

REPORT OF  
GEOTECHNICAL INVESTIGATION  
PROPOSED SINGLE FAMILY RESIDENCE  
1829 KIRKBY ROAD  
GLENDALE, CALIFORNIA

FOR  
MR. VAGRAM GALOUSTION

PROJECT NO. 16-470  
NOVEMBER 16, 2016

November 16, 2016

16-470

Mr. Vagram Galoustion  
1829 Kirkby Road  
Glendale, California 91208

Subject: Geotechnical Investigation  
Proposed Single Family Residence  
1829 Kirkby Road  
Glendale, California

Dear Mr. Galoustion:

### **INTRODUCTION**

This report presents the results of a geotechnical investigation for the subject project. During the course of this investigation, the engineering properties of the subsurface materials were evaluated in order to evaluate slope stability and to provide recommendations for design and construction of foundations and a retaining wall. The investigation included research, soil and rock 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 J. E. Gusman Engineering was used as reference. The map was dated January 12, 2015. In addition, a site plan showing the proposed addition by Vartan Jangozian & Associates was used as reference. The map was dated October 28, 2016.

The enclosed Drawing No. 1, shows approximate location of the obtained samples in relation to the site boundaries, existing building and the proposed construction areas. This drawing also shows the approximate location of the analyzed Cross Section A-A'. Drawing No. 2 shows the profile of the Cross Section A-A'.

The attached Appendix I, describes the method of field exploration. The attached Appendix II describes the laboratory testing procedures. Table I presents summaries of laboratory test results. Plate Nos. II-1 presents the results of direct shear tests performed on selected undisturbed samples.

## **PROJECT CONSIDERATIONS**

It is our understanding that the subject lot will be developed for a single family residence. The building is expected to be a two-story wood frame structure with a detached garage. The flooring system of the proposed building will be created in the form of terraces. As part of the creation of the terraces, some cutting will be made. The resulting vertical cuts will then be supported by retaining walls. Within the proposed building, the retaining walls will be integrated into the structure and will be part of the permanent building. The flooring system of the proposed building will be in a form of wooden raised floors. The approximate location of the proposed structures with respect to the site boundaries are shown on the enclosed Drawing No. 1.

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 50 kips, combined dead plus frequently applied live loads. The retaining wall footings are expected to have loads of on the order of 4 kips per lineal foot.

## **SITE GRADING**

Site grading is expected to basically involve minor cutting in order to create terrace-like finished grades. The excavated materials will then be used behind minor retaining walls on the downslope side of the residence.

## **SITE CONDITIONS**

### **SURFACE CONDITIONS**

The site is located on an south-facing slope in the northern portion of the San Rafael Hills in the City of Glendale, California. The site is an irregular shaped lot that is approximately 167 feet long in the maximum north-south direction and 170 feet long in the east-west direction. The lot is bound on all sides by developed lots with residences. The subject lot is also developed and is occupied by a one-story residence with, retaining walls above and below the residence and decking.

The south-facing ascending slope is a nearly 22-foot high slope with a 5:1 (horizontal to vertical) gradient. There are some retaining wall at the toe of this slope.

This slope is sparsely covered with low, non indigenous ground cover. Runoff from the slope is by sheetflow to the drain at the back of the wall.

## **GEOLOGIC AND SOIL CONDITIONS**

Observation of the on site cut slopes as well as review of published geologic maps indicates that the investigated portion of the lot is underlain by crystalline basement rock of Cretaceous age. A description of this unit is as follows:

**Artificial Fill (Af):** The existing fill consists of moderately compact, light-brown to brown, somewhat porous, slightly silty, fine to coarse grained sand containing variable amounts of gravel and cobble size clasts of granitic rock. The fill rests on natural soil and on rock. The existing fill generally covers the level pad to depths of two to 7 feet. However, in the area of our Test Pit No. 4, the fill was found to more than 7 feet thick. The bottom of the fill was not reached. The existing fill also should not be used for support of structural foundations.

**Colluvial Soil (Qsw):** The colluvial soil consists of medium dense, brown, somewhat porous, slightly silty, gravely, fine to coarse grained sand. This unit was encountered in our Test Pit Nos. 2.

**Basement Rock (gd) :** The basement rock consists of a very firm to moderately hard, yellow-brown to orange-brown, massive, medium-to fine-grained granodiorite. The rock is generally slightly to moderately fractured and moderately to highly weathered. This unit was encountered in our Test Pit Nos. 1, 2 and 3.

The rock was found to be generally very firm to hard. Such material is expected to provide adequate support for the proposed building and the detached garage.

No known faults cross or trend toward the site.

## **SCOPE OF WORK**

### **FIELD INVESTIGATION**

In order to define the subsurface conditions, four 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 7 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, and shear strength were determined from these tests. The laboratory test results are presented on Plate No. II-1, within Appendix II.

### **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.

## **LIQUEFACTION**

Based on our review of Seismic Hazard Map of the Pasadena 7.5' Quadrangle, the site does occur within the area of potential for liquefaction. The map has been developed for areas having primarily water table within 40 feet of the ground surface. Liquefaction phenomenon normally occurs where loose, saturated sand layers occur at shallow depths (normally less than 40 feet). The top zone of the site (within the foundation stress influence zone) consists of bedrock without groundwater, it is our

opinion that the chances of structural damages resulting from soil liquefaction is remote at the subject site.

## **SEISMIC DESIGN CONSIDERATIONS**

Based on soil properties, the site class is C. Ground motions (10% probability of being exceeded in 50 years) are expressed as a fraction of the acceleration due to gravity (g). The values of ground motion are determined from latitude and longitude (34.17164, -118.2253) utilizing the USGS Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration ASCE 7-10 Standard Ground motion values are **S<sub>Ms</sub> 0.2 sec 2.857g**, **S<sub>M1</sub> 1.0 sec 0.996g**, **S<sub>Ds</sub> = 1.905g**, and **PGA<sub>M</sub> = 1.062**.

## **EVALUATION AND RECOMMENDATIONS**

### **GENERAL**

Based on the geotechnical engineering data derived during this investigation, it is believed that the proposed construction at the subject site may be made as planned. It is our opinion that when the proposed addition is made, following the recommendations in this report, the site will be safe for the proposed structures against the hazard of liquefaction, landslide, settlement, or slippage. The proposed building will have no adverse influence on the geologic stability of properties outside of the project site.

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

Concrete grade slabs may be supported on properly compacted fill soils. In the areas of the building, therefore, any existing fill blanket should be excavated and properly recompacted for support of new grade slabs. Where grade slabs span between soil and rock, the rock should be over-excavated by some 12 inches and the excavated materials should be placed back as compacted fill. This will create a uniform

subgrade conditions beneath the concrete floors and reduce the chances of excessive uneven movements of the subgrade which would normally cause cracks on the grade slabs.

The following sections present our specific recommendations for foundations, lateral design, retaining walls, grade slabs, site grading, site drainage, and observations during construction.

### **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 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
5-12		1:1

---

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.

## **FOUNDATIONS**

The support system of the proposed retaining walls near the slope, should be in a form of deep foundations consisting of column pads, cast-in-place end-bearing caissons and/or friction piles. The end-bearing caissons shafts should have a minimum length of 5 feet. However, friction piles should have a minimum length of 10 feet. Any wall or retaining wall should have a horizontal setback of about  $H/3$  feet from the face of the descending slope surface.

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. All footings should be established at least 12 inches into rock and have a minimum depth of 18 inches.

An allowable maximum bearing value of on the order of 4,000 pounds per square foot may be used for continuous, 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 500 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 foundations in contact with the bedrock may be assumed to be the product of the dead load forces and a coefficient of friction of 0.30. Passive pressure on the face of foundations may also be used to resist lateral forces. A passive pressure of zero at the ground surface 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 the bedrock. For vertical shafts spaced greater than 2.5 times the pile diameter, the above given passive pressures may be doubled. For the purpose of moment calculations, the point of fixity of the vertical shafts may be taken at some 2 feet below the lowest unsupported outdipping bedrock surface. The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 15 feet.

The recommended value for passive could be used where the setback from the face of the rock is 5 feet. In order to get 5 feet setback from face of rock, the depth where the passive starts is below 3-5 feet in depth.

## **GRADE SLABS**

Concrete grade slabs may be supported on rock or properly compacted fill soils. Where grade slabs span between soil and rock, the rock should be over-excavated by some 12 inches and the excavated materials should be placed back as compacted fill. This will create a uniform subgrade conditions beneath the concrete floors and reduce the chances of excessive uneven movements of the subgrade which would normally cause cracks on the grade slabs.

For the purpose of this project, it is recommended that concrete grade slabs cast over soils have a minimum thickness of 5 inches and be reinforced with #4 bars placed at 24 inches on center.

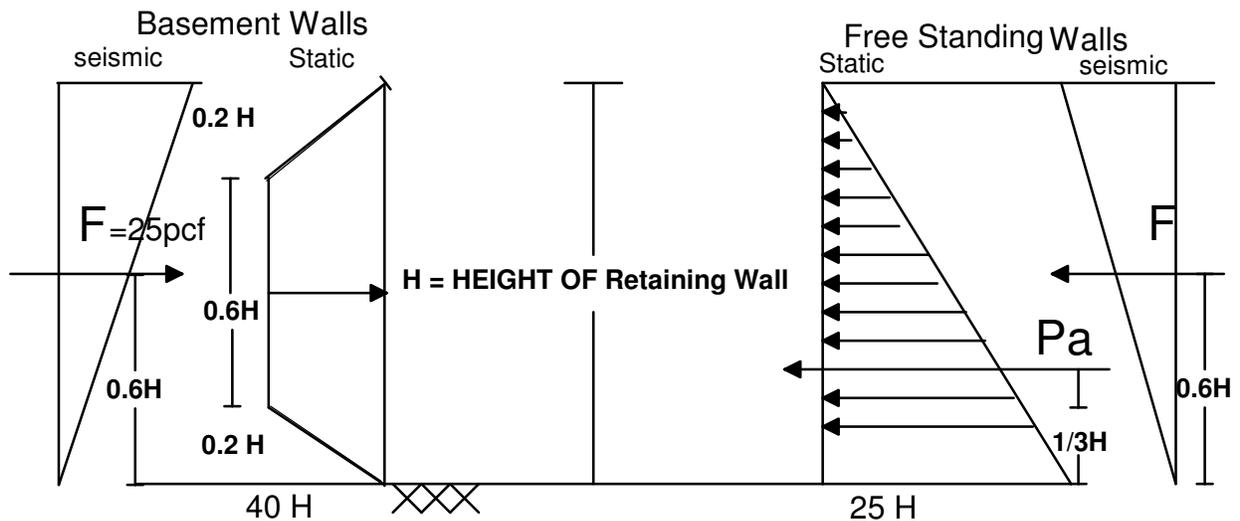
In the areas where moisture sensitive floor covering is used and slab dampness cannot be tolerated, a vapor barrier should be used beneath the slabs. This normally consists of a 10-mil polyethylene film covered with 2 inches of clean sand.

## RETAINING WALLS

As part of the proposed development, retaining walls will be constructed. Such walls are expected to be basically in a form of a cantilevered system with a maximum heights of 10 feet.

The perimeter walls of the basement of the proposed building are expected to be buried to maximum depths of about 10 feet. Static design of cantilevered retaining walls supporting cuts of compacted fill/ native soil with level ground conditions may be based on an equivalent fluid pressure of 35 pounds per cubic foot. Static design of these walls (being restrained against rotation designed at rest condition) could be based on an equivalent fluid pressure of 55 pounds per cubic foot. The effects of uniform surcharge may be computed using a coefficient of 0.30 times the assumed uniform loads.

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



It is recommended that the rear retaining wall which will be constructed against the hillside to have a free-board of at least 12 inches (above any structure and a paved drain to collect any minor debris washing down during rainy seasons). The free board should be designed as Impact wall based on an equivalent fluid pressure of 125 pounds per square foot per foot of depth. The free-board should then be cleaned after each rainy season.

## **SITE GRADING**

Site grading for the proposed project is expected to involve cutting in order to create the proposed finished grades. As part of site grading work, some wall backfilling will also be made.

All wall backfill should be granular in nature. The backfill should be properly benched into rock.

Prior to placing any fill, the Soil Engineer should observe the excavation bottoms. The areas to receive compacted fill should be scarified to a depth of about 8 inches, moistened as required to bring to approximately optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by the ASTM Designation D 1557-12 Compaction Method.

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. The excavated rock are considered to be satisfactory for reuse in the compacted fill areas and wall backfill. Rocks larger than 6 inches in diameter, however, should not be used as backfill.
2. 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 1557-12 for the material used.
3. 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.

4. 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.
5. 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 content is achieved.
6. Inspection and field density tests should be conducted by the Soil Engineer 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.

## **SITE DRAINAGE**

Site drainage should be provided to divert roof and surface waters from the property through non-erodible drainage devices to the street. In no case should the surface waters be allowed to pond behind the retaining walls or flow over the slope surfaces in an uncontrolled manner.

## **OBSERVATION DURING CONSTRUCTION**

The presented recommendations in this report assume that all structural foundations will be established in competent rock. 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. Please notify this office at least 24 hours before any observation/testing and/or inspection tasks are 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" test pits 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  
Site Plan-Drawing No. 1  
Vicinity Map -Drawing No. 2  
Appendix I Method of Field Exploration  
Plate No. I-1  
Appendix II Methods of Laboratory Testing  
Plate No. II-1

Respectfully Submitted,

**Advanced Geotechniques**



Peter Pailian  
Associate Engineer

Reviewed By:

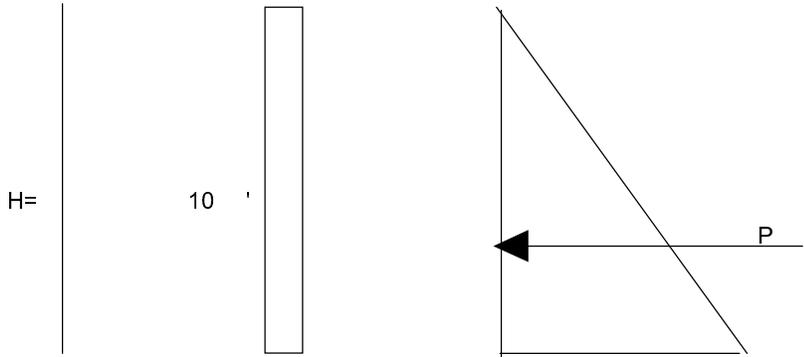


Ara Jitechian  
Civil Engineer  
CE 54893



PP/AJ

Distribution: (4)



**SOIL PARAMETERS**

$H = 10$  Feet      Height of retaining wall  
 $\gamma = 125$  PCF      Unit weight  
 $PGAm = 1.062$  g

$Kh = 1/2 PGAm / (2/3) = 0.354$

$F = 3/8 Kh \gamma H^2$

$F = 1.66$  kips

Resultent acting at a distance of  $(0.6 \cdot H)$  from base of wall

EQUIVALENT FLUID PRESSURE ( EFP ) =  $(2 \cdot UBF) / (h)^2$   
 Height of Retaining Wall = 10 Feet  
 $EFP = 33$  pcf      Use 33 PCF supporting Level grade

**SEISMIC LATERAL FORCE OF WALL**

Project Name: Mr. Vagram Tankazyan

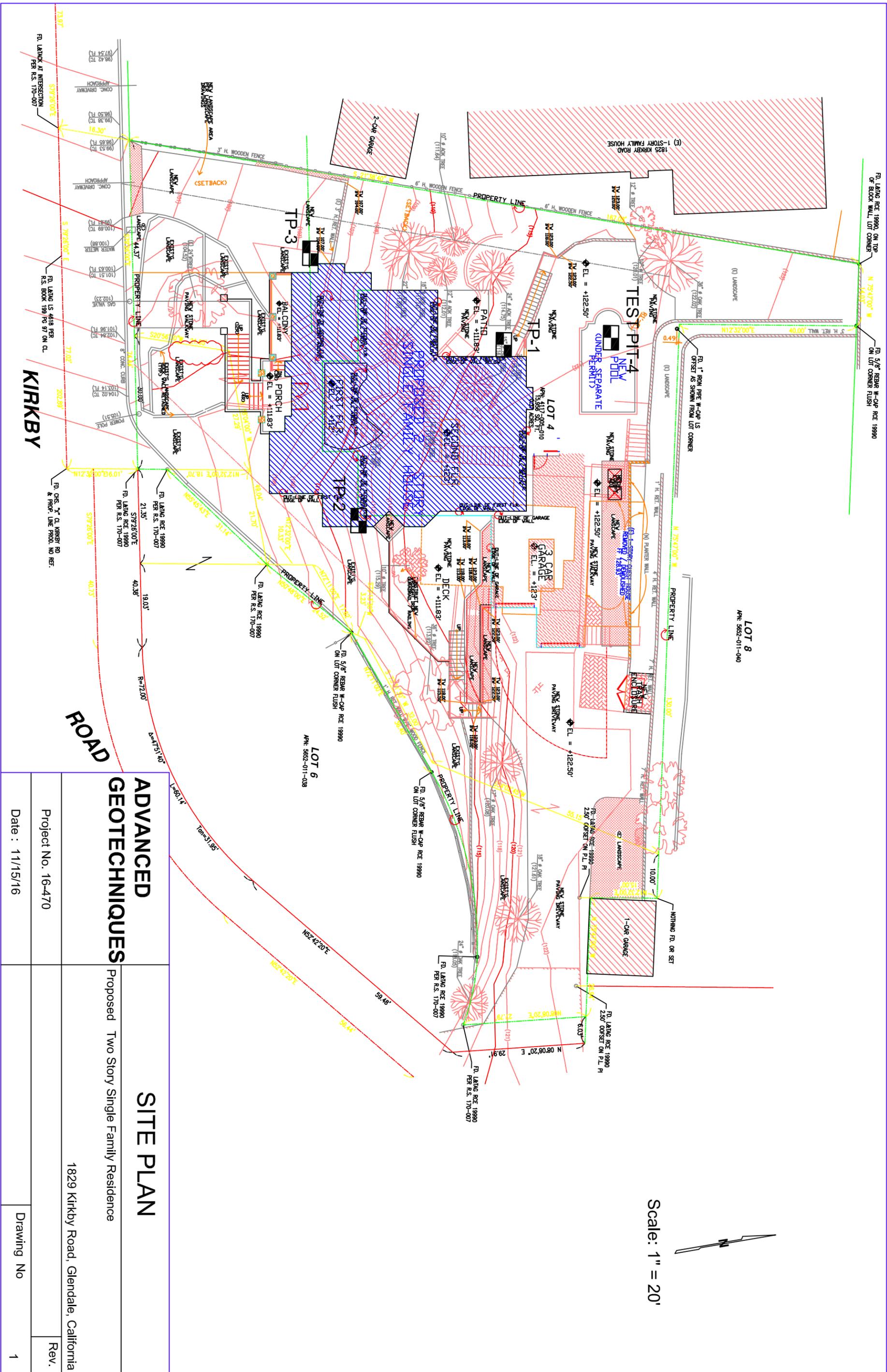
Date: 11/16/2016

Project No. 16-470



ADVANCED GEOTECHNIQUES  
 GEOTECHNICAL ENGINEERING CONSULTANTS

**SHEET No. 1**



LOT 8  
APN: 5652-011-000

LOT 6  
APN: 5652-011-008

Scale: 1" = 20'



**ADVANCED  
GEOTECHNIQUES**

**SITE PLAN**

Proposed Two Story Single Family Residence

1829 Kirkby Road, Glendale, California

Project No. 16-470

Date : 11/15/16

Drawing No

1

**KIRKBY**

**ROAD**

**TEST PT-4**  
NEW  
UNDER SEPARATE  
PERMIT

**LOT 4**  
APN: 5652-011-010  
0.850 ACRES

**3 CAR GARAGE**  
EL. = +123'

**DECK**  
EL. = +1183'

**1-CAR GARAGE**

**TP-3**

**TP-1**

**TP-2**

**TP-4**

**TP-5**

**TP-6**

**TP-7**

**TP-8**

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## **APPENDIX I**

### **METHOD OF FIELD EXPLORATION**

In order to define the subsurface conditions, four test pits were excavated to a depth of 8 feet. The approximate location of the test pits is shown on the enclosed Geologic Map & 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.

Relatively undisturbed soil samples were retained in brass liner rings 2.5 inches in diameter and 1.0 inch in height. The relatively undisturbed samples of the subsoil were obtained by driving a steel sampler with successive drops of a 10-pound sampling hammer free-falling a vertical distance of about 30 inches. The relatively undisturbed soil 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 November 9, 10 &14, 2016. The material excavated from the borings were 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.



## APPENDIX II

### LABORATORY TESTING PROCEDURES

#### Moisture Density

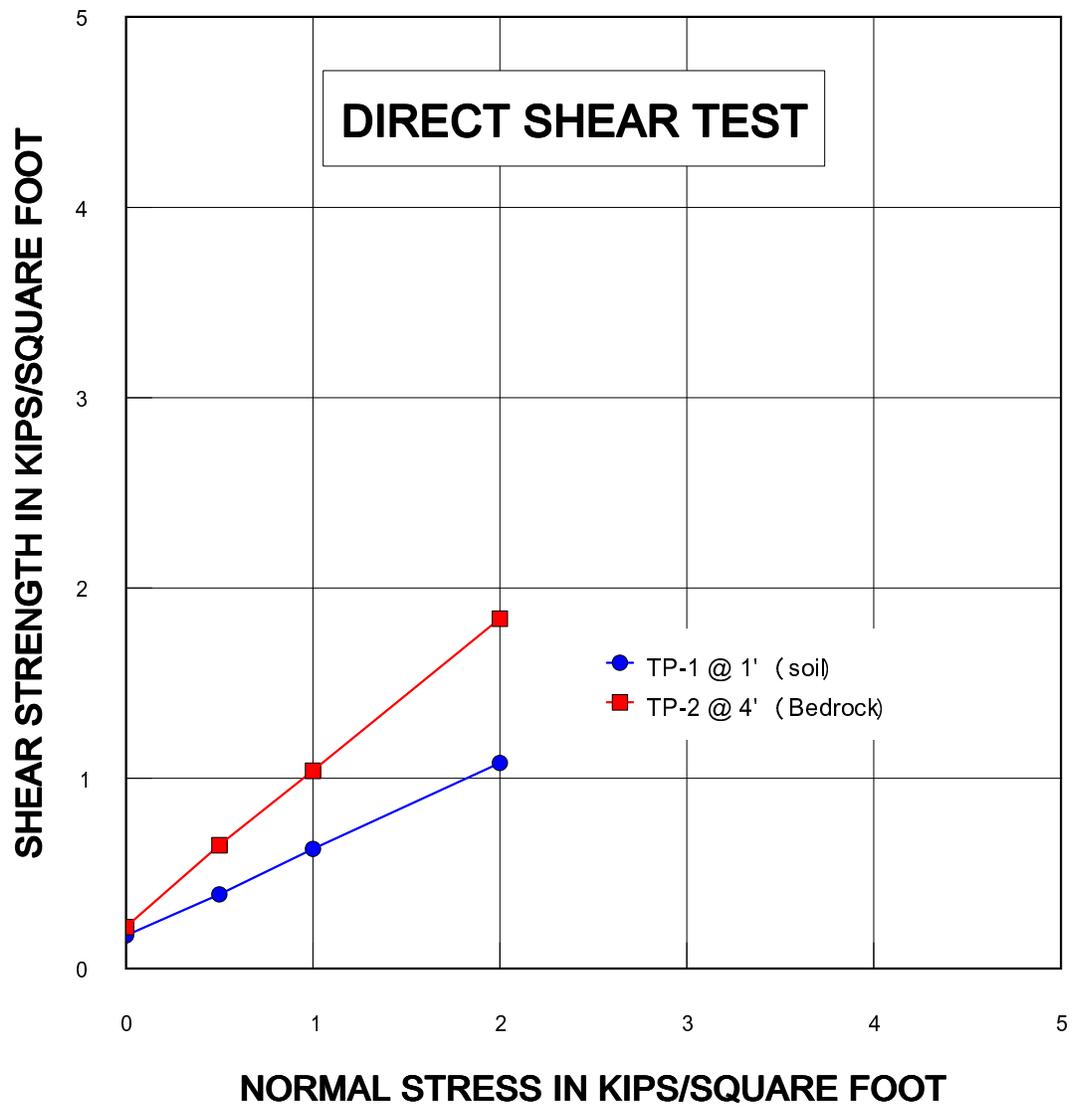
The moisture-density information provides a summary of soil consistency for each stratum and can also provide a correlation between soils found on this site and other nearby sites. The tests were performed using ASTM D2216-10 Laboratory Determination of water content Test Method. The dry unit weight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

#### SUMMARY OF LABORATORY TEST RESULTS

Sample No.	Depth (ft)	Dry density (pcf)	Moisture Content (% dry weight)
1	1'	108	4
2	2'	122	3

#### 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	$\gamma$ pcf	$\phi$ deg.	C psf	Type of Sample	$\gamma$ S pcf
TP-1 @ 1' ( soil)	109	24	175	Soil	133
TP-2 @ 4' ( Bedrock)	117	40	600	Bedrock	136

Project Name: Mr. Vagram Galoustion

Project No. 16-470



Advanced Geotechniques

Plate II-1

## STATEMENT

Hillside properties are typically subject to potential geotechnical hazards including slumps, mudflows, 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. Vagram Galoustion 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.

Respectfully,

**ADVANCED GEOTECHNIQUES,**

## 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 test pits 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 test pits 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 advice 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.



1829 Kirkby Road

Liquefaction Zone

1: 830

