APPENDIX O

GEOTECHNICAL REPORT

GEOTECHNICAL REVIEW REPORT
TENTATIVE TRACT MAP NO. 82349
CHADWICK RANCH, CITY OF BRADBURY
LOS ANGELES COUNTY, CALIFORNIA

NEVIS CAPITAL, LLC

October 9, 2019
J.N. 17-219

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NEVIS CAPITAL, LLC
335 N. Berry Street
Brea, CA 92821

Attention: Mr. Scott Yang

Subject: **Geotechnical Review Report, Tentative Tract Map No. 82349, Chadwick Ranch, City of Bradbury, Los Angeles County, California**

Dear Mr. Yang:

Petra Geosciences, Inc. (Petra) is submitting herewith our geotechnical review of the tentative tract map for the Chadwick Ranch project in the city of Bradbury, California. The major soils engineering/geologic issues identified within this document include:

- Unsuitable soil removals;
- Slope stability;
- Excavation and engineering characteristics of earth materials;
- Earthwork considerations;
- Seismic hazard evaluation; and
- Groundwater conditions and subsurface drainage.

Petra appreciates the opportunity to provide you with geotechnical consulting services. If you have any questions or should you require any additional information, please contact us at our Santa Clarita office at (661) 255-5790.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Theodore M. Wolfe
Senior Associate Geologist

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GEOTECHNICAL REVIEW REPORT
TENTATIVE TRACT MAP NO. 82349
CHADWICK RANCH, CITY OF BRADBURY
LOS ANGELES COUNTY, CALIFORNIA

PURPOSE AND SCOPE OF STUDY

The purpose of this study was to: 1) review existing geologic/geotechnical references considered germane to the site; 2) conduct a field investigation to obtain site specific data; 3) evaluate the engineering properties of the onsite soil materials; and 4) provide conclusions and recommendations for grading and construction of the proposed site improvements.

Specific items evaluated as part of this report include unsuitable soil removals, slope stability, excavation and engineering characteristics of the onsite earth materials, earthwork considerations, groundwater and subsurface drainage conditions, seismic hazard evaluation, slope and lot maintenance recommendations and preliminary foundation design considerations.

The scope of this study included the following tasks:

1. A review of both published and unpublished geotechnical documents relative to the site.
2. Geologic field mapping/reconnaissance.
3. Excavation of seven (7) flight auger borings (B-1 through B-7) and nine (9) backhoe test pits (TP-1 through TP-9).
4. A Seismic Refraction Survey consisting of four (4) traverses (S-1 through S-4) was conducted by an independent contractor.
5. Plotting of geotechnical information on the accompanying 80-scale Tentative Tract Map/Conceptual Grading Plan prepared by Proactive Engineering Associates, (Geotechnical Map, Plate 1).
6. Engineering and geologic analyses of the data developed during the current and previous field study and laboratory testing programs.
7. Preparation of this report.

SITE LOCATION AND PROJECT DESCRIPTION

Tentative Tract Map No. 82349 encompasses approximately 95± acres along the northern boundary of the city of Bradbury at the base/southern flank of the San Gabriel Mountains (i.e., the northern edge of the San Gabriel Valley). The irregularly shaped site is located near the northern terminus of Bliss Canyon Road and is bound by two major drainages and debris basins (see Figure 1). Bliss Canyon is located along the west side of the site and Spinks Canyon forms the eastern boundary. A Flood Control Road runs along the southern property boundary and serves to connect the two debris basin sites and provide gated vehicular access to the site. A graded/terraced hillside identified as the Spinks Debris Disposal Area forms the south-central limits of the property. The northerly limits of the site extend into hillside terrane of the San Gabriel Mountains.

The subject site is characterized by steep hillside topography. As depicted on the Geotechnical Map (Plate 1), a prominent north to northeasterly trending ridgeline transects the southwestern and central portions of the site. A broad fire road has been constructed/graded along this ridgeline. Steep natural slopes descend from the ridgeline. The northerly slopes descend to a major drainage that empties into the Bliss Canyon Debris Basin to the southwest. This drainage extends well past the northern site boundaries. Several north/south trending spur ridges extend from the main ridgeline to the south property boundary. The spur ridges are separated by two narrow drainages that extend to the southerly adjacent Flood Control Road. Natural slopes are relatively steep with slope gradients varying from approximately 1:1 to 2:1 (horizontal to vertical). Topographical relief from the north to south property limits is on the order of 900 feet. Within the limits of the proposed grading development (i.e. the approximate southern half of the site) the elevation difference is on the order of 500 feet from a low of approximately 800 feet above mean sea level (amsl) to a high of approximately 1,300 feet amsl. The natural slopes and drainages are covered with dense shrubs, brush and small trees. Large oak trees and several other tree species are located in the canyon bottoms and on the lower portions of the slopes. Grasses cover the main ridgeline where fire road construction has removed the natural vegetation.

PROPOSED DEVELOPMENT

The 80-scale conceptual grading plan indicates that Chadwick Ranch will be developed as a residential project with 13 building pads, one reservoir pad, 3 desilting basins and attendant streets and parkways. It is anticipated that conventional cut and fill grading techniques will be used to produce the proposed grades. Access will be provided from an existing flood control access road at the terminus/intersection of Bliss Canyon and Long Canyon Roads.

Both cut and fill slopes are designed to slope ratios of 2-horizontal to 1-vertical (2:1) or flatter. The highest proposed cut slope is an approximately 110± foot high, 2:1 slope that ascends from the north side of the reservoir pad. The maximum designed cut is approximately 90± feet. The highest proposed fill slope is an approximately 160± foot high, 2:1 slope that ascends from Pad 14. The maximum design fill depth is approximately 90± feet. Several retaining wall structures are proposed throughout the site. It is our understanding that these walls, which vary in height from 5 to 30 ± feet, will be designed as mechanically stabilized earth (MSE) structures.

FIELD INVESTIGATION

Geologic reconnaissance of the project site was conducted to observe existing site conditions and to prepare a base geologic map. The subsurface exploration program consisted of seven (7) flight auger borings (B-1 through B-7), and nine (9) backhoe test pits (TP-1 through TP-9). The exploratory borings and test pits were observed and logged by a representative of this firm's geologic/engineering staff. Logs of the exploratory borings and test pits are presented in Appendix I. The approximate locations of the exploratory excavations/soundings are shown on the Geotechnical Map (Plate 1).

GEOLOGIC CONDITIONS

Geologic and Geomorphic Setting

The subject site is located on the northern edge of the Los Angeles Basin within the Peninsular Ranges Geomorphic Province at the southern edge of the San Gabriel Mountains. The San Gabriel river is located 1-1/2 miles to the west and the topographically prominent Puente Hills are located 9 miles to the south of the site.

Based on regional geologic mapping and on a recent subsurface exploration, the subject property is underlain by Cretaceous age granitic rocks, which consist primarily of massive to foliated quartz diorite rock, granitic rock, and light-colored quartzo-feldspathic gneiss. These rocks are moderately fractured and deeply weathered. In the southern portion of the site, the igneous bedrock is mantled by dissected, older alluvial fan deposits (Pleistocene age), locally referred to as the San Dimas Formation. These deposits consist of gravel, sand, silt, and clay, which is poorly consolidated and moderately to slightly decomposed. This unit varies in thickness from a few feet to as much as 70 to 90 feet. Stream laid alluvial deposits are located in the canyon bottoms. These loose, granular materials are derived from near source granitics/fan deposits and are on the order of 15 to 20 feet in maximum depth.

The site is located several hundred feet north of the main splay of the Sierra Madre Fault Zone. This zone has been classified as "active" per the State of California Alquist-Priolo (AP) Earthquake Fault Act. As part of this act, a fault hazard zone is established along the trace of active faults. Detailed fault investigations are required in this zone for the siting of any habitable structures. The fault hazard zone encroaches into the extreme southern site limits.

Artificial fills (non-engineered), topsoil, alluvium, older alluvial fan deposits, and igneous bedrock were encountered during the field portion of our investigation and are described in greater detail in the following sections.

Stratigraphy

Artificial Fill (Af)

Non-engineered artificial fills are present in localized areas throughout the site. These materials are associated with previous grading to develop access roads and debris basins. A significant volume of fill has been placed in the Spinks Debris Disposal Area. We estimate these fills vary in thickness from about 5 feet to up to 20 feet. The fill materials observed generally consist of sand, silty sand and clayey sand, with scattered gravel and cobbles. These fills were typically loose to medium dense and may be susceptible to settlement. as well as erosion and/or failure within the sloping areas. Portions of the fill may contain some trash, vegetation, debris, and boulders.

Topsoil (No Map Symbol)

The majority of the site is covered with a thin mantle of topsoil. The topsoil generally consists of silty sand and clayey sand and some sandy clay which is generally fine to medium grained, dark grayish brown to dark reddish brown, dry to damp, loose to medium dense, and soft to firm, porous and locally desiccated with some gravel to cobble size bedrock fragments and roots. The thickness of the topsoil is estimated to vary from about 1 to 3 feet thick. Topsoil is susceptible to erosion and shallow slumping where exposed in steep natural slopes.

Alluvium (Qal)

Recent Holocene alluvial materials are present in the bottom of the canyon areas throughout the site. These materials generally consist of light to medium gray to gray-brown silty sand, sand, and gravelly sand which are fine to coarse grained, dry to very moist, loose to medium dense with occasional cobbles and boulders. Based on test pit excavations, the alluvium is estimated to vary in depth from 5 to 15± feet.

San Dimas Formation (Osd)

Pleistocene-age older alluvial fan deposits which are locally referred to as the San Dimas Formation, mantle/overlie the bedrock in the central and southern portions of the property. These materials generally consist of reddish brown to orange-brown silty sand and clayey sand that is fine to coarse grained. The older alluvial fan deposits were observed to be generally dry to slightly moist, loose to moderately dense to very dense, and contained some gravel and cobbles with occasional boulders. The upper 1 foot to 3 feet of the older alluvial fan deposits are weathered, locally porous and lower in density. Occasional small pores were observed in the borings along with some clay coating on sand grains and fractures with some ped development. These deposits are estimated to be vary in thickness from 25 feet to 75± feet.

Quartz Diorite (qd)

Bedrock consists of older Cretaceous age Quartz Diorite with occasional lenses of gneiss and quartz veins. These materials are generally gray to brownish gray, fine to medium grained, dry to damp, moderately hard to very hard, and massive to locally gneissoid. The quartz diorite was generally moderately to very weathered and moderately to very fractured within the upper 10 feet to 25 feet.

Geologic Structure

Geologic structure in the igneous rocks/quartz diorite, where observed, is generally depicted by localized jointing or fracturing. A defined structural trend is not apparent. The San Dimas Formation is characterized by granular, coarsening and fining sequences that is generally massive. Occasional bedding/depositional layers were observable with orientations that vary from shallow to medium angles that dip to the south.

Ground Water

Groundwater was not encountered during our field study. Minor seepage was reported near the bottom of Boring B-7 in the canyon bottom south of the proposed grading limits. The occurrence of groundwater/seepage is dependent upon seasonal variations in rainfall. It is likely that after prolonged periods of heavy rainfall, perched groundwater conditions will occur in the bottom of the canyons and swales and along the contact between the older alluvial fan deposits and the underlying quartz diorite.

Faulting

Research of published and unpublished geologic/geotechnical maps and geographic literature and review of aerial photographs indicates that the site is located in a seismically complex area. The site lies approximately 1 mile north of the Sierra Madre-San Fernando Fault, 3 miles northeast of the Raymond Hill Fault, 2 miles south of the Clamshell-Sawpit Fault, 7 miles south of the San Gabriel Fault, and 21 miles south of the San Andreas Fault. The subject site, in relation to the known active and potentially active faults in the region, is presented on Figure 2.

There are no mapped faults onsite. The site does not lie within the bounds of an "Earthquake Fault Zone," as defined by the state of California in the Alquist-Priolo (AP) Earthquake Fault Zoning Act. However, the site does lie just north of an AP Zone for the potentially active Sierra Madre Fault.

Review of aerial photographs revealed fairly weak northwesterly/southeasterly trending photo lineaments to the northwest of the site. These lineaments seemingly “die out” east of Bradbury and Bliss Canyons and do not project onsite. A feasibility study conducted in 1999 (see References) which covered approximately 300 acres that included the subject site, also identified these lineaments and depicted them traversing the central portion of the site. These lineaments were tied to shear zones observed/mapped hundreds of feet west of the proposed development. Field mapping was conducted along the central portions of the site in the canyon areas to observe bedrock/quartz diorite exposures in the canyon bottoms and on the slope flanks. Shear zones as described west of the site with "10 to 15-foot-wide intensely fractured bedrock" sections were not observed. Therefore, it is this firm's opinion that these weak lineaments are not associated with faulting.

Seismic Hazards

Earthquakes have occurred in the Los Angeles region and will, undoubtedly, occur in the future. The project site is, as is all of Los Angeles County, in a seismically active region. Forty-four (44) faults have been identified within a 100-kilometer radius from the project site. Design and construction of the structures should be in accordance with the applicable state and local codes pertaining to both primary and secondary seismic hazards.

Primary earthquake hazards include both surface rupture and ground motion (shaking). Secondary hazards resulting from major earthquakes include liquefaction, seismically induced flooding, and seismically induced landsliding.

Primary Hazards

Surface Rupture

The State of California has identified faults that are considered capable of producing "surface displacement within the Holocene time (about the last 11,000 years)". The closest such fault/zone is the Sierra Madre Fault which is located approximately 1 mile south of the property limits. There are no mapped faults shown on any of the published regional geological maps which cover the subject area (Dibblee, 1992; Morton, 1976; Weber, 1973), including the State of California Earthquake Fault Zone Maps (State of California, 1999).

Ground Motion (Shaking)

Because Chadwick Ranch is within a seismically active area, the potential exists for ground motion to affect future improvements. Petra has thus assessed free-field horizontal ground acceleration (PGA) using currently accepted methodology (Appendix II, herein). Distances to selected major faults are shown on the following table.

Fault Name	Approximate Distance in Miles
Sierra Madre	1.1
Raymond	2.0
Clamshell-Sawpit	3.4
San Gabriel	7.1
San Jose	8.9
Puente Hills (LA)	9.2
Elysian Park	9.4
Puente Hills (Coyote Hills)	10.1
San Andreas (Mojave S)	20.7

Secondary Hazards

Liquefaction and Dynamic Settlement

Seismic agitation of loose, saturated sands and silty sands can result in a build-up of pore water pressures (CDMG, 1997). If these pore water pressures are sufficient to overcome overburden stresses, a temporary quick condition known as liquefaction can result. This can be manifested as sand boils, lateral spreading, or dynamic settlement.

Potentially liquefiable soils are present on site in the form of loose/soft alluvium, colluvium and non-engineered artificial fill. Bedrock units are not liquefiable. Based on a review of the Seismic Hazard Map for the Azusa Quadrangle, no portion of the developable site area is located in a zone of required Liquefaction Potential. Potentially liquefiable materials will be removed as part of the remedial grading operations.

Seismically Induced Flooding

Seismically induced flooding normally includes flooding due to a tsunami (seismic sea wave), a seiche (wave generated in an enclosed body of water), or failure of a dam/reservoir or other water retention structure up-stream of the site. The site is located over 30 miles from the Pacific Ocean. In addition, there are no known up-canyon dams or reservoirs whose failure would impact the site. As such, the potential for seismically induced flooding is considered nil.

Seismically Induced Landsliding

The site is located within a hillside region and has been identified by the state mandated Seismic Hazards Mapping Act as requiring investigation for earthquake induced landslides. As part of the preparation of this report, this firm performed stability analyses of selected proposed cut, proposed fill and natural slopes within and adjacent to the proposed grading limits depicted on the Site Plan. Pseudo-static slope stability analyses were performed in accordance with guidelines for preparation of geotechnical reports. The results of these calculations (included herein) meet or exceed minimum requirements for both static and pseudo-static conditions.

ENGINEERING CHARACTERISTICS

Suitability of Onsite Soils for Fill and Oversized Rocks

The onsite soils/alluvium, artificial fill and bedrock are considered suitable for use as engineered fill provided they are free of organics, demolition debris or other deleterious materials. It is likely that oversize material will be generated from cuts in the hard bedrock areas. Oversize material is generally classified as rock like material that is greater than 12 inches in diameter that will not break down when processed/laid down as engineered fill. These materials will require special handling and grading equipment. Hold down zones will be required so that oversize materials are not placed beneath pad areas where footing excavations would be impacted or in street areas within the depth of utility excavations. Recommendations are presented in subsequent sections of this report.

Excavation Characteristics

The site is underlain by crystalline bedrock that is hard to very hard. These hard materials are exposed on the major ridgeline that defines the north and western limits of the proposed grading. The bedrock is mantled by dissected alluvial fan deposits on the southerly facing slopes. A Seismic Refraction Survey, consisting of four seismic lines (S-1 through S-4) was conducted to evaluate site excavation characteristics and evaluate the extent/depth of the alluvial fan deposits (see Appendix A). Seismic lines S-1 and S-2 are located along the main ridgeline where the deepest bedrock cuts, on the order of 80 to 90 feet are proposed. The seismic survey indicates that the upper 60± feet of these materials are rippable using conventional grading equipment. Excavation of bedrock materials below this depth will likely require heavy ripping (i.e. single shank with a DR9 dozer or equivalent). Blasting may be required in localized areas in the deeper cuts. It is recommended that an experienced grading contractor be retained for consultation regarding the possible scope of hard rock operations.

Earthwork Adjustments

Average earthwork adjustment factors are presented below. These factors are provided to assist in earthwork balance studies.

Geologic Unit	Map Symbol	Adjustment Factor
Granitic Bedrock	Qd	Bulk 5-15%
Alluvium Fan	Qsd	Shrink 0-10%
Alluvium/Soil	Qal	Shrink 10-15%
Non-engineered Artificial Fill	Af	Shrink 5-15%

These factors are estimates based upon the available site information and experience working in similar geologic units. Owing to the uncertainty of these estimates, contingencies should be made to adjust the earth work balance when grading is in progress and actual needs are better defined.

Oversized materials (greater than twelve inches in diameter) will be generated from the deeper bedrock cuts. Oversized materials should be handled as described in the Earthwork Considerations section of this report.

Compressibility

Soil, non-engineered artificial fill, alluvium and weathered bedrock and alluvial fan deposits are compressible in their existing state and will require removal from areas planned to receive fill. Estimated depths of unsuitable materials are indicated on the accompanying Geotechnical Map. These materials, once properly moisture conditioned, will be suitable for use as compacted fill.

Expansion Potential

Based on visual classification of site soils and experience on similar projects in the Bradbury area, the onsite materials are generally considered to possess "very low" to "low" expansion potential; although some finer-grained materials may exhibit "medium" or "high" expansion potential. Specific testing for expansion potential should be performed the grading plan review stage and on the as-graded near-surface materials at the completion of grading.

Geochemical Considerations

Based on a preliminary assessment of the site soils and a review of pertinent geotechnical literature indicates that soluble sulfates are relatively low and that no restrictions for cement type or maximum water-cement ratio for concrete would are anticipated.

It is anticipated that the on-site materials will be classified as "severely corrosive" to "moderately corrosive" towards on-site ferrous improvements.

Further evaluation of geochemical considerations should be performed at the grading plan review stage.

CONCLUSIONS AND RECOMMENDATIONS

Based on our subsurface investigation and analysis, development of the site as shown on the enclosed 80-Scale Grading Plan is considered feasible from a geotechnical point of view.

All grading shall be accomplished under the observation and testing of the project geotechnical engineer and engineering geologist or his/her authorized representative in accordance with the **recommendations** contained herein, the current **Building Code** of the City of Bradbury and this firm's **"Earthwork Specifications"** (Appendix IV).

Stripping and Deleterious Material Removal

Existing vegetation, trash, debris and any other deleterious material should be removed and wasted from the site prior to commencing removal of unsuitable soils and placement of compacted fill materials.

Removal of Unsuitable Material

All existing artificial fill, natural soils, alluvium and weathered bedrock and alluvial fan deposits shall be removed from areas planned to receive fill or where exposed at final grade. The resulting voids should be replaced to design grade with engineered fill. The exact extent of removals can best be determined in the field during grading when observation and evaluation can be performed by the soils engineer and/or engineering geologist. Removals should expose competent, unweathered bedrock/alluvial fan deposits and be observed and mapped by the engineering geologist prior to fill placement. Approximate depths of unsuitable material removal/overexcavation are indicated on the accompanying Geotechnical Map (Plate 1) and are depicted on the geologic/geotechnical cross-sections.

Pipelines, Oil and/or Water Wells, Cesspools and Septic Tanks/Vaults

Oil and/or water wells, if encountered within the areas proposed for development, should be abandoned in accordance with local and State of California Code requirements. Cesspools and septic tanks/vaults, if encountered, should be removed to a minimum of five (5) feet below existing grade or finish grade, whichever is lower. The portion of cesspools not removed should be pumped of their contents and filled with washed concrete sand, thoroughly jetted into place or, if in the influence zone of structures, with a lean 3-sack slurry mix. The remaining cavities should be filled with compacted fill as specified herein. An alternative to the above would be to excavate the entire cesspool to the bottom of the cesspool and to remove any surrounding wet, soft or unsuitable soils while laying back the excavation sides to approximately 2:1 (horizontal to vertical). Backfill of the resultant void could then be completed with onsite soils.

Slope Stability and Remediation

Shear Strength Parameters

Proposed slopes are planned at a 2:1 (horizontal to vertical) slope ratios or flatter with intervening terraces and drainage devices. The slope stability analyses performed for this study utilized shear strength values based data provided in the Seismic Hazard Zone Report for the Azusa Quadrangle and Petra's judgement and experience with similar units. The values are summarized below.

Shear Strength Parameters

Material	Cohesion (C, psf)	Angle of Internal Friction (Ø, degrees)
Quartz Diorite (qd)	450	34
San Dimas Formation (Qsd)	300	30
Compacted Fill	225	30

Fill Slopes

Fill slopes, when properly constructed at slope ratios of 2:1 (horizontal to vertical) and flatter, are considered to be grossly stable. The highest proposed 2:1 fill slope, which is approximately 160+ feet high, is depicted on Cross Section 2-2'. Stability analysis of Cross Section 2-2', which indicates factors of safety in excess of the required minimums for static and pseudostatic conditions, is presented in the Appendix.

Toe removals will be required for fill slopes located along the limits of grading. The removals should extend past the design toe a distance equal to the depth of removal (1:1 projection). A typical toe removal detail is provided as Plate G-6 (Appendix IV). If removals are limited due to property lines or physical impediments, then Restricted Use Zones or deepened foundation requirements may be established.

Cut Slopes

Cut slopes which expose granitic bedrock are considered to be grossly stable. The highest proposed cut slope is located superjacent to the reservoir pad and depicted on Cross Section 4-4'. Stability analysis of Cross Section 4-4', which indicates factors of safety in excess of the required minimums for static and pseudostatic conditions, is presented in the Appendix.

Cut slopes in the central and southern portions of the site will expose alluvial fan sediments. These sediments are layered and consist of granular and finer grained units which are generally considered unsuitable when exposed on cut slope faces. These slopes will require remediation in the form of stabilization fills. Recommendations for construction of these slopes is presented herein and typical details are presented in Appendix IV (Plates G-5 and G-6).

MSE Walls

Internal stability of the proposed MSE walls in under the purview of the wall designer. Global stability of the walls will be evaluated at the grading plan review stage. If necessary, additional recommendations can be presented at that time based on the results of the analyses.

Natural Slopes

Natural slopes descend from the north and south side of Building Pads 1 through 4 on the north side of the project. The slopes are underlain by granitic bedrock and are considered grossly stable. Stability analysis was conducted along Cross Section 3-3' which depicts the northerly facing slope Building Pad 2. Stability analysis of this section indicates factors of safety in excess of the required minimums for static and pseudostatic conditions (Appendix II).

Surficial Stability

Fill slopes are proposed to be constructed at a maximum slope ratio of 2:1, horizontal to vertical. The surfaces of the fill slopes within the site will be comprised of fill materials that consist of reconstituted native bedrock materials and alluvium, colluvium, and existing artificial fill materials. Surficial slope stability calculations were performed for fill slopes based on a depth of saturation of 4 feet below the slope face and assuming an infinite slope with seepage parallel to the slope face. A surficial stability analysis was also performed for a roughly 1.4:1 natural slope in the San Dimas formation. The stability calculations resulted in a factor of safety in excess of 1.5. Surficial stability calculations are presented in the Appendix.

Overexcavation of Building Pads and Steep Cut/Fill Transitions

Capped Building Pads

Building Pads 1 through 4 are design cut lots that will expose hard granitic bedrock. Hard rock conditions will make excavation of foundations difficult, therefore the building footprint should be overexcavated a minimum of 3 feet below the proposed bottom of footings or 5 feet, whichever is greater. In addition, consideration should be given to overexcavating the entire lot in order to facilitate future lot improvements such as pools and spas. The reservoir pad is also a cut lot that will expose hard bedrock and should be overexcavated. Depth of overexcavation should be evaluated when reservoir foundation plans are reviewed.

Cut/Fill Transition Lots

Cut portions of building pads that are transected by cut/fill transitions should be overexcavated a minimum depth of five (5) feet and replaced to design grade with compacted fill.

In order to reduce the differential settlement potential of lots with steep cut/fill transitions, the cut and shallow fill portions of the transition should be overexcavated such that the shallowest fill depth is at least equal to 1/3 the deepest fill section within the building pad footprint (15-foot maximum).

Subsurface Drainage

Canyon Subdrains

Subdrains should be installed in the bottom of the steep tributary canyons which drain to the southern property limits. Approximate subdrain locations are shown on the Geotechnical Map. Subdrains should be constructed in accordance with Plate G-2 (Appendix IV).

Backdrains

Backdrains will be required behind all stabilization fills, and skin fill slopes (reconstructed) in excess of 10 feet in height. Backdrains should be constructed in accordance with Plate G-3 (Appendix IV).

Temporary Construction Backcuts

The stability of temporary backcut slopes associated with stabilization fills is dependent on many factors which include slope angle, height, geologic structure of unsupported bedrock, shear strength along planes of weakness, groundwater conditions, nuisance water, and the length of time temporary cuts remain unsupported. Consequently, there is a risk of backcut failures during excavation of basal fill keys for stabilization fills. In order to minimize the potential for backcut failures, the following techniques should also be considered:

1. All basal fill keys should be excavated, observed by the project geologist, and then filled in the shortest practical period of time. Keyway excavations should never be allowed to stand open for prolonged periods of time.

2. Provisions should be made for preventing nuisance water and rainwater from collecting and ponding in keyway excavations.

3. Grading equipment and other construction traffic should never be allowed to traverse along the tops of temporary backcut slopes.

4. In addition to the above, all OSHA requirements should be followed with respect to excavation safety.

In consideration of inherent instability created by temporary construction backcuts for stabilization fills, buttress fills and unsuitable removals, it is imperative that grading schedules are coordinated to minimize unsupported exposure time of these excavations. Once started, these excavations and subsequent fill operations should be maintained to completion without intervening delays imposed by avoidable circumstances. In cases where five-day workweeks comprise a normal schedule, grading should be planned to avoid exposing at grade or near-grade excavations through a non-work weekend. Where improvements may be affected by temporary instability, either on/or offsite, further restrictions such as slot cutting, extending workday-weekend schedules, and/or other requirements considered critical to serving specific circumstances may be imposed.

Construction Staking and Survey

All removals and fill keys, including those excavated as part of stabilization fill construction, shall be surveyed by the Civil Engineer prior to observation and approval by the geotechnical engineer/geologist. Backdrain systems and subdrains should be survey located to verify location and gradients.

Settlement Monitoring

Post-grading settlement of deep fills will occur due to their own weight. Some areas of the major canyon fills are expected to exceed 90 feet in depth. The fills within the site will be derived from soil and bedrock materials that with expansion potentials ranging from “very low” to “medium”. Based on these conditions, it is expected that total primary consolidation of the new fill materials will be reached immediately at the completion of grading within lots underlain by 40 feet of compacted fill or less. In addition, considering the anticipated granular nature of the fill materials, long-term secondary settlement of these materials is not expected to be a significant design consideration. However, on lots underlain by 40 feet or more of compacted fill, it is recommended that settlement monitoring be performed. Surface monuments should be installed at finished grade in these deep fill areas immediately following completion of grading to verify post grading settlement. The survey monuments should be monitored on a weekly basis for the first three weeks, then once every two weeks for a total of one month. Subsequent readings should be taken once a month for three months, or whenever the settlement appears to stabilize. Building construction should not proceed until it is determined by this firm that primary consolidation has occurred and that any further anticipated settlement will be within acceptable tolerable limits.

Since near surface underground utilities within street areas generally have a greater tolerance for future differential and total settlements, the street areas may be released for construction following completion of grading at the discretion of the project geotechnical consultant.

Earthwork Considerations

Compaction Standards

Fill and processed natural ground should be compacted to a minimum relative compaction of 90 percent, as determined by ASTM Test Method D 1557. Each lift should be treated in a like manner until the desired finish grades are attained. In order to enhance the performance of the deep fill areas onsite and to aid in reducing the settlement monitoring potential, fills placed below forty (40) feet from ultimate finish grade shall be compacted to a minimum relative compaction of 93 percent as determined by ASTM Test Method D 1557. This criterion should take into consideration proposed future grades for residential development. Compaction shall be achieved at or slightly above optimum moisture content, and as generally discussed in the attached **"Earthwork Specifications"** (Appendix IV). Mixing and moisture conditioning will be required in order to achieve the required moisture conditions.

Observation of Excavations

All removal bottoms shall be observed and mapped by the engineering geologist and/or soils engineer prior to fill placement. Toe stakes should be provided by the Civil Engineer in order to verify required key dimensions and locations.

Treatment of Removal Bottoms

At completion of unsuitable soil removals and excavation of any required keyways, the exposed bottom shall be scarified to a minimum depth of eight (8) inches, moisture-conditioned (or dried back) to slightly above optimum conditions, and compacted to the standards set forth in this report.

If removal bottoms encounter wet materials (subject to pumping under the loading of standard earthmoving equipment), it may be necessary to stabilize the removal bottoms with gravel, or geofabric and gravel to provide a firm, working bottom to facilitate the placement of compacted fill.

Fill Placement

Following completion of remedial removals, exposed bottom surfaces in areas approved for placement of fill should first be scarified to a minimum depth of 6 inches, watered or air dried as necessary to achieve near optimum moisture conditions, and then compacted in place to a minimum relative compaction of 90 percent. Fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve near optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D 1557.

Benching

Fills placed against canyon walls, on natural slope surfaces inclining at 5:1, horizontal to vertical, or steeper, and against temporary backcut slopes associated with construction of stabilization fills should be placed on a series of level benches excavated into competent bedrock or competent native soil materials. These benches should be provided at vertical intervals of approximately 3 to 5 feet. Typical benching details are shown on Plates SG-5 through SG-8, Appendix IV.

Mixing

In order to prevent layering of different soil types and/or different moisture contents, mixing of materials may be necessary. The mixing should be accomplished prior to and as part of compaction of each fill lift. Discing may be required when either excessively dry or wet materials are encountered.

Fill Slope Construction

Fill slopes shall be overfilled to an extent determined by the contractor, but not less than two feet measured perpendicular to the slope face, so that when trimmed back to the compacted core, the required compaction is achieved.

Compaction of each fill lift should extend out to the temporary slope face. Backrolling during mass filling at intervals not exceeding four feet in height is recommended unless more extensive overfill is undertaken.

As an alternative to overfilling, fill slopes may be built to the finish slope face in accordance with the following recommendations:

- Compaction of each fill lift shall extend to the face of the slopes.
- Backrolling during mass grading shall be undertaken at intervals not to exceed four feet in height. Backrolling at more frequent intervals may be required.
- Care shall be taken to avoid spillage of loose materials down the face of the slopes during grading.
- At completion of mass filling, the slope surface shall be watered, shaped and compacted first with a sheepsfoot roller, then with a grid roller operated from a side boom Cat, or equivalent, such that compaction to project standards is achieved to the slope face.

Proper seeding and planting of the slopes should follow as soon as practical, to inhibit erosion and deterioration of the slope surfaces. Proper moisture control will enhance the long-term stability of the finished slope surface.

Oversized Materials

Oversize rock is defined as hard boulders or irreducible cemented bedrock fragments exceeding 12 inches in maximum dimension. It is anticipated that a significant amount of oversize rock will be generated in the deeper bedrock cuts. These materials should be placed in the lower portions of the deeper fills utilizing the typical detail shown on Plate SG-4, Appendix IV. Any oversize materials buried on site should be placed individually or in windows, and in a manner to avoid nesting, and then completely covered with granular on-site earth materials. The granular materials should be thoroughly watered and rolled to ensure closure of all voids. Oversize rock should not be placed within the upper 10 feet of finish grade within the building areas or street areas where they may interfere with footing and utility trenches, or in areas where they may interfere with the future construction of swimming pools and/or spas.

The above recommendations also apply to inert construction debris (concrete, brick, etc. but excludes metal debris such as re-bar or metal pipe) provided that the materials proposed for incorporation into the compacted fill section have been observed and approved by the project geotechnical engineer.

Haul Roads

Haul roads should be selected to avoid disturbing terrain which is to remain in a natural state. Also, haul roads traversing compacted fill areas should be coordinated and planned to avoid or minimize generation of loose spill fill thereon. When this condition is unavoidable, close coordination with the project geotechnical engineer and his/her representative will be required to eliminate intermingling of engineered and non-engineered fill.

During grading, special care should be exercised to avoid spilling and depositing of loose soil or debris onto slope areas and into areas programmed to remain in a natural state. Any loose slough, debris or other deleterious materials deposited or accumulated on natural areas will have to be removed by the contractor upon completion of grading.

Testing of Compacted Fill

Fill should be tested at the time of placement to ascertain that the required compaction is achieved.

Final Reports

The results of the observations and testing of all earthwork should be presented in a final geotechnical report following the completion of earthwork and grading.

DESIGN RECOMMENDATIONS

Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific acceleration response spectrum. The seismic parameters that were used to construct the acceleration response spectrum for analysis and design of the proposed site improvements were determined in accordance with the provisions of Section 1613 of the 2016 California Building Code (CBC), which incorporates the 2010 edition of the American Society of Civil Engineers (ASCE) document "*Minimum Design Loads for Buildings and Other Structures*" (ASCE/SEI 7-10).

To construct the site-specific acceleration response spectrum for this project, we performed a seismic hazard analysis to first determine the ground motion characteristics for the Risk Targeted Maximum Considered Earthquake (MCE_R) as required by Section 1613 of the 2016 CBC. We determined peak ground acceleration (PGA) levels for use in analysis and design as prescribed in Section 1803.5.12 of the 2016 CBC. The Building Seismic Safety Council (BSSC), in its commentary to Section 11.8.3 of ASCE/SEI 7-10, states that for ordinary design (including retaining walls), the use of the lower design level PGA is appropriate. However, for analysis of liquefaction, it states that the full MCE peak ground acceleration with a recurrence interval of approximately 2,475 years is to be used; due to the potentially catastrophic effect liquefaction can have on a building structure.

The MCE_R ground motion is determined using both probabilistic and deterministic methods and is defined as the level of ground motion that will produce 1 percent collapse risk in 50 years for a generic structure.

The probabilistic component is taken as the level of ground acceleration having a 2 percent chance of exceedance in 50 years (a 2,475-year recurrence interval). The deterministic models assume an 84th percentile ground motion to provide the upper bound subset for the likely ground motion at a site. Both types of analysis include directivity effects. The CBC also specifies that the MCE ground motion be scaled by a factor of ⅔ to determine the appropriate design values. This scaling is approximately equivalent to the level of ground motion that would result from a probabilistic analysis at a 10 percent chance of exceedance in 50 years (a 475-year recurrence interval).

To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used the computer application that is available at the Structural Engineers Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps web site https://seismicmaps.org/ to calculate the ground motion parameters. In addition, the USGS Earthquake Hazards Program web site https://earthquake.usgs.gov/hazards/interactive/ was used to determine the appropriate earthquake magnitude.

To run the above computer applications, site latitude, longitude, risk category and knowledge of "Site Class" are required. The site class definition depends on the average shear wave velocity, Vs30, within the upper 30 meters (approximately 100 feet) of site soils and bedrock. A shear wave velocity of 1,200 to 2,500 feet per second for the upper 100 feet was used for the site based on the result of seismic refraction survey, our engineering experience and judgment.

The following table, Table 1, provides parameters required to construct site-specific acceleration response spectrum for the site based 2016 CBC guidelines. Printouts of the computer output are attached in Appendix II.

TABLE 1

Seismic Design Parameters

Ground Motion Parameters	Reference	Parameter Value	Unit
Site Latitude (North)	-	34.1543	°
Site Longitude (West)	-	-117.9624	°
Site Class Definition [1, 2]	Section 1613.3.2	C	-
Assumed Risk Category [1]	Table 1604.5	II	-
M_w - Earthquake Magnitude [3]	USGS 2008 Interactive Deaggregation Tool	7.7	-
S_s - Mapped Spectral Response Acceleration [1, 2]	Figure 1613.3.1(1)	2.596	g
S_1 - Mapped Spectral Response Acceleration [1, 2]	Figure 1613.3.1(2)	0.970	g
F_a - Site Coefficient [1, 2]	Table 1613.3.3(1)	1.0	-
F_v - Site Coefficient [1, 2]	Table 1613.3.3(2)	1.3	-
S_{MS} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration [1, 2]	Equation 16-37	2.596	g
S_{M1} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration [1, 2]	Equation 16-38	1.261	g
S_{DS} - Design Spectral Response Acceleration [1, 2]	Equation 16-39	1.731	g
S_{D1} - Design Spectral Response Acceleration [1, 2]	Equation 16-40	0.84	g
T_o - ($0.2\ S_{D1}/\ S_{DS}$) [4]	Section 11.3	0.097	s
T_s - ($S_{D1}/\ S_{DS}$) [4]	Section 11.3	0.485	s
T_L - Long Period Transition Period [4]	Figure 22-12	8	S
F_{PGA} - Site Coefficient [4]	Figure 22-7	1.0	-
PGA_M - Peak Ground Acceleration at MCE [4, *]	Equation 11.8-1	0.972	g
Design PGA ≈ (⅔ PGA_M) - Slope Stability [2, †]	Similar to Equations 16-39 & 16-40	0.648	g
Design PGA ≈ (0.4 S_{DS}) – Short Retaining Walls [4, ‡]	Equation 11.4-5	0.692	g
C_{RS} - Short Period Risk Coefficient [4]	Figure 22-17	0.959	-
C_{R1} - Long Period Risk Coefficient [4]	Figure 22-18	0.949	-
Seismic Design Category [1, #]	Section 1613.3.5	E	-

References:

[1] California Building Code (CBC), 2016, California Code of Regulations, Title 24, Part 2, Volume I and II.
[2] Structural Engineers Association of California –https://seismicmaps.org/
[3] USGS Interactive Deaggregation Tool - https://earthquake.usgs.gov/hazards/interactive/
[4] American Society of Civil Engineers (ASCE/SEI), 2010, Minimum Design Load for Buildings and Other Structures, Standards 7-10.

Related References:

Federal Emergency Management Agency (FEMA), 2009, NEHERP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-750).

Notes:

* PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).

† PGA Calculated at the Design Level of ⅔ of MCE; approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).

‡ PGA Calculated for short, stubby retaining walls with an infinitesimal (zero) fundamental period.

The designation provided herein may be superseded by the structural engineer in accordance with Section 1613.3.5.1, if applicable.

Structural Design

It is anticipated that one- and two-story, wood frame and stucco residential structures with shallow foundations may be constructed on the lots. Maximum anticipated wall loads are expected to be less than 2 kips per foot. If larger, heavier structures are planned, the following design parameters may require revision.

Upon the completion of rough grading, finish grade samples should be collected and tested so as to provide specific recommendations as they relate to individual lots. These test results and corresponding design recommendations will be presented in a Final Rough Grading Report. Final foundation design recommendations should be made based upon specific structure-sitings, loading conditions, and as-graded soil conditions.

FOUNDATION DESIGN GUIDELINES

Allowable Soil Bearing Capacities

Pad Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade for pad footings that are not a part of the slab system and are used for support of such features as roof overhang, second-story decks, patio covers, etc. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Continuous Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Lateral Resistance

Provided that remedial grading is performed within the site in accordance with our "Earthwork" Recommendations, passive earth pressure of 250 pounds per square foot per foot of depth, to a maximum value of 2,500 pounds per square foot, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces.

It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

General Discussion of Footing and Slab on-Grade Design

It is anticipated that the majority of onsite soils will possess very low (E.I. ≤ 20) to low (E.I. 21 - 50) expansion potential when tested and classified in accordance with ASTM Test Method D 4829. However, some alluvial, colluvium and surface soils and the finer-grained materials within the site may possess medium (E.I. 51 - 90) and possibly even high (E.I. 91 - 130) expansion potential. For soils having medium to high expansion potential, consideration should be given to utilizing post-tensioned foundations. For preliminary design purposes, the following foundation design recommendations for both conventionally-reinforced and post-tensioned foundations systems are presented.

The 2016 CBC does not require special design of foundations and slabs-on-ground in order to resist potential effect of expansive soils for soils characterized as having very low (E.I. ≤ 20) expansion potential. However, the design of foundations and slabs-on-ground for soils classified as Low (E.I. 21 - 50) expansion potential (i.e., considered to be expansive per Section 1803.5.3 of the 2016 CBC) should be performed in accordance with the procedures outlined in Sections 1808.6.1 and 1808.6.2 of the 2016 CBC, respectively.

Briefly, Section 1808.6.1 of the 2016 CBC requires that foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Section 1808.6.2 of the 2016 CBC requires that non-prestressed slabs on-grade or mat foundations constructed on expansive soils be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations.* The 2016 CBC also requires that post-tensioned slabs on-grade or mat foundations placed on expansive soils be designed in accordance with *PTI DC10.5-12, "Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundation on Expansive Soils"* with the provision that the analyses used to determination of moments, shears and deflections are performed accordingly. It should be noted that, under certain conditions, the 2016 CBC allows for alternative, rational methods of analysis and design of such slabs provided that these methods account for soil-structure interaction, the deformed shape of the soil support, plate or stiffened plate action of the slab, as well as both center lift and edge lift conditions.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for reducing the effects of variability in composition and behavior within the site soils and long-term differential settlement. These recommendations have been developed on the basis of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.

It should also be noted that the recommendations for reinforcement provided herein are performance-based and intended only as guidelines to achieve adequate performance under the anticipated soil conditions. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion,) as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

Conventionally-Reinforced Slab on-Grade System

As stated above, onsite soils should be considered expansive per Section 1803.5.3 of the 2016 CBC. For soils that are considered expansive, Section 1808.6.2 of the 2016 CBC specifies that non-prestressed slab-on-grade foundations constructed on expansive materials should be designed in accordance with the latest edition of the Wire Reinforcement Institute (WRI) publication "Design of Slab-on-Ground Foundations". The design procedures outlined in the WRI publication are based on the weighted plasticity index of the various soil layers existing within the upper 15 feet of the building site.

The WRI publication states that the weighted plasticity index (P.I.) of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the supporting soil or bedrock materials. Since the buildings will be constructed on level building pads, and in consideration of the estimated unconfined compressive strength of the onsite soils, it is recommended that the weighted plasticity index, as provided herein, be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 9 of the WRI publication). For preliminary design purposes, the project structural engineer may assume an effective plasticity index of 14 for soils with Expansion Index (E.I.) between 21 and 50.

Final foundation design criteria should be made at the completion of grading, based on "as-graded" soil conditions. As such, the footing and slab configuration, and reinforcement recommendations should be considered preliminary and subject to review; pending evaluation of expansive soil characteristics and "as-graded" conditions at the conclusion of grading operations.

<u>Footings</u>

1. The following table presents recommended minimum depths, widths and reinforcement for exterior and interior continuous footings supporting one- and two-story light framed structures.

Continuous Footings – Conventionally-Reinforced Slabs		
Foundation Element	**Expansion Index (E.I.)**	
	E.I. $\leq$ 20[1]	**E.I. 21 – 50**
Exterior Continuous Footing Depth[2] (in.)	12	15
Interior Continuous Footing Depth[3] (in.)	10	12
Width (in.) per 2016 CBC Table 1809.7	12	12
Reinforcement	Two (2) #4 bars; one (1) top, one (1) bottom	Two (2) #4 bars; one (1) top, one (1) bottom
The recommended parameters presented above are applicable to one- and two-story light-frame construction. Additional recommendations can be provided for 3- and 4-story buildings, if necessary. [1] No special foundation design methodology is indicated by the 2016 CBC for soil characterized as having an Expansion Index less than or equal to 20. [2] Depth below lowest adjacent final grade. [3] Depth below top of finish floor.		

2. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in a similar manner as recommended above.

3. The following table presents recommended minimum dimensions, depths, and reinforcement for interior isolated pad footings supporting one- and two-story light framed structures.

Interior Isolated Pad Footings – Conventionally-Reinforced Slabs		
Foundation Element	**Expansion Index (E.I.)**	
	E.I. $\leq$ 20	E.I. 21 - 50
Footing Dimension (in.)	24 x 24	24 x 24
Footing Depth[1] (in.)	12	12
Reinforcement	#4 bars @ 18" o.c. both ways, placed near footing bottom	4 bars @ 18" o.c. both ways, placed near footing bottom
The recommended parameters presented are applicable to one- and two-story light-frame construction. Additional recommendations can be provided for 3- and 4-story buildings, if necessary. [1] Depth below top of finish floor.		

4. The following table presents recommended minimum dimensions, depths, and reinforcement for exterior isolated pad footings supporting roof overhangs such as second-story decks, patio covers and similar appurtenances.

Exterior Isolated Pad Footings – Conventionally-Reinforced Slabs		
Foundation Element	**Expansion Index (E.I.)**	
	E.I. ≤ 20	**E.I. 21 - 50**
Footing Dimension (in.)	24 x 24	24 x 24
Footing Depth[1] (in.)	18	18
Reinforcement[2]	#4 bars @ 18” o.c. both ways, placed near footing bottom	4 bars @ 18” o.c. both ways, placed near footing bottom
The recommended parameters presented are applicable to one- and two-story light-frame construction. Additional recommendations can be provided for 3- and 4-story buildings, if necessary. [1] Depth below lowest adjacent final grade. [2] Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.		

5. The spacing and layout of the interior concrete grade beam system required below floor slabs should be determined by the project architect or structural engineer in accordance with the WRI publication using the effective plasticity index value.

6. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2016 CBC) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

<u>Building Floor Slabs</u>

1. The following table presents recommended minimum thicknesses and reinforcement for concrete floor slabs. Slab dimension, reinforcement type, size and spacing should be designed by the structural engineer/slab designer to account for internal concrete forces that may occur (e.g., thermal, shrinkage and expansion), as well as external forces (e.g., applied loads).

Concrete Floor Slabs - Conventionally-Reinforced			
Slab Element		**Expansion Index (E.I.)**	
		E.I. ≤ 20	**E.I. 21 – 50**
Floor Slab Thickness (in.)		4	4
Reinforcement	Effective. P.I. < 20	#3 bars @ 24” o.c. (max.) both ways	#3 bars @ 18” o.c. (max.) both ways
	Effective P.I. ≥ 20	N/A	N/A
Alternative Reinforcement[1]		6x6/W2.9xW2.9 WWF	6x6/W2.9xW2.9 WWF
N/A – Not applicable. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid-depth. Care should be exercised to prevent warping of the welded wire mesh between the chairs in order to ensure its placement at the desired mid-slab position. [1] If recommended by the structural engineer - welded wire fabric (WWF) sheets only, no rolls.			

2. Living area concrete floor slabs and areas to receive moisture sensitive floor covering should be underlain with a vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). The membrane should be properly lapped and sealed was well as sealed around all plumbing lines and other openings. At least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. It is essential to prevent damage to the moisture retarder membrane. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch thick leveling course of sand across the pad surface prior to the placement of the membrane. Penetration of the membrane with screed guides during concrete placement should be avoided. A 4-inch thick layer of non-expansive sand and gravel should be placed below the vapor retarder membrane on lots where the soil is indicated as having an Expansion Index of 91 or greater.

   At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

3. Garage floor slabs should have the same minimum thickness and reinforcement as living area floor slabs. Garage floor slabs should be poured separately from adjacent wall footings with a positive separation maintained using ¾-inch minimum felt expansion joint material. To aid in reducing the propagation of shrinkage cracks, garage floor slabs should be quartered with weakened plane joints. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that recommended in Item 2 above, should the garage slab be overlain with moisture sensitive floor covering.

4. Prior to placing concrete, the subgrade soils below floor slabs should be pre-watered as recommended in the following table.

Subgrade Moisture Content – Conventionally-Reinforced Slabs		
Parameter	**Expansion Index (E.I.)**	
	E.I. $\leq$ 20	E.I. 21 - 50
Moisture Content (percent of optimum)	100	120
Pre-watering Depth Below Subgrade (in.)	12	12

5. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

6. In addition to the potential effects of expansive soils, the foundations should be designed in consideration of the estimated potential total and differential settlements presented herein.

<u>Post-Tensioned Slab-on-Grade System</u>

Certain assumptions regarding the site environmental condition and the composition of the subsurface soils were made in order to comply with Section 1808.6.2 of the 2016 CBC and the PTI publication. The following table presents soil and environmental parameters for preliminary design of post-tensioned slabs-on-grade based on laboratory testing, engineering analysis, as well as our engineering judgment and experience on similar sites.

Final foundation design criteria should be made at the completion of grading, based on "as-graded" soil conditions. As such, the following soil parameters, footing and slab configuration, and reinforcement recommendations should be considered preliminary and subject to review; pending evaluation of expansive soil characteristics and “as-graded” conditions at the conclusion of grading operations.

Tentative Design Parameters for PTI Procedure

Parameter	**Expansion Index (E.I.)**	
	E.I. ≤ 20*	**E.I. 21 - 50**
Liquid Limit (LL)	N/A	38
Plastic Limit (PL)	N/A	18
Plasticity Index (PI)	N/A	20
Percent Passing No. 200 Sieve (% < #200)	N/A	30
Percent Less than 2 Microns (% < 0.002 mm)	N/A	20
Percent Fine Clay	N/A	20
Expansion Index (EI)	N/A	50
Summary of Design Parameters		
Approximate Depth of Constant Suction, feet	9	9
Approximate Soil Suction, pF	3.9	3.9
Inferred Thornthwaite Index:	-20	-20
Average Edge Moisture Variation Distance, e_m in feet:		
Center Lift	9.0	9.0
Edge Lift	5.1	5.1
Anticipated Swell, y_m in inches:		
Center Lift	0.20	0.25
Edge Lift	0.40	0.60

* Since no special foundation design methodology is indicated by the 2016 CBC for soil characterized as having an Expansion Index less than or equal to 20, any rational and appropriate procedure may be chosen by the project structural engineer for the design of post-tensioned slabs on-ground. Should the design engineer choose to follow the procedures published by the Post-Tensioning Institute (PTI), the above design criteria are provided.

N/A – Not Applicable

Modulus of Subgrade Reaction

The modulus of subgrade reaction for design of load bearing partitions may be assumed to be 125 pounds per cubic inch for soils with either very low (E.I. ≤ 20) or low (E.I. 21 – 50) expansion potential.

Minimum Design Recommendations

The soil values provided above may be utilized by the project structural engineer to design post-tensioned slabs-on-ground in accordance with Section 1808.6.2 of the 2016 CBC and the PTI publication. Thicker floor slabs and larger footing sizes may be required for structural reasons and should govern the design if more restrictive than the minimum recommendations provided below:

1. The following table presents recommended minimum depths, widths and reinforcement for exterior and interior continuous footings supporting one- and two-story light framed structures.

Continuous Footings – Post-Tensioned Slabs		
Foundation Element	**Expansion Index (E.I.)**	
	E.I. ≤ 20	**E.I. 21 - 50**
Perimeter Footing/Thickened Edge Depth[1] (in.)	12	12
Interior Footing Depth[2] (in.)	10	12
Reinforcement[3]	Two (2) #4 bars; one (1) top, one (1) bottom	Two (2) #4 bars; one (1) top, one (1) bottom

The recommended parameters presented above are applicable to one- and two-story light-frame construction. Additional recommendations can be provided for 3- and 4-story buildings, if necessary.

[1] Depth below lowest adjacent final grade.
[2] Depth below top of finish floor.
[3] Alternatively, post-tensioned tendons may be utilized in the perimeter continuous footings in lieu of the reinforcement bars.

2. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in a similar manner as recommended above.

3. The following table presents recommended minimum dimensions, depths, and reinforcement for exterior isolated pad footings supporting roof overhangs such as second-story decks, patio covers and similar appurtenances.

Exterior Isolated Pad Footings – Post-Tensioned Slabs		
Foundation Element	**Expansion Index (E.I)**	
	E.I. ≤ 20	**E.I. 21 - 50**
Footing Dimension (in.)	24 x 24	24 x 24
Footing Depth[1] (in.)	18	18
Reinforcement[2]	#4 bars @ 18" o.c. both ways, placed near footing bottom	4 bars @ 18" o.c. both ways, placed near footing bottom

The recommended parameters presented are applicable to one- and two-story light-frame construction. Additional recommendations can be provided for 3- and 4-story buildings, if necessary.

[1] Depth below lowest adjacent final grade.
[2] Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

4. The thickness of the floor slabs should be determined by the project structural engineer with consideration given to the expansion potential of the onsite soils; however; we recommend that a minimum slab thickness of 4 inches be considered for soils classified as having very low (E.I. ≤ 20) to low (E.I. 21 - 50) expansion potential.

5. As an alternative to designing 4-inch thick post-tensioned slabs with perimeter footings as described in Items 1 and 2 above, the structural engineer may design the foundation system using a thickened slab design. The minimum thickness of this uniformly thick slab should be 8 inches for soils classified as having Very Low (E.I. ≤ 20) or Low (E.I. 21 - 50) expansion potential. The engineer in charge of post-tensioned slab design may also opt to use any combination of slab thickness and footing embedment depth as deemed appropriate based on their engineering experience and judgment.

6. Living area concrete floor slabs and areas to receive moisture sensitive floor covering should be underlain with a vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). The membrane should be properly lapped and sealed was well as sealed around all plumbing lines and other openings. At least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. It is essential to prevent damage to the moisture retarder membrane. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch thick leveling course of sand across the pad surface prior to the placement of the membrane. <u>Penetration of the membrane with screed guides during concrete placement should be avoided</u>. A 4-inch thick layer of non-expansive sand and gravel should be placed below the vapor retarder membrane on lots where the soil is indicated as having an Expansion Index of 91 or greater.

   At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures *uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.*

7. Garage floor slabs should have the same minimum thickness and reinforcement as living area floor slabs. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that recommended in Item 6 above, should the garage slab be overlain with moisture sensitive floor covering.

8. Prior to placing concrete, the subgrade soils below floor slabs should be pre-watered, as necessary, to achieve the recommended minimum values in the following table.

Subgrade Moisture Content – Post-Tensioned Slabs		
Parameter	**Expansion Index (E.I)**	
	E.I. ≤ 20	**E.I. 21 – 50**
Moisture Content (percent of optimum)	100	100
Pre-watering Depth Below Subgrade (in.)	12	12

9. The minimum footing dimensions and foundation design parameters recommended herein are based on our experience, judgement and professional interpretation of the prevailing site soils' characteristics and the inferred site environmental/climatic conditions. At this time, we do not have information regarding potential improvements located within the influence of the foundation system that could impact the foundation's performance. Such improvements may include but are not limited to: adjacent lawn/planter areas and the implemented irrigation regime; trees located within 4 horizontal feet of the foundation; and vertical and/or horizontal moisture barriers. A knowledge of these feature may allow us to perform more refined analysis of the proposed development that may provide for a modification in the design parameters. In the absence of such refined analysis, the minimum dimensions provided herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2016 CBC and PTI DC10.5-12) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

10. In addition to the potential effects of expansive soils, the foundations should be designed in consideration of the estimated potential total and differential settlements presented herein.

<u>General Foundation Design and Construction Considerations</u>

The following apply to both conventionally-reinforced and post-tensioned slabs.

1. Design and construction of the proposed foundations systems should be undertaken by firms that are experienced in this field. It is the responsibility of the foundation design engineer to select the design methodology and properly design the foundation systems for the soils conditions indicated herein. The slab designer should provide deflection potential to the project architect/structural engineer for incorporation into the design of the structure.

2. To reduce moisture penetration beneath the slab-on-grade areas, utility trenches should be backfilled with lean concrete slurry where they intercept the foundation perimeter. As an alternative, the utility trenches can be backfilled with on-site materials compacted to a minimum 90 percent of the applicable maximum dry density.

3. Soil materials from foundation excavations should not be placed on slab-on-grade areas unless it is compacted and tested.

4. All foundation excavations should be observed by a representative of this firm prior to the placement of forms, reinforcement or concrete. The excavations should be free of all loose and sloughed materials, be neatly trimmed and moisture conditioned at the time of concrete placement.

Differential Settlement Design Value

In addition to the potential effects of expansive soils, the proposed structures should be designed in anticipation of differential settlements presented below.

Residential Units

As previously recommended, all unsuitable artificial fill, alluvium, colluvium, and surficial older alluvium, landslide materials and weathered bedrock material will be removed down to either competent bedrock or competent native materials and then replaced as compacted fill. Based on these conditions, post-grading settlements beneath the site will result from settlement of competent native soils to be left in-place due to the weight of new fill materials, settlement of the proposed fills due to their own weight, and settlement of the near-surface soils due to the weight of the new buildings.

Maximum total settlements over a period of 50 years due to long-term settlement of the fill materials are expected to be on the order of less than ¼ inch within shallow fill lots and up to approximately 1 inch within lots underlain by up to approximately 40 feet of fill. A settlement monitoring program is recommended for lots with fill depths in excess of 40 feet (i.e. deep fill lots). A long-term differential settlement on the order of ¾ of an inch over a span of 30 feet has been preliminarily estimated for lots with deeper fills.

Foundation settlement can also occur due to the compression of the near-surface soils due to building loads. Under the recommended maximum allowable bearing capacity, the maximum total footing settlement due to building loads is estimated at ½ of an inch, and maximum differential settlement is estimated at ¼ inch over a span of 30 feet. The majority of this estimated footing settlement will occur during building construction or shortly thereafter as the loads are applied.

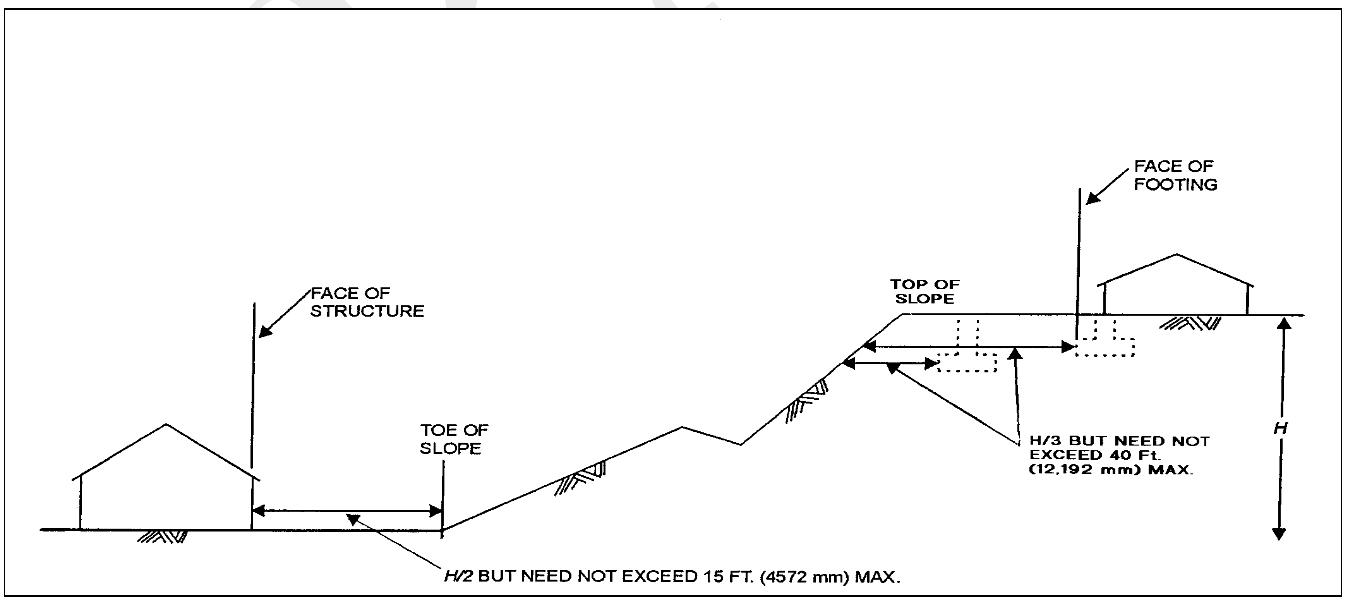
Based on the above discussion, the maximum total settlements within the site due to the combined effects of long-term settlement and new loads are estimated to range from roughly ¾ to 2 inches. Consequently, the maximum differential settlement is estimated to be on the order of 1 inch over a span of 30 feet.

The actual total and differential settlements within each individual lot will vary depending on the depths of fill, the engineering characteristics of the soil and bedrock materials below the fill, and the variations in fill depths across the building pads. Therefore, final settlement estimates for individual lots should be performed by the geotechnical consultant of record on a lot-by-lot basis based on actual as-graded conditions and the proposed on-site improvements.

Structure Setbacks and Deepened Footings

It is generally recognized that improvements constructed in proximity to natural slopes, cut slopes or properly constructed fill slopes may, over a period of time, be affected by natural processes including gravity forces, weathering of surficial soils, and long-term (secondary) settlement. Most building codes, including the CBC, require that structures be set back or footings deepened, where subject to the influence of these natural processes.

For the subject site, where foundations for residential structures are to exist in proximity to slopes, the footings should be embedded to satisfy the requirements presented in the following figure.



Future improvements may be constructed within the setback zone, however, the design and sitting of all such improvements should be reviewed by a qualified soils engineer familiar with hillside grading techniques, in general, and the site-specific conditions reported herein, in particular.

Conventional Retaining Wall Design Recommendations

The following retaining wall recommendations should be incorporated into the project design and specifications.

Allowable Bearing Capacity and Lateral Resistance

Retaining walls may be designed using the allowable bearing capacity and lateral resistance values recommended previously for residential building footings; however, when calculating lateral resistance, the resistance of the upper 6 inches of the soil covering the footings should be ignored in areas where the footings will not be covered with concrete flatwork, or where the thickness of soil cover over the top of the footing is less than 12 inches.

Where the retaining wall footing is constructed above ground sloping away at a 2:1 slope ratio, a reduced passive earth pressure of 150 pounds per square foot, per foot of depth, to a maximum value of 1,500 pounds per square foot should be used. In addition, the lateral resistance should be ignored for the upper portions of the wall footing located within the creep zone.

Active and At-Rest Earth Pressures

As of the date of this report, it is uncertain whether the proposed retaining wall will be backfilled with on-site soils with a Very Low (E.I. ≤ 20) to Medium (E.I. 51 - 90) expansion potential or imported granular materials. For this reason, active and at-rest earth pressures are provided below for both onsite soils and imported granular soils.

1. Onsite Soils Used for Backfill

   Assuming native soils exhibiting Medium expansion potential (E.I. 51 - 90) or less are used for backfill behind retaining walls, active earth pressures equivalent to fluids having densities of 46 and 76 pounds per cubic foot should be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 69 and 110 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system (see Figure RW-1). All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

It should be noted that the above earth pressures are based on a condition where on-site soils with Medium expansion potential (E.I. 51 - 90) are used for backfill. On-site materials selected for use as backfill should be tested by the project geotechnical engineer to evaluate the engineering properties. Final recommendations should be provided by the project geotechnical consultant at the completion of rough grading operations.

2. Imported Sand, Pea Gravel or Rock Used for Wall Backfill

Where sufficient area exists behind the proposed walls, imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, or pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater (see Figures RW-2 and RW-3). For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 45 and 62 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are also for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the recommended active and at-rest earth pressures.

Retaining wall plans should be provided to this firm for review prior to grading and construction phases.

Earthquake Loads on Conventional Retaining Walls

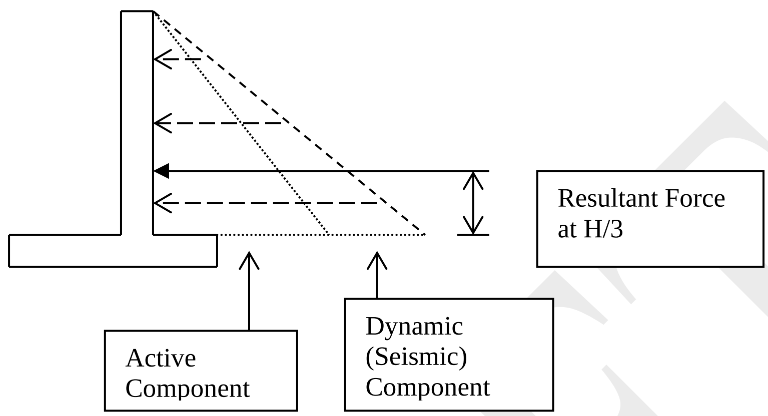
Section 1803.5.12 of the 2016 CBC requires the determination of lateral loads on retaining walls supporting more than 6 feet of backfill height from earthquake forces for structures in seismic design categories D through E. The 2016 CBC allows that the peak ground acceleration (PGA) may be assumed equal to 0.4 x S_{Ds}. This yields a PGA value of 0.692g for this site (0.4 x 1.731g). This value was used in the Seed and Whitman (1970) simplified calculation for level conditions behind retaining structures. According to the research of Sitar, et al. (2012), the simplified Seed and Whitman calculation is appropriate for use for both cantilever retaining walls and restrained basement walls.

The horizontal ground acceleration value K_h for cantilever retaining walls may be assumed to be equal to half of the peak ground acceleration. Thus, $K_h = ½ (a_g) = (0.5)(0.692g) = 0.35g$. From Seed and Whitman (1970), the lateral load on a retaining structure can be determined by the following equation:

$P_D = \Upsilon (¾) K_h$

where P_D = Dynamic Lateral Earth Pressure,
Υ = weight of soil = 125 pcf, and
K_h = horizontal ground acceleration
thus, P_D = (125 pcf) (¾) (0.35) = 32.8 pcf, use 35 pcf.

Sitar, et al. (2012) indicates that the seismic earth pressures have a triangular distribution with the largest load occurring at the bottom of the wall. The distribution of the seismic lateral load for both cantilever and basement types of walls is as follows:



Geotechnical Observation and Testing

Grading associated with retaining wall construction, including backcut excavations, observation of the footing trenches, installation of the subdrainage systems, and placement of backfill should be provided by a representative of the project geotechnical consultant.

Subdrainage

A perforated pipe-and-gravel subdrain should be constructed behind retaining walls exceeding a height of 3 feet (see Figure RW-1). Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or ABS SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of ¾-inch to 1½-inch open-graded gravel. If on-site soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-third the wall height or to a minimum height of 1.5 feet above the footing, whichever is greater. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

For retaining walls not exceeding a height of 3 feet, weepholes or open vertical masonry joints may be considered to reduce the potential for excess water to accumulate in the backfill soils. Weepholes, if used, should be 3-inches minimum diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch intervals. A continuous gravel fill, 3 inches by 12 inches, should be placed behind the weepholes or open masonry joints. The gravel should be wrapped in filter fabric to prevent infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

Damp-Proofing

The backfilled sides of retaining walls should be coated with a damp-proofing compound or covered with a similar material to reduce the potential for moisture migration through the walls.

Temporary Excavations

Temporary slopes may be cut at a gradient no steeper than 1:1 (h:v). However, the project geotechnical engineer should observe temporary slopes for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes.

Conventional Retaining Wall Backfill

Where on-site soils or imported sand are used for backfill, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve optimum or slightly above optimum moisture conditions, and then mechanically compacted in-place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to verify compliance with project specifications. If imported pea gravel or rock is used for backfill, the gravel should be placed in approximately 2- to 3-foot-thick lifts, thoroughly wetted but not flooded, and then mechanically tamped or vibrated into place. A representative of the project geotechnical consultant should observe the backfill procedures and probe the backfill to determine that an adequate degree of compaction is achieved.

To reduce the potential for the direct infiltration of surface water into the backfill, imported sand, gravel or rock backfill should be capped with at least 12 inches of on-site soil. Filter fabric such as Mirafi 140N, or equivalent, should be placed between the soil and the imported gravel or rock to prevent fines from penetrating into the backfill.

MSE Retaining Walls

The following preliminary MSE retaining wall recommendations may be utilized for conceptual design and budgeting purposes. Additional recommendations may be necessary depending on the type of MSE wall system selected.

Strength Parameters

Based on the anticipated soil and geologic conditions, the segmental walls may be preliminarily designed using the following soil parameters. These parameters should be further evaluated at the grading plan review stage and additional recommendations may be necessary.

Preliminary Shear Strength Parameters

MSE Wall Soil Type/Zone	Phi Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
Reinforced Fill	30	0	120
Retained Soil	30	0	120
Foundation Soil	30	225	120

In general, coarser onsite materials are anticipated to be acceptable for use in the various zones. However, a 2-inch particle size is generally the maximum allowable in the reinforced zone. Additional recommendations with regard to grain size distribution, expansion index and plasticity index will be presented at the grading plan review stage.

Wall Construction

Prior to placing the wall units, a minimum of 6 inches of ¾-inch diameter open-graded crushed rock should be spread for the leveling pad below the base of the wall. This leveling pad should have a minimum width of 24 inches. As an alternative to crushed rock, the leveling pad may be constructed using aggregate base materials or 2,000 psi unreinforced concrete. The aggregate base materials should be compacted to a minimum relative compaction of 95 percent. A subdrain should be constructed immediately behind the wall (for closed face wall types) and a backdrain system should be placed against the backcut. Recommendations for the construction of the subdrain and backdrain systems are presented in the following sections.

Following placement of the leveling pad, construction of the subdrain and backdrain, and placement of the lower units of the wall, geogrid placement should begin one block above the bottom of the wall. The minimum geogrid material specifications, geogrid lengths and vertical spacing intervals for each geogrid layer behind the wall should be determined by the wall designer or structural engineer.

In addition, wall corners and radii with tight curves have can cause a third direction of movement resulting in unit cracking and gapping. High quality granular backfill should be considered in these general areas to minimize the potential for cracking and gapping.

Wall Subdrain

For closed face MSE type walls, a wall subdrain should be incorporated into the design. The subdrain behind the wall should consist of 4-inch-diameter perforated Schedule 40 PVC or SDR-35 pipe. The pipe should be installed with the perforations facing down and embedded in 3/4-inch to 1 ½ inch open-graded gravel. Approximately 3 cubic feet of gravel should be provided for each linear foot of perforated subdrain pipe. The entire pipe and gravel subdrain assembly should be completely wrapped in filter fabric consisting of Mirafi 140N or equivalent. A non-perforated outlet pipe consisting of 4-inch-diameter Schedule 40 PVC or equivalent should be routed to a suitable discharge area such as a drainage swale or area drain which will ultimately discharge to the street. A minimum flow gradient of 2 percent should be maintained throughout the subdrain system. A minimum 12-inch-wide column of 3/4-inch, open-graded gravel should be placed above the pipe and gravel subdrain assembly and directly behind the MSE wall units.

Backdrain

In addition to the subdrain located immediately behind the wall, a backdrain system should be constructed against the backcut. The backdrain system should consist of minimum 4-inch diameter, perforated PVC Schedule 40 or ABS SDR-35 perforated pipe (perforations laid down) embedded in a minimum of 3 cubic feet per linear foot of 3/4- to 1 ½ inch-diameter, open-graded gravel. The gravel and pipe should be wrapped in Mirafi 140N geotextile filter fabric or equivalent. The gravel and pipe system should be installed along the bottom of the backcut. At least one non-perforated outlet pipe should be provided for every 100-foot length of perforated backdrain. To reduce the potential for subsurface water to migrate horizontally into the reinforced backfill zone, a drainage column should be constructed against the bottom two-thirds of the backcut. This drainage column may consist of 3/4-inch-diameter open-graded gravel. A layer of Mirafi 140N geotextile filter fabric or equivalent should be placed between the gravel column and the backcut. As an alternative to this gravel system, a composite geotechnical drainage material such as Mirafi G100N or equivalent may be placed against the bottom two-thirds of the backcut. The gravel column or composite drainage materials should be directly connected to the recommended pipe and gravel backdrain system.

Fill Placement

Wall backfill should be placed in lifts no greater than 6 to 8 inches in thickness, watered as necessary to achieve a uniform moisture equal to or slightly greater than optimum moisture content, and then compacted in place to a minimum relative compaction of 90 percent. Subsequent lifts should not be placed until the preceding lift has been approved by the geotechnical consultant. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557.

Other Design and Construction Recommendations

Concrete Flatwork and Lot Improvements

General

Near-surface compacted fill soils within the site are variable in fines content and expansion behavior with an expectation for the majority of these soils to exhibit an expansion potential in the very low to low categories. For this reason, we recommend that additional testing of subgrade soils be performed at the completion of precise grading in order to provide specific recommendations for all exterior concrete flatwork. However, owing to typical project scheduling constraints, it may not be feasible to collect additional samples of subgrade soils for testing to verify their expansion characteristics in a timely manner; i.e., immediately prior to pouring concrete. As such, we recommend that all exterior concrete flatwork such as sidewalks, patio slabs, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private and/or public vehicular parking, driveways and/or access roads within and adjacent to the site be designed by the project architect, civil and/or structural engineer with consideration given to mitigating the potential cracking, curling, uplift, etc. that can develop in soils exhibiting expansion index values that fall in the upper ranges of the values provided above.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project architect, civil engineer, structural engineer and/or landscape consultant as deemed appropriate. If sufficient time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results may dictate that a somewhat less conservative design could be used.

Subgrade Preparation

Compaction

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent. Where concrete public roads, concrete segments of roads and/or concrete access driveways and heavy recreational vehicles parking are proposed, the upper 6 inches of subgrade soil should be compacted to a minimum 95 percent relative compaction.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork distress, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning may be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Private driveways that will be designed for the use of passenger cars for access to private garages should also be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load in public access areas, segments of road that will be paved with concrete (such as bus stops and cross-walks) or access roads and driveways, which serve multiple residential units or garages, that will be subject to heavy truck loadings and recreational vehicles parking should have a minimum thickness of 5 inches and be provided with control joints spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design of the public and access roads.

Reinforcement

All concrete flatwork having their largest plan-view panel dimensions exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 18 inches for 4-inch-thick slabs and No. 4 bars spaced 24 inches for 5-inch-thick slabs on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designations for 4-inch-thick slabs and 6x6/W2.9xW2.9 designations for 5-inch-thick slabs in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs. All foot and equipment traffic on the reinforcement should be avoided or reduced to a minimum.

The reinforcement recommendations provided herein are intended as a guideline to achieve adequate performance for anticipated soil conditions. As such, this guideline may not satisfy certain acceptable approaches, e.g. the area of reinforcement to be equal to or greater that 0.2 percent of the area of concrete. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.

Edge Beams (Optional)

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas because they typically introduce excessive water into the subgrade soils and allow root invasion, both of which can cause heaving and cracking of the flatwork.

Utility Trench Excavation

All trenches should be shored or laid back in accordance with applicable OSHA Standards. Excavations should be constructed at slope ratios no steeper than 1:1 (horizontal to vertical) unless properly shored. No surcharge loads are permitted above unshored or unretained excavations.

This includes, but is not limited to, spoil piles, lumber, traffic, concrete blocks or other materials or construction equipment.

Precautions should be taken to prevent water from flowing into open excavations. Temporary provisions should be made at all times to adequately direct surface drainage from all sources away from the excavations.

The project geotechnical engineer, or his/her representative, should observe the bottom of the trench excavation, prior to installation/construction of subsurface lines to observe that the improvements are supported on competent material. If materials, which are not competent are encountered, the excavation should be deepened until competent materials are reached. The deepened excavation may then be filled with compacted soil until the desired bottom elevation is achieved.

Utility Trench Backfill

Trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM Test Method D 1557. Bedding materials should have an Expansion Index value of 20 or less and a Sand Equivalent Value of 30 or greater. Onsite soils will not be suitable for use as bedding but will be suitable for use as backfill provided oversized materials are removed. Compaction should be accomplished by mechanical means. Jetting of native soils will not be acceptable.

Temporary Excavation Considerations

Temporary excavations should be sloped or braced in accordance with CAL-OSHA/FED-OSHA regulations.

No surcharge loads are permitted above unshored or unretained excavations. This includes, but is not limited to, earth spoil piles, lumber, concrete trucks or other vehicles, concrete blocks, or other construction materials or construction equipment. Drainage above excavation should be directed away from the banks. Care should be taken to prevent saturation of the soils.

SLOPE AND LOT MAINTENANCE

The following sections briefly highlight slope and lot maintenance responsibilities incumbent upon the resident or owner in a hillside development. Further discussion of these and other issues is presented in the Homeowner's Maintenance and Improvement Considerations portion of the Appendix III.

Slope Planting

Slope planting should consist of ground cover, shrubs, and trees that possess deep dense root structures and require a minimum of irrigation. It is the responsibility of the resident to maintain such planting.

Slope Irrigation

The resident or owner is responsible for installation of proper irrigation systems, as well as maintenance and repair of such systems. Leaks should be repaired immediately. Sprinklers should be adjusted to provide maximum uniform coverage with a minimum of water usage and overlap. Overwatering with consequent wasteful runoff and serious ground saturation must be avoided. If automatic sprinkler systems are installed their use must be adjusted to account for natural rainfall conditions.

Lot Drainage

Design fine grade elevations should be maintained throughout the life of the structure. If design fine grade elevations are altered, adequate area drains should be installed in order to provide rapid discharge of water, away from the structures and slope areas. Surface drainage away from footings must be maintained.

Burrowing Animals

Residents or owners should undertake a program for the elimination of burrowing animals. This should be an ongoing program in order to maintain slope stability.

FUTURE PLAN REVIEW

This report represents a geotechnical review of the 80-scale Conceptual Grading Plan for Tentative Tract Map No. 82349. As the project design progresses, site specific geologic and geotechnical issues need to be incorporated into design and construction of the project. Consequently, future plan reviews will be necessary. These reviews may include evaluations of:

- Mass grading plan
- Precise grading plans
- Foundation plans
- Conventional retaining wall plans
- MSE wall plans

These plans should be forwarded to the project geotechnical engineer/engineering geologist for evaluation and comment, as necessary.

CLOSURE/REPORT LIMITATIONS

This report is based on the proposed project and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soils can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the construction phase of the project are essential to confirming the basis of this report. To provide the greatest degree of continuity between the design and construction phases, consideration should be given to retaining Petra for construction services.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and in the same time period. This report provides our professional opinions and as such, they are not to be considered a guaranty or warranty.

This report should be reviewed and updated after a period of one year or if the site conditions, ownership or project concept changes from that described herein. This report has not been prepared for use by parties or projects other than those named or described herein and may not contain sufficient information for other parties or other purposes.

This opportunity to be of service is sincerely appreciated. Please call if you have any questions pertaining to this report.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Ronald A. Reed Senior Associate Engineer GE 2524	Theodore M. Wolfe Senior Associate Geologist CEG 1626

AM/RAR/TMW/lv

W:\2014-2019\2017\200\17-219 Nevis Capital, LLC (Chadwick Ranch Project, Bradbury)\Reports\17-219 110 Tentative Plan Review Report.docx

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APPENDIX I

EXPLORATION LOGS AND SEISMIC REFRACTION STUDY

SITE LOCATION MAP – FIGURE 1

REGIONAL FAULT ACTIVITY MAP – FIGURE 2

GEOTECHNICAL MAP – PLATE 1

GEOTECHNICAL CROSS SECTIONS – PLATE 2

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**TP-1**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**818'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/10/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Clayey Sand (SC): Dark reddish-brown, slightly moist, fine- to coarse-grained sand, subangular cobbles up to 18-inch-diameter.				■			
5									
10		**SAN DIMAS FORMATION (Qsd)** Clayey Sandy Gravel (GC): Reddish-brown, slightly moist, very dense, coarse-grained sand, poorly graded, many subangular cobbles up to 18-inch-diameter.				■			
		Clayey Sand (SC): Yellowish-brown, moist, dense, medium- to coarse-grained sand, few gravel sized bedrock clasts.							
		Total Depth = 12.5' No Groundwater Encountered Backfilled with on-site soils 8/10/17.							
15									
20									
25									
30									
35									

PLATE

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EXPLORATION LOG

Project:	CHADWICK RANCH	Boring No.:	TP-2
Location:	BRADBURY, CALIFORNIA	Elevation:	837'
Job No.: 17-219	Client: NEVIS CAPITAL, LLC	Date:	8/10/17
Drill Method: Mini Track Excavator	Driving Weight: N/A	Logged By:	Evan Price

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Clayey Silty Sand (SC-SM): Dark brown, dry, loose, fine- to coarse-grained sand, many gravel, few subangular cobbles up to 6-inch-diameter, many roots.							
		SAN DIMAS FORMATION (Qsd) Clayey Sand (SC): Reddish-brown, moist, dense, medium- to coarse-grained sand, many fine-grained gravel, few subangular cobbles up to 6-inch-diameter.				■			
5									
		@8:' Decrease in cobbles to trace.							
10									
		Total Depth = 13.0' No Groundwater Encountered Backfilled with on-site soils 8/10/17.							
15									
20									
25									
30									
35									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	CHADWICK RANCH	Boring No.:	TP-3
Location:	BRADBURY, CALIFORNIA	Elevation:	838'
Job No.:	17-219 Client: NEVIS CAPITAL, LLC	Date:	8/10/17
Drill Method:	Mini Track Excavator Driving Weight: N/A	Logged By:	Evan Price

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Clayey Silty Sand (SC-SM): Dark brown, dry, loose, fine- to coarse-grained sand, many gravel, few subangular cobbles up to 6-inch-diameter, many roots.							
5		**SAN DIMAS FORMATION (Qsd)** Clayey Sand (SC): Reddish-brown, moist, dense, medium- to coarse-grained sand, many gravel and subangular cobbles up to 32-inch-diameter.				■			
10		@11': Orangish-brown, decrease in cobbles to few.							
		Total Depth = 12.0' No Groundwater Encountered Backfilled with on-site soils 8/10/17.							
15									
20									
25									
30									
35									

PLATE

Petra Geosciences, Inc.

E X P L O R A T I O N L O G

Project: **CHADWICK RANCH**		Boring No.:	**TP-4**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**816'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/10/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Clayey Silty Sand (SC-SM): Dark brown, dry, loose, fine- to coarse-grained sand, many gravel, few subangular cobbles up to 6-inch-diameter, many roots.							
		SAN DIMAS FORMATION (Qsd) Silty Sand (SM): Reddish-brown, moist, dense, medium- to coarse-grained sand, many fine-grained gravel, few subangular cobbles up to 18-inch-diameter.				■			
5		@6': Decrease in cobbles to trace.							
10									
		Total Depth = 11.0' No Groundwater Encountered Backfilled with on-site soils 8/10/17.							
15									
20									
25									
30									
35									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**TP-5**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**885'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/11/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**SLOPEWASH (Qsw)** Sandy Clay (CL): Dark reddish-brown, dry, very stiff, coarse-grained sand, few gravel, trace pinhole porosity, rootlets throughout.				■			
		SAN DIMAS FORMATION (Qsd) Clayey Sand (SC): Reddish-brown, slightly moist, very dense, medium- to coarse-grained sand, few gravel, trace cobbles consisting of local granitic bedrock clasts.							
5									
10		Total Depth = 9.0' No Groundwater Encountered Backfilled with on-site soils 8/11/17.							
15									
20									
25									
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35									

PLATE

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project: CHADWICK RANCH		Boring No.:	TP-6
Location: BRADBURY, CALIFORNIA		Elevation:	945'
Job No.: 17-219	Client: NEVIS CAPITAL, LLC	Date:	8/11/17
Drill Method: Mini Track Excavator	Driving Weight: N/A	Logged By:	Evan Price

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**SAN DIMAS FORMATION (Qsd)** Clayey Sand (SC): Reddish-brown, dry, very dense, coarse-grained sand, trace gravel.							
5						■			
10									
		Total Depth = 12.5' No Groundwater Encountered Backfilled with on-site soils 8/11/17.							
15									
20									
25									
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35									

PLATE

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EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**TP-7**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**923'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/11/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**TOPSOIL** Sandy Clay (CL): Light brown, dry, soft, many roots.							
		SAN DIMAS FORMATION (Qsd) Clayey Sand (SC): Reddish-brown, slightly moist, dense, fine- to coarse-grained sand, trace gravel, trace pinhole porosity, some layers grade to sandy clay.							
5									
10									
		Total Depth = 10.5' No Groundwater Encountered Backfilled with on-site soils 8/11/17.							
15									
20									
25									
30									
35									

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EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**TP-8**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**838'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/11/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Sandy Clay (CL): Dark brown, dry, soft, fine- to coarse-grained sand, many gravel, few subangular cobbles, many roots. Clayey Sand (SC): Dark brown, slightly moist, dense, fine- to coarse-grained sand, many fine-grained gravel, few subangular cobbles up to 6-inch-diameter. @3.5': Reddish-brown, few cobbles.							
5									
10		**SAN DIMAS FORMATION (Qsd)** Clayey Sand (SC): Dark yellowish-brown, moist, medium dense, coarse-grained sand, many gravel sized local granitic bedrock clasts, trace cobbles, few rootcasts and rootlets.				■			
		Total Depth = 10.5' No Groundwater Encountered Backfilled with on-site soils 8/11/17.							
15									
20									
25									
30									
35									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**TP-9**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**807'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**8/11/17**
Drill Method: **Mini Track Excavator**	Driving Weight: **N/A**	Logged By:	**Evan Price**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Sandy Clay (CL): Brown, slightly moist, stiff, fine- to coarse-grained sand, few gravel and subangular cobbles up to 12-inch-diameter, pinhole porosity.							
5						■			
10		**SAN DIMAS FORMATION (Qsd)** Clayey Sand (SC): Dark yellowish-brown, moist, medium dense, coarse-grained sand, many gravel sized local granitic bedrock clasts, trace cobbles, few rootcasts and rootlets.							
		Total Depth = 11.5' No Groundwater Encountered Backfilled with on-site soils 8/11/17.							
15									
20									
25									
30									
35									

PLATE

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EXPLORATION LOG

Project:	**CHADWICK RANCH**	Boring No.:	**B-1**
Location:	**BRADBURY, CALIFORNIA**	Elevation:	**±1170'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**7/8/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**TOPSOIL** Sand (SP): Light brown, dry, loose, medium- to coarse-grained sand, poorly graded.							
		BEDROCK - Quartz Diorite (Qd) Granite: highly weathered.							
5		@5': becomes dense.		8	■				
		@6': becomes reddish brown, blocky, aphanitic.							
		slightly moist, @8': becomes slightly moist to moist, phaneritic.							
		@9': quartz vein N55W, 50N.							
10		@10': Quartz Diorite(Qd), moderately weathered,some jointing.		13	■				
15		@15': less weathered.		17	■				
20									
		@22': becomes blocky, fractured.							
25		@25': joint - N70W, 65N.		27	■				
		@28': becomes hard, massive.							
30									
35		@35': too hard to sample.							

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	CHADWICK RANCH		Boring No.:	**B-1**
Location:	**BRADBURY, CALIFORNIA**		Elevation:	**±1170'**
Job No.:	**17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**7/8/17**
Drill Method:	**Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@36': massive, becomes hard to very hard.							
40									
		@41' joint - N80E, 50N.							
45									
		@47': refusal.							
		Total Depth 47 feet No Water Backfilled with cuttings.							
50									
55									
60									
65									
70									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**B-2**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**±1180'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**7/8/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**BEDROCK - Quartz Diorite (Qd)** Granitic, Quartz Diorite and Andesite: tan to light brown, dry, medium- to coarse-grained sand, highly weathered.							
5		@5': becomes slightly moist.		5	■				
		@7': becomes hard, less weathered.							
10		@10': too hard to sample.							
15				15	■				
		@18': Gneiss lens (coarser grained).							
20									
		@23': blocky fractures.							
25				22	■				
		@27': massive.							
30									
35				27	■				

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**CHADWICK RANCH**	Boring No.:	**B-2**
Location:	**BRADBURY, CALIFORNIA**	Elevation:	**±1180'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**7/8/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@36': massive.			■				
40		@40': slight fractures.							
45		@45': becomes hard to very hard.							
50		Total Depth 50 feet No Water Backfilled with cuttings.							
55									
60									
65									
70									

PLATE

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**B-3**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**±900'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/11/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**BEDROCK - San Dimas Formation (Qsd)** Clayey Sand (SC): Reddish-brown, dry to slightly moist, medium dense, fine- to coarse-grained sand, with fine gravel up to 0.5" in diameter.							
5		@5': occasional pebbles.		7	■				
10				12	■				
15				15	■				
		@16': coarsens, pebble/cobble conglomerate.							
20		@20': massive, slight coarsening downward.							
25				20	■				
		@28': blocky fractures.							
30									
		@31': occasional finer grained beds.							
35		@35': very little gravel.		22	■				

PLATE

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project: **CHADWICK RANCH**		Boring No.:	**B-3**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**±900'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/11/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lithology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@37': pebbles and cobbles to 4".							
40		@40': slight decrease in grain size, increase in moisture.							
45		massive.		30	■				
50									
55		@55': Approximately flat contact; medium grained clayey sand, dark reddish brown, moist to damp, slightly stiff.		35	■				
60									
65		Total Depth 65 feet No Water Backfilled with cuttings.							
70									

PLATE

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project:	**CHADWICK RANCH**	Boring No.:	**B-4**
Location:	**BRADBURY, CALIFORNIA**	Elevation:	**±938'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/11/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**BEDROCK - San Dimas Formation (Qsd)** Clayey Sand (SC): Reddish-brown, dry to slightly moist, medium dense, fine- to coarse-grained sand.							
5				6					
		slight fining and coarsening sequences.							
10				11					
		occasional pebbles to 2".							
15									
20									
		BEDROCK - Quartz Diorite (Qd) Granitic: Light reddish-brown, slightly moist, coarse-grained sand, moderately hard, thickly bedded to massive, @24': Approximate contact N50W, 51S.							
25				13					
		@28': becomes hard.							
30		@30': too hard to sample.							
		@32': blocky fractures.							
35		@35': softens slightly.		22					

PLATE

EXPLORATION LOG

Project: **CHADWICK RANCH**		Boring No.:	**B-4**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**±938'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/11/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
40		@40': hardens, finer grained, aphanitic.							
45									
50		@50': very slow drilling.							
		Total Depth 51 feet No Water Backfilled with cuttings.							
55									
60									
65									
70									

PLATE

EXPLORATION LOG

Project:	CHADWICK RANCH	Boring No.:	B-5
Location:	BRADBURY, CALIFORNIA	Elevation:	±962'
Job No.:	17-219 Client: NEVIS CAPITAL, LLC	Date:	9/11/17
Drill Method:	Lo-Dril w/ 24" auger Driving Weight: NA	Logged By:	KTM

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**BEDROCK - San Dimas Formation (Qsd)** Clayey Sand (SC): Reddish-brown, dry to slightly moist, medium dense, fine- to coarse-grained sand.							
5		occasional pebbles to 3".		5	■				
10		coarsening downward.		10	■				
15		massive.							
20				20	■				
25		**BEDROCK - Quartz Diorite (Qd)** Granitic: Tan to light orangish gray, coarse-grained sand, moderately hard, with few fine gravel up to 0.5" in diameter.							
30		@30': same as above.		16	■				
		@33': blocky fractures.							
35									

PLATE

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project: **CHADWICK RANCH**		Boring No.:	**B-5**
Location: **BRADBURY, CALIFORNIA**		Elevation:	**±962'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/11/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		massive.							
40									
		becomes hard to very hard.							
45									
50		Total Depth 50 feet No Water Backfilled with cuttings.							
55									
60									
65									
70									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	CHADWICK RANCH		Boring No.:	B-6
Location:	BRADBURY, CALIFORNIA		Elevation:	±803'
Job No.:	17-219	Client: NEVIS CAPITAL, LLC	Date:	9/12/17
Drill Method:	Lo-Dril w/ 24" auger	Driving Weight: NA	Logged By:	KTM

Depth (Feet)	Lithology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Silty Sand (SM): Gray to light brown, dry, loose, fine- to coarse-grained sand, abundant cobbles up to 8" in diameter, high caving potential.							
5									
6				7	■				
9				9	■				
10									
12		**BEDROCK - San Dimas Formation (Qsd)** Clayey Sand (SC): Reddish-brown, slightly moist, medium dense, medium- to coarse-grained sand, with coarse gravel up to 3" in diameter.							
14–18						■			
15									
20		@20': becomes very moist.		20	■				
25									
29		@29': groundwater/seepage encountered.							
30									
35									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**CHADWICK RANCH**	Boring No.:	**B-6**
Location:	**BRADBURY, CALIFORNIA**	Elevation:	**±803'**
Job No.: **17-219**	Client: **NEVIS CAPITAL, LLC**	Date:	**9/12/17**
Drill Method: **Lo-Dril w/ 24" auger**	Driving Weight: **NA**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Total Depth 38 feet Water @ 29 feet caving Backfilled with cuttings.							
40									
45									
50									
55									
60									
65									
70									

EXPLORATION LOG

Project:	CHADWICK RANCH	Boring No.:	B-7
Location:	BRADBURY, CALIFORNIA	Elevation:	±815'
Job No.: 17-219	Client: NEVIS CAPITAL, LLC	Date:	9/12/17
Drill Method: Lo-Dril w/ 24" auger	Driving Weight: NA	Logged By:	KTM

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**BEDROCK - San Dimas Formation (Qsd)** Clayey Sand (SC): Reddish-brown, dry to slightly moist, medium dense, coarse-grained sand, with fine gravel up to 0.25" in diameter.							
5		coarsens downward.			■				
10		slightly moist.			■				
15		finer grained, silty/sandy clay.			■				
20		occasional pebbles, coarsens to gravelly sand.			■				
22–27						■			
25									
30		@31': becomes wet.							
		@32': groundwater encountered.							
		@34': refusal.							
35		Total Depth 34 feet							

PLATE

EXPLORATION LOG

Project:	CHADWICK RANCH	Boring No.:	B-7
Location:	BRADBURY, CALIFORNIA	Elevation:	±815'
Job No.:	17-219 Client: NEVIS CAPITAL, LLC	Date:	9/12/17
Drill Method:	Lo-Dril w/ 24" auger Driving Weight: NA	Logged By:	KTM

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Water @ 32 feet caving Backfilled with cuttings.							
40									
45									
50									
55									
60									
65									
70									

PLATE

Petra Geosciences, Inc.

TERRA GEOSCIENCES

SEISMIC REFRACTION SURVEY

CHADWICK RANCH PROJECT

BRADBURY, LOS ANGELES COUNTY, CALIFORNIA

Project No. 173000-1

August 17, 2017

Prepared for:

Petra Geosciences, Inc.
28358 Constellation Road, Unit 680
Valencia, CA 91335

Petra Geosciences, Inc.
28358 Constellation Road, Unit 680
Valencia, CA 91335

August 17, 2017
Project No. 173000-1

Attention: Mr. Ted Wolfe, Vice President

Regarding: Seismic Refraction Survey
Chadwick Ranch Project
Bradbury, Los Angeles County, California
Petra Project No. 17-219

EXECUTIVE SUMMARY

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high velocity granitic bedrock (non-rippable) may be present. Additionally, the structure and seismic velocity distribution of the subsurface earth materials was also assessed. This report will describe in further detail the procedures used and the results of our findings, along with presentation of representative seismic models for each survey traverse.

For this study, four survey traverses (Seismic Lines S-1 through S-4) were performed across the subject site, as selected by your office. The traverses were located in the field by use of Google™ Earth imagery (2017), the provided topographic maps, and GPS coordinates. The approximate locations of our seismic traverses are presented on a captured Google™ Earth (2017) image, as presented on the Seismic Line Location Map (Overview), Plate 1, with detailed site location maps provided on Plate 2, using partial copies of the provided topographic maps, prepared by Don Read Corporation, Brea, California, dated May 5, 2017.

This opportunity to be of service is sincerely appreciated. If you should have questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

PROFESSIONAL GEOPHYSICIST
DONN C.
SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

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INTRODUCTION

The subject property (approximated in black outline on Figure 1 below) is generally located along the southern flank of the San Gabriel Mountains, in the Bradbury area of Los Angeles County, California. At this time, we understand that a proposed residential development will be developed, with grading to consist of excavations as deep as 87± feet (provided Cut/Fill Diagram).

Surficial geologic mapping by Morton (1973), as shown on Figure 1 below, indicates the subject property in the north to be predominantly underlain by Cretaceous age granitic rocks, which consist of a mixture of massive to foliated quartz diorite rock, granodioritic-to-granitic rock, and light-colored quartzo-feldspathic gneiss (map symbol qd_1). These rocks are described as being well- to intensely-fractured, locally sheared, and deeply weathered. Along the south, the site is mantled by dissected older alluvial fan deposits (Pleistocene age), locally referred to as the San Dimas Formation, presumably underlain at depth by the granitic rocks as described above. These deposits are generally described as consisting of gravel, sand, silt, and clay, which is poorly consolidated and moderately to slightly decomposed (map symbol Qsd).

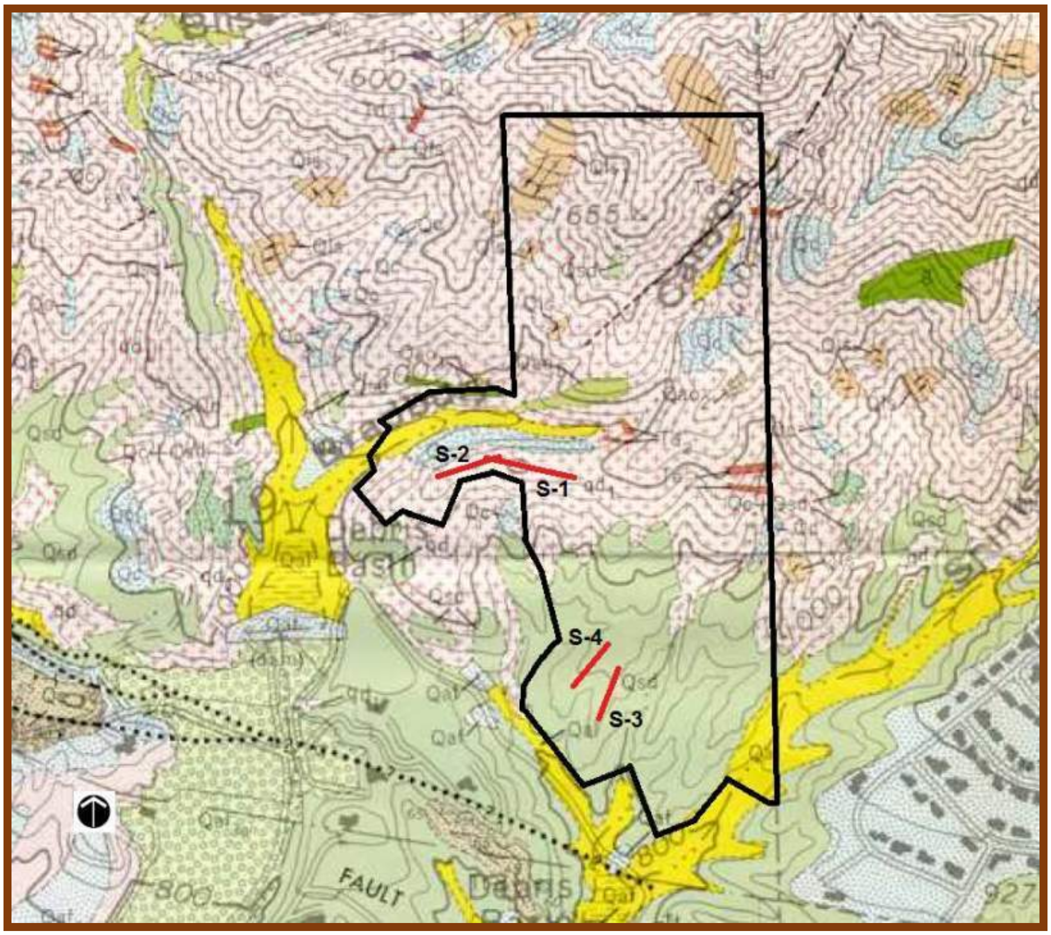


FIGURE 1- Geologic Map (Morton, 1973); seismic traverses shown as red lines.

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Four seismic refraction survey lines (Seismic Lines S-1 through S-4) have been performed along representative areas across the subject site as selected by you. The traverses were located in the field by use of Google™ Earth imagery (2017), GPS coordinates, and the provided topographic maps, and have been delineated on the Seismic Line Location Maps, as presented on Plates 1 and 2. Seismic Lines S-2 through S-4 each consisted of a total of twenty-four 14-Hertz geophones, spaced at regular 12-foot intervals (total length 300 feet), in order to detect both the direct and refracted waves. Seismic Line S-1 consisted of overlapping of two individual spreads (each with 24, 14-hertz geophones), using 10-foot spacings (total length 430 feet), with six overlapped geophones in between the two spreads. A 16-pound sledge-hammer was used as the energy source to produce the seismic waves.

Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. Multiple hammer impacts were utilized at each shot point location in order to increase the signal to noise ratio, which enhanced the primary seismic “P”-waves. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.12 seconds. No acquisition filters were used during data collection. During acquisition, the seismograph displays the seismic wave arrivals on the computer screen which were used to analyze the arrival time of the primary seismic “P”-waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. Each geophone and seismic shot location was surveyed using a hand level and ruler for topographic correction, with “0” being the lowest point along each survey line.

Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2017); and **Rayfract™** (Intelligent Resources, Inc., 1996-2017). All of the computer programs perform their individual analyses using exactly the same input data, which includes the first-arrival times of the “P”-waves and the survey line geometry.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the “Seismic Refraction Modeling by Computer” method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of six times to improve the accuracy of depths to the top of layer-2. This first-arrival picks were then used to generate the Layer Velocity Models using the **SIPwin** computer program, which presents the subsurface velocities as individual layers and are presented within Appendix A for reference. In addition, the associated Time-Distance Plot for each survey line, which shows the individual data picks of the first “P-wave” arrival times, also appears in Appendix A.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the ABC method, and the Generalized Reciprocal Method (GRM). These methods are used for defining irregular non-planar refractors and are briefly described below. The Delay-Time method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay-time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The ABC (intercept time) method makes use of critically refracted rays converging on a common surface position. This method involves using three surface to surface travel times between three geophones and the velocity of the first layer in an equation to calculate depth under the central geophone and is applied to all other geophones on the survey line. The GRM method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

- **Rayfract™** is seismic refraction tomography software that model's subsurface refraction, transmission, and diffraction of acoustic waves which generally indicates the relative structure and velocity distribution of the subsurface using first break energy propagation modeling. An initial 1D gradient model is created using the DeltatV method (Gebrande and Miller, 1985) which gives a good initial fit between modeled and picked first breaks. The DeltatV method is a turning-ray inversion method which delivers continuous depth vs. velocity profiles for all profile stations. These profiles consist of horizontal inline offset, depth, and velocity triples. The method handles real-life geological conditions such as velocity gradients, linear increasing of velocity with depth, velocity inversions, pinched-out layers and outcrops, and faults and local velocity anomalies. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion (Schuster and Quintus-Bosz, 1993).

  WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break. This computer program performs the analysis by using the same first-arrival P-wave times and survey line geometry that were generated during the initial layer velocity model analyses. The associated Refraction Tomographic Models which display the subsurface earth material velocity structure, is represented by the velocity contours (isolines shown in feet/second), supplemented with the color-coded velocity shading for visual reference, as presented within Appendix B.

The combined use of these seismic refraction computer programs provided a more thorough and comprehensive analysis of the subsurface structure and velocity characteristics. Each computer program has a specific purpose based on the objective of the analysis being performed. **SIPwin** and **Refractor** were primarily used for detecting generalized subsurface velocity layers providing "weighted average velocities." The processed seismic data of these two programs were compared and averaged to provide a final composite layer velocity model which provided a more thorough representation of the subsurface. **Rayfract™** provided tomographic velocity and structural imaging that is very conducive to detecting strong lateral velocity characteristics such as imaging corestones, dikes, velocity gradients, etc.

SUMMARY OF GEOPHYSICAL INTERPRETATION

It is important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, relic bedding, etc.), creating anisotropic conditions. Anisotropy (direction-dependent properties of materials) can be caused by "micro-cracks," jointing, foliation, layered or inter-bedded rocks with unequal layer stiffness, small-scale lithologic changes, etc. (Barton, 2007). Velocity anisotropy complicates interpretation and it should be noted that the seismic velocities obtained during this survey may have been influenced by the nature and character of any localized structural discontinuities within the bedrock underlying the subject site. Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Such variable directions can result in velocity differentials of between 2% to 40% depending upon the degree of the structural fabric (i.e., weakly-moderately-strongly foliated, respectively).

The first computer method described below used for data analysis is the traditional layer method (**SIPwin** and **Refractor**). Using this method, it should be understood that the data obtained represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders, dikes, or other local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

In general, the site where locally surveyed, was noted to be characterized by three major subsurface layers (Layers V1, V2, and V3, see Appendix A) with respect to seismic velocities. The following velocity layer summaries have been prepared with respect to the **SIPwin** and **Refractor** analysis, with the representative Layer Velocity Models being presented within Appendix A, along with their respective Time-Distance Plots for reference.

- **Velocity Layer V1**:

  The upper layer (V1) yielded a seismic velocity range of 1,261 to 1,963 fps, which is typical for near-surface unconsolidated surficial earth materials in the southern California region. These materials are most likely comprised of topsoil, colluvium, older alluvial deposits, and/or completely-weathered and fractured bedrock materials.

- **Velocity Layer V2**:

  The second layer (V2) yielded a velocity range of 2,597 to 3,240 fps, which is typical for both highly-weathered granitic bedrock and/or indurated older alluvial sediments (San Dimas Formation). For granitic rock, this velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively-fresher boulders within a highly decomposed bedrock matrix.

- **Velocity Layer V3:**

  The third layer (V3) indicates the presence of moderately-weathered bedrock, having a seismic velocity range of 6,653 to 8,111 fps. These higher velocities signify the decreasing effect of weathering as a function of depth and could indicate the presence of abundant widely-scattered buried fresh large crystalline boulders in highly-weathered matrix, or possibly a slightly-weathered to fresher crystalline bedrock matrix, that has a wide-spaced fracture system.

The following table summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer, as indicated on the Layer Velocity Models, presented within Appendix A.

TABLE 1- VELOCITY SUMMARY OF SEISMIC SURVEY LINES

Seismic Line	V1 Layer (fps)	V2 Layer (fps)	V3 Layer (fps)
S-1	**1,350**	**2,597**	**8,111**
S-2	**1,261**	**2,815**	**6,653**
S-3	**1,963**	**3,231**	**7,363**
S-4	**1,587**	**3,240**	**7,062**

Using **Rayfract™**, tomographic models were also prepared for comparative purposes to better illustrate the general structure and velocity distribution of the subsurface, using velocity contour isolines, as presented within Appendix B. Although no discrete velocity layers or boundaries are created, these models generally resemble the corresponding overall average layer velocities as presented within Appendix A. In general, the seismic velocity of the bedrock gradually increases with depth, with observable lateral velocity differentials suggesting the local presence of weathering differentials, buried corestones, and/or dike structures. The colors representing the velocity gradients have been standardized on all of the models for comparative purposes.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

A summary of the generalized rippability characteristics of bedrock based on a compilation of rippability performance charts prepared by Caterpillar, Inc. (2017; see Figure 2, Page 8), Caltrans (Stephens, 1978), and Santi (2006), has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. These seismic velocity ranges and rippability potentials have been tabulated below for reference.

TABLE 2- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

Granitic Rock Velocity	**Rippability**
< 6,800	Rippable
6,800 – 8,000	Moderately Rippable
> 8,000	Non-Rippable

Additionally, we have provided the Caltrans Rippability Chart as presented below within Table 3 for comparison. These values are from published Caltrans studies (Stephens, 1978) that are based on their experience and which appear to be more conservative than Caterpillar's rippability charts. It should be noted that the type of bedrock was not indicated.

TABLE 3- STANDARD CALTRANS RIPPABILITY CHART

Velocity (feet/sec ±)	Rippability
< 3,500	Easily Ripped
3,500 – 5,000	Moderately Difficult
5,000 – 6,600	Difficult Ripping / Light Blasting
> 6,600	Blasting Required

Table 4 is partially modified from the "Engineering Behavior from Weathering Grade" as presented by Santi (2006), which also provides velocity ranges with respect to rippability potentials, along with other rock engineering properties that may be pertinent.

TABLE 4- SUMMARY OF ROCK ENGINEERING PROPERTIES

ENGINEERING PROPERTY:	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely Weathered
Excavatability	Blasting necessary	Blasting to rippable	Generally rippable	Rippable
Slope Stability	½ :1 to 1:1 (H:V)	1:1 (H:V)	1:1 to 1.5:1 (H:V)	1.5:1 to 2:1 (H:V)
Schmidt Hammer Value	51 – 56	37 – 48	12 – 21	5 – 20
Seismic Velocity (fps)	8,200 – 13,125	5,000 – 10,000	3,300 – 6,600	1,650 – 3,300

For purposes of the discussion in this report with respect to the expected bedrock rippability characteristics, we are assuming that a D9R/D9T dozer will be used as a minimum, such as discussed further below. Smaller excavating equipment will most likely result in slower production rates and possible refusal within relatively lower velocity bedrock materials. It should be noted that the decision for blasting of bedrock materials for facilitating the excavation process is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

The Caterpillar D9R Ripper Performance Chart (Caterpillar, 2017) has been provided on Figure 2 below for reference.

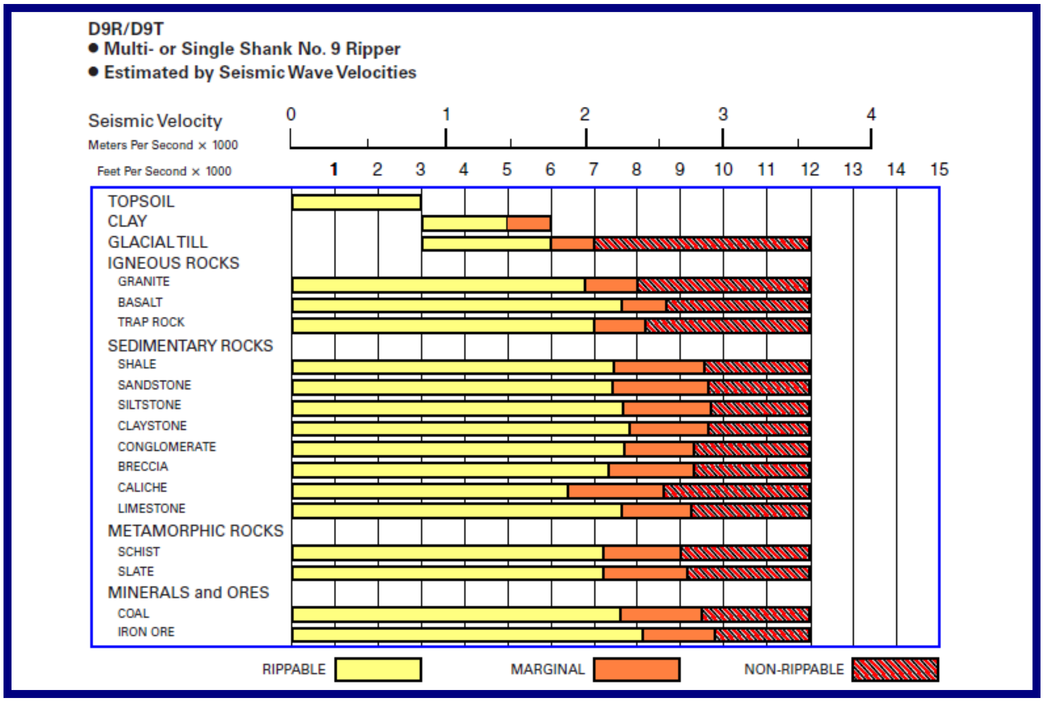


FIGURE 2- Caterpillar D9R Ripper Performance Chart (2017).

A summary of the generalized rippability characteristics of granitic bedrock has been provided below to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas that were surveyed. The velocity ranges described below are general averages of Tables 2 and 3 (see Pages 6 and 7) and assume typical, good-working, heavy excavation equipment, such as single shank D9R dozer, as described by Caterpillar, Inc. (2000 and 2017). However, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges as the performance charts are tailored for conventional bulldozer equipment and cannot be directly correlated. Trenching operations which utilize large excavator-type equipment within granitic bedrock materials, typically encounter very difficult to non-productable conditions where seismic velocities are generally greater than 4,000± fps, and less for smaller backhoe-type equipment.

- **Rippable Condition (0 - 4,000 ft/sec)**:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

- **<u>Marginally Rippable Condition (4,000 - 7,000 ft/sec)</u>**:

  This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **<u>Non-Rippable Condition (7,000 ft/sec or greater)</u>**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

GEOLOGIC & EARTHWORK CONSIDERATIONS

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report and/or information gathered for the subject project which may describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults, and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is foliated and/or fractured at depth, this structure could aid in excavation production. Unfavorable bedrock conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment that will be necessary and the production levels that can be anticipated for this project. A summary of excavation considerations is included within Appendix C in order to provide you and your grading contractor with a better understanding of the complexities of excavation in bedrock materials, so that proper planning and excavation techniques can be employed.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of good quality with moderate amounts of ambient “noise” that was introduced during our survey, from apparent radio/microwave interference. Analysis of the data and picking of the primary “P”-wave arrivals was therefore performed with slight difficulty, with interpolation of some data points being necessary. Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process, integrate, and display the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  The upper V1 layer (average weighted velocity of 1,261 to 1,963 fps) is believed to consist of topsoil, colluvium, older alluvial deposits, and/or completely-weathered and fractured bedrock materials. No excavation difficulties are expected within this velocity layer, however, isolated floaters (i.e., boulders, corestones, dikes, etc.) may be encountered and could produce somewhat difficult conditions locally.

- **Velocity Layer V2**:

  The lower V2 layer is believed to consist of highly-weathered granitic bedrock and/or indurated older alluvial sediments (San Dimas Formation), with an average weighted velocity range of 2,597 to 3,240 fps. This velocity range is typical for both earth material types and the two units cannot be distinguished separately. With respect to granitic rock, this layer may include relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. The indurated older alluvial sediments are expected to be generally homogenous but most likely get relatively harder with depth. Caterpillar (2017; see Figure 2) indicates this velocity range to be “rippable” using a D9R dozer or equivalent. No unusually hard excavations are anticipated during grading in this velocity layer.

- **Velocity Layer V3**:

  The third V3 layer is believed to consist of moderately-weathered granitic bedrock. Hard excavation difficulties within this layer (average weighted velocity range of 6,653 to 8,111 fps) should be anticipated during grading. This layer may consist of relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Blasting may be necessary to achieve desired grade, including any infrastructure when approaching the higher end of this velocity range. Caterpillar (2017; see Figure 2) indicates this velocity range to be “marginally rippable” to “non-rippable” using a D9R dozer or equivalent. Larger equipment may facilitate excavation potentials within this higher velocity layer.

The ray sampling coverage of the subsurface seismic waves that were acquired during the processing of the tomographic models using **Rayfract™**, appeared to be of good quality which was verified by having a Root Mean Square Error (RMS) of 0.9 to 1.5 percent (see lower right-hand corner of each model). The RMS error (misfit between picked and modeled first break times) is automatically calculated during the processing routine, with a value of less than 2.0% being preferred, of which all the models obtained. Based on the tomographic modeling and typical excavation characteristics observed within granitic bedrock of the southern California region, anticipation of gradual increasing hardness with depth should be anticipated during grading. Some lateral velocity variations should be expected to be encountered across the site generally due to the presence of buried corestones and/or dikes.

CLOSURE

The field geophysical survey was performed on August 8, 2017 by the undersigned using "state of the art" geophysical equipment and techniques along the selected traverse locations scattered across the site. The seismic data was further evaluated using recently developed computerized tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface velocity and structural conditions. It should be noted that our data presented within this report was obtained along four specific locations therefore other areas in the local may contain different velocity layers and depths not encountered during our field survey. Additional survey traverses may be necessary to further evaluate the excavation characteristics across other portions of the site where cut grading will be proposed, if warranted. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and in the interpretation and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied.

In summary, the results of this seismic refraction survey are to be considered as an aid to assessing the rippability and excavation potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative “test” excavations with the proposed type of excavation equipment for the proposed construction should be considered, so that they may be correlated with the data presented within this report.

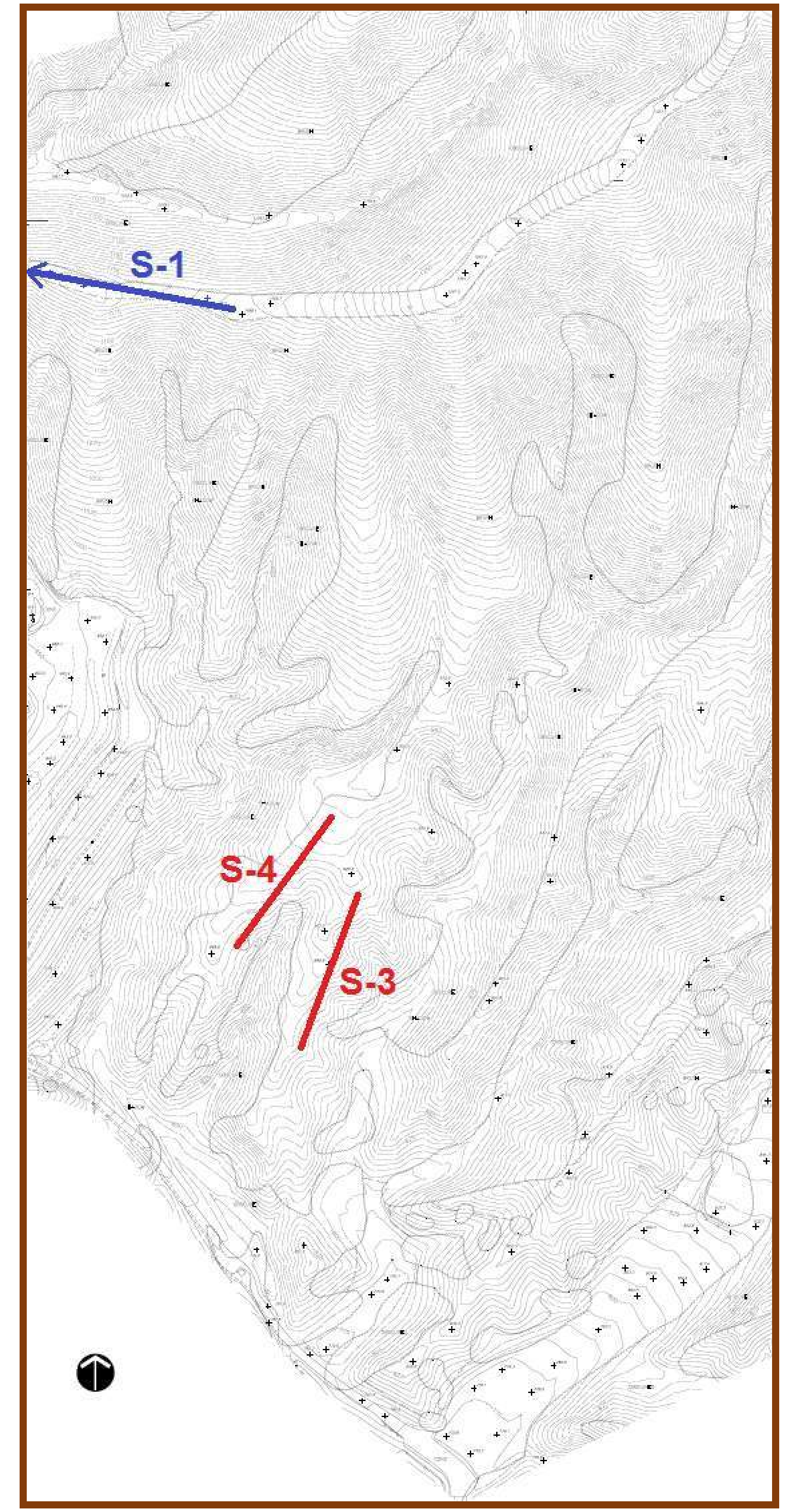
SEISMIC LINE LOCATION MAP

(OVERVIEW)

From Google™ Earth imagery (2017).

PROJECT NO. 173000-1 **PLATE 1**

SEISMIC LINE LOCATION MAP

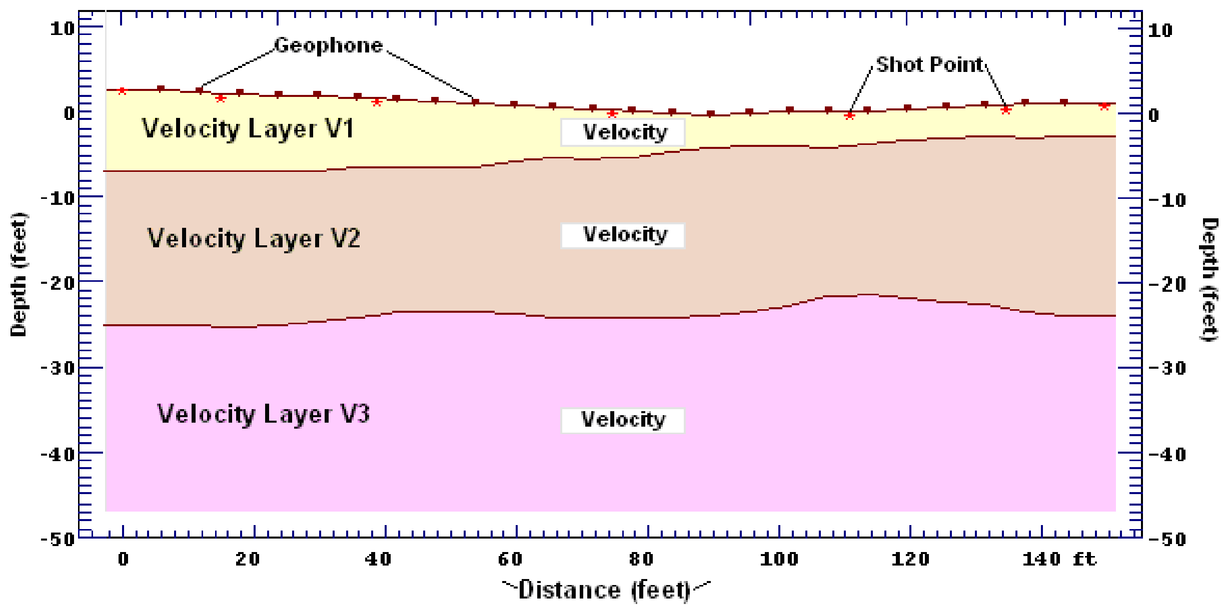


APPENDIX A

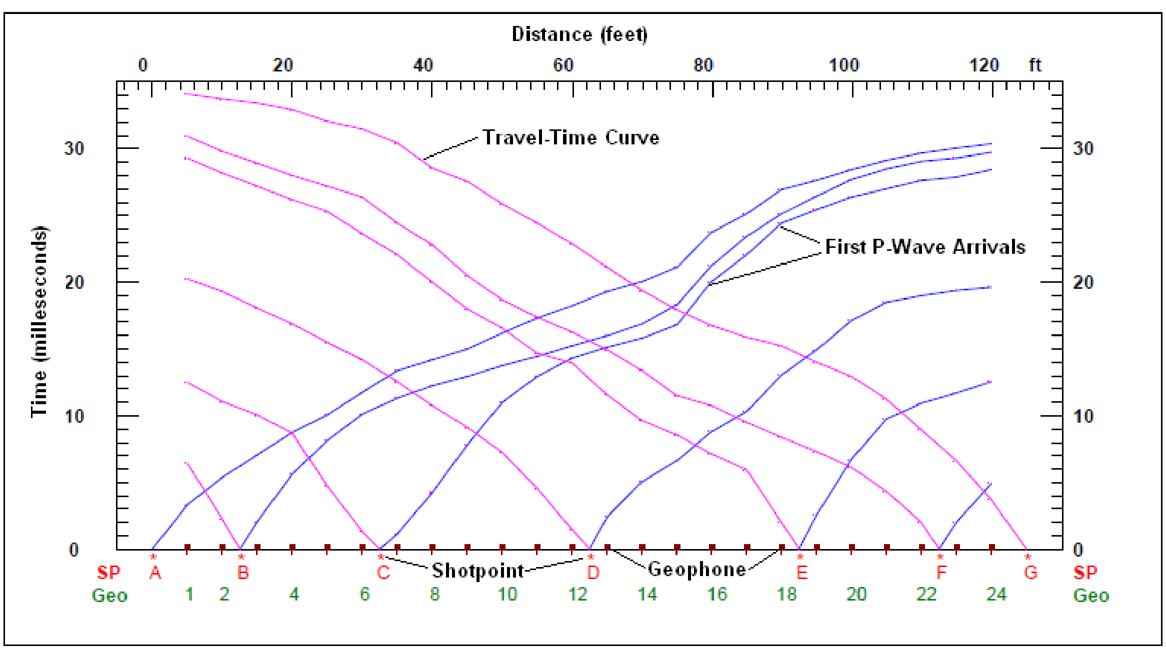
LAYER VELOCITY MODELS

LAYER VELOCITY MODEL LEGEND

LAYER VELOCITY MODEL



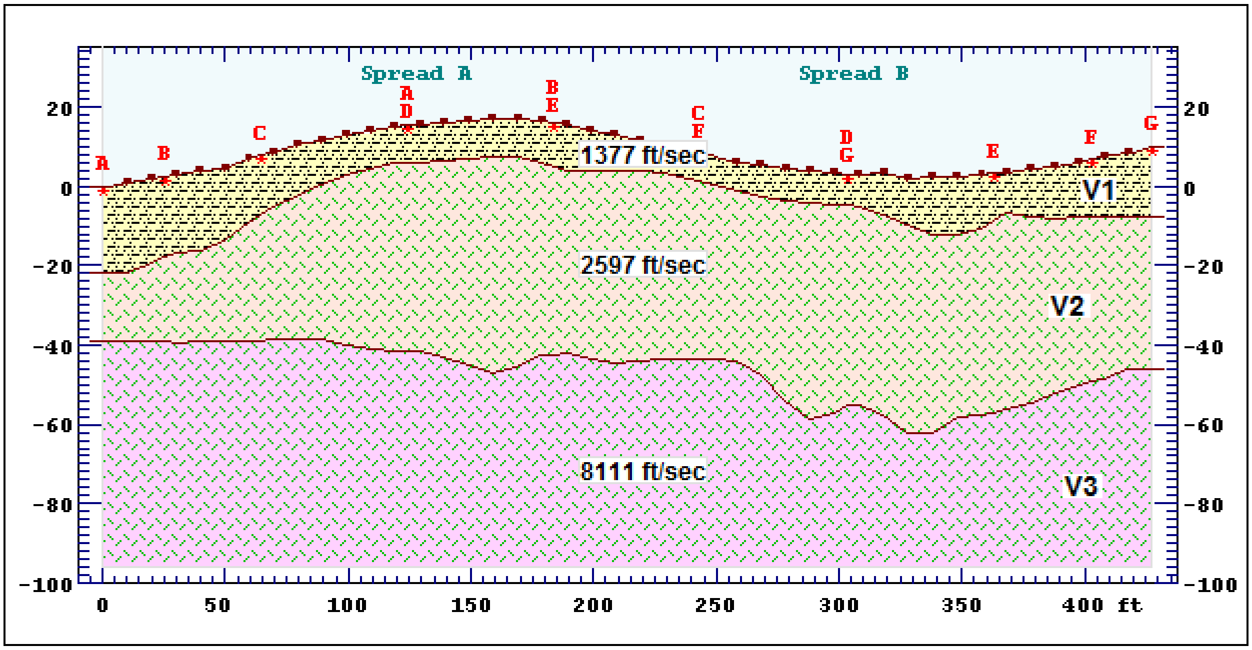
TIME-DISTANCE PLOT



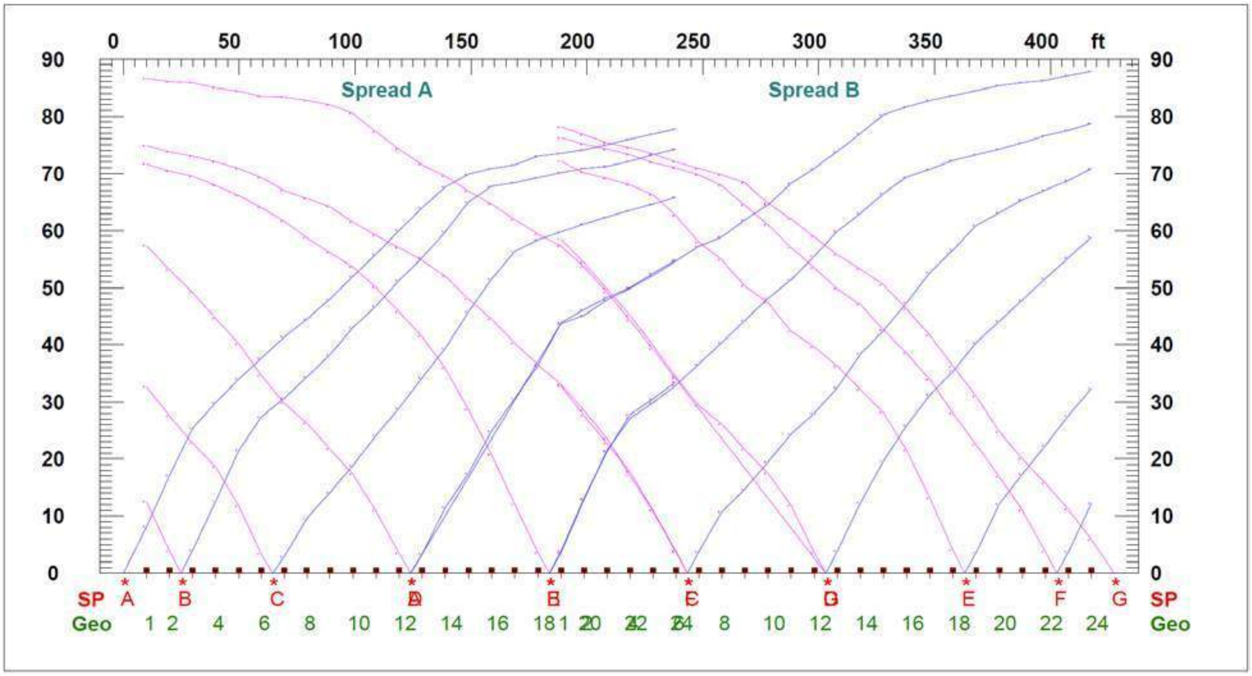
SEISMIC LINE S-1

South 81° East >

LAYER VELOCITY MODEL



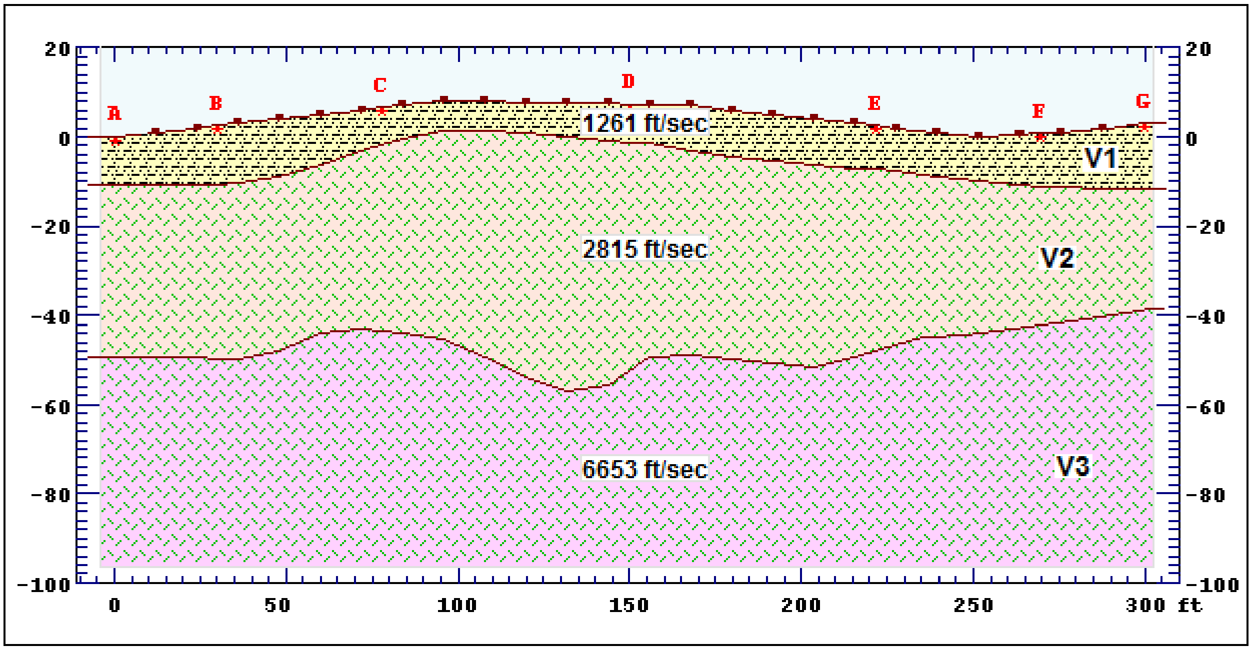
TIME-DISTANCE PLOT



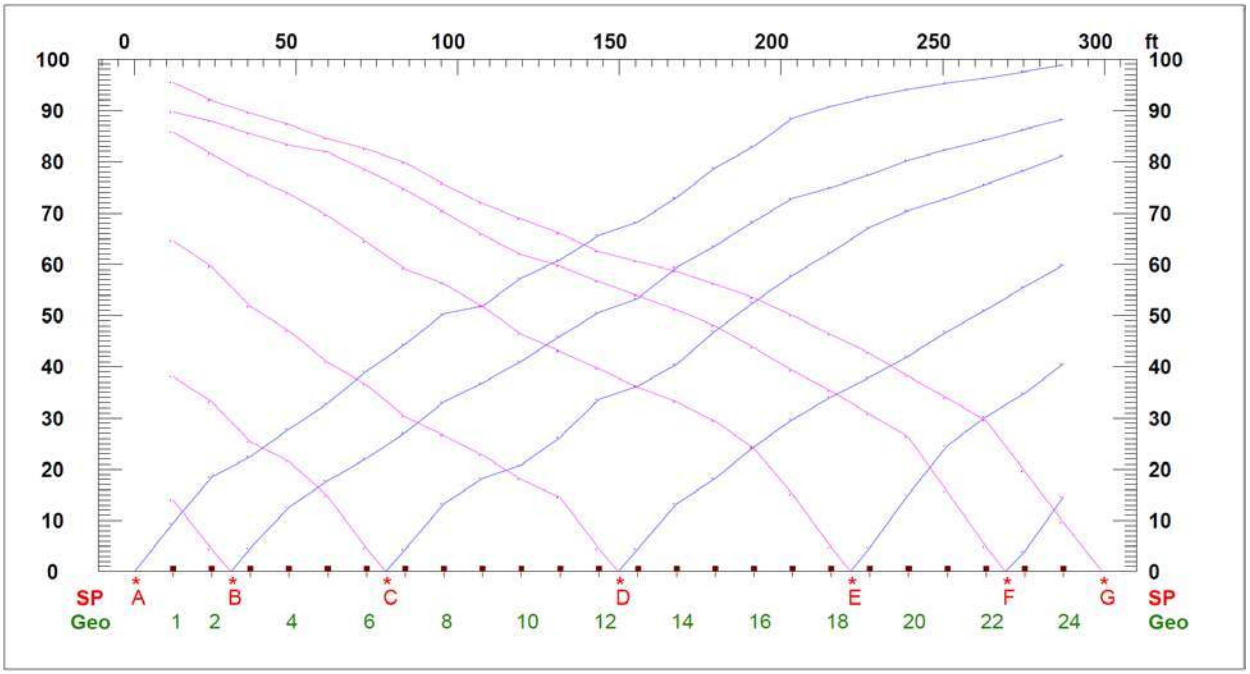
SEISMIC LINE S-2

North 69° East >

LAYER VELOCITY MODEL



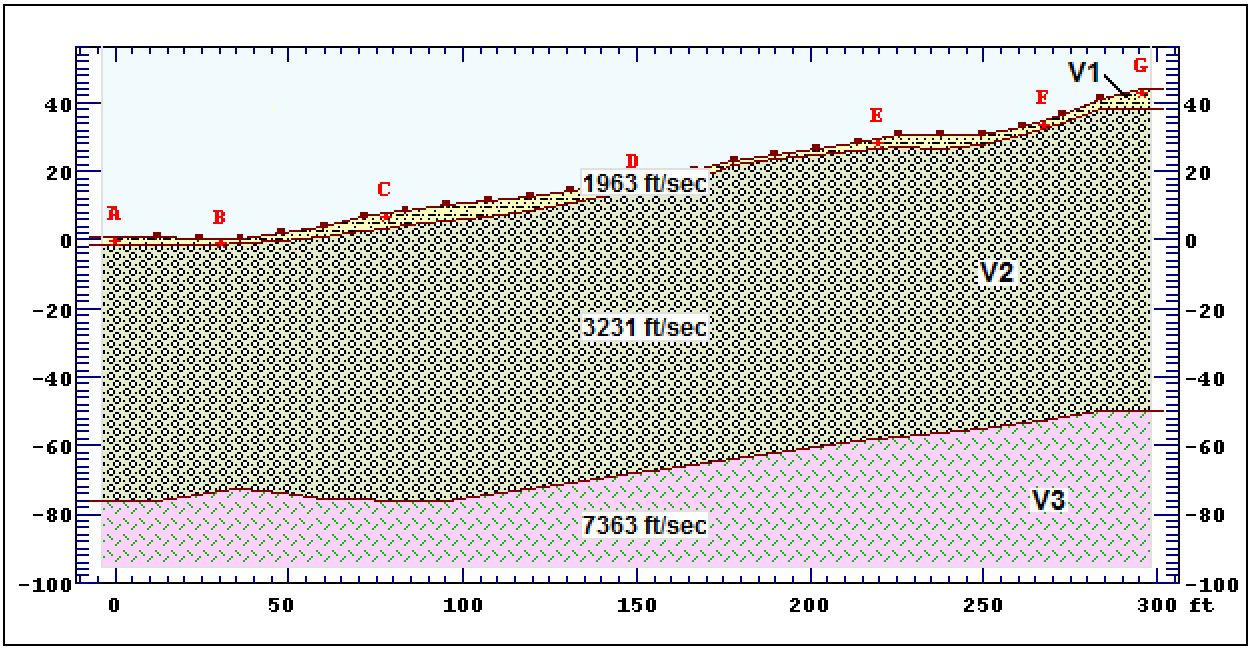
TIME-DISTANCE PLOT



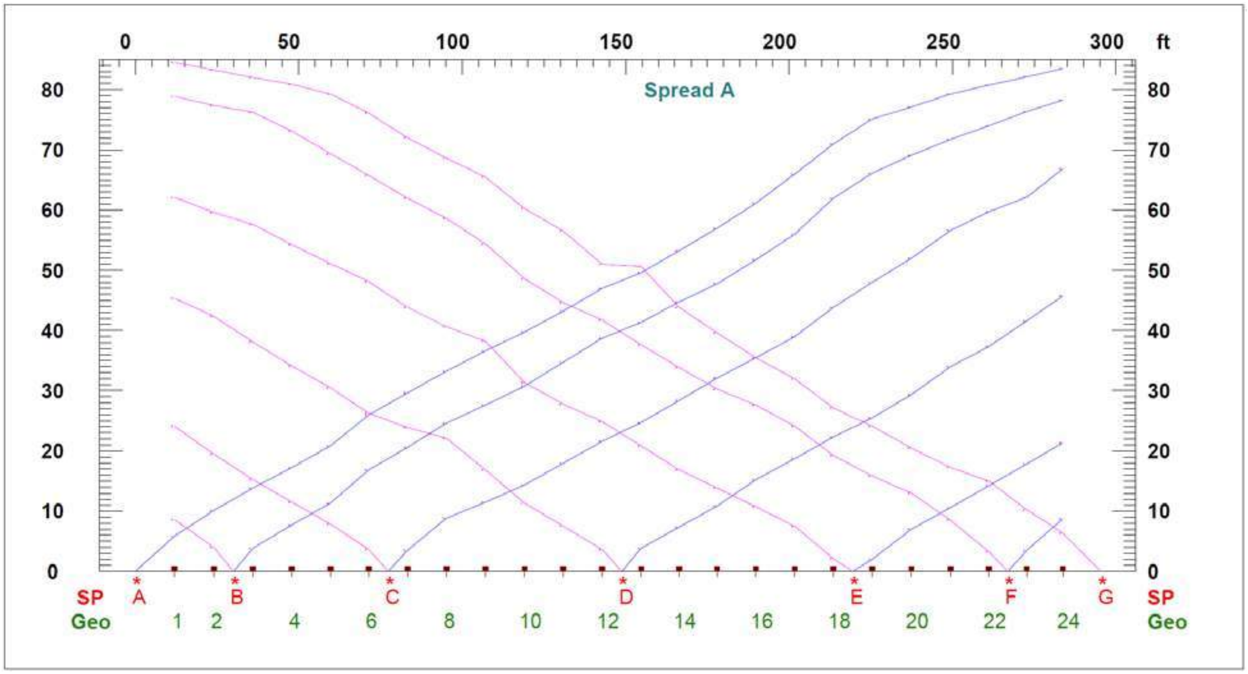
SEISMIC LINE S-3

North 21° East >

LAYER VELOCITY MODEL



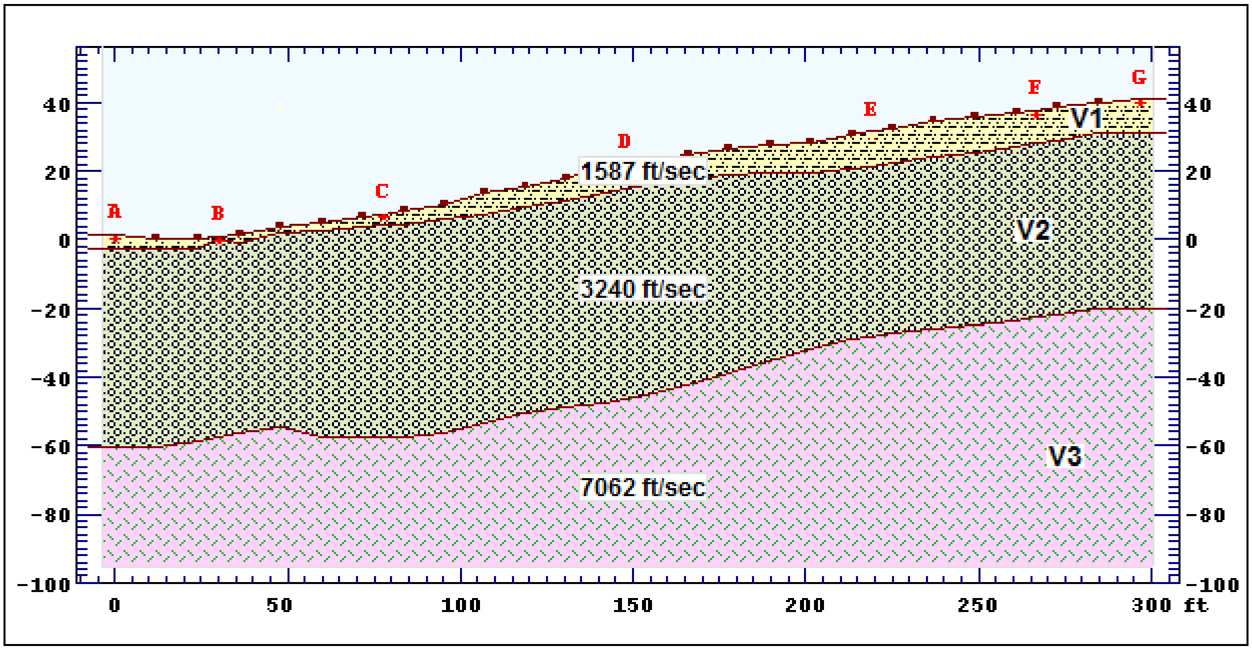
TIME-DISTANCE PLOT



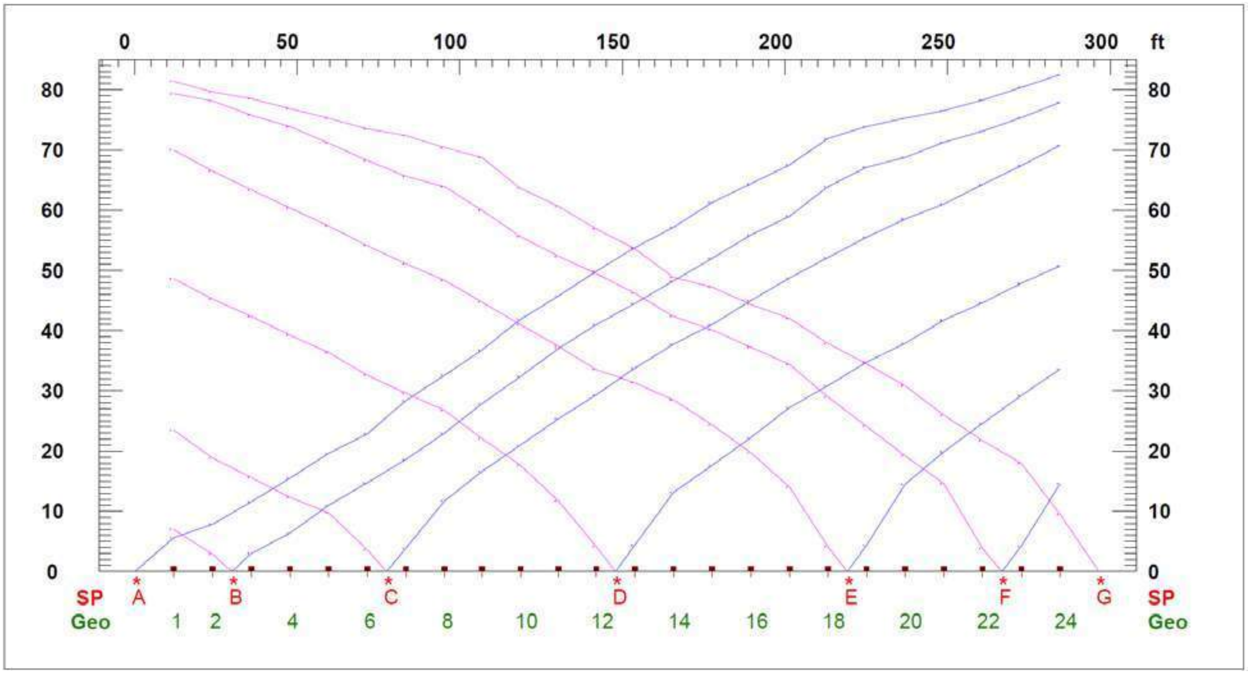
SEISMIC LINE S-4

North 38° East >

LAYER VELOCITY MODEL



TIME-DISTANCE PLOT



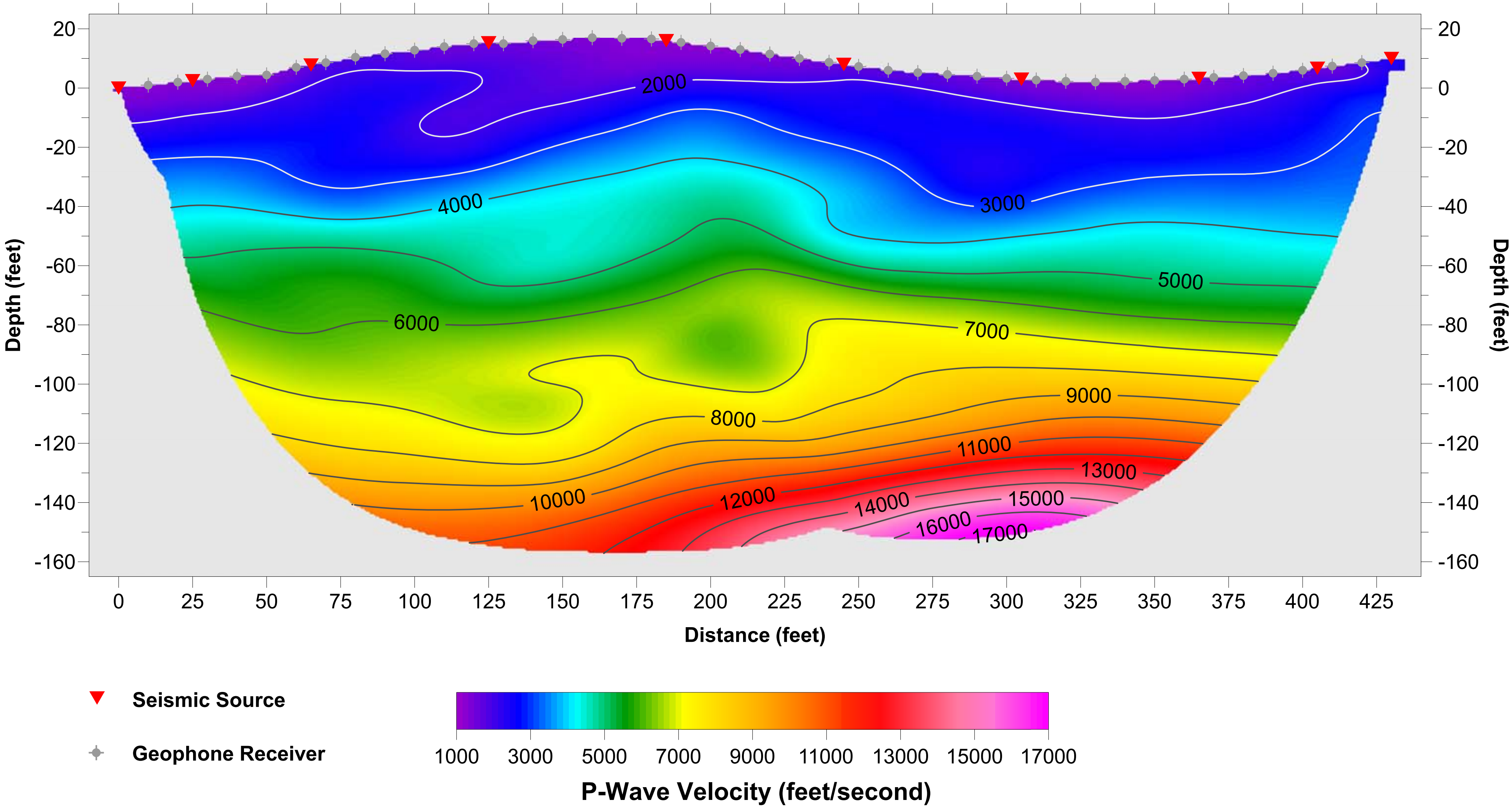
APPENDIX B

REFRACTION TOMOGRAPHIC MODELS

SEISMIC LINE S-1

South 81° East →

REFRACTION TOMOGRAPHIC MODEL



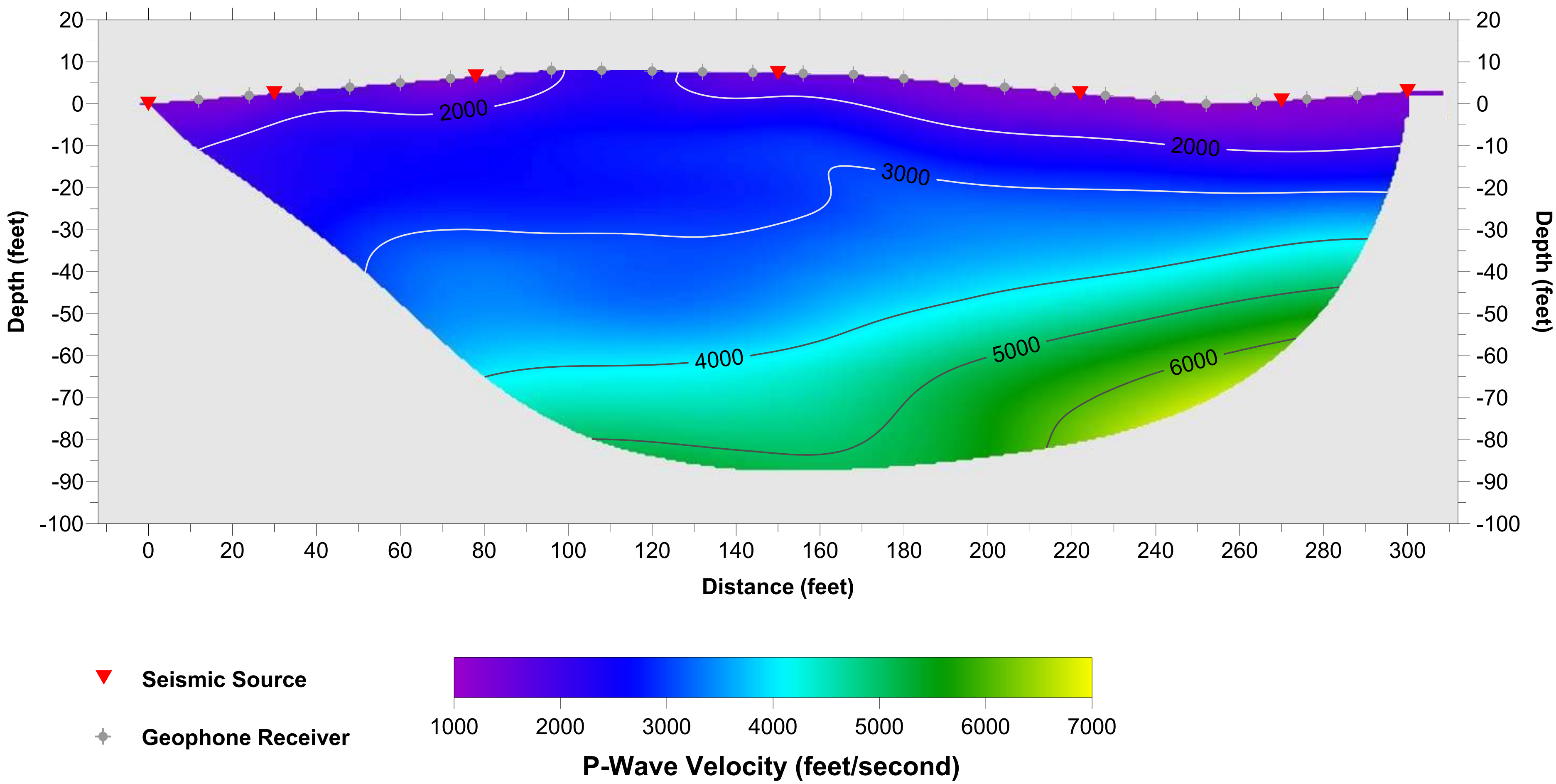
SCALE: 1:1 (Horizontal = Vertical)

RMS error 0.9%, Rayfract Version 3.35

SEISMIC LINE S-2

North 69° East →

REFRACTION TOMOGRAPHIC MODEL



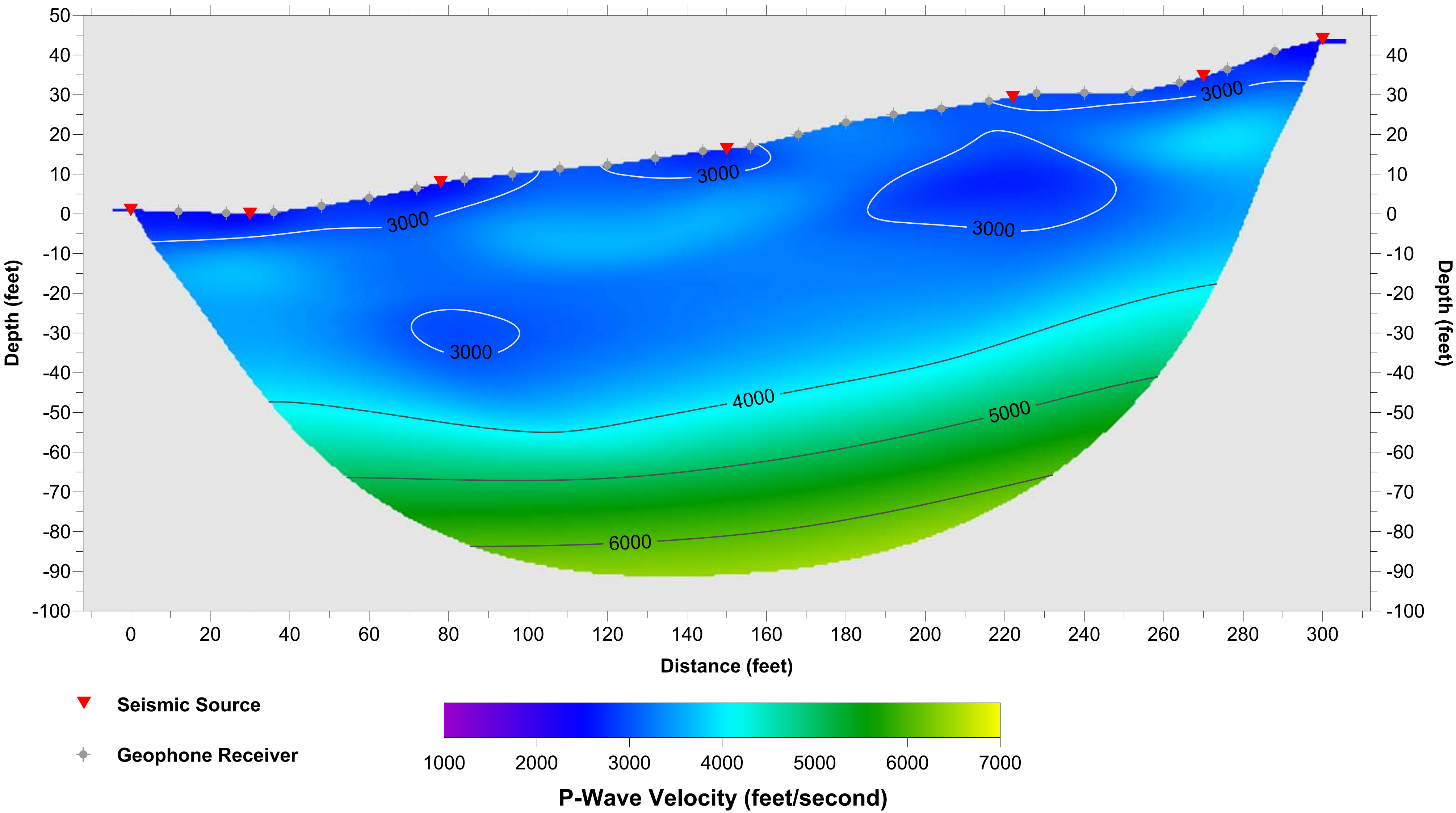
SCALE: 1:1 (Horizontal = Vertical)

RMS error 1.2%, Rayfract Version 3.35

SEISMIC LINE S-3

North 21° East →

REFRACTION TOMOGRAPHIC MODEL



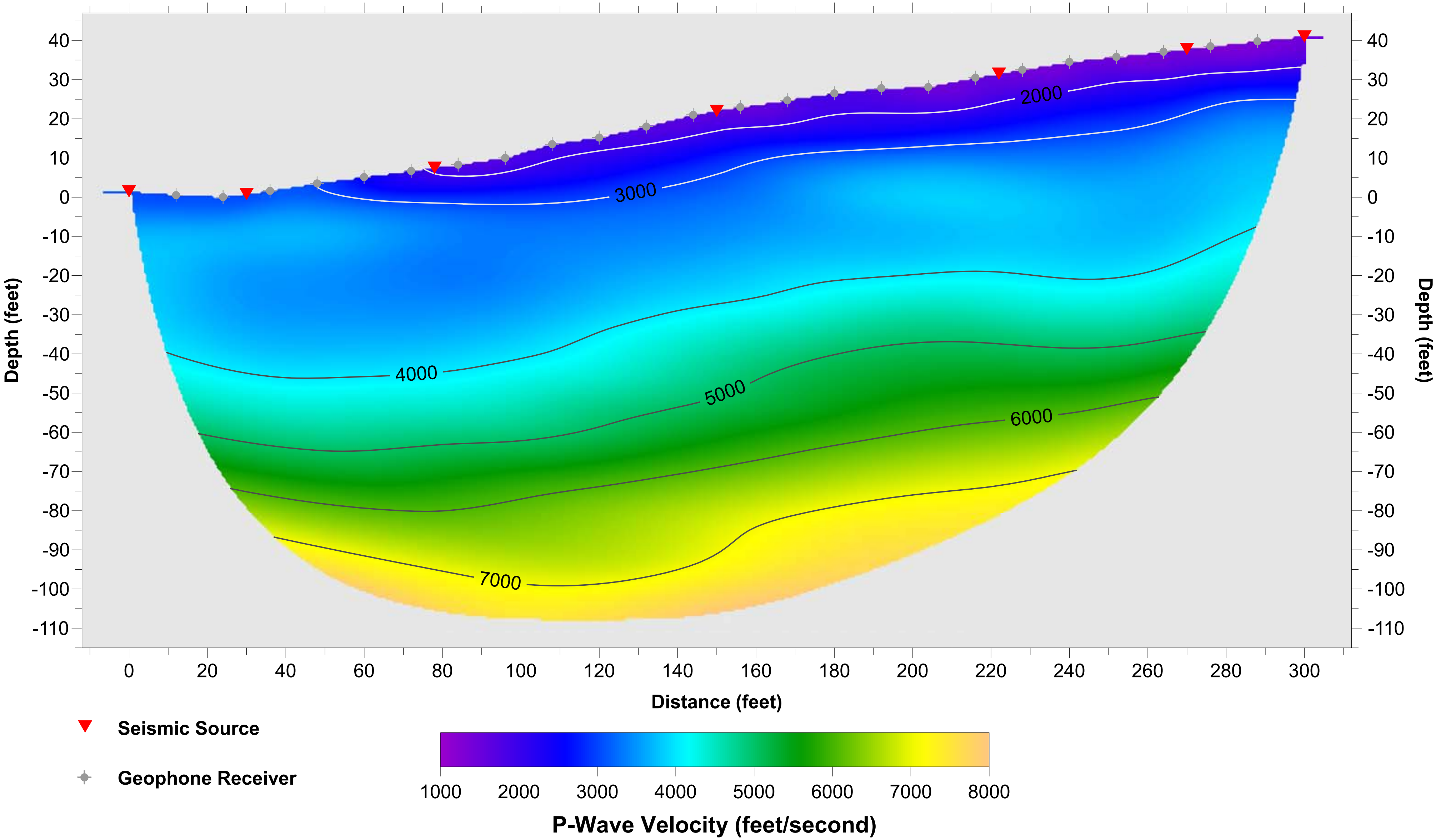
SCALE: 1:1 (Horizontal = Vertical)

RMS error 1.5%, Rayfract Version 3.35

SEISMIC LINE S-4

North 38° East →

REFRACTION TOMOGRAPHIC MODEL



SCALE: 1:1 (Horizontal = Vertical)

RMS error 0.9%, Rayfract Version 3.35

APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the client's responsibility to ensure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ***Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,***
- ***Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,***
- ***Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.***

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also, important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to ensure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX D

REFERENCES

REFERENCES

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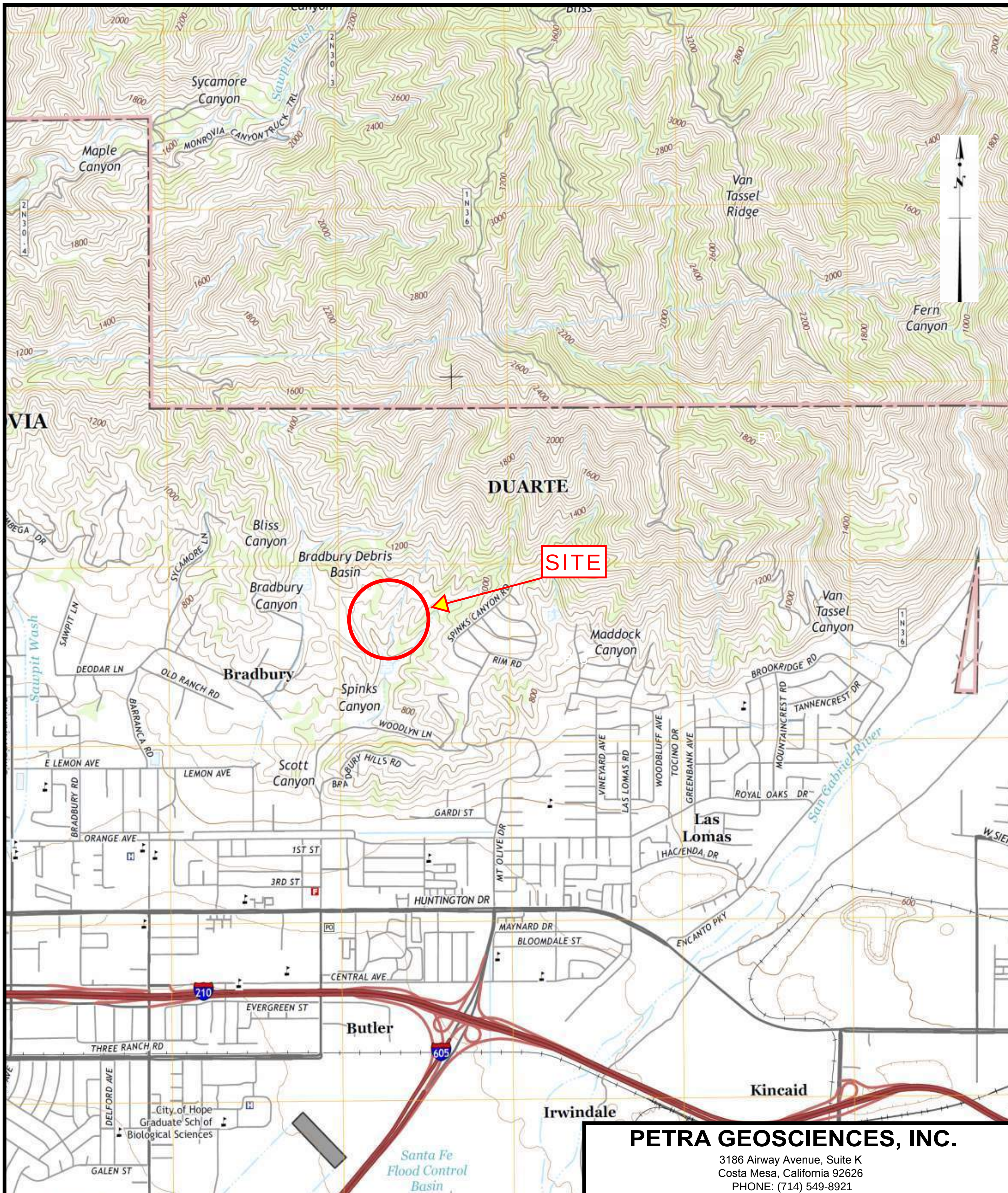


Base Map:

Portion of USGS Azusa Quadrangle Topographic Series, 2015

PETRA GEOSCIENCES, INC.
3186 Airway Avenue, Suite K
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

SITE LOCATION MAP

Chadwick Ranch Project
City of Bradbury, Los Angeles County, California

PETRA GEOSCIENCES

DATE: October, 2019

J.N.: 17-219

Figure 1

SYMBOL EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along faultbreak.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

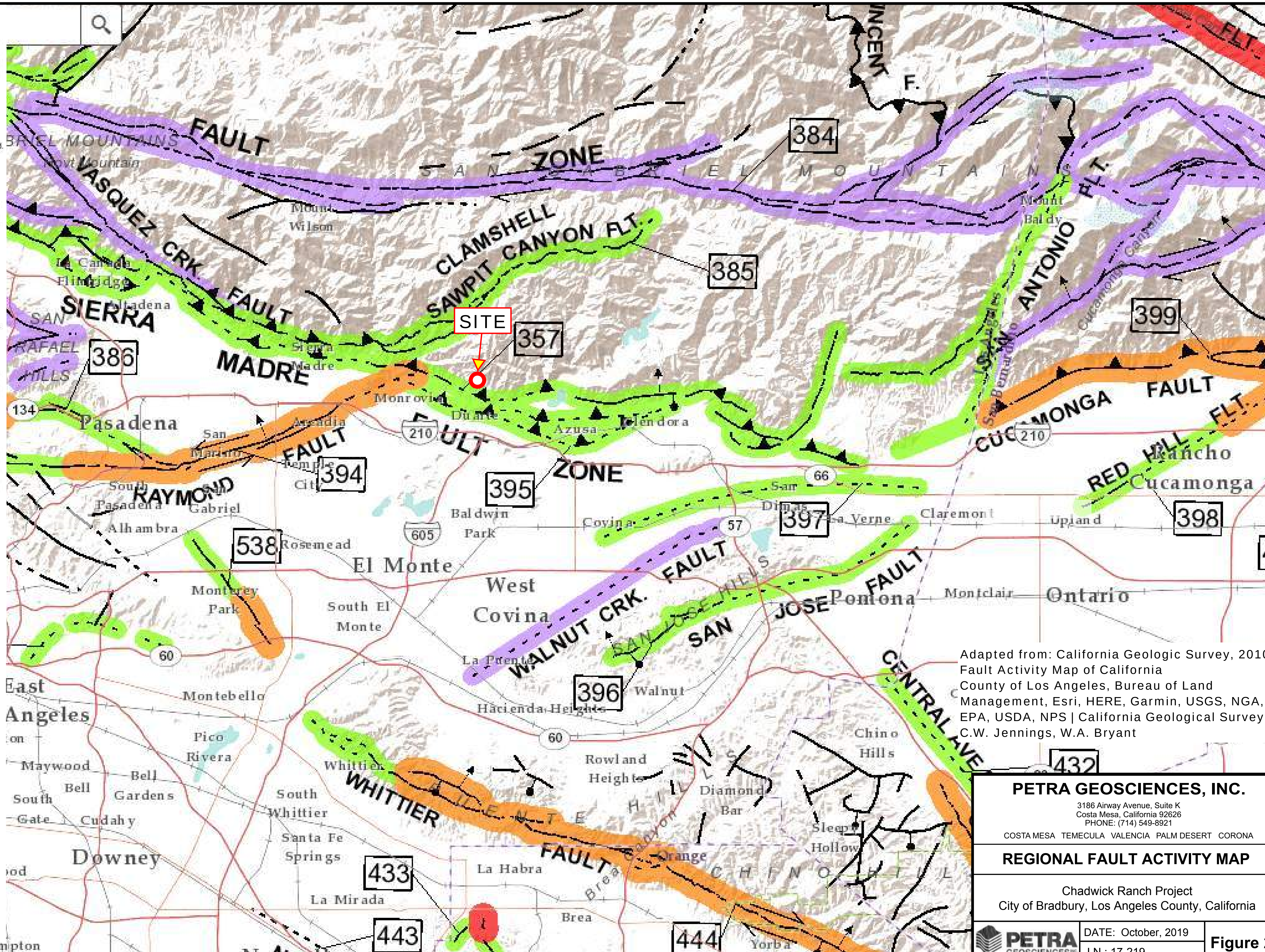
Low angle fault (barbs on upper plate).

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.



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REGIONAL FAULT ACTIVITY MAP

Chadwick Ranch Project
City of Bradbury, Los Angeles County, California

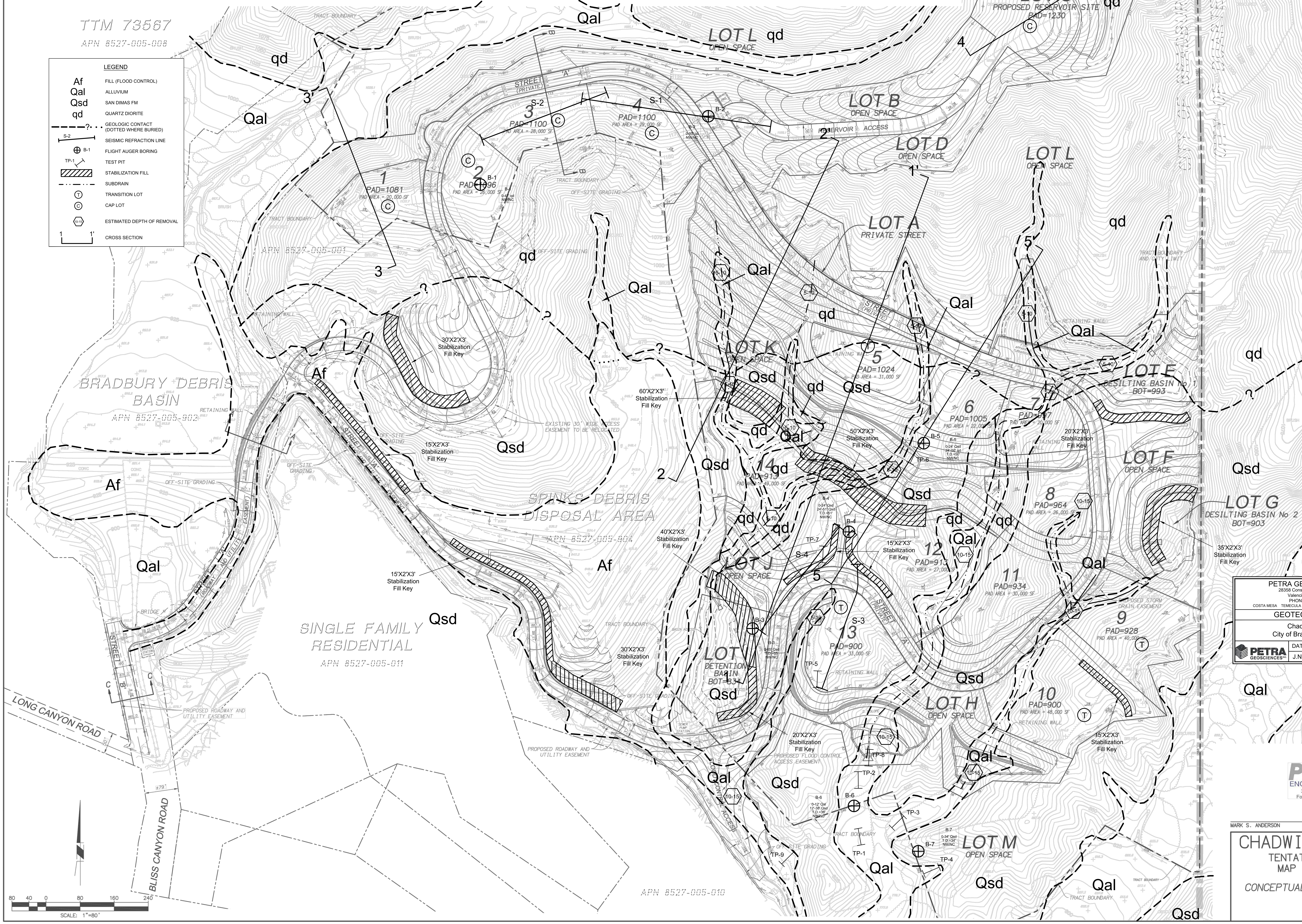
PETRA GEOSCIENCES INC

DATE: October, 2019
J.N.: 17-219

Figure 2

TENTATIVE TRACT MAP No. 82349

CITY OF BRADBURY



PETRA GEOSCIENCES, INC.
28358 Constellation Road, Unit 680
Valencia, California 91355
PHONE: (661) 255-5790
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

GEOTECHNICAL MAP

Chadwick Ranch
City of Bradbury, California

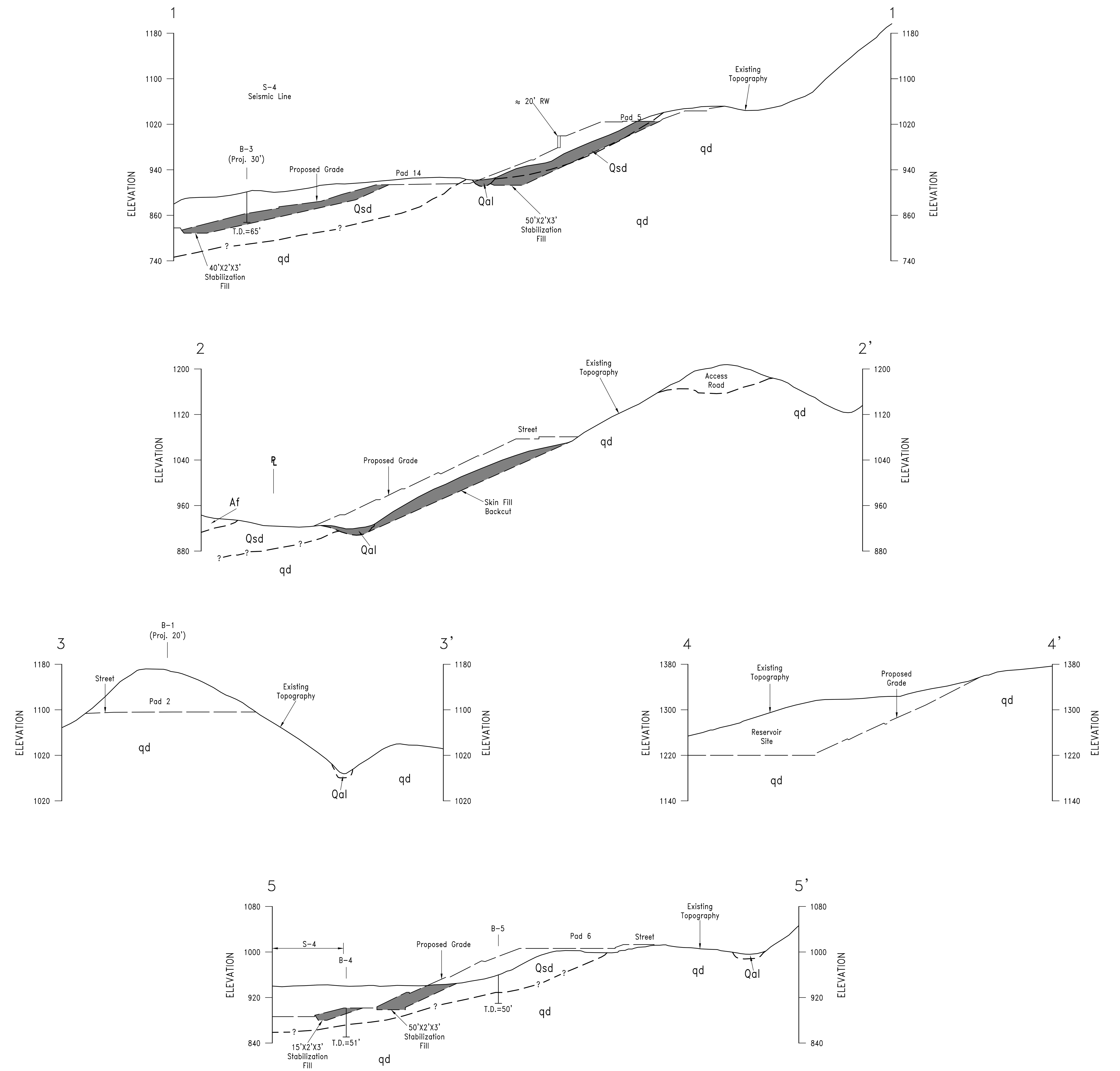
DATE: OCT. 2019
J.N: 17-219
PLATE 1

PROACTIVE ENGINEERING CONSULTANTS
27042 Towne Centre Drive, Suite 110
Foothill Ranch, CA 92610 (949) 716-7460

MARK S. ANDERSON DATE

CHADWICK RANCH
TENTATIVE TRACT MAP No 82349
CONCEPTUAL GRADING PLAN

SHEET 2 OF 3
September 26, 2019



Shading Indicates
Remedial Grading

CROSS–SECTIONS 1–1', 2–2', 3–3', 4–4' and 5–5'

SCALE H&V 1"=80'

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28358 Constellation Road, Unit 680
Valencia, California 91355
PHONE: (661) 255-5790
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

GEOLOGIC CROSS SECTIONS

Chadwick Ranch
City of Bradbury

PETRA GEOSCIENCES

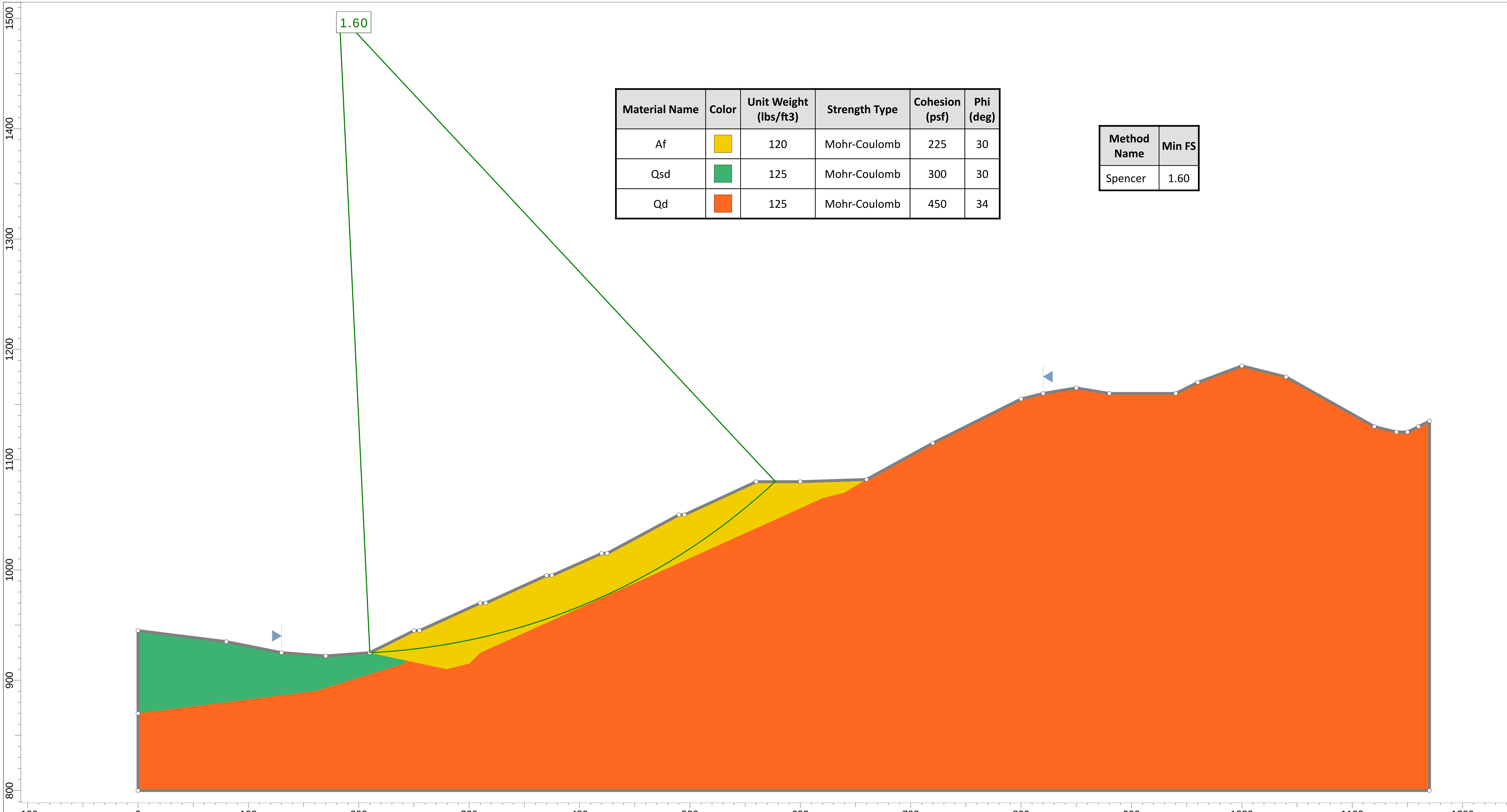
DATE: 10/2019
J.N: 17-219

PLATE 2

APPENDIX II

SLOPE STABILITY ANALYSES

USGS SEISMIC DESIGN PARAMETERS



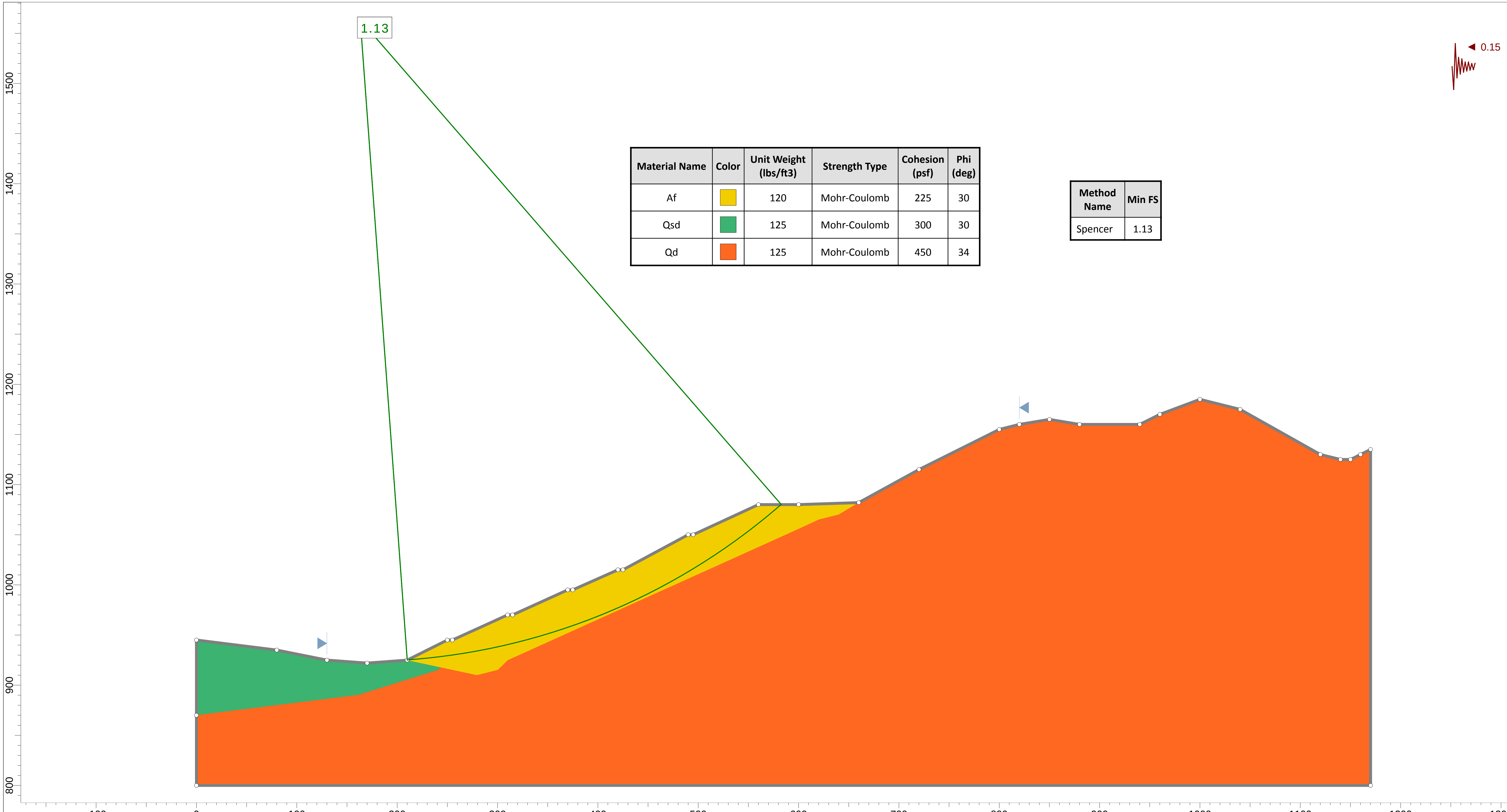
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af		120	Mohr-Coulomb	225	30
Qsd		125	Mohr-Coulomb	300	30
Qd		125	Mohr-Coulomb	450	34

Method Name	Min FS
Spencer	1.60

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project	17-219 Chadwick Ranch Project
Analysis Description	Section 2-2'
Drawn By	AM
Scale	1:1000
Company	Petra Geosciences, Inc.
Date	October 2019
File Name	Slide1.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af		120	Mohr-Coulomb	225	30
Qsd		125	Mohr-Coulomb	300	30
Qd		125	Mohr-Coulomb	450	34

Method Name	Min FS
Spencer	1.13

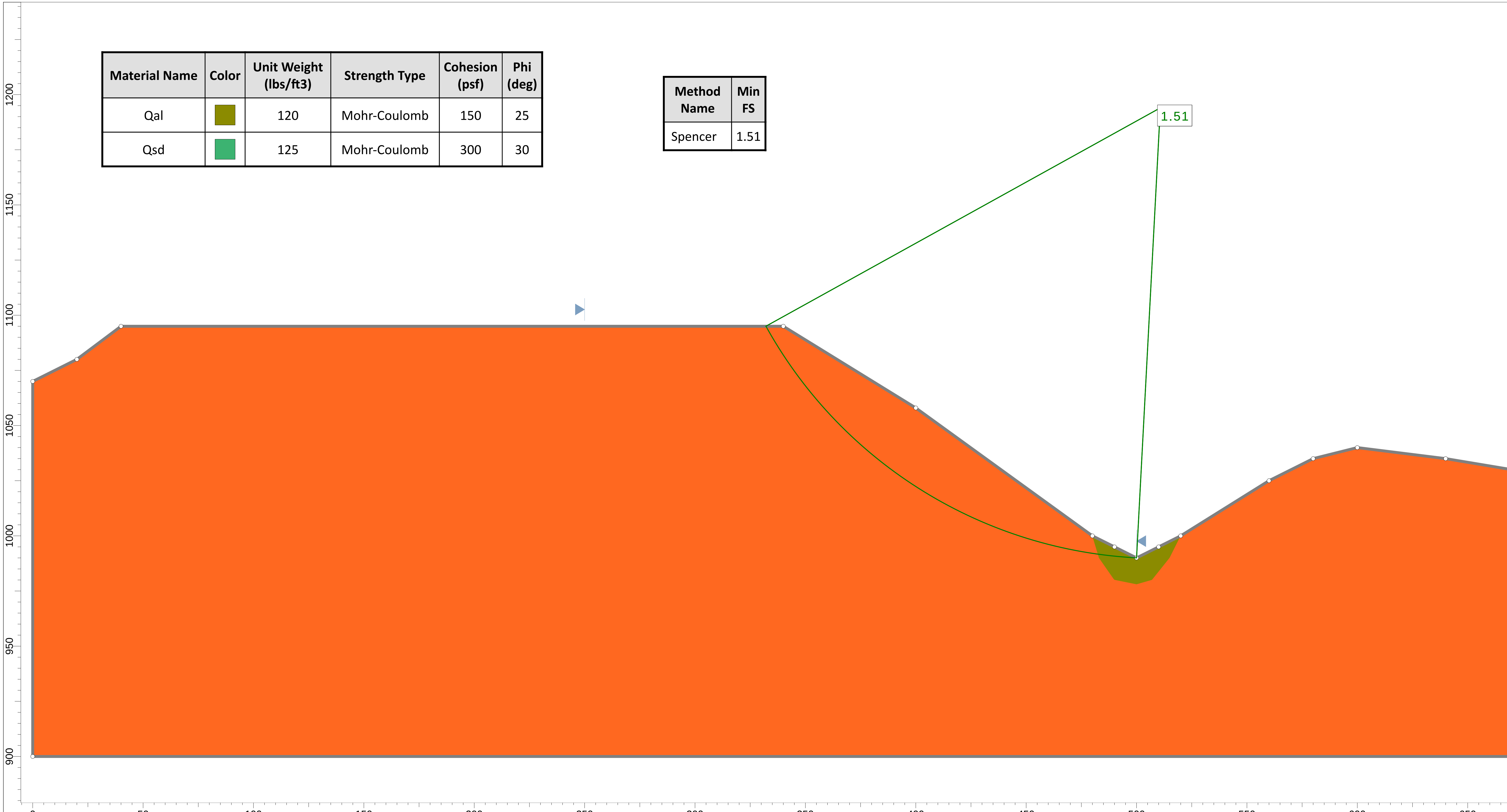
PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project	17-219 Chadwick Ranch Project		
Analysis Description	Section 2-2'		
Drawn By	AM	Scale	1:1100
Company	Petra Geosciences, Inc.		
Date	October 2019	File Name	Slide1.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qal		120	Mohr-Coulomb	150	25
Qsd		125	Mohr-Coulomb	300	30

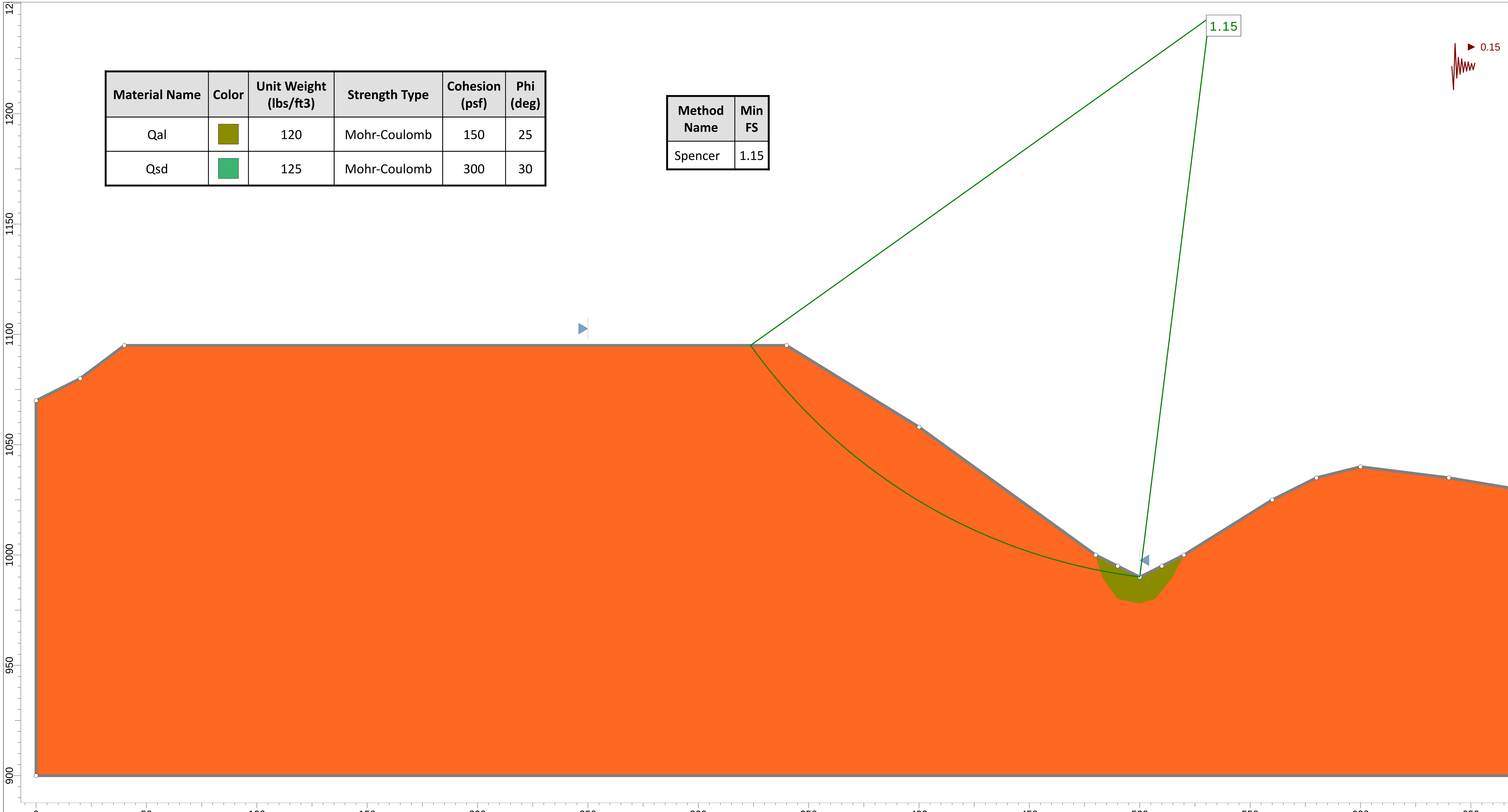
Method Name	Min FS
Spencer	1.51



PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project	17-219 Chadwick Ranch Project
Analysis Description	Section 3-3'
Drawn By	AM
Scale	1:500
Company	Petra Geosciences, Inc.
Date	October 2019
File Name	Slide1.slmd



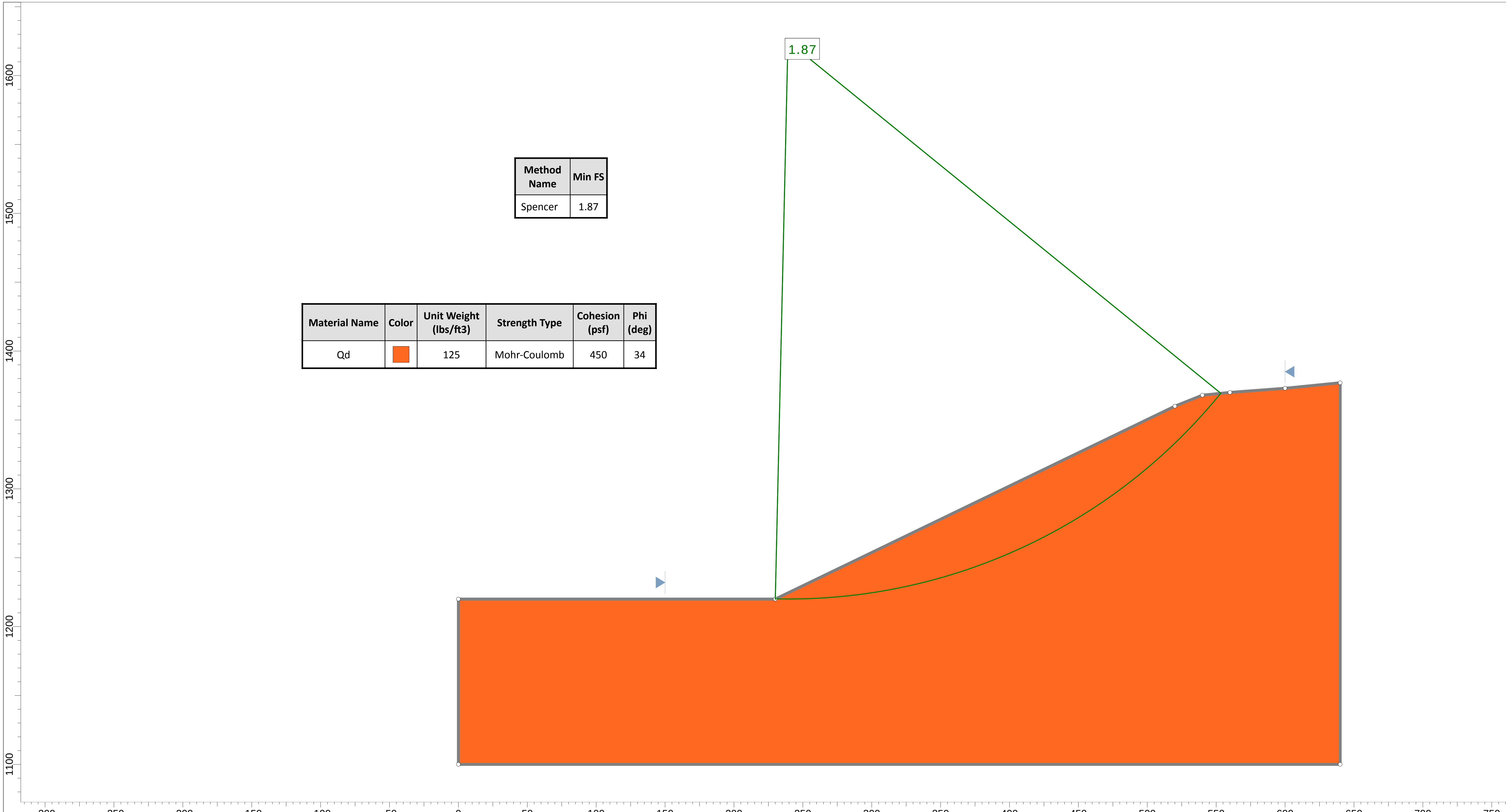
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qal		120	Mohr-Coulomb	150	25
Qsd		125	Mohr-Coulomb	300	30

Method Name	Min FS
Spencer	1.15

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project	17-219 Chadwick Ranch Project		
Analysis Description	Section 3-3'		
Drawn By	AM	Scale	1:500
Company	Petra Geosciences, Inc.		
Date	October 2019	File Name	Slide1.slmd



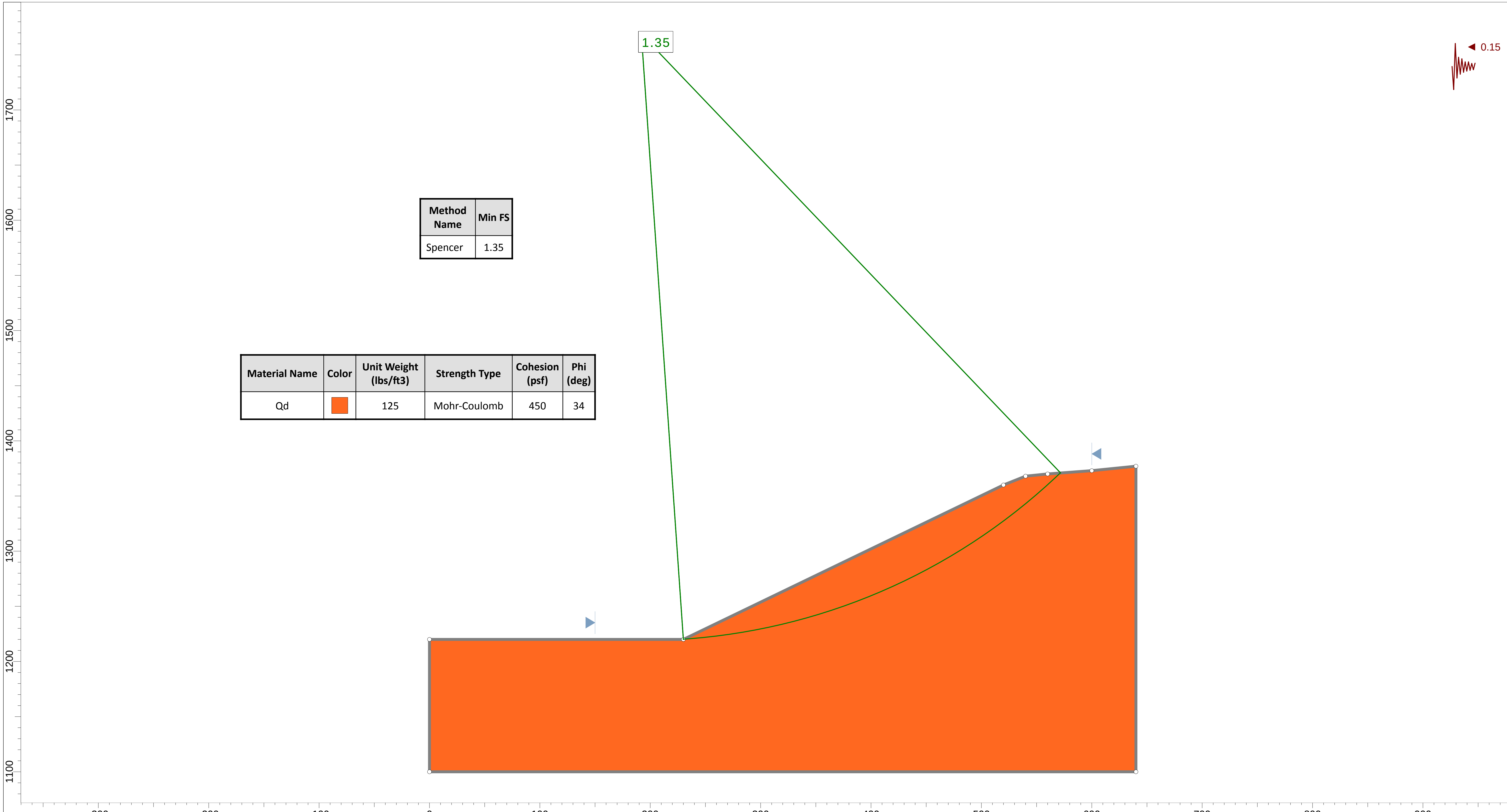
Method Name	Min FS
Spencer	1.87

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qd		125	Mohr-Coulomb	450	34

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project	17-219 Chadwick Ranch Project		
Analysis Description	Section 4-4'		
Drawn By	AM	Scale	1:800
Company	Petra Geosciences, Inc.		
Date	October 2019	File Name	Slide1.slmd



Method Name	Min FS
Spencer	1.35

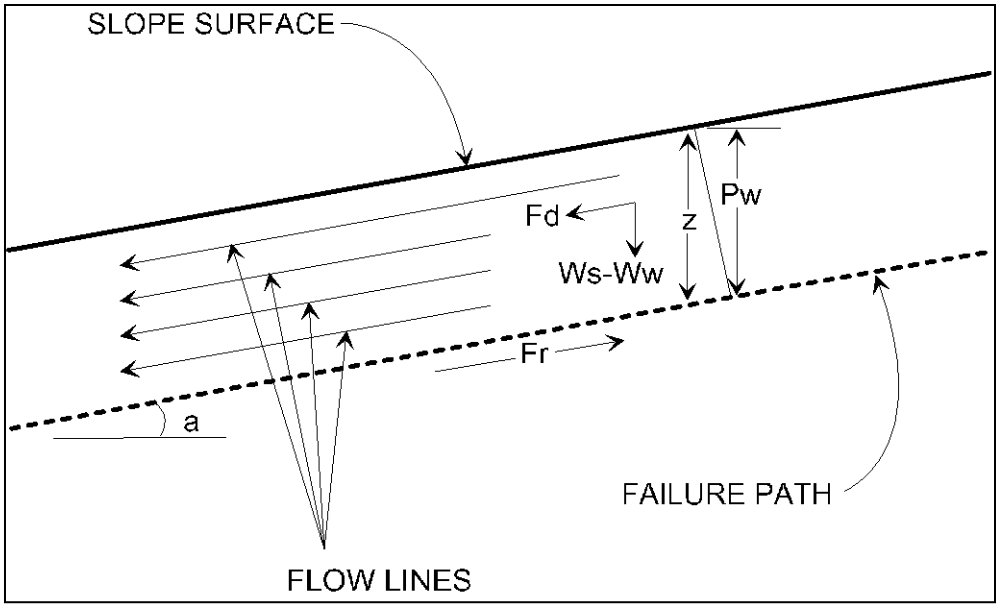
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Qd		125	Mohr-Coulomb	450	34

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.021

Project: 17-219 Chadwick Ranch Project			
Analysis Description: Section 4-4'			
Drawn By: AM	Scale: 1:1000	Company: Petra Geosciences, Inc.	
Date: October 2019		File Name: Slide1.slmd	

SURFICIAL STABILITY ANALYSIS



Assume: (1) Saturation To Slope Surface
(2) Sufficient Permeability To Establish Water Flow

Pw = Water Pressure Head=(z)(cos^2(a))
Ws = Saturated Soil Unit Weight
Ww = Unit Weight of Water (62.4 lb/cu.ft.)
u = Pore Water Pressure=(Ww)(z)(cos^2(a))
z = Layer Thickness
a = Angle of Slope
phi = Angle of Friction
c = Cohesion
Fd = (0.5)(z)(Ws)(sin(2a))
Fr = (z)(Ws-Ww)(cos^2(a))(tan(phi)) + c
Factor of Safety (FS) = Fr/Fd

2:1 Fill Slope

Given:

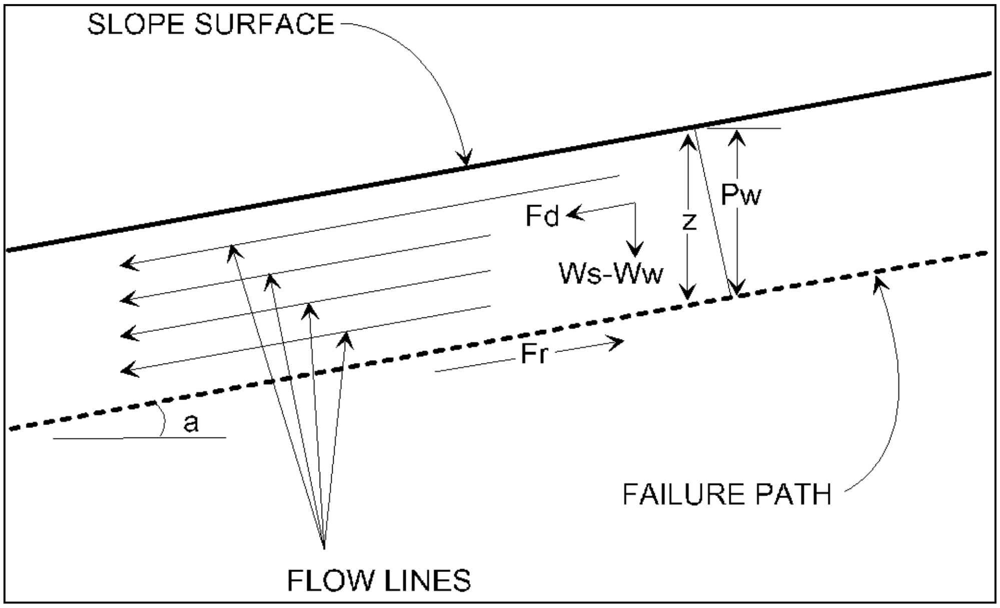
Ws	z	a		phi		c
(pcf)	(ft)	(degrees)	(radians)	(degrees)	(radians)	(psf)
120	4	26.6	0.464258	30	0.523599	225

Calculations:

Pw	u	Fd	Fr	**FS**
3.20	199.56	192.18	331.35	**1.72**

Job No. 17-219

SURFICIAL STABILITY ANALYSIS



Assume: (1) Saturation To Slope Surface
(2) Sufficient Permeability To Establish Water Flow

Pw = Water Pressure Head=(z)(cos^2(a))
Ws = Saturated Soil Unit Weight
Ww = Unit Weight of Water (62.4 lb/cu.ft.)
u = Pore Water Pressure=(Ww)(z)(cos^2(a))
z = Layer Thickness
a = Angle of Slope
phi = Angle of Friction
c = Cohesion
Fd = (0.5)(z)(Ws)(sin(2a))
Fr = (z)(Ws-Ww)(cos^2(a))(tan(phi)) + c
Factor of Safety (FS) = Fr/Fd

1.4:1 Natural Slope - San Dimas Formation (Section 3-3')

Given:

Ws	z	a		phi		c
(pcf)	(ft)	(degrees)	(radians)	(degrees)	(radians)	(psf)
125	4	35.5	0.619592	30	0.523599	300

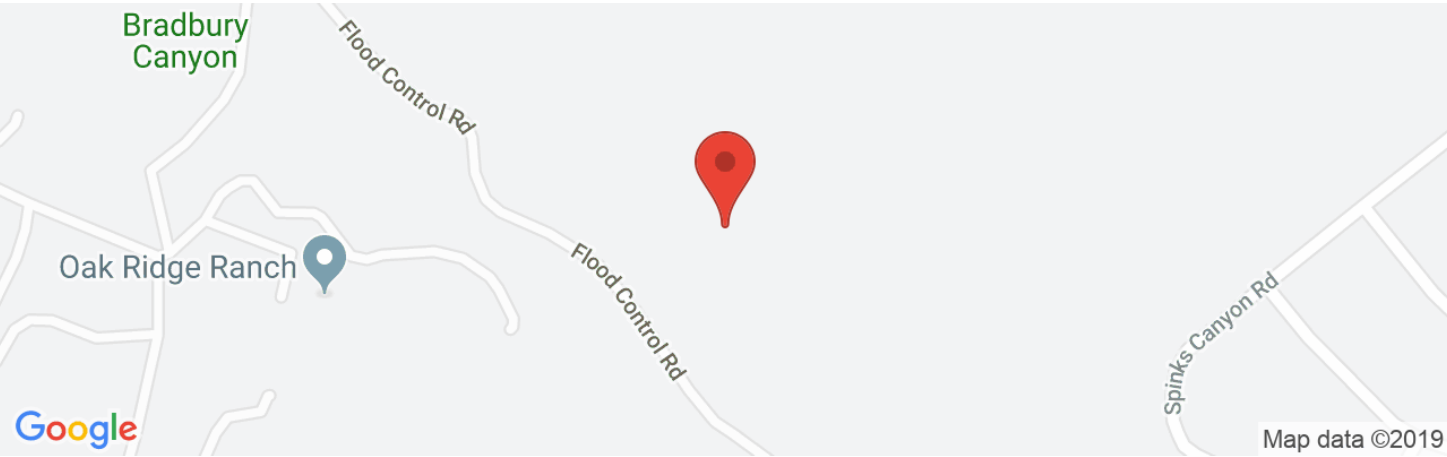
Calculations:

Pw	u	Fd	Fr	**FS**
2.65	165.43	236.38	395.82	**1.67**

Job No. 17-219

The Chadwick Ranch

Latitude, Longitude: 34.154331, -117.962452

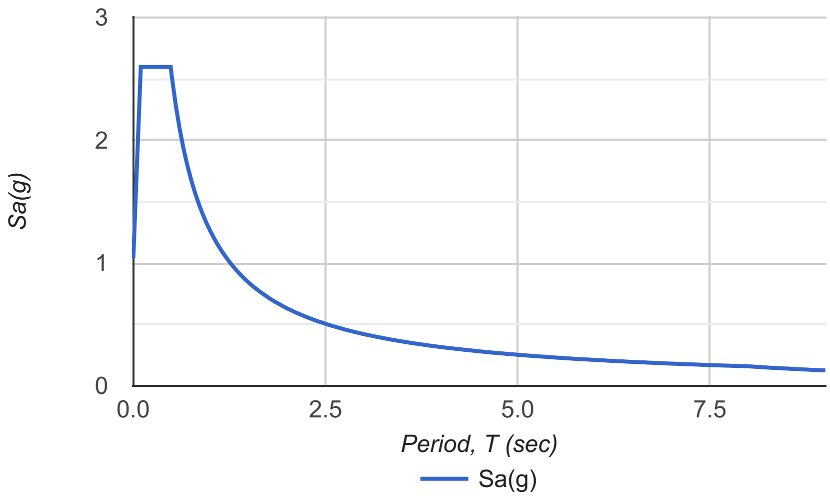


Date	10/4/2019, 5:06:22 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

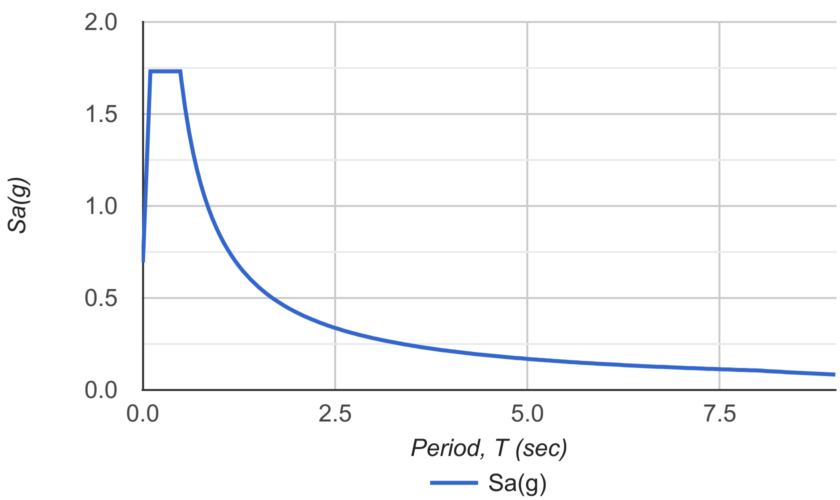
Type	Value	Description
S_S	2.596	MCE_R ground motion. (for 0.2 second period)
S_1	0.97	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.596	Site-modified spectral acceleration value
S_{M1}	1.261	Site-modified spectral acceleration value
S_{DS}	1.731	Numeric seismic design value at 0.2 second SA
S_{D1}	0.84	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.3	Site amplification factor at 1.0 second
PGA	0.972	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.972	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.596	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.707	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.95	Factored deterministic acceleration value. (0.2 second)
S1RT	0.97	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.022	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.244	Factored deterministic acceleration value. (1.0 second)
PGAd	1.134	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.959	Mapped value of the risk coefficient at short periods
C_{R1}	0.949	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum



U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Latitude

Decimal degrees

34.154331

Longitude

Decimal degrees, negative values for western longitudes

-117.962452

Site Class

537 m/s (Site class C)

Spectral Period

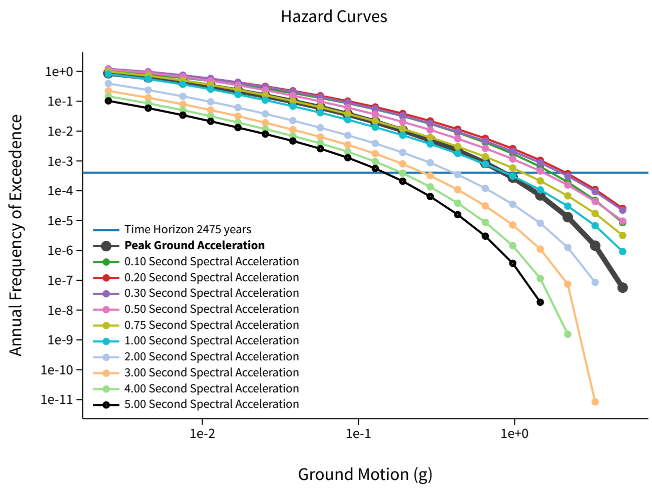
Peak Ground Acceleration

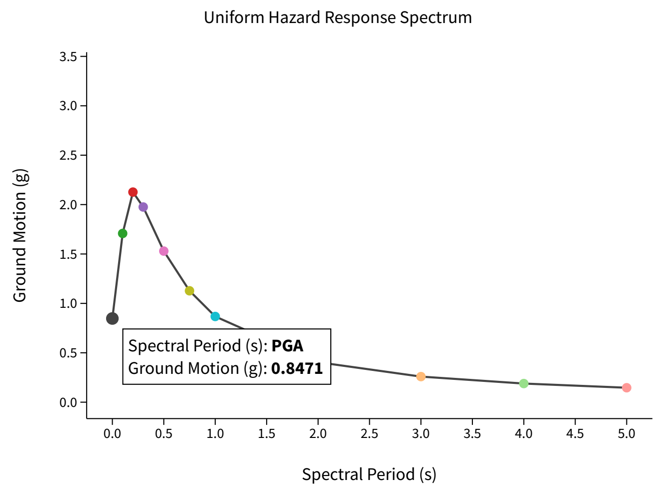
Time Horizon

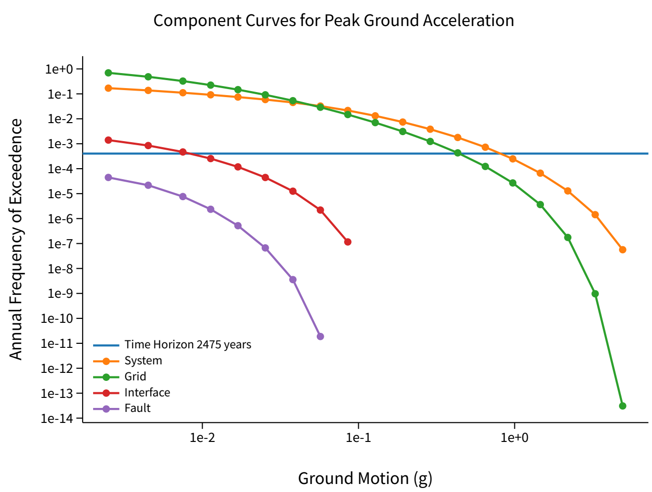
Return period in years

2475

Hazard Curve





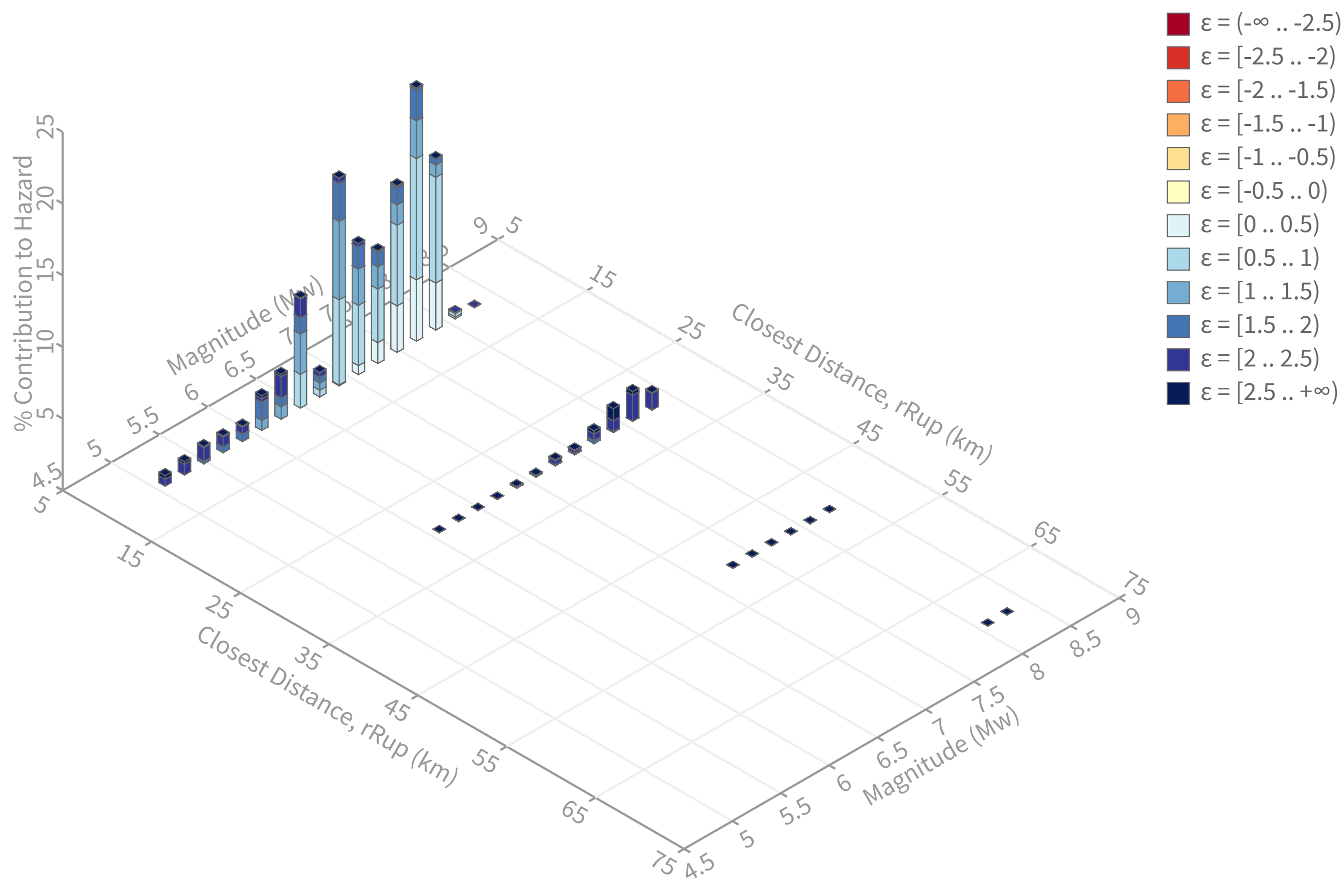


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.84711392 g

Recovered targets

Return period: 2798.449 yrs
Exceedance rate: 0.0003573408 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.07 %

Mean (over all sources)

m: 7.22
r: 7.88 km
ε_0: 1.16 σ

Mode (largest m-r bin)

m: 7.7
r: 5.56 km
ε_0: 0.82 σ
Contribution: 17.66 %

Mode (largest m-r-ε_0 bin)

m: 7.7
r: 2.64 km
ε_0: 0.74 σ
Contribution: 8.46 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							46.09
Sierra Madre [2]		1.87	7.64	0.59	117.961°W	34.165°N	6.24	15.39
Raymond [0]		3.28	7.04	0.84	117.991°W	34.166°N	295.76	12.41
Puente Hills (LA) [0]		14.78	7.14	1.36	118.116°W	33.990°N	217.92	3.40
San Andreas (Mojave S) [10]		33.32	8.07	2.39	117.802°W	34.423°N	26.25	2.47
Elysian Park (Upper) [0]		15.07	7.10	1.76	118.097°W	34.077°N	235.28	2.02
Puente Hills (Coyote Hills) [1]		16.20	7.32	1.55	118.044°W	33.915°N	195.83	1.82
San Jose [2]		14.36	7.08	1.50	117.881°W	34.043°N	148.78	1.29
Clamshell-Sawpit [1]		5.40	7.36	1.02	117.997°W	34.192°N	322.85	1.17
San Gabriel (Extension) [4]		11.36	7.46	1.64	117.967°W	34.256°N	357.66	1.14
UC33brAvg_FM31	System							43.75
Sierra Madre [2]		1.87	7.63	0.59	117.961°W	34.165°N	6.24	15.48
Raymond [0]		3.28	7.01	0.86	117.991°W	34.166°N	295.76	11.66
San Andreas (Mojave S) [10]		33.32	8.07	2.39	117.802°W	34.423°N	26.25	2.47
Elysian Park (Upper) [0]		15.07	6.52	2.05	118.097°W	34.077°N	235.28	2.46
Puente Hills [1]		15.67	7.42	1.55	117.967°W	33.944°N	181.06	2.02
Sierra Madre [3]		2.75	7.55	0.64	117.980°W	34.170°N	317.18	1.50
Clamshell-Sawpit [1]		5.40	7.26	1.06	117.997°W	34.192°N	322.85	1.23
San Gabriel (Extension) [4]		11.36	7.57	1.58	117.967°W	34.256°N	357.66	1.15
San Jose [2]		14.36	7.11	1.57	117.881°W	34.043°N	148.78	1.11
UC33brAvg_FM32 (opt)	Grid							5.10
PointSourceFinite: -117.962, 34.213		7.79	5.83	1.86	117.962°W	34.213°N	0.00	1.39
PointSourceFinite: -117.962, 34.213		7.79	5.83	1.86	117.962°W	34.213°N	0.00	1.39
UC33brAvg_FM31 (opt)	Grid							5.06
PointSourceFinite: -117.962, 34.213		7.81	5.82	1.87	117.962°W	34.213°N	0.00	1.41
PointSourceFinite: -117.962, 34.213		7.81	5.82	1.87	117.962°W	34.213°N	0.00	1.41

APPENDIX III

HOMEOWNER'S MAINTENANCE AND IMPROVEMENT CONSIDERATIONS

HOMEOWNER'S MAINTENANCE AND IMPROVEMENT CONSIDERATIONS

Expansive Soils

Some of the earth materials, which may be moved as part of the site grading, may be identified as being expansive in nature. As such, these materials are susceptible to large volume changes upon variations in their moisture content. These soils will swell upon the introduction of water and shrink upon drying. The forces associated with these volume changes can have significant negative impacts (in the form of differential movement) on foundations, walkways, and other lot improvements.

Homeowners purchasing property and living in an area containing expansive soils must assume a certain degree of responsibility for homeowner improvements and for maintaining conditions around their home. Provisions should be incorporated into the design and construction of homeowner improvements to account for the expansive nature of the on-site soils material. Lot maintenance and landscaping should also be conducted in consideration of expansive soil characteristics. Of primary importance is minimizing the moisture variation below all lot improvements. Such design, construction and homeowner maintenance provisions may include:

- Employing contractors for homeowner improvements who design and build in recognition of local building code and specific site soils conditions.
- Establishing and maintaining positive drainage away from all foundations, walkways, driveways, patios, and other hardscape improvements.
- Avoiding the construction of raised planters adjacent to structural improvements. Alternatively, planter sides/bottoms can be sealed with an impermeable membrane and drained away from the improvements via subdrains into approved disposal areas.
- Sealing and maintaining construction/control joints within concrete slabs and walkways to reduce the potential for moisture infiltration into the subgrade soils.
- Utilizing landscaping schemes with vegetation that requires minimal watering. Alternatively, watering should be done in a uniform manner as equally as possible on all sides of the foundation, keeping the soil "moist" but not allowing the soil to become saturated.
- Maintaining positive drainage away from structures.
- Roof gutters are considered an effective means of drainage. The roof gutters, if installed, should be outletted in such a way that positive drainage away from all structures and planters is maintained.
- Avoiding the placement of trees closer to the proposed structures than a distance of one-half the mature height of the tree. Alternate placement of trees (closes to the structures) may be performed based on recommendations from a qualified landscape architect.
- Observation of the soil conditions around the perimeter of the structure during extremely hot/dry or unusually wet weather conditions so that modifications can be made in irrigation programs to maintain relatively constant moisture conditions.

Sulfates

Homeowners and/or residents should be cautioned against the import and use of certain inorganic fertilizers, soil amendments, and/or other soils from offsite sources in the absence of specific information relating to their chemical composition. Some fertilizers have been known to leach sulfate compounds into soils otherwise containing "negligible" sulfate concentrations and increase the sulfate concentrations in near-surface soils to significant levels. In some cases, concrete improvements constructed in soils containing high levels of soluble sulfates may be affected by deterioration and loss of strength.

Site Drainage

The homeowners and/or residents should be made aware of the potential problems, which may develop when drainage is altered through construction of retaining walls, swimming pools, paved walkways, patios, etc. Ponded water, drainage over the slope face, leaking irrigation systems, overwatering or other conditions that could lead to ground saturation must be avoided.

- No water should be allowed to flow over the slopes. No alteration of pad gradients should be allowed that will prevent pad and roof runoff from being directed to approved disposal areas.

- As part of site maintenance by the homeowners and/or residents, all roof and pad drainage should be directed away from slopes and around structures to approved disposal areas. All berms constructed and compacted as part of fine grading and should be maintained by the resident. The recommended drainage patterns established at the time of the fine grading should be maintained throughout the life of the structure. No alterations to these drainage patterns should be made unless designed by qualified professionals in compliance with local code requirements.

Slope Drainage

The homeowners and/or residents should be made aware of the importance of maintaining and cleaning all interceptors' ditches, drainage terraces, downdrains and any other drainage devices installed to promote slope stability. Backdrain and subdrain outlet pipes, that may protrude through slope surfaces at the completion of grading operations, are designed to conduct subsurface water away from compacted fill sections and buttress/ stabilization fills. These pipes, in conjunction with the graded features, are designed to promote project stability and must be protected in-place and not altered or damaged in any way.

Planting and Irrigation

Seeding and planting of the slopes should be planned to achieve, as rapidly as possible, a well-established and deep-rooted vegetal cover requiring minimal watering. It should be the responsibility of the landscape architect to provide such plants initially and of the residents to maintain such planting. Alteration of such a planting scheme is at the resident's risk. The homeowners and/or residents are responsible for proper irrigation and for maintenance and repair of properly installed irrigation systems. Leaks should be fixed immediately.

Burrowing Animals

The homeowners and/or residents must undertake a program to eliminate burrowing animals. This must be an ongoing program in order to promote slope stability.

Homeowner Improvements

Homeowner and/or resident improvements (pools, spas, patio slabs, retaining walls, planters, etc.) should be designed to account for the nature of the project. Design considerations on any given lot may need to include provisions for differential bearing materials, ascending/descending slope conditions, perched (irrigation) water, special surcharge loading conditions, and long-term creep/settlement.

All homeowner and/or resident improvements should be designed and constructed by qualified professionals utilizing appropriate design methodologies which account for the on-site soils and geologic conditions. Each lot and proposed improvement should be evaluated on an individual basis.

APPENDIX IV

EARTHWORK SPECIFICATIONS AND TYPICAL GRADING DETAILS

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for projects on which Petra Geosciences, Inc. (Petra) is the geotechnical consultant. No deviation from these specifications will be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Soils Engineer and Engineering Geologist of record (Geotechnical Consultant).

I. GENERAL

A. The Geotechnical Consultant is the Owner's or Builder's representative on the project. For the purpose of these specifications, participation by the Geotechnical Consultant includes that observation performed by any person or persons employed by, and responsible to, the licensed Soils Engineer and Engineering Geologist signing the soils report.

B. The contractor should prepare and submit to the Owner and Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" and the estimated quantities of daily earthwork to be performed prior to the commencement of grading. This work plan should be reviewed by the Geotechnical Consultant to schedule personnel to perform the appropriate level of observation, mapping, and compaction testing as necessary.

C. All clearing, site preparation, or earthwork performed on the project shall be conducted by the Contractor in accordance with the recommendations presented in the geotechnical report and under the observation of the Geotechnical Consultant.

D. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Consultant and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Consultant. The Contractor shall also remove all material considered unsatisfactory by the Geotechnical Consultant.

E. It is the Contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction to project specifications. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

F. After completion of grading a report will be submitted by the Geotechnical Consultant.

II. SITE PREPARATION

A. Clearing and Grubbing

1. All vegetation such as trees, brush, grass, roots, and deleterious material shall be disposed of offsite. This removal shall be concluded prior to placing fill.

2. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, etc., are to be removed or treated in a manner prescribed by the Geotechnical Consultant.

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.

3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide sufficient survey control for determining locations and elevations of processed areas, keys, and benches.

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

IV. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

B. Oversize Materials

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.

D. Import

If importing of fill material is required for grading, proposed import material should meet the requirements of the previous section. The import source shall be given to the Geotechnical Consultant at least 2 working days prior to importing so that appropriate tests can be performed and its suitability determined.

V. FILL PLACEMENT AND COMPACTION

A. Fill Layers

Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed 6 inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

B. Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly above optimum moisture content.

C. Compaction

Each layer shall be compacted to 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557-02, will be used.)

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soils condition, the area to received fill compacted to less than 90 percent shall either be delineated on the grading plan or appropriate reference made to the area in the soils report.

D. Failing Areas

If the moisture content or relative density varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

E. Benching

All fills shall be keyed and benched through all topsoil, colluvium, alluvium or creep material, into sound bedrock or firm material where the slope receiving fill exceeds a ratio of 5 horizontal to 1 vertical, in accordance with the recommendations of the Geotechnical Consultant.

VI. SLOPES

A. Fill Slopes

The contractor will be required to obtain a minimum relative compaction of 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure that produces the required compaction.

B. Side Hill Fills

The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the soils report. (See detail on Plate SG-5.)

C. Fill-Over-Cut Slopes

Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soils prior to placing fill. (see detail on Plate SG-6).

D. Landscaping

All fill slopes should be planted or protected from erosion by other methods specified in the soils report.

E. Cut Slopes

1. The Geotechnical Consultant should observe all cut slopes at vertical intervals not exceeding 10 feet.

2. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be evaluated by the Geotechnical Consultant, and recommendations shall be made to treat these problems (Typical details for stabilization of a portion of a cut slope are given in Plates SG-2 and SG-3.).

3. Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.

4. Unless otherwise specified in the soils and geological report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.

5. Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.

VII. GRADING OBSERVATION

A. General

All cleanouts, processed ground to receive fill, key excavations, subdrains, and rock disposals must be observed and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas are ready.

B. Compaction Testing

Observation of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading. Location and frequency of tests shall be at the Consultants discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations may be selected to verify adequacy of compaction levels in areas that are judged to be susceptible to inadequate compaction.

C. Frequency of Compaction Testing

In general, density tests should be made at intervals not exceeding 2 feet of fill height or every 1000 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

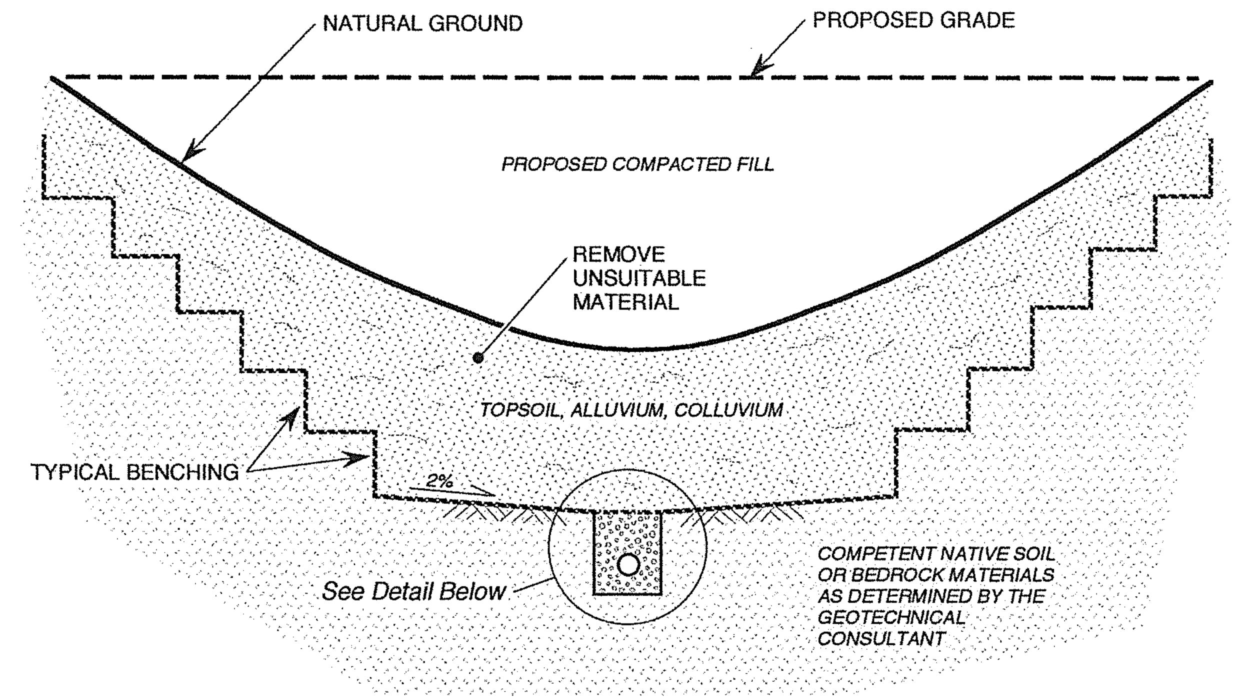
VIII. CONSTRUCTION CONSIDERATIONS

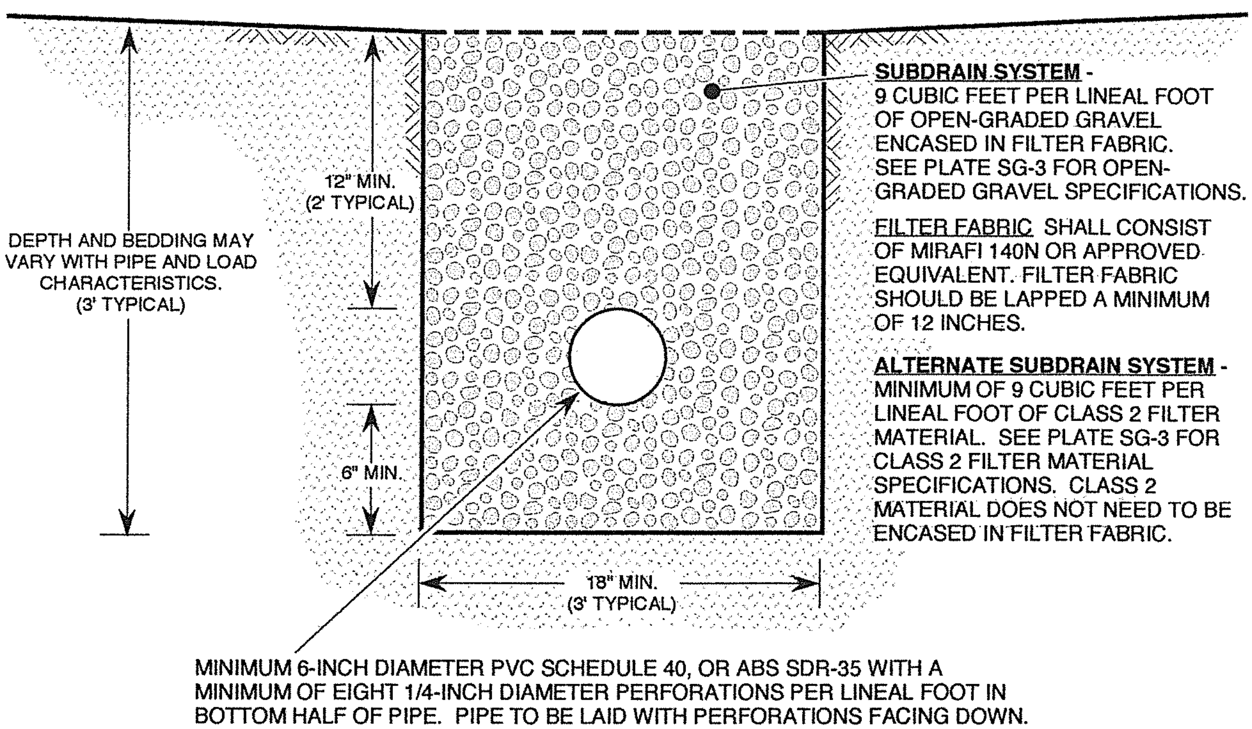
A. Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.

S:\!BOILERS-WORK\REPORT INSERTS\STANDARD GRADING SPECS

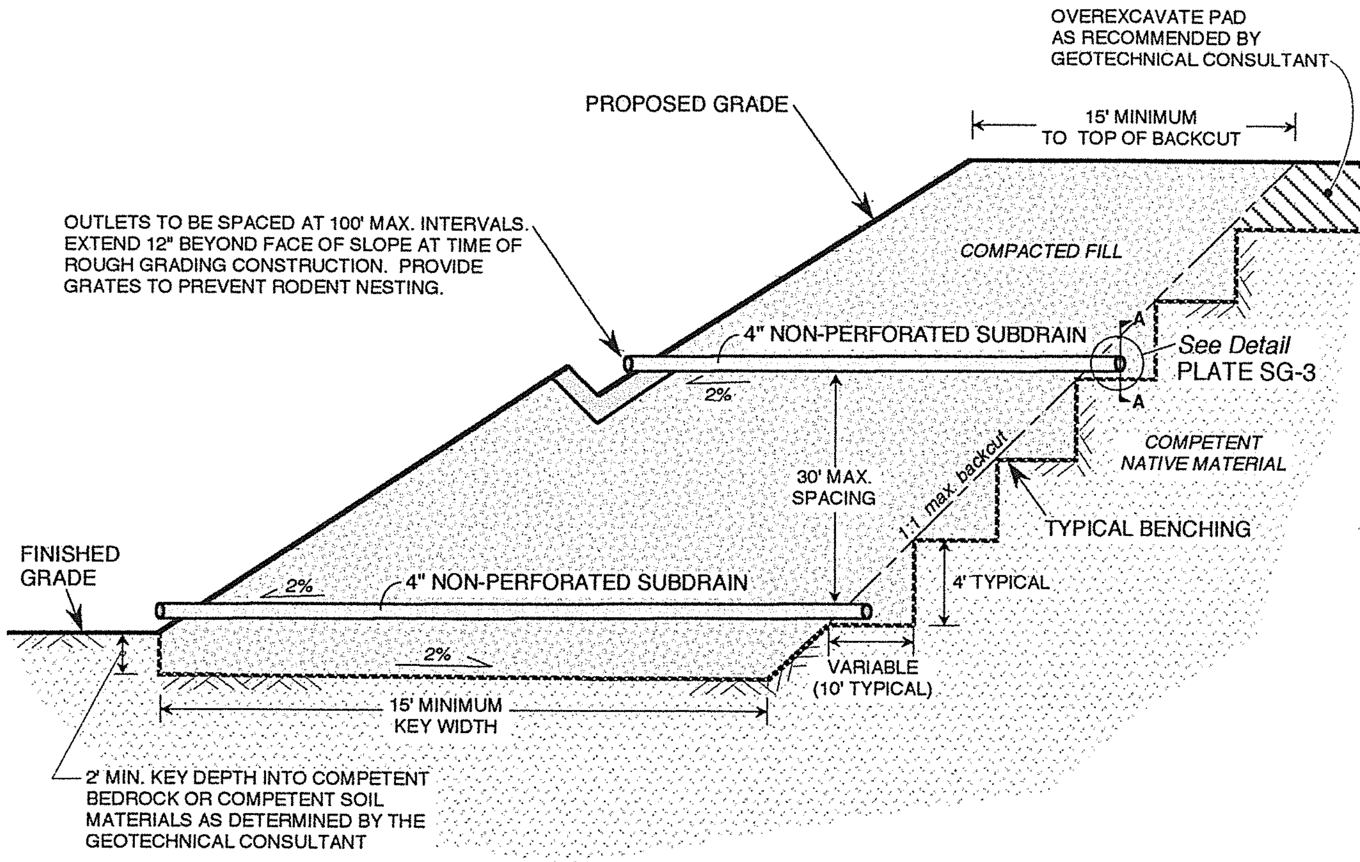




NOTES:

1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.

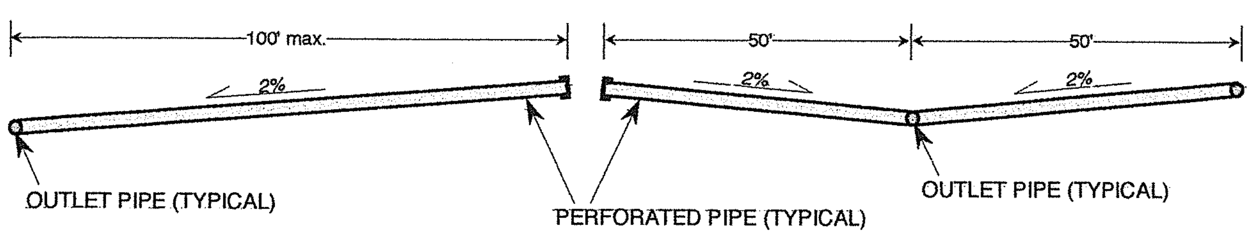
PETRA	CANYON SUBDRAIN DETAIL	PLATE SG-1



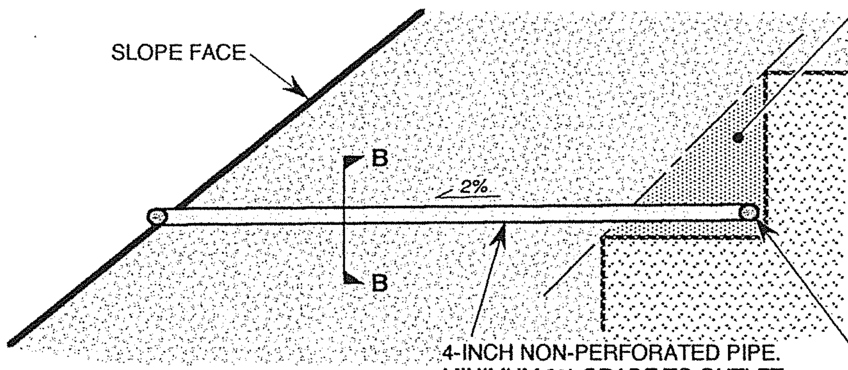
NOTES:

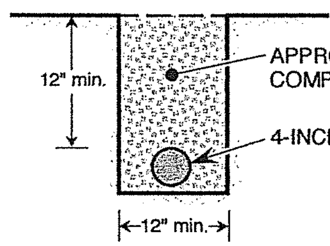
1. 30' MAXIMUM VERTICAL SPACING BETWEEN SUBDRAIN SYSTEMS.
2. 100' MAXIMUM HORIZONTAL DISTANCE BETWEEN NON-PERFORATED OUTLET PIPES. *(See Below)*
3. MINIMUM GRADIENT OF 2% FOR ALL PERFORATED AND NON-PERFORATED PIPE.

SECTION A-A (PERFORATED PIPE PROFILE)



PETRA	BUTTRESS OR STABILIZATION FILL DETAIL	PLATE SG-2





SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:

1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:

OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFI 140N OR EQUIVALENT)

OPEN-GRADED GRAVEL

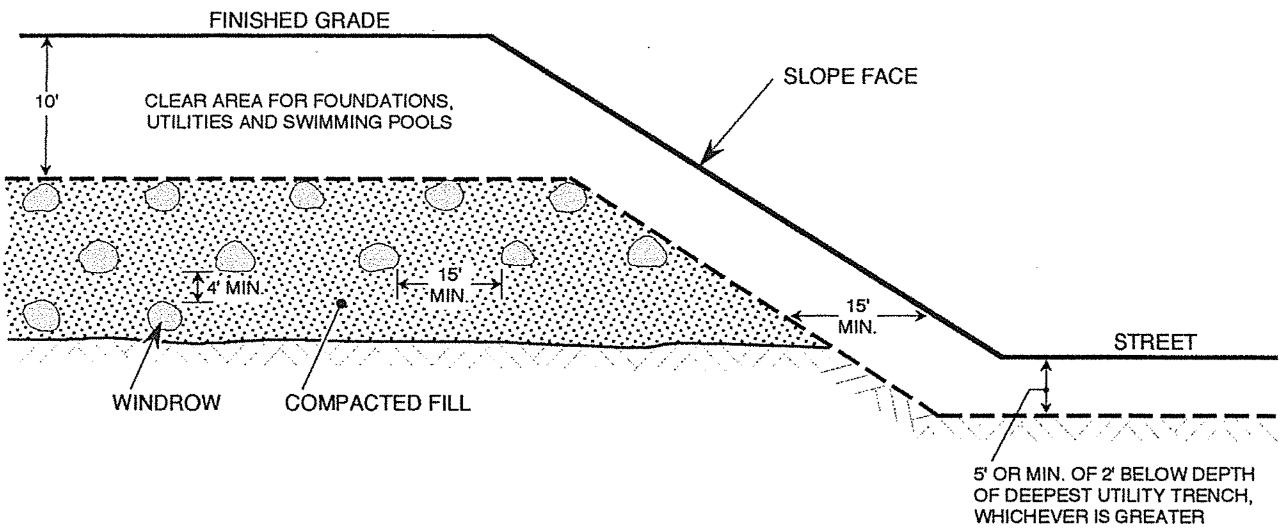
SIEVE SIZE	PERCENT PASSING
1 1/2-INCH	88 - 100
1-INCH	5 - 40
3/4-INCH	0 - 17
3/8-INCH	0 - 7
No. 200	0 - 3

ALTERNATE:

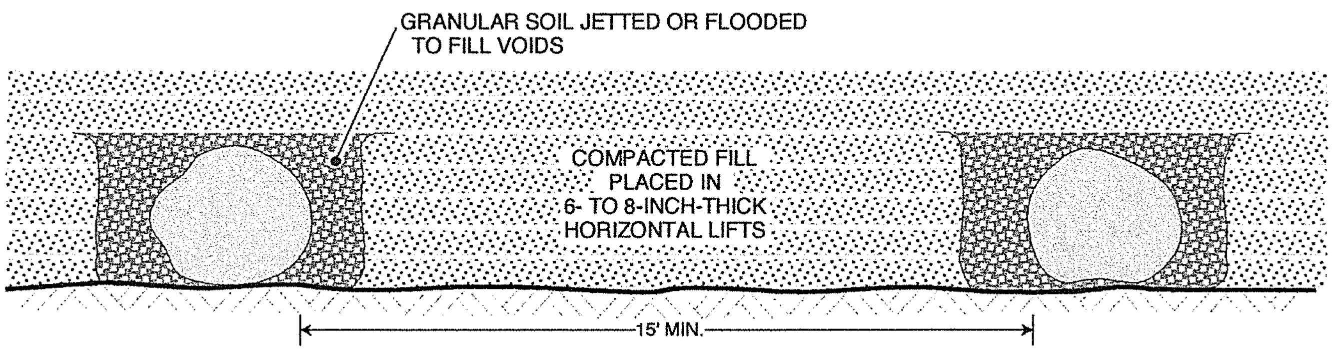
CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS STANDARD SPECIFICATION 68-1.025.

CLASS 2 FILTER MATERIAL

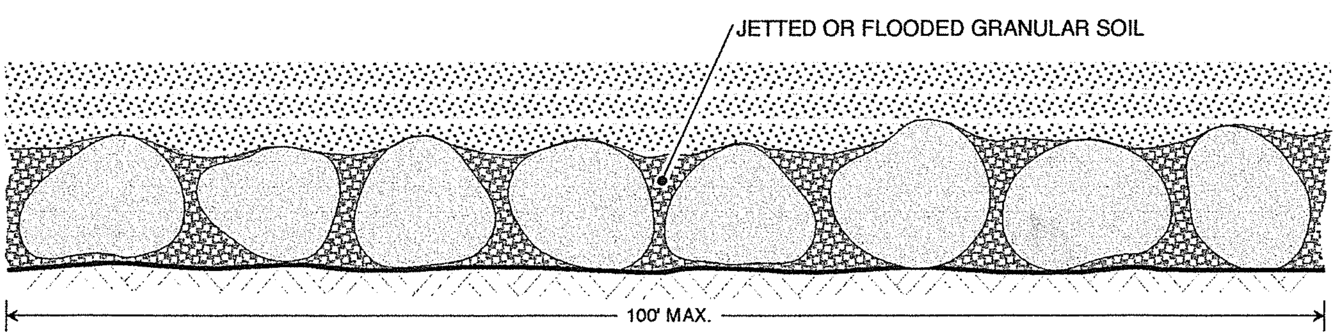
SIEVE SIZE	PERCENT PASSING
1-INCH	100
3/4-INCH	90 - 100
3/8-INCH	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. -30	5 - 15
No. -50	0 - 7
No. 200	0 - 3



TYPICAL WINDROW DETAIL (END VIEW)

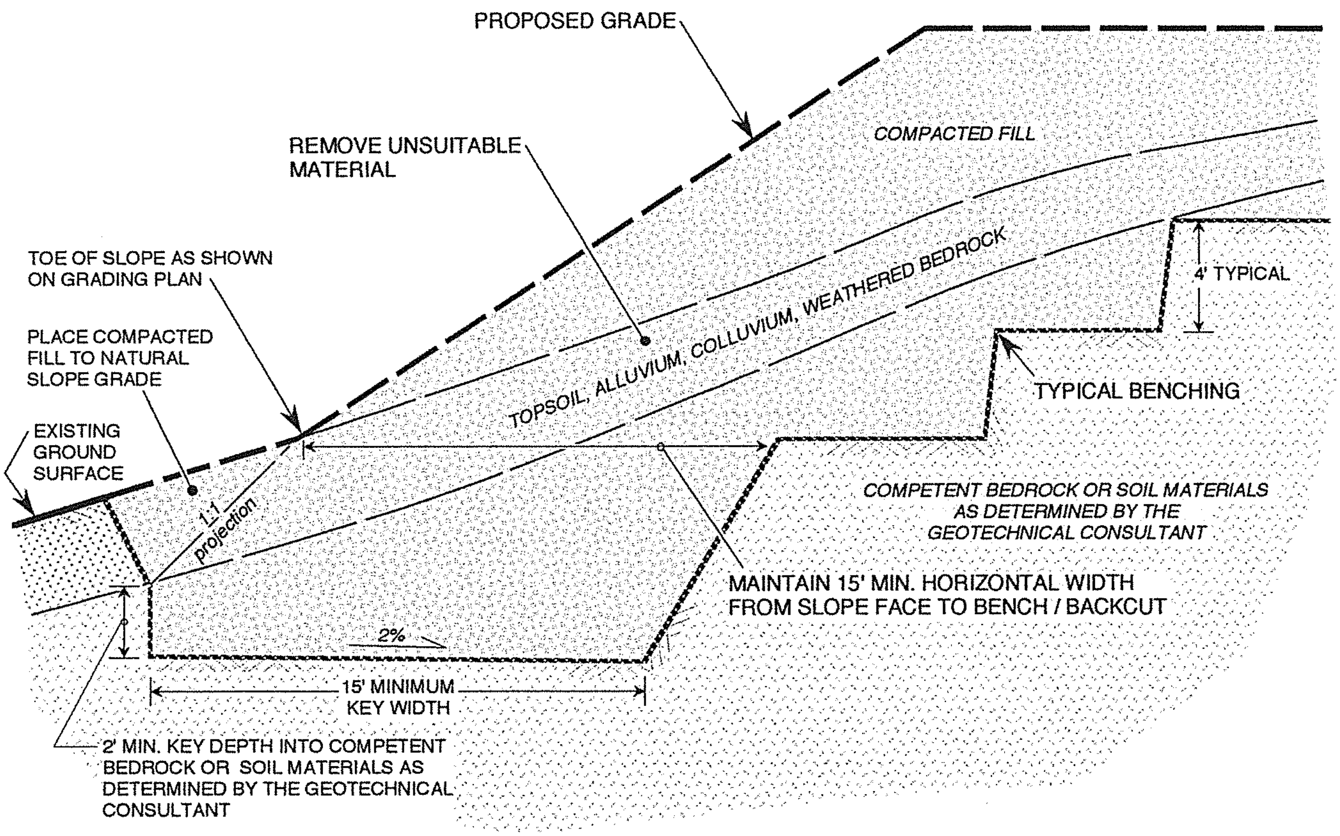


TYPICAL WINDROW DETAIL (PROFILE VIEW)



NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER

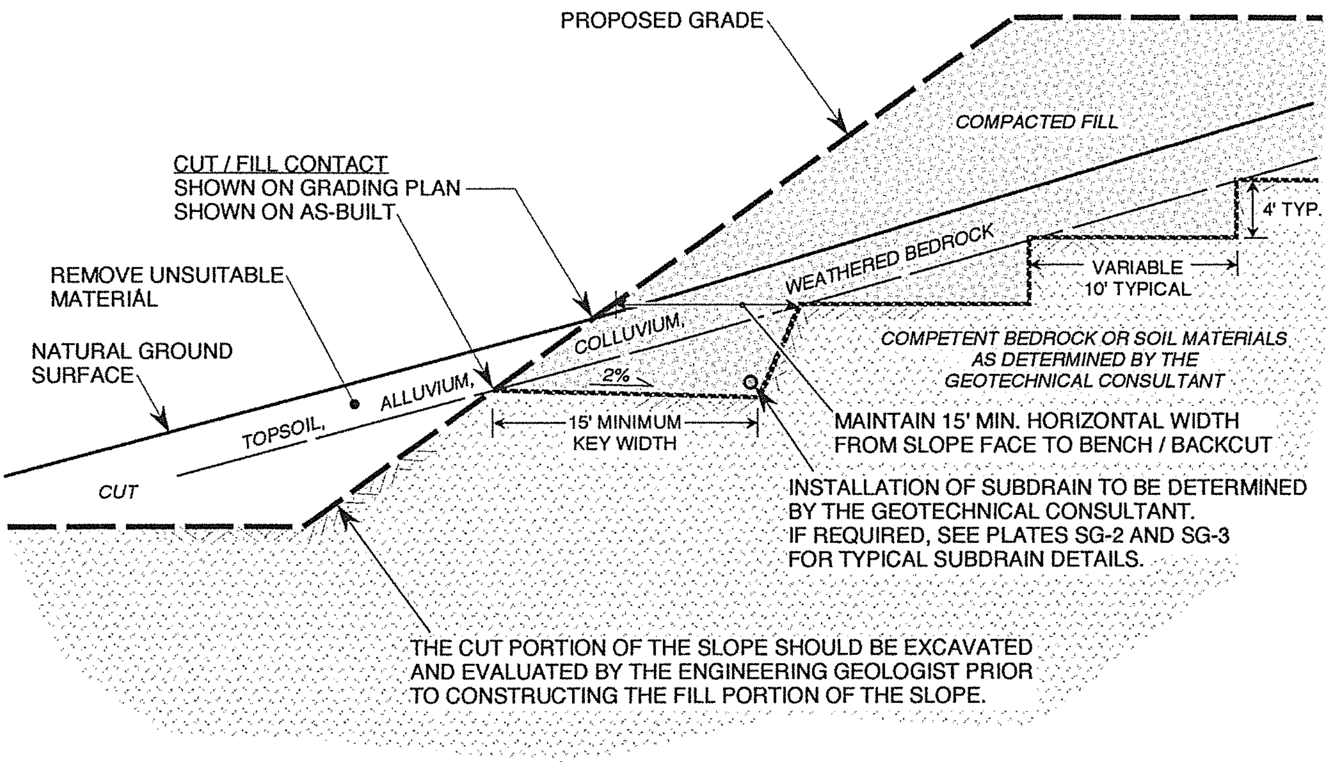
PETRA	TYPICAL ROCK DISPOSAL DETAIL	PLATE SG-4



NOTES:

1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.

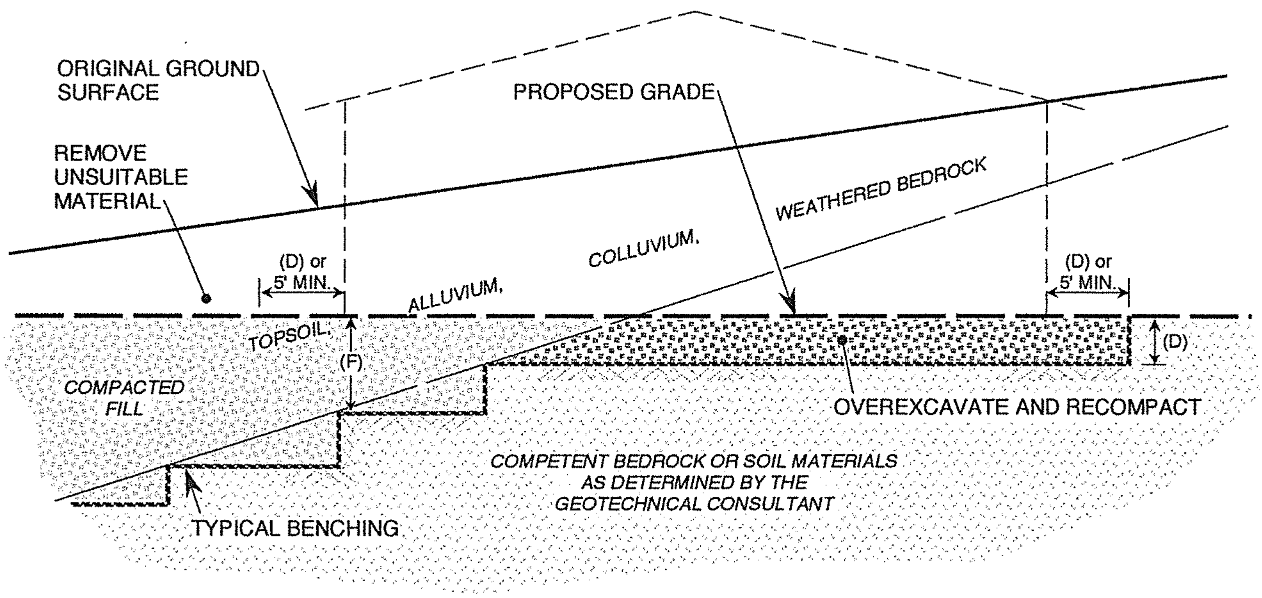
PETRA	FILL SLOPE ABOVE NATURAL SLOPE	PLATE SG-5



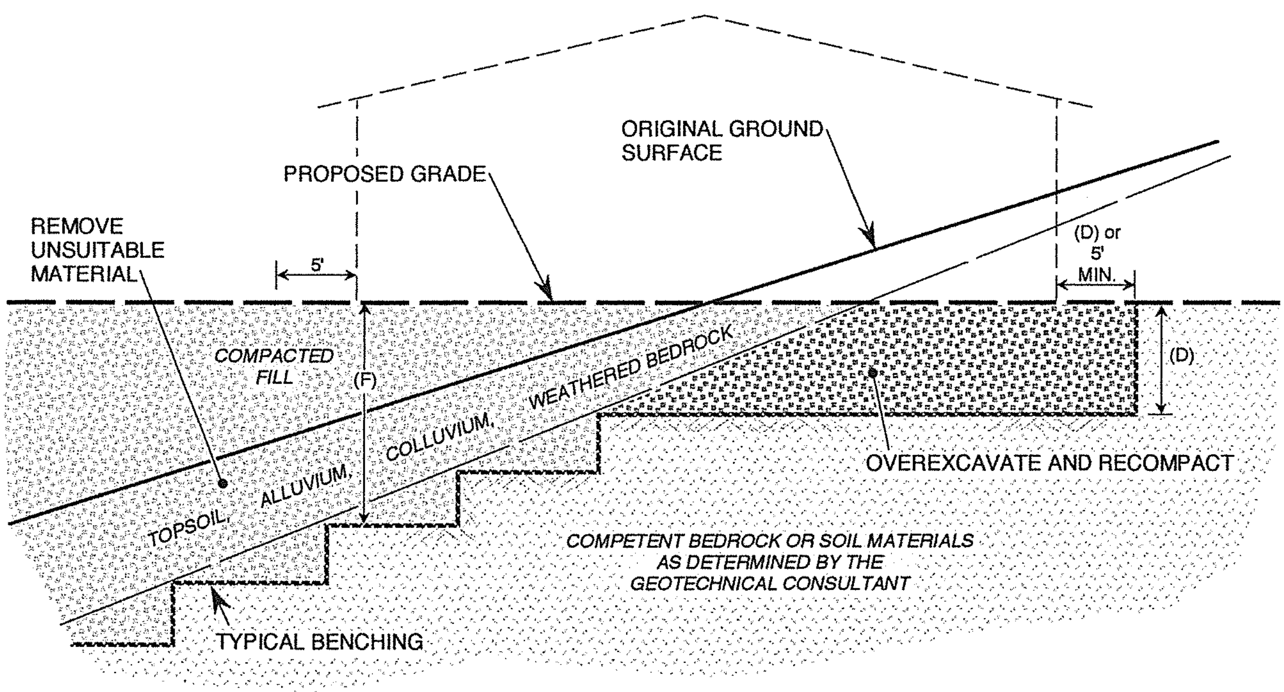
PETRA	FILL SLOPE ABOVE CUT SLOPE	PLATE SG-6

CUT LOT

UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD

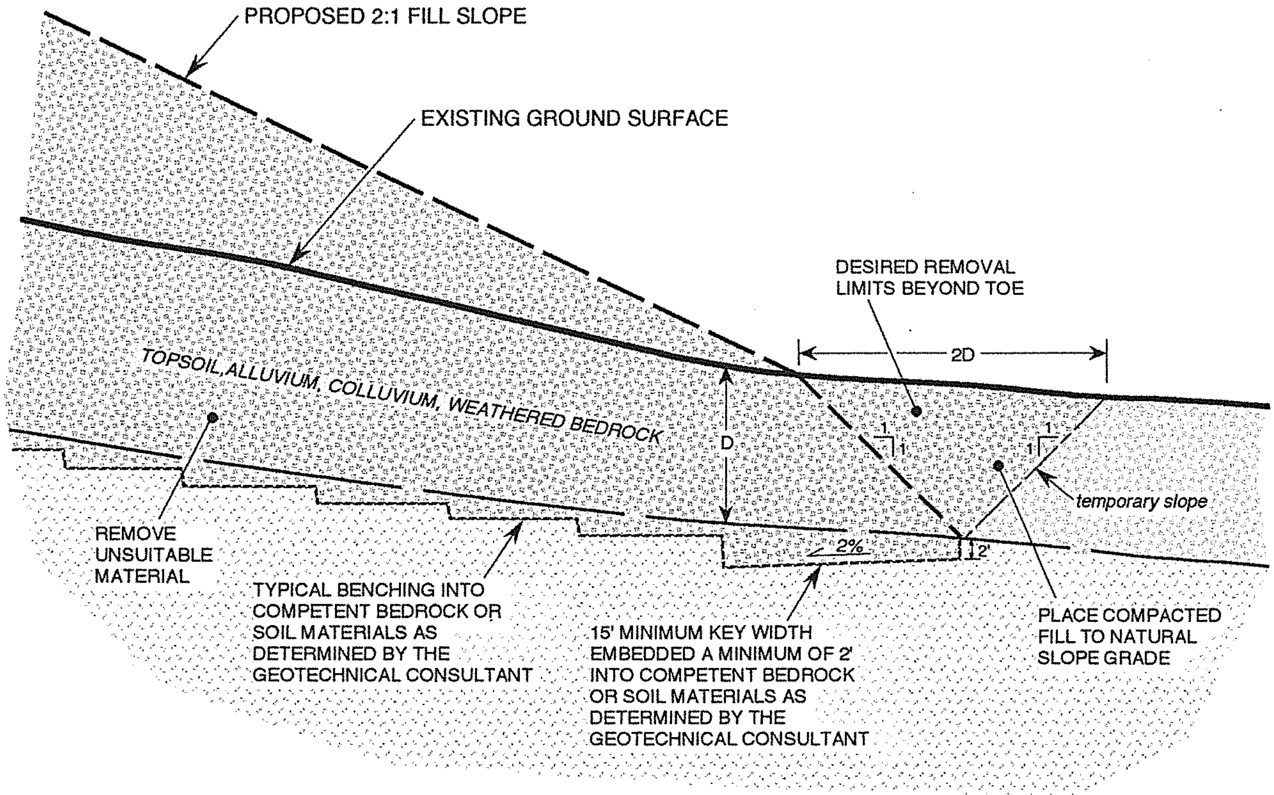


CUT-FILL TRANSITION LOT



<u>MAXIMUM FILL THICKNESS (F)</u>	<u>DEPTH OF OVEREXCAVATION (D)</u>
FOOTING DEPTH TO 3 FEET	EQUAL DEPTH
3 TO 6 FEET	3 FEET
GREATER THAN 6 FEET	1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM

PETRA	CUT LOTS AND CUT-FILL TRANSITION LOTS	PLATE SG-7

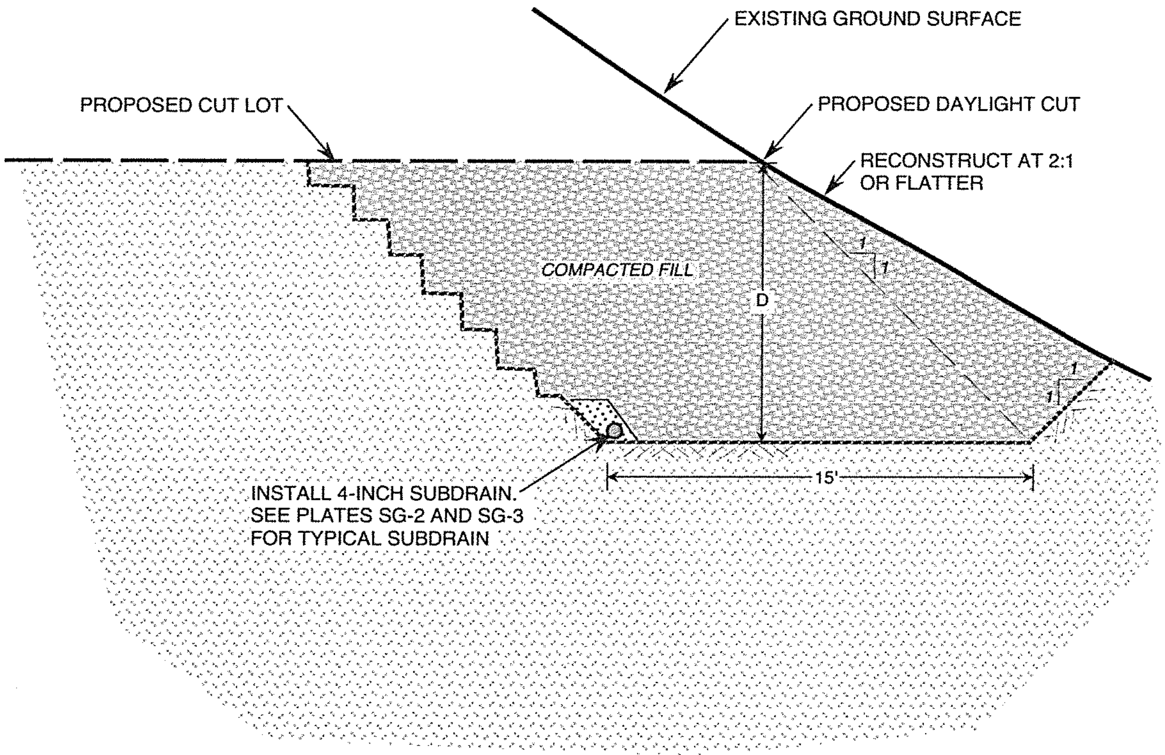


D = RECOMMENDED DEPTH OF REMOVAL
PER GEOTECHNICAL REPORT

PETRA

TYPICAL REMOVALS BEYOND TOE OF PROPOSED FILL SLOPE

PLATE SG-8



NOTE:

1. "D" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA	SHEAR KEY ON DAYLIGHT CUT LOTS	PLATE SG-9

TENTATIVE TRACT MAP No. 82349
CITY OF BRADBURY
TTM 73567
APN 8527-005-008
LEGEND
Af FILL (FLOOD CONTROL)
Qal ALLUVIUM
Qsd SAN DIMAS FM
qd QUARTZ DIORITE
GEOLOGIC CONTACT (DOTTED WHERE BURIED)
SEISMIC REFRACTION LINE
FLIGHT AUGER BORING
TEST PIT
STABILIZATION FILL
SUBDRAIN
TRANSITION LOT
CAP LOT
ESTIMATED DEPTH OF REMOVAL
CROSS SECTION
LOT C
PROPOSED RESERVOIR SITE
PAD=1230
LOT L
OPEN SPACE
LOT B
OPEN SPACE
RESERVOIR ACCESS
LOT D
OPEN SPACE
LOT A
PRIVATE STREET
LOT K
OPEN SPACE
LOT E
DESILTING BASIN No 1
BOT=993
LOT F
OPEN SPACE
LOT G
DESILTING BASIN No 2
BOT=903
LOT J
OPEN SPACE
LOT I
DETENTION BASIN
BOT=834
LOT H
OPEN SPACE
LOT M
OPEN SPACE
PAD=1081
PAD=1100
PAD=1024
PAD=1005
PAD=913
PAD=964
PAD=934
PAD=928
PAD=900
BRADBURY DEBRIS BASIN
APN 8527-005-902
SPINKS DEBRIS DISPOSAL AREA
APN 8527-005-904
SINGLE FAMILY RESIDENTIAL
APN 8527-005-011
APN 8527-005-010
LIMITS OF DISTURBANCE
LONG CANYON ROAD
BLISS CANYON ROAD
SCALE: 1"=80'
CITY OF BRADBURY
CITY OF DUARTE
PETRA GEOSCIENCES, INC.
28358 Constellation Road, Unit 680
Valencia, California 91355
PHONE: (661) 255-5790
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA
GEOTECHNICAL MAP
Chadwick Ranch
City of Bradbury, California
DATE: OCT. 2019
J.N: 17-219
PLATE 1
PROACTIVE
ENGINEERING CONSULTANTS
27042 Towne Centre Drive, Suite 110
Foothill Ranch, CA 92610 (949) 716-7460
MARK S. ANDERSON
DATE
CHADWICK RANCH
TENTATIVE TRACT
MAP No 82349
CONCEPTUAL GRADING PLAN
SHEET 2 OF 3
September 26, 2019