REPORT OF

GEOTECHNICAL INVESTIGATION

PROPOSED SINGLE FAMILY RESIDENCE

LOTS 147, 148 AND 170 OF TRACT NO. 6759

APNS 5679-016-001, 5679-016-002 AND 5679-016-024

1248 CORONA DRIVE

GLENDALE, CALIFORNIA 91205

FOR

MR. EDUARDO J. CARRILLO

PROJECT NO. 19-327-22

APRIL 1, 2019

April 1, 2019 19-327-22

Mr. Eduardo J. Carillo
8207 Brookgreen Road
Downey, California 90240

Subject: Report Of Geotechnical Investigation
Proposed Single Family Residences
Lots 147, 148 And 170 Of Tract No. 6759
APNs 5679-016-001, 5679-016-002 And 5679-016-024
1248 Corona Drive
Glendale, California 91205

Dear Mr. Carillo:

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 temporary excavations, retaining walls, foundations, and grading. 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 the course of this investigation, the provided topographic survey map of the site and the plans were used as reference. The plans, dated March 23, 2019, were prepared by the offices of EC+Associates, Engineering.

The enclosed Geologic Map & Site Plan; Drawing No. 1, shows the 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 locations of the Geologic Cross Sections A-A' and B-B’. Drawing Nos. 2 and 3 show the profiles of the Geologic Cross Sections A-A' and B-B’.

Figure No. 1 show the Site Vicinity Map. Figure No. 2 shows the Regional Topographic Map. Figure No. 3 shows the Regional Geologic Map. Figure No. 4 shows the Historically Highest Groundwater (Contour Map).

The attached Appendix I, describes the method of field exploration. Figure Nos. I-1 through I-6 present summaries of the materials encountered at the location of our exploratory test holes. Figure No. I-7 presents a key to the log of exploratory test pits.

The attached Appendix II describes the laboratory testing procedures. Figure Nos. II-1 and II-2 present the results of direct shear and consolidation tests on selected undisturbed samples.

PROJECT CONSIDERATIONS

It is our understanding that the proposed project will consist of construction of a single family residence on the site. The proposed building is expected to be a 2-story structure constructed over basement garage beneath the northern end of the proposed building. The approximate location of the proposed building is shown on the enclosed Drawing No. 1. Cross Sections A-A' and B-B'; Drawing Nos. 2 and 3, show the profiles of the proposed building with respect to the existing and proposed finished grades.

It is expected that the finished grades will be created through mainly cutting operations. The resulting vertical cuts will then be supported by retaining walls of as high as 25 feet. Most of the retaining walls will be integrated into the proposed residence and will be part of the permanent structure. These walls will be designed as "restrained walls" (their tops will be restrained against rotation). The upper most retaining wall supporting the ascending slope will be designed as cantilevered system. This retaining wall will have a freeboard of at least 2 feet and a concrete paved drain (swale) to divert surface water and collect normal erosion debris which will be cleaned after rainy seasons. See the enclosed Cross Sections A-A' and B-B'.

Based on the results of our investigation, it is expected that the planned excavation will be made through mainly bedrock with minor amount of soil (fill and native) on the top. The bedrock will be free of throughgoing plane of weakness. On this basis, the earth-retaining structures for this project (temporary and permanent) can be designed based on normal lateral earth pressures.

Due to the magnitude of the planned depth of excavation to create the proposed finished grades, and in order to reduce the volume of over-excavation, temporary shoring should be used during the course of the site grading work. Such shoring system should consist of cantilevered soldier piles. The shallower excavations (within the front) exposing supported bedding can be made using unsupported/open cuts with gradients as recommended in this report.

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 120 kips, combined dead plus frequently applied live loads. The retaining wall footings are expected to have loads of on the order of 9 kips per lineal foot.

ANTICIPATED SITE GRADING WORK

It is expected that the site grading work will involve excavation (cutting operations) in order to create the proposed finished grades. As part of the site grading work, some wall backfilling will also be made within the over-excavation areas. The excavated site materials, broken down to acceptable pieces (less than 4 inches in diameter) can be used for wall backfilling. It is anticipated that, at the completion of the site grading work, significant volume of materials will be exported from the site.

SITE SURFACE CONDITIONS

The site consists of three lots; 147, 148 & 149 of Tract No. 6759), known as 1248 Corona Drive, Glendale, California. Access to the site is along Corona Drive.

The site is nearly rectangular in shape covering a surface area of about 8,743 square feet. The site is bounded by vacant lots to the north and south and by single family dwellings to the east.

The project consists of construction of a new, multilevel single family dwelling. Access to the residence will be directly from street along Corona Drive. Living spaces are planned along the second and third level, with garage at ground (at or near the street level). Some free standing retaining walls are planned for behind the dwelling to provide the required building setback from the ascending slope.

We are in receipt of a topographic survey, and site plan and cross section drawings of the proposed dwelling by the project designer, EC+Associates Engineering with updated drawings dated 03/23/2019, which we used a “base map” for preparation of our geologic map and cross section drawings.

REGIONAL GEOLOGY

The site is situated in the southern portion of the San Rafael Hills, part of the Transverse Ranges Geomorphic Province of California. The local bedrock underlying this area is assigned to the lower to middle Miocene-age Topanga formation marine sedimentary rocks consisting of sandstone and conglomerate with large clasts of diorite (see Figure No. 3 – Regional Geologic Map). Regional maps show the bedrock to range from massive to crudely stratified toward the north.

The site is located approximately 1 mile north of the inferred location of the Raymond Fault, which extends west-east roughly along the alignment of York Boulevard (see Figure No. 3- Regional Geologic Map). According to the Southern California Earthquake Center, this fault has ruptured in Holocene time, which indicates it is active. However, the property is not located in an Alquist-Priolo Earthquake Fault zone, and a fault study was not a part of this scope of work. Owners of properties near active faults are encouraged to protect their homes against damage from earthquakes and fault ruptures, such as by purchasing earthquake insurance.

GEOLOGIC AND SOIL CONDITIONS

Our geologic study consisted of the excavation/inspection/sampling and logging of six test pits, a review of published geologic maps and published reports, and on-site and near-site geologic reconnaissance and mapping. As part of our effort we advanced six test pits along the site and surface logged exposed bedrock outcrops. This activity indicates that the area of the proposed new dwelling and retaining wall is underlain by minor surficial fill, native soils and sedimentary sandstone bedrock. A Geologic Map and Site Plan is provided in Drawing 1, while Drawings 2 and 3 include Geologic Cross Sections A and B. Test pit logs are provided in Appendix I. A brief description of the units and their distribution are as follows:

Artificial Fill (Af): A thin veneer of fill material was found to overlie the soil and bedrock along our test pits and consists of medium to dark brown fine to medium grained gravelly silty sand. Debris and trash were encountered in Test Pit 4 (TP-4) and is presumed to be from the grading efforts from the buildings to the east. Maximum thickness of the fill was in the order of three feet, in Test Pit No. 3. However, it may differ along different parts of the site.

Soil (Qc): Mixed colluvial and residual soil consisting of highly weathered bedrock material and colluvial soil was found locally between the surficial fill and bedrock. It generally consists of a silty sand matrix with few fine gravel, slightly moist, medium dense to dense.

Sandstone and Conglomerate Bedrock (Ttqdb): Bedrock was encountered in all of the test pits. Bedrock consisted of thick bedded to massive sandstone conglomerate with medium to coarse, subangular to subrounded grains of diorite and shale. The bedrock was found to be moderately well indurated, crumbly to slightly friable, medium dense to dense, orange brown to medium brown, slightly moist. In our exploration and field mapping, we did not encounter bedding or stratification in the bedrock.

ENGINEERING-GEOLOGIC CONSIDERATIONS

Water was not found in the test pits to the maximum depth explored. There are no springs noted on-site, nor does the site have any surface streams passing through it.

Bedding attitudes were not found in our test pit exposures due to the massive and thick-bedded nature of the conglomerate and sandstone.

Fill, and residual soils are considered not suitable for support of any foundations. All foundations for the proposed retaining walls shall be established in bedrock.

From an engineering-geologic point of view, the proposed new dwelling project can proceed as planned, provided all proposed structures are founded in sandstone bedrock to sufficient depth, and with proper drainage; surface water runoff on the site is controlled; and preventive slope maintenance is regularly performed.

PREVENTIVE SLOPE MAINTENANCE

For the ascending slopes, and all slopes in general, it is important to reduce the risk of problems relating to slope instability. It is recommended that the owners implement a program of normal slope maintenance. This maintenance program should include annual clean out of drains, elimination of gophers and 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.

SEISMIC DESIGN CONSIDERATIONS

In accordance with the 2016 California Building Code (CBC 2016), the project site can be classified as site “C”. The mapped spectral accelerations of S_S=2.808 (short period) and S_1 =0.996 (1-second period) can be used for this project. These parameters corresponds to site Coefficients values of F_a =1.0 and F_V =1.3, respectively.

The seismic design parameters would be as follows:

Sms= Fa (Ss) = 1.0 (2.808) = 2.808
Sm1=Fv (S1) = 1.3 (0.996) = 1.295
Sds=2/3 (Sms) = 2/3 (2.808) = 1.872, and
Sd1=2/3 (Sm1) = 2/3 (1.295) = 0.863

EVALUATION AND RECOMMENDATIONS

GENERAL

Based on the geotechnical engineering data derived during this investigation, it is believed that the proposed construction may be made as planned. It is our opinion that when the proposed construction and grading are made, following the recommendations in this report, the site will be safe for the proposed structures against the hazard of landslide, settlement, or slippage.

It is anticipated that the planned excavation will be made through minor amount of soil (fill and native) and mainly massive sandstone bedrock. On this basis, the earth-retaining structures for this project (temporary and permanent) can be designed based on normal lateral earth pressures.

The planned excavation is expected to automatically eliminate the existing nonconforming cut slope condition within the area of the proposed construction. Outside the areas of construction, the existing steep cut slope should be shaved to inclination of no steeper than 1:1. Close to the side property lines, the cut slope should be gradually blended into the current inclination.

Bedrock will be exposed at the finished grades after the planned excavation in a form of terraces are made. The rock is expected to provide very good support for the proposed residence and the associated retaining walls through conventional spread footing. Piles will be used for support of high walls where temporary shoring is used.

The resulting vertical cuts from the planned grading (excavation) work will be supported by retaining walls. Most of the retaining walls will be integrated into the proposed building and will be part of the permanent structure. These walls will be designed as “restrained walls” (their tops will be restrained against rotation). The upper most retaining wall supporting the ascending slope will be designed as cantilevered system. This retaining wall will have a freeboard of at least 2 feet and a concrete paved drain (swale) to divert surface water and collect normal erosion debris which will be cleaned after rainy seasons. See the enclosed Cross Sections A-A’ and B-B’.

For support of high cuts, use of temporary shoring will reduce the volume of over-excavation and the subsequent backfilling. The shoring should consist of cantilevered soldier piles. The piles can then be incorporated into the retaining walls and be part of the permanent structures. The lower portions of the shoring piles (below the base of the excavation) can be used to provide vertical support through skin friction.

The results of our analysis indicated that the subject lot, with the planned grading work, will remain grossly stable with respect to deep-seated slope instability (having a factor of safety of greater than 1.5). See the enclosed engineering calculation sheets.

The soil cover (fill and native) above the proposed residence was is considered to be creep prone and subject to potential surficial instability. Due to limited slope area, a 2-feet high freeboard is considered to be adequate to retain debris associated with erosion. The freeboard should be cleaned after rainy seasons. The freeboard portion should be designed based on an equivalent fluid pressure of 125 pounds per square foot per foot of depth.

For the purpose of the subject project, it is recommended that all permanent slopes be covered with erosion resistant vegetation. A landscape architect may be consulted for selection of proper ground cover for the subject site.

Grade slabs may be cast directly over bedrock, or properly compacted fill soils. Where grade slabs span between soil and bedrock, the bedrock should be over-excavated by some 12 inches and the excavated materials could be used for the compacted fill (compacted to at least 90 percent relative compaction at optimum moisture content). This will create uniform subgrade conditions beneath grade slabs and reduce the chances of uneven subgrade movements. Because of granular nature of the site materials, soil expansion will not be an issue of this site. The grade slabs for this project, however, should be at least 5 inches thick and be reinforced with # 3 bars placed at every 18 inches on center.

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

TEMPORARY EXCAVATION

Unshored Excavations: It is expected that temporary excavations will be made during the course of site grading work to create the proposed finished grades. The excavation will be made through minor amount of native soils and massive sandstone bedrock.

Based upon the engineering characteristics of the subsurface materials, it is our opinion that temporary excavation slopes through soil and bedrock with supported bedding may be made in accordance with the following table:

Maximum Depth of Cut (FT)	Maximum Slope Ratio (Horizontal:Vertical) Soil	Bedrock
0-5	1/2:1	Vertical
5-12	1:1	Vertical
>12	1:1	1:1

It is recommended that the Engineering Geologist inspect the cut slopes within larger scale excavations as soon as five feet of bedrock is exposed in order to confirm the results of our findings. Modification to our recommendations may be necessary if variations are noted.

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 raveling 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 wash into the excavation.

Cantilevered Soldier Piles: In order to reduce the volume of over-excavation and the subsequent backfill, cantilevered soldier piles should be used as a means of temporary shoring for the higher cuts. Soldier piles consist of structural steel beams encased in concrete (below the basement garage level) and slurry mix within the exposed depths of excavation. For the purpose of this project, caisson type shoring piles with reinforcing cages can also be used. The caissons can be incorporated into the retaining walls and be part of the permanent structures. The lower portions of the shoring piles (below the base of the excavation) can be used to provide vertical support through skin friction.

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 7,000 pounds per square foot. The maximum center-to-center spacing of the vertical shafts should be maintained no greater than 12 feet.

For design of temporary support, active pressure on the shoring piles supporting cuts of bedrock with supported bedding may be computed using an equivalent fluid density of 30 pounds per cubic foot. The west-facing shoring piles supporting cuts of bedrock with daylighted bedding should be designed based on an equivalent fluid density of 35 pounds per cubic foot. Uniform surcharge may be computed using an active pressure coefficient of 0.30 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 12 inches below the base of the excavation. 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. Lagging may also be required in bedrock, if sloughing is experienced on the exposed granular bedrock between the piles.

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

GRADING RECOMMENDATIONS

The major portion of the site grading work will involve excavation (cutting operation) in order to create the proposed finished grades. Some wall backfilling will also be made in the over-excavation areas behind the retaining walls. All wall backfill should be granular in nature.

Prior to placing any fill, the Soil Engineer and Engineering Geologist should observe the excavation bottoms. The areas to receive fill should be scarified and compacted in-place to a relative compaction of at least 90 percent at 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 before site grading work is initiated;

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 4 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 for the material used. All new fill should be benched into rock;

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 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 walls or flow over the slope surfaces in an uncontrolled manner. A minimum surface slope of one and two percent should be maintained in paved and unpaved areas, respectively.

The site drainage recommendations should also include the following:

1. Having positive slope away from the buildings, as recommended above;
2. Installation of roof drains, area drains and catch basins with appropriate connecting lines;
3. Managing landscape watering;
4. Regular maintenance of the drainage devices;
5. Installing waterproofing or damp proofing, whichever appropriate, beneath concrete grade slabs and behind the basement walls.
6. The owners should be familiar with the general maintenance guidelines of the City requirements.

FOUNDATIONS

It is anticipated that, after the planned excavation is made, rock will be exposed at the finished grades. The rock is expected to provide very good support for the proposed residence and the associated retaining walls through conventional spread footing foundation system.

The retaining wall footings should be at least 24 inches wide and should be established at least 24 inches into bedrock. The footings of the proposed residence should be at least 18 inches wide and should be established at least 18 inches into bedrock.

It should be noted that the above recommended foundation dimensions are the minimum required. The actual foundation dimensions may be greater depending upon the magnitude of the imposed loads.

Properly designed and constructed spread footings established in bedrock may be based on allowable maximum bearing pressure of 6,000 pounds per square foot.

For the purpose of estimating vertical capacity of individual piles, an allowable maximum skin friction value of 750 pounds per square foot may be used for the top 10 feet of the bedrock. The allowable maximum skin friction value can be increased to 900 pounds per square foot for the portion of piles extended deeper than 10 feet into bedrock. Uplift capacity may be assumed one half of the downward capacity.

The above given allowable maximum bearing and skin friction values are for the total of dead, plus frequently applied live loads. For short duration transient loading; wind or seismic forces, the given value may be increased by one third.

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, should not be left open overnight.

During the course of our field investigation, no caving was experienced in the test holes. On this basis, caving is expected not to occur within drilled holes. If the foundations are excavated with hand tools, proper shoring should be implemented for workmen safety where soil is exposed.

Total and differential settlements of the proposed residence and the associated retaining walls (with foundations established in rock) are expected to be within tolerable limits; less than 3/8 and 1/4 of one inch, respectively. The major portion of the settlements are expected to occur during construction.

LATERAL DESIGN

Lateral resistance at the base of footings in contact with bedrock may be assumed to be the product of the dead load forces and a coefficient of friction of 0.4. Passive pressure on the face of footings or developed against the vertical shafts, may also be used to resist lateral forces. For the purpose of the subject project, a passive pressure of 300 pounds per square foot at the surface of bedrock and increasing at a rate of 300 pounds per square foot per foot of depth to a maximum value of 3,500 pounds per square foot may be used.

It should be noted that, if the individual shafts are spaced at least 2.5 times the pile diameters (isolated shafts) the above given values can be doubled. For the purpose of moment calculations, the point of fixity of the vertical shafts on slope may be taken some 12 inches below the surface of the bedrock.

GRADE SLABS

Grade slabs may be cast directly over bedrock, or properly compacted fill soils. Where grade slabs span between soil and bedrock, the bedrock should be over-excavated by some 12 inches and the excavated materials could be used for the compacted fill (compacted to at least 90 percent relative compaction at optimum moisture content). This will create uniform subgrade conditions beneath grade slabs and reduce the chances of uneven subgrade movements. Because of granular nature of the site materials, soil expansion will not be an issue of this site. The grade slabs for this project, however, should be at least 5 inches thick and be reinforced with # 3 bars placed at every 18 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 6-mil polyethylene film covered with 2 inches of clean sand.

RETAINING WALLS

As part of the site grading work, many retaining walls will be constructed. Such walls are expected to be designed as restrained (within the building) and cantilevered outside the building. Maximum height of the restrained walls are expected to be on the order of 25 feet. The vertical heights of the cantilevered retaining walls are expected to be on the order of 10 feet.

Static design of cantilevered retaining walls supporting cuts of massive bedrock may be based on an equivalent fluid pressure of 35 pounds per square foot per foot of depth. The retaining walls that are restrained against rotation at top should be based on an equivalent fluid pressure of 44 pounds per square foot per foot of depth. The cantilevered retaining walls supporting ascending slope should be designed based on an equivalent fluid pressure of 45 pounds per square foot per foot of depth. The freeboard section of the cantilevered retaining wall should be designed based on an equivalent fluid density of 125 pounds per cubic foot.

It is noted that, based on the new Code requirement, the basement walls should be designed not only for static, but also for seismic lateral earth pressures. For the purpose of this project, the magnitude of seismic lateral earth pressure should be maximum at the ground surface and decrease at a rate of 36 pounds per square foot per foot of depth to a value of zero at the base of the retaining wall (see the enclosed supporting engineering calculations). The point of application of the lateral thrust of the seismic pressure should be assumed 0.6 time the wall height, measured from the bottom of the wall.

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. The backdrain system should consist of 4-inch diameter perforated pipes encased in free draining gravel; at least one cubic foot per lineal foot of the pipe.

The retaining walls supporting all ascending slope should have a minimum freeboard of 2 feet and a paved drain to collect minor debris washed down during rainy season. The freeboard should then be cleaned after rainy seasons.

EXCAVATION

During the course of our field investigation, site materials, soils and upper bedrock were excavated using hand tools. Such materials were explored without significant difficulty. It is anticipated, however, that when excavating deeper bedrock, coring and/or use of jackhammer will be required.

OBSERVATION DURING CONSTRUCTION

The presented recommendations in this report assume that all structural foundations (footings and piles) will be established in bedrock. All foundation excavations should be observed and approved by a representative of this office, before the reinforcing is placed. 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. All shoring piles should be inspected by a Grading Deputy.

The project engineering geologist should observe the temporary cut slopes. Modification to our recommendations may be necessary if significant variations are noted in the geologic features of the underlying bedrock.

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
Drawing No. 1 - Geologic Map & Site Plan
Drawing No. 2 - Geologic Cross Section A-A'
Drawing No. 3 - Geologic Cross Section B-B'
Figure No. 1 - Site Vicinity Map
Figure No. 2 - Regional Topographic Map
Figure No. 3 - Regional Geologic Map
Figure No. 4 - Historically Highest Groundwater (Contour Map)
Appendix I - Method of Field Exploration
Figure Nos. I-1 through I-7
Appendix II - Methods of Laboratory Testing
Figure Nos. II-1 and II-2

Respectfully Submitted,
APPLIED EARTH SCIENCES

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CJM/SM/la

Distribution: (3)

Bedrock Strength Parameters

Saturated Unit Weight = γs = 133 pcf

Value of Fiction Angle = ϕ = 42 °

$$K_o = 1 - \sin(\phi)$$

$$K_o = 1 - \sin 42^{o}$$

$$K_o = 1 - 0.67$$

$$K_o = 0.33$$

$$\gamma_o = K_o * \gamma$$

$$\gamma_o = 0.33 * 133$$

$$\gamma_o = 44.0$$

At-Rest Equivalent Fluid Density, $\boxed{\gamma_o = \mathbf{44}\ \text{PCF}}$

AT-REST LATERAL EARTH PRESSURE

Basement Walls

FOR: 1248 Corona Drive	**DATE:** 4/1/19	**PROJECT NO.:** 19-327-22
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		**CALC SHEET No. 1**

<u>Bedrock Strength Parameters</u>

Saturated Unit Weight	ϒ=	133	PCF
Height of Wall	H=	26	Ft.
	PGA_M=	1.077	

* FIGURE 2 of Naval Facilities Engineering Command

$$P_{AE} = \frac{3}{8}\gamma H^2(K_h) \qquad *7.2\text{-}78$$

$$K_h = \frac{\frac{2}{3} * PGA_M}{2}$$

K_h= 2/3 * 1.077 / 2

K_h= 0.36

P_{AE} = 3/8 * 133 * 676 * 0.36

P_{AE} = 12104 lb.

<u>Equivelent Fliud Pressure (EFP)</u>

$$EFP = (\frac{2xP_{AE}}{H^2})$$

EFP= 2 * 12104 / 676

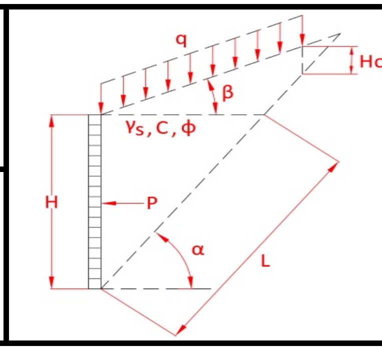
EFP= 35.81 PCF

SEISMIC LATERAL EARTH PRESSURE

Retaining Walls

FOR: 1248 Corona Drive	**DATE:** 4/1/19	**PROJECT NO.:** 19-327-22
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		**CALC SHEET NO. 2**

Bedrock Strength	Unit Weight	γ_s	133	PCF
	Cohesion	C	650	PSF
	Friction Angle	φ	42	°
	Estimated Failure Surface Angle	α	51	°
Retaining Wall Parameters	Height of Wall	H	26	ft.
	Average Inclination of Ground Surface Above Wall	β	31	°
	Assumed Surcharge Load	q	300	PSF



Moblized Strength	Factor of Safety	F.S.	**1.25**	**1.5**	
	Mobilized Cohesion	$C_m = c/F.S.$	520	433.33	PSF
	Mobilized Friction Angle	$\varphi_m = \tan^{-1}(\tan\varphi/F.S.)$	36	31	°
Tension Crack	Coefficient of Lateral Earth Active Pressure	$K_a = \tan^2(45^o-(\varphi_m/2))$	0.26	0.32	
	Height of Tension Crack	$H_c=(2C_m)/[(\gamma_s)(K_a^{0.5})]$	15.3	11.5	ft.

For F.S = 1.25 (Temporary Conditions)	Failure Surface Angle (α > β)	α	46	51	56	°
	Length of Potential Sliding Surface Across Wedge	$L = \frac{(H-H_C)\cos\beta}{sin(\alpha-\beta)}$	35.54	26.89	21.76	ft.
	Weight of Soil in Wedge Above Potential Sliding Area	$W = 0.5\gamma_S\left[HL + \frac{H_C(H-H_C)cos\beta}{sin(\alpha-\beta)}\right] * cos\alpha$	67750	46447	33400	lb.
	Additional Lateral Load	$E= (K_a qL \cos\alpha)/(\cos\beta)$	1664.8	1141.3	820.7	lb.
	Resultant Horizontal Force	$P= (W-c_m L \sin\alpha)(\tan(\alpha-\Phi_m))-c_m L\cos\alpha+E$	-1340.5	2030	3345	lb.
	Equivalent Fluid Density	$G_h=2P/H^2$	*-4.0*	*6.0*	*9.9*	PCF

For F.S = 1.5 (Permanent Conditions)	Failure Surface Angle (α > β)	α	46	51	56	°
	Length of Potential Sliding Surface Across Wedge	$L = \frac{(H-H_C)\cos\beta}{sin(\alpha-\beta)}$	47.98	36.31	29.39	ft.
	Weight of Soil in Wedge Above Potential Sliding Area	$W = 0.5\gamma_S\left[HL + \frac{H_C(H-H_C)cos\beta}{sin(\alpha-\beta)}\right] * cos\alpha$	83147	57002	40991	lb.
	Additional Lateral Load	$E= (K_a qL \cos\alpha)/(\cos\beta)$	2746.4	1882.8	1354.0	lb.
	Resultant Horizontal Force	$P= (W-c_m L \sin\alpha)(\tan(\alpha-\Phi_m))-c_m L\cos\alpha+E$	6605.9	8299	8441	lb.
	Equivalent Fluid Density	$G_h=2P/H^2$	*19.5*	*24.6*	*25.0*	PCF

EFD	For Temporary Wall Design, Use Equivalent Fluid Density	G_h=	30	PCF
	For Permanent Wall Design, Use Equivalent Fluid Density	G_h=	35	PCF

LATERAL EARTH PRESSURE CALCULATIONS

SECTION B-B' - SOUTH FACING RETAINING WALLS

FOR: 1248 Corona Drive | **DATE:** 4/1/19 | **PROJECT NO.:** 19-327-22

APPLIED EARTH SCIENCES
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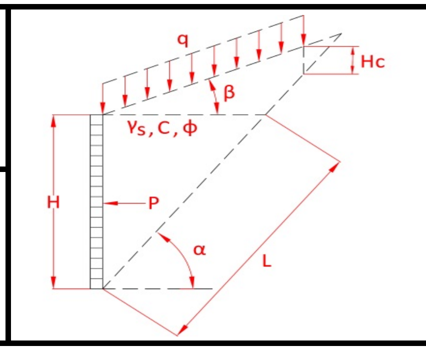
TABLE No. 1

Bedrock Strength	Unit Weight	γ_s	133	PCF
	Cohesion	C	650	PSF
	Friction Angle	φ	42	°
	Estimated Failure Surface Angle	α	66	°
Retaining Wall Parameters	Height of Wall	H	26	ft.
	Average Inclination of Ground Surface Above Wall	β	31	°
	Assumed Surcharge Load	q	300	PSF



Moblized Strength	Factor of Safety	F.S.	**1.25**	**1.5**	
	Mobilized Cohesion	$C_m = c/F.S.$	520	433.33	PSF
	Mobilized Friction Angle	$\varphi_m = \tan^{-1}(\tan\varphi/F.S.)$	36	31	°
Tension Crack	Coefficient of Lateral Earth Active Pressure	$K_a = \tan^2(45^o-(\varphi_m/2))$	0.26	0.32	
	Height of Tension Crack	$H_c = (2C_m)/[(\gamma_s)(K_a^{0.5})]$	15.3	11.5	ft.

For F.S = 1.25 (Temporary Conditions)	Failure Surface Angle (α > β)	α	61	66	71	°
	Length of Potential Sliding Surface Across Wedge	$L = \frac{(H-H_C)\cos\beta}{\sin(\alpha-\beta)}$	18.40	16.04	14.31	ft.
	Weight of Soil in Wedge Above Potential Sliding Area	$W = 0.5\gamma_S\left[HL + \frac{H_C(H-H_C)\cos\beta}{\sin(\alpha-\beta)}\right] * \cos\alpha$	24476	17900	12785	lb.
	Additional Lateral Load	$E = (K_a qL\cos\alpha)/(\cos\beta)$	601.4	439.8	314.2	lb.
	Resultant Horizontal Force	$P = (W-c_m L\sin\alpha)(\tan(\alpha-\Phi_m)) - c_m L\cos\alpha + E$	3556.0	3041	1953	lb.
	Equivalent Fluid Density	$G_h = 2P/H^2$	*10.5*	*9.0*	*5.8*	PCF

For F.S = 1.5 (Permanent Conditions)	Failure Surface Angle (α > β)	α	61	66	71	°
	Length of Potential Sliding Surface Across Wedge	$L = \frac{(H-H_C)\cos\beta}{\sin(\alpha-\beta)}$	24.84	21.65	19.32	ft.
	Weight of Soil in Wedge Above Potential Sliding Area	$W = 0.5\gamma_S\left[HL + \frac{H_C(H-H_C)\cos\beta}{\sin(\alpha-\beta)}\right] * \cos\alpha$	30038	21968	15691	lb.
	Additional Lateral Load	$E = (K_a qL\cos\alpha)/(\cos\beta)$	992.2	725.6	518.3	lb.
	Resultant Horizontal Force	$P = (W-c_m L\sin\alpha)(\tan(\alpha-\Phi_m)) - c_m L\cos\alpha + E$	7693.7	6299	4322	lb.
	Equivalent Fluid Density	$G_h = 2P/H^2$	*22.8*	*18.6*	*12.8*	PCF

EFD	For Temporary Wall Design, Use Equivalent Fluid Density	G_h=	30	PCF
	For Permanent Wall Design, Use Equivalent Fluid Density	G_h=	35	PCF

LATERAL EARTH PRESSURE CALCULATIONS

SECTION B-B' - SOUTH FACING RETAINING WALLS

FOR: 1248 Corona Drive | **DATE:** 4/1/19 | **PROJECT NO.:** 19-327-22

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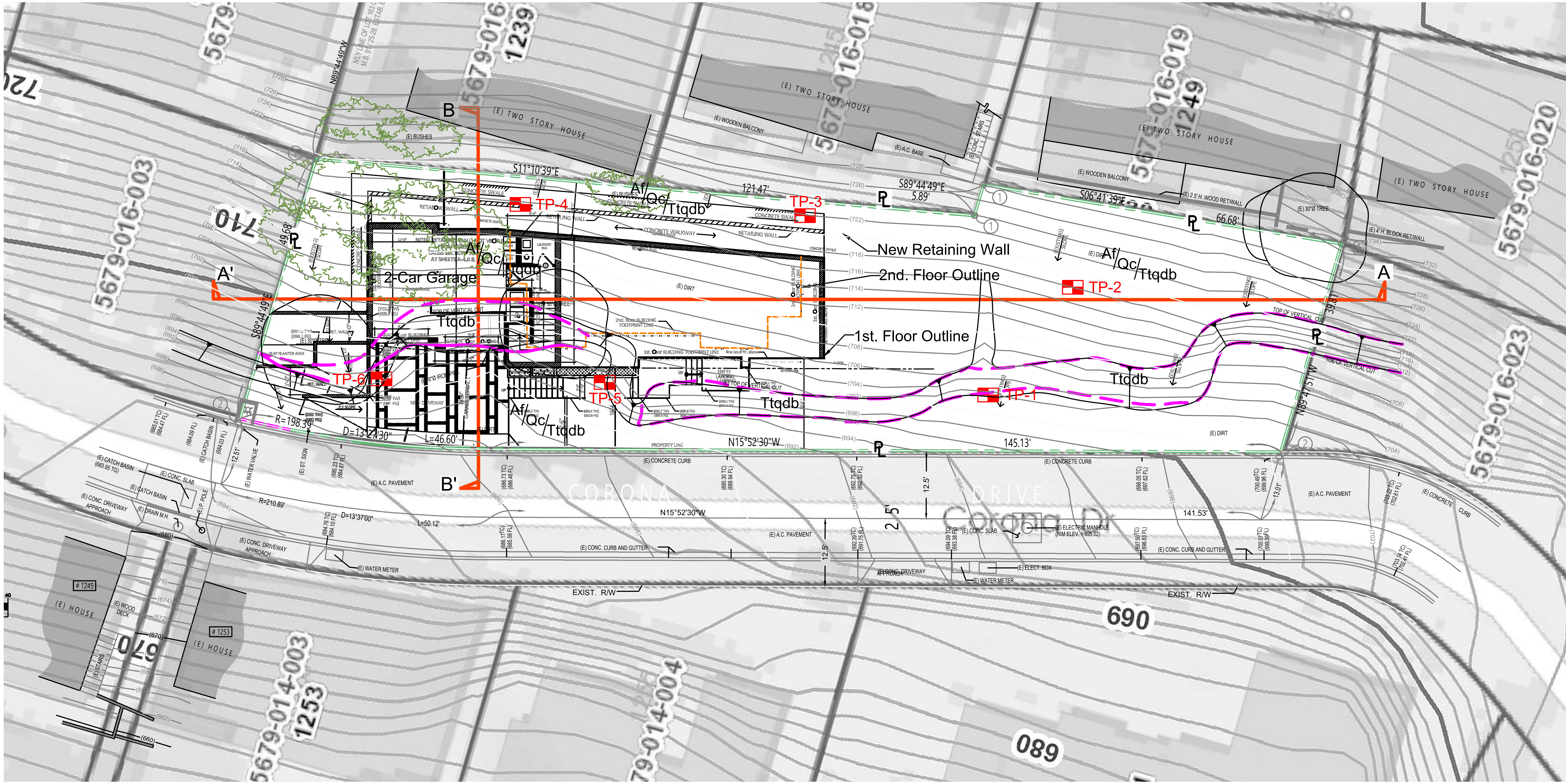
TABLE No. 2



Note:

Site plan and sections prepared by using of topography survey prepared by Architectural Plans by:

-EC+Associates Engineering

-Topographic Lines outside subject property are based on LA County GIS Map.

LEGEND:

Af = Artificial Fill

Qc = Colluvium

Ttqdb = Bedrock

TP-6 = Location & Number of Test Pit

= Geological Contact Approximately Located

N

Scale: 1" = 20'

GEOLOGIC MAP & SITE PLAN

PROJECT No:	19-327-22
DESCRIPTION:	Proposed Single Family Residence
DATE:	04 / 01 / 2019
FOR:	Mr. Eduardo Carillo
DRAWN BY:	TG
ADDRESS:	1248 Corona Drive, Glendale, CA 91205 (APNs 5679-016-001, 002, 024)
CHECKED BY:	SM
DRAWING No:	1

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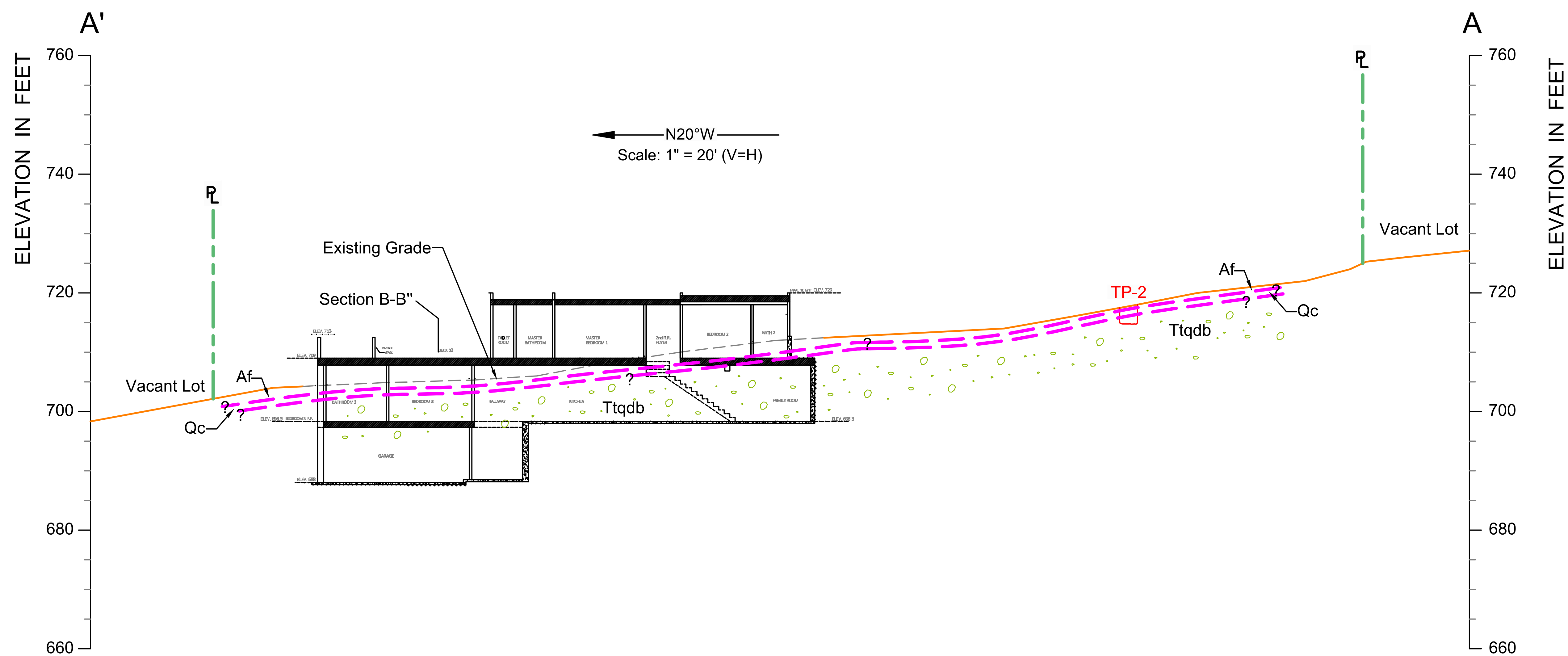

LEGEND:

Af = Artificial Fill

Qc = Colluvium

Ttqdb = Bedrock

TP-6 = Location & Number of Test Pit

= Geological Contact Approximately Located

GEOLOGIC CROSS SECTION A-A'

PROJECT No:	19-327-22
DESCRIPTION:	Proposed Single Family Residence
DATE:	04 / 01 / 2019
FOR:	Mr. Eduardo Carillo
DRAWN BY:	TG
ADDRESS:	1248 Corona Drive, Glendale, CA 91205 (APNs 5679-016-001, 002, 024)
CHECKED BY:	SM
DRAWING No:	2

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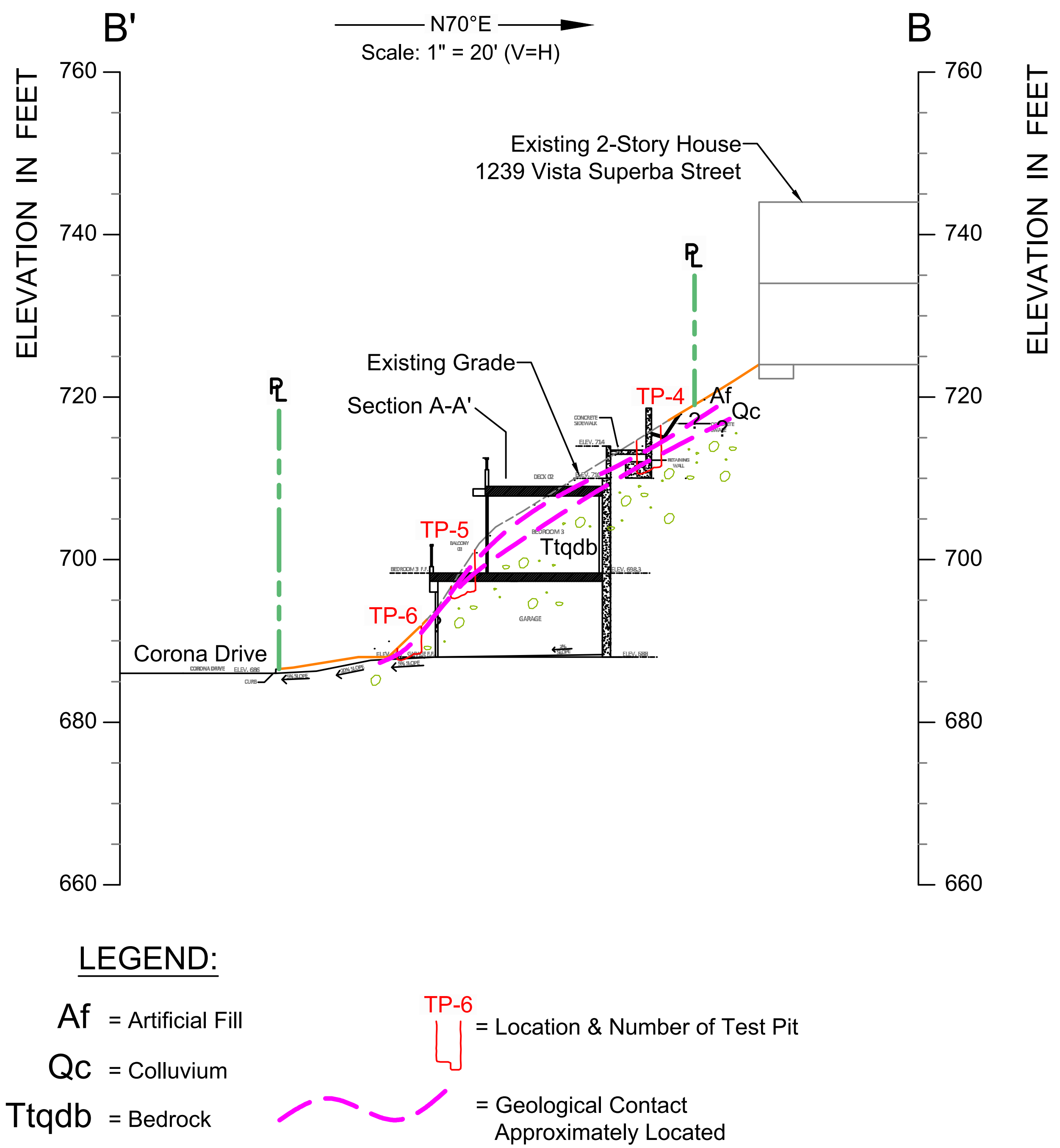
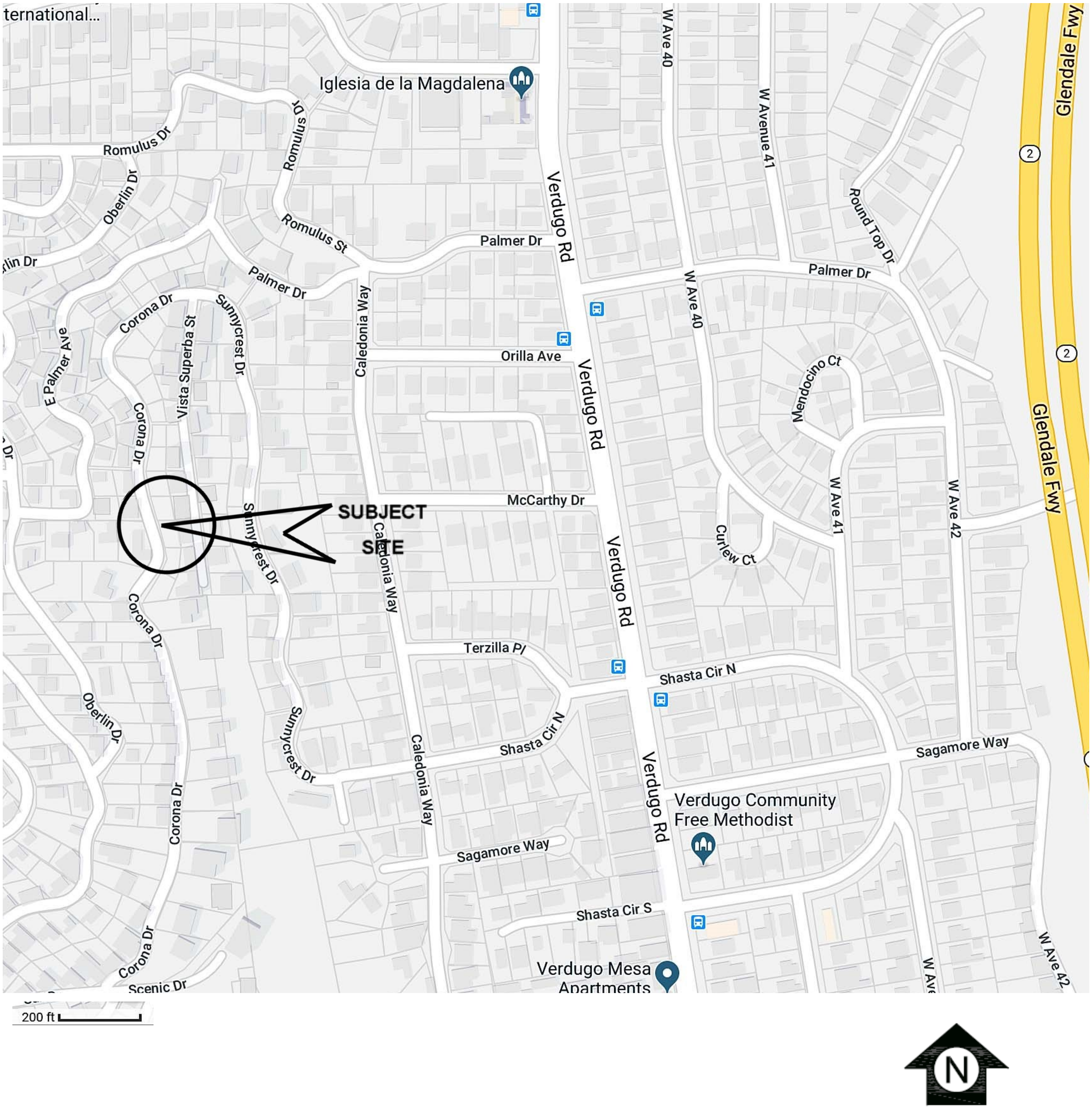


GEOLOGIC CROSS SECTION B-B'

DESCRIPTION:	Proposed Single Family Residence	PROJECT No:	19-327-22
		DATE:	04 / 01 / 2019
FOR:	Mr. Eduardo Carillo	DRAWN BY:	TG
ADDRESS:	1248 Corona Drive, Glendale, CA 91205 (APNs 5679-016-001, 002, 024)	CHECKED BY:	SM
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS — www.aessoil.com (818) 552-6000	DRAWING No:	3



Reference: Portion of Google Maps

SITE VICINITY MAP

Proposed Single Family Residence

1248 Corona Drive, Glendale, CA 91205

FOR	DATE	PROJECT No.
Mr. Eduardo Carillo	04 / 01 / 2019	19-327-22

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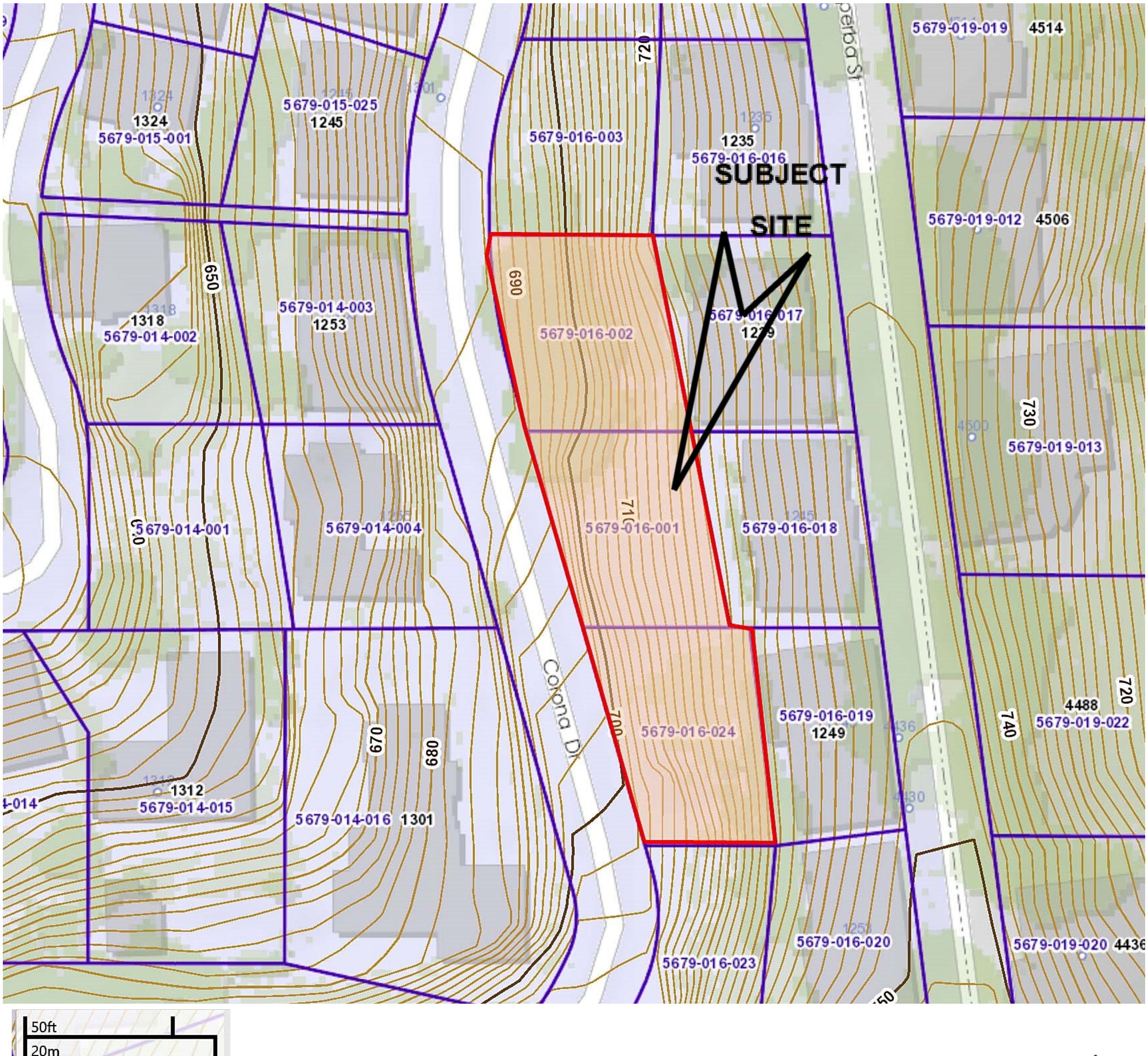
FIGURE No. 1



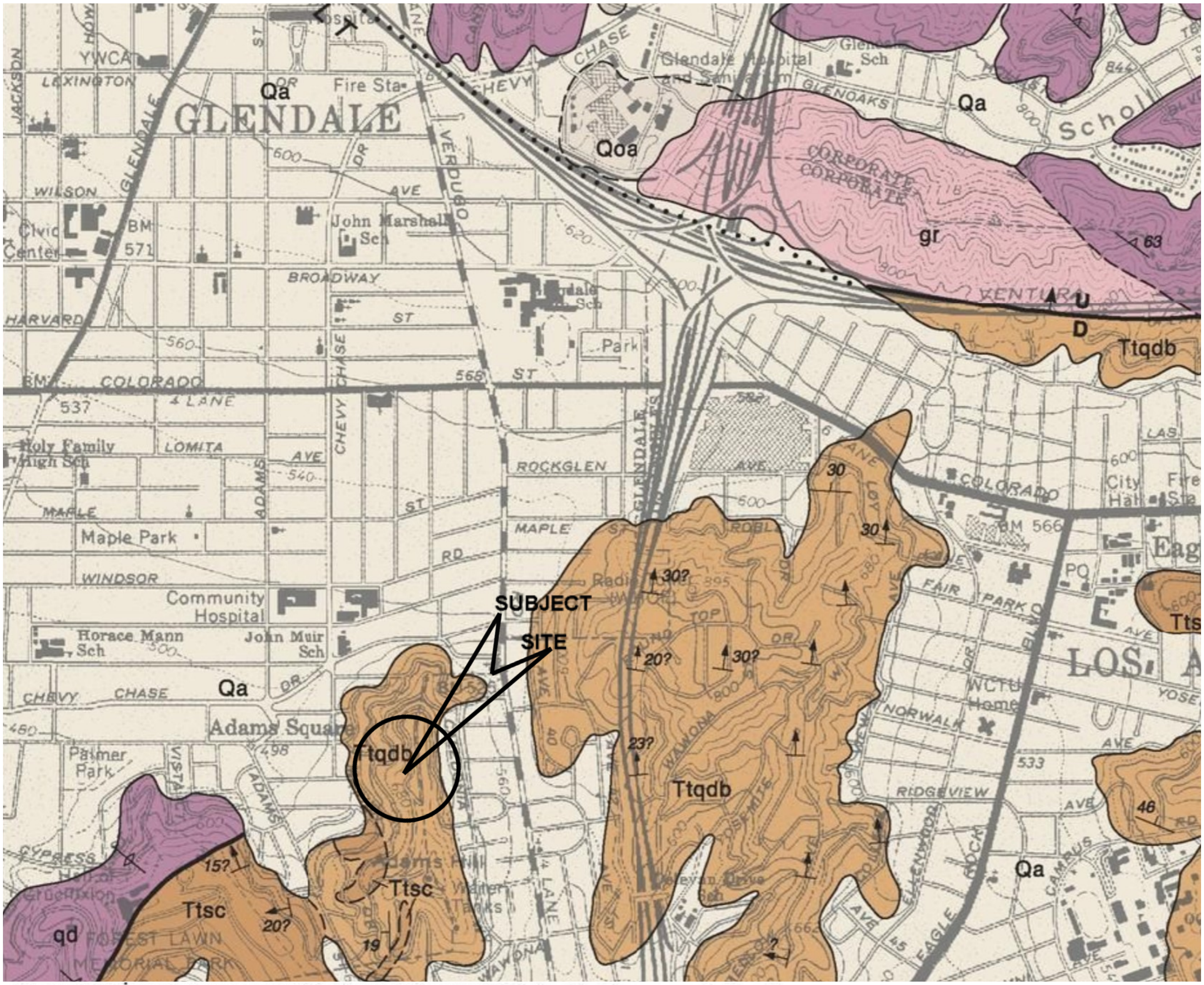
Reference: Los Angeles County GIS Map

REGIONAL TOPOGRAPHIC MAP

Proposed Single Family Residence

1248 Corona Drive, Glendale, CA 91205

FOR	DATE	PROJECT No.
Mr. Eduardo Carillo	04 / 01 / 2019	19-327-22
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 2



Ttsc *Light gray to brown, semi-friable sandstone, and interbedded brown sandy to silty shale, semisiliceous shale, and pebble-cobble conglomerate of quartz diorite detritus; Luisian(?) Stage*

Ttqdb *Gray to brown breccia, massive to vaguely bedded, composed of angular detritus and a few rounded cobbles and boulders, all of biotite hornblende quartz diorite; at Adams Hill includes lenses of sandy to silty shale and sandstone* **(Ttsc)***; probably overlies quartz diorite basement rocks*

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

Reference: Dibblee Geologic Map of the Pasadena Quadrangle

REGIONAL GEOLOGIC MAP

Proposed Single Family Residence

1248 Corona Drive, Glendale, CA 91205

FOR	DATE	PROJECT No.
Mr. Eduardo Carillo	04 / 01 / 2019	19-327-22

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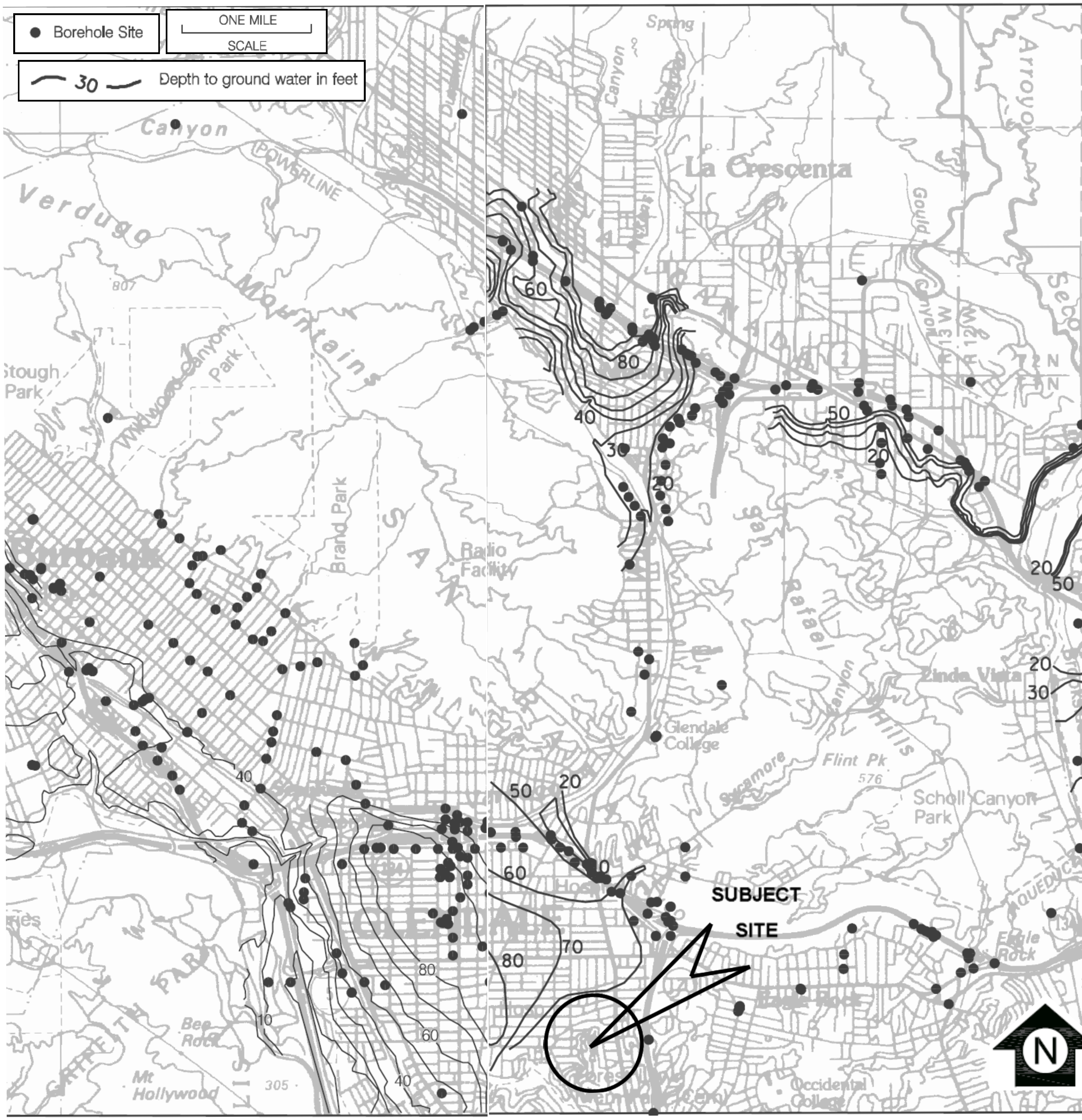
FIGURE No. 3



Reference: Pasadena & Burbank 7.5 Minute Quadrangle

HISTORICALLY HIGHEST GROUNDWATER (Contour Map)

Proposed Single Family Residence

1248 Corona Drive, Glendale, CA 91205

FOR	DATE	PROJECT No.
Mr. Eduardo Carillo	04 / 01 / 2019	19-327-22
APPLIED EARTH SCIENCES GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS		FIGURE No. 4

APPENDIX I
METHOD OF FIELD EXPLORATION

In order to define the subsurface conditions, six test pits were excavated on the site. The approximate location of the excavated test pits 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 Figure Nos. I-1 through I-6 within this Appendix. These figures also show the number and approximate depths of each of the recovered soil and rock 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 soil and bedrock 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 February 15, 2019. The material excavated from the test pits 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.

Date: February 27, 2019
Project No: 19-327-22

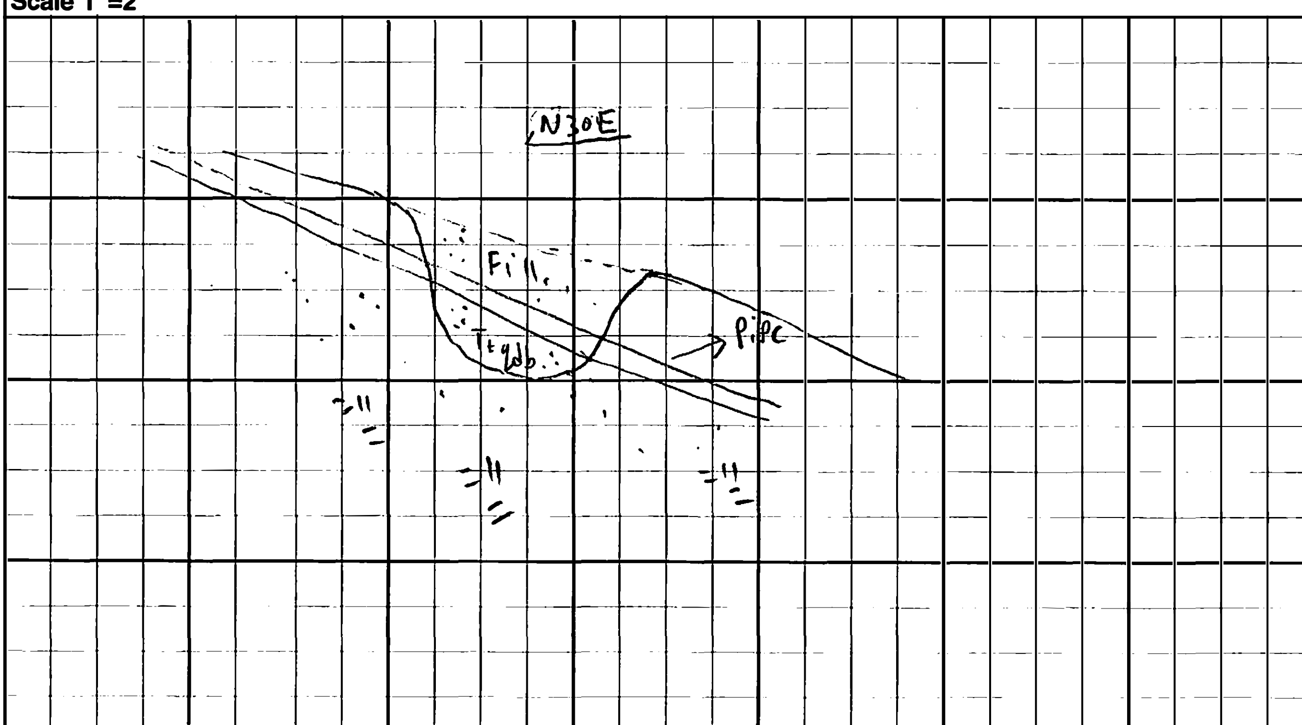
Figure No. I-1

EXPLORATORY TEST PIT NO. 1

PROJECT LOCATION:Lots 147,148 &170 of Tract 6759, Glendale
PROJECT TYPE: Proposed New SFR
DATE LOGGED: February 15, 2019
LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
102 @ 1'	13			Fill (Af)	0'- 2': FILL: Backfilled pipe trench, silty sand (SM) with fine granitic fragments as gravel, appears to be reworked bedrock material as fill, slightly to moderately compacted, moist, yellowish to dark brown.
117 @ 2.5'	12			Bedrock (Ttqdb)	2': SEDIMENTARY BEDROCK: Massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, yellowish brown.Weathered along the surface, grades to very dense with depth. Total Depth 2 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



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Date: February 27, 2019
Project No: 19-327-22

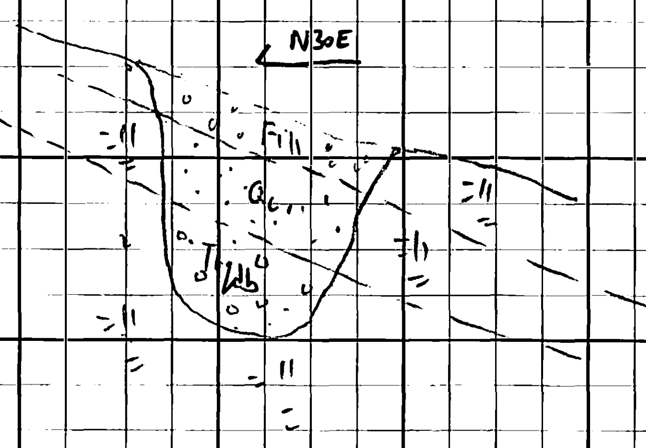
Figure No. I-2

EXPLORATORY TEST PIT NO. 2

PROJECT LOCATION:Lots 147,148 &170 of Tract 6759, Glendale   PROJECT TYPE: Proposed New SFR
DATE LOGGED: February 15, 2019   LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
				Fill (Af)	0'- 0.5': FILL-TOP SOIL: Silty Sand (SM), loose, moist to very moist, porous, dark brown, heavy rootlets, little gravel, creep prone.
99 @ 1.5'	16			Colluvium (Qc)	0.5' - 1.5': NATIVE SOIL: Silty Sand (SM) with fragments of granite, material is composed of highly weahtered underlying bedrock mixed with overylying top soil, creep prone, moist to very moist, slightly consolidated.
122 @ 3'	10			Bedrock (Ttqdb)	1.5' - 3': SEDIMENTARY BEDROCK: Highly weathered along surface and grades to massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, yellowish brown, creep prone in upper 6 inches of and grades to competent with depth.
					Total Depth 3 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



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Date: February 27, 2019
Project No: 19-327-22

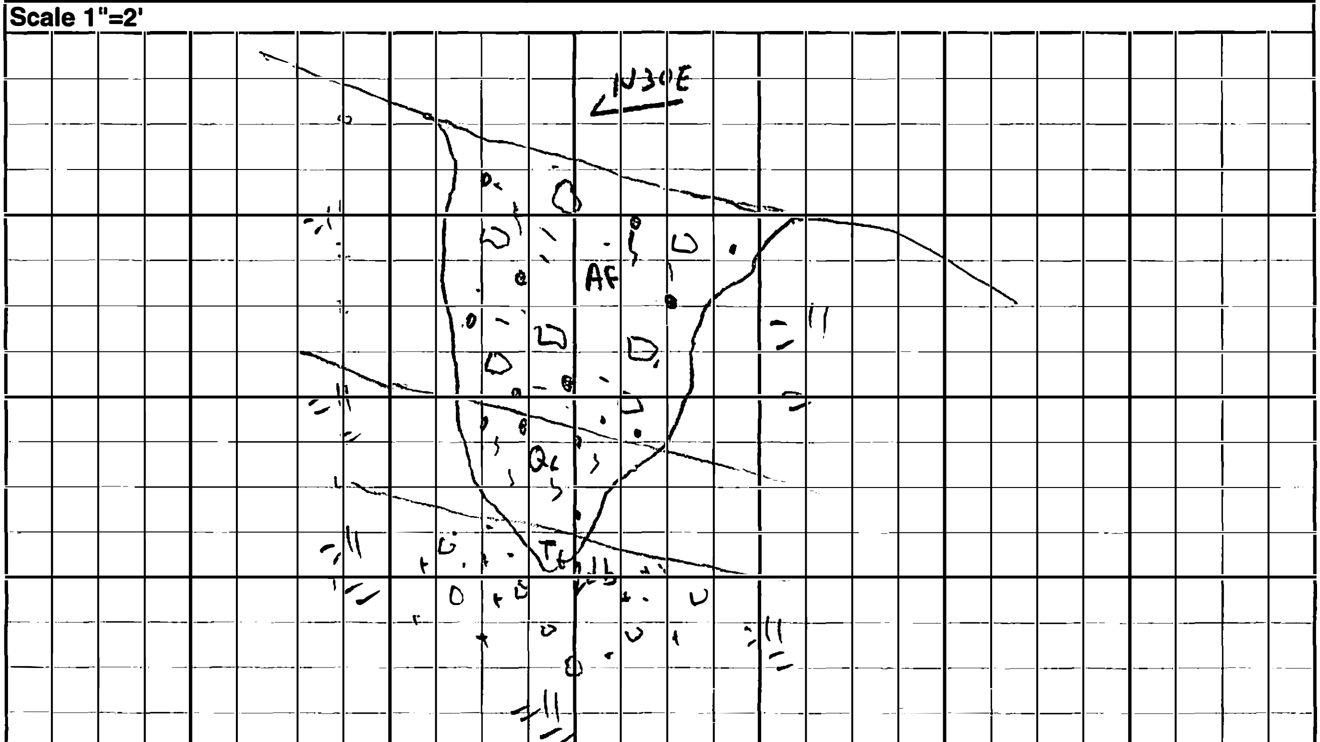
Figure No. I-3

EXPLORATORY TEST PIT NO. 3

PROJECT LOCATION:Lots 147,148 &170 of Tract 6759, Glendale   PROJECT TYPE: Proposed New SFR
DATE LOGGED: February 15, 2019   LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
101 @ 3'	15			Fill (Af)	0'- 3': FILL-: Silty Sand (SM) with gravel fragments of shale, debris, loose to moderately compact, heavy rootlets along underlaying layer with Qc, caving and loose along upper two feet, material is presumed to be from past grading of adjacent property.
				Colluvium (Qc)	3' - 4': NATIVE SOIL: Silty Sand (SM) with fragments of granite, material is composed of highly weahtered underlying bedrock mixed with overylying top soil, creep prone, moist to very moist, slightly consolidated.
118 @ 5'	14			Bedrock (Ttqdb)	4' - 5': SEDIMENTARY BEDROCK: Highly weathered along surface and grades to massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, yellowish brown, creep prone in upper 6 inches of and grades to competent with depth, no discernable bedding orientation observed. Total Depth 5 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



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Date: February 27, 2019
Project No: 19-327-22

Figure No. I-4

EXPLORATORY TEST PIT NO. 4

PROJECT LOCATION:Lots 147,148 &170 of Tract 6759, Glendale

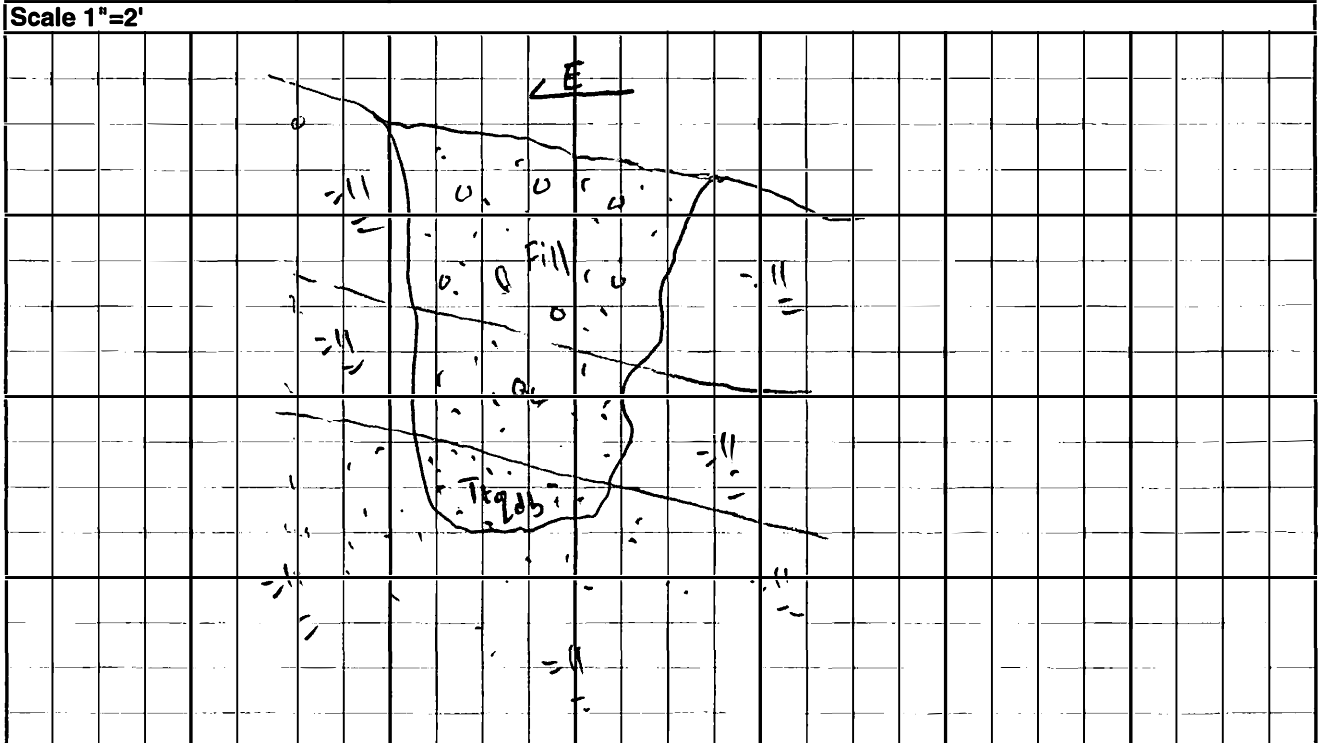
PROJECT TYPE: Proposed New SFR

DATE LOGGED: February 15, 2019

LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
				Fill (Af)	0'- 2': FILL-: Silty Sand (SM) with gravel fragments of shale, loose to moderately compact, heavy rootlets along underlaying layer with Qc, caving and loose along upper two feet, material is presumed to be from past grading of adjacent property, debris (cut wires, trash).
107 @ 3.5'	17			Colluvium (Qc)	2' - 3.5': NATIVE SOIL: Silty Sand (SM) with fragments of granite, material is composed of highly weahtered underlying bedrock mixed with overylying top soil, creep prone, moist to very moist, slightly consolidated.
121 @ 4.5'	13			Bedrock (Ttqdb)	3.5' - 4.5': SEDIMENTARY BEDROCK: Highly weathered along surface and grades to massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, yellowish brown, creep prone in upper 6 inches of and grades to competent with depth, no discernable bedding orientation observed.
					Total Depth 4.5 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



Date: February 27, 2019
Project No: 19-327-22

Figure No. I-5

EXPLORATORY TEST PIT NO. 5

PROJECT LOCATION: Lots 147,148 &170 of Tract 6759, Glendale

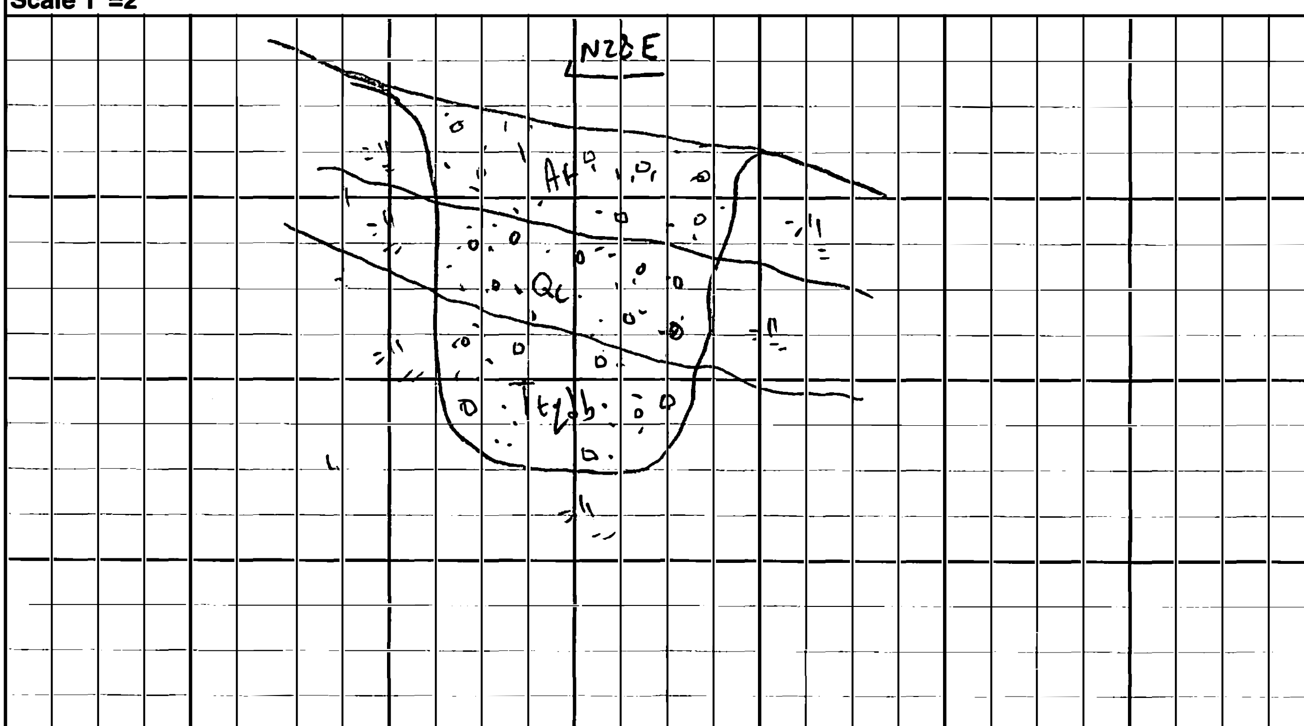
PROJECT TYPE: Proposed New SFR

DATE LOGGED: February 15, 2019

LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
				Fill (Af)	0'- 1': FILL-: Silty Sand (SM) with granitic fragments as gravel, moist to very moist, brown, loose, creep prone.
				Colluvium (Qc)	1' - 2': NATIVE SOIL: Silty Sand (SM) with fragments of granite, material is composed of highly weahtered underlying bedrock mixed with overylying top soil, creep prone, moist to very moist, slightly consolidated.
				Bedrock (Ttqdb)	2' - 4': SEDIMENTARY BEDROCK: Highly weathered along surface and grades to massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, yellowish brown, creep prone in upper 6 inches of and grades to competent with depth, no discernable bedding orientation observed.
					Total Depth 4 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



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Date: February 27, 2019
Project No: 19-327-22

Figure No. I-6

EXPLORATORY TEST PIT NO. 6

PROJECT LOCATION: Lots 147,148 &170 of Tract 6759, Glendale

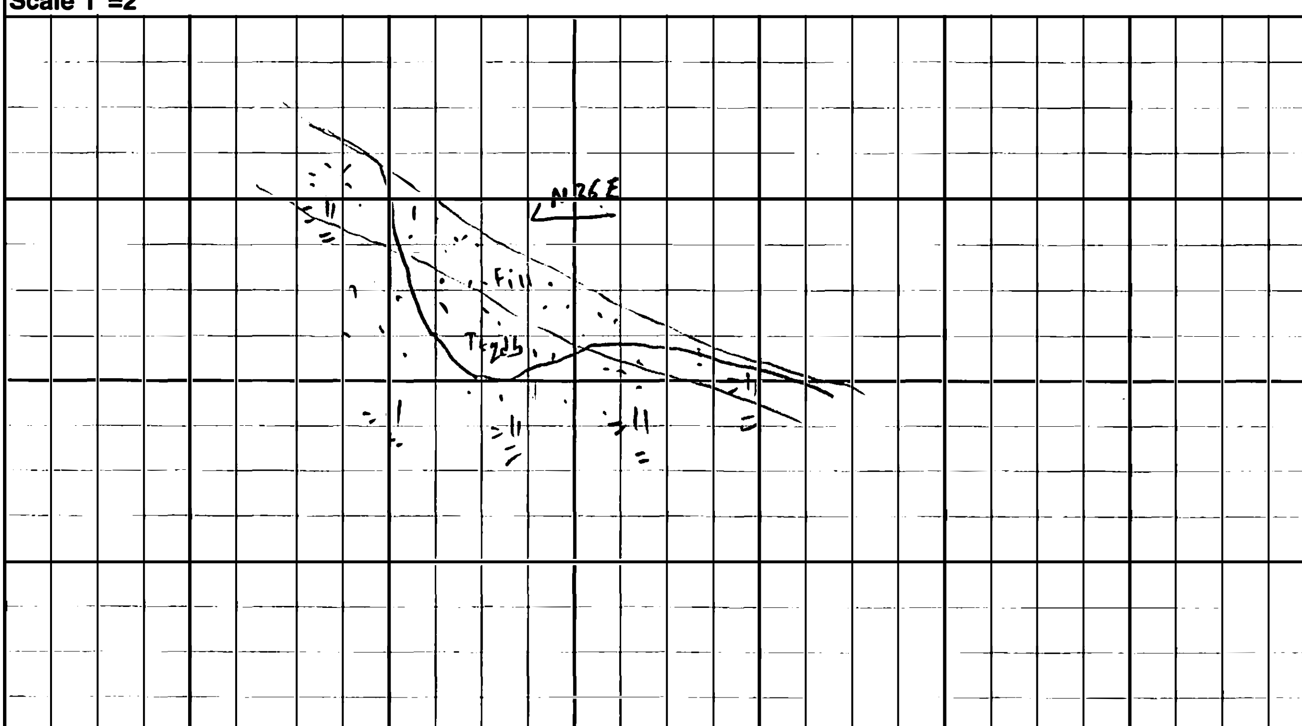
PROJECT TYPE: Proposed New SFR

DATE LOGGED: February 15, 2019

LOGGED BY: Ted

DRY DENSITY (PCF)	FIELD MOISTURE (% DRY WEIGHT)	% Silt + Clay	BLOWS PER FOOT	GEOLOGIC UNIT	MATERIAL DESCRIPTION (USCS)
92 @ 1'	14			Fill (Af)	0'- 1': FILL-: Silty Sand (SM) with granitic fragments as gravel, moist to very moist, brown, loose, creep prone.
123 @ 2'	9			Bedrock (Ttqdb)	1' - 2': SEDIMENTARY BEDROCK: Highly weathered along surface and grades to massive breccia composed of well rounded to subangular granitic clasts (qd), fine sand matrix, well to highly cemented with depth, crumbles easily when mild pressure applied, slightly moist to moist, grades to competent with depth, no discernable bedding orientation observed. Total Depth 2 Feet. No water, No caving. Test Pit nominally backfilled to surface level.

Scale 1"=2'



MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAME
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Organic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Organic clay of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Organic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Organic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 · NO. 40 · NO. 10 · NO. 4 · 3/4 in. · 3 in. · (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

JOB NAME :	Propose New Single Family Residence 1248 Corona Drive, Glendale, CA 91205	JOB No. 19-327-22

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FIGURE No. I-7

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 dry unit weight and field moisture content were determined for each undisturbed sample, and the results are shown on the log of exploratory borings.

SHEAR AND RE-SHEAR TESTS

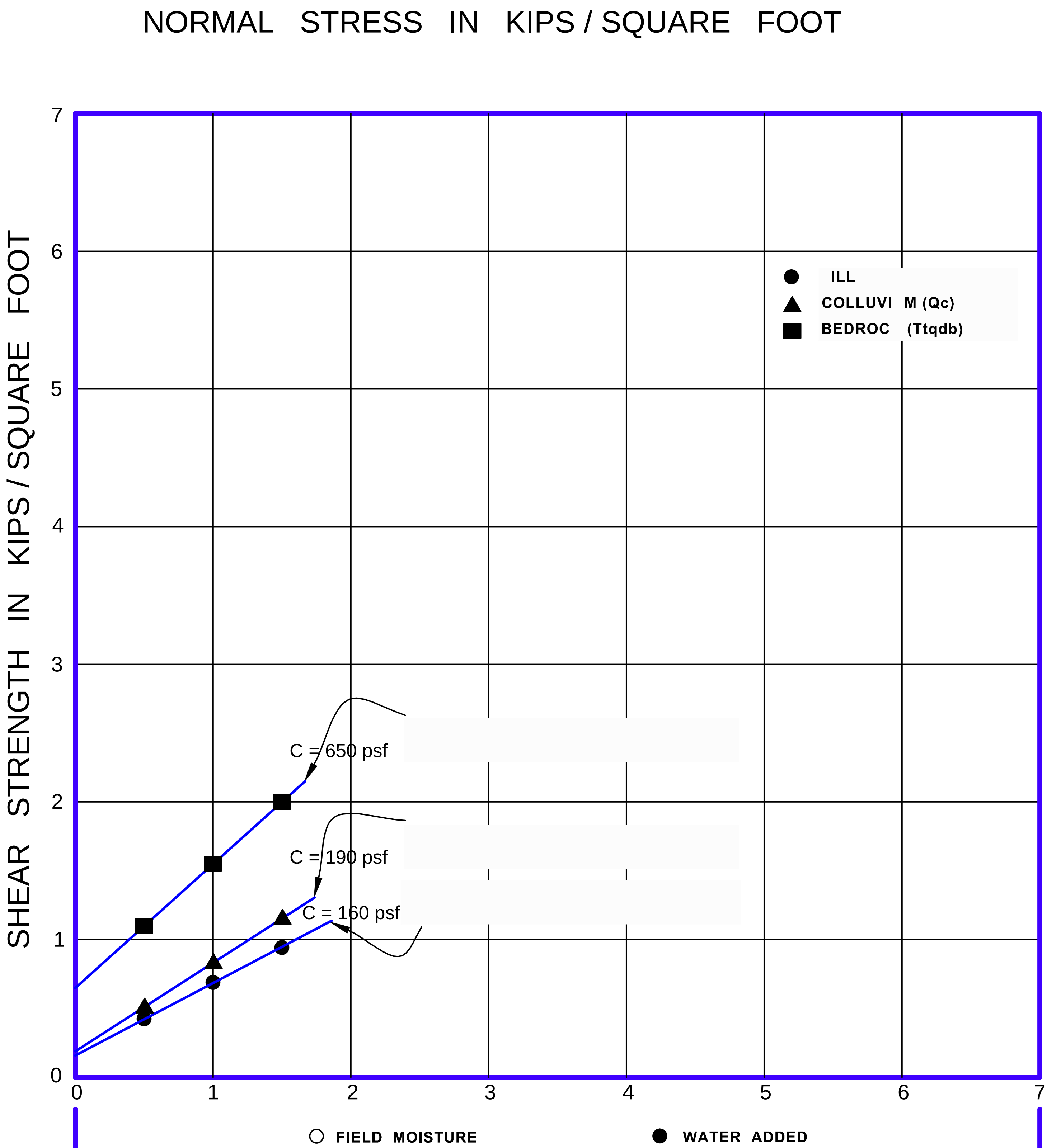
After the samples are pre-soaked overnight under initial confining pressure, a range of normal stresses are applied vertically, and the shear strengths are progressively determined under each load in order to determine the internal angle of friction and the cohesion of the sample. After application of each of the confining pressures, and before the shearing tests, sufficient amount of time is allowed for any excess pore pressure to dissipate. During the course of shear test, the sample is allowed to undergo volume change under a given confining pressure. Under each load, the direct sear tests are continued until the ultimate strength or about 3 percent strain (whichever is lower) is reached. The sample is then allowed to relax to remove the major portion of the viscous component of the shear strength. It should be noted that due to normal disturbance during sampling and laboratory extruding, the measured bedrock strengths are normally significantly lower than the actual values.

In order to determine the strength of the bedrock along bedding, foliation or joint planes or landslide debris strengths, the sample is soaked overnight under initial confining pressure. The sample is then re-sheared several times until the least strengths are obtained. During typical testing, the shearing of the samples are continued until the residual strengths are developed (the shear strengths remain constant, after the peak has been reached, or about 5 percent strain corresponding to approximately 0.100 inches of shearing deformation has occurred). At this point, the tests are stopped. The samples are then pushed back to their original position. The shear test procedure is then repeated along the previously sheared plane. This procedure is repeated several times until constant residual strengths are obtained.

CONSOLIDATION

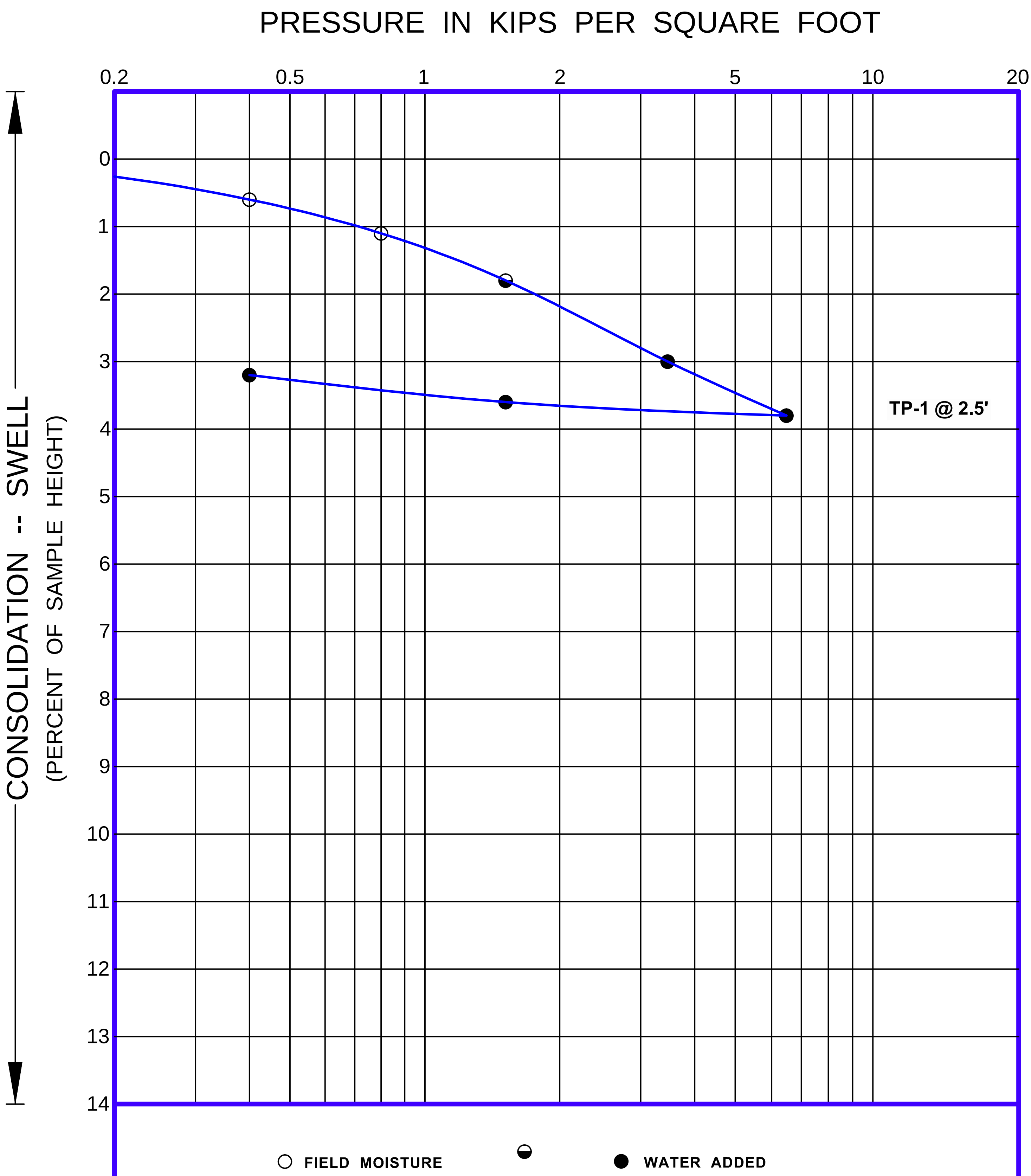
The apparatus used for the consolidation tests is designed to receive the undisturbed brass ring of soil as it comes from the field. Loads were applied to the test specimen in several increments, and the resulting deformations were recorded at selected time intervals. Porous stones were placed in contact with the top and bottom of the specimen to permit the ready addition or release of water.

Undisturbed specimens were tested at the field and added water conditions. The test results are shown on Figure No. II-2 within this Appendix.



DIRECT SHEAR TESTS

JOB NAME :	Propose New Single Family Residence 1248 Corona Drive, Glendale, CA 91205	JOB No. 19-327-22
Applied Earth Sciences	GEOTECHNICAL . GEOLOGY . ENVIRONMENTAL ENGINEERING CONSULTANTS www.aessoil.com (818) 552-6000	FIGURE No. II - 1



SWELL - CONSOLIDATION TESTS

JOB NAME : Propose New Single Family Residence
1248 Corona Drive,
Glendale, CA 91205

JOB No. 19-327-22

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FIGURE No. II - 2