Attachment #8

CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

PRELIMINARY GEOLOGIC AND GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Single-Family Residence and Swimming Pool

AIN: 5648-029-015

1766 Cielito Drive

Glendale, CA

for

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1766 Cielito Drive

Glendale, CA

Project 200416

August 2, 2021

PRELIMINARY GEOLOGIC AND GEOTECHNICAL ENGINEERING INVESTIGATION

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INTRODUCTION

This report presents the results of a Preliminary Geologic and Geotechnical Engineering Investigation on a portion of the subject property. The purpose of this investigation has been to ascertain the subsurface conditions pertaining to the proposed project. The work performed for the project included reconnaissance mapping, description of earth materials, determining geologic structure, obtaining representative samples of earth materials, laboratory testing, engineering analyses, and preparation of this report. Results of the project include findings, conclusions, and appropriate recommendations.

SCOPE

The scope of this investigation included the following:

- Review of preliminary plans by Alajajian Marcoosi Architects, Inc.
- Review of nine (9) test pit explorations. Explorations were backfilled with the excavated materials but not compacted.
- Preparation of the enclosed Geologic Map and Cross Sections, (see Appendix I).
- Sampling of representative earth materials, laboratory testing, and engineering analyses (see Appendix II).
- Review of referenced materials and available public reports at the City of Glendale (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

Ray Lombera & Associates, Inc. prepared the topographic base map utilized in this investigation. Preliminary building plans were prepared by Alajajian Marcoosi Architects, Inc and utilized/incorporated onto the base map for this investigation. Analysis and/or professional opinions generated from this plan are only as accurate as the plan(s) provided to our office. If discrepancies are found to exist between the plan(s) and the actual site condition, they should be brought to our immediate attention so that revisions may be made as required.

The scope of this investigation is limited to the project area explored as depicted on the Geologic Map. This report is not a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for purposes other than the proposed project. Creative Geotechnical, Inc. should be consulted to determine if additional work is required when our work is used by others or if the scope of the project has changed. If the project is delayed for more than one year, this office should be contacted to verify the current site conditions and to prepare an update report.

This report is prepared for the use of the client and authorized agents only and should not be considered transferable. Prior to use by others, the site should be revisited and this report should be reviewed by Creative Geotechnical, Inc. Following review, additional work may be required to update this report.

PROPOSED DEVELOPMENT

It is our understanding that the site will be developed with a single-family residence and a swimming pool. Anticipated foundations will range from 1 to 2 kips per lineal foot and 20-40 kips for column foundations. The proposed development is depicted on the enclosed Geologic Map and Cross Sections.

Grading will consist of conventional cut and fill methods. Final plans have not been prepared and await the conclusions and recommendations of this investigation. These plans should be reviewed by Creative Geotechnical, Inc. to ensure that our recommendations have been followed.

SITE DESCRIPTION

Location and Description

Access to the property is via Cielito Drive from Deermont Road (see Location Map in Appendix I). The site is developed with a one-story single-family residence with attached garage, swimming pool and deck.

The pad has a light to moderately dense growth of vegetation consisting of grasses, lawn areas, shrubs and trees. Vegetation is moderately dense to dense on the ascending and descending slopes consisting of grasses, ground cover, shrubs, and trees.

Adjacent sites to the north and south have been developed with single-family residences. Adjacent structures to the north and south are more than 15 feet from the property line.

Topography

Topographically, the property is situated on the east west flank of a north-south trending ridge within the southwest portion of the San Gabriel Mountains. The property essentially consists of a near-level pad with ascending slopes to the north and descending slopes to the south. Maximum topographic relief on-site is about 90 feet. Ascending slopes from the pad on the north portion of the site have a general gradient of 1.5:1 or less, (horizontal to vertical). Descending slopes display a general gradient of 0.3:1 or less, (horizontal to vertical). Some slopes are partially supported by retaining walls and board revetment. Details of the topography are depicted on the Location Map and Geologic Map in Appendix I.

Drainage

Surface water at the site consists of direct precipitation onto the property and runoff from surrounding slopes to the north. Much of this water drains as sheet flow down descending slopes to low-lying areas, area drains, paved swale drains, offsite and/or to the street. The residence has been provided with roof gutters and downspouts. Portions of the yard are serviced by an irrigation system. Area drains are located in the front and rear yard areas, and pool deck. A subdrain outlet pipe was observed in the curb and near the southwest corner of the property.

Groundwater

No active surface groundwater seeps or springs were observed on the subject site. The subsurface exploration did not encounter groundwater to a depth of 10 feet. Due to the elevated nature of the property, groundwater is not anticipated to pose a problem to the proposed project. Seasonal fluctuations of groundwater levels may occur by tidal changes and varying amounts of rainfall, irrigation and recharge.

FIELD EXPLORATION

The scope of the field exploration was developed based on the preliminary plans of the proposed development available at the time of the exploration and was limited to the area of the proposed development. The locations of the explorations are depicted on the Geologic Map and Cross Sections. The field exploration was limited by existing structures, hardscape, and/or underground utilities on the site.

The field exploration of the site was conducted on April 18, 2020. The geotechnical and geologic conditions were mapped by a representative of this office (refer to Exploration Logs). Subsurface exploration was performed by manual excavation into the underlying earth materials. Explorations were excavated to a maximum depth of 10 feet. Down-hole observation of the earth materials exposed in the explorations was performed by the project geologist where subsurface conditions were deemed to be safe. All explorations were backfilled and tamped upon completion of down-hole observation. However, some settlement within exploration areas should be anticipated.

Detailed descriptions of the geologic materials encountered during the field exploration are provided in the Exploration Logs and Boring Logs in Appendix I.

Undisturbed and bulk samples representative of the earth materials were obtained and transported to our laboratory. Undisturbed Modified California (MC) samples were obtained within the explorations through the use of a thin-walled steel hand-held sampler. MC samples were retained in brass rings of two and one-half inches in diameter and one inch in height. The samples were transported in moisture tight containers. The results of the laboratory testing and a summary of the test procedures are included within Appendix II.

SUMMARY OF FINDINGS

Previous Work

The subject property was developed circa 1961 prior to the County of Los Angeles Grading Ordinance. The City of Santa Monica, Building Department had been contacted for geology and/or geotechnical reports covering the subject property. However, the building official would not allow review of the records and had refused to conduct a search for available records.

Stratigraphy

The encountered earth materials on the site are briefly described below. Detailed descriptions of the explorations and approximate depths of the earth materials are given in the enclosed Exploration Logs (see Appendix A).

Artificial Fill (Af)

Artificial fill was encountered on the subject site. Fill materials were presumably placed during pad grading and construction of the residence. Fill was encountered in all nine of the test pits ranging from 1 to more than 10 feet in thickness. The contact between the fill and the underlying bedrock was exposed within five of the exploratory test pits. No evidence of engineered keys or benches was observed. Fill generally consists of silty sand and with abundant rock fragments that generally range between 1 and 10 inches in length. The approximate limit of the existing fill is shown on the attached Geologic Map and Cross Sections.

Bedrock (gqd)

Bedrock exposed on-site and underlying the fill consists of intrusive granitic rocks of Cretaceous time with varying compositions of granodiorite and quartz diorite. They are generally dark yellowish brown to black and white, very dense, slightly fractured, and slightly to moderately weathered. The contact between the artificial fill and bedrock is approximately located on the Geologic Map and Cross Sections.

Excavation Characteristics

Subsurface exploration was performed through the use of hand labor excavating into fill and bedrock. The bedrock encountered during the exploration consists of granite. At the site, bedrock was observed to be moderately to slightly weathered and slightly fractured. Although excavation difficulty is considered normal, it should be noted that the bedrock is a layered formation and hard or well cemented bedrock may be encountered. Extremely hard layers of bedrock were encountered in the explorations. Thus, excavating into the bedrock during construction will be difficult. Typically, the hardness of bedrock increases with depth. If hard or well cemented bedrock is encountered, coring or the use of heavy jack hammers may be necessary.

Geologic Structure

The local area has been uplifted and intruded by a granitic pluton. No dominant patterns of adversely orientated fractures or joints were observed during the subsurface investigation. The proposed retaining walls will not be surcharged by the bedrock.

Critical anticipated bedrock structure is depicted on Geologic Cross Sections. Preliminary geologic data indicates the proposed development is favorable from the standpoint of geology and geotechnical engineering, provided the recommendations contained herein are followed and maintained.

Landslides

Landslides are a mass wasting phenomenon in mountainous and hillside areas which include a wide range of movements. In Southern California common slope movements include shallow surficial slumps and flows, deep-seated rotational and translational bedrock failures, and rock falls. Landslides occur when the stability of the slopes change to an unstable condition resulting from a number of factors. Common natural factors include the physical and/or chemical weathering of earth materials, unfavorable geologic structure relative to the slope geometry, erosion at the toe of a slope, and precipitation. These factors may be further aggravated by human activities such as excavations, removal of lateral support at the toe of a slope, surcharge at the top of a slope, clearing of vegetation, alteration of drainage, and the addition of water from irrigation and leaking pipes.

Ancient or recent bedrock landslides were not observed on the property. Also, no recent surficial slope failures or slumps were observed within the proposed project area on the property.

Slope Stability

A gross stability analysis was performed on cross section A that are considered to represent the most critical profile based on the geologic structure and topography. The analysis was performed using the computer program stabl for windows. A summary of the stability analysis is provided below with calculations contained herein.

Summary of shear strength parameters:

Material	γ (pcf)	γ_{sat} (pcf)	C (psf)	Phi (deg)
Bedrock	130	140	906	41.3

Summary of slope stability analyses:

Section	Static Factor of Safety	Pseudo-Static Factor of Safety
A	2.00	1.19

The assessment of surficial slope stability was based on the infinite slope with seepage parallel to the slope surface model. The analysis indicates that the slope is surficially stable.

Seismic Hazards

Earthquake Faults

The Alquist-Priolo Earthquake Fault Zoning (AP) Act was passed into law following the destructive February 9, 1971 San Fernando earthquake. The intent of the Act is to increase public safety by reducing the siting of most structures for human occupancy across an active fault. The Act only addresses the hazard of surface fault rupture and is not directed toward other earthquake hazards. The property is not located within an Alquist-Priolo Earthquake Fault Zone. The general locations of major faults within Southern California are depicted on a fault map provided by the USGS in Appendix I.

Active Faults

The following active faults are capable of producing seismic waves (ground shaking) on the subject property. A summary description of the closest active faults and potentially active faults to the site are described herein and labeled by number on the map below. An active fault, as defined by the State Mining and Geology Board, is one, which has "had surface displacement within Holocene time (about the last 11,000 years)".

The San Andreas Fault zone (42) is the dominant active fault in California. Geologic studies show that over the past 1,400 to 1,500 years large earthquakes have occurred at about 150-year intervals on the southern San Andreas Fault. It consists of numerous subparallel faults of varied lengths in a zone generally 0.3 to 1.5 km wide in Southern California. The dip of the fault is near vertical and the sense of motion is right lateral. Historically, the 1857 Fort Tejon earthquake with an estimated magnitude of 7.9 ruptured the ground surface from the vicinity of Cholame (near Paso Robles) to somewhere between the Cajon Pass and San Gorgonio Pass (Wrightwood), approximately 200 miles. Studies of offset stream channels indicate that as much as (29) feet of movement occurred in 1857. The fault extends from the Gulf of California northward to the Cape Mendocino area where it continues along the ocean floor, approximately 750 miles in length.

The Northridge earthquake occurred on January 17, 1994, in the San Fernando Valley. The epicenter was about 1 mile south-southwest of Northridge at a focal depth of 12 miles. The surface wave magnitude was issued by the National Earthquake Information Center at Mw=6.7. This event occurred on a previously unrecognized south-dipping blind reverse fault without surface rupture. This earthquake produced the strongest ground motions ever instrumentally recorded in an urban setting in North America. Damage was wide-spread with sections of major freeways collapsed include some parking structures and office buildings. Common surface disruptions included buckled curbs and sidewalks, fissured concrete and asphalt, and rupture of utility lines which are generally aligned in northwest and east-west directions. Shattered ridges were reported along Mulholland Drive in the Sherman Oaks area, consisting of intense ground disturbances associated with strong vibratory ground motions within the north trending ridges underlain by shale of the Lower Modelo formation.

The Whittier-Elsinore fault zone (60) consists of several subparallel, overlapping and en echelon fault strands in a zone up to 1.2 km wide. It extends nearly 125 miles from the Mexican border to the northern edge of the San Fernando Valley. Seismicity includes the Whittier Narrows earthquake of October 1, 1987 with a magnitude of 5.9 and an epicenter in the city of Rosemead. This earthquake occurred on a previously unknown and concealed thrust fault. There was no reported surface rupture from the earthquake. Also, numerous close and scattered small earthquakes have occurred in historic time near and along the fault.

The San Fernando fault (45) consists of five major en echelon strands at least 9.5 miles in length. The "San Fernando" earthquake of February 9, 1971 produced a magnitude of Mw 6.5 at a depth of 8.4 km along an east west trending reverse fault with a northerly dip. The length of the surface rupture was about 9.5 miles and ground shaking lasted for approximately 60 seconds. The earthquake ruptured the northwestern end of the Sierra Madre Fault zone forming the San Fernando Fault. Major damage included the Olive View and Veterans Administration Hospitals and collapse of freeway overpasses. Landslides occurred in the Upper Lake area of Van Norman Lakes. Additionally, the Van Norman Dam and the Pacoima Dam were severely damaged.

The eastern portion of the Santa Susana fault (52) ruptured during the 1971 San Fernando Earthquake. The Santa Susana fault consists of several strands in a zone as wide as 1 km. It generally strikes from north 75 degrees west to north 50 degrees east and dips to the north. The fault is a high angle reverse fault. The fault appears to have been generated by northeast-southwest oriented compressional stress.

The Newport-Inglewood fault zone (31) consists of several strands that extend from offshore by Laguna

Beach to either merge with or be truncated by the Malibu-Santa Monica fault zone near Beverly Hills. The fault has a length of about 45 miles. It was the source of the "Long Beach" earthquake, which occurred on March 10, 1933 with a magnitude of 6.3. Numerous small earthquakes have occurred in historic time along and near the fault zone. The fault zone is easily observed by an alignment of hills and mesas including Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill, Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, and Newport Mesa.

In June 1995, two portions of the Malibu Coast fault zone (27) were reclassified as active fault zones by the State of California. On August 16, 2007, the fault zone near the east side of Malibu Bluff Park was removed from the State of California Earthquake Fault Zone map by the State of California. The east west trending Malibu Coast fault consists of several subparallel strands in a zone as wide as 0.5 km, with a length of at least 17 miles. It strikes east west and dips (45) to (80) degrees to the north. The Malibu Coast fault has the potential to produce a large Maximum Credible Peak and Repeatable Acceleration on the subject property. The duration of the Malibu Coast fault is estimated at (11) seconds assuming fault end nucleation and unidirectional rupture propagation, (Bolt, 1981). The Malibu Coast fault is thought to be part of other faults such as the Santa Monica fault and Hollywood fault that separate the Transverse Ranges on the north from the Peninsula Range on the south. Two Malibu Earthquakes occurred with Magnitudes of M_L 5.2 and M_L 5.0 on January 1, 1979 and January 18, 1989, respectively. It was reported that only minor damage occurred in the areas closest to the epicenter.

The Hollywood fault zone (22) extends along the base of the Santa Monica Mountains. This fault was added to the list of active faults by the State of California in 2014. Generally, the Hollywood fault extends eastward for a distance of 15 km through Beverly Hills, West Hollywood, and Hollywood to the Los Angeles River. The fault is primarily expressed at the ground surface by scarp-like features. This is a left–reverse fault with an estimated slip rate between 0.33 mm/yr and 0.75 mm/yr, (Petersen and Wesnousky 1994).

The Raymond fault (39) is a combination fault with reverse and left slip movement that acts as a groundwater barrier within the densely populated San Gabriel Valley. The activity of the fault is attested to by the numerous geomorphic features found along its entire length of approximately 14 miles. Scattered small earthquakes have occurred north of the fault trace. It may be the source of the 1855 Los Angeles earthquake. The Raymond fault is an east-trending fault made up of other faults such as the Hollywood and Santa Monica faults that separate the Transverse Ranges on the north form the Peninsula Range on the south.

The Sierra Madre fault zone (53) is often divided into five main segments; Vasquez Creek fault, Clamshell fault (10), Sawpit Canyon fault (10), Duarte fault and the Cucamonga fault (14). The Sierra Madre earthquake of June 28, 1991 (Mw5.8) was in the San Gabriel Mountains. An estimated 33.5 million dollars of damage has been reported. The Sierra Madre fault zone is about 75 km long. It's a thrust fault system along the south edge of the San Gabriel Mountains. The east end of the Sierra Madre fault zone intersects the San Jacinto fault and the San Andreas Fault. The 1971 San Fernando earthquake occurred on the San Fernando-Sunland segment of the Sierra Madre fault zone.

The San Gabriel fault (46) consists of several en echelon fault strands in a zone approximately 0.5 km wide, with a length of about 90 miles. The fault trends northwestward and subparallel to the San Andreas Fault. As of March 1, 1988, a portion of the Newhall segment of the fault zone was reclassified as an active fault. Fault activity has been dated between 1550 and 3500 years before present within the

Newhall segment. The youngest ground rupture event has broken alluvial beds to within five feet of the ground surface. Geologic evidence suggests 38 miles of right lateral offset has occurred between 14 million and 3 million years ago and may have functioned as an ancestral branch of the San Andreas Fault. Recent studies suggest that the major strike slip movement has become inactive and dip slip movement is active at the present time.

Potentially Active Faults

A potentially active fault, as defined by the State Mining and Geology Board, is one, which has had surface displacement during Quaternary time (last 1.6 million years). "These faults are those based on available data along which no known historical ground surface ruptures or earthquakes have occurred. These faults, however, show strong indications of geologically recent activity". The following list provides potentially active faults that are capable of producing seismic waves (ground shaking) on the property.

The Santa Monica fault (50) extends east from the coastline in Pacific Palisades through Santa Monica and West Los Angeles and merges with the Hollywood fault. Several local geologists believe portions of the Santa Monica fault zone are active. Currently, it is listed by the State of California as a potentially active fault. Portions of the fault zone may change to "active" and be placed within the Alquist-Priolo Earthquake Fault Zone as additional geologic reports are submitted to the State containing evidence of Holocene activity. The Santa Monica fault consists of one or more fault strands, with a poorly known geometry. Generally, the fault strikes northeast 60 to 80 degrees and dips 45 to 65 degrees northwest at depth with a few near vertical surface traces. The length of the fault is at least 25 miles. The composite local mechanism of fault displacement is a reverse left lateral along the Santa Monica-Hollywood-Raymond fault zone. The Santa Monica and Hollywood faults may be part of a larger fault system that includes Malibu Coast, Raymond and Cucamonga fault system. This fault zone forms the central portion of a major tectonic boundary separating the east west trending Transverse Ranges province to the north from the northwest trending Peninsular Ranges province to the south.

The Benedict Canyon fault zone trends eastward through the Santa Monica Mountains. The fault may be part of the Hollywood-Santa Monica-Raymond fault system. The activity of the fault is based on offsets in groundwater bearing sediments that correlate with steep dipping gravity gradients. The fault extends through Universal City and along the north side of the eastern part of the Santa Monica Mountains.

The Simi fault (54) consists of a single strand that bifurcates at the western end. Generally, it strikes north 70-80 degrees east and dips 60 to 75 degrees north with a length of about 31-km.

The Mission Hills fault (30) is an east west trending fault with a length of about 9 km. The fault is presumed to be a single strand that strikes north 80 degrees east to east west and dips about 80 degrees to the north.

The Chatsworth fault (8) is a reverse fault which juxtaposes Cretaceous Chatsworth formation and Paleocene Martinez formation over Miocene Modelo formation within the San Fernando Valley.

The Palos Verdes Hills fault (35) consists of several en echelon strands locally in a zone as wide as 2 km with a length of 50 miles. It strikes north between 20 and 60 degrees west with dips of 70 degrees to the southwest.

Seismic Effects

During an earthquake there are several primary geologic hazards such as ground rupture, ground shaking, landslides, and liquefaction that can adversely affect property, structures, and improvements. On hillside properties, the potential exists for landsliding from ground shaking which may adversely affect property, structures, and improvements. Properties near and along the coastline may potentially be affected by inundation due to tsunamis generated from a seismic event. The State of California has prepared maps that detail areas which may require assessment for ground rupture, landsliding and/or liquefaction. Strong ground shaking is the primary hazard that causes damage from earthquakes and these areas have been zoned with a high level of seismic shaking hazard. The historical earthquake record in Southern California is less than 200 years; therefore, potential damage from a seismic event is not limited areas that have experienced damage in the past. Based on the above discussion, earthquake insurance with building code upgrades is suggested.

There are several active and/or potentially active faults that could possibly affect the site within Los Angeles County. Although all of Southern California is within a seismically active region, some areas have a higher potential for seismic damage than others. The current scientific technology does not provide for accurate prediction of the time, location, or magnitude of an earthquake event.

It should be understood that the following discussion is an evaluation of risk and degree of potential damage to a structure if a fault were to rupture on or near the site and does not imply that a fault may or may not be present beneath the site. An assessment of damage to the structure is based on the Modified Mercalli Intensity Scale which is correlated to observed damage from seismic events. Intensity/damage associated with an earthquake is not directly correlated to magnitude. For a given magnitude of an earthquake, the intensity/damage to a structure may vary depending on the subsurface earth materials, type of fault rupture, hypocenter depth, and local building practices in effect during the construction of a structure.

An evaluation of the seismic effects on a property is designed to provide the client with rational and believable seismic data that could affect the property during the lifetime of the proposed improvements. The minimum design acceleration for a project is listed in the Building Code. It is recommended that the structural design of the proposed project be based on current design and acceleration practices of similar projects in the area. The project structural designer should review and verify all of the seismic design parameters prior to utilizing the information for the design.

Ground Rupture

Ground rupture is the result of movement from an active fault. A fault is a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. No known active fault is mapped on the subject site.

Ground Shaking

Ground shaking caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several active and potentially active faults. Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which are a response to earthquake magnitudes relative to the fault distance from the subject property. Southern California major earthquakes are generally the result of large-scale earth processes in which the Pacific plate slides northwestward relative to the North American plate at about 2 inches/year.

The potential for lurching, surface manifestations, landslides, and topographic related features from ground/seismic shaking can occur almost anywhere in Southern California. Proper maintenance of properties can mitigate some of the potential for these types of manifestations, but the potential cannot be completely eliminated. Many structures were built before earthquake codes were adopted; others were built according to codes formulated when less was known about the intensity of near-fault shaking. Therefore, the margin of safety is difficult to quantify.

A publicly available computer program provided by the United States Geological Survey (USGS) was utilized for the probabilistic prediction of peak horizontal ground acceleration from digitized design maps of Maximum Considered Earthquake (MCE) ground response. A summary of the seismic design parameters is provided in Appendix III. The project structural designer should verify all of the input parameters and review all of the resulting seismic design parameters prior to utilizing the information for the design.

Earthquake Induced Landslides

The State of California has prepared Seismic Hazard Zone Reports to regionally map areas of potential increased risk of permanent ground displacement based on historic occurrence of landslide movement, local topographic expression, and geological and geotechnical subsurface conditions. The maps may not identify all areas that have potential for earthquake-induced landslides, strong ground shaking, or other earthquake-related geologic hazards. The subject site is not located within an earthquake-induced landslide hazard zone on the State of California Seismic Hazard Map.

Liquefaction

The State of California has prepared Seismic Hazard Zone Reports to regionally map areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement. The maps may not identify all areas that have potential for liquefaction, strong ground shaking, and other earthquake and geologic hazards. The subject site is not located within a liquefaction hazard zone on the State of California Seismic Hazard Zone Map.

A detailed subsurface analysis can be performed to determine the liquefaction potential on the subject site and provide recommendations to mitigate the effects of liquefaction. A proposal for a detailed analysis will be prepared if requested.

Liquefaction is a process by which sediments below the water table temporarily lose strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; occasionally gravel liquefies. Liquefaction can occur when seismic waves, primarily shear waves, pass through saturated granular layers distorting the granular structure, and causing loosely packed groups of particles to collapse. These collapses increase the pore-water pressure between grains if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid.

In the liquefied condition, soil may deform with little shear resistance; deformations large enough to cause damage to buildings and other structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the material, the depth, thickness and areal extent of the liquefied layer, the ground slope and the distribution of loads applied by buildings and other structures.

Liquefaction induced ground deformations (detailed below) will have an effect on the proposed and existing development that can result in significant structural damage, collapse or partial collapse of a structure, especially if there is significant differential settlement or lateral spreading between adjacent structural elements. Even without collapse, significant settlement or lateral spreading could result in significant structural damage including, but not limited to, blocked doors and windows that could trap occupants.

Surface Manifestations

The determination of whether surface manifestation of liquefaction (such as sand boils, ground fissures etc.) will occur during earthquake shaking at a level-ground site can be made using the method outlined by Ishihara (1985). It is emphasized that settlement may occur, even with the absence of surface manifestation. Youd and Garris (1994 and 1995) evaluated the Ishihara method and concluded that the method is not appropriate for level ground sites subject to lateral spreading and/or ground oscillation.

Lateral Spreads

Whereas the potential for flow slides may exist at a building site, the degradation in undrained shear resistance arising from liquefaction may lead to limited lateral spreads (of the order of feet or less) induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear stress biases on essentially horizontal ground (Youd, 1995). At larger cyclic shear strains, the effects of dilation may significantly increase post liquefaction undrained shear resistance. However, incremental permanent deformations will still accumulate during portions of the earthquake load cycles when low residual resistance is available. Such low resistance will continue even while large permanent shear deformations accumulate through a ratcheting effect. Such effects have recently been demonstrated in centrifuge tests to study liquefaction induced lateral spreads, as described by Balakrishnan et al. (1998). Once earthquake loading has ceased, the effects of dilation under static loading can mitigate the potential for a flow slide.

It is clear from past earthquakes that damage to structures can be severe, if permanent ground displacements on the order of several feet occur. However, during the Northridge earthquake significant damage to building structures (floor slab and wall cracks) occurred with less than one (1) foot of lateral spread. The complexities of post-liquefaction behavior of soils noted above, coupled with the additional complexities of potential pore water pressure redistribution effects and the nature of earthquake loading on the sliding mass, lead to difficulties in providing specific guidelines for lateral spread evaluations.

Seismically Induced Settlements

Seismic settlement occurs when cohesionless soils densify as result of ground shaking. Typically, seismically induced settlement is greatest in loose cohesionless sands. Lee and Albaisa (1974) and Yoshimi (1975) studied the volumetric strains (or settlements) in saturated sands due to dissipation of excess pore pressures generated in saturated granular soils by the cyclic ground motions. The volumetric strain, in the absence of lateral flow or spreading, results in settlement. Liquefaction-induced settlement could result in collapse or partial collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Even without collapse, significant settlement could result in blocked doors and windows that could trap occupants.

CONCLUSIONS

1. Based on the results of this investigation and a thorough review of the proposed development, as discussed, the project is suitable for the intended use providing the following recommendations are incorporated into the design and subsequent construction of the project. Also, the development must be performed in an acceptable manner conforming to building code requirements of the controlling governing agency.

2. Based on the State of California Seismic Hazard Maps, the subject site is not located within a liquefaction hazard zone.

3. Based on the State of California Seismic Hazard Maps, the subject site is partially located within an earthquake-induced landslide hazard zone.

4. The SITE CLASS based on California Building Code is C.

5. Based upon field observations, laboratory testing and analysis, the bedrock found in the explorations has sufficient strength to support the proposed development.

RECOMMENDATIONS

Specific

1. To create a uniform building pad for the proposed driveway, the existing fill should be removed to competent bedrock and replaced as compacted fill. In addition, the proposed removals should extend a minimum of two feet below the proposed foundations and outside of the building footprint.
2. The proposed residence and swimming pool should be supported on foundations embedded into bedrock.
3. The proposed swimming pool should be supported entirely on piles embedded into bedrock. The swimming pool should be designed as outlined in the Pool section below.
4. The soils chemistry results should be incorporated into the design of the proposed project. Our firm does not practice corrosion mitigation, it is recommended that a corrosion specialist be retained to perform pertinent laboratory testing after completion of earthworks to verify the corrosivity potential of finish grades, and develop adequate and relevant prevention recommendations; if any are needed.
5. The property owner shall maintain the site as outlined in the Drainage and Maintenance Section.

Building Setbacks

The construction of buildings and structures on or adjacent to slopes steeper than 3:1 (horizontal to vertical) in gradient shall be setback from the slopes in accordance with the requirements of the applicable governmental agency.

In general, all foundations on or adjacent to a descending slope shall be located a distance of one-third of the vertical height of the slope (H/3) to provide vertical and lateral support for the foundation. This distance is measured horizontally from the face of the foundation to the face of the bearing material. This horizontal distance does not need to exceed 40 feet. Where the slope is steeper than 1:1 (horizontal to vertical), the required setback shall be measured from an imaginary plane at 45 degrees to the horizontal, projected upward from the toe of the slope.

In general, buildings and structures on or adjacent to an ascending slope shall be located a distance of one-half of the vertical height of the slope (H/2) to provide sufficient protection from slope drainage, erosion, and shallow failures. This distance is measured horizontally from the face of the building/structure to the toe of the slope. This horizontal distance does not need to exceed 15 feet. Where the slope is steeper than 1:1 (horizontal to vertical), the toe is considered to be at the intersection of a horizontal plane from the top of the foundation and an imaginary plane tangent to the slope at 45 degrees to the horizontal.

The construction of swimming pools on or adjacent to slopes shall maintain setback distances equal to one-half of the setback distances for buildings and foundations. Swimming pools on or adjacent to a descending slope shall be located a distance of one-sixth of the vertical height of the slope (H/6). This horizontal distance does not need to exceed 20 feet. Swimming pools on or adjacent to an ascending slope shall be located a distance of one-fourth of the vertical height of the slope (H/4). This horizontal distance does not need to exceed (7½) feet.

Drainage and Maintenance

Maintenance of properties must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The property owner shall be fully responsible for dampness or water accumulation caused by alteration in grading, irrigation or installation of improper drainage system, and failure to maintain drain systems. The following are specific drainage, maintenance, and landscaping recommendations. Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to the improvements. It is the responsibility of the property owner to ensure that improvements, structures and drainage devices are maintained in accordance with the following recommendations and the requirements of all applicable government agencies.

Drainage

Positive pad drainage should be incorporated into the final plans. The pad should slope away from the footings at a minimum five percent slope for a horizontal distance of ten feet. In areas where there is insufficient space for the recommended ten foot horizontal distance concrete or other impermeable surface should be provided for a minimum of three feet adjacent the structure. Pad drainage should be at a minimum of two percent slope where water flow over lawn or other planted areas. Drainage swales should be provided with area drains about every fifteen feet. Area drains should be provided in the rear and side yards to collect drainage. All drainage from the pad should be directed so that water does not pond adjacent to the foundations or flow toward them. Roof gutters and downspouts are required for the proposed structures and should be connected into a buried area drain system. All drainage from the site should be collected and directed via non-erosive devices to a location approved by the building official. Area drains, subdrains, weep holes, roof gutters and downspouts should be inspected periodically to ensure that they are not clogged with debris or damaged. If they are clogged or damaged, they should be cleaned out or repaired.

Landscaping (Planting)

The property owner is advised not to develop planter areas between patios, sidewalk and structures. Planters placed immediately adjacent to the structures are not recommended. If planters are proposed immediately adjacent to structures, impervious above-grade or below-grade planter boxes with solid bottoms and drainage pipes away from the structure are suggested. All slopes should be maintained with a dense growth of plants, ground-covering vegetation, shrubs and trees that possess dense, deep root structures and require a minimum of irrigation. Plants surrounding the development should be of a variety that requires a minimum of watering. It is recommended that a landscape architect be consulted regarding planting adjacent to improvements. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes should be reviewed by the landscape architect.

Irrigation

An adequate irrigation system is required to sustain landscaping. Over-watering resulting in runoff and/or ground saturation must be avoided. Irrigation systems must be adjusted to account for natural rainfall conditions. Any leaks or defective sprinklers must be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems must be adjusted for rainy seasons. A landscape architect should be consulted to determine the best times for landscape watering and the proper usage.

Pools/Plumbing

Leakage from a swimming pool or plumbing can produce a perched groundwater condition that may cause instability or damage to improvements. Therefore, all plumbing should be leak-free.

Grading and Earthwork

Proposed grading will consist of removal & recompaction and foundation excavations.

Foundations

It is recommended that the proposed structure be founded into bedrock. All foundations shall maintain the required code setback from any slope.

The minimum skin-friction pile diameter is 24 inches. Piles should extend into the bedrock a minimum of 10 feet. The piles may be proportioned using the Pile Capacity Chart in Appendix III. All piles shall be considered fixed 3 feet into competent bedrock. All piles should be designed to resist a creep force of 1000 pounds per lineal foot for each foot of shaft exposed to the soils above the bedrock.

The minimum continuous footing size is 24 inches wide and 24 inches deep into the bedrock, measured from the lowest adjacent grade. Continuous footings may be proportioned, using a bearing value of 3000 pounds per square foot. Column footings placed into the bedrock may be proportioned, using a bearing value of 3500 pounds per square foot, and should be a minimum of 2 feet in width and 24 inches deep, below the lowest adjacent grade.

The bearing values given above are net bearing values; the weight of concrete below grade may be neglected. These bearing values may be increased by one-third (1/3) for temporary loads, such as, wind and seismic forces.

Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the bedrock. A coefficient of friction of 0.3 may be used between the foundations and the bedrock. The passive resistance may be assumed to act as a fluid with a density of 500 pounds per square foot, with a maximum earth pressure of 5000 pounds per square foot. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third. Piles may be considered isolated if the distance between piles is greater that (2.5) time the pile diameter. For isolated poles, the allowable passive earth pressure may be doubled.

All footing excavation depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing materials will not be acceptable to this office.

Settlement

Settlement of the proposed residence and swimming pool will occur. Settlement of 1/8 to 1/4 inches between walls, within 20 feet or less, of each other, and under similar loading conditions, are considered normal. Total settlement on the order of 1/2 inches should be anticipated.

Expansive Soils

Expansive soils were not encountered on the subject property. Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil. Expansive soils heave when moisture is introduced and contract as they dry. During inclement weather and/or excessive landscape watering, moisture infiltrates the soil and causes the soil to heave (expansion). When drying occurs the soils will shrink (contraction).

Repeated cycles of expansion and contraction of soils can cause pavement, concrete slabs on grade and foundations to crack. This movement can also result in misalignment of doors and windows. To reduce the effect of expansive soils, foundation systems are usually deepened and/or provided with additional reinforcement design by the structural engineer. Planning of yard improvements should take into consideration maintaining uniform moisture conditions around structures. Soils should be kept moist, but water should not be allowed to pond. These designs are intended to reduce, but will not eliminate deflection and cracking and do not guarantee or warrant that cracking will not occur.

Excavations

Excavations ranging in vertical height up to (16) feet will be required for the proposed basement. Conventional excavation equipment may be used to make these excavations. Excavations should expose fill. The fill is suitable for vertical non-surcharged excavations up to (5) feet, cuts above (5) feet in height shall be trimmed back at 1:1 (H:V) slope gradient or shored per our recommendations. The bedrock is suitable for vertical non-surcharged excavations up to (5) feet, cuts above (5) feet in height shall be trimmed back st 1:1 (H:V) slope gradient or shored per our recommendations Surcharged cuts and/or cuts removes support from any adjacent properties/structures *(Note: lateral support shall be considered removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property)* should be shored. This should be verified by the project geotechnical engineer during construction so that modifications can be made if variations in the soil occur.

Caving was not noted in any of our test pits, however the chances of caving will increase within larger scale excavations and should be anticipated particularly in coarse-grained material and under groundwater table, and saturated fine-grained material may cave as well. Since caving can be affected by many known and unknown factors, test boring(s) is/are suggested before initiating drilling on site to understand the need for caving mitigation on site.

<u>Proper installation of shoring is the responsibility of the contractor. The adjacent property owners must be advised of the risks and the owner and builder should provide arrangements to repair any possible damages.</u>

The cantilevered retrained shoring shall be designed per the following table. A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. In addition to lateral earth pressure, these retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings. The design values provided in the following table assuming that water table will be maintained below the bottom of the cut until the permanent subgrade structure is constructed.

Depth of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (p.c.f.) *Triangular Distribution of Pressure*	Restrained Shoring System Lateral Earth Pressure (p.s.f./ft)* *Trapezoidal Distribution of Pressure*
Up to 12	20	15H

In addition to lateral earth pressure, these retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge.

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

The minimum shoring pile diameter is (18) inches. Shoring piles should extend into the alluvium a minimum of (10) feet below the bottom of the proposed excavation. All piles shall be considered fixed (3) feet into bedrock. Lagging is required.

Constriction excavations shall be made under the supervision of a qualified "competent person" along with periodic review performed by this office. A "competent person" as defined by California/OSHA, is one who is capable of identifying existing and predictable hazards that are unsanitary or dangerous to employees. The competent person has the authority to impose prompt corrective measures to eliminate these hazards.

All excavations should be stabilized within 30 days of initial excavation. If this time is exceeded, the project geotechnical engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of the excavation, nor to flow toward it. All excavations should be protected from inclement weather. This is required to keep the surface of the open excavation from becoming saturated during rainfall. Saturation of the excavation may result in a relaxation of the soils which may result in failures. Excavations should be kept moist, not saturated, to reduce the potential for raveling and sloughing during construction. No vehicular surcharge should be allowed within three feet (3') of the top of cut.

Temporary Shoring

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than two diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be taken from the attached pile capacity chart. The minimum depth of embedment for shoring piles is 10 feet below the bottom of the excavation plane.

Casing may be required should caving be experienced in the saturated earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

To allow for continuous placement of the pile foundations, it is recommended that piles be placed under the continuous inspection of the project Soils Engineer.

Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 10 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present.

<u>Lagging</u>

Due to the cohesionless nature of the underlying earth materials, it is anticipated that lagging will be required throughout the entire depth of the excavation. It is anticipated that lagging will be required throughout the entire depth of the excavation. Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the earth materials, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but may be limited to a maximum of 400 pounds per square foot.

The maximum vertical cut during the lagging installation shall be limited to (5) feet in height. If loose or cohesionless material is encountered during the lagging installation, the maximum vertical cut during the lagging installation shall be limited to (2-3) feet in height.

<u>Tied-Back Anchors</u>

Tie-back anchors may be used to resist lateral loads. Friction anchors consisting of high stress thread bars are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities.

For preliminary design purposes, the drilled friction anchors may be designed for a skin friction of 300 pounds per square foot. Pressure grouted anchor may be designed for a skin friction of 2,000 pounds per square foot. Where belled anchors are utilized, the capacity of belled anchors may be designed by assuming the diameter of the bonded zone is equivalent to the diameter of the bell. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. The capacities of the anchors should be determined by testing of the initial anchors as outlined below under the Tie-back Anchor Testing section.

Tie-Back Anchor Testing

The installation of the anchors and the testing of the completed anchors should be observed by are presentative of a qualified geotechnical firm. The geotechnical engineer or his representative should select at least two of the initial anchors for 24-hour 200% tests and two additional anchors for "quick" 200% tests to verify in the field the friction value assumed in this report. Also, we recommend that the 200% tests be performed at representative locations around the site and not be concentrated in a single area, and the total number of 200% tests (quick and 24-hour) performed should not be less than minimum of 10 percent of the number of installed tiebacks.

The total deflection during 24-hour 200% tests should not exceed 12 inches during loading; the anchor deflection should not exceed ¾ inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than ½ inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

For the quick 200% tests, the test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed ¼ inch during the 30-minute period.

All of the production anchors should be pre-tested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked off at the design load. The locked-off load should be verified by rechecking the load on the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked off within 10% of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors be installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by a representative of this firm. Minor caving during drilling of the anchors should be anticipated.

Raker Braces

The proposed soldier piles may be laterally supported by raker braces supported by temporary footings, or dead-men. Temporary footings inclined at an angle of 45 degrees to the horizontal may be designed for an allowable bearing value of 1500 psf. To utilize this allowable bearing pressure, the inclined footings should be a minimum of 24 inches in width, and should be embedded a minimum of 24 inches below the lowest adjacent grade. The allowable bearing provided above may be increased by 10% for each additional foot of depth to maximum of (3000) pounds per square foot.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. The maximum deflection shall not exceed one-half inch (1/2) inch at the top of the shored embankment where a structure is within 1:1 (h:v) plane projected up from the base of the excavation, and for a maximum lateral deflection of (1) inch provided there are no structures within a 1:1 (h:v) plane projected up from the base of excavation. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent streets and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring and Pre-Construction Survey

Because of the depth of the excavation, periodic shoring monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable. The intent of this program will be to produce an accurate and on-going record of the horizontal and vertical deflections of the temporary shoring system.

Monitoring of the excavation performance should be started prior to the beginning of the initial excavation. The weekly schedule of performance monitoring may be modified as the job progresses. Once the subterranean structure has been constructed, monitoring of the performance will no longer be required.

Complete documentation of the pre- and post-construction conditions of the adjacent improvements shall be undertaken. A sufficient number of photographs to establish the existing condition of all adjacent structures.

It is recommended that the client's representative prepare a pre-construction survey prior to site development. The pre-construction survey should document existing site conditions and performance of offsite structures prior to construction (where applicable). It is recommended that any temporary shoring excavations at the site be conducted with frequent observation by a representative from this office. If adverse conditions are encountered during excavations, additional recommendations may be necessary.

The excavations should be monitored by a representative from this office. The monitoring may be provided by a licensed surveyor during construction to determine deformation monitoring of adjacent structures and possible deflection of the shoring piles and/or temporary excavations. It is recommended that the survey monitoring performed by others, be provided weekly for the first month and monthly afterward for a period of 6 months or as determined by your representatives (if applicable). Additional design recommendations (i.e. bracing, tie back) may be necessary depending on field conditions, and should be determined by the project engineer (if applicable).

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many building officials require that shoring installation should be performed during the **continuous observations** of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

Excavations Maintenance – Erosion Control

The following recommendations should be considered a part of the excavation/erosion control plan for the subject site and are intended to supplement, but not supersede nor limit the erosion control plans produced by the Project Civil Engineer and/or Qualified SWPPP Developer. These recommendations should be implemented during periods required by the Building Code (typically between the months of October and April) or at any time of the year prior to a predicted rain event. Consideration should also be given to potential local sources of water/runoff such as existing drainage pipes or irrigation systems that remain in operation during construction activities.

Open Excavations:

All open excavations shall be protected from inclement weather, including areas above and at the toe of the excavation. This is required to keep the excavations from becoming saturated. Saturation of the excavation may result in a relaxation of the soils which may result in failures. Water/runoff should be diverted away from the excavation and not be allowed to flow over the excavation in a concentrated manner.

Hillside Excavations:

All hillside excavations shall be protected during inclement weather and should extend beyond the edges of the excavations in all directions. Plastic sheeting along with stakes, ropes and sandbags may be used to provide protection of the excavations. Water/runoff should be diverted away from the excavation and not be allowed to flow over the excavation.

The project Civil Engineer should provide a plan depicting the required limits of erosion control. Slopes around an open excavation should be trimmed to slope away from the open excavation so that water/runoff will not drain into the excavation. Any trees or planters that might cause failures around an open excavation shall be anchored safely. After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work.

Open Trenches/Foundation Excavations:

No water should be allowed to pond adjacent to or flow into open trenches. All open trenches shall be covered with plastic sheeting that is anchored with sandbags. Areas around the trenches should be sloped away from the trenches to prevent water runoff from flowing into or ponding adjacent to the trenches.

After the inclement weather has ceased, the excavations shall be reviewed by the project geotechnical engineer and geologist for safety prior to recommencement of work. Foundation excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment and contact with the bearing material have been maintained.

Open Pile/Caisson Excavations:

All pile/caisson excavations should be reviewed and poured prior to the onset of inclement weather. It is not recommended that any pile/caisson excavations remain open through any inclement weather. However, if it is necessary to leave pile/caisson excavations open during inclement weather, all water and runoff shall be diverted away from and prevented from entering the pile/caisson excavations. Pile/caisson excavations that remain open during inclement weather shall be reviewed by the project geotechnical engineer and geologist prior to the placement of steel and concrete to ensure that proper embedment has been maintained. The base of all end-bearing caissons shall be re-cleaned to ensure contact with the proper bearing material. All stockpiled cuttings from the pile borings shall be removed.

Grading In Progress:

During the inclement time of the year, or during periods prior to the onset of rain, all fill that has been spread and is awaiting compaction shall be compacted before stopping work for the day or before stopping work because of inclement weather. These fills, once compacted, shall have the surface sloped to drain to one area where water may be removed.

Additionally, it is suggested that all stock-piled fill materials be covered with plastic sheeting. This action will reduce the potential for the moisture content of the fill from becoming too high for compaction. If the fill stockpile is not covered during inclement weather, then aerating the fill to reduce the moisture content would be required. This action is generally very time consuming and may result in construction delays.

Work may recommence, after the rain event, once the site has been reviewed by the project geotechnical engineer.

Retaining Walls

Cantilever retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls up to (12) feet in height may be designed per the following table. The 'active' pressure assumes that the wall will be allowed to deflect 0.01H to 0.02H. Basement walls and other walls where horizontal movement is restricted at the top or not allowed to deflect shall be designed for at-rest pressure.

Surface Slope of Retained Material Horizontal to Vertical	Active Equivalent Fluid Weight p.c.f.	At-Rest Pressure Fluid Weight p.c.f.
Level	30	60

The active and at-rest equivalent fluid pressure presented herein are based on assumed certified compacted fill shear strength parameters, the proposed compacted fill shear strength parameters shall be verified during the compacted fill construction. If expansive soils were to be used in the compacted fill behind the proposed retaining wall, the active and at-rest equivalent fluid pressure shall be increased by (15) p.c.f.

In addition to lateral earth pressure, these retaining walls should be designed to resist the surcharge imposed by the proposed structures, footings, any adjacent buildings, or by adjacent traffic surcharge, per the attached figures 11 and 12 obtained from the Naval Facilities Engineering Command, Design Manual 7.02 (Foundation and Earth Structures, pages 74 and 75).

The wall pressure stated assumes that the wall has been backfilled as outlined below with a permanent drainage system. Proper compaction of the backfill is recommended to provide lateral support to adjacent properties. Even with proper compaction of required backfill, settlement of the backfill may occur. Accordingly, utility lines, footings, slabs, or falsework should be planned and designed to accommodate potential settlement.

Walls to be backfilled must be reviewed by the project Geotechnical Engineer prior to commencement of the backfilling operation.

1. Adequate permanent drainage is required behind the wall to minimize the buildup of hydrostatic pressures. A perforated pipe, with perforations placed down, shall be installed at the base of the wall footing. The pipe shall be encased in at least one cubic foot (1') of three-quarter inch (3/4") gravel. The pipe shall exit from behind the retaining wall and drain to a location approved by the architect or civil engineer.

   When space does not permit the installation of standard pipe and gravel drainage system, i.e. walls adjacent the property line, a flat drainage product is acceptable subject to approval of the governing agency. It is recommended that a drainage composite geotextile (such as MiraDrain / QuickDrain) be placed at the base of the proposed retaining wall. The drainage composite geotextile will provide comparable drainage to the conventional four inch perforated pipe encased in gravel per Code Sections 1805.4.2 and 1805.4.3.

   Where shoring does not allow for the installation of a standard subdrainage system behind the proposed retaining wall, a rock pockets system may be utilized. The client shall submit a letter of modification and obtain the City of Glendale approval prior to using this option. The rock pockets system should drain through the proposed retaining wall. The (3/4") gravels pockets should be a minimum of one cubic foot (1'X1'X1' or 2'X2'X4") and should be installed no more than (8) feet on center.

   If a drainage system is not provided the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure in Retaining Wall section. The entire wall should be design for full hydrostatic pressure based on a water level at the ground surface. In addition, floors would need to be designed for hydrostatic uplift and waterproofed.

2. A continuous vertical drain, consisting of a gravel blanket six inches (6") thick or geotextile vertical drainage system, shall be placed along the back side of the wall to within 2 feet of the ground surface.

3. Water and moisture affecting retaining walls is a common post-construction complaint. Poorly applied or omitted waterproofing can lead to standing water inside the building or efflorescence on the wall.

   It is recommended that the retaining walls be waterproofed. Waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer. Creative Geotechnical, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing.

4. After the wall backdrain system has been placed and the waterproofing installed, fill may be placed, if sufficient room allows, in layers not exceeding four inches (4") in thickness and compacted to 90 percent of the maximum density, as determined by ASTM D 1557. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for fill, the fill material shall be compacted to a minimum of (95) percent of the maximum dry density.

5. Where space does not permit compaction of material behind the wall (<24 inches wide, up to 10 feet in height), a granular backfill shall be used. This granular backfill shall consist of one-half inch (1/2") to three-quarter inch (3/4") crushed rock and should be densified by tamping into place. The crushed rock backfill should not exceed a depth of ten feet.

6. All granular free-draining wall backfills shall be capped with a clayey compacted soil within the upper two feet (2') of the wall backfill. This compacted material should start below the required wall freeboard.

7. A concrete-lined swale drain should be placed behind any retaining wall that can intercept surface runoff from upslope areas. This surface runoff shall be transferred to an area approved by the building official.

8. A minimum freeboard of two (2) feet shall be maintained at all times. Any slough, debris or trash should be removed immediately. Swales shall be maintained, by sealing any and all cracks or repairing breaks that occur over the life of the swale.

<u>Lateral Earth Pressure Due to Earth Motion</u>

Retaining walls should be designed to resist an active earth pressure due to earth motion, if required by the building official, distributed as a triangle pressure. Retaining walls up to (12) feet in height may be designed per the following table. The seismic equivalent fluid pressure is in addition to static earth pressures.

The seismic loading is based on a horizontal acceleration coefficient of ½ of 2/3 $PGA_M = 0.36$.

Surface Slope of Retained Material Horizontal to Vertical	Seismically Induced Earth Pressure - Equivalent Fluid Weight p.c.f.
Level	10

Slabs on Grade

Slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way and supported on certified compacted fill and/or bedrock. Alternatively, the slab on grade may be structurally supported on the recommended foundation system. Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs, and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet, control joints are not required for structural slab supported on the recommended foundations system option. Two-car garage slabs should be quartered or saw cut slabs and isolated from the stem wall footing to mitigate cracking. These recommendations are considered minimums unless superseded by the project structural engineer. Prior to placing the vapor retarder/waterproofing the moisture content of the subgrade should be raised to 120 percent of the optimum moisture content to a depth of 18 inches.

It is recommended that a vapor retarder/waterproofing be placed below the concrete slab on grade. Vapor/moisture transmission through slabs does occur and can impact various components of the structure.

Vapor retarder/waterproofing design and inspection of installation is not the responsibility of the geotechnical engineer (most often the responsibility of the architect). Creative Geotechnical, Inc. does not practice in the field of water and moisture vapor transmission evaluation/mitigation. Therefore, we recommend that a qualified person/firm be engaged/consulted to evaluate the general and specific water and moisture vapor transmission paths and any impact on the proposed development. This person/firm should provide recommendations for mitigation of potential adverse impact of water and moisture vapor transmission on various components of the structure as deemed necessary. The actual waterproofing design shall be provided by the architect, structural engineer or contractor with experience in waterproofing

In order to promote good building practices and alert the rest of the design/construction team of some of the appropriate standards and expert recommendations pertaining to vapor barriers/retarders, the waterproofing designer should consider recommending and citing specific performance characteristics. The following paragraph includes some of the standards and expert recommendations and should be considered for use waterproofing designer own recommendations:

Vapor barrier shall consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditions (ASTM E 1745 Section 7.1 and Sub-Paragraph 7.1.1-7.1.5): less than 0.01 perms [grains/(ft^2-hr-inHg)] and comply with the ASTM E 1745 Class A requirements. Install vapor barrier according to ASTM E1643, including proper perimeter seal. Basis of design: Stego Wrap Vapor Barrier 15 mil and Stego Crete Claw Tape (perimeter seal tape). Approved Alternatives: Vaporguard by Reef Industries, Sundance 15 mil Vapor Barrier by Sundance Inc.

Decking

Exterior decking slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at 16 inches on center each way and supported on certified compacted fill and/or bedrock. Alternatively, the slab on grade may be structurally supported on the recommended foundation system. Provisions for cracks should be incorporated into the design and construction of the decking. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. Decking planned adjacent to lawns, planters or adjacent to descending slopes should be provided with a 12-inch thickened edge. The deck reinforcement should be bent down into the edge. These recommendations are considered minimums unless superseded by the project structural engineer. Prior to placing the concrete the subgrade should be raised to 120 percent of the optimum moisture content to a depth of 18 inches.

Slough Protection

Some surficial erosion/surficial slope failures may occur during inclement weather. In order to mitigate this possible occurrence from impacting the rear yard area and the proposed structure, it is recommended that the freeboard on the rear yard retaining wall be a minimum of two feet.

The sloughed materials behind these walls must be cleaned out each time deposition occurs, to allow them to function as envisioned.

Some surficial erosion/surficial slope failures may occur during inclement weather. In order to mitigate this possible occurrence from impacting improvements all slopes should be planted and maintained as described in the Drainage and Maintenance section. In addition, deep-rooted shrubs should be planted in staggered rows that do not exceed 10 feet on center over the slope face.

Trench Backfill

Standard construction techniques should be sufficient for site utility trench excavations. It is recommended that utility trenches not be planned or constructed parallel to and below a 2:1 (h:v) plane projected down from the base of the outer edge of conventional foundations. If utilities are required within this zone, foundations should be deepened to satisfy this recommendation. The surface of utility trenches frequently settles, even when backfill is placed under optimum conditions. Structural units or pavement placed over the trench backfill should be designed to accommodate such movements.

Backfill of all utility trenches should be placed by mechanical compaction methods and should be tested and certified. Flooding and/or jetting of other trench backfill does not create compact trench backfill. Sand (with a minor Sand Equivalent of 30+) may be placed around utility lines and be properly jetted and should be limited to around the pipe and 6 inches above the pipe.

Utilities bedded in sand can serve as conduits to bring subsurface water onto the site. It is recommended that a slurry seal be placed around the pipes at their entrance to the property.

Soil Shrinkage and Bulking

Bulking is the increase of volume of the earth material when it is excavated. Shrinkage is the decrease in volume of the earth material when it is compacted. Contingencies should be made to adjust the earthwork balance when grading is in progress and the bulk-shrink is better defined.

The bulking factor is estimated to range from 10 to 20 percent. The shrinkage factor is estimated to range from 10 to 20 percent. These estimated values are based on the limited data collected from the subsurface exploration, available information at the time of this report, laboratory test data, and our experience in the site area.

These estimates may vary depending on contractor methods and few known & unknown factors during the site grading. Losses from site clearing and grubbing operations mat effect quantity calculations and should be taken into account. Actual shrinkage/bulking of the soil may vary.

Jobsite Safety

Neither the processional activities of Creative Geotechnical, Inc., nor the presence of Creative Geotechnical, Inc. employees and subconsultants at a construction/project site, shall relieve the contractor of its obligations, duties and responsibilities including, but not limited to, construction means, methods, techniques, or procedures necessary for performing, superintending and coordination the work in accordance with the contract documents and any health or safety precaution required by any regulatory agencies. Creative Geotechnical, Inc. and its personal have no connection with their work or any health or safety programs or procedures. The general contractor shall be the sole responsible for jobsite safety.

REVIEWS

Plan Review and Plan Notes

The final grading, building, and/or structural plans shall be reviewed and approved by the consultants to ensure that all recommendations are incorporated into the design or shown as notes on the plan.

The final plans should reflect the following:

1. The Preliminary Geologic and Geotechnical Engineering Investigation by Creative Geotechnical, Inc. is a part of the plans.

2. Plans must be reviewed and signed by Creative Geotechnical, Inc.

3. The project geotechnical engineer and/or geologist must review all grading.

4. The project geotechnical engineer and/or geologist shall review all foundations.

Construction Review

Reviews will be required to verify all geologic and geotechnical work. It is required that all footing excavations, seepage pits, and grading be reviewed by this office. This office should be notified at least **two working days** in advance of any field reviews so that staff personnel may be made available.

The property owner should take an active role in project safety by assigning responsibility and authority to individuals qualified in appropriate construction safety principles and practices. Generally, site safety should be assigned to the general contractor or construction manager that is in control of the site and has the required expertise, which includes but not limited to construction means, methods and safety precautions. When excavations exist on a site, the area should be fenced, and warning signs posted. All pile excavations must be properly covered and secured. Soil generated from excavations and cuts should not be spilled over descending slopes or piled against fences.

LIMITATIONS

General

This report and the explorations are subject to the following conditions. Please read this section carefully; it limits our liability.

The validity of our recommendations presented herein is dependent upon review of the engineering geology and geotechnical engineering aspects of the project during construction by this firm. This report is intended to be used only in its entirety. No portion or section of the report, by itself, is designed to completely represent any aspect of the project described herein. If any reader requires additional information or has questions regarding this report, Creative Geotechnical, Inc. should be contacted.

The subsurface conditions described herein have been projected from limited subsurface explorations and laboratory testing. The explorations and testing presented in our report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions. Subsurface conditions were interpreted on the basis of field explorations and past experience. Although, between exploratory excavations, subsurface earth materials may vary in type, strength and many other properties from those interpreted. The findings, conclusions and recommendations presented herein are for the soil conditions encountered in the specific locations. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Other soil conditions due to non-uniformity of the soil conditions or manmade alterations may be revealed during construction. If subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that further recommendations may be made if required. If it is desired to minimize the possibility of such changes, additional explorations and testing can/should be performed.

The exploration was performed only on portion of the site, and cannot be considered as indicative of the portions of the site not explored.

Creative Geotechnical, Inc. should be consulted to determine if additional work is required when our work is used by others or if the scope of the project has changed. If the project is delayed for more than

one year, this office should be contacted to verify the current site conditions and to prepare an update report.
Findings, conclusions and recommendations presented herein are based on experience and background. Therefore, findings, conclusions and recommendations are professional opinions and are not meant to indicate a control of nature.

Fluctuations in groundwater level may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be hazardous to health and property and saturation of earth materials can cause subsidence or slippage of the site.

This preliminary report provides information regarding the findings on the subject property. It is not designed to provide a guarantee that the site will be free of hazards in the future, such as but not limited to, landslides, slippage, liquefaction, expansive soils, differential settlement, debris flows, seepage, concentrated drainage or flooding. It may not be possible to eliminate all hazards, but homeowners must maintain their property and improve deficiencies to minimize these hazards. Any liability in connection herewith shall not exceed the fee for this report.

The exploration was performed only on portion of the site, and cannot be considered as indicative of the portions of the site not explored.

Engineering for the proposed project should not begin until approval of the investigation is granted by the local building official. Significant changes in our recommendations may result due to the building department review process.

This report was prepared on the basis of the preliminary development plan furnished. Final plans should be reviewed by our office as additional geotechnical work may be required.

This report may not be copied. If you wish to purchase additional copies, you may order them from this office.

CONSTRUCTION NOTICE

Construction can be challenging. Creative Geotechnical, Inc. has provided this report to advise you of the general site conditions, geotechnical feasibility of the proposed project, and overall site stability. It must be understood that the professional opinions provided herein are based upon subsurface data, laboratory testing, analyses, and interpretation thereof. Recommendations contained herein are based upon surface reconnaissance and minimum subsurface explorations deemed suitable by your consultants.

Although quantities for foundation concrete and steel may be estimated based on the findings provided in this report, provision should be made for possible changes in quantities during construction. If it is desired to minimize the possibility of such changes, additional exploration and testing should be considered. However, you must be aware that depths and magnitudes will most likely vary between explorations given in the report.

The observations and testing during construction are beyond the scope of this investigation and budget and are conducted on a time and material basis.

We appreciate the opportunity of serving you on this project. If you have any questions concerning this report, please contact the undersigned.

Respectfully submitted,
CREATIVE GEOTECHNICAL, INC.

Raymond M. Haddad
Project Engineer
GE 2985
RMH/PRK: -200416-1

Pedram Rahimikian
Project Geologist
CEG 2683

Distribution: (3) Addressee

APPENDIX I

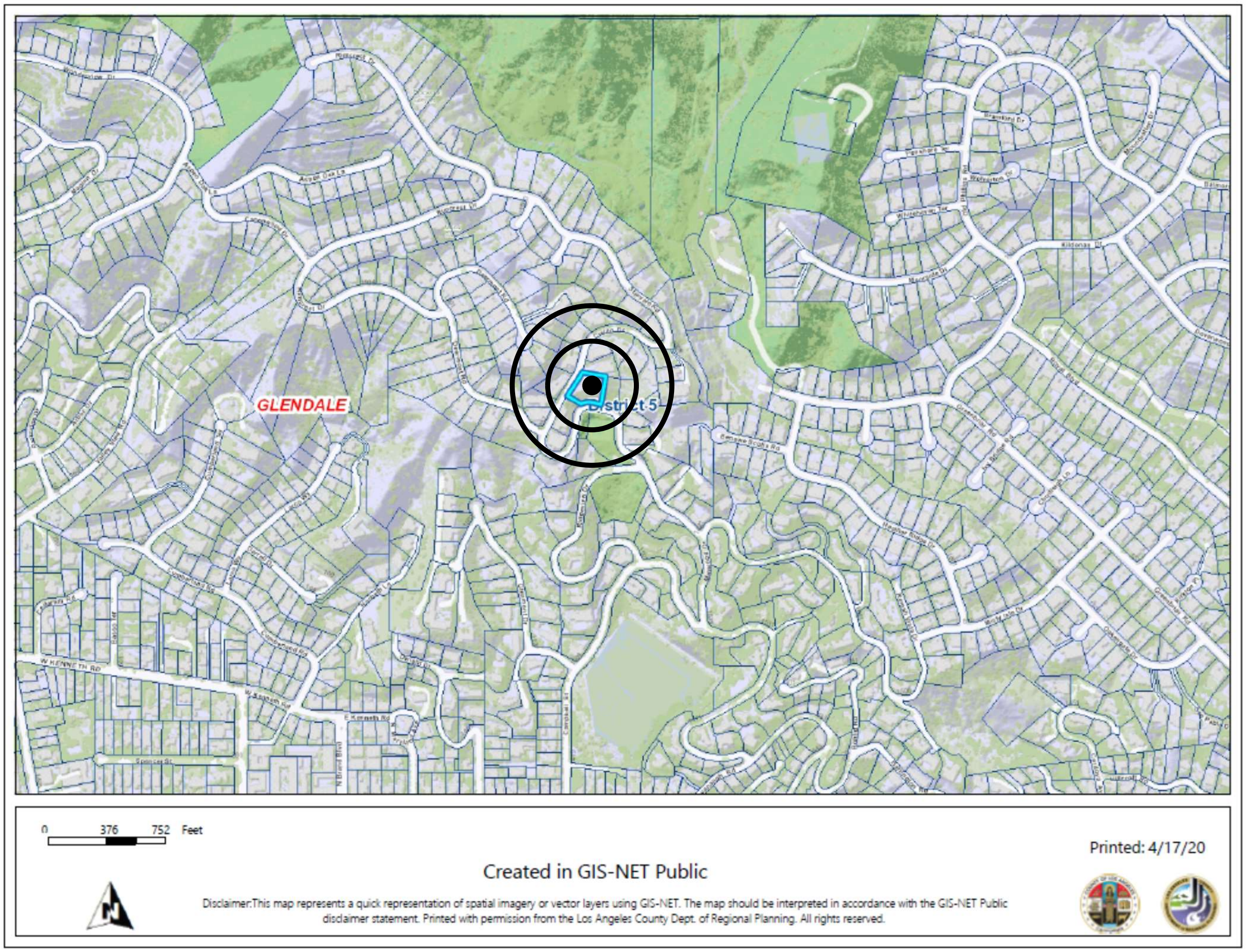
SITE INFORMATION

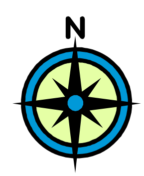
Location Map
Groundwater Map
Regional Geologic Map
Seismic Hazard Map

Geologic Map
Cross Sections

Field Exploration
Exploration Logs TP-1 through TP-9

LOCATION MAP



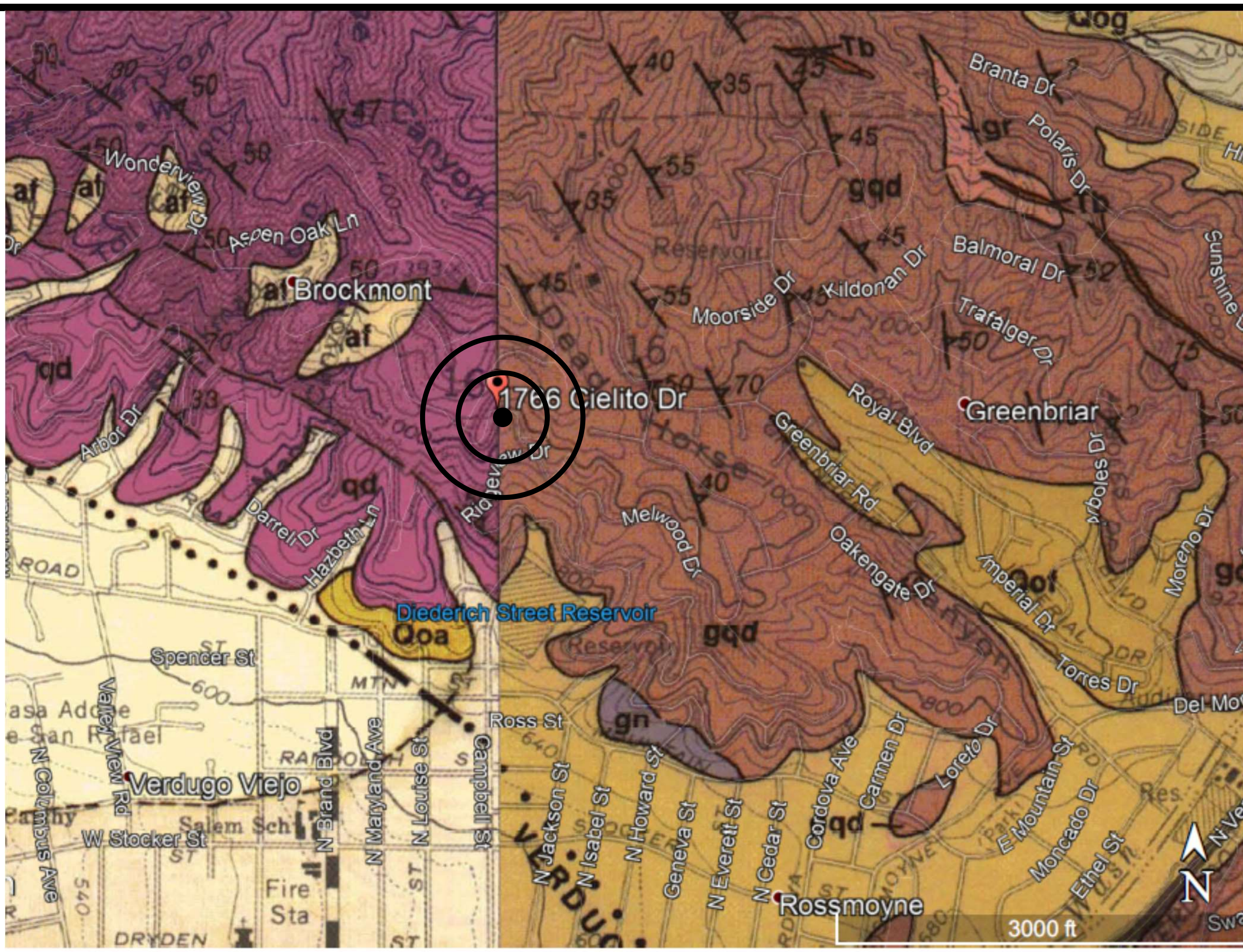


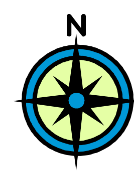
Reference:	GIS-NET	Scale: As Shown

GROUNDWATER MAP

Reference:	State of California Seismic Hazard Report, Burbank and Pasadena Quadrangle	Scale: As Shown

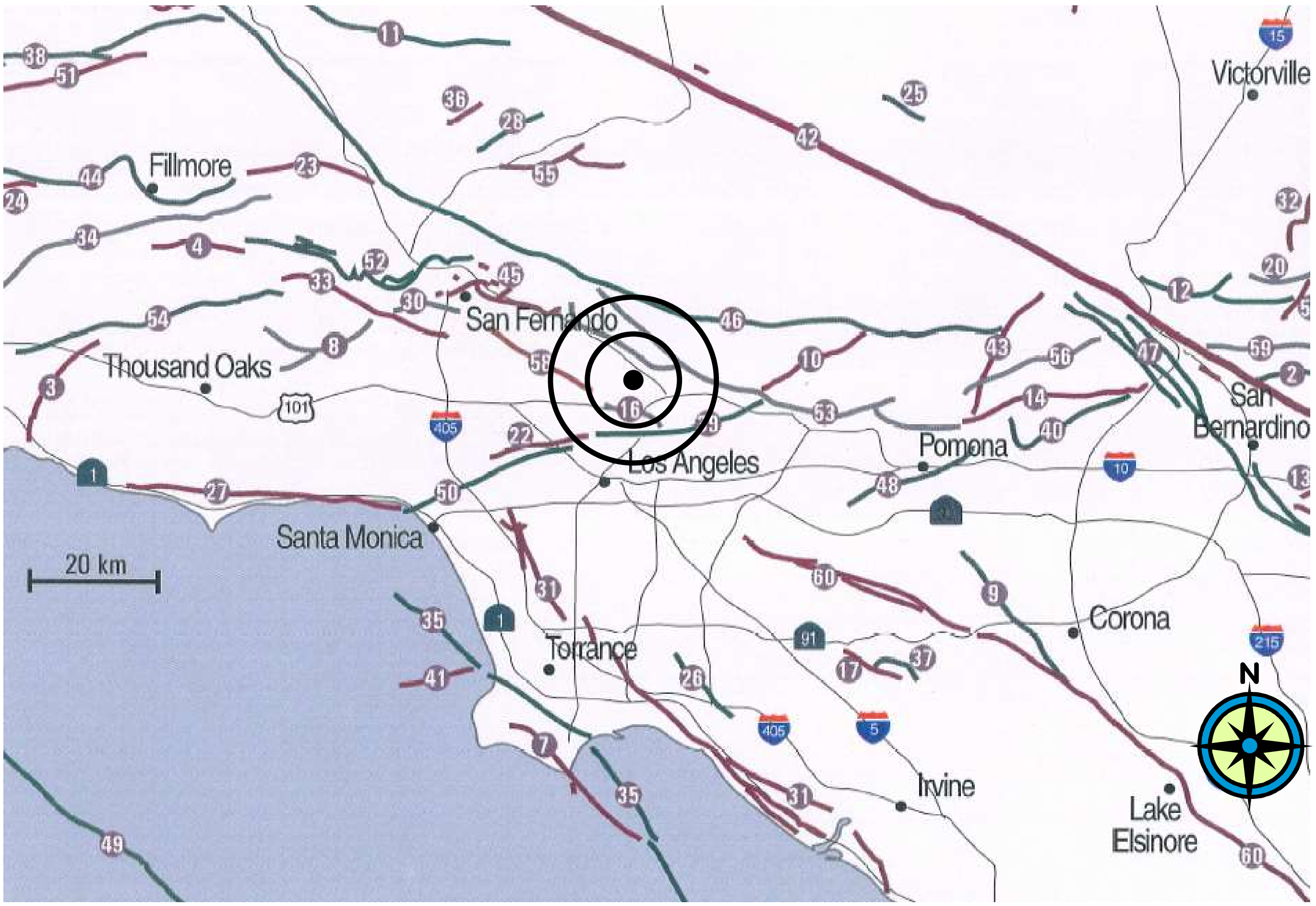
REGIONAL GEOLOGIC MAP





Reference:	Dibblee Geologic Foundation, Burbank & Pasadena Quadrangle	Scale: As Shown

USGS FAULT MAP

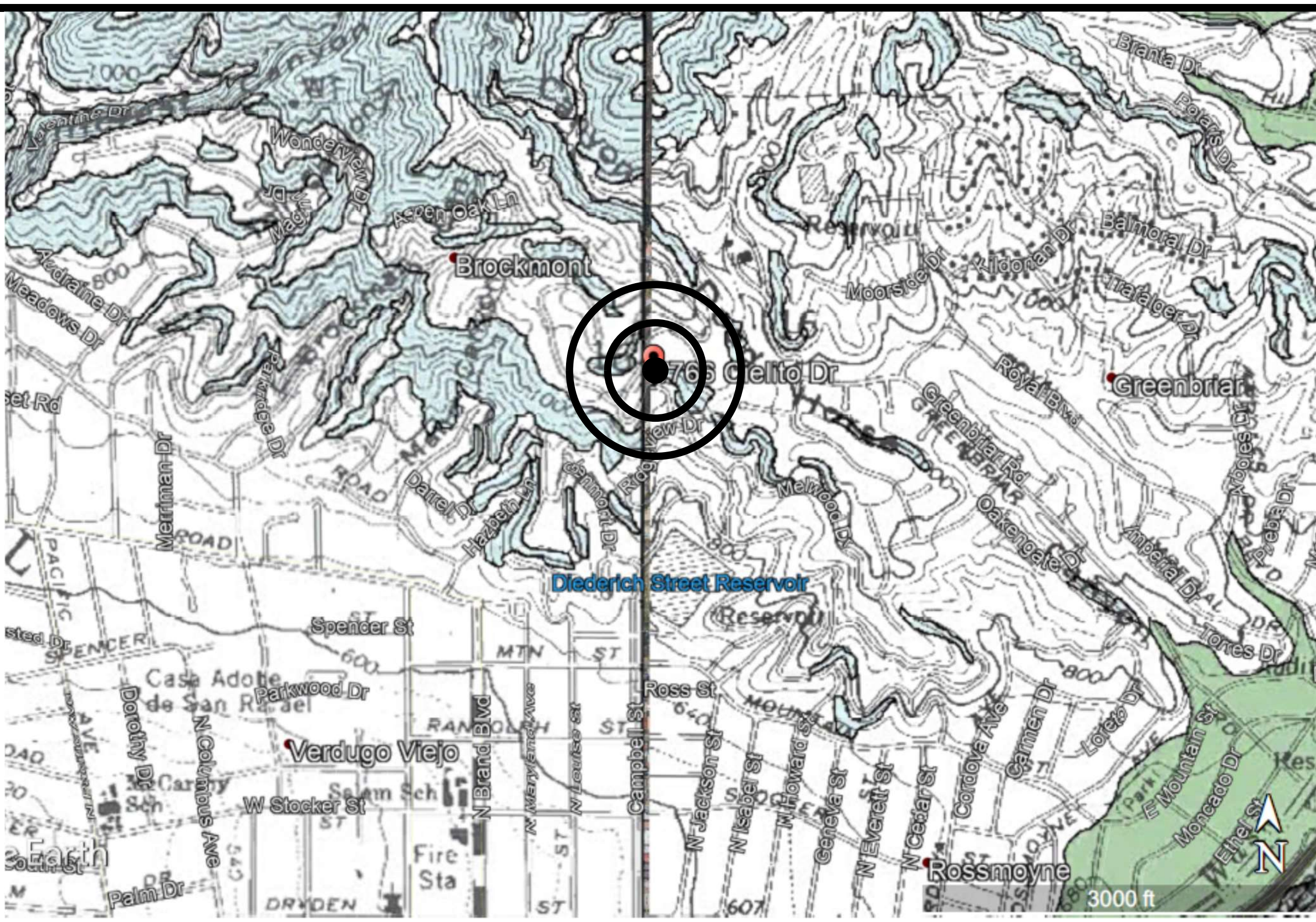


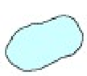
Lisa Wald, U.S. Geologic Survey (modified from SCEC).

1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek
16 Eagle Rock fault
17 El Modeno
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

Reference:	U. S. G. S: active fault (red) and potentially active fault (green)

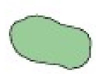
SEISMIC HAZARD MAP





Earthquake-Induced Landslides

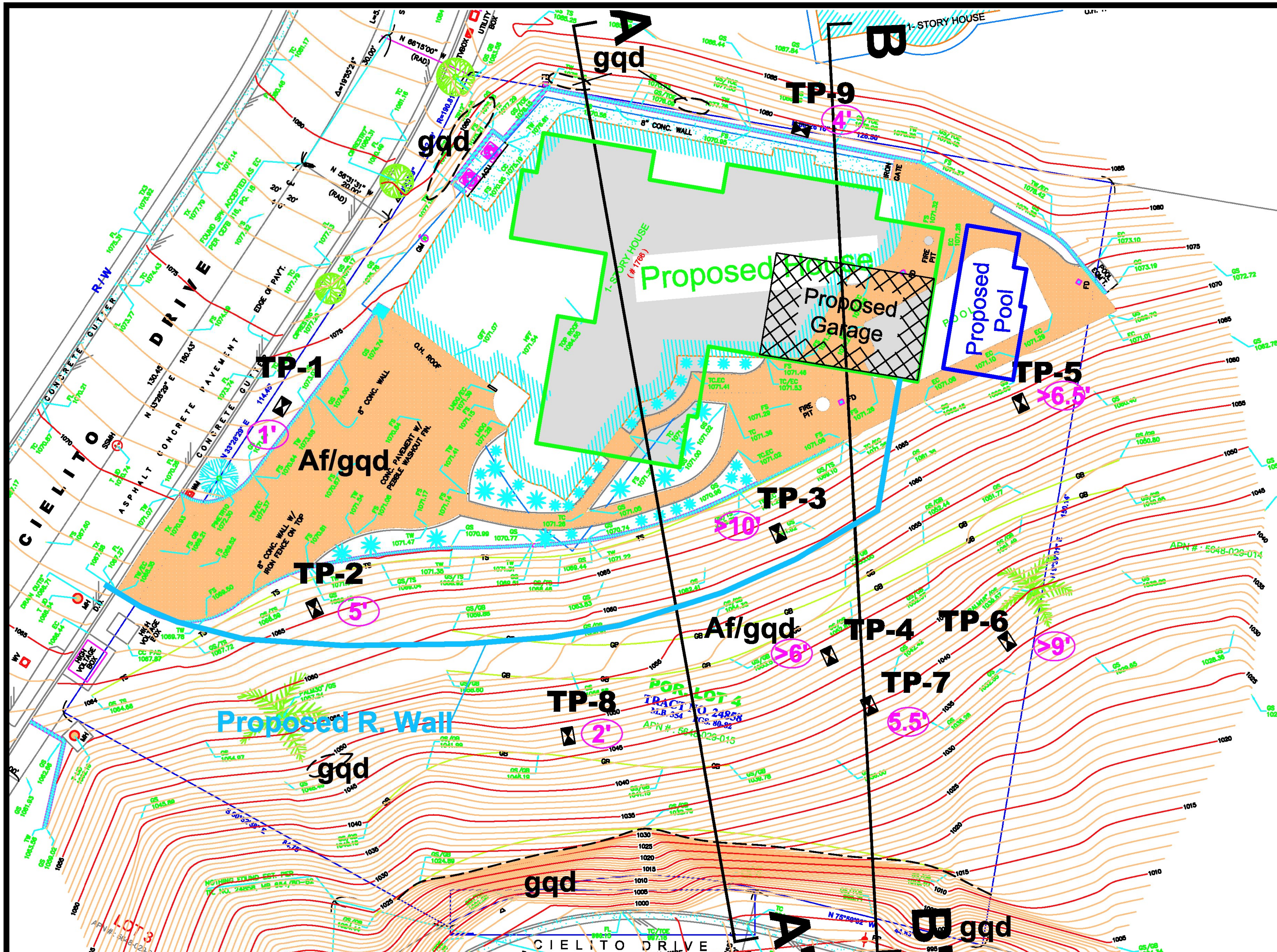
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Reference:	State of California, Seismic Hazard Map of the Burbank & Pasadena Quadrangle	Scale: As Shown



Explanation	
Af	Fill
gqd	Granitic Bedrock
TP-9	Location of Test Pits
10'	Depth of Bedrock (ft)
A — A'	Line of Cross Section
	Contact

CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

Description: **Geologic Map**

Site Survey Prepared By: Ray Lombera & Associates, Inc.

Project Address: 1766 Cielito Dr. Glendale, California

Date: Jul. 2021
Scale: 1" = 20'
Job No. 200416-2

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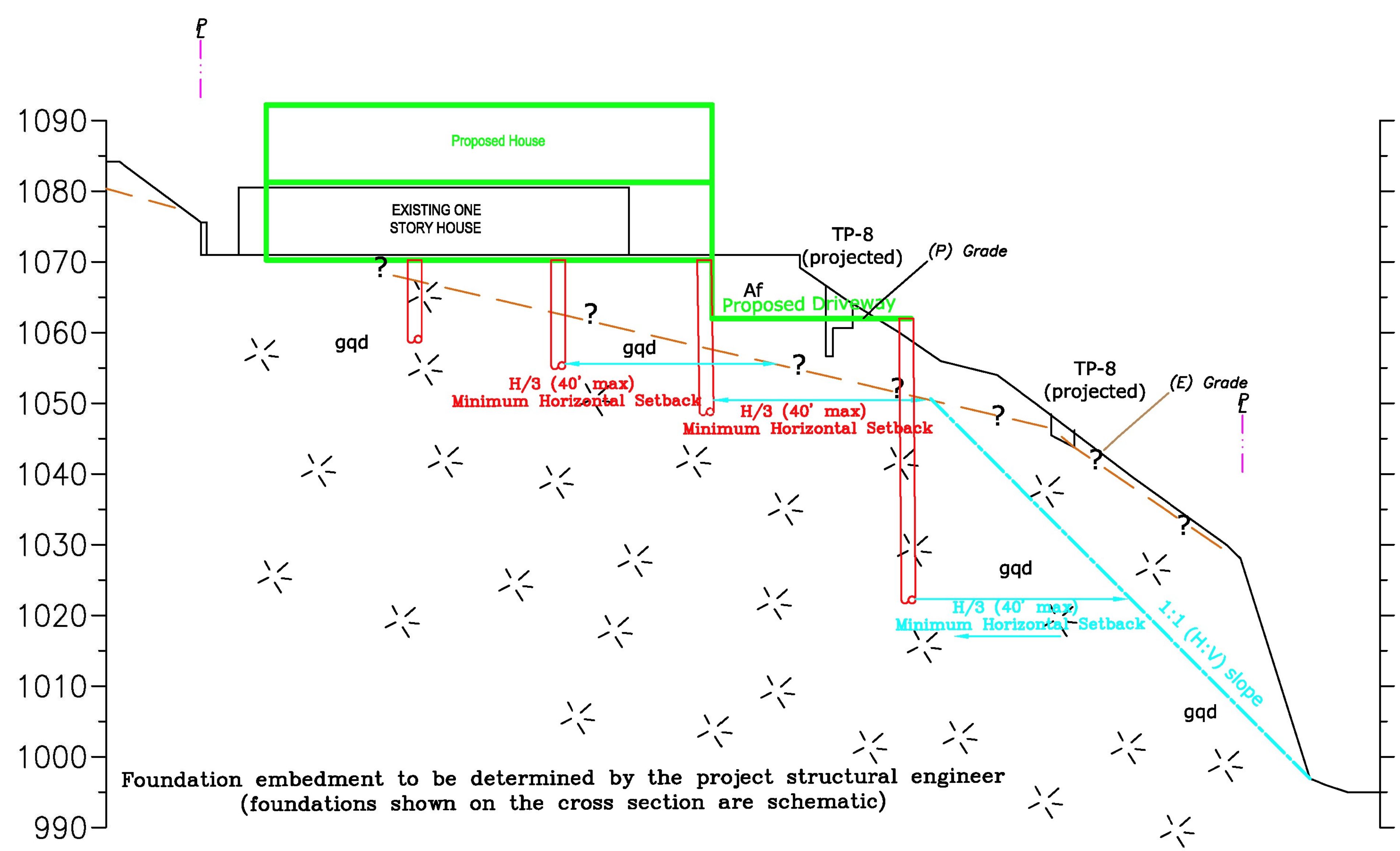

CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

Description:	Project Address:	Date: Jul. 2021
Geologic Cross Section A - A'	1766 Cielito Dr. Glendale, California	Scale: 1" = 20'
Site Survey Prepared By: Ray Lombera & Associates, Inc.		Job No. 200416-2

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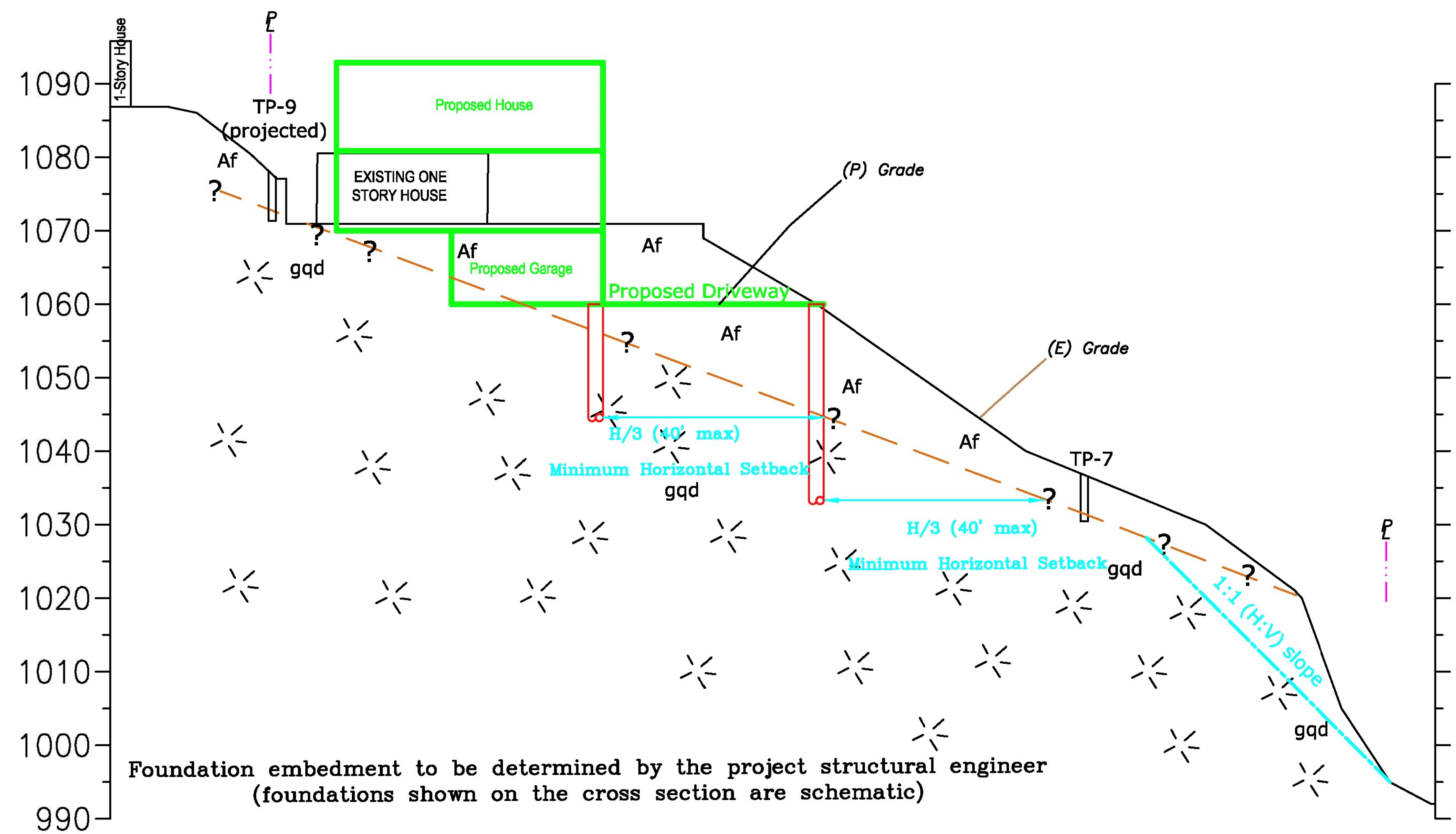

CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

Description:
Geologic Cross Section A - A'

Site Survey Prepared By:
Ray Lombera & Associates, Inc.

Project Address:
1766 Cielito Dr.
Glendale, California

Date: Jul. 2021
Scale: 1" = 20'
Job No. 200416-2

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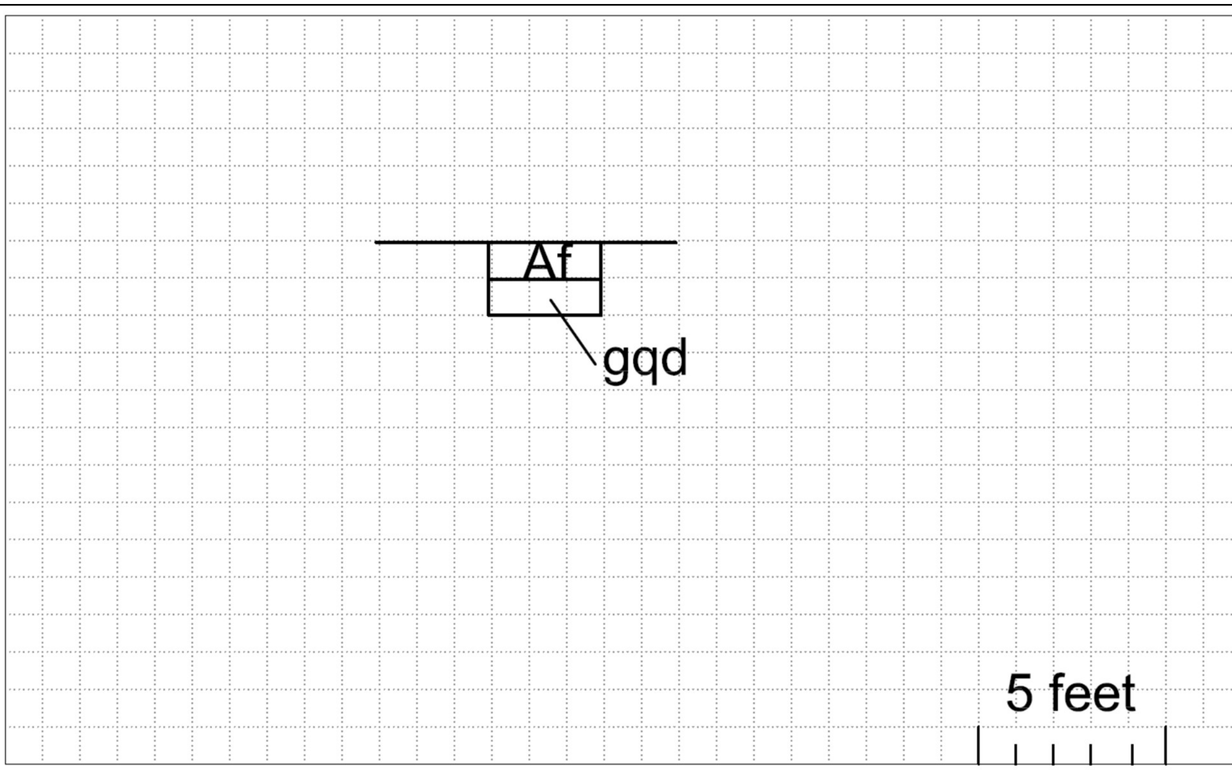
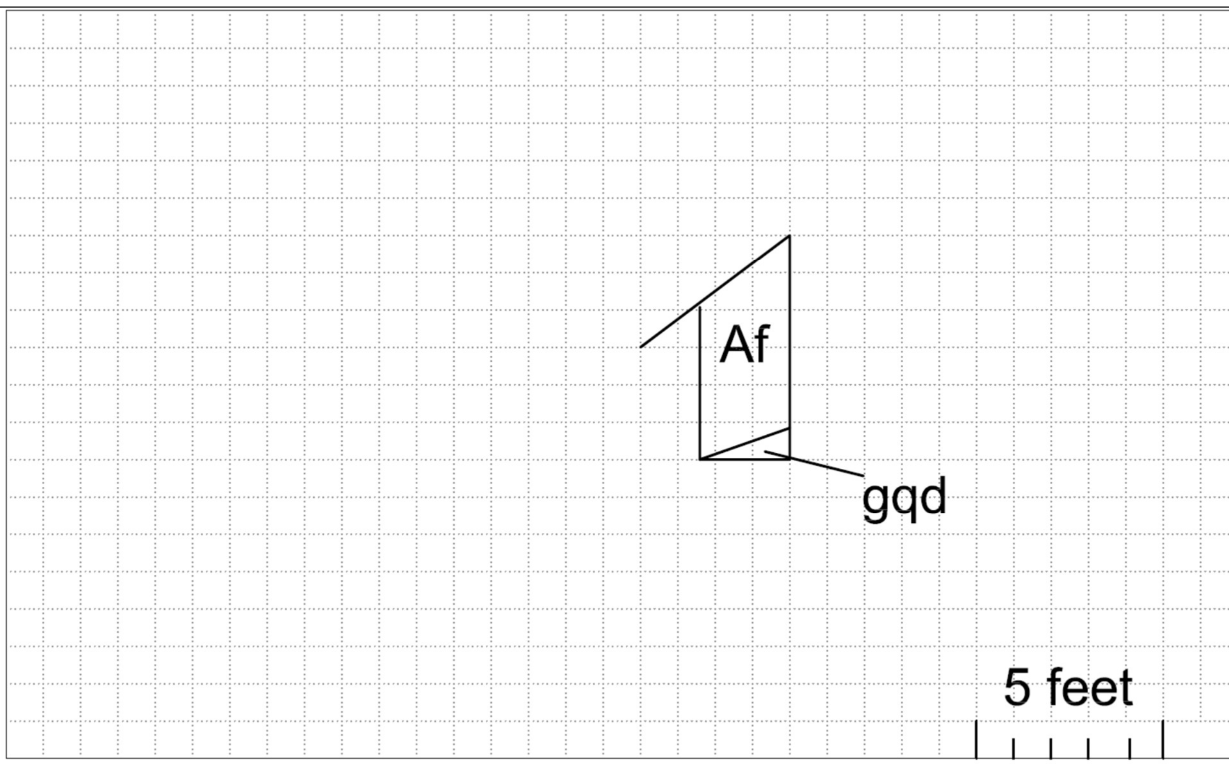
CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-1**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 1.0' ARTIFICIAL FILL (Af), silty sand, dark yellowish brown, loose **1.0' – 2.0' BEDROCK (gqd)**, granite, dark gray, slightly weathered, very dense		
TOTAL DEPTH: 2.0 ft. NO GROUNDWATER NO CAVING		

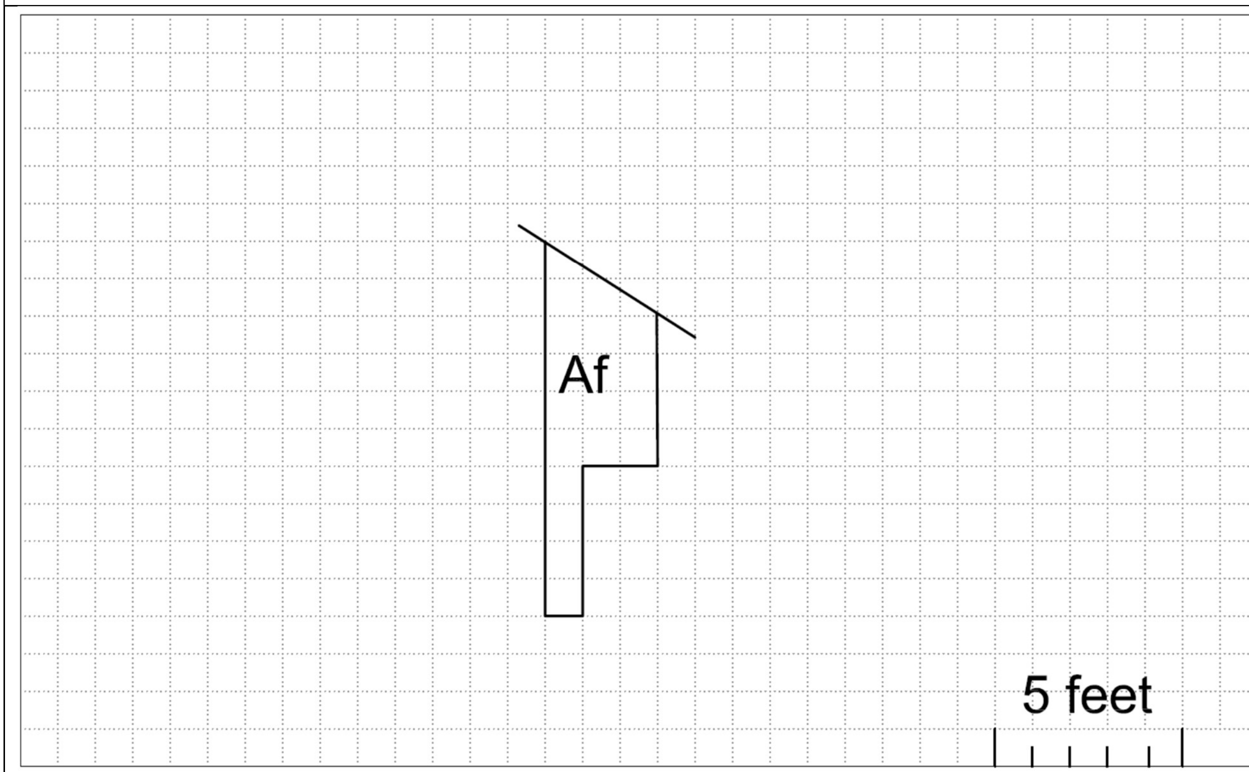


CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-2**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 5.0' ARTIFICIAL FILL (Af), silty sand with gravels and cobbles, dark yellowish brown, moderately dense, abundant roots, moist **5.0' – 6.0' BEDROCK (gqd)**, granite, white to dark gray, slightly weathered and fractured, very dense		
TOTAL DEPTH: 6.0 ft. NO GROUNDWATER NO CAVING		

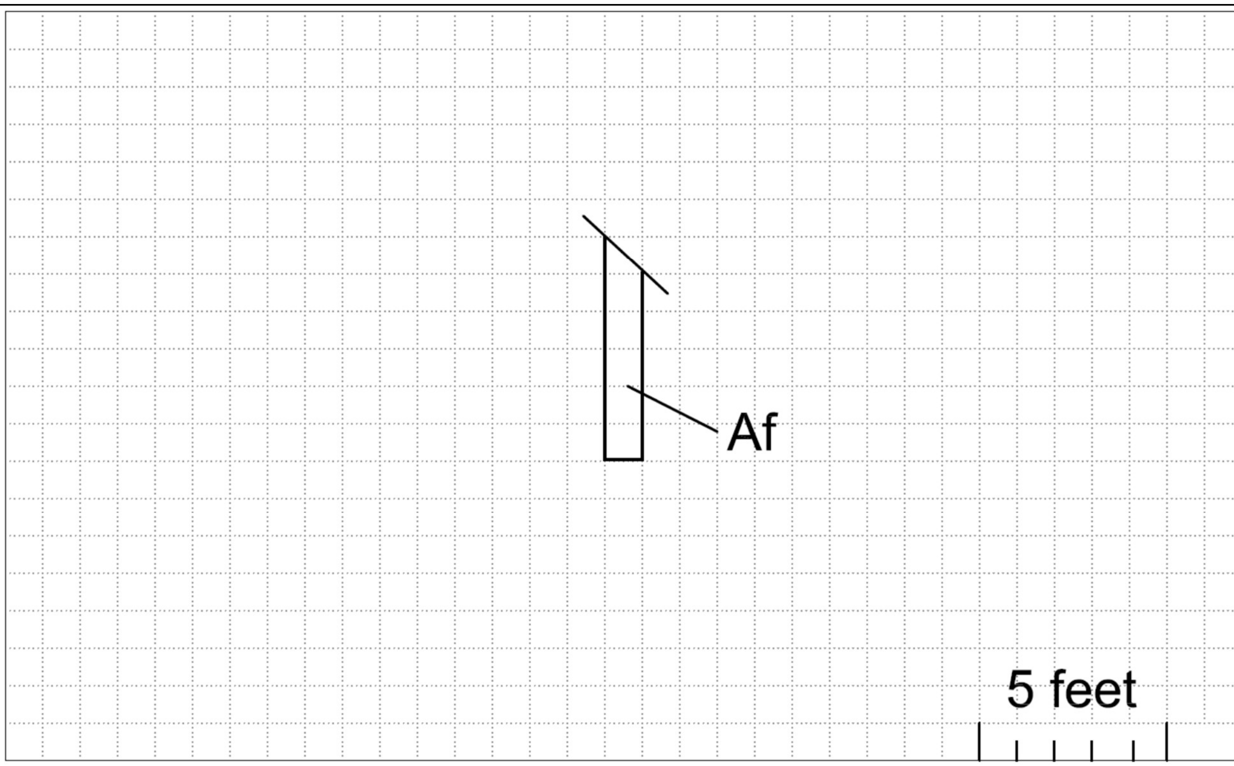


<table>
<tr><td>CreativeGeotechnical, Inc.
Project #: 200416
Address: 1766 Cielito Drive</td><td>Date of Excavation: 04.18.2020
Logger: PRK</td><td>Test Pit
TP-3</td></tr>
<tr><td colspan="2">DESCRIPTIONS</td><td>Attitudes
Bedding: (b) Joint: (j)
Shear: (s) Fault: (f)</td></tr>
<tr><td colspan="2">0 – 10.0’ ARTIFICIAL FILL (Af), silty sand with gravels and cobbles, dark yellowish brown, moderately dense, abundant roots, moist

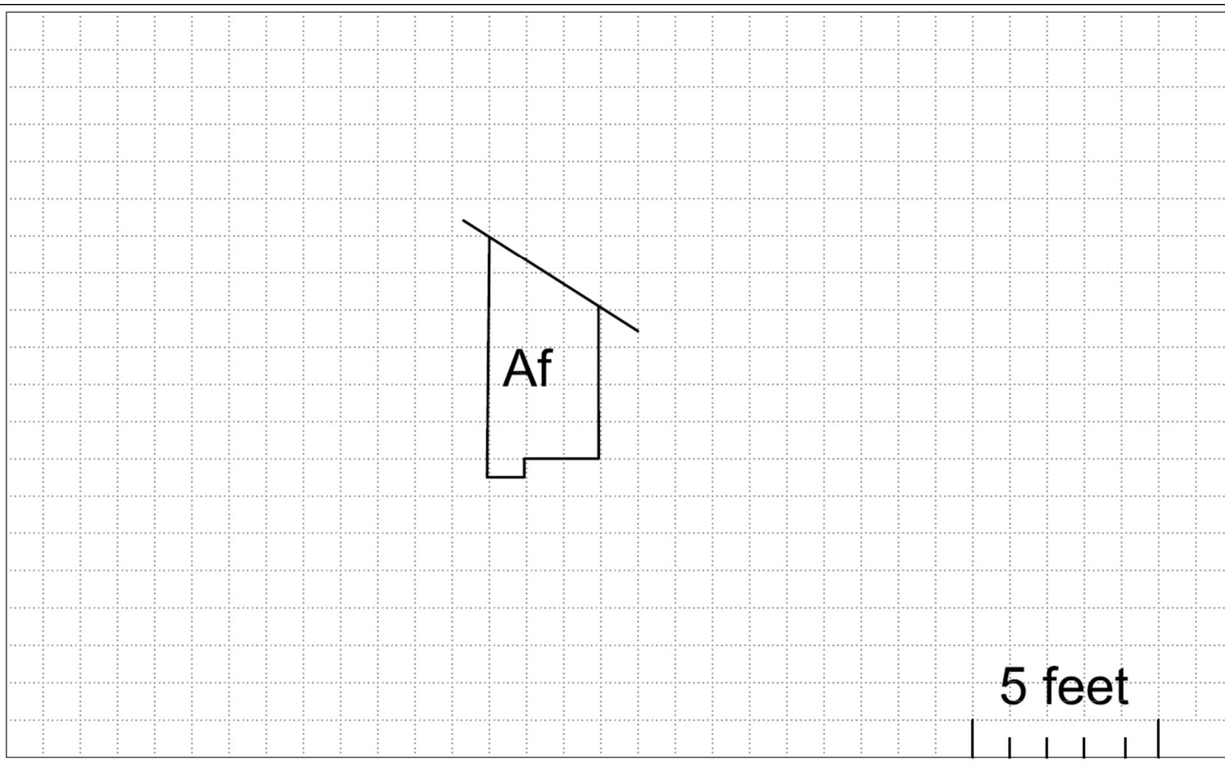
@10’ Refusal due to gravels</td><td></td></tr>
<tr><td colspan="3">TOTAL DEPTH: 10.0 ft. NO GROUNDWATER NO CAVING</td></tr>
</table>



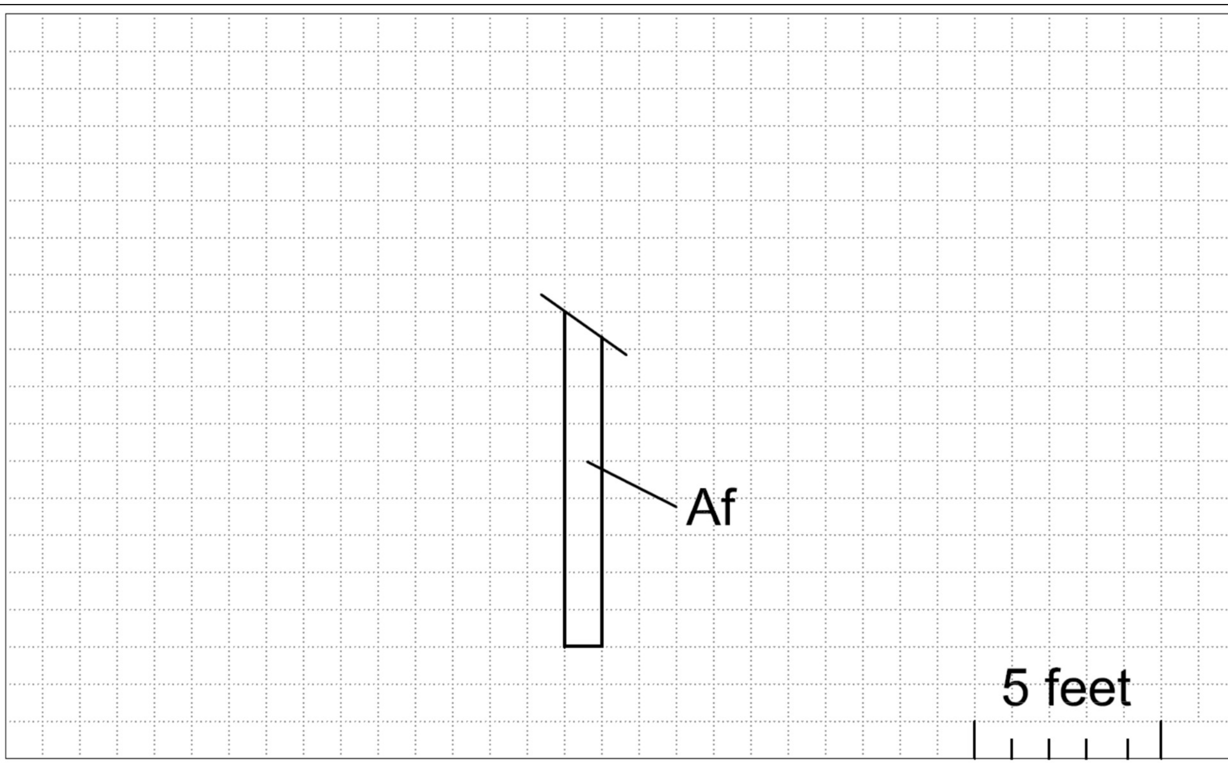
CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-4**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 6.0’ ARTIFICIAL FILL (Af), silty sand with gravels and cobbles, dark yellowish brown, moderately dense, abundant roots, moist @6’ Refusal due to gravels		
TOTAL DEPTH: 6.0 ft. NO GROUNDWATER NO CAVING		



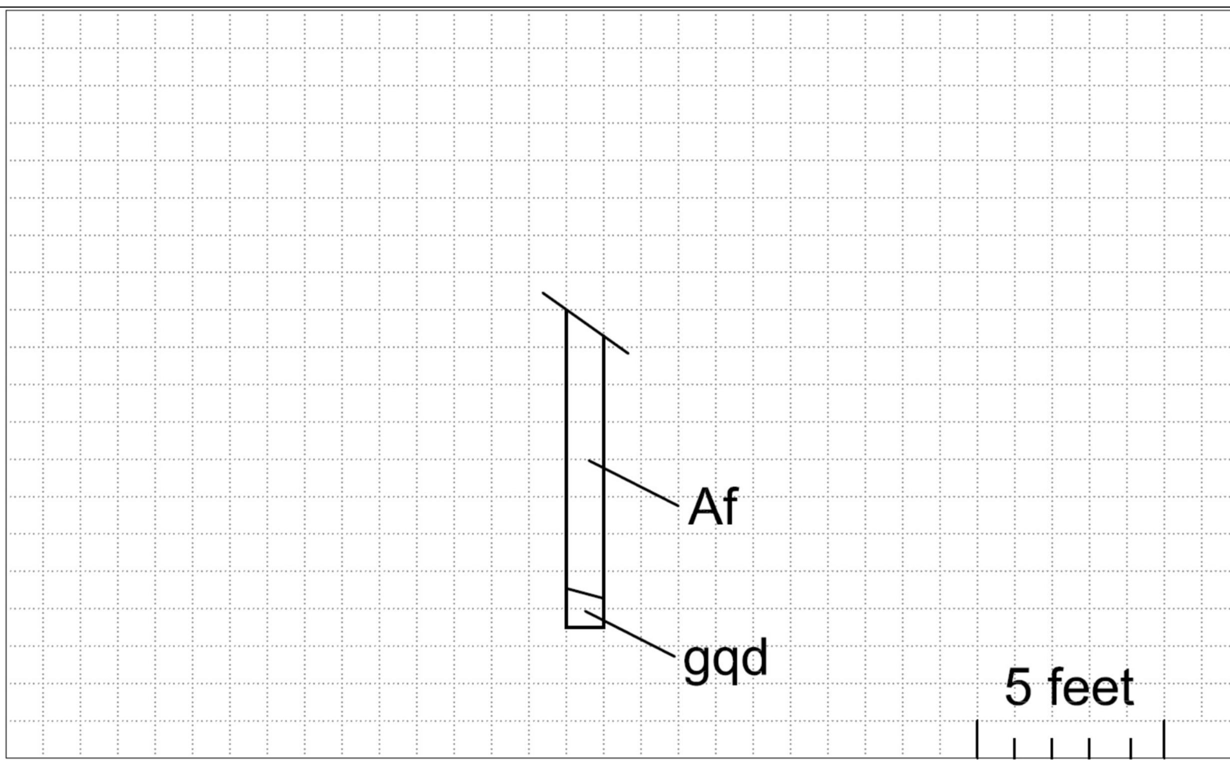
CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-5**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 6.5’ ARTIFICIAL FILL (Af), silty sand with gravels, dark yellowish brown, moderately dense, abundant roots, moist @6.5’ Refusal due to gravels		
TOTAL DEPTH: 6.5 ft. NO GROUNDWATER NO CAVING		



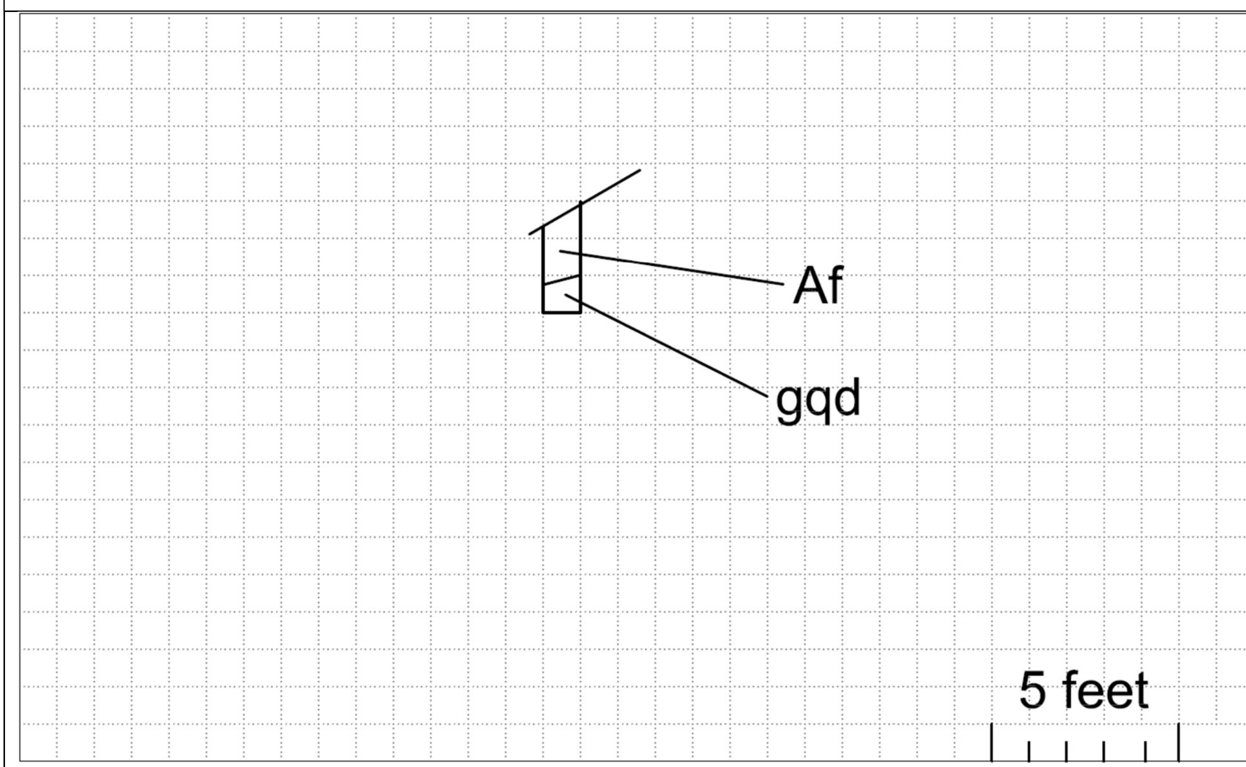
CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-6**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 9.0' ARTIFICIAL FILL (Af), silty sand with gravels, dark yellowish brown, moderately dense, abundant roots, moist @ 9' Refusal due to gravels		
TOTAL DEPTH: 9.0 ft. NO GROUNDWATER NO CAVING		



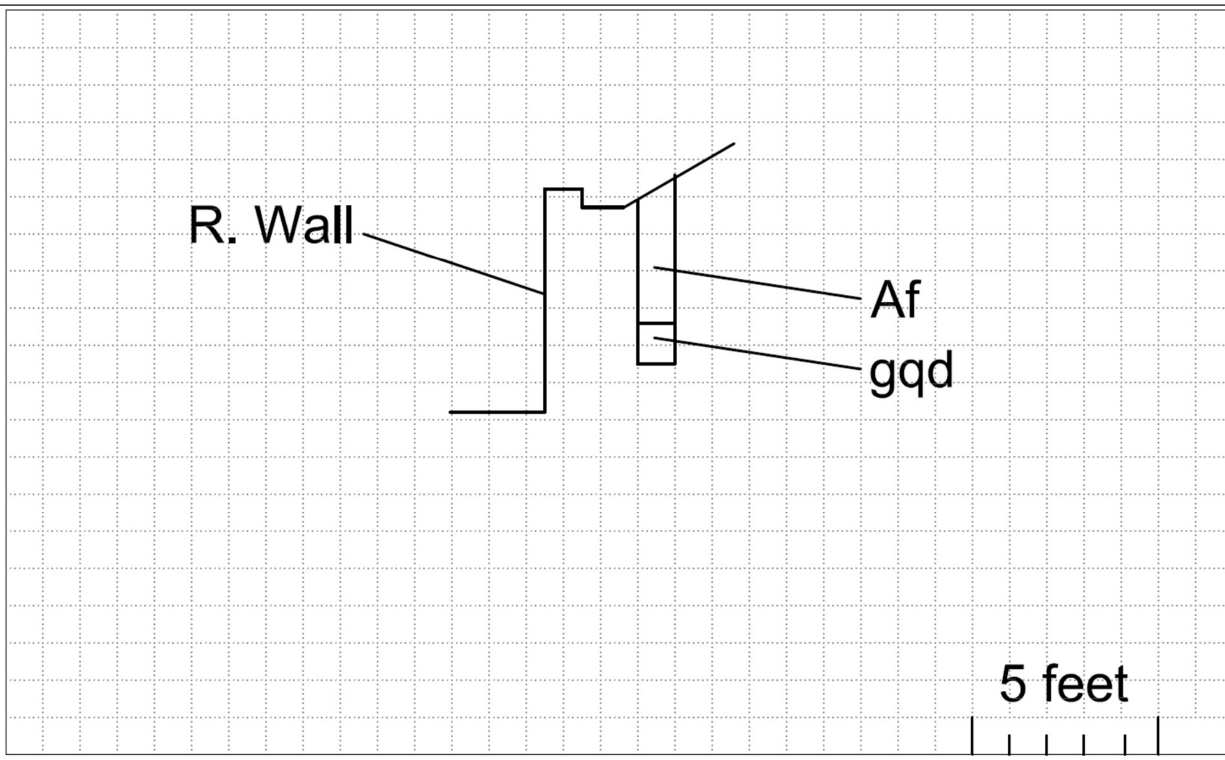
CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-7**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 5.5' ARTIFICIAL FILL (Af), silty sand with gravels and cobbles, dark yellowish brown, moderately dense, abundant roots, moist **5.5' – 6.5' BEDROCK (gqd)**, granite, white to dark gray, moderately to slightly weathered and fractured, very dense		
TOTAL DEPTH: 6.5 ft. NO GROUNDWATER NO CAVING		



CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-8**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 2.0' ARTIFICIAL FILL (Af), silty sand, dark yellowish brown, loose **2.0' – 3.0' BEDROCK (gqd)**, granite, white to dark gray, slightly weathered, very dense		
TOTAL DEPTH: 3.0 ft. NO GROUNDWATER NO CAVING		



CreativeGeotechnical, Inc. Project #: 200416 Address: 1766 Cielito Drive	Date of Excavation: 04.18.2020 Logger: PRK	**Test Pit TP-9**
DESCRIPTIONS		Attitudes Bedding: (b) Joint: (j) Shear: (s) Fault: (f)
0 – 4.0' ARTIFICIAL FILL (Af), silty sand with gravels, dark yellowish brown, loose, slightly moist **4.0' – 5.0' BEDROCK (gqd)**, granite, white to dark gray, moderately to slightly weathered, very dense		
TOTAL DEPTH: 5.0 ft. NO GROUNDWATER NO CAVING		



APPENDIX II

LABORATORY TESTING

Laboratory testing was performed on samples obtained as outlined in the Field Exploration section of this report. All samples were sent to the laboratory for examination, testing in general conformance to specified test methods, and classification, using the Unified Soil Classification System and group symbol.

Moisture and Density Tests

The dry unit weight and moisture content of the undisturbed samples were determined. The results are tabulated in the Laboratory Recapitulation - Table 1.

Shear Tests

Direct single-shear tests were performed with a direct shear machine. The desired normal load is applied to the specimen and allowed to come to equilibrium. The rate of deflection on the sample is approximately 0.005 inches per minute. The samples are tested at higher and/or lower normal loads in order to determine the angle of internal friction and the cohesion. The results are plotted on the Shear Test Diagrams and the results tabulated in the Laboratory Recapitulation - Table 1.

pH (CTM 643)

A sample of dry soil and distilled water are placed in a flask and allowed to stand for approximately an hour to stabilize. The pH is measured using a pH meter that has been compensated for temperature. The results are tabulated in the Laboratory Recapitulation - Table 2.

Minimum Resistivity (CTM 643)

The electrical resistivity of each soil specimen is conducted in a two-stage process using the soil box method. The first stage measures the resistivity of the soil in its as-received condition and the second stage records the value after saturation with distilled water. The results are tabulated in the Laboratory Recapitulation - Table 2.

Chloride Content (CTM 422)

A sample of dry soil is mixed with distilled water and allowed to stand overnight. The top aliquot of the sample is mixed with chloride indicator and titrated over silver nitrate solution. The chloride content is determined by the difference of the volumes required to complete titration. The results are tabulated in the Laboratory Recapitulation - Table 2.

Sulfate Content (CTM 417)

A sample of dry soil is mixed with distilled water and allowed to stand overnight. The top aliquot is mixed with distilled water and a conditioning agent. The solution is then placed in a photometer and the value recorded. The process is repeated with the addition of barium chloride. The sulfate content is determined by the difference of the photometer readings. The results are tabulated in the Laboratory Recapitulation - Table 2.

PROJECT NO.: 200416
PROJECT
ADDRESS: 1766 Cielito Drive

LABORATORY RECAPITULATION 1

Explorations	Depth (ft)	Material	Dry Density (p.c.f.)	Moisture Content (%)
Outcrop	0.0	gqd	120	4
TP-1	2.0	gqd	121	3
TP-2	6.0	gqd	124	5
TP-3	6.0	Af	119	8
TP-5	4.0	Af	125	5
TP-7	3.0	Af	122	5

LABORATORY RECAPITULATION 2

Explorations	Depth (ft)	pH	As-Is Soil Resistivity (ohm-cm)	Minimum Soil Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
TP-2	6.0	6.70	60,000	5,600	18	22

LABORATORY RECAPITULATION 3

Exploration	Depth (ft)	Expansion Index
TP-3	6	0

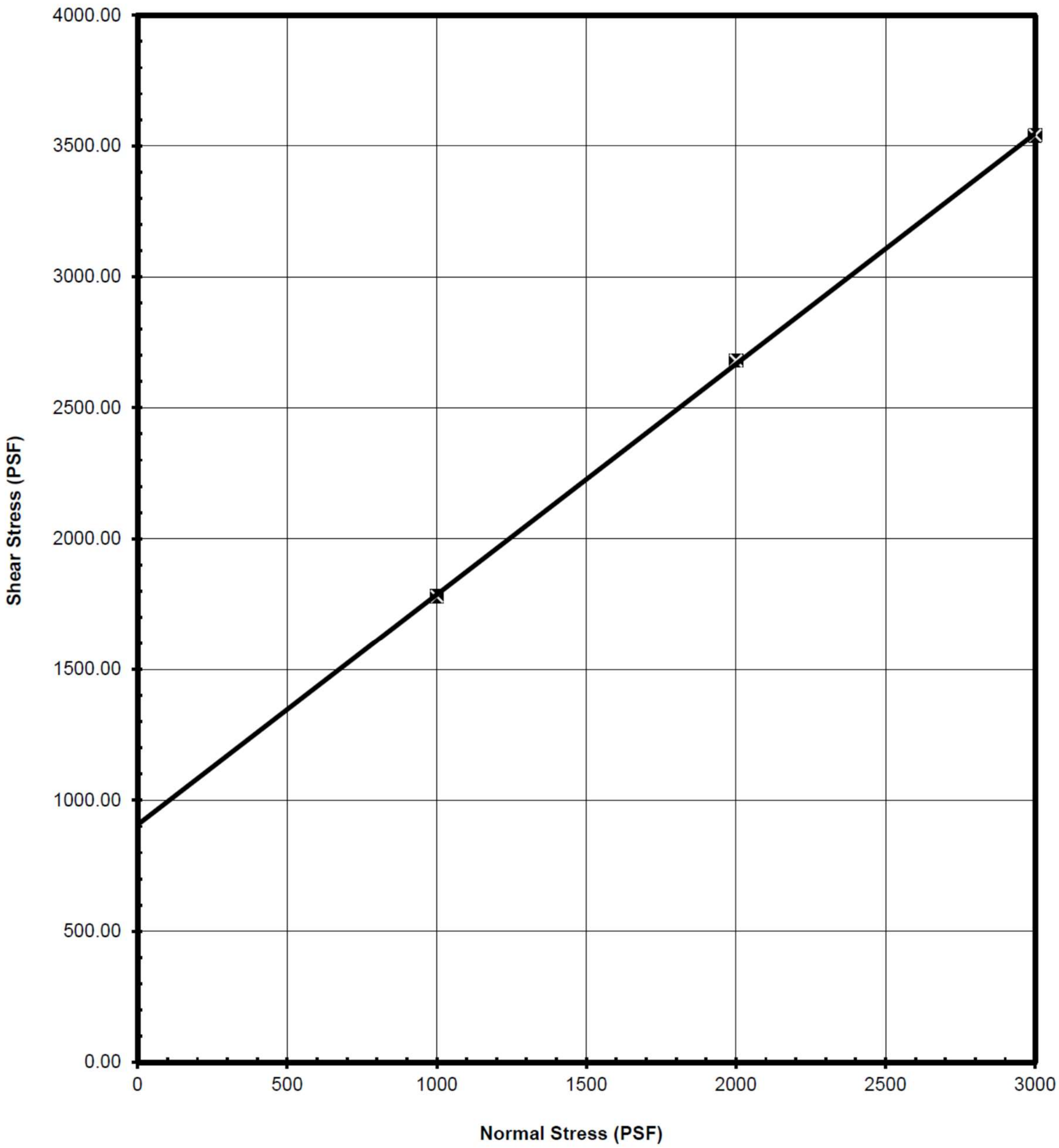
Direct Shear Test Diagram (D-3080)

PLATE: S-1

P.N. 200416

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
gqd	TP-1 @ 2.0'	Ultimate	Saturated	1

Soil Dry Density (PCF)	121	**Shear Strength Values:**	
Soil Moisture Content (%)	13.5	**Phi (Degrees)**	**41.3**
Soil Saturation (%)	97.6	**Cohesion (PSF)**	**906.7**



Direct Shear Test Diagram (D-3080)

PLATE: S-2

P.N. 200416

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
gqd	TP-2 @ 6.0'	Ultimate	Saturated	1

Soil Dry Density (PCF)	124	Shear Strength Values:	
Soil Moisture Content (%)	12	Phi (Degrees)	42.8
Soil Saturation (%)	95.3	Cohesion (PSF)	916.7

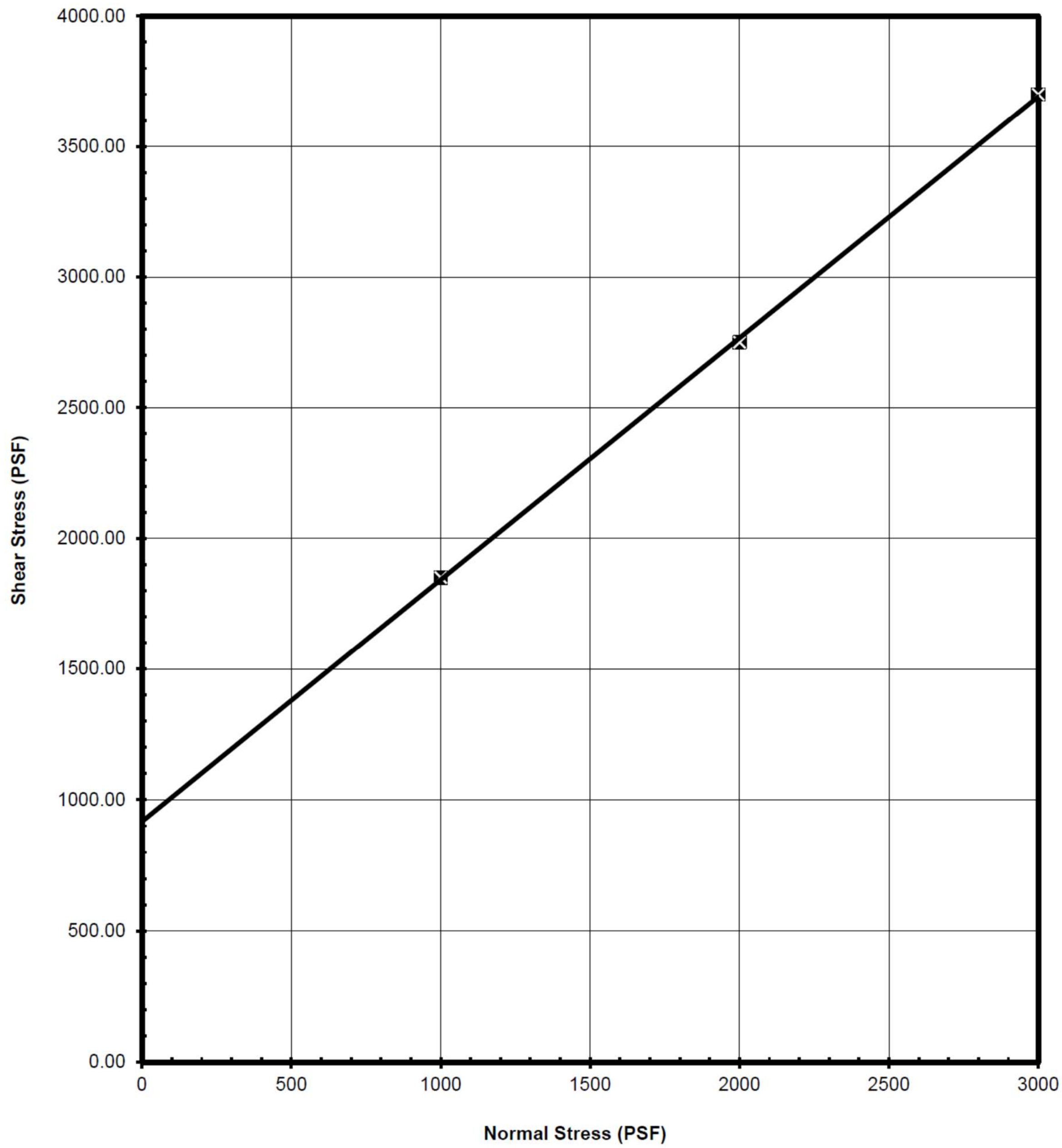


PLATE: S-3

P.N. 200416

Direct Shear Test Diagram (D-3080)

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Af	TP-3 @ 6.0'	Ultimate	Saturated	1

Soil Dry Density (PCF)	119	Shear Strength Values:	
Soil Moisture Content (%)	14.3	Phi (Degrees)	34.8
Soil Saturation (%)	97.3	Cohesion (PSF)	313.3

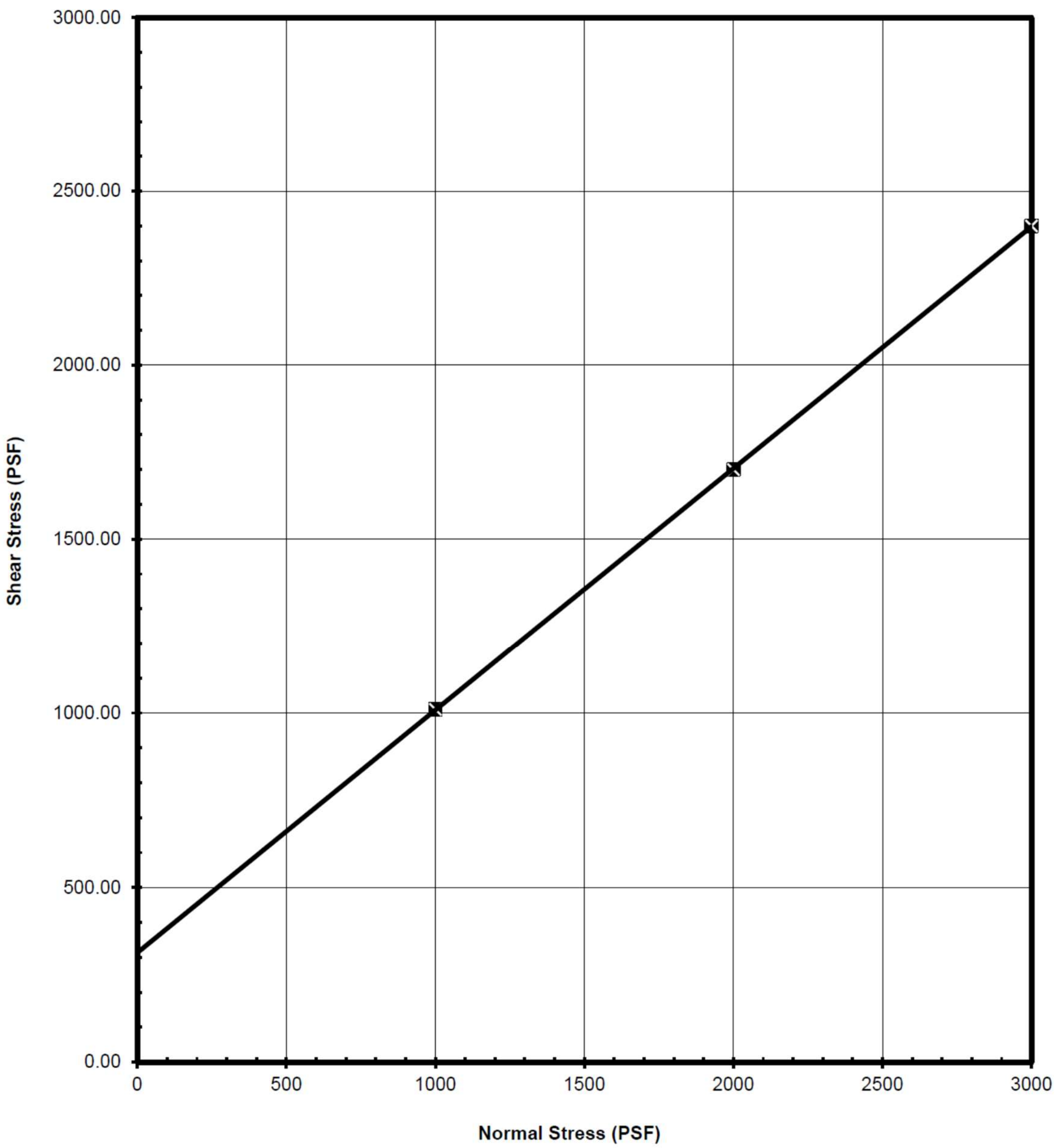
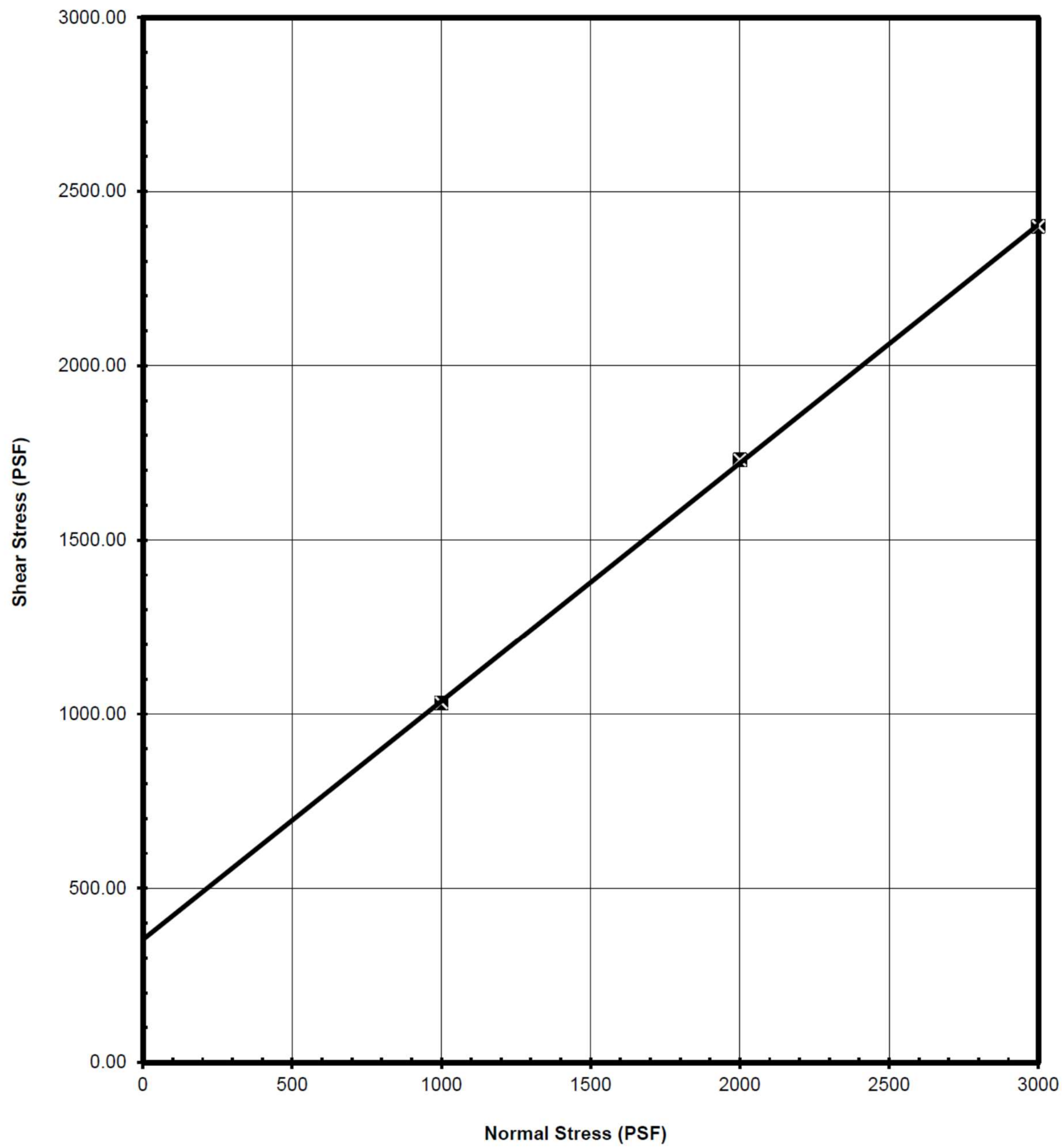


PLATE: S-4
P.N. 200416

Direct Shear Test Diagram (D-3080)

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Af	TP-7 @ 3.0'	Ultimate	Saturated	1

Soil Dry Density (PCF)	122	Shear Strength Values:	
Soil Moisture Content (%)	13	Phi (Degrees)	**34.4**
Soil Saturation (%)	96.9	Cohesion (PSF)	**352.7**



APPENDIX III

ANALYSES

Bearing Capacity

Lateral Design

Slope Stability

Seismic Evaluation

BEARING CAPACITY ANALYSIS

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	gqd	EMBEDMENT DEPTH:	2 feet
SHEAR DIAGRAM:	0	FOOTING LENGTH:	50 feet
COHESION:	906 psf	FOOTING WIDTH:	2 feet
PHI ANGLE:	41.3 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	130 pcf	FOOTING INCLINATION:	0 degrees
SAFETY FACTOR:	4		
FOOTING TYPE:	S Strip		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	77.14	Dq =	1.20	Sy =	0.98
Nc =	86.67	Gc =	1.00	Dy =	1.00
Ny =	100.33	Bc =	1.00	Iy =	1.00
Sc =	1.04	Iq =	1.00	Gy =	1.00
Sq =	1.04	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

CALCULATED ULTIMATE BEARING CAPACITY (Qult)	151,652.0 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	37,913.0 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	7.8%

BEARING CAPACITY ANALYSIS

CALCULATE THE ULTIMATE AND ALLOWABLE BEARING CAPACITIES OF THE BEARING MATERIAL LISTED BELOW USING HANSEN'S METHOD. (REFERENCE: J. BOWLES, *FOUNDATION ANALYSIS AND DESIGN*, 1988, p. 188-194).

CALCULATION PARAMETERS

EARTH MATERIAL:	gqd	EMBEDMENT DEPTH:	2 feet
SHEAR DIAGRAM:	0	PAD LENGTH:	2 feet
COHESION:	906 psf	PAD WIDTH:	2 feet
PHI ANGLE:	41.3 degrees	SLOPE ANGLE:	0 degrees
DENSITY:	130 pcf	PAD INCLINATION:	0 degrees
SAFETY FACTOR:	4		
FOOTING TYPE:	p Pad		

CALCULATED RESULTS

HANSEN'S SHAPE, DEPTH, AND INCLINATION FACTORS

Nq =	77.14	Dq =	1.20	Sy =	0.60
Nc =	86.67	Gc =	1.00	Dy =	1.00
Ny =	100.33	Bc =	1.00	Iy =	1.00
Sc =	1.89	Iq =	1.00	Gy =	1.00
Sq =	1.88	Ic =	1.00	Gq =	1.00
Dc =	1.40	Bq =	1.00	By =	1.00

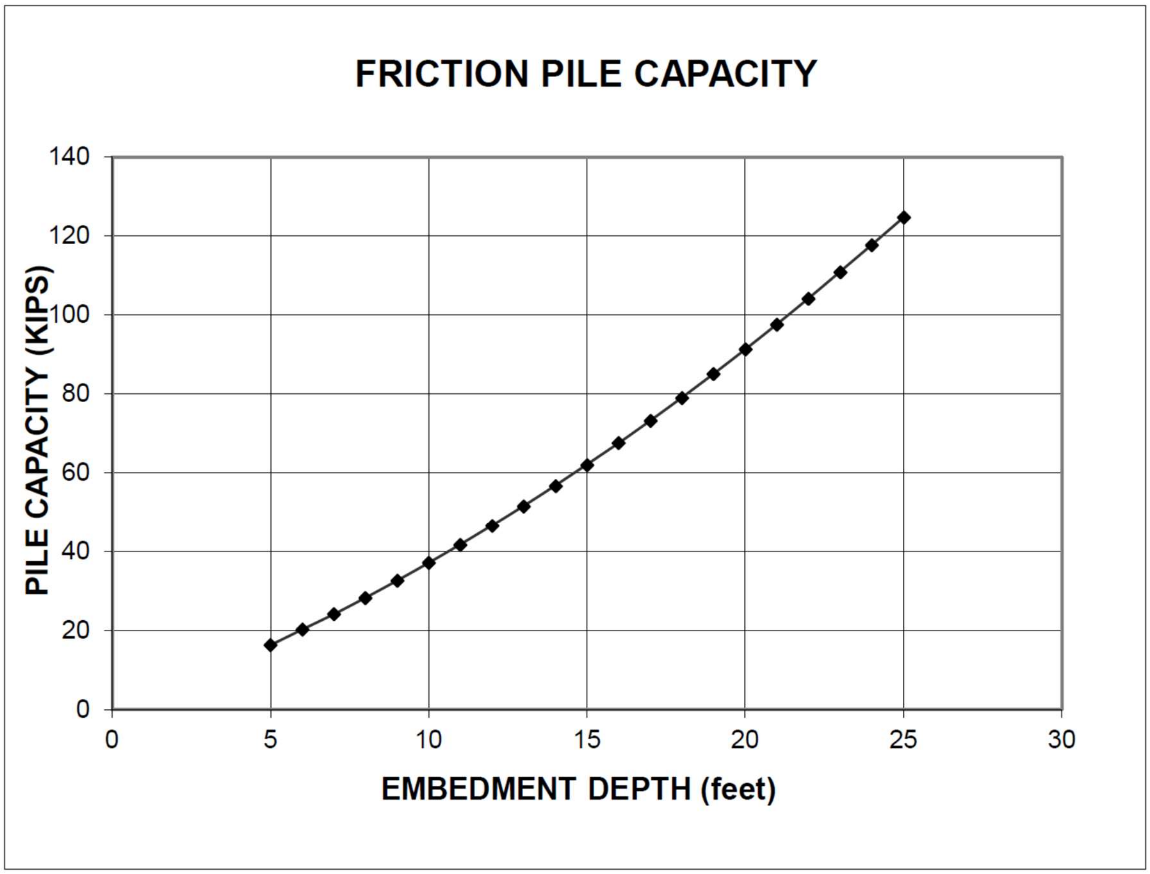
CALCULATED ULTIMATE BEARING CAPACITY (Qult)	260,921.2 pounds
ALLOWABLE BEARING CAPACITY (Qa = Qult / fs)	65,230.3 pounds
PERCENT INCREASE FOR EMBEDMENT DEPTH	8.2%

PILE CAPACITY

CALCULATE ALLOWABLE SKIN FRICTION RESISTANCE FOR DRILLED, CAST IN PLACE CONCRETE PILES. SKIN FRICTION IS TABULATED AS A FUNCTION OF EMBEDMENT DEPTH. (REFERENCES: NAVFAC DM-7.2, PAGES 193-193 AND J.E. BOWLES, "*FOUNDATION ANALYSIS AND DESIGN*," 1988.)

CALCULATION PARAMETERS

EARTH MATERIAL:	gqd	PILE DIAMETER:	2 feet
SHEAR DIAGRAM:	0	INITIAL PILE DEPTH:	5 feet
COHESION:	906 psf	FINAL PILE DEPTH:	25 feet
PHI ANGLE:	41.3 degrees	EXTERNAL SURCHARGE:	0 pounds
DENSITY:	130 pcf	ADHESION VALUE:	1.00
SAFETY FACTOR:	2	PILE/SOIL FRICTION:	31.0 degrees
COMPRESSION/TENSION:	C	LATERAL COEFF. (Ko)	0.70
PILE TYPE:	COMPRESSION PILE	NO GROUNDWATER	0.00 feet



CONCLUSIONS:

THE CALCULATED CAPACITY OF 24 INCH DIAMETER PILES, AS A FUNCTION OF EMBEDMENT, ARE SHOWN IN THE GRAPH.

PASSIVE EARTH PRESSURE

USE RANKINE'S METHOD TO CALCULATE THE PASSIVE EARTH PRESSURE. USE THE PROCEDURE IN NAVFAC DM-7, 1982, (p 7.2-21, Figure 2).

CALCULATION PARAMETERS

EARTH MATERIAL:	gqd	SAFETY FACTOR (fs):	1.5
SHEAR DIAGRAM:	0	INITIAL SEARCH DEPTH:	1
COHESION:	906 psf	FINAL SEARCH DEPTH:	21
PHI ANGLE:	41.3 degrees	LIMIT PASSIVE (Y OR N):	Y
DENSITY:	130 pcf	MAXIMUM PASSIVE:	100,000.0 pounds
		Cd (C/fs):	604.0 psf
		PhiD = atan(tan(phi)/fs) =	30.4 degrees

FOOTING DEPTH (feet)	TOTAL PASSIVE FORCE Pp (pounds)	PASSIVE EARTH PRESSURE AT DEPTH - SigmaP (psf)	INCREASE IN PASSIVE EARTH PRESSURE WITH EMBEDMENT DEPTH (psf/f)
1	2,305.3	2,503.1	2,503.1
2	5,006.2	2,898.8	395.7
3	8,102.8	3,294.4	395.7
4	11,595.0	3,690.1	395.7
5	15,482.9	4,085.7	395.7
6	19,766.5	4,481.4	395.7
7	24,445.7	4,877.0	395.7
8	29,520.6	5,272.7	395.7
9	34,991.1	5,668.4	395.7
10	40,857.3	6,064.0	395.7
11	47,119.1	6,459.7	395.7
12	53,776.6	6,855.3	395.7
13	60,829.8	7,251.0	395.7
14	68,278.6	7,646.7	395.7
15	76,123.1	8,042.3	395.7
16	84,363.2	8,438.0	395.7
17	92,999.0	8,833.6	395.7
18	102,030.5	9,229.3	395.7
19	111,457.6	9,624.9	395.7
20	121,280.4	10,020.6	395.7
21	131,498.8	10,416.3	395.7

TEMPORARY EXCAVATION HEIGHT

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	gqd	WALL HEIGHT:	10 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	906 psf	SURCHARGE:	0 pounds
PHI ANGLE:	41.3 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	130 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	3 feet
CD (C/FS):	724.8 psf	FINAL TENSION CRACK:	10 feet
PHID = ATAN(TAN(PHI)/FS) =	35.1 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	46 degrees
AREA OF TRIAL FAILURE WEDGE	25.3 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	3294.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	408 trials
LENGTH OF FAILURE PLANE	4.3 feet
DEPTH OF TENSION CRACK	6.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	3.0 feet
CALCULATED HORIZONTAL THRUST	**-1973.6 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-39.5 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**10.0 feet**

TEMPORARY EXCAVATION HEIGHT

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af		WALL HEIGHT:	5	feet
SHEAR DIAGRAM:	0		BACKSLOPE ANGLE:	0	degrees
COHESION:	313	psf	SURCHARGE:	0	pounds
PHI ANGLE:	34.8	degrees	SURCHARGE TYPE:	U	Uniform
DENSITY:	130	pcf	INITIAL FAILURE ANGLE:	20	degrees
SAFETY FACTOR:	1.25		FINAL FAILURE ANGLE:	70	degrees
WALL FRICTION:	0	degrees	INITIAL TENSION CRACK:	3	feet
CD (C/FS):	250.4	psf	FINAL TENSION CRACK:	10	feet
PHID = ATAN(TAN(PHI)/FS) =		29.1	degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	44	degrees
AREA OF TRIAL FAILURE WEDGE	10.7	square feet
TOTAL EXTERNAL SURCHARGE	0.0	pounds
WEIGHT OF TRIAL FAILURE WEDGE	1385.1	pounds
NUMBER OF TRIAL WEDGES ANALYZED	408	trials
LENGTH OF FAILURE PLANE	4.2	feet
DEPTH OF TENSION CRACK	2.1	feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	3.0	feet
CALCULATED HORIZONTAL THRUST	**-575.4**	**pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-46.0**	**pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**5.0**	**feet**

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	313 psf	SURCHARGE:	0 pounds
PHI ANGLE:	34.8 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	130 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	208.7 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 24.9 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	41.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	5390.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	9 feet
DEPTH OF TENSION CRACK	4.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	5.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**1279.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**17.8 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**30.0 pcf**

RETAINING WALL

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	313 psf	SURCHARGE:	0 pounds
PHI ANGLE:	34.8 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	130 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	313.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 34.8 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0.36 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	50.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	6571.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	9 feet
DEPTH OF TENSION CRACK	4.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	6.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**1665.0 pounds**

SHORING PILE

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED RETAINING WALLS. THE WALL HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. THE MONONOBE-OKABE METHOD USED TO CALCULATE SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Af	RETAINED LENGTH	12 feet
SHEAR DIAGRAM:	0	BACKSLOPE ANGLE:	0 degrees
COHESION:	313 psf	SURCHARGE:	0 pounds
PHI ANGLE:	34.8 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	130 pcf	INITIAL FAILURE ANGLE:	40 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	250.4 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 29.1 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	41.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	5390.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	9 feet
DEPTH OF TENSION CRACK	4.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	5.0 feet
CALCULATED THRUST ON PILE	**543.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**7.5 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**20.0 pcf**

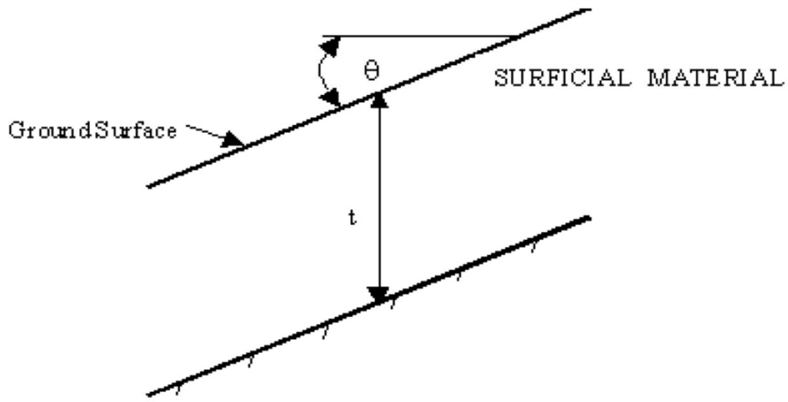
SURFICIAL STABILITY

CALCULATE THE SURFICIAL STABILITY OF THE EARTH MATERIAL USING THE INFINITE SLOPE ANALYSIS WITH PARALLEL SEEPAGE. THIS METHOD WAS RECOMMENDED BY THE ASCE AND THE BUILDING AND SAFETY ADVISORY COMMITTEE (8/16/78). MODIFIED FROM SKEMPTON & DeLORY, 1957.

CALCULATION PARAMETERS

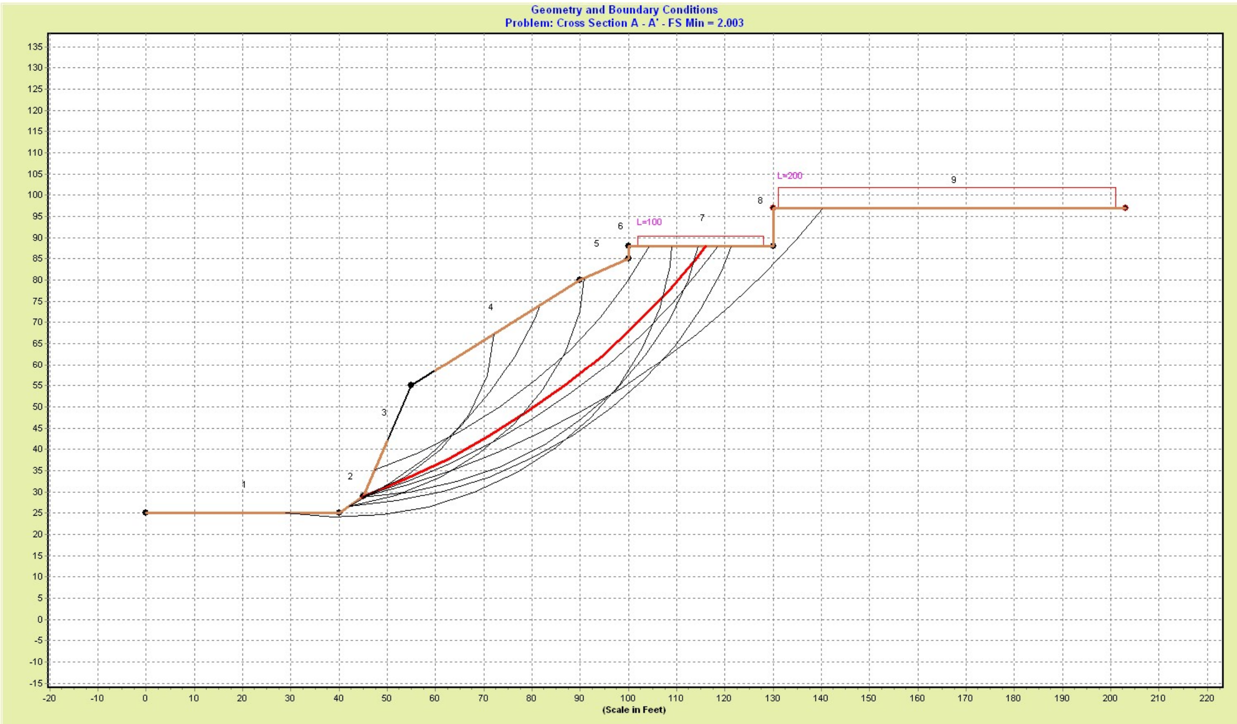
EARTH MATERIAL: Af

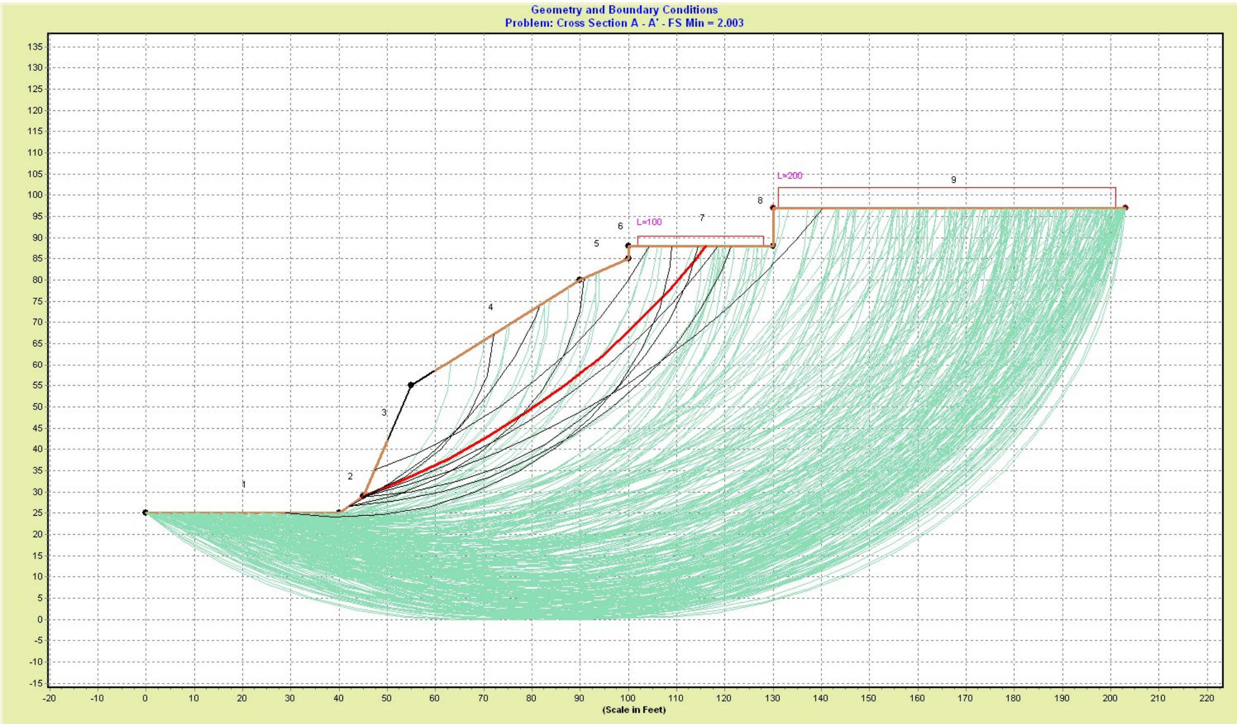
COHESION:	313 psf	SHEAR DIAGRAM:	0
PHI ANGLE:	34.8 degrees	SLOPE ANGLE:	34 degrees
DENSITY:	125 pcf	SATURATION DEPTH (t):	4.0 feet



$$FS = \frac{C + (\gamma_{soil} - \gamma_{water}) \bullet t \bullet \cos^2\theta \tan\Phi}{\gamma_{soil} \bullet t \bullet \cos\Phi \sin\Phi}$$

SAFETY FACTOR = 1.87


Geometry and Boundary Conditions
Problem: Cross Section A - A' - FS Min = 2.003
L=200
L=100
(Scale in Feet)


Geometry and Boundary Conditions
Problem: Cross Section A - A' - FS Min = 2.003
L=100
L=200
1
2
3
4
5
6
7
8
9
(Scale in Feet)

** PCSTABL6 **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION   Cross Section A - A'

BOUNDARY COORDINATES

9 Top   Boundaries
9 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	25.00	40.00	25.00	1
2	40.00	25.00	45.00	29.00	1
3	45.00	29.00	55.00	55.00	1
4	55.00	55.00	90.00	80.00	1
5	90.00	80.00	100.00	85.00	1
6	100.00	85.00	100.10	88.00	1
7	100.10	88.00	130.00	88.00	1
8	130.00	88.00	130.10	97.00	1
9	130.10	97.00	203.00	97.00	1

1

ISOTROPIC SOIL PARAMETERS

1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	130.0	140.0	906.0	41.3	0.00	0.0	0

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	102.00	128.00	100.0	0.0
2	131.00	201.00	200.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

1

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

```
 20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between  X =   0.00 ft.
                             and  X =  50.00 ft.


Each Surface Terminates Between   X =  60.00 ft.
                            and   X = 203.00 ft.


Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =  0.00 ft.


10.00 ft. Line Segments Define Each Trial Failure Surface.
```

1

```
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined.  They Are Ordered - Most Critical
First.


* * Safety Factors Are Calculated By The Modified Bishop Method * *


Failure Surface Specified By 11 Coordinate Points
```

Point No.	X-Surf (ft)	Y-Surf (ft)
1	44.74	28.79
2	53.79	33.04
3	62.57	37.83
4	71.04	43.14
5	79.18	48.96
6	86.95	55.25
7	94.32	62.00
8	101.28	69.19
9	107.79	76.78
10	113.83	84.75
11	116.00	88.00

```
Circle Center At X =  -21.1 ; Y =  180.8  and Radius,  165.6

      ***     2.003   ***


     Individual data on the    16  slices
```

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.3	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	8.8	10793.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	1.2	3158.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	7.6	21574.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	8.5	25269.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	8.1	24658.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	7.8	23175.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	3.1	8835.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	4.3	11838.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	5.7	13760.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.1	241.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	1.2	2981.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0

13	0.7	1719.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	5.8	10991.8	0.0	0.0	0.0	0.0	0.0	0.0	579.1
15	6.0	5684.9	0.0	0.0	0.0	0.0	0.0	0.0	604.0
16	2.2	459.2	0.0	0.0	0.0	0.0	0.0	0.0	217.1

** PCSTABL6 **

by
Purdue University

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

PROBLEM DESCRIPTION   Cross Section A - A'

BOUNDARY COORDINATES

9 Top   Boundaries
9 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	25.00	40.00	25.00	1
2	40.00	25.00	45.00	29.00	1
3	45.00	29.00	55.00	55.00	1
4	55.00	55.00	90.00	80.00	1
5	90.00	80.00	100.00	85.00	1
6	100.00	85.00	100.10	88.00	1
7	100.10	88.00	130.00	88.00	1
8	130.00	88.00	130.10	97.00	1
9	130.10	97.00	203.00	97.00	1

1

ISOTROPIC SOIL PARAMETERS

1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	130.0	140.0	906.0	41.3	0.00	0.0	0

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	102.00	128.00	100.0	0.0
2	131.00	201.00	200.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

A Horizontal Earthquake Loading Coefficient
Of0.360 Has Been Assigned

A Vertical Earthquake Loading Coefficient

```
          Of0.000 Has Been Assigned

          Cavitation Pressure =    0.0 (psf)
1


          A Critical Failure Surface Searching Method, Using A Random
          Technique For Generating Circular Surfaces, Has Been Specified.


          400 Trial Surfaces Have Been Generated.


           20 Surfaces Initiate From Each Of 20 Points Equally Spaced
          Along The Ground Surface Between  X =   0.00 ft.
                                       and  X =  50.00 ft.


          Each Surface Terminates Between   X =  60.00 ft.
                                      and   X = 203.00 ft.


          Unless Further Limitations Were Imposed, The Minimum Elevation
          At Which A Surface Extends Is  Y =  0.00 ft.


          10.00 ft. Line Segments Define Each Trial Failure Surface.


1

          Following Are Displayed The Ten Most Critical Of The Trial
          Failure Surfaces Examined.  They Are Ordered - Most Critical
          First.


          * * Safety Factors Are Calculated By The Modified Bishop Method * *


          Failure Surface Specified By 11 Coordinate Points
```

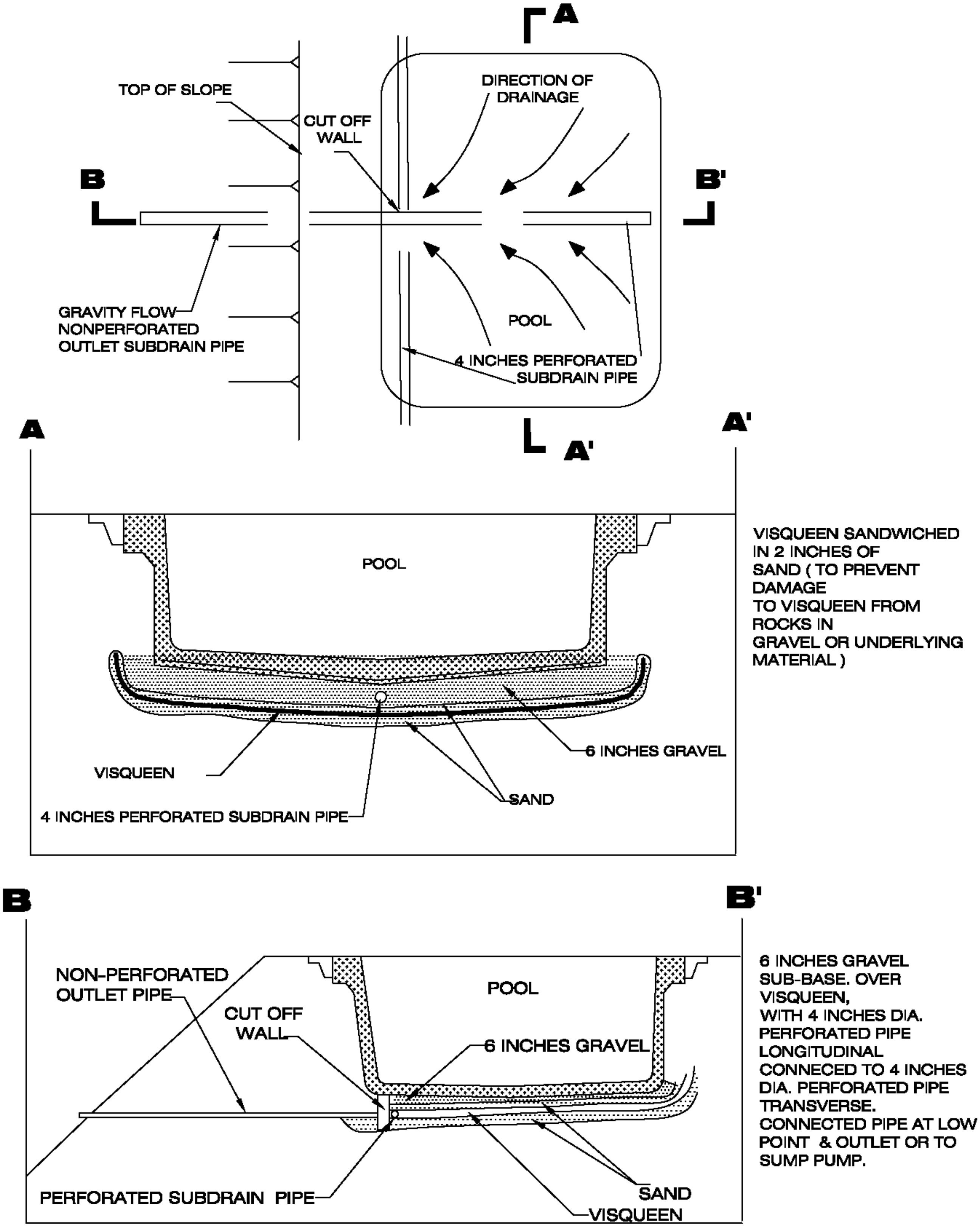
Point No.	X-Surf (ft)	Y-Surf (ft)
1	44.74	28.79
2	53.79	33.04
3	62.57	37.83
4	71.04	43.14
5	79.18	48.96
6	86.95	55.25
7	94.32	62.00
8	101.28	69.19
9	107.79	76.78
10	113.83	84.75
11	116.00	88.00

```
          Circle Center At X =  -21.1 ; Y =  180.8  and Radius,  165.6

                ***     1.194   ***



               Individual data on the    16  slices
```

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)

1	0.3	1.5	0.0	0.0	0.0	0.0	0.5	0.0	0.0
2	8.8	10793.0	0.0	0.0	0.0	0.0	3885.5	0.0	0.0
3	1.2	3158.7	0.0	0.0	0.0	0.0	1137.1	0.0	0.0
4	7.6	21574.3	0.0	0.0	0.0	0.0	7766.8	0.0	0.0
5	8.5	25269.7	0.0	0.0	0.0	0.0	9097.1	0.0	0.0
6	8.1	24658.9	0.0	0.0	0.0	0.0	8877.2	0.0	0.0
7	7.8	23175.3	0.0	0.0	0.0	0.0	8343.1	0.0	0.0
8	3.1	8835.7	0.0	0.0	0.0	0.0	3180.8	0.0	0.0
9	4.3	11838.0	0.0	0.0	0.0	0.0	4261.7	0.0	0.0
10	5.7	13760.0	0.0	0.0	0.0	0.0	4953.6	0.0	0.0
11	0.1	241.6	0.0	0.0	0.0	0.0	87.0	0.0	0.0
12	1.2	2981.8	0.0	0.0	0.0	0.0	1073.4	0.0	0.0
13	0.7	1719.8	0.0	0.0	0.0	0.0	619.1	0.0	0.0
14	5.8	10991.8	0.0	0.0	0.0	0.0	3957.1	0.0	579.1
15	6.0	5684.9	0.0	0.0	0.0	0.0	2046.6	0.0	604.0
16	2.2	459.2	0.0	0.0	0.0	0.0	165.3	0.0	217.1

POOL SUBDRAIN DETAIL

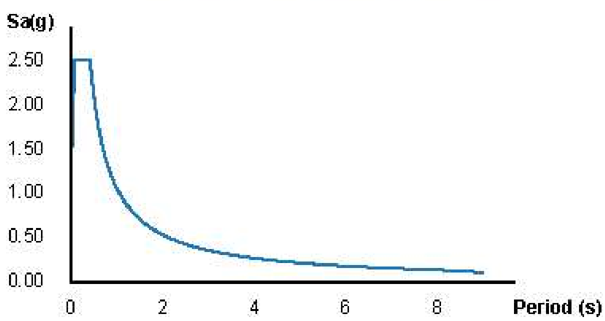


ATC Hazards by Location

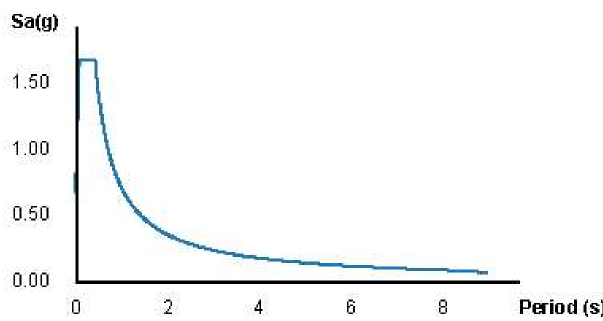
Search Information

Address:	1766 Cielito Dr, Glendale, CA 91207, USA
Coordinates:	34.1729274, -118.2507893
Elevation:	1085 ft
Timestamp:	2020-04-17T16:57:14.776Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	C

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	2.099	MCE_R ground motion (period=0.2s)
S_1	0.764	MCE_R ground motion (period=1.0s)
S_{MS}	2.518	Site-modified spectral acceleration value
S_{M1}	1.07	Site-modified spectral acceleration value
S_{DS}	1.679	Numeric seismic design value at 0.2s SA
S_{D1}	0.713	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	E	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	1.4	Site amplification factor at 1.0s

CR_S	0.894	Coefficient of risk (0.2s)
CR_1	0.894	Coefficient of risk (1.0s)
PGA	0.9	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.081	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	2.099	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.347	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.331	Factored deterministic acceleration value (0.2s)
S1RT	0.764	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.855	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.778	Factored deterministic acceleration value (1.0s)
PGAd	0.933	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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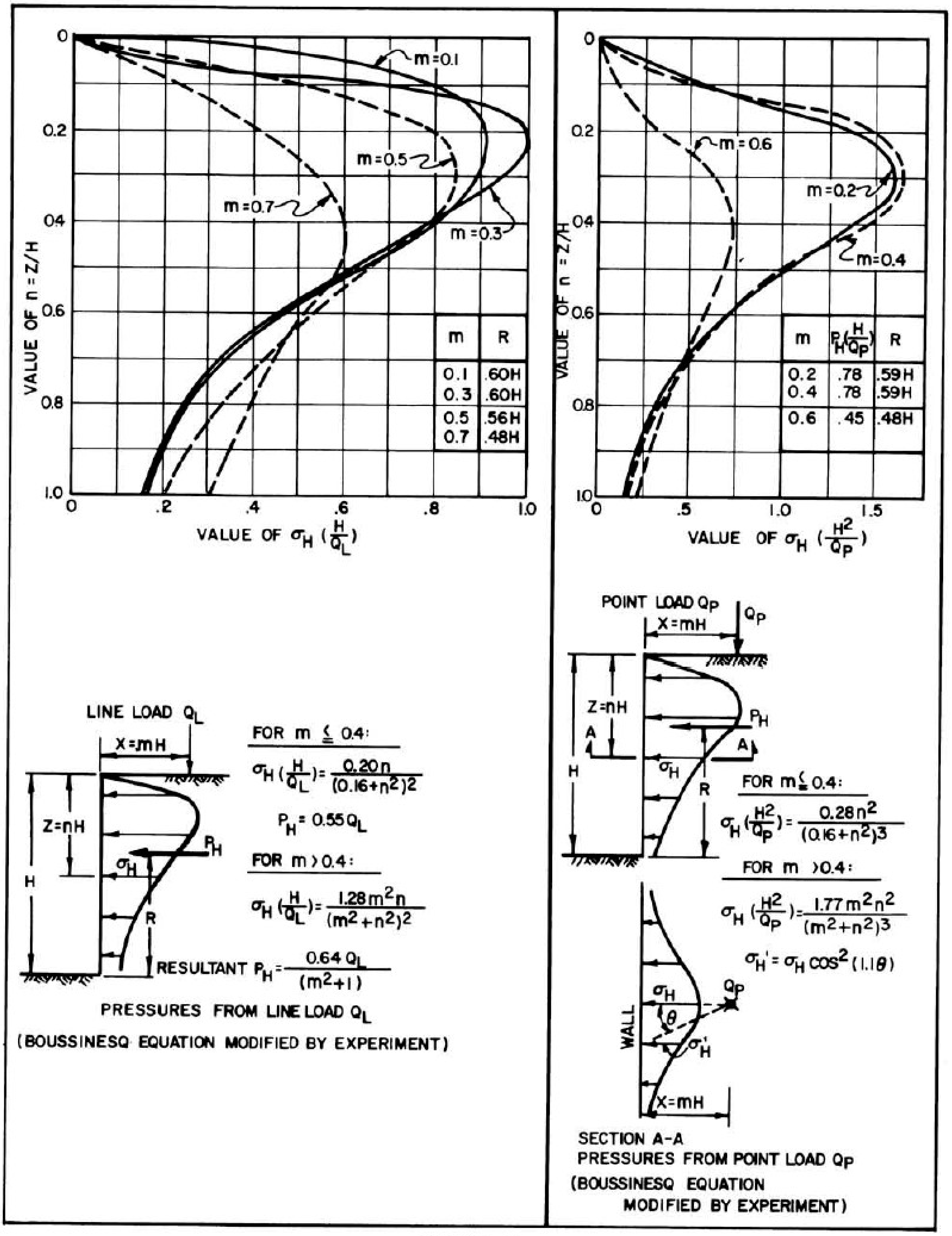

FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

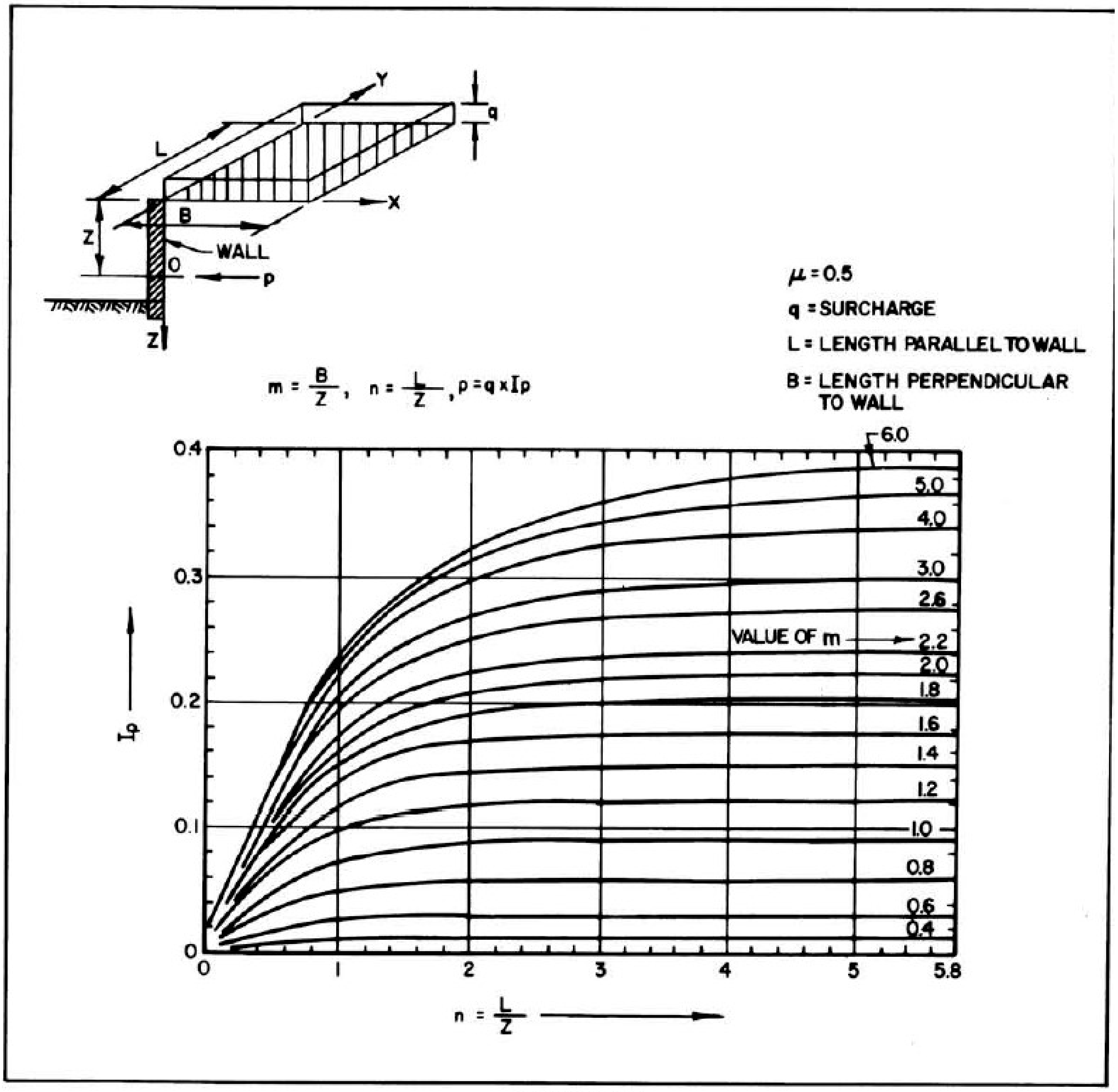

FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

APPENDIX IV

REFERENCES

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CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

June 25, 2021 Project 200416

Dr. Jack Demirchian
1766 Cielito Drive
Glendale, CA

Subject: **SUPPLEMENTAL REPORT No. 1**
1766 Cielito Drive
Glendale, CA

Dear Dr. Demirchian:

With regard to the non-certified fill, about 20 feet thick on top bedrock, existing on your property, it is our firm's opinion that trimming the fill thickness will reduce the potential for future instability on the steep slope portion. Should you have any questions regarding this report, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,
CREATIVE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
RAYMOND M. HADDAD
No. 2985
GEOTECHNICAL
STATE OF CALIFORNIA

Raymond M. Haddad
Project Engineer
GE 2985
RMH: -2

Distribution: (1) Addressee Via Email

DRAFT

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CREATIVE GEOTECHNICAL, INC.
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

September 7, 2022 Project 200416

Dr. Jack Demirchian
1766 Cielito Drive
Glendale, CA

Subject: **SUPPLEMENTAL REPORT No. 2**
1766 Cielito Drive
Glendale, CA

References:

1) Preliminary Geologic and Geotechnical Engineering report by Creative Geotechnical, Inc. covering the subject site, dated August 2, 2021.

Dear Dr. Demirchian:

With regard to the non-certified fill, about 20 feet thick on top bedrock, existing on your property, it is our firm's opinion that trimming the fill down to bedrock will reduce the potential for future instability on the steep slope portion (refer to cross sections A-A' and B-B' in Ref. No.1 above). Should you have any questions regarding this report, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,
CREATIVE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
RAYMOND M. HADDAD
No. 2985
GEOTECHNICAL
STATE OF CALIFORNIA

Raymond M. Haddad
Project Engineer
GE 2985
RMH: -3

Distribution: (1) Addressee Via Email