot is approximately 112 feet wide in the maximum east-west direction and approximately 150 feet long in the maximum north-south direction. The lot is bound on the east by Arch Place, and on all other sides by developed lots. The subject lot has not been developed. From the street, the site ascends westerly some 80 feet on a slope with an average gradient of approximately 1.16 :1 (horizontal:vertical). Beyond the west property line, the gradient is flat.

Runoff from the site slopes is by sheetflow to the lot below.

The entire site is covered with annual weeds and grasses and some small to medium size trees.

No evidence of shallow ground water, seepage or springs was observed anywhere on the surface of the site.

REGIONAL GEOLOGY

The project site is located in the Verdugo Mountains, which make up part of the Transverse Ranges Geomorphic province. The Transverse Ranges Geomorphic province forms a relatively youthful physiographic feature in response to accelerated uplift that accommodates concentrated north-south crustal shortening in a region of compression where the North American and Pacific plate boundary trends obliquely to their overall motion vector. The San Andreas Fault, which forms the northern boundary of the Transverse Ranges geomorphic province in this area, exhibits a more westerly orientation in this area as opposed to its' overall northern orientation.

The Verdugo Mountains are a structural block located between the Santa Monica and San Gabriel Mountains, and are bound to the south by the Verdugo Fault Zone and to the north by north-dipping San Fernando and Sierra Madre Fault Zones. The project site is located along the southern margin of the Verdugo Mountains within the northeastern block of the Los Angeles Basin (Yerkes and others, 1965). Eastern basement rocks are exposed in the Verdugo Mountains.

GEOLOGIC AND SOIL CONDITIONS

Observations of the test pits as well as review of published geologic maps indicated that the lot is underlain by artificial fill, colluvial soil and crystalline basement rock, of Cretaceous age. Descriptions and distribution of these units are as follows:

Artificial Fill (Af)

The existing fill consists of moderately compact, brown, porous, silty, coarse to very fine sand. The fill overlies natural soil on the lower slope and is 1 to 2 feet thick. Such materials are believed to have been generated from the local sources.

Colluvial Soil (Qsw)

The colluvial soils consists of moderately loose to medium dense, brown, porous, silty, coarse to very fine grained sand. The soil zone is approximately 1 to 3 feet thick.

Basement Rock (Kgd)

The Rock beneath the slope consists of Very firm to moderately hard, gray-brown, to brown-gray Granodiorite. The rock is slightly to moderately fractured, moderately weathered and quite competent.

ENGINEERING GEOLOGY

The engineering geologic factors addressed herein that are considered significant to the construction of the retaining wall are fracturing within bedrock, excavation characteristics, landslides and groundwater.

Fractures

Fractures as observed in the test pits are not well developed within the rock.

Excavation Characteristics

Bedrock at the site is moderately resistant to excavation. Standard equipment, however, can be used to excavate bedrock. Jack hammers will be necessary within hard bedrock. If drilling equipment is used for foundation excavation, coring may be occasionally necessary.

Due to granular in nature, and relatively poor degree of compaction, the fill soil at the subject site is considered to be susceptible to caving. Excavations should be shored or trimmed back to a safe angle.

Landslides

Evidence of ancient or recent landslides was not observed on the property at the time of our field investigation. Regional geologic maps published on the area did not exhibit and landslides in the region. No evidence of surficial failures were observed on the site.

Groundwater

Shallow ground water, seepage or springs were not observed during our investigation. No ground water seepage was observed in our exploratory excavations. The ground water level is considered below an influential level within bedrock.

SCOPE OF WORK

FIELD INVESTIGATION

In order to define the subsurface conditions, two test pits were excavated within the subject site. The approximate locations of the test pits are shown on the enclosed Drawing No. 1. The test pits were extended to a maximum depth of 10 feet below existing grades.

Continuous logs of the subsurface conditions, as encountered in the drilled test pits, were recorded in the field and are presented on the Log of Exploratory test pits; Plate No. I-1 within Appendix I.

LABORATORY TESTING

The laboratory tests were conducted on representative samples in order to determine certain physical properties of the subsurface materials. Field moisture content, in-situ density, shear strength, consolidation were determined from these tests. The laboratory test results are presented on Plate No. II-1 within Appendix II.

The shear strength used in our calculations is representative of the existing materials. The shear value used is the weakest material strength of the existing bedrock. As indicated in the shear test results as the material gets deeper the shear value increases with depth.

ENGINEERING ANALYSIS

The results of our field and laboratory investigations were evaluated. Based on the results of the laboratory testing, engineering analyses were performed in order to formulate recommendations for design and construction of foundations.

SEISMIC HAZARD EVALUATION

The site is located in Southern California, an area of known seismic activity and the site will be subjected to intense ground shaking during potentially large earthquakes on nearby faults. The site is not located within an Alquist-Priolo Earthquake Fault Zones. Although no known surface traces of active faults traverse the site, the site will be subjected to intense ground shaking during potentially large earthquakes on nearby faults.

The nearest seismic source exposed at the surface is believed to be the Verdugo Fault located some 1.85 km north of the site. It is postulated that the Verdugo Fault is capable of producing probable earthquake magnitudes ranging from 6.8. The fault is considered to be active with slip rates is 0.5 mm/yr. The estimated interval between major rupture is uncertain.

Potential ground shaking at the site was evaluated at the U.S.G.S. Unified Hazard Tool website using the USGS NSHM 2014 Dynamic Edition. Analysis for the total deaggregation using a peak ground acceleration spectral period and a time horizon of 10% in 50 years yields a mean magnitude event of 6.9 with an epicentral distance of 10.55 km. Calculating these statistics for a time horizon of 2% in 50 years yields a mean magnitude of 7.02 with an epicentral distance of 6.39 km. Using Figure 1 of SP117a, (Blake and others, 2002), a seismic coefficient of 0.33 should be used in the seismic analysis where a 5 cm threshold is required. The subject site is not within seismic landslide zone.

Ground shaking resulting from earthquakes common to southern California can be expected within the lifespan of the structure. No major problems are anticipated as a result of fault displacement or ground lurching resulting from earthquakes provided the foundation system is constructed as herein recommended.

SEISMIC DESIGN CONSIDERATIONS

Based on soil properties, the site class is C. Ground motions are expressed as a fraction of the acceleration due to gravity (g). The values of ground motion are determined from the address, utilizing ASCE Hazard tools website and the USGS Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration 7-22 Standard Ground motion values are:

PGA_M	1.02	Site modified peak ground acceleration
S_{MS}	2.46	Site-modified spectral acceleration value
S_{M1}	1.12	Site-modified spectral acceleration value
S_{DS}	1.64	Numeric seismic design value at 0.2 second SA
S_{D1}	0.75	Numeric seismic design value at 1.0 second SA
TL	8	Long-period transition period in seconds
S_s	2.42	MCER ground motion. (for 0.2 second period)
S_1	0.78	MCER ground motion. (for 1.0s period)
VS30	530	Time-averaged shear-wave velocity to a depth of 30 meters.

PREVENTIVE SLOPE MAINTENANCE

For the descending slopes and all slopes in general, it is important to reduce the risk of problems relating to slope instability. It is recommended that you implement a program of normal slope maintenance. This maintenance program should include annual cleanout of drains, elimination of earth burrowing rodents, maintaining low water consumptive, fire retardant, deep rooted ground cover and proper irrigation.

Hillside properties are typically subject to potential geotechnical hazards including mudslides, spalling of slopes, erosion and concentrated flows. It must be emphasized that responsible maintenance of these slopes, and the property in general, by the owner, using proper methods, can reduce the risk of these hazards significantly.

EVALUATION AND RECOMMENDATIONS

GENERAL

Based on the geotechnical engineering data derived from this investigation, it is our opinion that the proposed construction may be made as planned. The proposed deck structure supported through deep foundations is believed to enhance the overall stability of the existing slope, at least within the upper areas. Such construction will reduce the amount of direct rain fall on the slope. It is our opinion that the existing slope with the imposed surcharge from the proposed single family deck will remain grossly stable (see the enclosed Engineering Calculation Sheets).

For the purpose of establishing the proposed finished grades, cutting will be made in order to create the basement garage. Such excavation will remove a portion of the existing, potentially unstable materials (fill, native soils) from the site. The vertical cuts resulting from the planned excavation will be supported through cantilevered retaining wall systems having vertical heights of 4 to 12 feet. All retaining walls constructed as part of the proposed project can be designed based on normal lateral earth pressure.

The proposed residence near the slope and the associated retaining walls should be supported through deep foundations consisting of friction piles. These piles should then be connected at their top with a grade beam. In addition, it is recommended that the proposed The outside face of the vertical shafts at the bottom should have a clear horizontal distance of 25 feet from the face of the descending bedrock surface.

If the new backfill is not benched into the bedrock, the vertical shafts should be designed not only for gravity loads, but also against lateral creep forces. For the purpose of this project, a lateral sustained load with a magnitude of 1,000 pounds for each foot of the vertical shaft in contact with the existing fill and native soils should be used for this project. The point of application of the lateral creep force should be taken at the surface of bedrock. The center-to-center spacing of the vertical shafts should not exceed 15 feet.

For the temporary shoring for the retaining walls along the street, cantilever soldier piles should be used. The cantilever soldier piles can then be incorporated into the permanent retaining walls and act as deep foundation support system.

It should be noted that the length of the piles should be determined for both temporary support of the bank and permanent support of the structure. The longest pile will govern the design.

The grade slabs may be cast directly over bedrock, or properly compacted granular fill soils. Where grade slabs span between soil and bedrock, the bedrock should be over-excavated by some 12 inches and the excavated materials should be used as compacted fill. This will create uniform subgrade conditions beneath grade slabs and reduce the chances of uneven subgrade movements.

The following sections present our specific recommendations for temporary shoring, site grading, foundations, lateral design, grade slabs, retaining walls, site drainage, and observations during construction. The grade slabs may be cast directly over bedrock, or properly compacted granular fill soils.

TEMPORARY SHORING

Unsupported/open Cuts: As part of the site grading work, temporary excavations will be required to establish the proposed finished grades. The excavations are expected to be made through the existing fill, native soils Bedrock. Where space limitations permit, unshored temporary excavation slopes can be used. Based upon the engineering characteristics of the subsurface materials, it is our opinion that temporary excavation slopes in accordance with the following table should be used:

Maximum Depth of Cut (Ft)	Maximum Slope Ratio (Horizontal:Vertical)	
	Soil	Bedrock
0-5	Vertical	Vertical
5-10	1:1	Vertica

Water should not be allowed to flow over the top of the excavation in an uncontrolled manner. No surcharge should be allowed within a 45-degree line drawn from the bottom of the excavation. Excavation surfaces should be kept moist but not saturated to retard ravelling and sloughing during construction. It would be advantageous, particularly during wet season construction, to place polyethylene plastic sheeting over the slopes. This will reduce the chances of moisture changes within the soil banks and material washing into the excavation.

Cantilevered Soldier Piles: Cantilevered soldier piles should be used for support of the front and side cut. Soldier piles consist of structural steel beams encased in concrete (below the base of the excavation) and slurry mix within the exposed depths of excavation.

The lateral resistance for cantilevered soldier piles may be assumed to be offered by available passive pressure below the basement level. An allowable passive pressure of 700 pounds per square foot per foot of depth may be used below the basement level for soldier piles having center-to-center spacing of at least 2-1/2 times the pile diameter. Maximum allowable passive pressure should be limited to 4,000 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 10 feet.

For temporary construction excavations, the active pressure on cantilevered soldier piles may be computed using an equivalent fluid density of 21 pounds per cubic foot. Uniform surcharge may be computed using an active pressure coefficient of 0.25 times the uniform load.

When using cantilevered soldier piles for temporary shoring, the point of fixity (for the purpose of moment calculations), may be assumed to occur at some one foot below the bottom of excavation (bedrock).

In order to limit local sloughing, it is recommended that lagging be used where soil is exposed between the soldier piles. All wood members left in ground should be pressure treated.

It should be noted that the recommendations presented in the "TEMPORARY EXCAVATION" section of this report is for use in design and for cost estimating purposes prior to construction. The contractor is solely responsible for safety during construction.

SITE GRADING

Site grading for the proposed project is expected to involve cutting operations in order to create finished grades. As part of the site grading work, some wall backfilling will also be made behind retaining walls and in the areas where temporary excavations have been made using unsupported, sloped cuts. Only non-expansive, granular soils should be used for wall backfilling. In addition the existing swimming pool will be demolished and backfield.

Prior to placing any fill, the Soil Engineer and Engineering Geologist should observe the excavation bottoms and subdrains. The areas to receive fill should be scarified and compacted in-place to a relative compaction of at least 90 percent at near the optimum moisture content.

General guidelines regarding site grading are presented below in an itemized form which may be included in the earthwork specification. It is recommended that all fill be placed under engineering observation and in accordance with the following guidelines:

1. All vegetation should be shaved and removed from the site;

2. Subdrain should be installed behind all retaining walls. All subdrain should be observed and approved by this office before backfilling;

3. The subdrain pipes should be laid at a minimum grade of two percent for self cleaning.

4. The excavated materials from the site may be reused in the areas of new fill. Wall backfill, however, should consist of granular materials.

5. Rocks larger than 6 inches in diameter should be excluded from the areas of compacted fill .

6. Fill material, approved by the Soil Engineer, should be placed in controlled layers. Each layer should be compacted to at least 90 percent of the maximum unit weight as determined by ASTM designation D 1557-09 for the material used.

7. The fill material shall be placed in layers which, when compacted, shall not exceed 8 inches per layer. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material in each layer.

8. When moisture content of the fill material is too low to obtain adequate compaction, water shall be added and thoroughly dispersed until the moisture content is near optimum.

9. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material shall be aerated by blading or other satisfactory methods until near optimum moisture condition is achieved.

10. Inspection and field density tests should be conducted by the Soil Engineer every vertical 2 feet during grading work to assure that adequate compaction is attained. Where compaction of less than 90 percent is indicated, additional compactive effort should be made with adjustment of the moisture content or layer thickness, as necessary, until at least 90 percent compaction is obtained.

FOUNDATIONS

Spread footing foundation system placed in bedrock may be used for support of the upper portion of the proposed building. All footings should also be established in bedrock.

The support system of the proposed single family residence near the slope, decking and the associated retaining walls should be in a form of deep foundations consisting of cast-in-place end-bearing caissons and/or friction piles. The vertical shafts near the slope should have a minimum length of 23 feet. At this depth, the horizontal setback of the vertical shafts (at the bottom) will be about H/3 feet from the face of the descending slope surface which is considered to be adequate for the structures. See the enclosed Section A-A' - Drawing No. 2.

End-bearing caissons should have a minimum shaft diameter of 30 inches to facilitate excavation and inspection. The diameter of the friction piles should be at least 16 inches.

An allowable maximum bearing value of on the order of 4,000 pounds per square foot may be used for continuos, spread footings and/or end-bearing caissons established in the bedrock. The footings should be placed (one foot into bedrock) at minimum depths of 24 inches below the lowest adjacent final grades. All footings should have a minimum width of 12 inches. For the purpose of estimating the vertical capacities of the friction piles, an allowable maximum skin friction value of 600 pounds per square foot should be used for the bedrock. No capacity should be allowed for the existing fill and .

For friction pile design, the weight of the shafts can be assumed to be taken by end-bearing, therefore, need not be added to the structural loads. All piles should be concreted as soon as they are excavated and, for safety reasons not be left open overnight .

The above given allowable bearing and skin friction values are for dead, plus frequently applied live loads. For short duration transient loadings, such as wind or seismic forces, the given values may be increased by one third.

Under the allowable maximum soil pressure, total and differential settlements of the proposed single family residence and the associated structures (retaining walls and decking) are expected to be within tolerable limits, less than 1/2 and 1/4 of one inch, respectively. The major portion of the settlements are expected to occur during construction.

It should be noted that, if the caissons are excavated with hand tools (due to difficult access with machine) the vertical shaft excavations should be properly shored for workman safety. All the applicable construction safety laws of OSHA should be followed by the project contractor.

LATERAL DESIGN

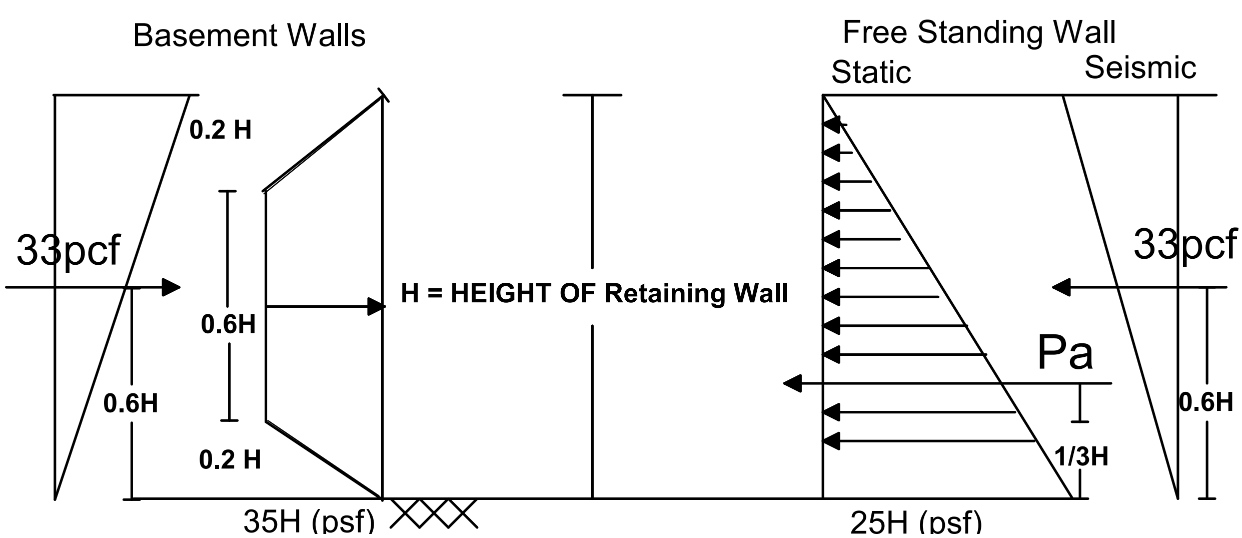
Lateral resistance at the base of the spread footings in contact with bedrock may be assumed to be the product of the dead load forces and a coefficient of friction of 0.35. Passive pressure against foundations may also be used to resist lateral loads.
For the purpose of this project, a passive pressure of 100 pounds per square foot at 3 feet from the surface of bedrock (below the lowest unsupported bedding) and increasing at a rate of 300 pounds per square foot per foot of depth to a maximum value of 4,000 pounds per square foot may be used for foundations poured against

bedrock. No passive pressure should be allowed for the upper soils or unsupported bedding.

It is anticipated that the pile support system on the slope will be designed as "flag pole". Assuming that the horizontal spacing of the vertical shafts will be greater than 2.5 times the pile diameter, the given passive pressures may be doubled. For the purpose of moment calculations, the point of fixity of the vertical shafts may be about 3 feet below the 22 degree line drawn from the south property line (lowest unsupported Bedding). The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 15 feet.

RETAINING WALLS

As part of the proposed development, retaining walls with heights of less than 12 feet will be constructed. Such walls will support cuts of compacted fill and Bedrock. Static design of cantilevered retaining walls supporting cuts of compacted fill with level ground conditions may be based on an equivalent fluid pressure of 35 pounds per square foot per foot of depth. The retaining wall supporting 2:1 slope should be designed based on an equivalent fluid pressure of 45 pounds per square foot per foot of depth. Static design of these walls (being restrained against rotation designed at rest condition) could be based on an equivalent fluid pressure of 50 pounds per cubic foot.



If the height of the retaining wall is greater than 6 feet, the lateral pressures on retaining walls due to earthquake motions is could be based on an equivalent fluid pressure for level backfill of 33 pounds per cubi c foot.

It is recommended that the retaining walls supporting the ascending slope have a minimum freeboard of 12 inches and a paved drain to divert surface water from the slope and collect minor debris washed down during heavy rain. The freeboard should be cleaned after the rainy season.

The above given pressures assume that hydrostatic pressure will be relieved from the back of the retaining walls through a properly designed and constructed backdrain system. Backdrain normally consists of 4-inch diameter perforated pipes encased in free draining gravel; at least one cubic foot per lineal foot of the pipe. The wall backfill should be granular in nature constructed on and benched into bedrock. The gradients of all new wall backfill should be maintained at no steeper than 2:1 (horizontal:vertical).

GRADE SLABS

Grade slabs may be cast on bedrock, or compacted granular fill. Where grade slabs span between soil and bedrock, the bedrock should be over-excavated by some 12 inches and the excavated materials should be used as compacted fill.

In the areas where moisture sensitive floor covering is used and slab dampness cannot be tolerated, a vapor-barrier should be used beneath the slabs. This normally consists of a 6-mil polyethylene film covered with 2 inches of clean sand.

SITE DRAINAGE

Site drainage should be provided to divert roof and surface waters from the property through nonerodible drainage devices to the street. In no case should the surface waters be allowed to pond adjacent to buildings, behind the retaining walls or flow over the slope surfaces in an uncontrolled manner. A minimum slope of one and two percent is recommended for paved and unpaved areas, respectively.

OBSERVATION DURING CONSTRUCTION

The presented recommendations in this report assume that all structural foundations will be established in competent bedrock. All foundation excavations should be observed by a representative of this office. It is essential to assure that all excavations are made at proper dimensions, are established in the recommended bearing material and are free of loose and disturbed soils.

Site grading work should be made under continuous observation and testing by a representative of this firm. For proper scheduling, please notify this office at least 24 hours before any inspection work is required.

CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either express or implied.

It is noted that the conclusions and recommendations presented are based on exploration "window" borings and excavations which is in conformance with accepted engineering practice. Some variations of subsurface conditions are common between "windows" and major variations are possible.

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The following Plates and Appendices are attached and complete this report:

Engineering Calculation Sheets
Geologic Map and Site Plan-Drawing No. 1
Geologic Cross Section A-A'- Drawing No. 2
Vicinity Map - Drawing No. 3
Appendix I   Method of Field Exploration
Plate No. I-1
Appendix II - Methods of Laboratory Testing
Plate Nos. II-1

Respectfully Submitted,

ADVANCED GEOTECHNIQUES

Peter Pailian
Associate Engineer

Reviewed By:

Ara Jitechian
Civil Engineer
CE 54893

PP/AJ/SBM
Distribution: (4)

GEOLOGIC REFERENCES

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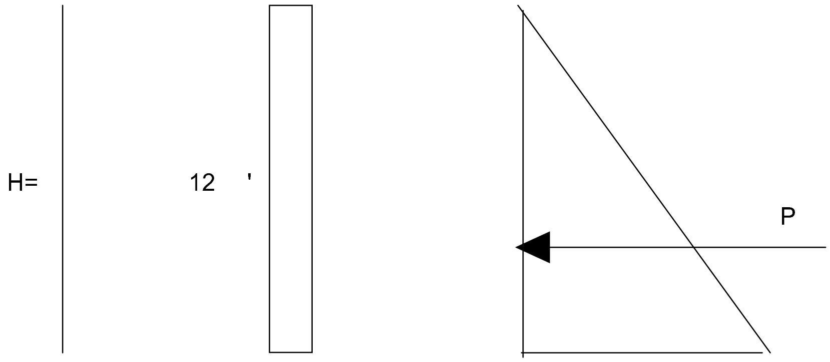


SOIL STRENGTH PARAMETERS

H=	12	Feet	Height of retaining wall
γ=	127	PCF	In situ Unit weight
γ=	140	PCF	Wet density
C=	700	PSF	Cohesion of Layer Along Potential Sliding Surface
ϕ=	40	DEG	Friction angle of layer along potential sliding surface

The Coefficient of earth pressure

$K_0 = 1 - \sin\phi$

$K_0 = 1 - 0.64279 = 0.36$

AT 12 Feet $\sigma_h = K_0\, \sigma_v$

$\sigma_v =$ 12 x 140 = 1,680.00 LB/ft2

$\sigma_h =$ 0.36 x 1680 = 600.12 LB/ft2

USE

at rest P = $1/2\, \sigma_v * H^2 =$ 3,601 3,601 LB/ft

EQUIVALENT FLUID PRESSURE (EFP)= $(2\text{*UBF})/(h^2)$
Height of Retaining Wall= 12 Feet

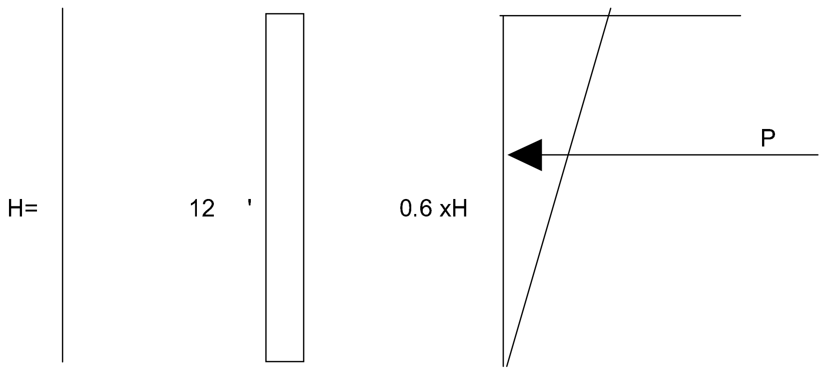
EFP= 50 pcf Use 50 PCF supporting Level grade

LATERAL EARTH FORCE AT REST ANALYSIS

Project Name: Mr. Arsen Aghajanian Date: **06/21/2024** Project No. **23-417**

ADVANCED GEOTECHNIQUES
GEOTECHNICAL ENGINEERING CONSULTANTS

SHEET No. 1



SOIL PARAMETERS

H=	12	Feet	Height of retaining wall
γ=	127	PCF	Unit weight
PGAm=	1.02	g	

The seismic lateral earth pressureis coefficient equivalent to one-half of two-thirds of two-thirds of PGAM;

Kh = 1/2PGAm/ (2/3) = 0.34

F= 3/4 Kh* γ

EFP= F= 32.4 pcf Use 33 PCF supporting Level grade

Resultent acting at a distance of (0.6*H) from base of wall

SEISMIC LATERAL FORCE OF WALL

Project Name: Mr. Arsen Aghajanian Date: **6/21/2024** Project No. **24-422**

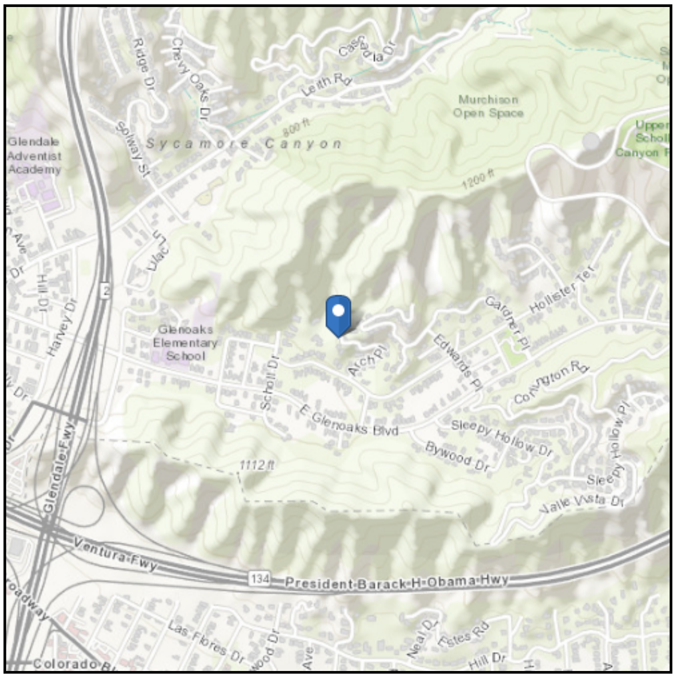
ADVANCED GEOTECHNIQUES
GEOTECHNICAL ENGINEERING CONSULTANTS

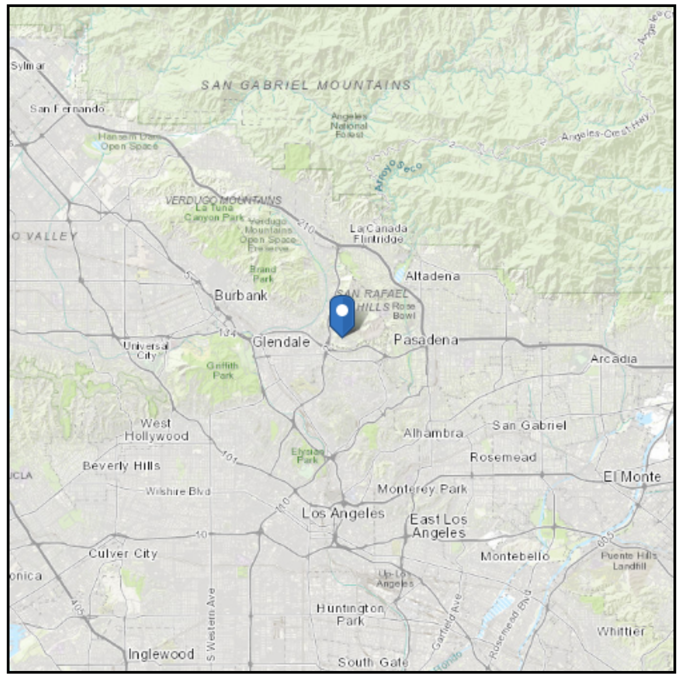
SHEET No. 2

ASCE Hazards Report

Address:
589 Arch Pl
Glendale, California
91206

Standard:	ASCE/SEI 7-22	**Latitude:**	34.152322
Risk Category:	II	**Longitude:**	-118.215887
Soil Class:	C - Very Dense Soil and Soft Rock	**Elevation:**	903.0247774499416 ft (NAVD 88)





ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

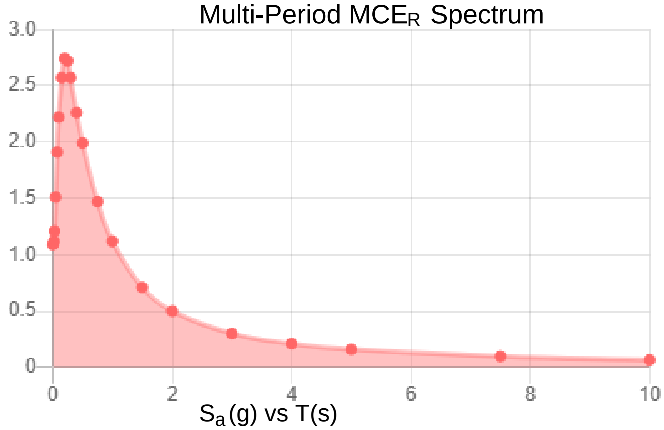
Site Soil Class: C - Very Dense Soil and Soft Rock

Results:

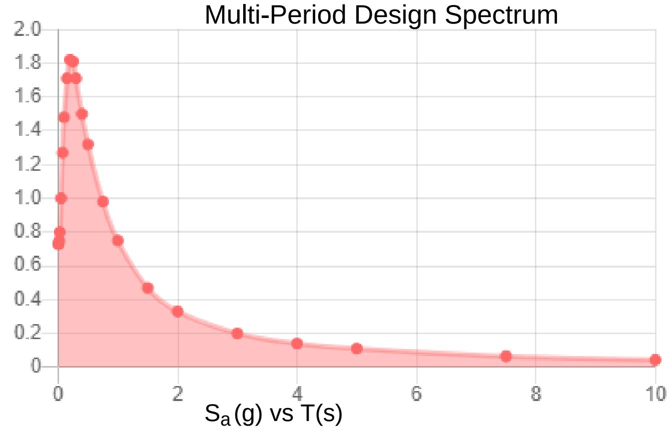
PGA_M :	1.02	T_L :	8
S_{MS} :	2.46	S_S :	2.42
S_{M1} :	1.12	S_1 :	0.78
S_{DS} :	1.64	V_{S30} :	530
S_{D1} :	0.75		

Seismic Design Category: E

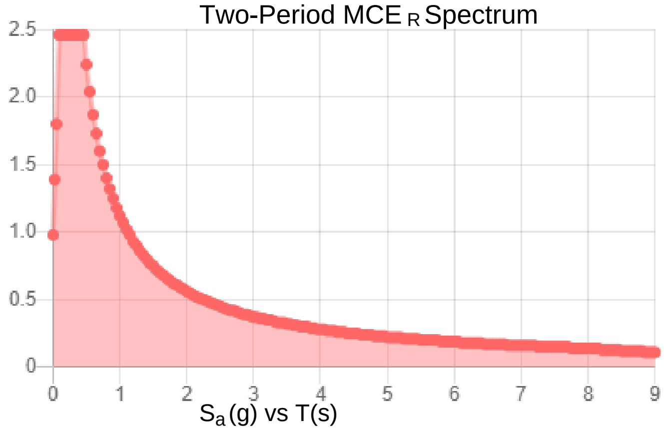
Multi-Period MCE_R Spectrum



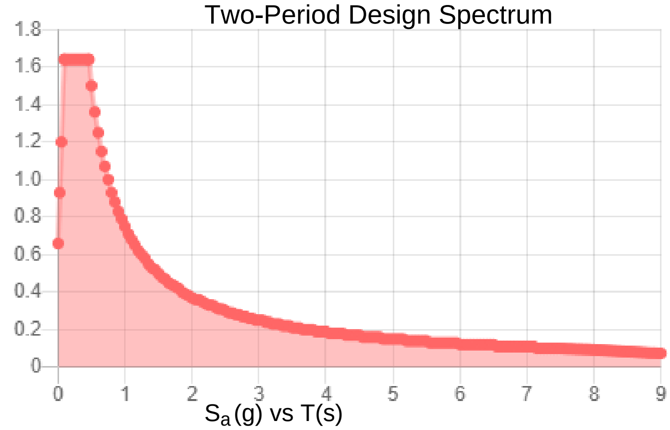
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Fri Jun 21 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

34.152322

Longitude

Decimal degrees, negative values for western longitudes

-118.215887

Site Class

537 m/s (Site class C)

Spectral Period

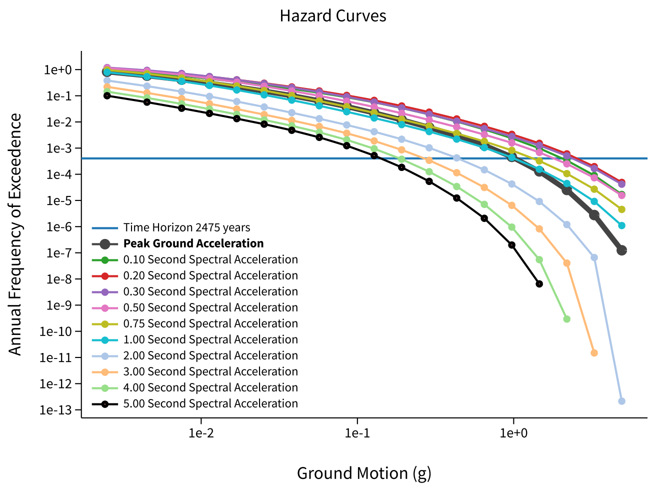
Peak Ground Acceleration

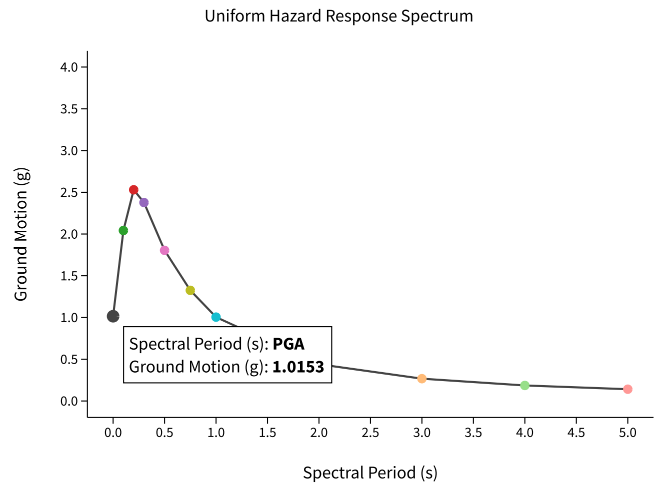
Time Horizon

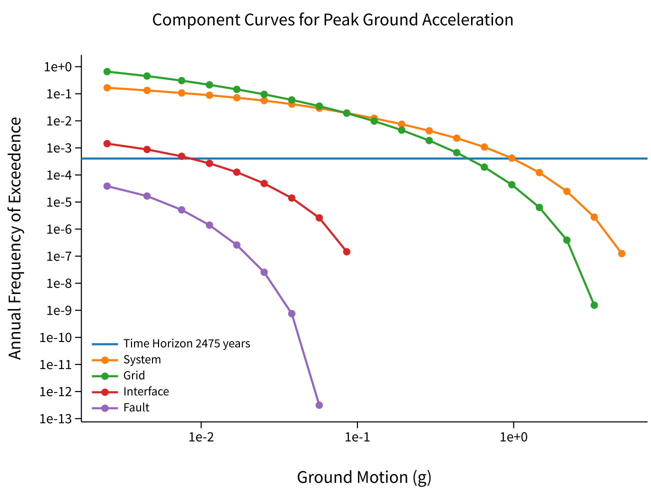
Return period in years

2475

Hazard Curve





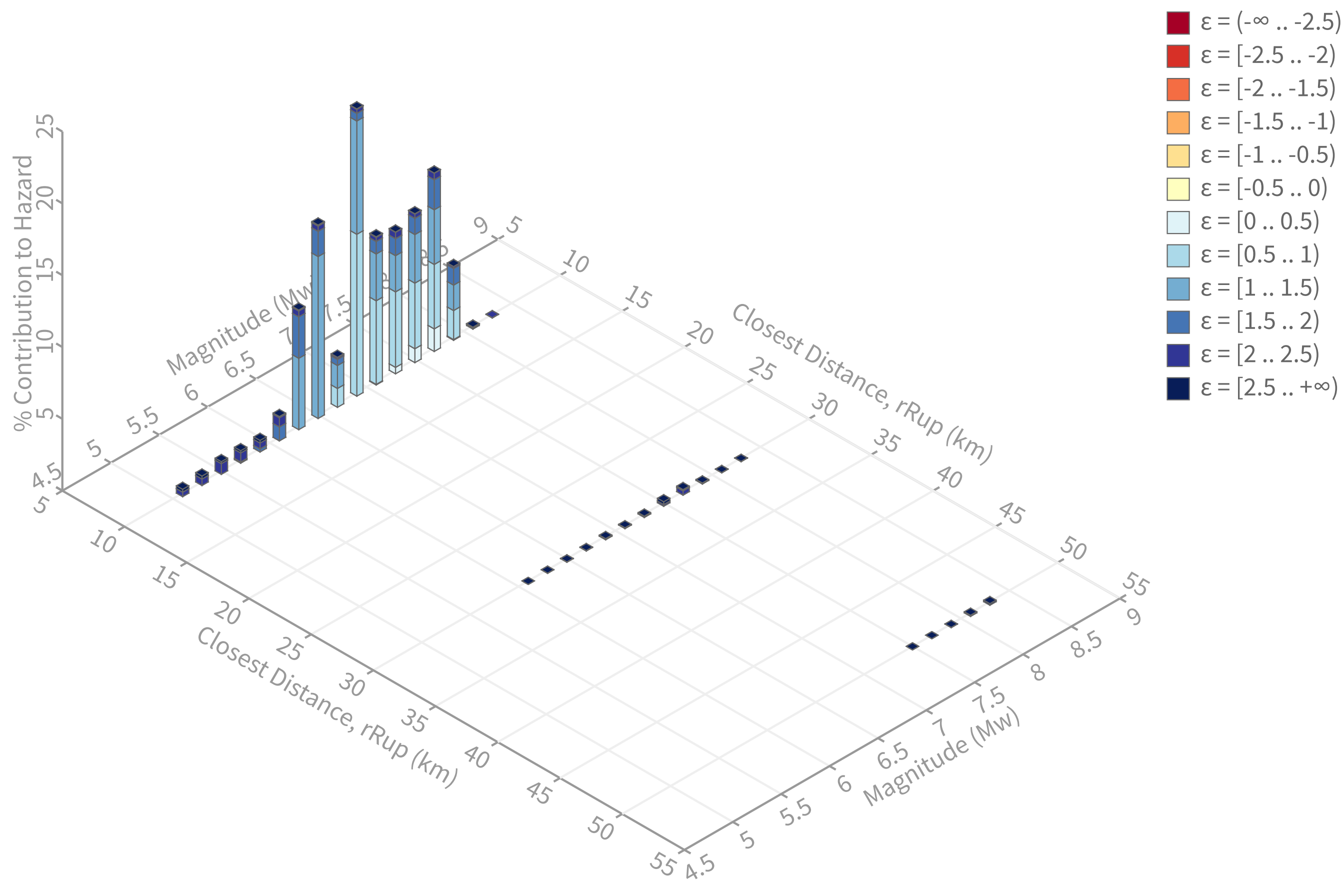


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.015337 g

Recovered targets

Return period: 2800.2563 yrs
Exceedance rate: 0.00035711017 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 7.02
r: 6.39 km
ε_0: 1.19 σ

Mode (largest m-r bin)

m: 6.9
r: 4.99 km
ε_0: 1.01 σ
Contribution: 19.99 %

Mode (largest m-r-ε_0 bin)

m: 6.55
r: 6.56 km
ε_0: 1.22 σ
Contribution: 11.27 %

Discretization

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

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							47.37
Hollywood [0]		3.39	7.27	0.84	118.225°W	34.127°N	196.24	12.37
Elysian Park (Upper) [2]		8.40	6.63	1.24	118.270°W	34.104°N	223.18	7.24
Elysian Park (Upper) [1]		8.96	6.35	1.44	118.239°W	34.081°N	195.14	5.93
Raymond [2]		3.52	6.80	1.10	118.214°W	34.124°N	176.03	5.74
Verdugo [1]		1.85	7.43	0.85	118.209°W	34.161°N	31.86	5.63
Sierra Madre [5]		8.24	7.66	1.50	118.179°W	34.219°N	24.90	3.20
Puente Hills [4]		10.16	7.06	1.12	118.291°W	34.073°N	218.19	2.51
UC33brAvg_FM32	System							45.21
Hollywood [0]		3.39	7.10	0.88	118.225°W	34.127°N	196.24	18.02
Raymond [2]		3.52	7.01	0.92	118.214°W	34.124°N	176.03	6.39
Verdugo [1]		1.85	7.41	0.86	118.209°W	34.161°N	31.86	4.97
Elysian Park (Upper) [1]		8.96	6.99	1.07	118.239°W	34.081°N	195.14	3.58
Sierra Madre [5]		8.24	7.69	1.50	118.179°W	34.219°N	24.90	3.05
Elysian Park (Upper) [2]		8.40	6.63	1.28	118.270°W	34.104°N	223.18	1.59
Puente Hills (LA) [1]		9.83	7.11	0.96	118.308°W	34.044°N	215.04	1.31
Puente Hills (Santa Fe Springs) [1]		16.12	7.05	1.85	118.144°W	33.926°N	165.18	1.28
Santa Monica alt 2 [0]		7.77	7.37	1.25	118.288°W	34.117°N	239.68	1.03
UC33brAvg_FM32 (opt)	Grid							3.87
PointSourceFinite: -118.216, 34.202		7.09	5.85	2.01	118.216°W	34.202°N	0.00	1.40
PointSourceFinite: -118.216, 34.202		7.09	5.85	2.01	118.216°W	34.202°N	0.00	1.40
UC33brAvg_FM31 (opt)	Grid							3.55
PointSourceFinite: -118.216, 34.202		7.20	5.80	2.04	118.216°W	34.202°N	0.00	1.30
PointSourceFinite: -118.216, 34.202		7.20	5.80	2.04	118.216°W	34.202°N	0.00	1.30

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

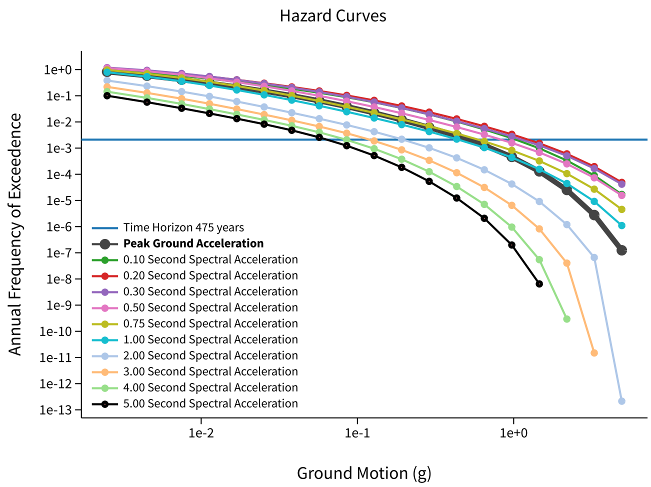
Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

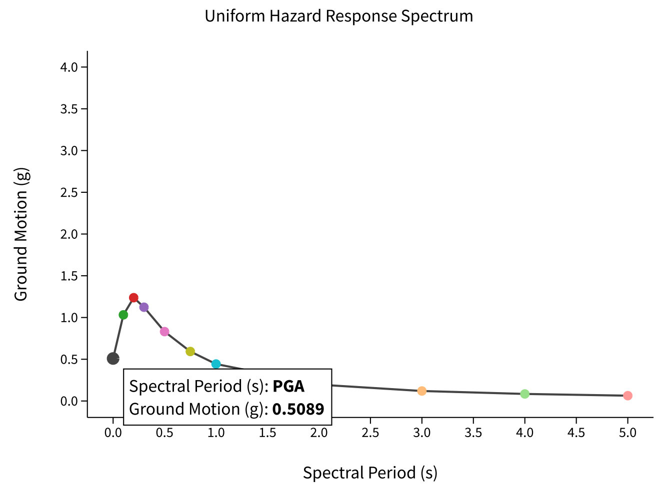
Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

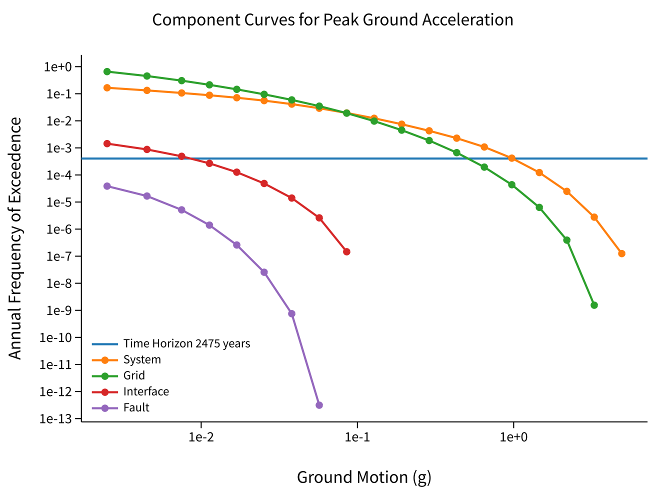
Input

Edition	Dynamic: Conterminous U.S. 2014 (u…
Spectral Period	Peak Ground Acceleration
Latitude (Decimal degrees)	34.152322
Time Horizon (Return period in years)	475
Longitude (Decimal degrees, negative values for western longitudes)	-118.215887
Site Class	537 m/s (Site class C)

Hazard Curve





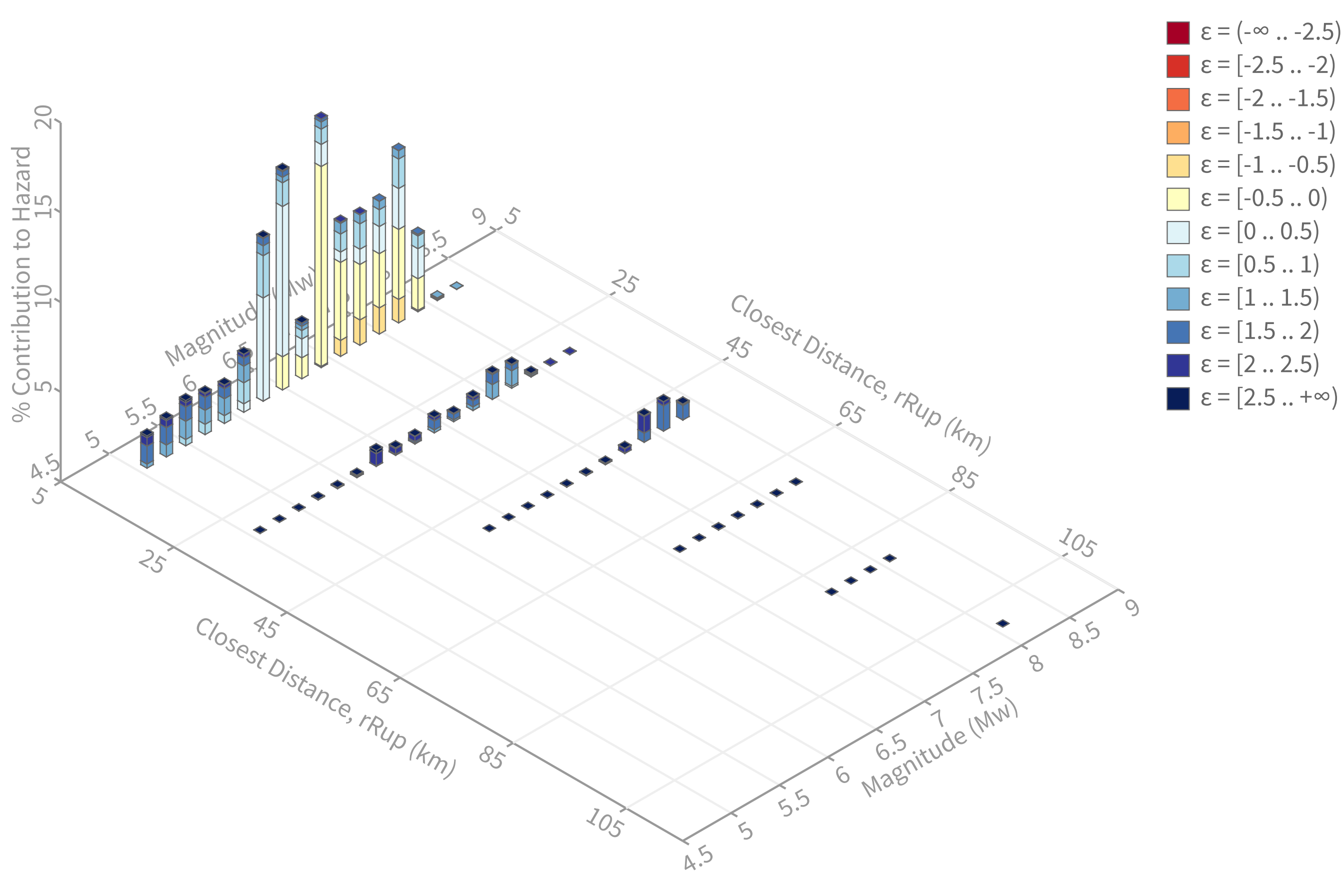


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.50894136 g

Recovered targets

Return period: 495.10145 yrs
Exceedance rate: 0.0020197881 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.1 %

Mean (over all sources)

m: 6.9
r: 10.55 km
ε_0: 0.5 σ

Mode (largest m-r bin)

m: 6.9
r: 5.79 km
ε_0: -0.09 σ
Contribution: 13.83 %

Mode (largest m-r-ε_0 bin)

m: 6.9
r: 4.66 km
ε_0: -0.24 σ
Contribution: 11.13 %

Discretization

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

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							42.46
Hollywood [0]		3.39	7.26	-0.34	118.225°W	34.127°N	196.24	7.16
Elysian Park (Upper) [2]		8.40	6.61	0.07	118.270°W	34.104°N	223.18	5.86
Elysian Park (Upper) [1]		8.96	6.35	0.31	118.239°W	34.081°N	195.14	5.73
Raymond [2]		3.52	6.77	-0.08	118.214°W	34.124°N	176.03	4.12
Sierra Madre [5]		8.24	7.64	0.35	118.179°W	34.219°N	24.90	3.38
Verdugo [1]		1.85	7.40	-0.30	118.209°W	34.161°N	31.86	3.34
San Andreas (Mojave S) [7]		43.94	8.05	1.93	118.001°W	34.506°N	26.58	2.03
Puente Hills [4]		10.16	7.04	-0.04	118.291°W	34.073°N	218.19	1.85
Compton [3]		19.25	7.40	0.78	118.414°W	33.865°N	209.76	1.46
UC33brAvg_FM32	System							39.36
Hollywood [0]		3.39	7.08	-0.30	118.225°W	34.127°N	196.24	10.76
Raymond [2]		3.52	6.96	-0.24	118.214°W	34.124°N	176.03	3.97
Sierra Madre [5]		8.24	7.66	0.35	118.179°W	34.219°N	24.90	3.21
Verdugo [1]		1.85	7.39	-0.29	118.209°W	34.161°N	31.86	2.97
Elysian Park (Upper) [1]		8.96	6.97	-0.09	118.239°W	34.081°N	195.14	2.52
San Andreas (Mojave S) [7]		43.94	8.05	1.93	118.001°W	34.506°N	26.58	2.04
Puente Hills (Santa Fe Springs) [1]		16.12	7.00	0.75	118.144°W	33.926°N	165.18	1.95
Compton [3]		19.25	7.48	0.73	118.414°W	33.865°N	209.76	1.38
Elysian Park (Upper) [2]		8.40	6.62	0.10	118.270°W	34.104°N	223.18	1.33
UC33brAvg_FM32 (opt)	Grid							9.34
PointSourceFinite: -118.216, 34.202		7.34	5.71	1.11	118.216°W	34.202°N	0.00	2.41
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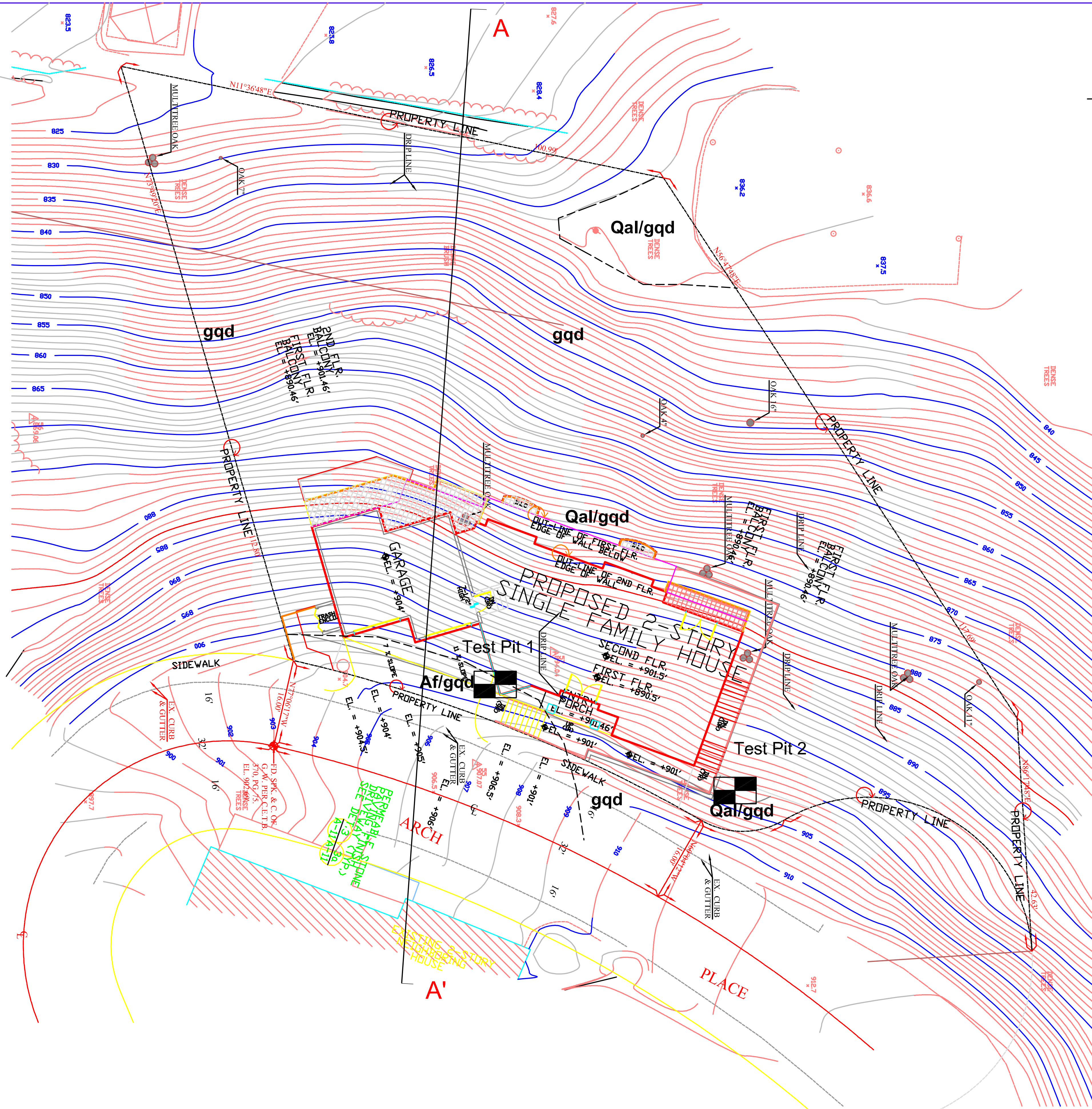
PointSourceFinite: -118.216, 34.238		9.94	5.85	1.43	118.216°W	34.238°N	0.00	1.08
UC33brAvg_FM31 (opt)	Grid							8.84
PointSourceFinite: -118.216, 34.202		7.41	5.68	1.14	118.216°W	34.202°N	0.00	2.30
PointSourceFinite: -118.216, 34.202		7.41	5.68	1.14	118.216°W	34.202°N	0.00	2.30



GEOTECHNICAL EXPLANATIO

Qal - Quaternary Alluvium

gqd - Bedrock - Quartz Diorite

- Approximate Geologic Con

TP-1 - Test Pit

A A' - Cross Section Line

Scale: 1" = 20'

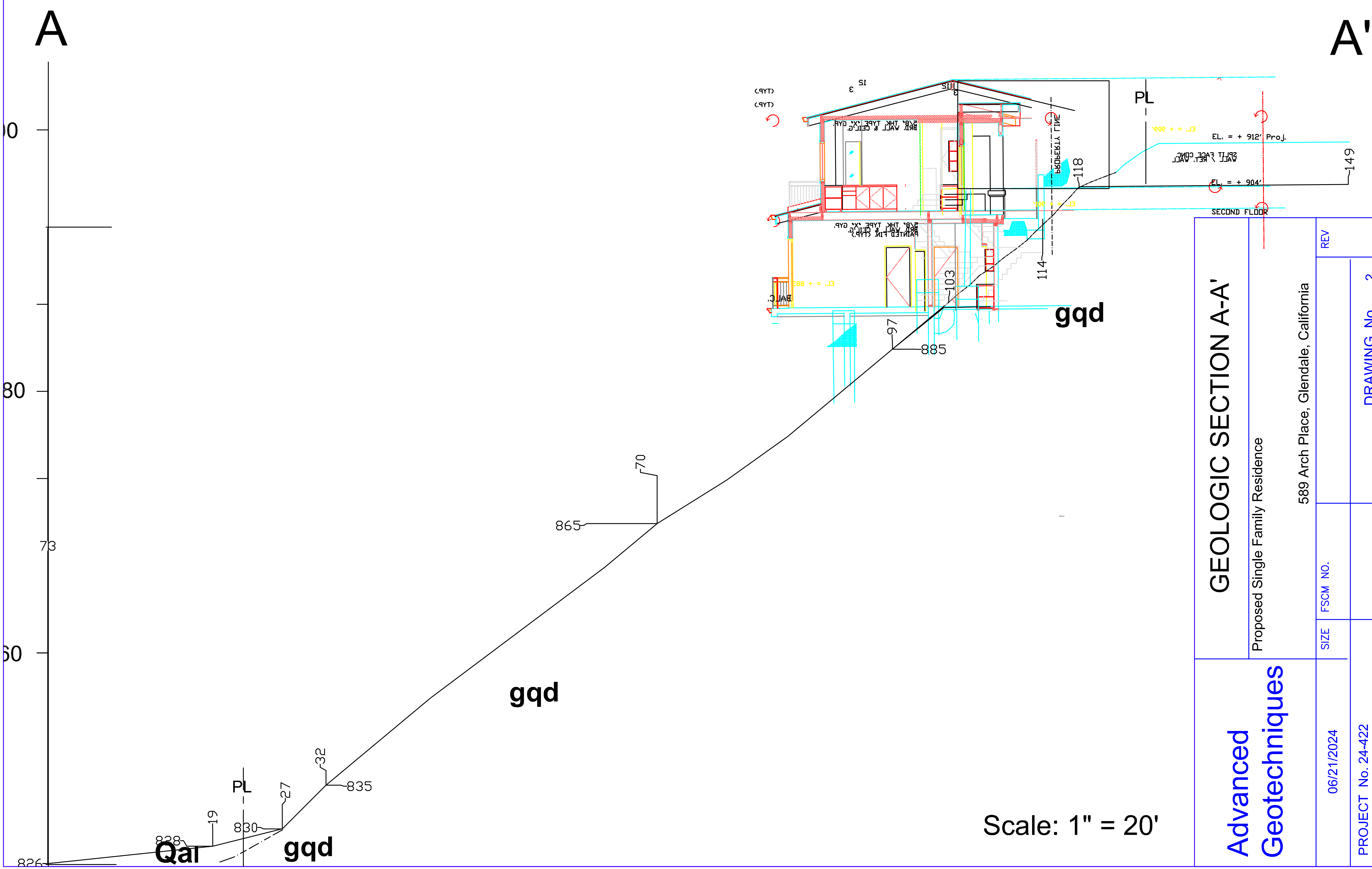
GEOLOGIC MAP & SITE PLAN

Proposed Single Family Residence

589 Arch Place, Glendale, California

Advanced Geotechniques

SIZE	FSCM NO.	REV
06/21/2024	PROJECT No. 24-422	DRAWING No. 1



GEOLOGIC SECTION A-A'

Proposed Single Family Residence

589 Arch Place, Glendale, California

Advanced Geotechniques

06/21/2024

PROJECT No. 24-422

DRAWING No. 2

REV

FSCM NO.

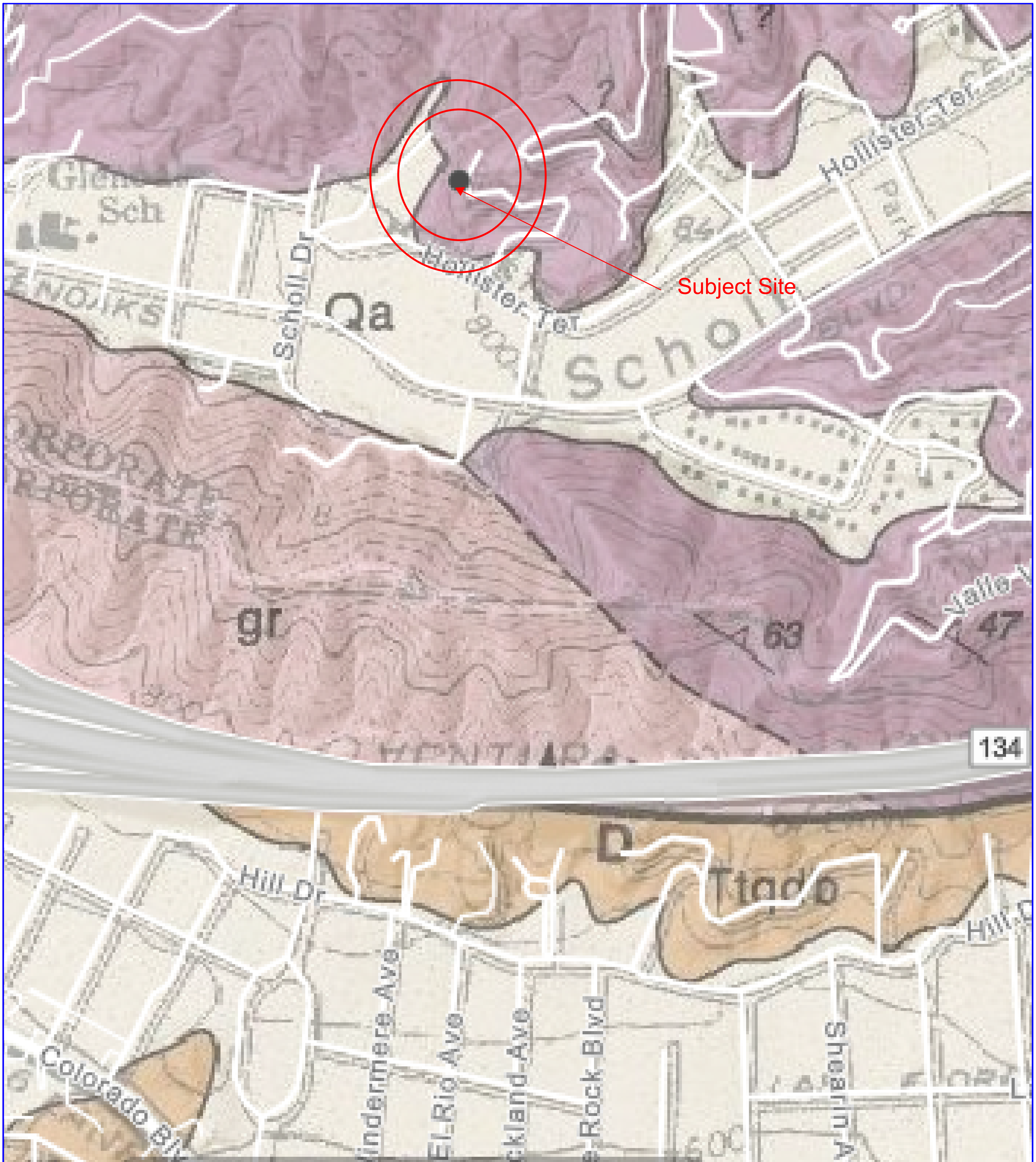
SIZE

0 39 78 Feet

Created in GIS-NET Public

Printed: 6/21/24

LA COUNTY PLANNING



N

ADVANCED GEOTECHNIQUES

VICINITY MAP

Portion of Geologic Map of Pasadena Quadrangle By Dibblee

Proposed Single Family Residence

589 Arch Place, Glendale, California

Date: 06/21/2024

PROJECT No. 24-422

Plate A

APPENDIX I

METHOD OF FIELD EXPLORATION

In order to define the subsurface conditions, two test pits were excavated on the site. Test pits were excavated by hand tools. The test pits, extended to depths of 10 feet, were excavated using a backhoe.

The approximate location of the excavated test pits and boring are shown on the enclosed Site Plan. Continuous logs of the subsurface conditions, as encountered in the test pits, were recorded during the field work and are presented on Plate No. I-1 within this Appendix. The plates also show the number and approximate depths of each of the recovered samples.

Relatively undisturbed samples of the subsurface materials were obtained by driving successive drops of a 36-pound metal weight free-falling a vertical distance of about 30 inches. The relatively undisturbed samples were retained in brass liner rings 2.5 inches in diameter and 1.0 inch in height.

Field investigation for this project was performed on May 3, 2024. The material excavated from the test pits and borings was placed back and compacted upon completion of the field work. Such material may settle. The owner should periodically inspect these areas and notify this office if the settlement creates a hazard to persons or property.

TEST PIT LOG

PROJECT NO. 24-422
LOGGED BY PP

SITE ADDRESS 589 Arch Place
DATE LOGGED 05/03/24

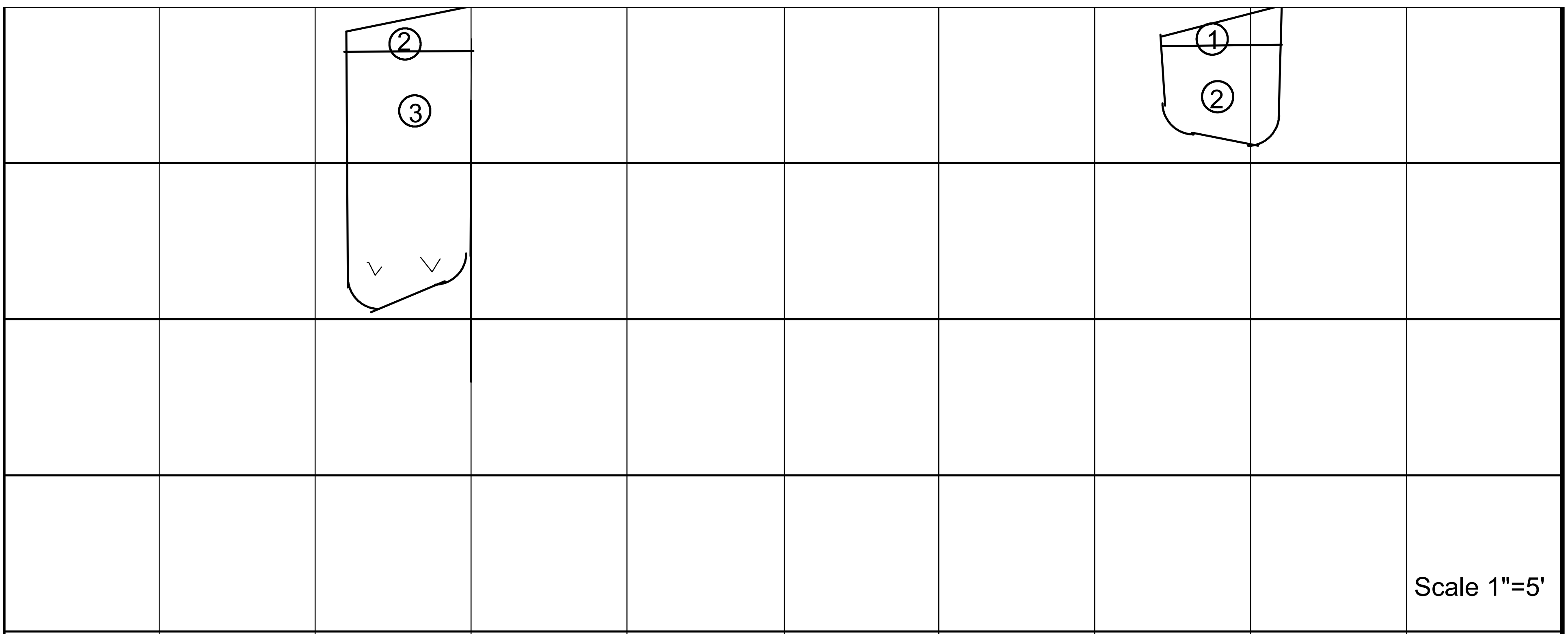
1) Artificial Fill (Af); Silty Sand, yellowish brown, moderately compact with abundant root structures, dry

2) Colluvium (Qc); Silty Sand, yellowish, brown, dense, dry, slightly porous, some gravel

3) Bedrock-quartz Diorite; tan, highly weathered, firm to tight, dense, dry, moderately fractured, massive, fractures are tight not continuos

TP-1

TP-2



APPENDIX II

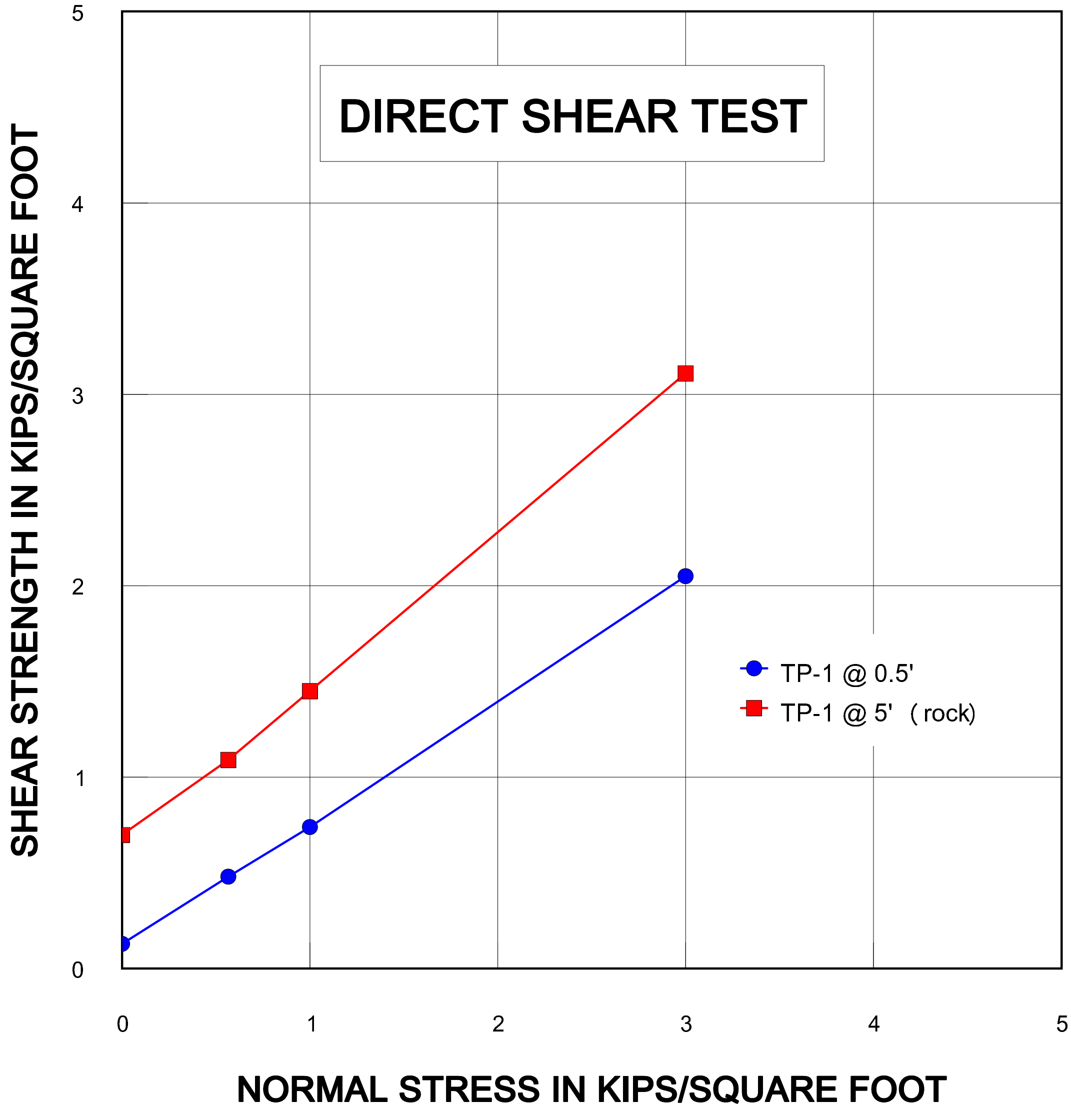
LABORATORY TESTING PROCEDURES

Moisture Density

The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The tests were performed using ASTM D2216-10 Laboratory Determination of water content Test Method. The dry unit weight and field moisture content were determined for each undisturbed sample.

Shear Tests

After the samples are saturated for 48 hours under initial confining pressure, the test is performed by deforming a specimen at a constant controlled strain rate on or near a single shear plane near the middle of selected sample between brass rings. Three specimens are tested, each under a different normal load, to determine the effects of shear resistance and displacement, and strength properties. This test determines the consolidated drained shear strength of the material (the internal angle of friction and the cohesion). The rate of shearing is slowed to ensure drained conditions (0.005 inches/min). The tests were performed using ASTM Laboratory Direct Shear Test Method. The Ultimate shear strength results of direct D3080-11 shear tests are presented on Plate No. II-1 within this Appendix.



Sample	γ pcf	ϕ deg.	C psf	Type of Sample	γ S pcf
TP-1 @ 0.5'	108	34	130	soil	130
TP-1 @ 5' (rock)	125	40	700	Rock	140

Project Name: Mr. Arsen Aghajanian   Project No. 24-422

Advanced Geotechniques

Plate II-1

STATEMENT

Graded properties are typically subject to potential geotechnical hazards including slumps, erosion, and concentrated runoff. It must be emphasized that responsible maintenance of these slopes, and the property in general, by the owner, using proper methods, can reduce the risk of these hazards.

Closing

A set of building and grading plans should be submitted to this office for review and approval prior to initiation of construction.

It is recommended that all foundation excavations be observed by this firm prior to placing concrete or steel. Any fill which is placed should be tested for compaction if used for engineering purposes.

The soils to be penetrated by the proposed excavation may vary significantly across the site. Preliminary information on vertical and lateral soil extent is based solely on the observations made at the test pits. The contractor should verify that similar conditions exist throughout the proposed excavation area. If different subsurface conditions from those described herein, are encountered at the time of construction, we recommend that we be contacted immediately to evaluate the conditions encountered.

It is advised that the client contact **Advanced Geotechniques,** at least **1 week** in advance of commencing grading to allow for contractual agreements for geotechnical services during the construction phases of your project.

Please advise this office at least 24 hours prior to any required verification.

Representatives of Advanced Geotechniques, will observe work in progress, perform tests on soil, and observe excavations and trenches. It should be understood that the contractor or others shall supervise and direct the work and they shall be solely responsible for all construction means, methods, techniques, sequences and procedures, and shall be solely and completely responsible for conditions of the job site, including safety of all persons and property during the performance of the work. We are providing this information solely as a service to our client. Under no circumstances should the information provided herein be interpreted to mean that Advanced Geotechniques is assuming the responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

Periodic observation by Advanced Geotechniques, is not intended to include verification of dimensions or review of the adequacy of the contractor's safety measures in, on, or near the construction site.

REMARKS

This report has been compiled for the exclusive use of Mr. Arsen Aghajanian. and their authorized representatives. It shall not be transferred to, or used by, a third party, to another project or applied to any other project on this site, other than as described herein, without consent and/or thorough review by this facility.

Should the project be delayed beyond the period of one year after the date of this report, the site should be examined and the report reviewed to consider possible changed conditions.

The owner and the contractor should make themselves aware of and become familiar with the applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to assure that the information and recommendations contained herein are called to the attention of the designers and builders for the project.

The limits of our liability for data contained in this report are presented on the following page.

Please call if you have any questions.

LIMITATIONS

This report is based on the development plans provided to our office. In the event that any significant changes in the design or location of the structure(s); as outlined on this report, are planned, the conclusions and recommendations contained in this report may not be considered valid unless the changes are reviewed and the conclusions of this report are modified or approved by the geotechnical engineer and engineering geologist in writing.

The subsurface conditions, excavations, characteristics and geologic structure described herein and shown on the enclosed cross section(s) have been projected from individual borings or test pits placed on the subject property. The subsurface conditions and excavation characteristics, and geologic structure should in no way be construed to reflect any variations which may occur between these borings or test pits.

It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, over-watering, and other factors not evident at the time measurements were made and reported herein. Advanced Geotechniques, assumes no responsibility for variations in groundwater levels that may occur across the site or in time.

If conditions encountered during construction appear to differ from those disclosed, this office shall be notified to consider the need for modifications. No responsibility for construction compliance with design concepts, specifications or recommendations is assumed unless on-site construction review is performed during the course of construction that pertains to the specific recommendations contained herein.

This report has been prepared in accordance with sound, generally accepted engineering practices common to the region. No warranties, either expressed or implied, are made regarding the professional advice provided under the terms of the agreement and included in this report.

This report is intended to aid your design professionals in their design of your project. Utilization of the advice presented herein is intended to reduce the risk associated with the construction projects. The professional opinions and geotechnical advise contained in this report are not intended to imply total performance of the project or guarantee that unusual conditions will not be discovered during or after construction.