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March 11, 2016
P/W 1406-06
Report No. 1406-06-B-6

**Attention:** **Mr. Kirk Philo**

**Subject:** ***Preliminary Geotechnical Investigation of San Elijo – Questhaven Property, South of San Elijo Road, San Marcos Area, County of San Diego, California***

**References:** **See Appendix A**

Gentlemen:

Pursuant to your request Advanced Geotechnical Solutions, Inc. (AGS) has prepared this preliminary geotechnical investigation of the San Elijo - Questhaven property located south of San Elijo Road in the San Marcos area of San Diego County, California. Based upon our conversations, review of available documents and plans, AGS understands that the project will be developed to support both single- and multi-family residences, as well as commercial/light industrial use.

The recommendations presented herein are based on a review of available geologic and geotechnical literature and maps pertinent to the proposed construction, the review of site specific investigations by others, the results of our recent subsurface exploration at the project site, associated laboratory testing, and our general experience in the area. Included in this report are: 1) engineering characteristics of the onsite soils; 2) discussion of the onsite geologic units; 3) limited geologic hazard analysis; 4) grading recommendations; and 5) geotechnical design recommendations for the proposed building, retaining walls and associated surface improvements.

AGS appreciates the opportunity to provide you with geotechnical consulting services and professional opinions. If you have any questions, please contact the undersigned at (619) 867-0487.

Respectfully Submitted,

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Distribution: (3) Addressee
Attachments:
Figure 1 – Site Location Map
Plate 1– Geologic Map and Exploration Location Plan

***Preliminary Geotechnical Investigation***
***San Elijo - Questhaven Property***
***South of San Elijo Road, San Marcos Area***
***County Of San Diego, California***

**ADVANCED GEOTECHNICAL SOLUTIONS, INC.**

## 1.0 SCOPE OF SERVICES

This study is aimed at providing geotechnical information as it relates to: 1) existing site soil conditions; 2) discussion of the geologic units onsite; 3) limited geologic hazard analysis; 4) engineering characteristics of the onsite soils; 5) excavation characteristics of earth materials; 6) preliminary geotechnical design for the proposed structures and associated improvements.

The scope of our study included the following tasks:

- Review of pertinent published and unpublished geologic and geotechnical literature, maps, and aerial photographs.
- Conduct 4 seismic refraction traverses to evaluate rippability of the onsite bedrock.
- Excavate, log and sample twelve excavator test pits (EX-1 through EX-12) at selected locations within the limits of the proposed development.
- Laboratory testing including maximum density, expansion potential, grain size analysis, and remolded shear strength testing.
- Prepare a plan depicting the onsite geologic contacts and excavator pit locations as well as subsurface locations from previous investigations, utilizing the 60-scale Site Concept Plan prepared by Excel Engineering (Plate 1).
- Conduct a geotechnical and geologic hazard analysis of the site.
- Develop general remedial grading recommendations for unsuitable soils and determine overexcavation recommendations for cut/bedrock areas.
- Evaluate presence of suitable capping materials and rippability of onsite bedrock.
- Conduct a limited seismicity analysis.
- Determine site specific seismic design parameters for use in the structural design.
- Determine design parameters of onsite soils as a foundation medium including bearing and friction values for foundation soils.
- Preparation of this geotechnical report with exhibits summarizing our findings. This report is suitable for design, contractor bidding, and regulatory review.

## 2.0 GEOTECHNICAL STUDY LIMITATIONS

The conclusions and recommendations in this report are professional opinions based on the data developed during this study and previous studies by others.

The materials immediately adjacent to or beneath those observed may have different characteristics than those observed. No representations are made as to the quality or extent of materials not observed. Any evaluation regarding the presence or absence of hazardous material is beyond the scope of this firm's services.

## 3.0 SITE LOCATION AND DESCRIPTION

The roughly rectangular shaped site (Figure 1) encompasses approximately 69 acres and is located south of San Elijo Road approximately one mile east of Ranch Santa Fe Road in the San Marcos area of San Diego County, California. The overall site is bounded to the north by San Elijo Road, to the west and south by open space, and to the east by the old San Marcos Landfill. In general, the site is characterized by a topographic saddle in the northerly/northeasterly portion with relatively broad low-relief drainages flowing to the northwest and southeast and gently to moderately sloping hillside flanking the saddle to the south and north.

Approximate elevations within the overall site limits range from a low elevation of 490 msl in the southeastern drainage to a high of 930 msl near the southwestern property limit. The property is currently vacant and undeveloped with the exception of a few dirt roads and SDG&E high voltage transmission lines transecting the site. The lower portion of the site is covered with a light to moderate growth of weeds and grass with localized stands of mature trees and bushes. The higher southern portion of the site is covered with a moderate to dense growth of chaparral.

## 4.0 PROPOSED DEVELOPMENT

Current conceptual plans depict a sheet graded configuration with a larger south parcel, two northeast parcels, and a smaller northwest parcel fronting east bound San Elijo Road. It is our understanding that the site will ultimately support a combination of single- and multi-family residences (southerly super pad), as well as commercial and industrial structures (northwest and northeasterly pads). Site concept plans prepared by Excel Engineering indicate the deepest cuts are on the order of 50 feet and the deepest fills are on the order of 40 feet. Cut and fill slopes on site are designed at 2:1 (horizontal to vertical). The highest proposed cut slope is approximately 80 feet tall, and the highest proposed fill slope is approximately 50 feet tall. Access to the site will be afforded by private drives off eastbound San Elijo Road and a central cul-de-sac.

## 5.0 FIELD AND LABORATORY INVESTIGATION

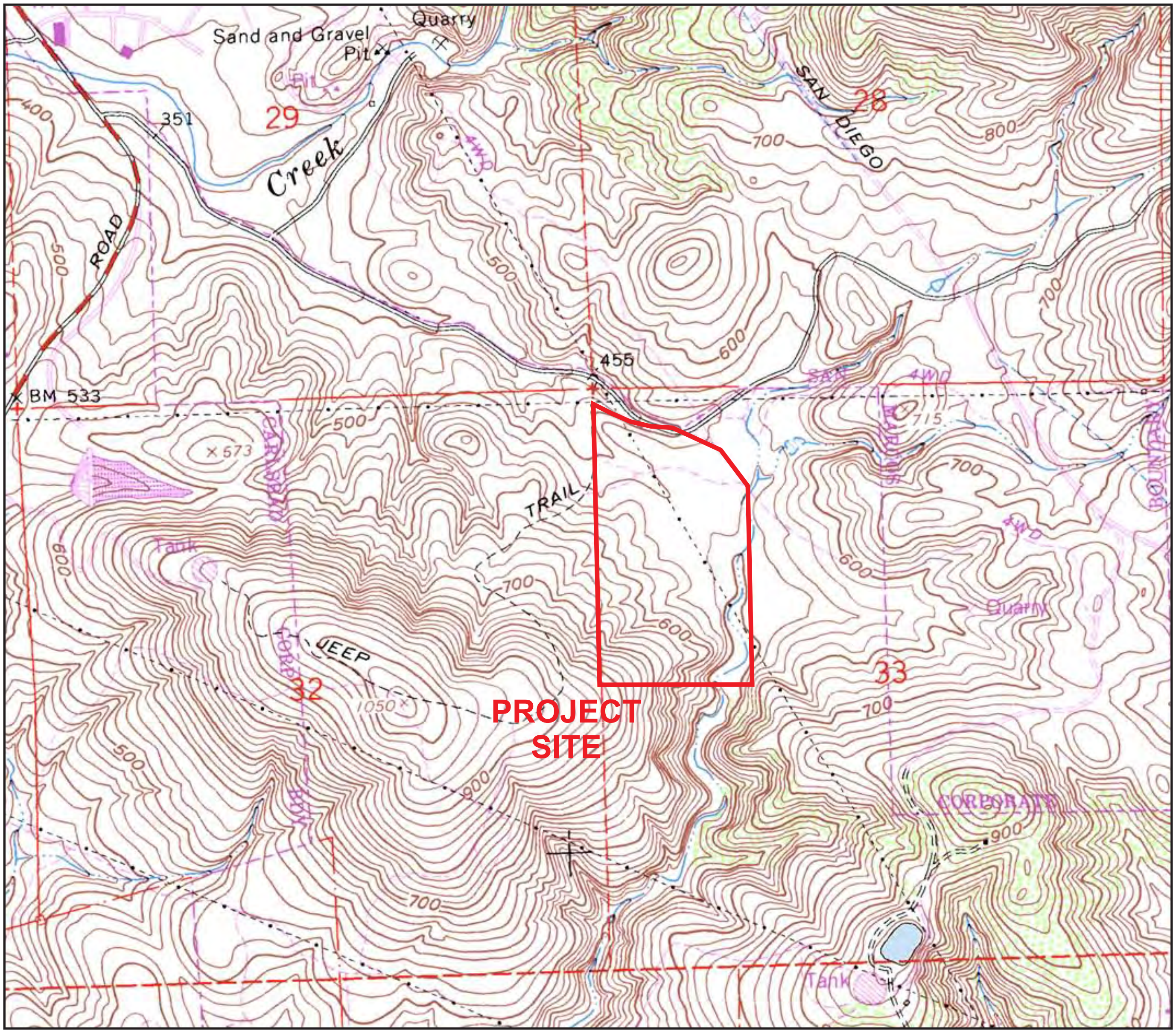
### 5.1. <u>Previous Studies</u>

#### 5.1.1. By Others

A Preliminary Geotechnical Investigation Report was prepared by Geotechnics Inc. in 2003 (Geotechnics, 2003). This study consisted of limited mapping and conducting five seismic refraction surveys. Logs of these seismic refraction surveys are presented in Appendix B. A bulk sample was taken from seismic line 4 for laboratory testing. Lab results are presented in Appendix C. The report also presents conclusions and recommendations for site development as planned at that time

#### 5.1.2. By AGS

As part of a “Due-Diligence” study, AGS conducted a limited subsurface exploration at the site on July 10, 2014 which consisted of the excavation, logging and sampling of eight (8) trackhoe (Cat 328D excavator (88,000lb machine) equipped with a 2 foot

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# SITE LOCATION MAP
# QUESTHAVEN
# SAN MARCOS, CALIFORNIA

P/W 1406-06

FIGURE 1

SOURCE MAP - TOPOGRAPHIC MAP OF THE
RANCHO SANTA FE 7.5 MINUTE QUADRANGLE,
SAN DIEGO COUNTY, CALIFORNIA


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bucket) tests pits. The depths of these test pits ranged from 6.5 to 19 feet. Logs of these excavations are presented in Appendix B.

AGS also conducted an infiltration study in November 2015. As part of this study representatives of AGS excavated and logged four rubber tire backhoe test pits for use in determining the soil lithology. Six (6) infiltration borings were excavated to 4-5 feet below ground surface and infiltration testing was performed. Locations of these excavations are presented in Appendix B. Laboratory testing was beyond the scope of this study.

### 5.2. Current Study

As part of the current study, AGS excavated, logged, and sampled 12 excavator test pits with a Cat 336 excavator (approximate weight of 80,000lb, equipped with a 2 foot bucket). The depths ranged from 7 to 24.5 feet below ground surface. Logs of these test pits are presented in Appendix B.

AGS retained Southwest Geophysics, Inc. to conduct seismic refraction traverses (SL-1 through SL-4) at predetermined locations onsite, as shown on Plate 1. This information was used in the rippability evaluation of the onsite bedrock units and to further define the geometries of the subsurface bedrock units onsite. The report by Southwest Geophysics is presented in Appendix B.

### 5.3. Laboratory Testing

Laboratory testing was conducted on representative bulk samples obtained from the subsurface excavations. Testing consisted of expansion testing, maximum density and optimum moisture content, remolded direct shear strength testing, and chemical/resistivity analyses. The results of laboratory testing are presented in Appendix C.

## 6.0 ENGINEERING GEOLOGY

### 6.1. Regional Geologic and Geomorphic Setting

The subject site is situated within the western portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges province occupies the southwestern portion of California, extending southward from the Transverse Ranges and Los Angeles Basin to the southern tip of Baja California. In general the province consists of young, steeply sloped, northwest trending mountain ranges underlain by Late Jurassic to Early Cretaceous-age metavolcanic and metasedimentary rock and Cretaceous-age igneous plutonic rock of the Peninsular Ranges Batholith. The westernmost portion of the province is predominantly underlain by younger marine and non-marine sedimentary rocks. The Peninsular Ranges' dominant structural feature is northwest-southeast trending crustal blocks bounded by active faults of the San Andreas transform system.

### 6.2. Site Geology

Published regional geologic maps indicate the site is underlain by metamorphic Santiago Peak Volcanics. Geotechnics Inc. (2003) also concluded the site is underlain by Santiago Peak Volcanics based on subsurface information attained through seismic refraction surveys. However, based on information gathered during our previous and recent subsurface explorations, the site is underlain by metamorphic Santiago Peak Volcanics and a sedimentary unit likely associated with the Santiago Formation. These units are mantled by relatively thin veneers of surficial soils including undocumented artificial fill, colluvium and residual soil. The following section contains a summary of the soil and bedrock units encountered onsite. The approximate distribution of these units are shown on Plate 1. Description of these geologic units, as observed during our investigation, are presented below. Test pit logs are presented in Appendix B.

#### 6.2.1. Artificial Fill-undocumented (afu)

Undocumented artificial fill soils were locally encountered in test pit TP-6 to a depth of eight (8) feet. As encountered these materials can generally be described as brown to gray, silty clay in a dry to moist and loose/soft to stiff condition. Based on a review of historical satellite imagery of the project site, it appears that minor grading/mining operations were conducted during the mid-1990's in the lower, central and southeasterly portions of the site.

#### 6.2.2. Alluvium

Alluvium was encountered in several test pits at the southeasterly boundary of the site. As encountered, these materials can generally be described as brown, silty to clayey sand with gravel in a moist and loose condition. The alluvium ranged from 3 to 9 feet in thickness. Alluvium is also anticipated to exist within the northwesterly drainage onsite. Subsurface exploration within this area was precluded due to environmental constraints.

#### 6.2.3. Colluvium (Qcol)

A relatively thin veneer colluvium mantles a majority of the project site and was encountered the majority of the test pits. The colluvium can generally be described as grayish brown/brown to reddish brown, silty to sandy clay in a dry to moist and loose to stiff condition. The colluvium ranged from 3 to 8 feet in thickness.

#### 6.2.4. Santiago Formation (Map Symbol Tsa)

Sedimentary bedrock materials which appear to be related to the Tertiary-aged Santiago Formation were encountered across the site below the surficial units and were observed to non-conformably overlie the Santiago Peak Volcanics. These materials ranged from 3 to 23 feet in thickness. As encountered, these materials can generally be described as gray to greenish gray to light brown, soft to hard, clayey sandstone and claystone.

#### 6.2.5. Santiago Peak Volcanics (Map Symbol Jsp)

Santiago Peak Volcanics were encountered at depth in many of the test pits across the site and are anticipated to underlie the remaining portions of the site beneath the Santiago Formation. The Santiago Peak Volcanics are generally comprised of metavolcaniclastic and metasedimentary bedrock. As encountered, these materials are completely to slightly weathered and moderately hard to very hard, generally reducing to 8-inch minus rock fragments in the highly weathered zones and 12-inch minus in the moderately weathered zones. Some rock fragments greater than 12-inches were encountered. A residual soil horizon on the order of two (2) feet thick locally mantled the intact bedrock in several test pits. Jointing observed within the unit typically ranged from tight to blocky and widened with depth. The excavator encountered refusal in the Santiago Peak Volcanics at depths between 6.5 feet and 19 feet during our due diligence investigation.

## 6.3. Groundwater

Groundwater was not encountered in our exploratory excavations, nor was groundwater observed on site by Geotechnics during their previous study. No natural groundwater condition is known to exist at the site that would impact the proposed site development. Intermittent surface water within the onsite drainages is anticipated during heavy and/or prolonged rain events.

It should be noted that localized perched groundwater may develop at a later date, most likely at or near fill/bedrock contacts, due to fluctuations in precipitation, irrigation practices, or factors not evident at the time of our field explorations.

## 6.4. Geologic Hazards

### 6.4.1. Landslides

No landslides have been mapped at the site. No topographic features were observed at the site that would indicate existing landslides. In addition, given the hard, relatively massive nature of the metavolcanic rock, the potential for landsliding is considered very low.

### 6.4.2. Flooding

According to available FEMA maps, the site is not in a FEMA identified flood hazard area.

### 6.4.3. Subsidence/Ground Fissuring

Due to the presence of the hard underlying metavolcanic rock and the proposed compacted fill that will be placed during grading activities, the potential for subsidence and ground fissuring due to settlement is very low.

## 6.5. Seismic Hazards

The site is located in the tectonically active Southern California area, and will therefore likely experience shaking effects from earthquakes. The type and severity of seismic hazards affecting the site are to a large degree dependent upon the distance to the causative fault, the intensity of the seismic event, and the underlying soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction or dynamic settlement. The following is a site-specific discussion of ground motion parameters, earthquake-induced landslide hazards, settlement, and liquefaction. The purpose of this analysis is to identify potential seismic hazards and propose mitigations, if necessary, to reduce the hazard to an acceptable level of risk.

#### 6.5.1. Surface Fault Rupture

No faults have been mapped within the project site. The nearest known active fault to the site is the Newport-Inglewood-Rose Canyon fault zone which is approximately 9 miles west of the project site. Accordingly, the potential for fault surface rupture on the subject site is very low. This conclusion is based on literature review and aerial photographic analysis.

#### 6.5.2. Seismicity

As noted, the site is within the tectonically active southern California area, and is approximately 9 miles from the Newport-Inglewood-Rose Canyon fault zone. The potential exists for strong ground motion that may affect future improvements.

At this point in time, non-critical structures (commercial, residential, and industrial) are usually designed according to the California Building Code (2013) and that of the controlling local agency.

#### 6.5.3. Liquefaction

Due to the hard metavolcanic rock that underlies the site, and given the lack of shallow groundwater at the site, and the dense compacted fill that will be placed during grading, the potential for liquefaction at the site is very low.

#### 6.5.4. Dynamic Settlement

Dynamic settlement occurs in response to an earthquake event in loose sandy earth materials. Given the fact that hard metavolcanic rock underlies the site, and the proposed removals, the potential for dynamic settlement is considered to be remote.

#### 6.5.5. Seismically Induced Landsliding

Evidence of landsliding at the site was not observed during our field explorations, nor were any geomorphic features indicative of landslides noted during our review of aerial photos and published geologic maps. metavolcanic rock at the site is not usually susceptible to seismically induced landsliding. Therefore, the potential for landslides to impact the proposed development is low.

## 7.0 GEOTECHNICAL ENGINEERING

Presented herein is a general discussion of the geotechnical properties of the various soil types and the analytic methods used in this report.

### 7.1. Excavation Characteristics

Based on our subsurface investigation and previous experience with similar projects near the subject site, it is anticipated that excavations within the undocumented artificial fill, alluvium, colluvium, Santiago Formation, and the completely- to highly-weathered portions of the Santiago Peak Volcanics bedrock can be accomplished with conventional equipment. It is likely that oversized "float" will be encountered in surface outcrops and shallow cuts which will require special handling. Deeper cuts within the Santiago Peak Volcanics will require specialized equipment, such as large excavators equipped with hoe rams, D-9 bulldozer with a single shank ripper and/or blasting to efficiently excavate to the proposed grades and overexcavation grades. In addition, where proposed underground utilities are located in hard rock areas, overexcavation during grading is recommended to facilitate the installation of wet and dry utilities.

#### 7.1.1. Rippability

As part of the current investigation, four seismic refraction surverys were performed (SL-1 through SL-4). Geotechnics, Inc. (2003), previously conducted five seismic refraction surveys (S-1 through S-5). Copies of the seismic refraction surveys are included in Appendix B. Approximate locations of the survey lines are shown on Plate 1.

SL-1 had primary compression wave velocities greater than 5,000 feet/second at depths below 15 to 20 feet. SL-2 had primary compression wave velocities greater than 5,000 feet/second at depths below 25 to 35 feet. SL-3 had greater than 5,000 feet/second at depths below 5 to 30 feet. SL-4 had greater than 5,000 feet/second at depths below 7 to 15 feet.

S-1 had primary compression wave velocities on the order of 5,500 feet/second at depths below 10 to 13 feet. These velocities are generally indicative of rippable to marginally rippable materials. S-2 had primary compression wave velocities on the order of 11,000 feet/second at depths below 25 feet. These velocities are generally indicative of non-rippable materials requiring blasting. S-3 had primary compression wave velocities on the order of 8,300 feet/second at depths below 6 to 8 feet. These velocities are generally indicative of non-rippable to marginally rippable materials. S-4 had primary compression wave velocities on the order of 6,600 feet/second at depths below 30 feet. These velocities are generally indicative of rippable materials. S-5 had primary compression wave velocities on the order of 3,300 feet/second at depths below 12 to 15 feet. These velocities are generally indicative of rippable materials.

As a means to further characterize the excavateability/rippability of the metamorphic bedrock of the Santiago Peak Volcanics, twelve (12) test pits (EX-1 through EX-12) were excavated with a Caterpillar 336 tracked excavator (~80,000 lbs.) and eight (8) test pits (TP-1 through TP-8) were excavated with a Caterpillar 328 tracked excavator (~88,000 lbs.). Logs of these excavations are presented in Appendix B. Based on these track-hoe test pit excavations and AGS's experience with grading of other sites in similar bedrock, the hard rock conditions encountered at the project site will likely require very heavy ripping, specialized grading techniques (Hoe-Rams, Breakers, etc.) and/or localized blasting at depths where the trackhoe encountered refusal. Typically this depth ranges from six and one-half (6.5) to nineteen (19) feet below existing ground surface in the metamorphic bedrock areas shown on Plate 1. The shallowest refusal depths encountered were in test pits TP-3 and TP-7 located in the upper/southwesterly portion of the site. These areas are characterized by two ridgelines with increased topographic relief. The more recent excavator test pits indicate generally rippable bedrock to the depth of designed cuts. However, exploration with the excavator near the southerly and southwesterly limits of the site (area of deepest design cuts) was precluded due to environmental restrictions.

Oversized materials will be generated from cuts in the bedrock and these oversized materials should be handled as discussed in Section 8.4. Recommended undercuts to remove hard rock from the near pad grade and within utility alignments are presented in Section 8.5.

Should blasting be required within 300 to 500 feet of the existing SDG&E transmission lines the owner should be aware that detailed monitoring by a licensed seismologist will be required.

### 7.2. Groundwater

Groundwater is not anticipated to impact the proposed site improvements.

### 7.3. Compressibility

Onsite the undocumented artificial fill, alluvium, colluvium, and highly weathered formational materials are considered to be moderately compressible in their present condition. These materials should be removed and recompacted within structural fill areas. The proposed compacted fill will have low compressibility.

### 7.4. Earthwork Adjustments

Table 7.4 summarizes estimated bulk/shrink factors which should be used by the design engineer for earthwork balance estimates. As is the case with every project, contingencies should be made to adjust the earthwork balance when grading is in progress and actual conditions are better defined.

**TABLE 7.4**
**Earthwork Adjustment Factors**

| Geologic Unit | Adjustment Factor |
|---|---|
| Undocumented Artificial Fill | Shrink 8%-10% |
| Alluvium/Colluvium | Shrink 9%-12% |
| Santiago Formation | Bulk 5 to 10% |
| Santiago Peak Volcanics (Rippable) | Bulk 15%-20% |
| Santiago Peak Volcanics (Blasting) | Bulk 20%-25% |

### 7.5. Collapse Potential/Hydro-Consolidation

Given the hard and dense volcanic rock below the site and the fact that the alluvium, colluvium, and undocumented artificial fill will be removed, the potential for hydro-consolidation is considered remote at the subject site.

## 7.6. Expansion Potential

As part of our investigation, representative bulk samples of near surface soils were collected and tested to evaluate their potential for expansion. Testing was performed in general accordance with ASTM D 4829. Test results by AGS indicate that the soils tested possess an expansion index (EI) within the range of 3 to 187, which corresponds to a "Very Low" to "Very High" expansion potential. Post-grading testing should be conducted to define as-graded expansive soil characteristics. The results of those tests and the final as-graded conditions will govern design of foundations and street pavement sections.

## 7.7. Shear Strength

Shear strength testing was conducted on "re-molded" samples of the onsite soils. Based upon our test results and familiarity with the onsite geologic units, AGS has summarizes the recommended shear strengths in Table 7.7 for the various geologic units and compacted fill onsite.

**TABLE 7.7**
**SHEAR STRENGTH**

| Material | Cohesion (psf) | Friction Angle (degrees) |
|---|---|---|
| Artificial Fill | 200 | 28 |
| Santiago Formation | 300 | 33 |
| Santiago Peak Volcanics | 400 | 38 |

## 7.8. Chemical/Resistivity Test Results

The onsite soils are generally classified as having a "negligible" to "high" sulfate exposure condition when classified in accordance with ACI 318-11 Table 4.2.1 (per 2013 CBC). Where clayey sedimentary units like those encountered onsite are located adjacent to the Santiago Peak Volcanics these soils tend to have higher sulfate concentrations. Accordingly, for budgetary purposes "moderate" sulfate exposure should be considered for foundation elements. The relatively low resistivity demonstrated by the site soils indicate they should be considered "moderately to severely" corrosive to metals in contact with those soils. Final determination should be based upon post-graded testing of near-surface soils. Determination as to the need and specification for protection of metal construction materials should be determined by engineers(s) specializing in corrosion analysis.

## 7.9. Bearing Capacity and Lateral Earth Pressures

Ultimate bearing capacity values were obtained using the graphs and formulas presented in *NAVFAC DM-7.1*. Allowable bearing was determined by applying a factor of safety of at least three (3) to the ultimate bearing capacity.

Static lateral earth pressures were calculated using *Rankine* methods for active and passive cases. If it is desired to use *Coulomb* forces, a separate analysis specific to the application can be conducted.

# 8.0 GRADING RECOMMENDATIONS

Based on the information presented it is the opinion of AGS that the proposed development is feasible from a geotechnical point of view. All grading shall be accomplished under the observation and testing of the project Geotechnical Consultant in accordance with the recommendations contained herein, the current codes practiced by the County of San Diego and this firm's Earthwork Specifications (Appendix E).

## 8.1. Site Preparation

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

## 8.2. Unsuitable Soils Removals

All topsoil/residual soil, alluvium, colluvium, highly weathered formational materials, and any undocumented fill will require removal in structural areas. It is anticipated that the depth of removal could range from a few feet to depths of 10 feet. Localized areas may require deeper removals. Minimally the removals should extend a lateral distance of at least 5 feet beyond the limits of settlement sensitive structures or the building pad, whichever is greater. If deeper removals are performed, the removals should extend a lateral distance equal to the depth of removal beyond the improvement limits. Removal bottoms should expose competent formational materials in a firm and unyielding condition. The resulting removal bottoms should be observed by a representative of AGS to verify that adequate removal of unsuitable materials have been conducted prior to fill placement.

In general, soils removed during remedial grading will be suitable for reuse in compacted fills, provided they are properly moisture conditioned and do not contain deleterious materials. Grading shall be accomplished under the observation and testing of the project soils engineer and engineering geologist or their authorized representative in accordance with the recommendations contained herein, the current grading ordinance of the County of San Diego.

## 8.3. Earthwork Considerations

### 8.3.1. Compaction Standards

All fills should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557. Prior to the placement of fill, the upper 6 to 8 inches should be ripped, moisture conditioned to optimum moisture or slightly above optimum, and compacted to a minimum of 90 percent of the maximum dry density (ASTM D1557). For roadways/driveway areas and other flatwork subjected to vehicular loading, a minimum compaction standard of 95 percent of the laboratory maximum dry density should be used within the upper 12 inches.

#### 8.3.2. Treatment of Removal Bottoms

At the completion of unsuitable soil removals, the exposed bottom should be scarified to a minimum depth of eight inches, moisture conditioned to above optimum moisture and compacted in-place to the standards set forth in this report.

#### 8.3.3. Fill Placement

Fill should be placed in thin lifts (eight-inch bulk), moisture conditioned to at or slightly above the optimum moisture content, uniformly mixed, and compacted by the use of both wheel rolling and kneading type (sheep's foot) compaction equipment until the designed grades are achieved.

#### 8.3.4. Benching

Where the natural slope is steeper than 5-horizontal to 1-vertical and where determined by the project Geotechnical Engineer or Engineering Geologist, compacted fill material shall be keyed and benched into competent materials.

#### 8.3.5. Mixing and Moisture Control

In order to prevent layering of different soil types and/or different moisture contents, mixing and moisture control of materials will be necessary. The preparation of the earth materials through mixing and moisture control should be accomplished prior to and as part of the compaction of each fill lift. Water trucks or other water delivery means may be necessary for moisture control. Discing may be required when either excessively dry or wet materials are encountered.

#### 8.3.6. Haul Roads

All haul roads, ramp fills, and tailing areas shall be removed prior to engineered fill placement.

#### 8.3.7. Compaction Equipment

Compaction equipment on the project shall include a combination of rubber-tired and sheepsfoot rollers to achieve proper compaction. Adequate water trucks/pulls should be available to provide sufficient moisture and dust control.

### 8.4. <u>Oversized Materials</u>

Oversized rock material [i.e., rock fragments greater than eight (8) inches] will be produced during the excavation of the design cuts and undercuts. Provided that the procedure is acceptable to the developer and governing agency, this rock may be incorporated into the compacted fill section to within three (3) feet of finish grade within residential areas and to one (1) foot below the deepest utility in street and house utility connection areas. Maximum rock size in the upper portion of the hold-down zone is restricted to eight (8) inches. Disclosure of the above rock hold-down zone should be made to prospective homebuyers explaining that excavations to accommodate swimming pools, spas, and other appurtenances will likely encounter oversize rock [i.e., rocks greater than eight (8) inches] below three (3) feet. Rock disposal details are presented on Detail 10, Appendix E. Rocks in excess of eight (8) inches in maximum dimension may be placed within the deeper fills, provided rock fills are handled in a manner described below. In order to separate oversized materials from the rock hold-down zones, the use of a rock rake may be necessary

### 8.4.1. Rock Blankets

Rock blankets consisting of a mixture of gravel, sand and rock to a maximum dimension of two (2) feet may be constructed. The rocks should be placed on prepared grade, mixed with sand and gravel, watered and worked forward with bulldozers and pneumatic compaction equipment such that the resulting fill is comprised of a mixture of the various particle sizes, contains no significant voids, and forms a dense, compact, fill matrix.

Rock blankets may be extended to the slope face provided the following additional conditions are met: 1) no rocks greater than twelve (12) inches in diameter are allowed within six (6) horizontal feet of the slope face; 2) 50 percent (by volume) of the material is three-quarter- (3/4) inch minus; and 3) backrolling of the slope face is conducted at four- (4) foot vertical intervals and satisfies project compaction specifications..

### 8.4.2. Rock Windrows

Rocks to maximum dimension of four (4) feet may be placed in windrows in deeper fill areas in accordance with Detail 10 (Appendix E). The base of the windrow should be excavated an equipment-width into the compacted fill core with rocks placed in single file within the excavation. Sands and gravels should be added and thoroughly flooded and tracked until voids are filled. Windrows should be separated horizontally by at least fifteen (15) feet of compacted fill, be staggered vertically, and separated by at least four (4) vertical feet of compacted fill. Windrows should not be placed within ten (10) feet of finish grade, within two (2) vertical feet of the lowest buried utility conduit in structural fills, or within fifteen (15) feet of the finish slope surface unless specifically approved by the developer, geotechnical consultant, and governing agency.

### 8.4.3. Individual Rock Burial

Rocks in excess of four (4) feet, but no greater than eight (8) feet may be buried in the compacted fill mass on an individual basis. Rocks of this size may be buried separately within the compacted fill by excavating a trench and covering the rock with sand/gravel, and compacting the fines surrounding the rock. Distances from slope face, utilities, and building pad areas (i.e., hold-down depth) should be the same as windrows.

### 8.4.4. Rock Disposal Logistics

The grading contractor should consider the amount of available rock disposal volume afforded by the design when excavation techniques and grading logistics are formulated. Rock disposal techniques should be discussed and approved by the geotechnical consultant and developer prior to implementation.

## 8.5. Overexcavation of Building Pads and Streets

### 8.5.1. Cut/Fill Transition Lots

Where design grades and/or remedial grading activities create a cut/fill transition, the cut and shallow fill portions of the building pad shall be overexcavated a minimum depth of three feet or 18 inches below the bottom of the proposed footings (whichever is deeper) and replaced with compacted fill. These remedial grading measures are recommended in order to minimize the potential for differential settlements between cut and fill areas. The undercut should be graded such that a gradient of at least one percent is maintained toward deeper fill areas or the front of the lot.

### 8.5.2. Cut Lots

It is recommended that for cut lots founded in hard bedrock, the bedrock should be overexcavated a minimum of 3 feet and replaced with compacted fill to facilitate utility and foundation construction. The bottom of the overexcavation should be graded such that a gradient of at least one percent is maintained toward the front of the lot.

### 8.5.3. Overexcavation of Streets/Driveways

Street/Driveway undercuts in hard rock areas should be based on depth of utilities within “right of way". The depth of undercut for streets should be at least one (1) foot below the deepest utility. In lieu of street undercutting and replacement with select soil the streets can be line shot should hard rock be encountered.

## 8.6. Slope Stability

### 8.6.1. Cut Slopes

The highest proposed cut slope is approximately 80 feet high, designed at a slope ratio of 2:1 (horizontal: vertical). Slope stability analyses cut slopes are presented in Appendix D. Surficial Stability calculations for the 2:1 cut slopes are presented in Appendix D. Based upon our analysis we anticipate that proposed cut slopes graded at slope ratios of 2:1 in Santiago Peak Volcanics will be grossly stable as designed.

Cut slopes should be observed by the Geotechnical Consultant during grading. Where cut slopes expose unfavorable geology, uncemented or poorly consolidated sandy materials, replacement of the unsuitable portions of the cut with a stabilization fill will be recommended.

### 8.6.2. Fill Slope Construction

Fill slopes on the project are designed at 2:1 ratios (horizontal to vertical). The highest anticipated fill slope is approximately 50 feet high. Slope stability analysis for the highest proposed fill slope is presented in Appendix D. Surficial Stability calculations for 2:1 fill slopes are presented in Appendix D. Fill slopes, when properly constructed with onsite materials, are expected to be grossly and surficially stable as designed.

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 finish slope surface.

8.6.2.1. *Overbuilding Fill Slopes*

Fill slopes should be overfilled to an extent determined by the contractor, but not less than 2 feet measured perpendicular to the slope face, so that when trimmed back to the compacted core, the compaction of the slope face meets the minimum project requirements for compaction.

Compaction of each lift should extend out to the temporary slope face. The sloped should be back-rolled at fill intervals not exceeding 4 feet in height unless a more extensive overfilling is undertaken.

8.6.2.2. *Compacting The Slope Face*

As an alternative to overbuilding the fill slopes, the slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Back-rolling at more frequent intervals may be required. Compaction of each fill should extend to the face of the slope. Upon completion, the slopes should be watered, shaped, and track-walked with a D-8 bulldozer or similar equipment until the compaction of the slope face meets the minimum project requirements. Multiple passes may be required.

### 8.7. Seepage

Seepage, when encountered during grading, should be evaluated by the Geotechnical Consultant. In general, seepage is not anticipated to adversely affect grading. If seepage is excessive, remedial measures such as horizontal drains or under drains may need to be installed.

## 9.0 CONCLUSIONS AND RECOMMENDATIONS

Development of the site for the proposed residential and commercial structures is considered feasible, from a geotechnical standpoint, provided that the conclusions and recommendations presented herein are incorporated into the design and construction of the project. As with all projects, changes in observed conditions may result in alternative construction techniques and/or possible delays. The contractor should be aware of these possibilities and provide contingencies in his bids to account for them.

### 9.1. Preliminary Foundation Design

The proposed structures can be supported on conventional or post-tensioned shallow foundations and slab-on-grade systems. The expansion potential of the underlying soils is classified as "Low" to “Very High”. The design of foundation systems should be based on as-graded conditions as determined after the completion of grading.

The following values may be used in the preliminary foundation design.

| | |
|---|---|
| **Allowable Bearing**: | 2000 lbs./sq.ft. (assuming a minimum embedment depth of 12 inches and a minimum width of twelve inches) |
| **Lateral Bearing**: | 250 lbs./sq.ft. at a depth of 12 inches plus 125 lbs./sq.ft. for each additional 12 inches embedment to a maximum of 2000 lbs./sq.ft. |
| **Sliding Coefficient**: | 0.35 |

The above values may be increased as allowed by Code to resist transient loads such as wind or seismic. Building Code and structural design considerations may govern. Depth and reinforcement requirements should be evaluated by the Structural Engineer.

## 9.2. Under Slab

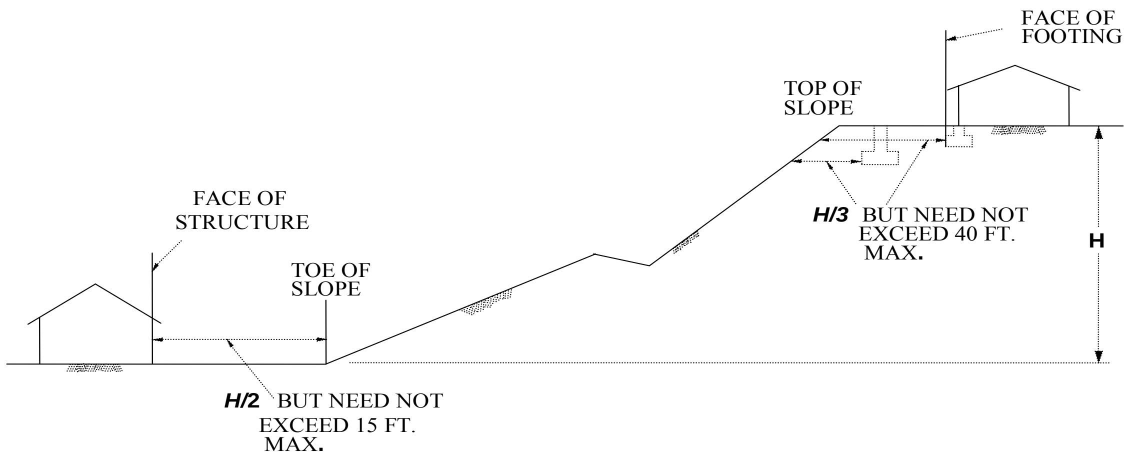
Prior to concrete placement the subgrade soils should be moisture conditioned in accordance to the expansion potential: a minimum of 110 (low expansive soils), 120 (medium expansive soils), 130 (High expansive soils) percent of optimum moisture content.

A moisture and vapor retarding system should be placed below the slabs-on-grade in portions of the structure considered to be moisture sensitive. The retarder should be of suitable composition, thickness, strength and low permeance to effectively prevent the migration of water and reduce the transmission of water vapor to acceptable levels. Historically, a 10-mil plastic membrane, such as *Visqueen*, placed between one to four inches of clean sand, has been used for this purpose. More recently Stego® Wrap or similar underlayments have been used to lower permeance to effectively prevent the migration of water and reduce the transmission of water vapor to acceptable levels. The use of this system or other systems, materials or techniques can be considered, at the discretion of the designer, provided the system reduces the vapor transmission rates to acceptable levels.

## 9.3. Deepened Footings and Structural Setbacks

It is generally recognized that improvements constructed in proximity to natural slopes or properly constructed, manufactured slopes can, 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 California Building Code (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 Figure 2.




**9.4. <u>Concrete Design</u>**

Preliminary testing indicates onsite soils exhibit a “negligible” to “high” sulfate exposure when classified in accordance with ACI 318-05 Table 4.3.1 (per 2013 CBC). However, historically where these clayey sedimentary units located adjacent to the Santiago Peak Volcanics these soils tend to have higher sulfate concentrations. Additionally, some fertilizers have been known to leach sulfates into soils otherwise containing "negligible" sulfate concentrations and increase the sulfate concentrations to potentially detrimental levels. It is incumbent upon the owner to determine whether additional protective measures are warranted to mitigate the potential for increased sulfate concentrations to onsite soils as a result of the future homeowner’s actions. Accordingly, for budgetary purposes “moderate” sulfate exposure should be considered in the design of the foundation elements.

**9.5. <u>Seismic Design Parameters</u>**

The following seismic design parameters are presented to be code compliant to the California Building Code (2013). The project site is considered to be Site Class "D" in accordance with CBC, 2013, Section 1613.3.2 and ASCE 7, Chapter 20. The site is located at Latitude 33.091˚ N and Longitude 117.208˚ W. Utilizing this information, the United States Geological Survey (USGS) web tool (http://earthquake.usgs.gov/designmaps) and ASCE 7 criterion, the mapped seismic acceleration parameters $S_S$, for 0.2 seconds and $S_1$, for 1.0 second period (CBC, 2013, 1613.3.1) for Risk-Targeted Maximum Considered Earthquake ($MCE_R$) can be determined. The mapped acceleration parameters are provided for Site Class “B”. Adjustments for other Site Classes are made, as needed, by utilizing Site Coefficients $F_a$ and $F_v$ for determination of $MCE_R$ spectral response acceleration parameters $S_{MS}$ for short periods and $S_{M1}$ for 1.0 second period (CBC, 2013 1613.3.3). Five-percent damped design spectral response acceleration parameters $S_{DS}$ for short periods and $S_{D1}$ for 1.0 second period can be determined from the equations in CBC, 2013, Section 1613.3.4.

| Seismic Design Criteria | |
|---|---|
| Mapped Spectral Acceleration (0.2 sec Period), $S_S$ | 1.002g |
| Mapped Spectral Acceleration (1.0 sec Period), $S_1$ | 0.391g |
| Site Coefficient, $F_a$ (CBC, 2013, Table 1613.3.3(1)) | 1.099 |
| Site Coefficient, $F_v$ (CBC, 2013, Table 1613.3.3(2)) | 1.618 |
| $MCE_R$ Spectral Response Acceleration (0.2 sec Period), $S_{MS}$ | 1.102g |
| $MCE_R$ Spectral Response Acceleration (1.0 sec Period), $S_{M1}$ | 0.632g |
| Design Spectral Response Acceleration (0.2 sec Period), $S_{DS}$ | 0.734g |
| Design Spectral Response Acceleration (1.0 sec Period), $S_{D1}$ | 0.426g |

Utilizing a probabilistic approach, the CBC recommends that structural design be based on the peak horizontal ground acceleration (PGA) having of 2 percent probability of exceedance in 50 years (approximate return period of 2,475 years) which is defined as the Maximum Considered Earthquake (MCE). Using the United States Geological Survey (USGS) web-based ground motion calculator, the site class modified $PGA_M$ ($F_{PGA}$*PGA) was determined to be 0.388g. This value does not include near-source factors that may be applicable to the design of structures on site.

## 9.6. Conventional Retaining Walls

The following earth pressures are recommended for the preliminary design of conventional retaining walls onsite:

**Static Case**

| Level Backfill | Rankine Coefficients | Equivalent Fluid Pressure (psf/lin.ft.) |
|---|---|---|
| Coefficient of Active Pressure: | $K_a = 0.36$ | 45 |
| Coefficient of Passive Pressure: | $K_p = 2.77$ | 346 |
| Coefficient of at Rest Pressure: | $K_o = 0.53$ | 66 |

| 2 : 1 Backfill | Rankine Coefficients | Equivalent Fluid Pressure (psf/lin.ft.) |
|---|---|---|
| Coefficient of Active Pressure: | $K_a = 0.65$ | 81 |
| Coefficient of Passive Pressure: | | |
| Descending | $K_p$ (-) = 1.23 | 154 |
| Coefficient of At Rest Pressure: | $K_o = 0.77$ | 96 |

**Seismic Case**

In addition to the above static pressures, unrestrained retaining walls should be designed to resist seismic loading. In order to be considered unrestrained, retaining walls should be allowed to rotate a minimum of roughly 0.004 times the wall height. The seismic load can be modeled as a thrust load applied at a point 0.6H above the base of the wall, where H is equal to the height of the wall. This seismic load (in pounds per lineal foot of wall) is represented by the following equation:

$$Pe = \frac{3}{8} * \gamma * H^2 * k_h$$

Where:

H = Height of the wall (feet)

$\gamma$ = soil density = 125 pounds per cubic foot (pcf)

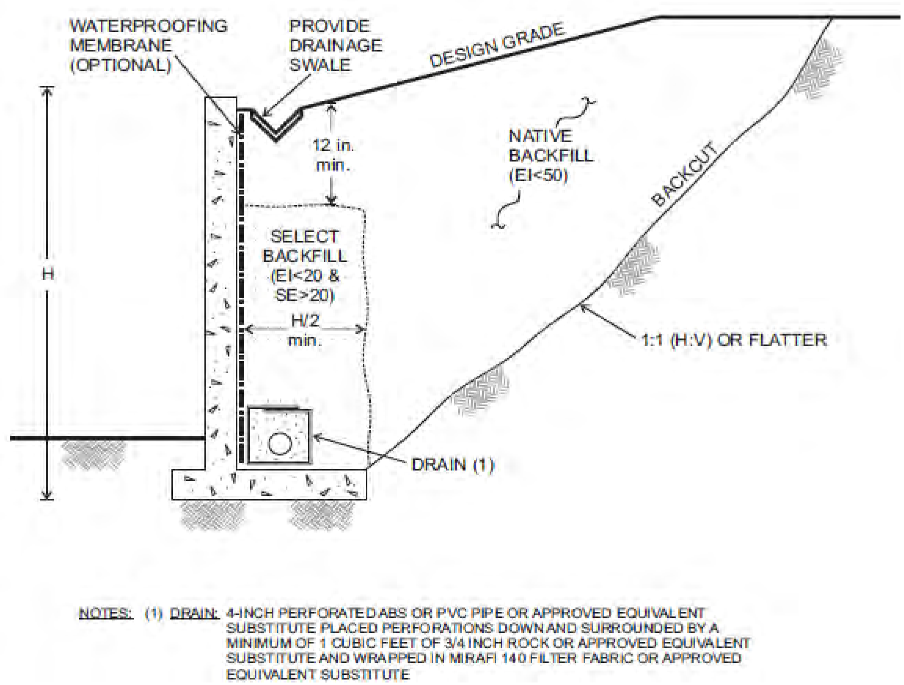
$k_h$ = seismic pseudostatic coefficient = 0.5* peak horizontal ground acceleration / g

Peak ground acceleration onsite is estimated at 0.388g.

Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

The foundations for retaining walls of appurtenant structures structurally separated from the building structures, may bear on properly compacted fill. A bearing value of 2,000 psf may be used for design of retaining walls. Retaining wall footings should be designed to resist the lateral forces by passive soil resistance and/or base friction as recommended for foundation lateral resistance. To relieve the potential for hydrostatic pressure wall backfill should consist of a free draining backfill (sand equivalent "SE" >20) and a heel drain should be constructed. The heel drain should be place at the heel of the wall and should consist of a 4-inch diameter perforated pipe (SDR35 or SCHD 40) surrounded by 4 cubic feet of crushed rock (3/4-inch) per lineal foot, wrapped in filter fabric (Mirafi® 140N or equivalent).

Proper drainage devices should be installed along the top of the wall backfill, which should be properly sloped to prevent surface water ponding adjacent to the wall. In addition to the wall drainage system, for building perimeter walls extending below the finished grade, the wall should be waterproofed and/or damp-proofed to effectively seal the wall from moisture infiltration through the wall section to the interior wall face.




The wall should be backfilled with granular soils placed in loose lifts no greater than 8-inches thick, at or near optimum moisture content, and mechanically compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. The soils engineer or his representative should observe the retaining wall footings, backdrain installation and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

## 9.7. Utility Trench Excavation

All utility trenches should be shored or laid back in accordance with applicable OSHA standards. Excavations in bedrock areas should be made in consideration of underlying geologic structure. AGS should be consulted on these issues during construction.

## 9.8. Utility Trench Excavation

Mainline and lateral utility trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557. Onsite soils will not be suitable for use as bedding material but will be suitable for use in backfill, provided oversized materials are removed. No surcharge loads should be imposed above excavations. This includes spoil piles, lumber, concrete trucks or other construction materials and equipment. Drainage above excavations should be directed away from the banks. Care should be taken to avoid saturation of the soils.

Compaction should be accomplished by mechanical means. Jetting of native soils will not be acceptable.

## 9.9. Exterior Slabs and Walkway

The subgrade below exterior slabs, sidewalks, driveways, patios, etc. should be moisture conditioned to a minimum of 110 (low expansive soils), 120 (medium expansive soils), 130 (High expansive soils) percent of optimum moisture content prior to concrete placement, dependent upon the expansion potential of the subgrade soils.

### 9.9.1. Slab Thickness

Concrete flatwork and driveways should be designed utilizing four-inch minimum thickness.

### 9.9.2. Control Joints

Weakened plane joints should be installed on walkways at intervals of approximately eight to ten feet. Exterior slabs should be designed to withstand shrinkage of the concrete.

### 9.9.3. Flatwork Reinforcement

Consideration should be given to reinforcing any exterior flatwork.

### 9.9.4. Thickened Edge

Consideration should be given to construct a thickened edge (scoop footing) at the perimeter of slabs and walkways adjacent to landscape areas to minimize moisture variation below these improvements. The thickened edge (scoop footing) should extend approximately eight inches below concrete slabs and should be a minimum of six inches wide.

## 9.10. Plan Review

Once final grading plans become available, they should be reviewed by AGS to verify that the design recommendations presented are consistent with the proposed construction.

## 9.11. Geotechnical Review

As is the case in any grading project, multiple working hypotheses are established utilizing the available data, and the most probable model is used for the analysis. Information collected during the grading and construction operations is intended to evaluate the hypotheses, and some of the assumptions summarized herein may need to be changed as more information becomes available. Some modification of the grading and construction recommendations may become necessary, should the conditions encountered in the field differ significantly than those hypothesized to exist.

# 10.0 SLOPE AND LOT MAINTENANCE

Maintenance of improvements is essential to the long-term performance of structures and slopes. Although the design and construction during mass grading is planned to create slopes that are both grossly and surficially stable, certain factors are beyond the control of the soil engineer and geologist. The homeowners must implement certain maintenance procedures.

The following recommendations should be implemented.

### 10.1. Slope Planting

Slope planting should consist of ground cover, shrubs and trees that possess deep, dense root structures and require a minimum of irrigation. The resident should be advised of their responsibility to maintain such planting.

### 10.2. Lot Drainage

Roof, pad and lot drainage should be collected and directed away from structures and slopes and toward approved disposal areas. Design fine-grade elevations should be maintained through the life of the structure or if design fine grade elevations are altered, adequate area drains should be installed in order to provide rapid discharge of water, away from structures and slopes. Residents should be made aware that they are responsible for maintenance and cleaning of all drainage terraces, down drains and other devices that have been installed to promote structure and slope stability.

### 10.3. Slope Irrigation

The resident, homeowner and Homeowner Association should be advised of their responsibility to maintain irrigation 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 run-off and ground saturation should be avoided. If automatic sprinkler systems are installed, their use must be adjusted to account for natural rainfall conditions.

### 10.4. Burrowing Animals

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

## 11.0 LIMITATIONS

This report is based on the project as described and the information obtained from the excavations at the approximate locations indicated on the Plate 1. The findings are based on the results of the field, laboratory, and office investigations combined with an interpolation and extrapolation of conditions between and beyond the excavation locations. The results reflect an interpretation of the direct evidence obtained. Services performed by AGS have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other representation, either expressed or implied, and no warranty or guarantee is included or intended.

The recommendations presented in this report are based on the assumption that an appropriate level of field review will be provided by geotechnical engineers and engineering geologists who are familiar with the design and site geologic conditions. That field review shall be sufficient to confirm that geotechnical and geologic conditions exposed during grading are consistent with the geologic representations and corresponding recommendations presented in this report. AGS should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Such changes or variations may require a re-evaluation of the recommendations contained in this report.

The data, opinions, and recommendations of this report are applicable to the specific design of this project as discussed in this report. They have no applicability to any other project or to any other location, and any and all subsequent users accept any and all liability resulting from any use or reuse of the data, opinions, and recommendations without the prior written consent of AGS.

AGS has no responsibility for construction means, methods, techniques, sequences, or procedures, or for safety precautions or programs in connection with the construction, for the acts or omissions of the CONTRACTOR, or any other person performing any of the construction, or for the failure of any of them to carry out the construction in accordance with the final design drawings and specifications.

# Appendix A

**References**

## References

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Kennedy, M.P. and Tan, S.S. (2007). Geologic Map of the Oceanside 30’ x 60’ Quadrangle, California, 1:100,000 Scale.

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# Appendix B

## Field Data

Project Questhaven
Date Excavated 2/1/2016
Logged by PWM
Equipment Cat 336 excavator w/ 2 foot bucket

# LOG OF EXCAVATOR PITS

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-1 | 0.0 – 5.5 | SM | **Alluvium (Qal):**<br>SILTY SAND with CLAY, fine to medium grained, brown, slightly moist, loose; some gravel and cobble to 4” diameter.<br>@4 ft. Medium dense. |
| | 5.5 – 9.0 | | **Santiago Formation (Tsa):**<br>CLAYEY SANDSTONE, fine to medium grained, yellowish brown to olive, soft.<br>@8 ft. moderately hard |
| | | | TD = 9 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-2 | 0.0 – 3.0 | SM | **Alluvium (Qal):**<br>SILTY SAND with CLAY, fine to medium grained, brown, moist, loose. |
| | 3.0 – 9.0 | | **Santiago Peak Volcanics (Jsp):**<br>METAVOLCANIC ROCK, yellowish to olive brown, highly weathered, soft; coming up in angular blocks to 4”.<br>@4 ft. Bluish gray with weathered surfaces, harder; coming up in angular blocks to 10”.<br>@7 ft. Moderately hard. |
| | | | TD = 9 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-3 | 0.0 – 1.0 | SM | **Topsoil:**<br>SILTY SAND with CLAY, fine to medium grained, brown, moist, loose. |
| | 1.0 – 4-0 | CH | **Colluvium (Qcol):**<br>CLAY, mottled yellow and grayish brown, moist, firm. |
| | 4.0 – 7.0 | | **Santiago Peak Volcanics (Jsp):**<br>METAVOLCANIC ROCK, mottled yellow, red, and gray, highly weathered, soft; approx. 50% cobble to 4" and 50% soil.<br>@5.5 ft. Bluish gray, reduces to angular generally 3" minus clasts.<br>@6.5 ft. Hard<br><br>TD = 7 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-4 | 0.0 – 1.5 | SM | **Topsoil:**<br>SILTY SAND with CLAY, fine to medium grained, brown, moist, loose. |
| | 1.5 – 20.5 | CH | **Santiago Formation (Tsa)**<br>Weathered to SANDY CLAY, light gray mottled yellow and orange, moist, firm.<br>@5 ft. predominantly light gray, slight yellow and orange staining.<br>@13 ft. slightly moist.<br>@15 ft. CLAYSTONE, gray mottled yellow, orange, and red, soft<br>@19 ft. CLAYSTONE, olive gray and purplish gray mottled orange, soft; some rounded cobble and boulders to 12". |
| | 20.5 – 21.5 | | **Santiago Peak Volcanics (Jsp):**<br>METAVOLCANIC ROCK, gray mottled orange, highly weathered, moderately hard; breaking into angular blocks generally 1" to 3".<br>@21 ft. Hard.<br><br>TD = 21.5 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-5 | 0.0 – 1.5 | SC | **Colluvium:** CLAYEY SAND, fine to medium grained, brown, very moist, loose. |
| | | CH | @1.5 ft. SANDY CLAY, fine to medium grained sand, reddish brown, very moist, soft to firm. |
| | 1.5 – 24.5 | | **Santiago Formation (Tsa):** SANDY CLAY, yellowish brown, moist, firm. @3.5 ft. CLAYEY SAND, fine to coarse grained, light gray mottled orange, slightly moist, medium dense. @6.5 ft. CLAYSTONE, light gray mottled yellow, soft; coming up in blockly clasts to 4”. @8 ft. CLAYEY SANDSTONE, fine to medium grained sand, light yellowish brown, moist, soft; with rounded cobble clasts up to 8” diameter. @17 ft. Moderately hard, slower digging. |
| | | | TD = 24.5 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-6 | 0.0 – 1.5 | CH | **Colluvium:** CLAY, brown, slightly moist, firm. |
| | 1.5 – 3.5 | | **Santiago Formation (Tsa):** SANDY CLAY, mottled yellow and light brown, moist, firm; weathered. @3.5’ CLAYEY SANDSTONE, fine to medium grained sand with some gravel, mottled gray and yellow, slightly moist, soft. @6’ CLAYSTONE, mottled olive gray and yellow, soft. @8 ft. SANDY CLAYSTONE, yellowish brown. |
| | 3.5 – 15.0 | | **Santiago Peak Volcanics (Jsp):** METAVOLCANIC ROCK, bluish gray mottled red and yellow, soft; highly weathered; coming up in blocks up to about 6”. @13ft. Less weathered, predominantly bluish gray, moderately hard. |
| | | | TD = 15 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-7 | 0.0 – 3.0 | SC | **Colluvium:**<br>CLAYEY SAND, fine to medium grained, brown, moist, loose. |
| | 3.0 – 11.0 | | **Santiago Formation (Tsa):**<br>CLAYEY SAND, fine to coarse grained with some gravel to 1" light yellowish brown, moist, medium dense.<br>@5 ft. CLAYSTONE with some sand and gravel, light gray, moist, soft.<br>@9 ft. Yellowish brown, moderately weathered, soft, reducing to soil with some blocky clasts up to 4". |
| | 11.0 – 15.0 | | **Santiago Peak Volcanics (Jsp):**<br>METAVOLCANIC ROCK, moderately weathered, bluish gray moderately hard; coming up in blocky clasts up to 6".<br>@14 ft. Hard digging. |
| | | | TD = 15 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-8 | 0.0 – 8.0 | SC | **Colluvium:**<br>CLAYEY SAND, fine to medium grained, brown, moist, loose. |
| | | CH | @1.5 ft. SANDY CLAY, fine to medium grained sand, yellowish brown, moist, firm; some clasts to 8" diameter. |
| | 8.0 – 14.0 | | **Santiago Formation (Tsa):**<br>CLAYSTONE, purplish gray and light greenish gray mottled orange, moderately weathered, soft.<br>@12 feet. Slightly weathered. |
| | 14.0 – 14.5 | | **Santiago Peak Volcanics (Jsp):**<br>METAVOLCANIC ROCK, bluish gray, moderately hard. |
| | | | TD = 14.5 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-9 | 0.0 – 6.0 | SC | **Colluvium:** CLAYEY SAND, fine to medium grained, brown to reddish brown, loose. @1.5 ft. CLAYEY SAND, fine to coarse grained with some gravel and cobble clasts to 10" diameter, yellowish brown, moist, medium dense. |
| | 6.0 – 9.0 | | **Santiago Formation (Tsa):** CLAYSTONE, purplish gray and light greenish gray mottled orange, moderately weathered, soft. @12 ft. Slightly weathered. |
| | 9.0 – 13.0 | | **Santiago Peak Volcanics (Jsp):** METAVOLCANIC ROCK, highly weathered, bluish gray with red and yellow on weathered surfaces, soft; breaking up to blocky clasts up to 8". @10.5 ft. Moderately hard. |
| | | | TD = 13 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-10 | 0.0 – 6.0 | SC | **Colluvium:** CLAYEY SAND, fine to medium grained, brown, moist, loose. @2 ft. CLAYEY SAND, fine to medium grained with some gravel to cobble clasts, yellowish brown mottled orange, moist, medium dense. |
| | 6.0 – 9.0 | | **Santiago Formation (Tsa):** SANDY CLAYSTONE, light gray and greenish, soft, coming up in blocky clasts to 6". @9 ft. moderately hard. |
| | 9.0 – 13.5 | | **Santiago Peak Volcanics (Jsp):** METAVOLCANIC ROCK, bluish gray, moderately hard, breaking up to 4" clasts. |
| | | | TD = 13.5 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-11 | 0.0 – 3.0 | SM | **Colluvium:** SILTY SAND, fine to medium grained, brown to reddish brown, moist, loose. |
| | | SC | @0.5 ft. CLAYEY SAND, fine to medium grained, reddish brown, moist, loose. |
| | 3.0 – 11.0 | | **Santiago Formation (Tsa):** Weathered to SANDY CLAY, light gray mottled orange, stiff. @5 ft. SANDY CLAYSTONE, light gray with less orange, soft. @7 ft. Breaking up into clasts up to 3”. @10 ft. Moderately hard. |
| | | | TD = 11 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| EX-12 | 0.0 – 3.0 | SM | **Colluvium:** SILTY SAND with CLAY, fine to medium grained, brown, moist, loose. |
| | | SC | @1 ft. CLAYEY SAND, fine to medium grained, reddish brown, moist, loose. |
| | 3.0 – 14.0 | | **Santiago Formation (Tsa):** CLAYEY SANDSTONE, gray to yellowish gray mottled orange, soft, moderately weathered. @5 ft. light gray, moderately hard. |
| | | | TD = 14 FT. NO WATER, NO CAVING |

| | |
|---|---|
| Project | Questhaven |
| Date Excavated | 11/12/2015 |
| Logged by | FE |
| Equipment | JD 460 |

## LOG OF TEST PITS

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| T-1 | 0.0 – 0.5 | SM | **Topsoil:**<br>SILTY SAND; grayish brown, dry to slightly moist, loose, fine to coarse grained. |
| | 0.5 – 9.5 | | **Santiago Formation (Tsa):**<br>Sandy Claystone; Light brown to light reddish brown, dry, highly weathered, fine grained sand, soft.<br>@ 2.0 ft. red, moist, moderately soft, slightly weathered.<br>@ 4.0 ft. some sub-angular meta-volcanic clasts to 3-inch diameter.<br>@ 4.5 ft. Clayey Sandstone; yellow, red and gray, slightly moist to moist, moderately soft, slightly weathered, some rounded volcanic clasts to 1-inch diameter.<br>@ 6.0 ft. Some rounded Rhyolite clasts to 6-inch diameter.<br>@ 6.5 ft. Sandy Claystone: light gray and yellow, moist, moderately soft, fine grained. |
| | 9.5 – 10 | | **Santiago Peak Volcanics (Jsp):**<br>Meta-Volcanic Rock; Light gray on fresh surfaces, reddish brown on weathered surfaces, dry, hard, slightly weathered.<br><br>REFUSAL @ 10 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| T-2 | 0.0 – 1.0 | CL | **Colluvium (Qcol):**<br>SANDY CLAY; brown and reddish brown, dry to slightly moist, soft, fine to coarse grained. |
| | 1.0– 12.0 | | **Santiago Formation (Tsa):**<br>Sandy Claystone; Light gray with yellowish red iron oxide, moist, soft, moderately weathered, fine grained sand.<br>@ 5.0 ft. Clayey Sandstone; light gray, moist, moderately soft, fine to coarse grained, slightly weathered.<br>@ 8.0 ft. some rounded volcanic clasts to 6-inch diameter.<br>@ 10.0 ft. Sandy Claystone; gray and reddish brown, moist, moderately soft, fine grained sand, slightly weathered. |
| | 12.0 | | **Santiago Peak Volcanics (Jsp):**<br>Meta-Volcanic Rock; gray, dry, very hard.<br><br>REFUSAL @ 12 FT.<br>NO WATER, NO CAVING |
| T-3 | 0.0 – 9.0 | GP | **Alluvium (Qal):**<br>SILTY GRAVEL; Sub-angular gravel to 2-inch diameter in a silty matrix, brown, slightly moist, loose, some clay, trace of fine grained sand, some rootlets. |
| | 9.0– 12.5 | | **Santiago Formation (Tsa):**<br>Sandy Claystone; Light olive to light gray, moist, soft, slightly weathered, fine grained sand.<br>@ 12.0 ft. some purple hues along shrink/swell fractures.<br><br>TOTAL DEPTH @ 12.5 FT.<br>NO WATER, NO CAVING |
| T-4 | 0.0 – 11.0 | | **Santiago Formation (Tsa):**<br>Sandy Claystone; Red to yellowish red, dry, soft, blocky, highly weathered, fine grained sand.<br>@ 1.0 ft. Slightly moist, moderately soft, slightly weathered.<br>@ 3.0 ft. light gray and yellow, thinly bedded.<br>@ 9.0 ft. some rounded volcanic clasts to 3-inch diameter. |
| | 11.0 | | **Santiago Peak Volcanics (Jsp):**<br>Dacite; Light gray to white, dry, hard to very hard.<br><br>REFUSAL @ 11 FT.<br>NO WATER, NO CAVING |

Project- Questhaven
Date Excavated - 7/10/2014
Logged by - PWM
Equipment -Cat 328D excavator w/ 2 foot bucket

## LOG OF TEST PITS

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-1 | 0.0 – 2.0 | CL | **COLLUVIUM (Qcol):** SILTY CLAY with SAND, brown to reddish brown, dry, loose. |
| | 2.0 – 3.0 | | Becomes moist, stiff. |
| | 3.0 – 14.0 | | **SANTIAGO PEAK VOLCANICS (Jsp):** (RESIDUAL SOIL) SANDY CLAY, completely weathered, soft, mottled orange brown to red brown.<br>@ 5 ft. METAMORPHIC BEDROCK, moderately weathered, hard; tight to blocky fracture spacing, some clay development on fracture surfaces, generally breaks into <8” rocks with occasional boulders up to 28”.<br>@ 11.5 ft. becomes slightly weathered, very hard.<br>REFUSAL AT 14.0 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-2 | 0.0 – 4.0 | CL | **COLLUVIUM (Qcol):** SILTY to SANDY CLAY, brown to reddish brown, dry, soft. |
| | 4.0 – 5.0 | | Becomes slightly moist, very stiff. |
| | 5.0 – 17.5 | | **SANTIAGO PEAK VOLCANICS (Jsp):** (RESIDUAL SOIL) SANDY CLAY, completely weathered, soft, red brown to dusky yellow brown. |
| | | | @ 7.0 ft. METAMORPHIC BEDROCK, highly to moderately weathered, hard, generally breaks into <6” rocks. @ 10.5 ft. breaks into rock mostly <8” |
| | | | @ 14.5 ft. becomes slightly weathered, hard to very hard @ 15.5 ft. becomes very hard. |
| | | | REFUSAL AT 17.5 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-3 | 0.0 – 3.5 | CL | **COLLUVIUM (Qcol):** SILTY CLAY with SAND, brown to reddish brown, dry, soft; occasional subrounded to subangular gravel, cobbles, and boulders up to 30”. |
| | 3.5 – 9.0 | | **SANTIAGO PEAK VOLCANICS (Jsp):** METAMORPHIC BEDROCK, slightly to moderately weathered, hard, generally breaks into <8” rocks with some >12”. @ 4.5 ft. becomes slightly weathered, hard to very hard; 18-24+” joint spacing |
| | | | REFUSAL AT 9.0 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-4 | 0.0 – 2.0 | CL | **COLLUVIUM (Qcol):** SILTY CLAY with SAND, brown, dry,soft. |
| | 2.0 – 4.0 | | Becomes reddish brown, slightly moist, stiff. |
| | 4.0 – 16.0 | | **SANTIAGO FORMATION (Tsa):** SILTY CLAYSTONE with SAND AND GRAVEL, completely weathered, soft, slightly moist; @ 6.5 ft. horizon of orange iron oxide development. |
| | | | @ 7.5 ft. becomes moderately weathered, moderately hard, mottled orange/pale brown/ gray @ 12.0 ft. becomes light gray, hard. @ 14.0 ft. becomes mottled greenish gray/red/orange. |
| | | | PRACTICAL REFUSAL AT 16.0 FT. (likely at Jsp contact) NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-5 | 0.0 – 1.0 | SM | **TOPSOIL:**<br>SILTY SAND, fine grained, dark brown, dry, loose. |
| | | | **COLLUVIUM (Qcol):** |
| | 1.0 – 3.5 | CL | CLAY, mottled dark red/gray, slightly moist to moist, firm to stiff. |
| | 3.5 – 12.5 | | **SANTIAGO FORMATION (Tsa):**<br>CLAYEY SANDSTONE, greenish gray to gray, slightly moist, soft; highly weathered.<br>@ 4.5 ft. becomes SANDY CLAYSTONE, slightly moist, soft; highly weathered.<br>@ 7.0 ft. becomes moderately hard to hard; occasional subrounded gravel and small cobble. |
| | | | **SANTIAGO PEAK VOLCANICS (Jsp):** |
| | 12.5 – 16.5 | | METAMORPHIC BEDROCK, moderately weathered, hard: tight to blocky fracture spacing.<br>@ 14.0 ft. becomes slightly weathered, very hard, breaking into rock mostly <8”. |
| | | | REFUSAL AT 16.5 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-6 | 0.0 – 8.0 | CL | **ARTIFICIAL FILL (afu):**<br>SILTY CLAY with SAND, very dark brown, dry to slightly moist, soft.<br>@ 3 ft. becomes SILTY CLAY, slightly moist to moist, soft, light orange brown to light gray. |
| | 8.0 – 19.0 | | **SANTIAGO FORMATION (Tsa):**<br>SILTY to SANDY CLAYSTONE, highly weathered, soft to moderately hard, light gray.<br>@ 10.0 ft. becomes slightly to moderately weathered, moderately hard to hard<br>@ 17.0 ft. becomes CLAYEY SANDSTONE, hard, pale yellowish brown.<br>TOTAL DEPTH 19.0 FT.<br>NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-7 | 0.0 – 3.0 | SM | **COLLUVIUM (Qcol):** SILTY SAND, fine grained, reddish brown, dry, loose; some gravel and trace roots. |
| | | SC | @ 1.0 ft. becomes CLAYEY SAND with gravel |
| | 3.0 – 6.5 | | **SANTIAGO PEAK VOLCANICS (Jsp):** METAMORPHIC BEDROCK, highly to moderately weathered, moderately hard, generally breaks into gravel-size rock fragments with occasional cobble size fragments. @ 4.5 ft. slightly weathered, hard to very hard, generally breaks into <8” rock fragments. @ 5.0 ft. becomes very hard, very slightly weathered to fresh; very difficult excavation. |
| | | | REFUSAL AT 6.5 FT. NO WATER, NO CAVING |

| Test Pit No. | Depth (ft.) | USCS | Description |
|---|---|---|---|
| TP-8 | 0.0 – 1.5 | SM | **COLLUVIUM (Qcol):** SILTY SAND, fine to medium grained, reddish brown, dry, loose; some gravel size rock fragments. |
| | 1.5 – 3.0 | SC | CLAYEY SAND, reddish brown, dry to slightly moist, loose; some cobble size rock fragments. |
| | 3.0 – 14.0 | | **SANTIAGO FORMATION (Tsa):** SILTY to SANDY CLAYSTONE, highly weathered, soft, slightly moist, light gray. @ 7.0 ft. becomes moderately weathered, soft to moderately hard. @ 12.0 ft. slightly weathered, moderately hard. |
| | | | PRACTICAL REFUSAL AT 14.0 FT. (likely at Jsp contact) NO WATER, NO CAVING |

**SEISMIC REFRACTION SURVEY**
**SAN ELIJO HILLS**
**SAN MARCOS, CALIFORNIA**

**PREPARED FOR:**
Advanced Geotechnical Solutions, Inc.
9707 Waples Street, Suite 150
San Diego, CA 92121

**PREPARED BY:**
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

February 10, 2016
Project No. 116005

SOUTHWEST GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

February 10, 2016
Project No. 116005

Mr. P.J. DeRisi
Advanced Geotechnical Solutions, Inc.
9707 Waples Street, Suite 150
San Diego, CA 92121

Subject: Seismic Refraction Survey
San Elijo Hills
San Marcos, California

Dear Mr. DeRisi:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the proposed San Elijo Hills retail development project located along San Elijo Road in San Marcos, California. Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions please contact the undersigned at your convenience.

Sincerely,
**SOUTHWEST GEOPHYSICS, INC.**

Patrick Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/18
STATE OF CALIFORNIA

PFL/HV/hv

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**TABLE OF CONTENTS**

## 1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the proposed San Elijo Hills retail development project located along San Elijo Road in San Marcos, California (Figure 1). Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

## 2. SCOPE OF SERVICES

Our scope of services included:

- Performance of four seismic P-wave refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results, conclusions and recommendations.

## 3. SITE DESCRIPTION

The project site is generally located along the south side of San Elijo Road just east of Rancho Santa Fe Road in San Marcos, California (Figure 1). The site slopes up from San Elijo Road toward the south before sloping down into a valley and then sloping up again toward the south. Several dirt roads and trails cross portions of the site. Vegetation in the area consists of annual grass, scattered brush, and small trees. Figures 2, 3a, and 3b depict the site conditions in the area of the seismic traverses.

Based on our discussions with you it is our understanding that the project involves the construction of a retail development and associated infrastructure. Cuts up to 25 feet deep along the north end of the property and up to 50 feet deep along the south end of the property may be performed.

## 4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the areas surveyed. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of sur-surface vertical component geophones and recorded with a 24-channel Geometrics StrataView seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Four seismic lines (SL-1 through SL-4) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint for a total of seven shot points.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth. For trenching operations**,** the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

| **Table 1 – Rippability Classification** | |
|---|---|
| **Seismic P-wave Velocity** | **Rippability** |
| 0 to 2,000 feet/second | Easy |
| 2,000 to 4,000 feet/second | Moderate |
| 4,000 to 5,500 feet/second | Difficult, Possible Blasting |
| 5,500 to 7,000 feet/second | Very Difficult, Probable Blasting |
| Greater than 7,000 feet/second | Blasting Generally Required |

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2011). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

## 5. ANALYSIS AND RESULTS

As previously indicated, four seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Figures 4a through 4d present the velocity models generated from our study. The approximate locations of the seismic refraction traverses are shown on the Seismic Line Location Map (Figure 2). In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

## 6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil overlying crystalline bedrock with varying degrees of weathering. Distinct vertical and lateral velocity variations are evident in the models. These inhomogeneities are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. It is also evident in the tomography models that the depth to bedrock is highly variable across the site.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be required depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

## 7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

## 8. SELECTED REFERENCES

Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

Optim, Inc., 2008, SeisOpt Pro, V-5.0.

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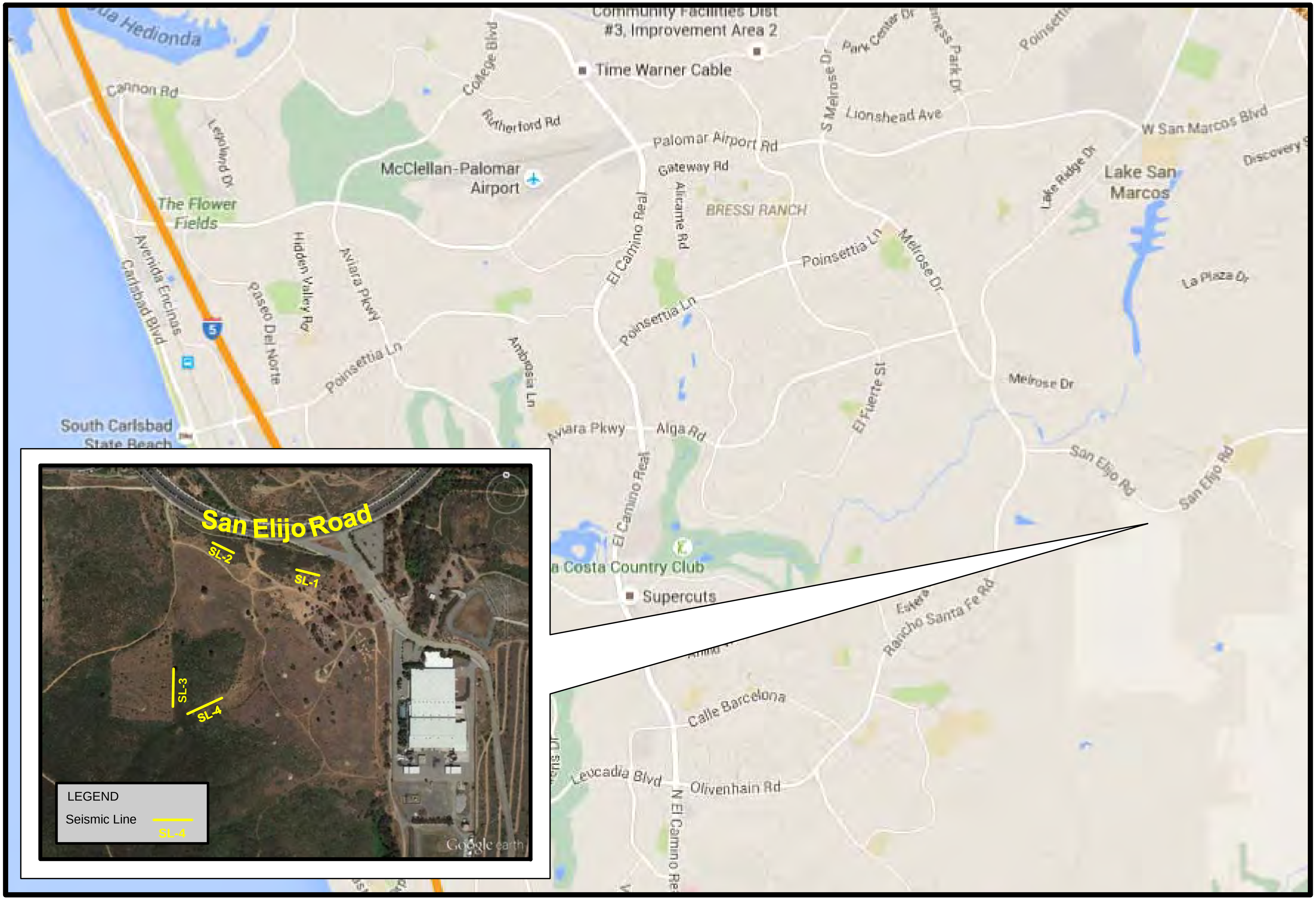
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.




**SITE LOCATION MAP**

N

San Elijo Hills
San Marcos, California

Project No.: 116005 | Date: 02/15

SOUTHWEST GEOPHYSICS INC.

Figure 1


0 200 400 600

approximate scale in feet

| SEISMIC LINE LOCATION MAP | N | San Elijo Hills<br>San Marcos, California | | SOUTHWEST GEOPHYSICS INC.<br>Figure 2 |
|---|---|---|---|---|
| | | Project No.: 116005 | Date: 02/15 | |



| | | |
|---|---|---|
| **SITE PHOTOGRAPHS** | San Elijo Hills<br>San Marcos, California | SOUTHWEST GEOPHYSICS INC. |
| | Project No.: 116005 / Date: 02/15 | Figure 3a |



**SITE PHOTOGRAPHS**

San Elijo Hills
San Marcos, California

Project No.: 116005

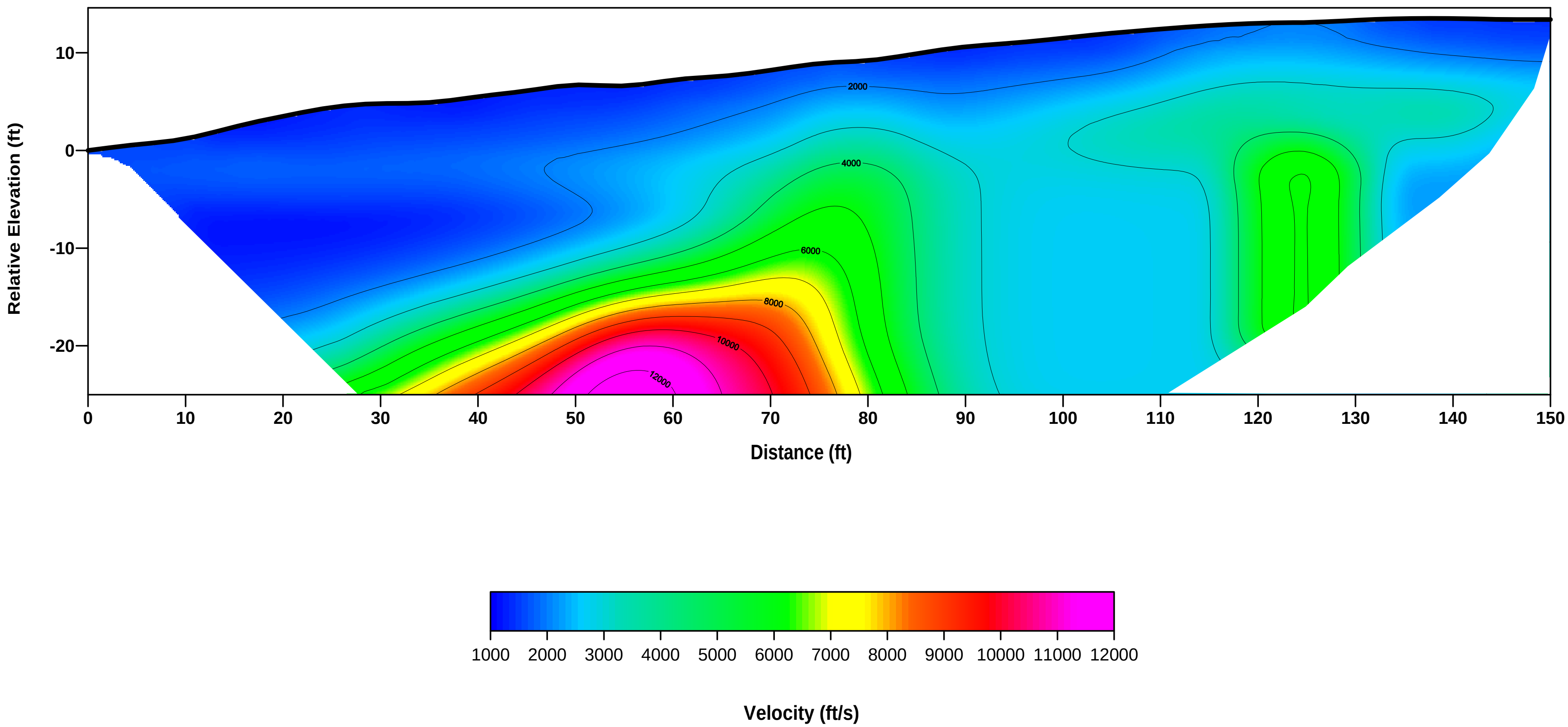
Date: 02/15

SOUTHWEST GEOPHYSICS INC.

Figure 3b

# TOMOGRAPHY MODEL

## SL-1




**SEISMIC PROFILE**

San Elijo Hills
San Marcos, California

Project No.: 116005 | Date: 02/15

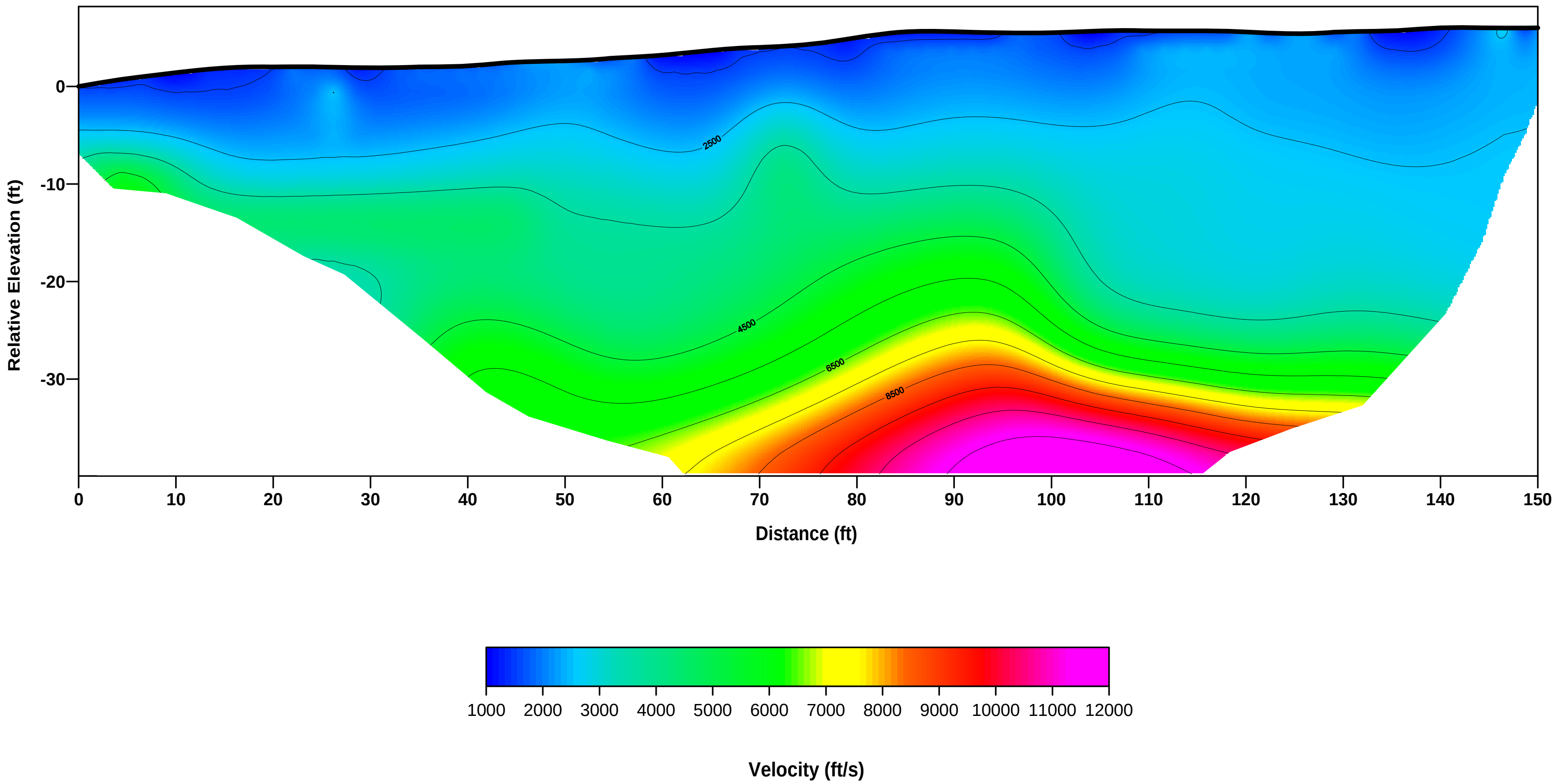
SOUTHWEST GEOPHYSICS INC.

Figure 4a

**Note: Contour Interval = 1,000 feet per second**

# TOMOGRAPHY MODEL

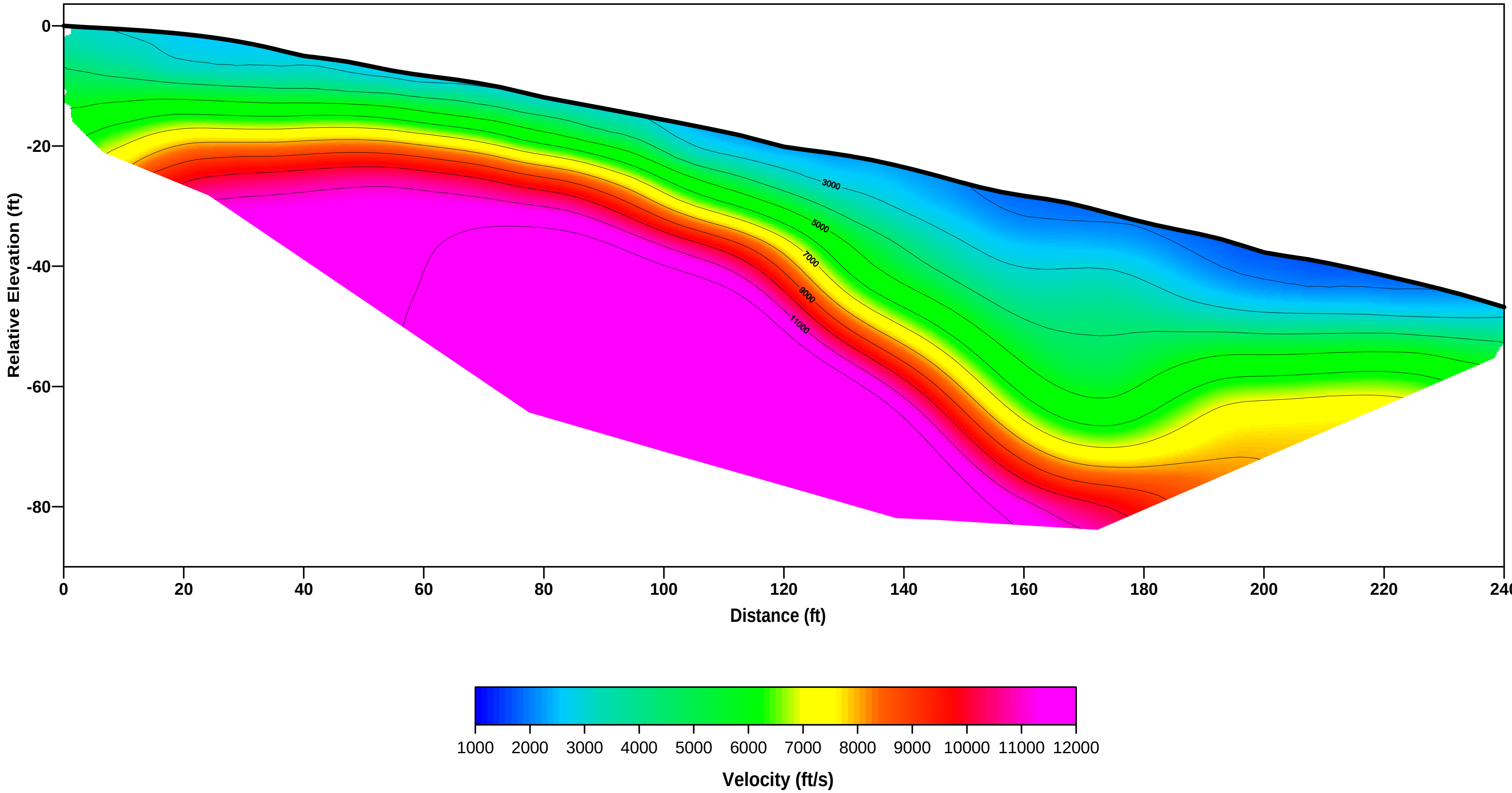
## SL-2




| | | | |
|---|---|---|---|
| **SEISMIC PROFILE** | San Elijo Hills<br>San Marcos, California | SOUTHWEST GEOPHYSICS INC.<br>Figure 4b | **Note: Contour Interval = 1,000 feet per second** |
| | Project No.: 116005 / Date: 02/15 | | |

# TOMOGRAPHY MODEL

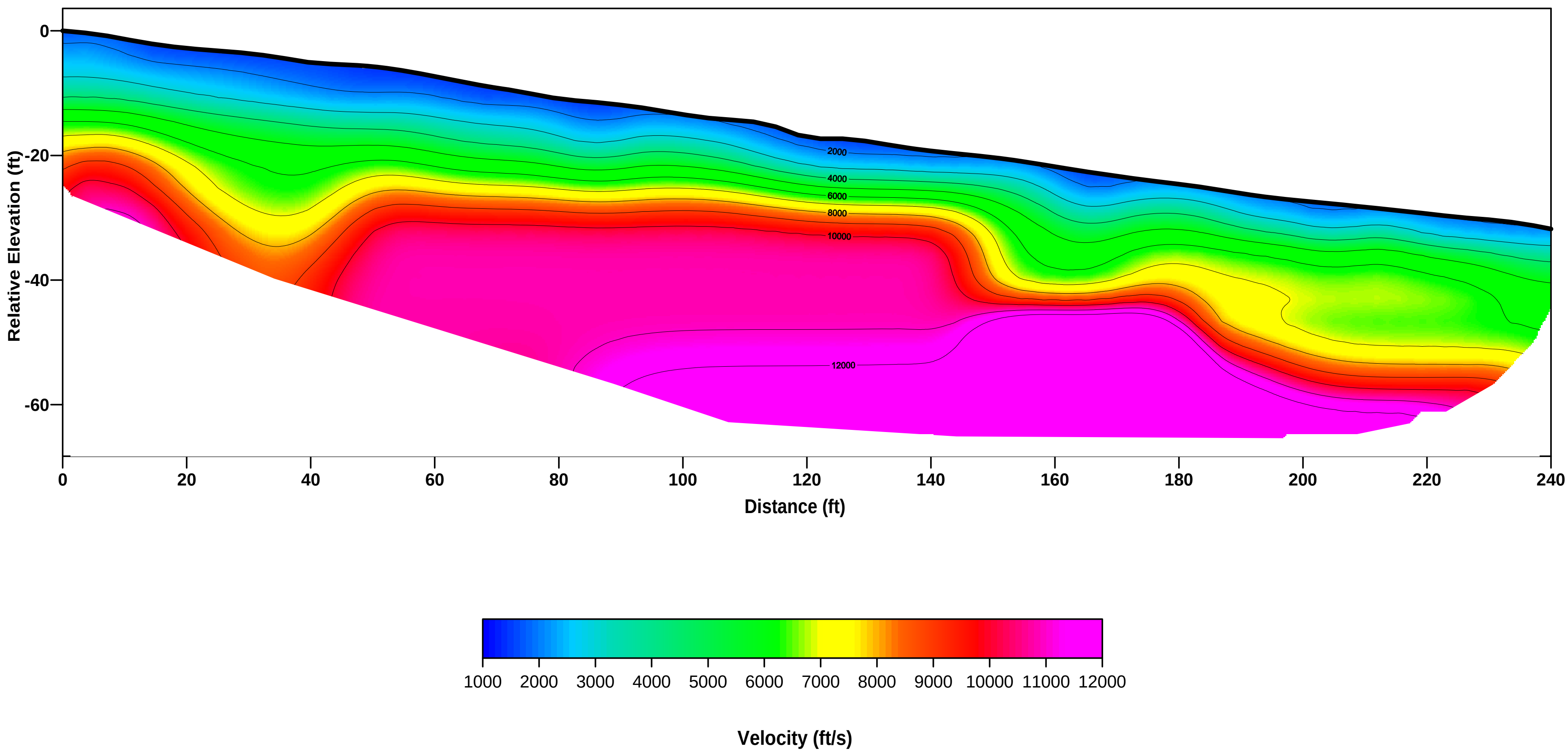
## SL-3



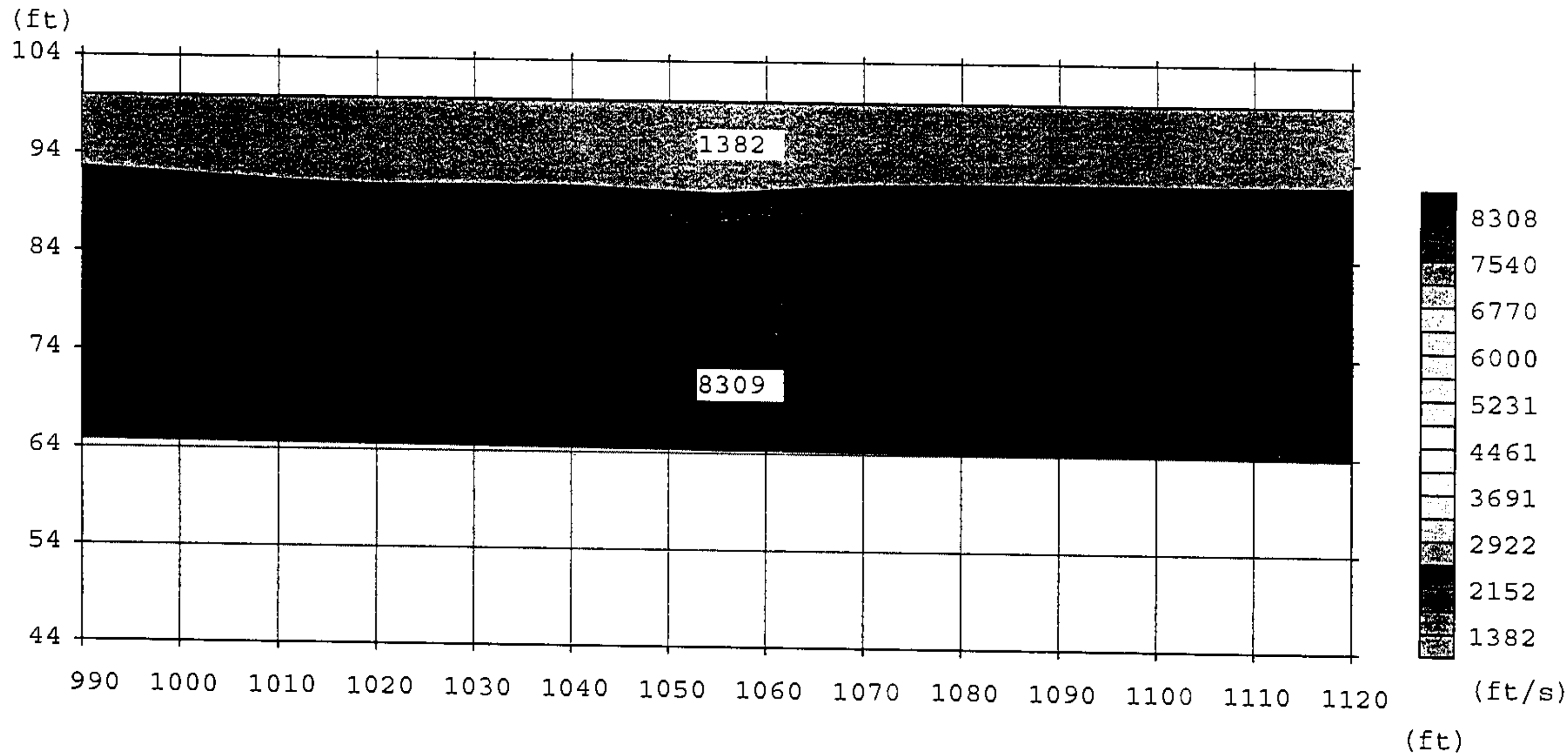

| | | | |
|---|---|---|---|
| **SEISMIC PROFILE** | San Elijo Hills<br>San Marcos, California | SOUTHWEST GEOPHYSICS INC.<br>Figure 4c | **Note: Contour Interval = 1,000 feet per second** |
| | Project No.: 116005 / Date: 02/15 | | |

# TOMOGRAPHY MODEL

## SL-4




| | | | | |
|---|---|---|---|---|
| **SEISMIC PROFILE** | San Elijo Hills<br>San Marcos, California | | SOUTHWEST GEOPHYSICS INC.<br>Figure 4d | **Note: Contour Interval = 1,000 feet per second** |
| | Project No.: 116005 | Date: 02/15 | | |




Geotechnics Incorporated

**SEISMIC LINE NUMBER 3**

Project No. 0280-005-00
Document No. 03-1116

**FIGURE C-3**

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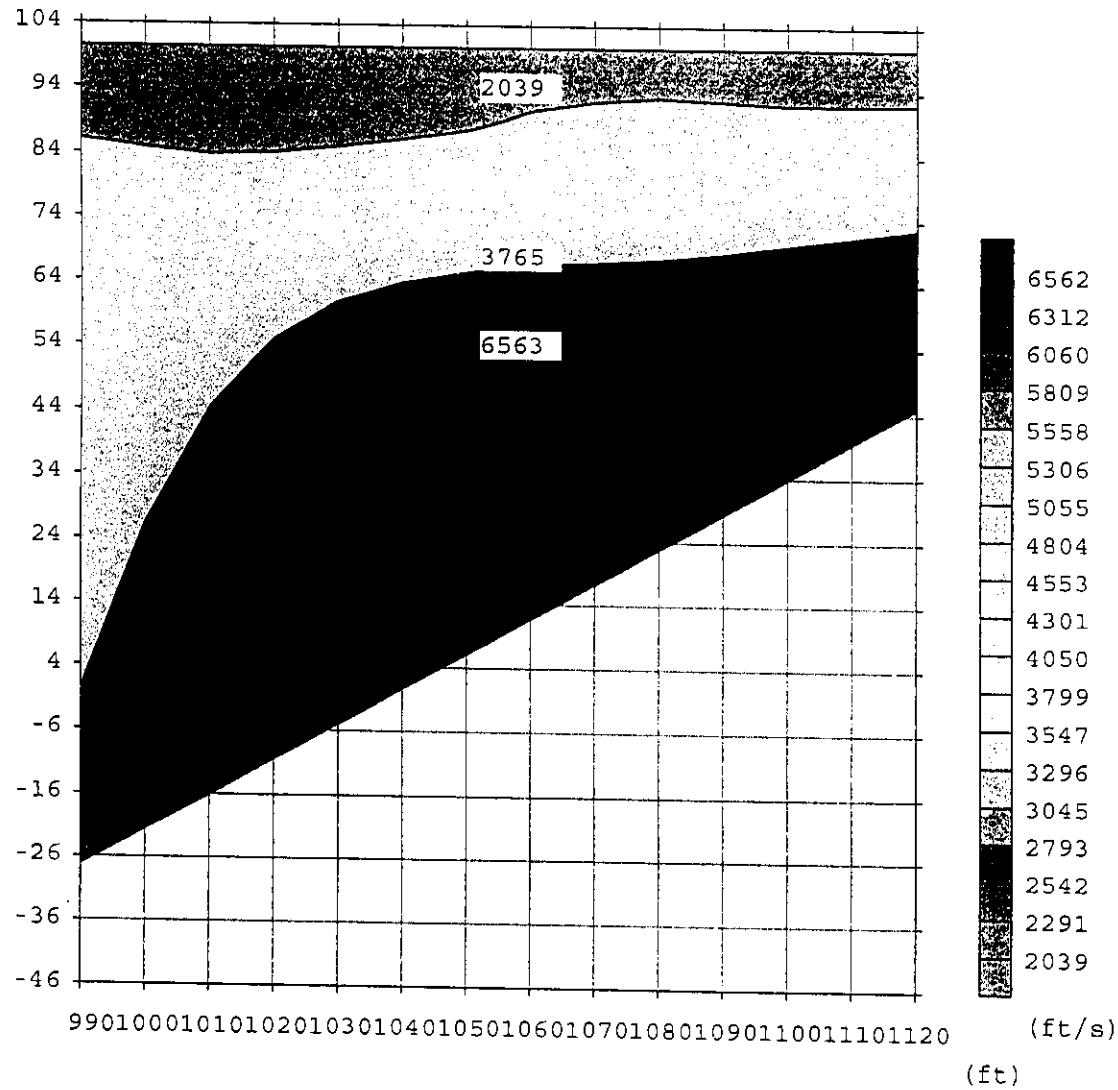


Geotechnics Incorporated

**SEISMIC LINE NUMBER 4**

Project No. 0280-005-00
Document No. 03-1116

**FIGURE C-4**

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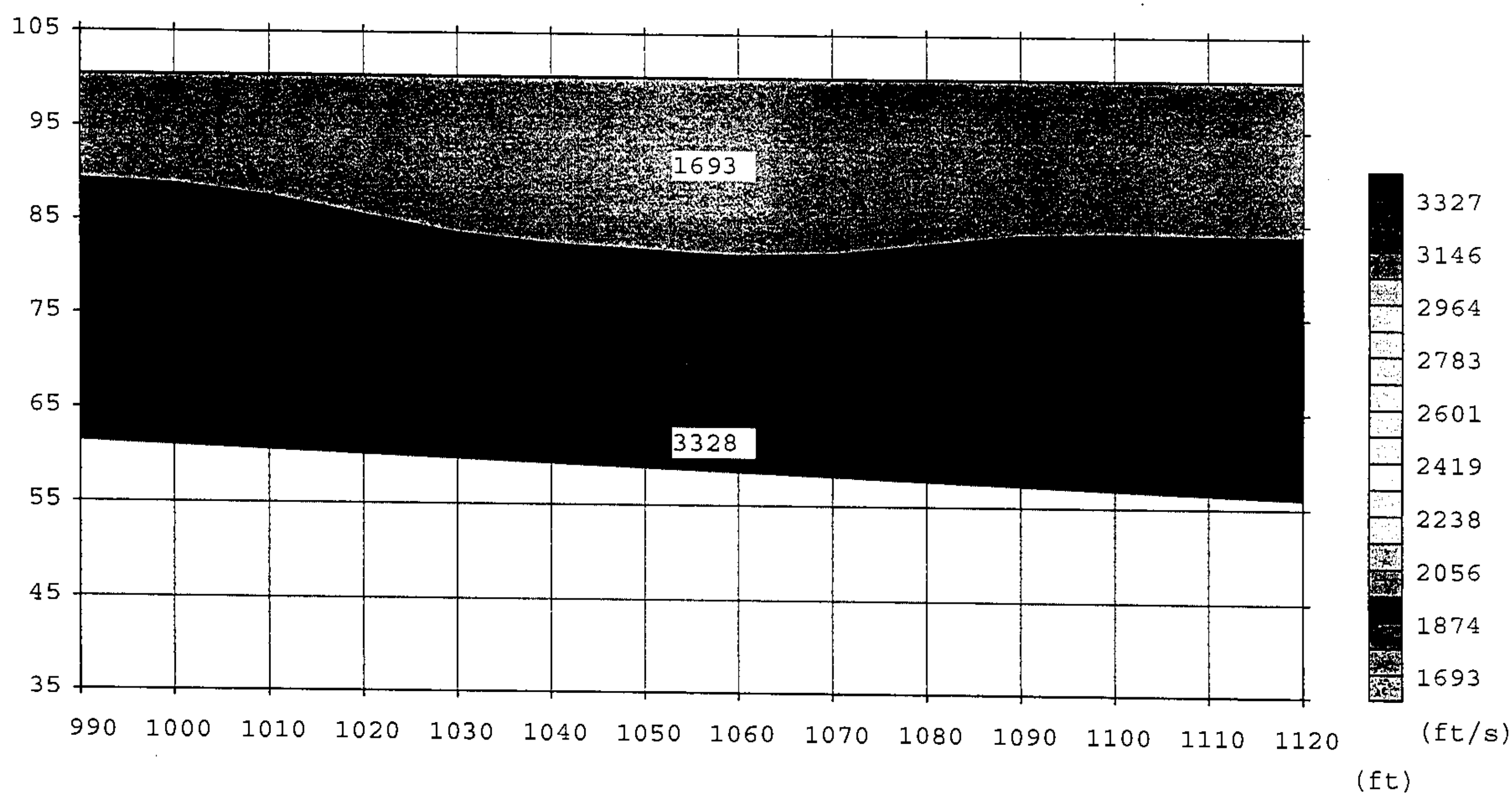


Geotechnics Incorporated

**SEISMIC LINE NUMBER 5**

Project No. 0280-005-00
Document No. 03-1116

**FIGURE C-5**

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# Appendix C

## Laboratory Data

**ADVANCED GEOTECHNICAL SOLUTIONS, INC.**

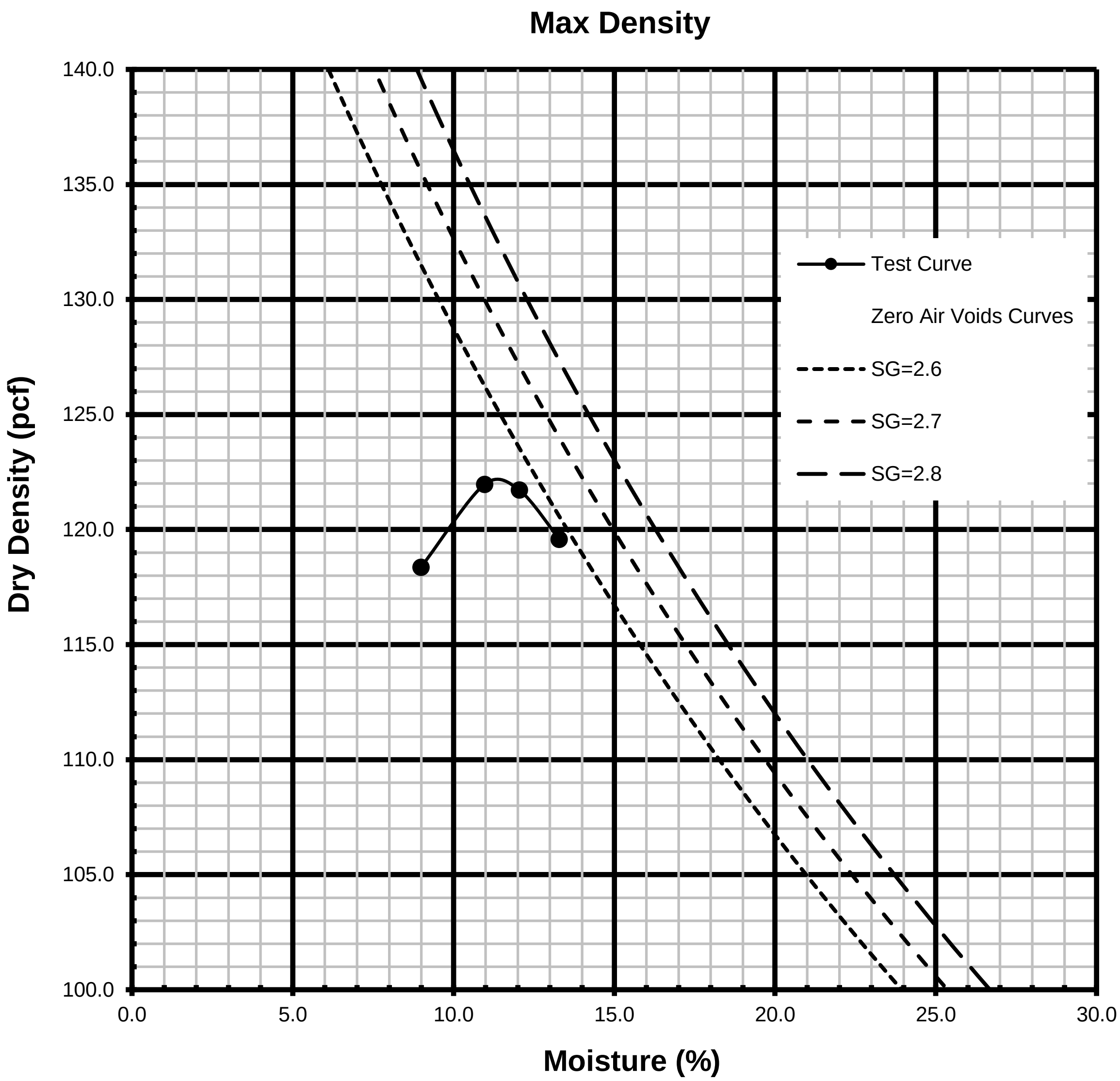
**MAXIMUM DENSITY - ASTM D1557**

Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 2/26/2016

Excavation: EX-2
Depth: 0-2'
Description: Dark brown Silty Sand with Gravel
By: H-M

Method A

| | | | | |
|---|---|---|---|---|
| Test Number | 1 | 2 | 3 | 4 |
| Dry Density (pcf) | 118.4 | 122.0 | 121.7 | 119.6 |
| Moisture Content (%) | 9.0 | 11.0 | 12.0 | 13.3 |




Maximum Density **122.0** pcf

Optimum Moisture **11.5** %

## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## MAXIMUM DENSITY - ASTM D1557

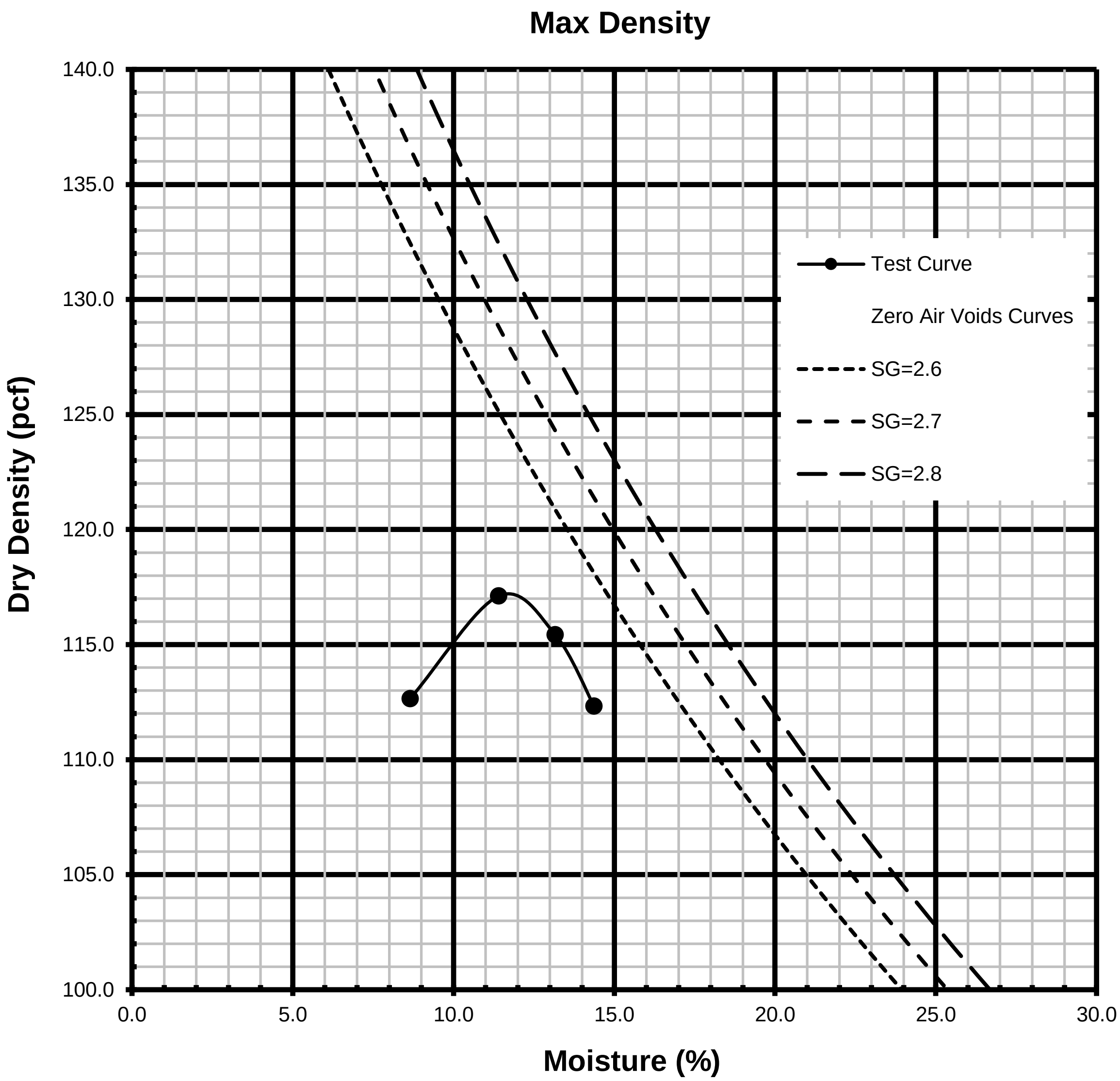
Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 2/26/2016

Excavation: EX-10
Depth: 0-1'
Description: Dark Brown Clayey Sand

By: H-M

Method A

| Test Number | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Dry Density (pcf) | 112.7 | 117.1 | 115.4 | 112.3 |
| Moisture Content (%) | 8.7 | 11.4 | 13.2 | 14.4 |




Maximum Density **117.0** pcf

Optimum Moisture **12.0** %

## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## MAXIMUM DENSITY - ASTM D1557

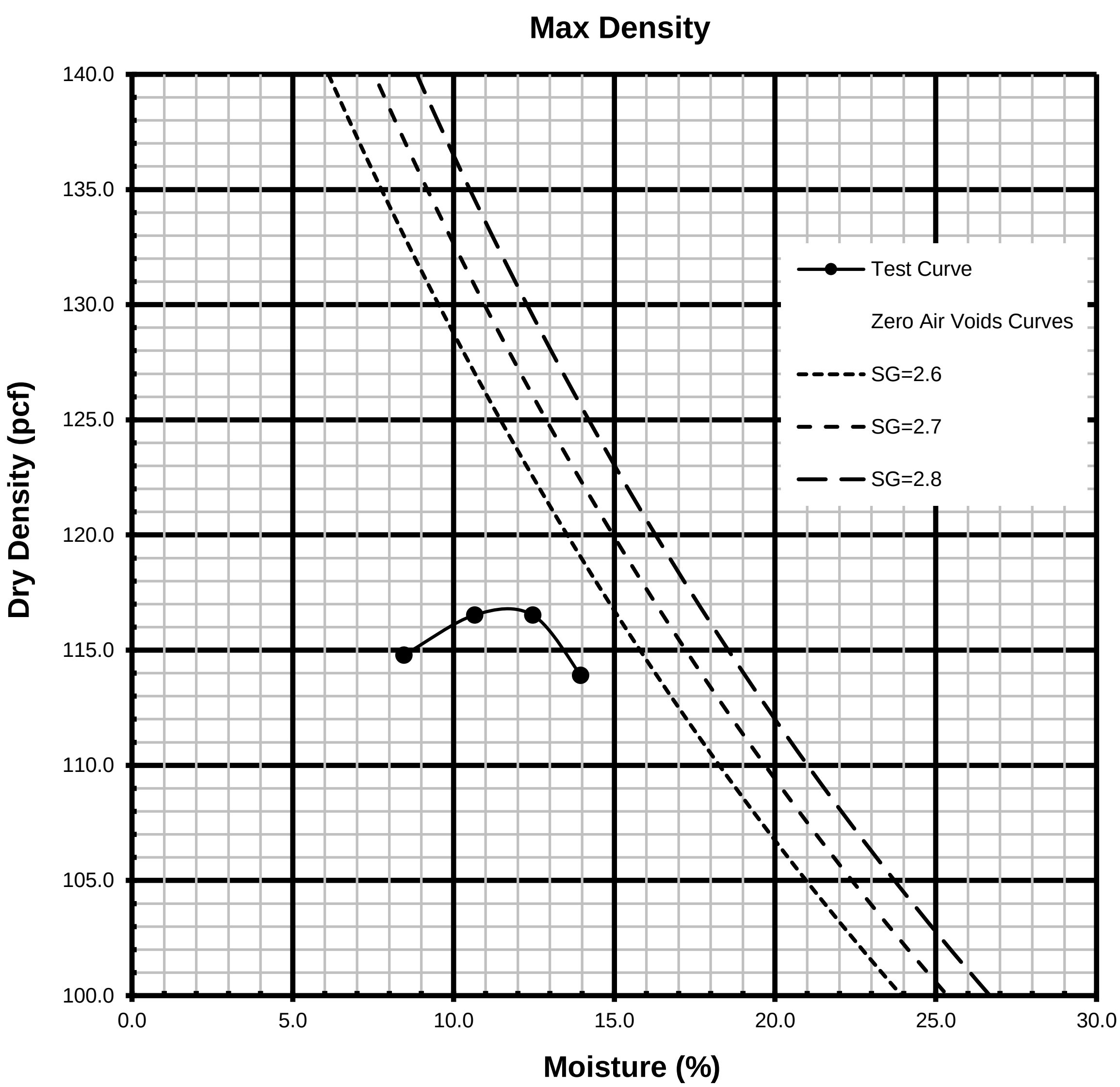
Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 2/26/2016

Excavation: EX-12
Depth: 3-6'
Description: Light brown Clayey Sand

By: H-M

Method A

| Test Number | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Dry Density (pcf) | 114.8 | 116.5 | 116.5 | 113.9 |
| Moisture Content (%) | 8.5 | 10.7 | 12.5 | 14.0 |




Maximum Density **117.0** pcf    Optimum Moisture **12.0** %

## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## DIRECT SHEAR - ASTM D3080

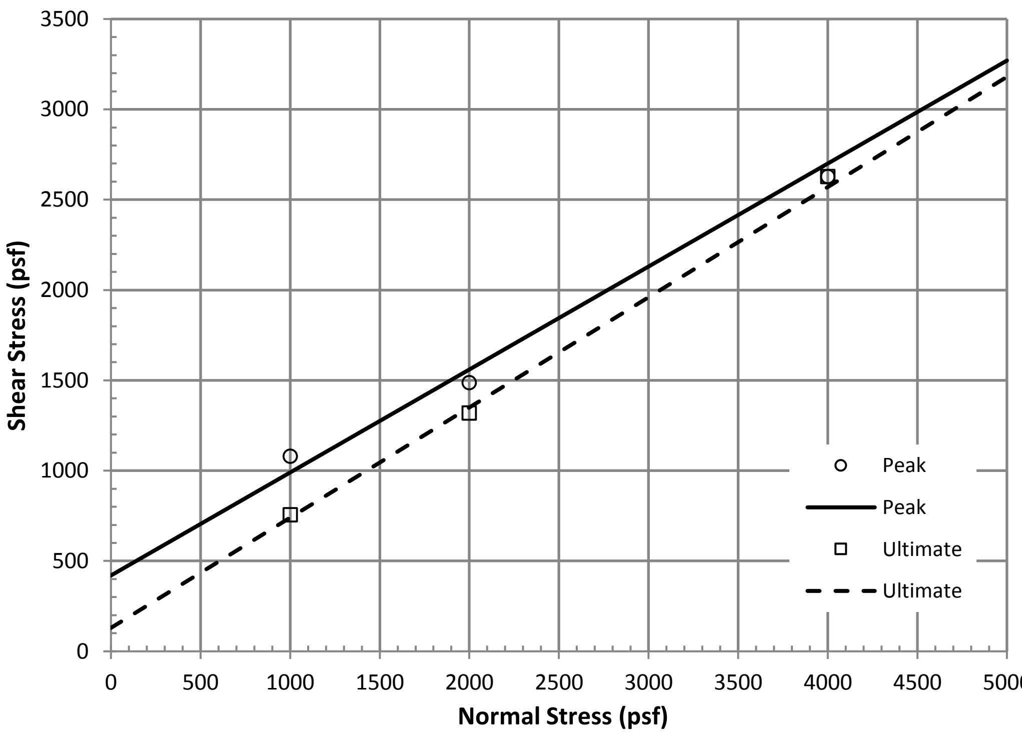
Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 2/28/16

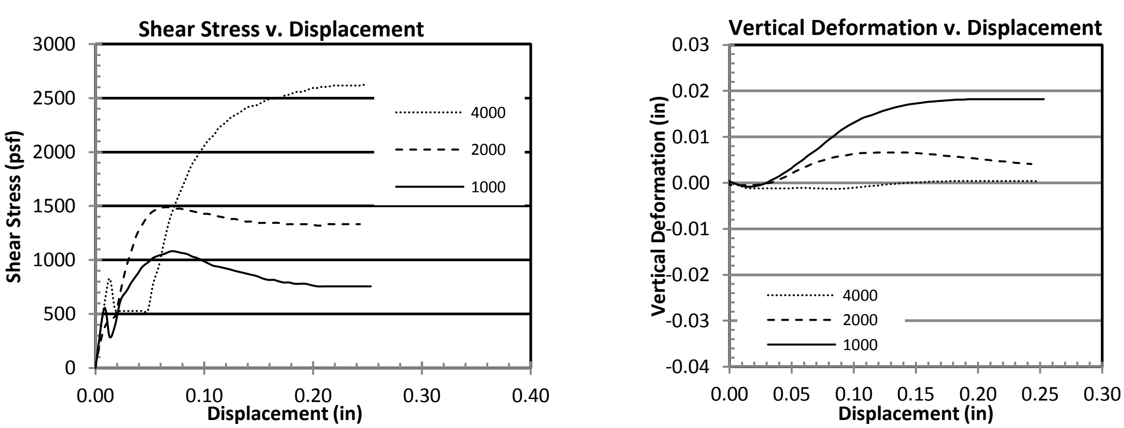
Excavation: EX-2
Depth: 0-2'
Sample Type: Remolded to 90%
By: HM

| Samples Tested | 1 | 2 | 3 |
|---|---|---|---|
| Normal Stress (psf) | 1000 | 2000 | 4000 |
| Maximum Shear Stress (psf) | 1080 | 1488 | 2628 |
| Ultimate Shear Stress (psf) | 756 | 1320 | 2628 |
| Initial Moisture Content (%) | 11.5 | 11.5 | 11.5 |
| Initial Dry Density (pcf) | 109.8 | 109.8 | 109.8 |

Method: Drained
Consolidation: Yes
Saturation: Yes
Shearing Rate (in/min): 0.04

| | Peak | Ultimate |
|---|---|---|
| Friction Angle, phi (deg) | 30 | 31 |
| Cohesion (psf) | 420 | 130 |



## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

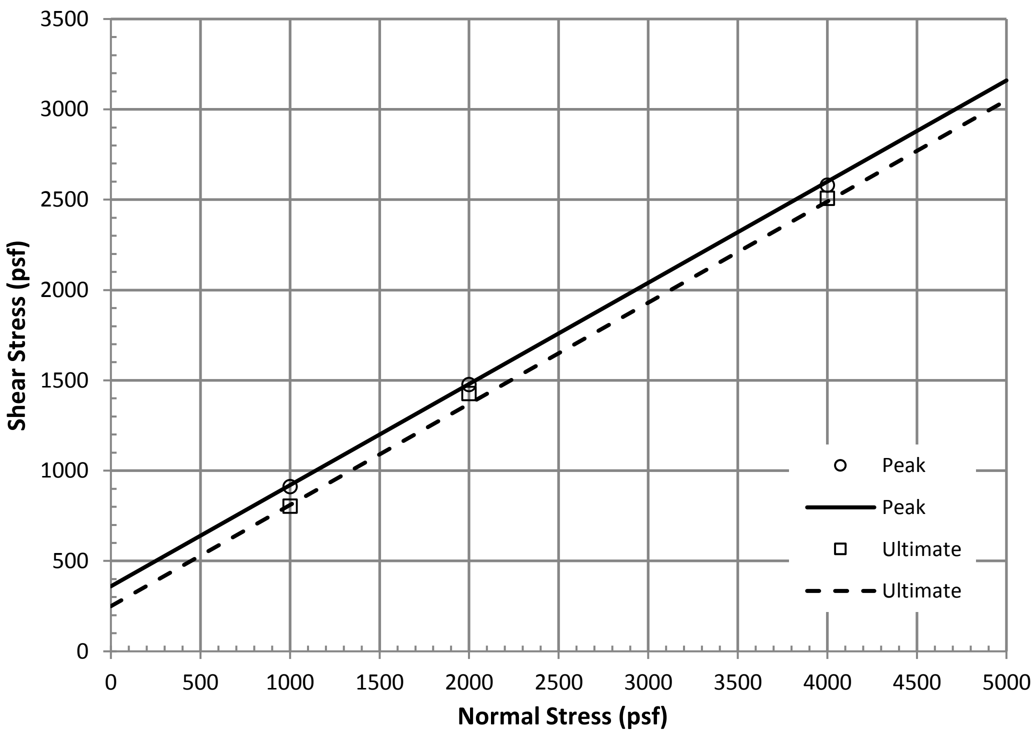
## DIRECT SHEAR - ASTM D3080

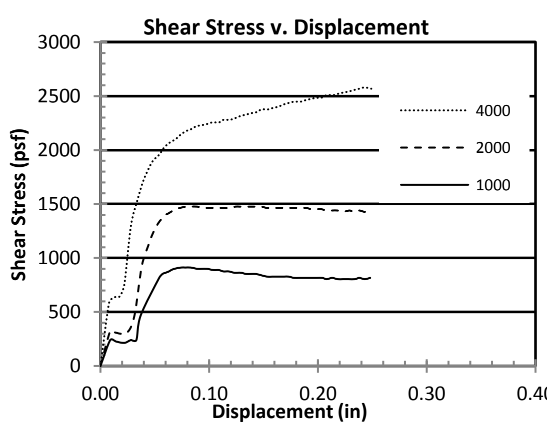
| | | | |
|---|---|---|---|
| Project Name: | Questhaven | Excavation: | EX-10 |
| Location: | San Diego County | Depth: | 0-1 |
| Project No.: | 1406-06 | Sample Type: | Remolded to 90% |
| Date: | 2/29/16 | By: | HM |

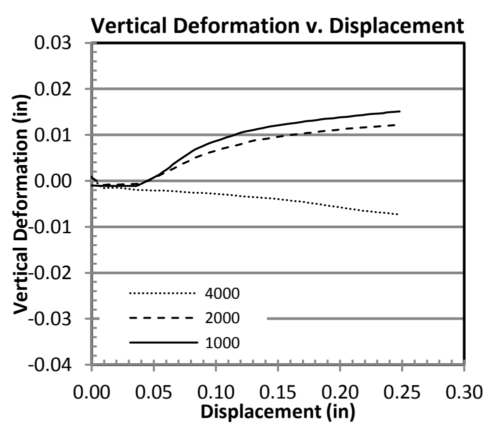
| Samples Tested | 1 | 2 | 3 |
|---|---|---|---|
| Normal Stress (psf) | 1000 | 2000 | 4000 |
| Maximum Shear Stress (psf) | 912 | 1476 | 2580 |
| Ultimate Shear Stress (psf) | 804 | 1428 | 2508 |
| Initial Moisture Content (%) | 12.0 | 12.0 | 12.0 |
| Initial Dry Density (pcf) | 105.3 | 105.3 | 105.3 |

| | |
|---|---|
| Method: | Drained |
| Consolidation: | Yes |
| Saturation: | Yes |
| Shearing Rate (in/min): | 0.04 |

| | Peak | Ultimate |
|---|---|---|
| Friction Angle, phi (deg) | 29 | 29 |
| Cohesion (psf) | 360 | 250 |



## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## DIRECT SHEAR - ASTM D3080

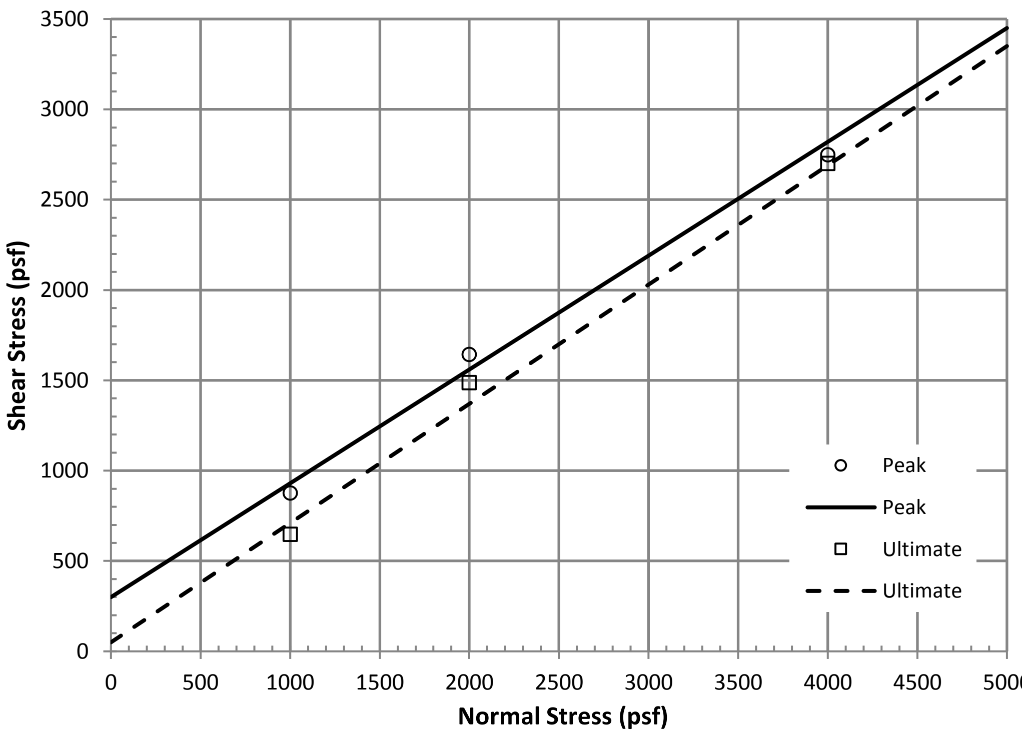
Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 3/2/16

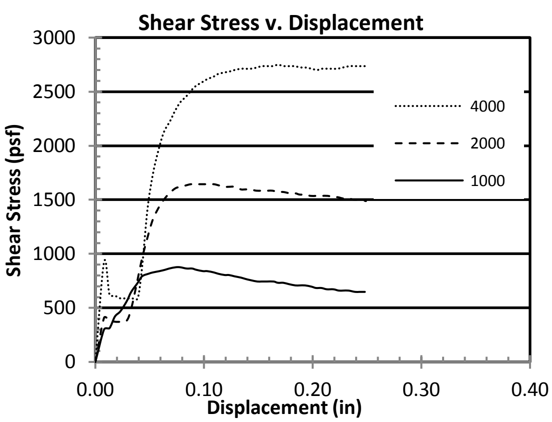
Excavation: EX-12
Depth: 3-6'
Sample Type: Remolded to 90%
By: HM

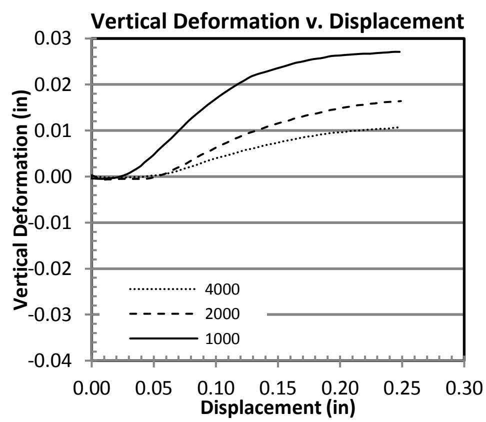
| Samples Tested | 1 | 2 | 3 |
|---|---|---|---|
| Normal Stress (psf) | 1000 | 2000 | 4000 |
| Maximum Shear Stress (psf) | 876 | 1644 | 2748 |
| Ultimate Shear Stress (psf) | 648 | 1488 | 2700 |
| Initial Moisture Content (%) | 12.0 | 12.0 | 12.0 |
| Initial Dry Density (pcf) | 105.3 | 105.3 | 105.3 |

Method: Drained
Consolidation: Yes
Saturation: Yes
Shearing Rate (in/min): 0.04

| | Peak | Ultimate |
|---|---|---|
| Friction Angle, phi (deg) | 32 | 33 |
| Cohesion (psf) | 300 | 50 |



## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## EXPANSION INDEX - ASTM D4829

| | | | |
|---|---|---|---|
| Project Name: | Questhaven | Excavation: | EX-4 |
| Location: | San Diego County | Depth: | 2-4 ' |
| File No: | 1406-06 | Description: | Light Olive Sandy Clay |
| Date: | 2/29/16 | By: | H-M |

| Expansion Index - ASTM D4829 | |
|---|---|
| Initial Dry Density (pcf): | 89.3 |
| Initial Moisture Content (%): | 16.5 |
| Initial Saturation (%): | 50.3 |
| Final Dry Density (pcf): | 97.8 |
| Final Moisture Content (%): | 31.0 |
| Final Saturation (%): | 94.4 |
| Expansion Index: | 187 |
| Potential Expansion: | Very High |

| ASTM D4829 - Table 5.3 | |
|---|---|
| Expansion Index | Potential Expansion |
| 0 - 20 | Very Low |
| 21 - 50 | Low |
| 51 - 90 | Medium |
| 91 - 130 | High |
| >130 | Very High |

## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## EXPANSION INDEX - ASTM D4829

| | | | |
|---|---|---|---|
| Project Name: | Questhaven | Excavation: | EX-10 |
| Location: | San Diego County | Depth: | 0-1 ' |
| File No: | 1406-06 | Description: | Dark brown clayey sand |
| Date: | 2/27/16 | By: | H-M |

| Expansion Index - ASTM D4829 | |
|---|---|
| Initial Dry Density (pcf): | 105.1 |
| Initial Moisture Content (%): | 11.0 |
| Initial Saturation (%): | 49.2 |
| Final Dry Density (pcf): | 96.0 |
| Final Moisture Content (%): | 21.9 |
| Final Saturation (%): | 98.2 |
| Expansion Index: | 3 |
| Potential Expansion: | Very Low |

| ASTM D4829 - Table 5.3 | |
|---|---|
| Expansion Index | Potential Expansion |
| 0 - 20 | Very Low |
| 21 - 50 | Low |
| 51 - 90 | Medium |
| 91 - 130 | High |
| >130 | Very High |

## ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## EXPANSION INDEX - ASTM D4829

| | | | |
|---|---|---|---|
| Project Name: | Questhaven | Excavation: | EX-12 |
| Location: | San Diego County | Depth: | 3-6' |
| File No: | 1406-06 | Description: | Light brown Clayey Sand |
| Date: | 2/27/16 | By: | H-M |

| Expansion Index - ASTM D4829 | |
|---|---|
| Initial Dry Density (pcf): | 104.1 |
| Initial Moisture Content (%): | 11.3 |
| Initial Saturation (%): | 49.3 |
| Final Dry Density (pcf): | 95.9 |
| Final Moisture Content (%): | 22.4 |
| Final Saturation (%): | 98.0 |
| Expansion Index: | 13 |
| Potential Expansion: | Very Low |

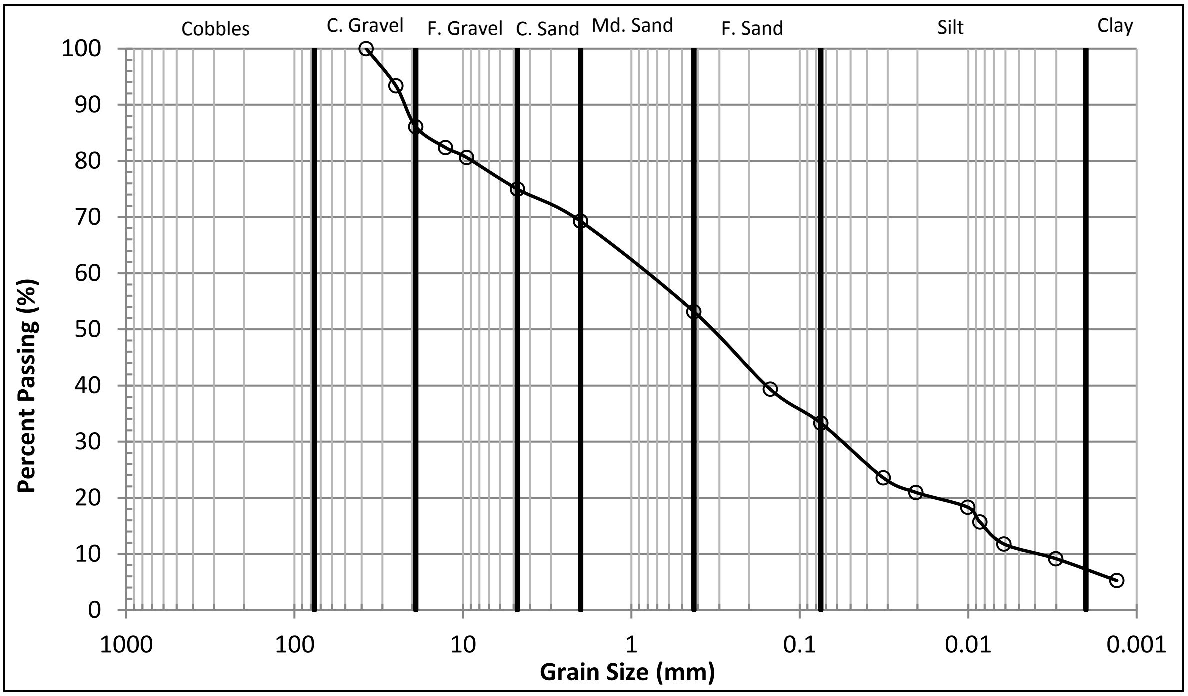
| ASTM D4829 - Table 5.3 | |
|---|---|
| Expansion Index | Potential Expansion |
| 0 - 20 | Very Low |
| 21 - 50 | Low |
| 51 - 90 | Medium |
| 91 - 130 | High |
| >130 | Very High |

# ADVANCED GEOTECHNICAL SOLUTIONS, INC.

## PARTICLE SIZE ANALYSIS - ASTM D422

Project Name: Questhaven
Location: San Diego County
Project No.: 1406-06
Date: 2/28/2016

Excavation: EX-2
Depth: 0-2 '
By: H M




| Grain Size (in/#) | Grain Size (mm) | Amount Passing (%) |
|---|---|---|
| 3 " | 75.00 | |
| 2 1/2 " | 63.00 | |
| 2 " | 50.00 | |
| 1 1/2 " | 37.50 | 100.00 |
| 1 " | 25.00 | 93.39 |
| 3/4 " | 19.05 | 86.10 |
| 1/2 " | 12.70 | 82.38 |
| 3/8 " | 9.53 | 80.63 |
| # 4 | 4.75 | 74.99 |
| # 10 | 2.00 | 69.25 |
| # 20 | 0.85 | #N/A |
| # 30 | 0.60 | #N/A |
| # 40 | 0.425 | 53.15 |
| # 50 | 0.30 | #N/A |
| # 60 | 0.212 | #N/A |
| # 100 | 0.15 | 39.35 |
| # 200 | 0.075 | 33.31 |
| Hydro | 0.0319 | 23.56 |
| Hydro | 0.0204 | 20.95 |
| Hydro | 0.0101 | 18.33 |
| Hydro | 0.0085 | 15.71 |
| Hydro | 0.0061 | 11.78 |
| Hydro | 0.0030 | 9.16 |
| Hydro | 0.0013 | 5.24 |

| Summary | |
|---|---|
| % Gravel = | 25.0 |
| % Sand = | 41.7 |
| % Silt = | 26.1 |
| % Clay = | 7.2 |
| Sum = | 100.0 |

# ANAHEIM TEST LAB, INC

3008 ORANGE AVENUE
SANTA ANA, CALIFORNIA 92707
PHONE (714) 549-7267


DATE: 03/07/16

Advanced Geotechnical Solutions, Inc
2842 Walnut Avenue, Suite C-1
Tustin, CA 92780

P.O. NO.: Verbal

LAB NO.: B-9161

SPECIFICATION: CA-417/422/643

Attn: Sean Donovan

MATERIAL: Soil

---

J.N.: 1406-06
Project: Questhaven

## ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

| | PH | SOLUBLE SULFATES per CA. 417 ppm | SOLUBLE CHLORIDES per CA. 422 ppm | MIN. RESISTIVITY per CA. 643 ohm-cm |
|---|---|---|---|---|
| EX-2 @ 0-2’ | 6.5 | 99 | 47 | 4,400 |
| EX-12 @ 3-6’ | 4.3 | 247 | 918 | 800 |

RESPECTFULLY SUBMITTED

WES BRIDGER CHEMIST

# Expansion Index
## (ASTM D4829)

| | | | |
|---|---|---|---|
| G Force Lab No. | 10065 | Sample No: | TP-1 |
| Date Sampled: | 07/10/14 | By: | PWM |
| Date Submitted: | 07/25/14 | By: | PJD |
| Sample Location: | On Site | | |
| Sample Depth: | 2-3' | | |
| Sample Description: | Pale Yellowish Tan Clayey Silt (ML) | | |

| | |
|---|---|
| Initial Water Content, % | 14.4% |
| Dry Density, pcf | 95.8 |
| Saturation, % | 51.4% |
| Initial Dial Reading, in. | 0.0000 |
| Final Dial Reading, in. | 0.1822 |
| Final Water Content, % | 33.0% |
| **Expansion Index** | **184** |
| **Potential Expansion** | **Very High** |

Reviewed by: Joe Bouknight
Joseph Bouknight, P.E., C81517

# Expansion Index
## (ASTM D4829)

| | | | |
|---|---|---|---|
| G Force Lab No. | 10067 | Sample No: | TP- 5 |
| Date Sampled: | 07/10/14 | By: | PWM |
| Date Submitted: | 07/25/14 | By: | PJD |
| Sample Location: | On Site | | |
| Sample Depth: | 3.5-4.5' | | |
| Sample Description: | Gray Silty Clay (CL/MH) | | |

| | |
|---|---|
| Initial Water Content, % | 13.4% |
| Dry Density, pcf | 99.3 |
| Saturation, % | 51.9% |
| Initial Dial Reading, in. | 0.0000 |
| Final Dial Reading, in. | 0.1391 |
| Final Water Content, % | 32.4% |
| **Expansion Index** | **141** |
| **Potential Expansion** | **Very High** |

Reviewed by: Joe Bouknight
Joseph Bouknight, P.E., C81517

**AGS Inc.** **July 29, 2014**
**Questhaven** **GF13798**

**AGS Inc. Project No:1406-06**

# LABORATORY COMPACTION CURVE

| | | | |
|---|---|---|---|
| G Force Lab No.: | **10066** | Depth, ft.: | **2-4'** |
| Sample Location: | **TP-4** | Sampled By: | **PWM** |
| Soil Description: | **Red Brown Clay (CL)** | Date Sampled: | **7/10/2014** |
| Source of Soil: | **On Site** | | |

Test Designation: **ASTM_D1557** Method **A**

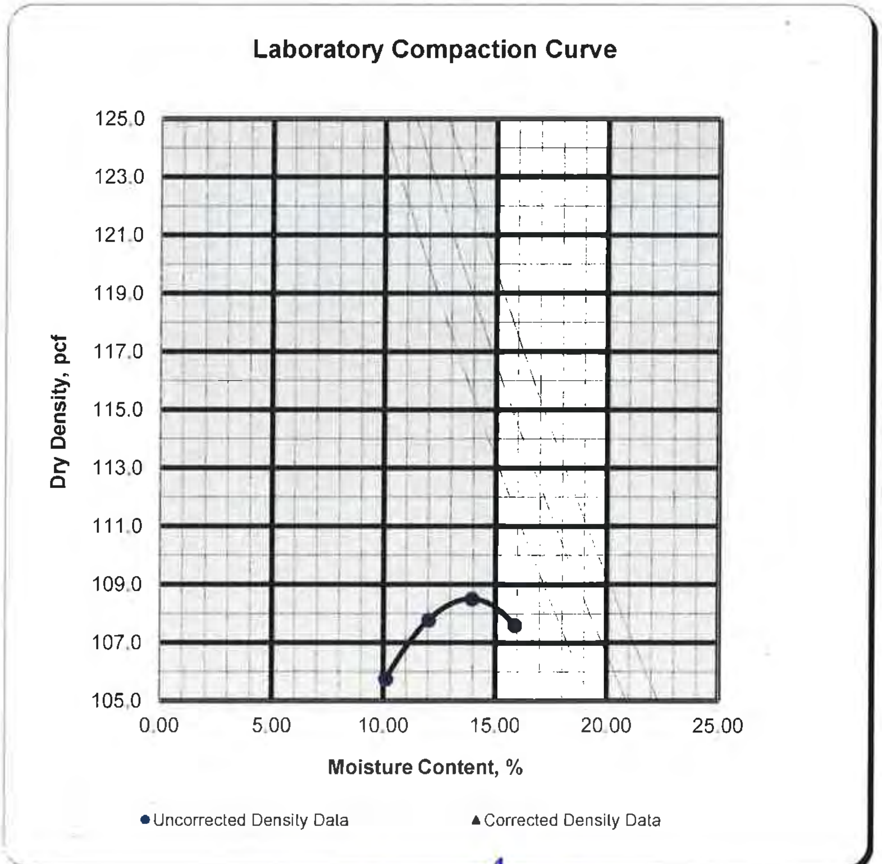
% +3/4" **0.0** % +3/8" **0.0** % +#4 **0.0**

Oversize Correction Applied? **No**

Method of Sample Preparation: **Dry**

Type of Hammer Used: **Manual**

**M/D Curve No. TP-4**




### Test Results

| | |
|---|---|
| Maximum Density, pcf | **108.5** |
| Optimum Moisture, % | **13.8** |

### Oversize Corrected Results

| | |
|---|---|
| Maximum Density, pcf | **N/A** |
| Optimum Moisture, % | **N/A** |

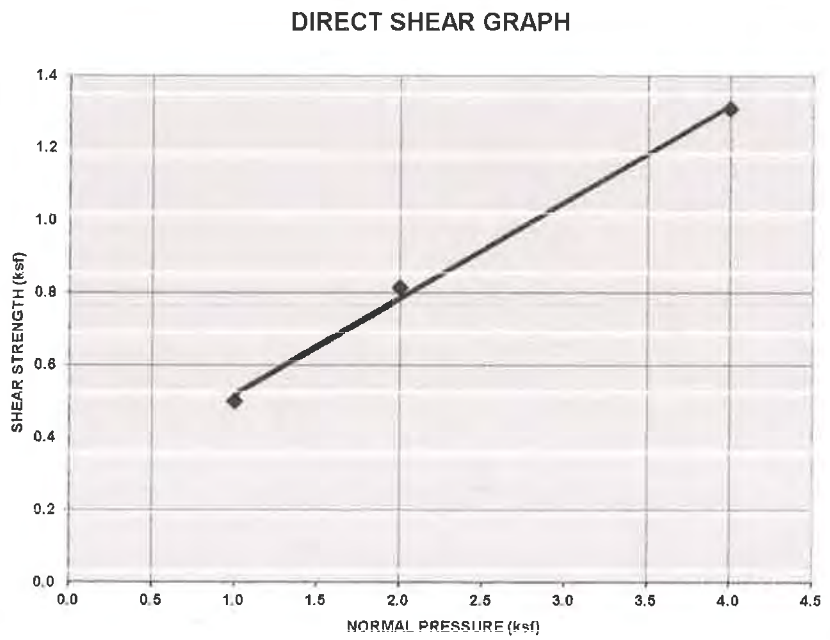
Reviewed by: Joe Bouknight
Joseph Bouknight, P.E., C81517

♦ *4035 Pacific Highway, San Diego, CA 92110* ♦ *Tel: 619-583-6633* ♦
*www.gforceca.com*

AGS Inc. Project No: 1406-06

# DIRECT SHEAR TEST REPORT

| | |
|---|---|
| G-FORCE LAB NO.: | 10066 |
| SAMPLE LOCATION: | TP- 4 @ 2-4' |
| SOIL TYPE: | Redish brown Clay (CL) |
| SAMPLE TYPE: | Remolded Max 108.5 @ 14.0 % M.C. 90% R.C. |




CALCULATED DATA

| | | | | | |
|---|---|---|---|---|---|
| INITIAL | | | | | |
| | WET DENSITY | pcf | 111.2 | 111.2 | 111.4 |
| | DRY DENSITY | pcf | 97.6 | 97.5 | 97.7 |
| | MOISTURE | % | 14.0 | 14.0 | 14.0 |
| FINAL, at failure | | | | | |
| | MOISTURE | % | 35.3 | 33.3 | 31.3 |

| | | | |
|---|---|---|---|
| NORMAL PRESSURE, ksf | 1.00 | 2.00 | 4.00 |
| SHEAR STRENGTH, ksf | 0.50 | 0.81 | 1.31 |
| FRICTION ANGLE, degrees | | 14.9 | |
| COHESION, ksf | | 0.25 | |

Reviewed by: Joe Bouknight
Joseph Bouknight, P.E. C81517

G FORCE®
DVBE SDVOSB SBE

## SULFATE / CHLORIDE CONTENT TEST RESULTS

| SAMPLE | Water-Soluble Sulfate Content (% of Dry Soil Wt.) (ASTM D 516) |
|---|---|
| S-4 @ 0' – 2' | 0.02 |

| Water Soluble Sulfate ($SO_4$) Content in % of Dry Soil Wt. | General Degree of Reactivity with Concrete |
|---|---|
| over 2.00 | Very Severely Reactive |
| 0.2 to 2.00 | Severely Reactive |
| 0.10 to 0.20 | Moderately Reactive |
| 0.00 to 0.10 | Negligible |

# Soil Corrosivity

(ASTM D4972,CTM 417,CTM 422)

| Lab Number | Boring No. | Depth | Sulfate % | Chloride % | PH | Resistivity (OHM-cm) |
|---|---|---|---|---|---|---|
| 10067 | TP-5 | 3.5-4.5' | 0.072 | 0.003 | 7.30 | 304 |

**Sulfate and Chloride content test were performed by So. Cal. Soils & Testing Inc.**

Date Sampled: 7/10/2014
Sampled By: PWM
Date Submitted: 7/25/2014
Submitted By: PJD

Reviewed by: Joe Bouknight

Joseph Bouknight, P.E., C81517

4035 Pacific Highway, San Diego, CA 92110 (619)583-6633
www.gforceca.com

## EXPANSION INDEX TESTS
(ASTM D 4829)

| SAMPLE NO. | SAMPLE DESCRIPTION | EXPANSION INDEX | EXPANSION POTENTIAL |
|---|---|---|---|
| S-4 @ 0'-2' | Brown clayey SAND | 31 | Low |

## 1997 UBC TABLE NO. 18-I-B, CLASSIFICATION OF EXPANSIVE SOIL

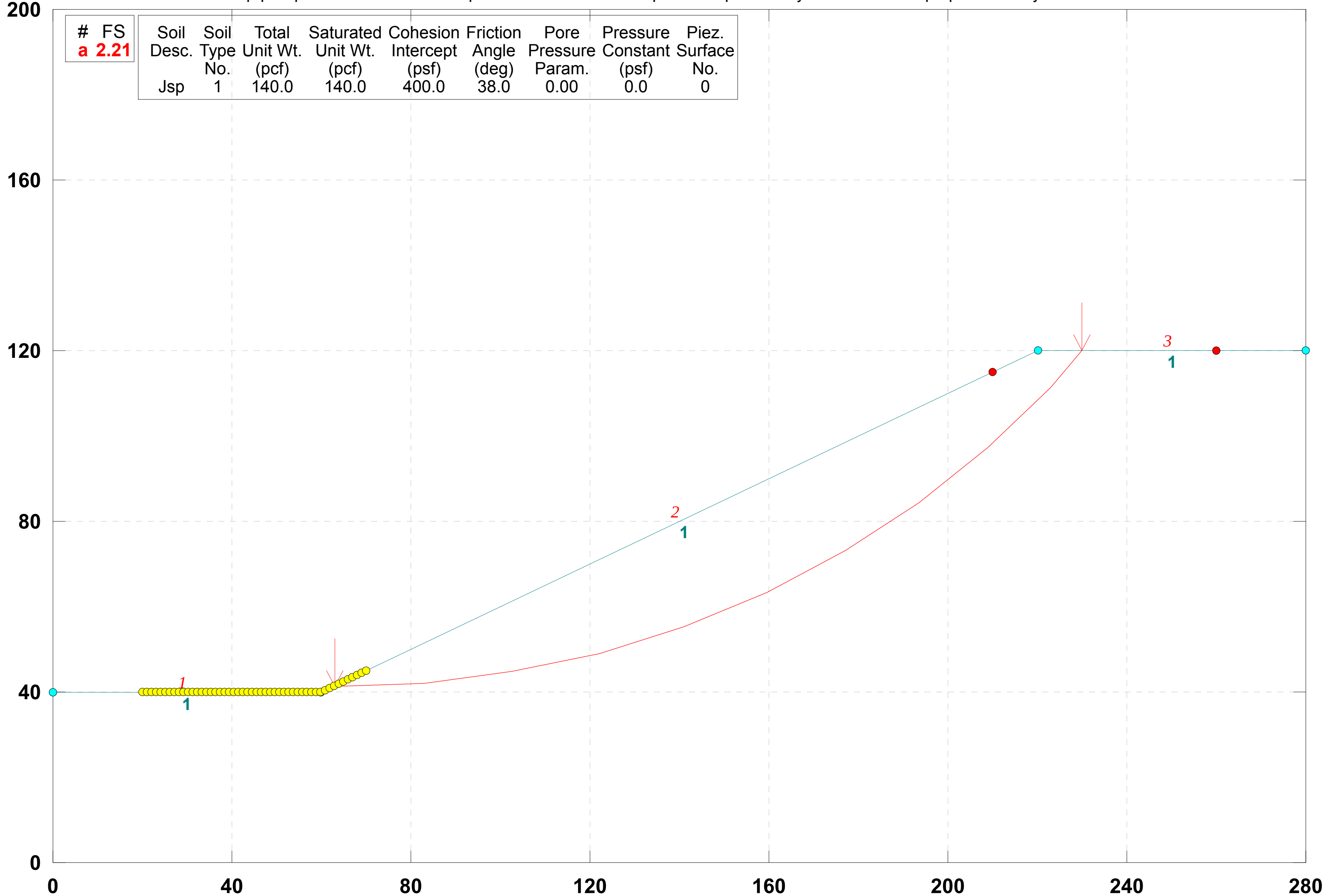
| EXPANSION INDEX | POTENTIAL EXPANSION |
|---|---|
| 0-20 | Very Low |
| 21-50 | Low |
| 51-90 | Medium |
| 91-130 | High |
| Above 130 | Very High |

# Appendix D

## Slope Stability

## 1406-06 Questhaven 80ft Cut Slope

c:\users\nick\desktop\phillip\14\1406-06 colrich - questhaven\1406-06-b-6 prelim\slope stability\1406-06 cut slope.pl2 Run By: PWM 3/11/2016 09:42AM




**GSTABL7 v.2 FSmin=2.21**

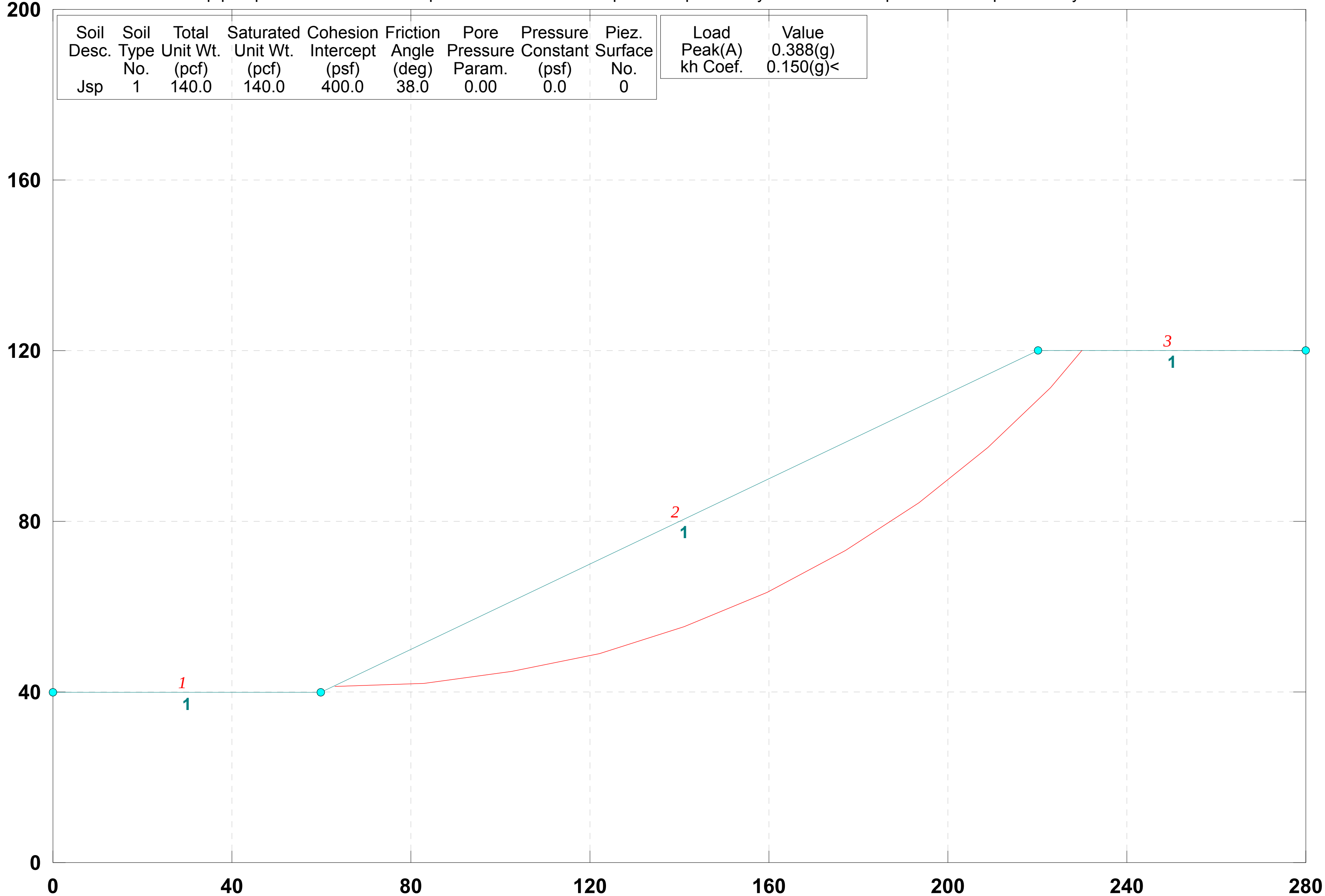
**Safety Factors Are Calculated By The Modified Bishop Method**

# 1406-06 Questhaven 80ft Cut Slope (PS)

c:\users\nick\desktop\phillip\14\1406-06 colrich - questhaven\1406-06-b-6 prelim\slope stability\1406-06 cut slope surface #1.plt   Run By: PWM   3/11/2016   10:01AM

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Jsp | 1 | 140.0 | 140.0 | 400.0 | 38.0 | 0.00 | 0.0 | 0 |

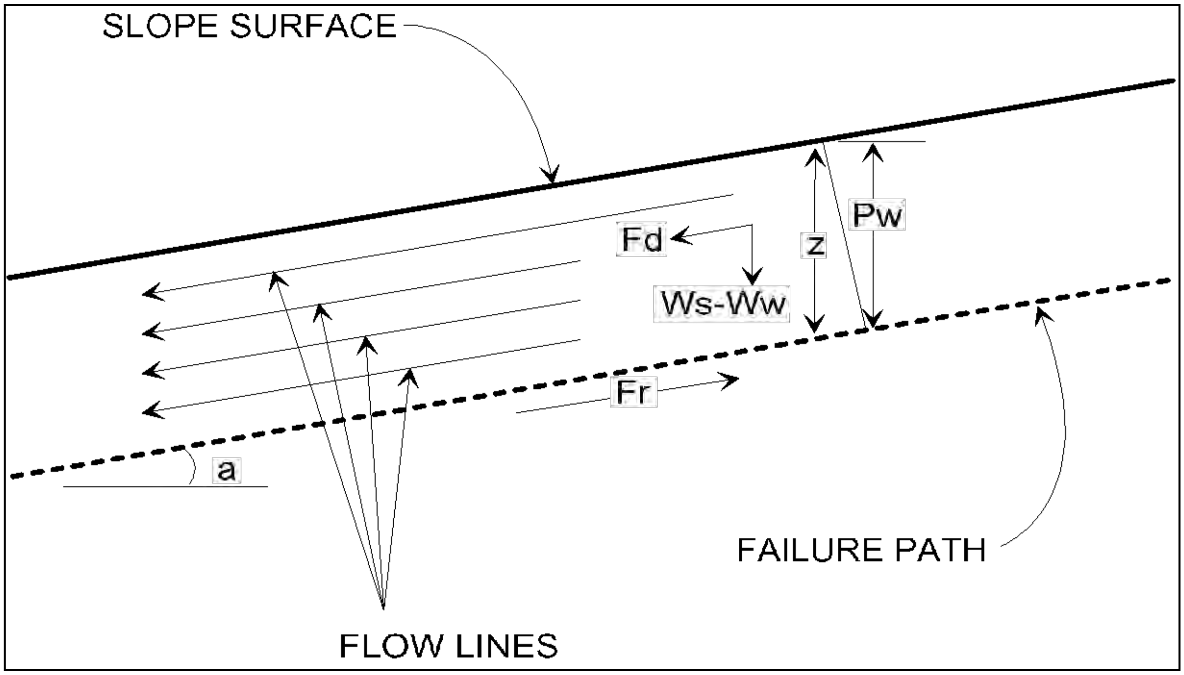
| Load | Value |
|---|---|
| Peak(A) | 0.388(g) |
| kh Coef. | 0.150(g)< |




**GSTABL7 v.2  FSmin=1.61**

**Factor Of Safety Is Calculated By The Modified Bishop Method**

# SURFICIAL SLOPE STABILITY 2:1 CUT




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

Given:

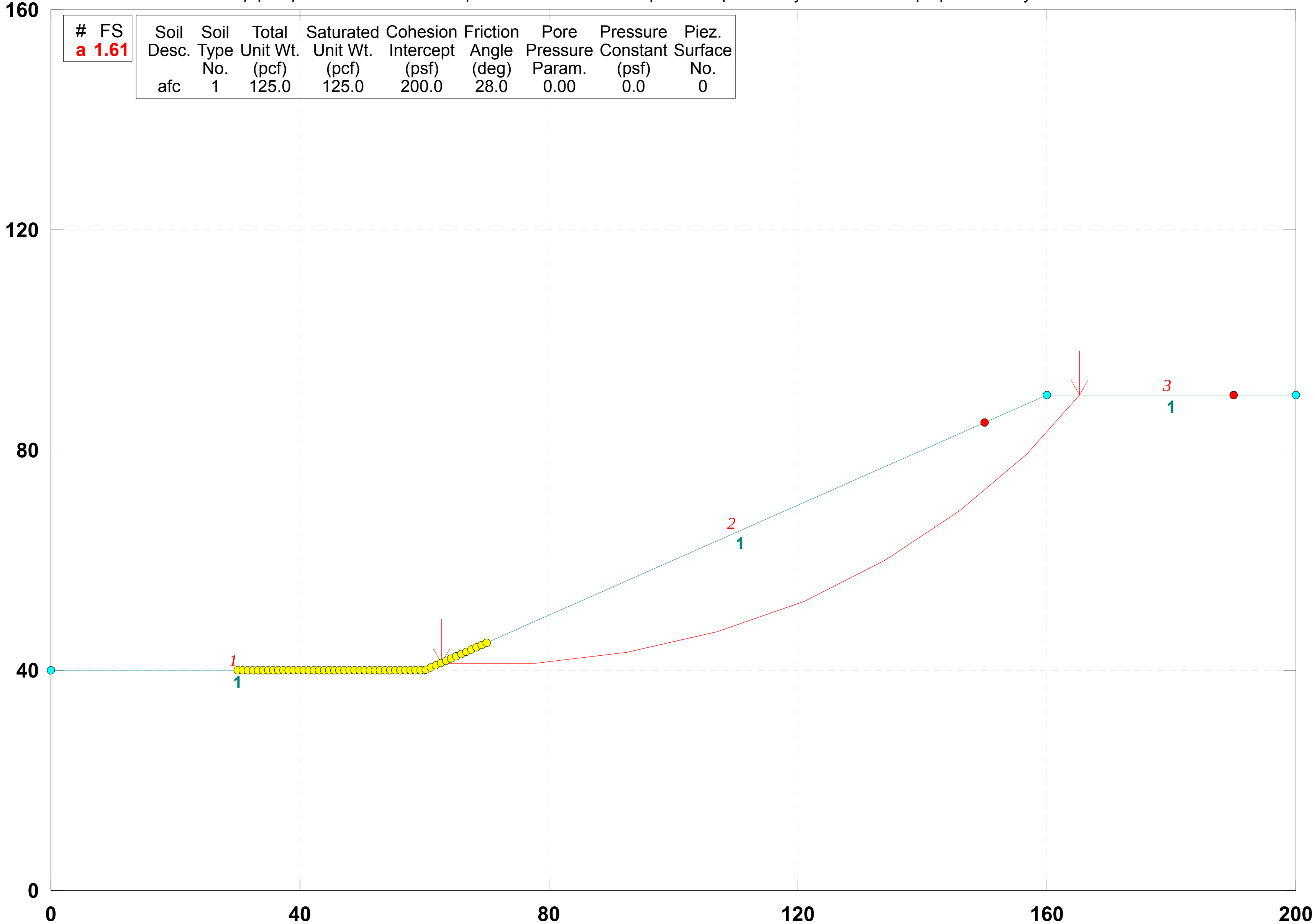
| Ws | z | a | | phi | | c |
|---|---|---|---|---|---|---|
| (pcf) | (ft) | (degrees) | (radians) | (degrees) | (radians) | (psf) |
| 140 | 5 | 26.56505 | 0.463648 | 38 | 0.663225 | 400 |

Calculations:

| Pw | u | Fd | Fr | **FS** |
|---|---|---|---|---|
| 4.00 | 249.60 | 280.00 | 642.51 | **2.29** |

# 1406-06 Questhaven 50ft Fill Slope

c:\users\nick\desktop\phillip\14\1406-06 colrich - questhaven\1406-06-b-6 prelim\slope stability\1406-06 fill slope.pl2 Run By: PWM 3/11/2016 09:45AM

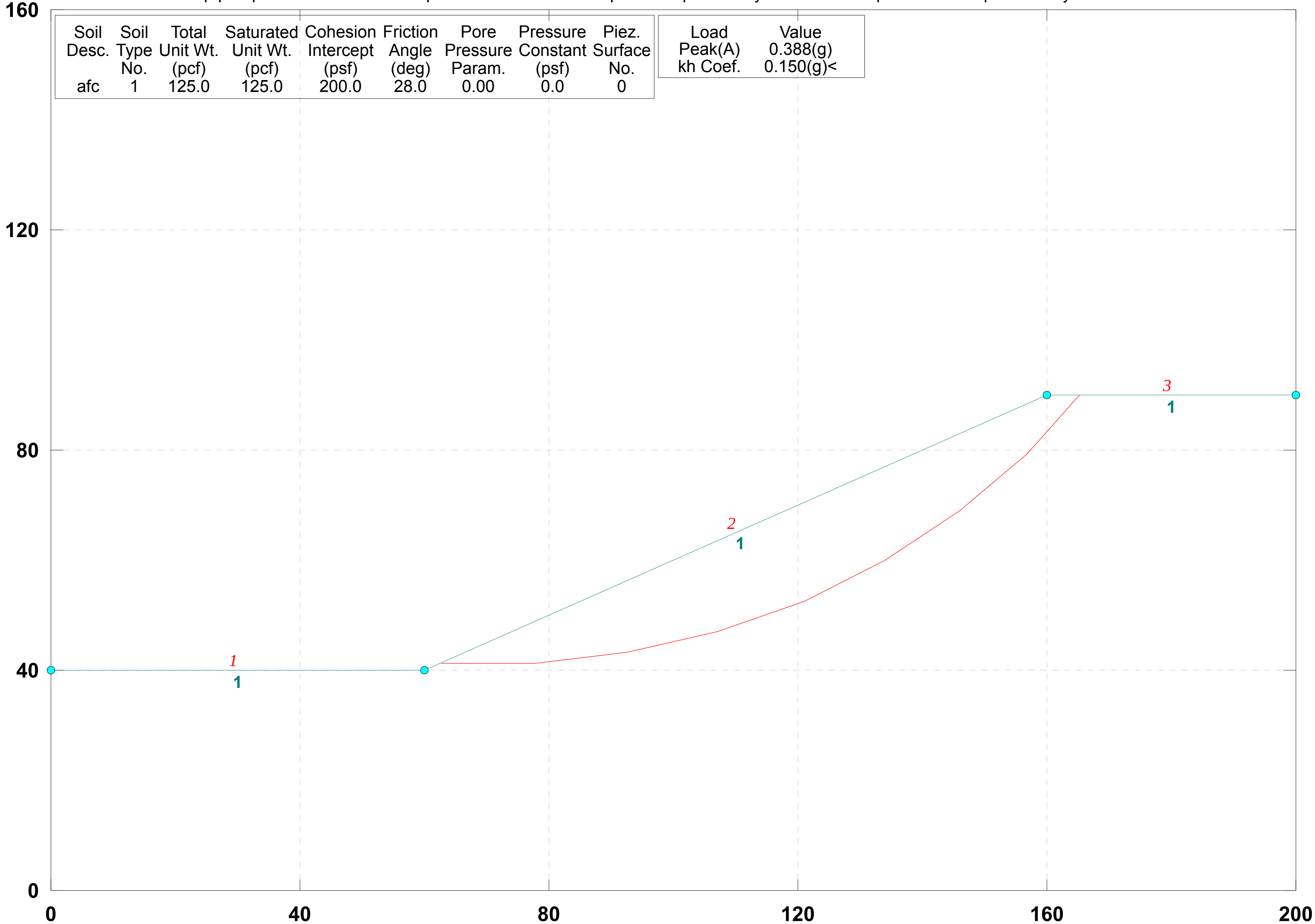



GSTABL7 v.2 FSmin=1.61

Safety Factors Are Calculated By The Modified Bishop Method

# 1406-06 Questhaven 50ft Fill Slope (PS)

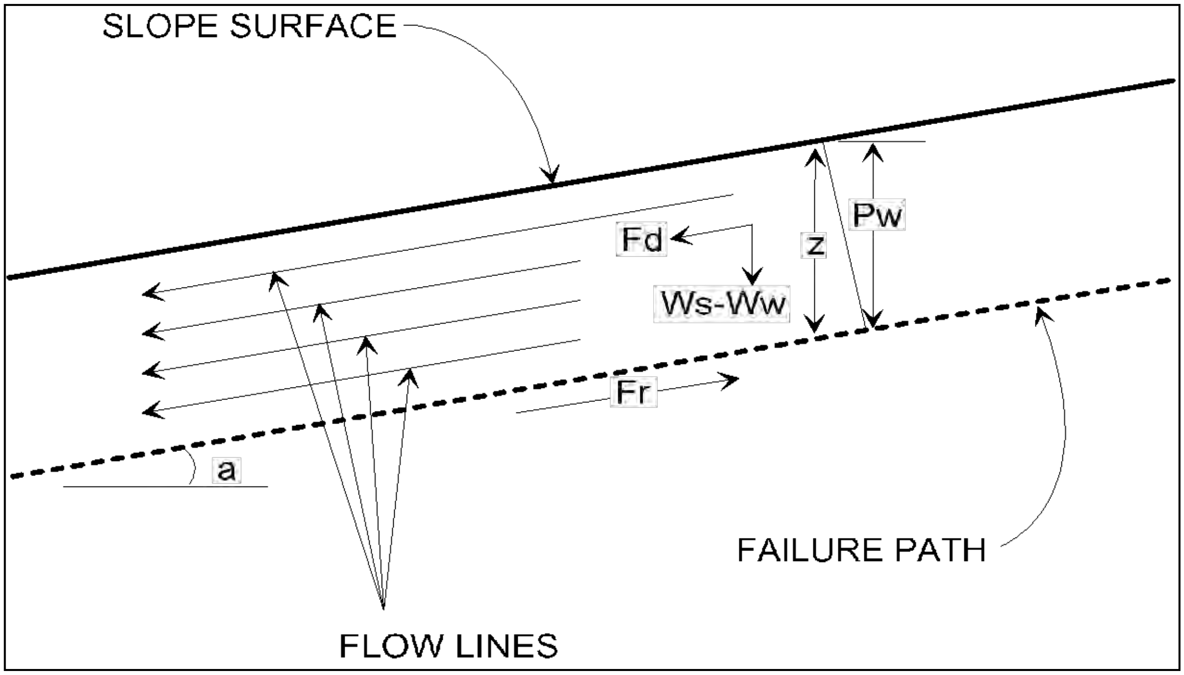
c:\users\nick\desktop\phillip\14\1406-06 colrich - questhaven\1406-06-b-6 prelim\slope stability\1406-06 fill slope surface #1.plt Run By: PWM 3/11/2016 10:01AM




**GSTABL7 v.2 FSmin=1.17**

**Factor Of Safety Is Calculated By The Modified Bishop Method**

# SURFICIAL SLOPE STABILITY 2:1 FILL




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

Given:

| Ws (pcf) | z (ft) | a (degrees) | a (radians) | phi (degrees) | phi (radians) | c (psf) |
|---|---|---|---|---|---|---|
| 125 | 4 | 26.56505 | 0.463648 | 28 | 0.488692 | 200 |

Calculations:

| Pw | u | Fd | Fr | **FS** |
|---|---|---|---|---|
| 3.20 | 199.68 | 200.00 | 306.51 | **1.53** |

# Appendix E

## General Earthwork Specifications and Grading Guidelines

GENERAL EARTHWORK SPECIFICATIONS

**I. General**

A. General procedures and requirements for earthwork and grading are presented herein. The earthwork and grading recommendations provided in the geotechnical report are considered part of these specifications, and where the general specifications provided herein conflict with those provided in the geotechnical report, the recommendations in the geotechnical report shall govern. Recommendations provided herein and in the geotechnical report may need to be modified depending on the conditions encountered during grading.

B. The contractor is responsible for the satisfactory completion of all earthwork in accordance with the project plans, specifications, applicable building codes, and local governing agency requirements. Where these requirements conflict, the stricter requirements shall govern.

C. It is the contractor's responsibility to read and understand the guidelines presented herein and in the geotechnical report as well as the project plans and specifications. Information presented in the geotechnical report is subject to verification during grading. The information presented on the exploration logs depicts conditions at the particular time of excavation and at the location of the excavation. Subsurface conditions present at other locations may differ, and the passage of time may result in different subsurface conditions being encountered at the locations of the exploratory excavations. The contractor shall perform an independent investigation and evaluate the nature of the surface and subsurface conditions to be encountered and the procedures and equipment to be used in performing his work.

D. The contractor shall have the responsibility to provide adequate equipment and procedures to accomplish the earthwork in accordance with applicable requirements. When the quality of work is less than that required, the Geotechnical Consultant may reject the work and may recommend that the operations be suspended until the conditions are corrected.

E. Prior to the start of grading, a qualified Geotechnical Consultant should be employed to observe grading procedures and provide testing of the fills for conformance with the project specifications, approved grading plan, and guidelines presented herein. All remedial removals, clean-outs, removal bottoms, keyways, and subdrain installations should be observed and documented by the Geotechnical Consultant prior to placing fill. It is the contractor's responsibility to apprise the Geotechnical Consultant of their schedules and notify the Geotechnical Consultant when those areas are ready for observation.

F. The contractor is responsible for providing a safe environment for the Geotechnical Consultant to observe grading and conduct tests.

**II. Site Preparation**

A. Clearing and Grubbing: Excessive vegetation and other deleterious material shall be sufficiently removed as required by the Geotechnical Consultant, and such materials shall be properly disposed of offsite in a method acceptable to the owner and governing agencies. Where applicable, the contractor may obtain permission from the Geotechnical Consultant, owner, and governing agencies to dispose of vegetation and other deleterious materials in designated areas onsite.

B. Unsuitable Soils Removals: Earth materials that are deemed unsuitable for the support of fill shall be removed as necessary to the satisfaction of the Geotechnical Consultant.

C. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, other utilities, or other structures located within the limits of grading shall be removed and/or abandoned in accordance with the requirements of the governing agency and to the satisfaction of the Geotechnical Consultant.

D. Preparation of Areas to Receive Fill: After removals are completed, the exposed surfaces shall be scarified to a depth of approximately 8 inches, watered or dried, as needed, to achieve a generally uniform moisture content that is at or near optimum moisture content. The scarified materials shall then be compacted to the project requirements and tested as specified.

E. All areas receiving fill shall be observed and approved by the Geotechnical Consultant prior to the placement of fill. A licensed surveyor shall provide survey control for determining elevations of processed areas and keyways.

**III. Placement of Fill**

A. Suitability of fill materials: Any materials, derived onsite or imported, may be utilized as fill provided that the materials have been determined to be suitable by the Geotechnical Consultant. Such materials shall be essentially free of organic matter and other deleterious materials, and be of a gradation, expansion potential, and/or strength that is acceptable to the Geotechnical Consultant. Fill materials shall be tested in a laboratory approved by the Geotechnical Consultant, and import materials shall be tested and approved prior to being imported.

B. Generally, different fill materials shall be thoroughly mixed to provide a relatively uniform blend of materials and prevent abrupt changes in material type. Fill materials derived from benching should be dispersed throughout the fill area instead of placing the materials within only an equipment-width from the cut/fill contact.

C. Oversize Materials: Rocks greater than 8 inches in largest dimension shall be disposed of offsite or be placed in accordance with the recommendations by the Geotechnical Consultant in the areas that are designated as suitable for oversize rock placement. Rocks that are smaller than 8 inches in largest dimension may be utilized in the fill provided that they are not nested and are their quantity and distribution are acceptable to the Geotechnical Consultant.

D. The fill materials shall be placed in thin, horizontal layers such that, when compacted, shall not exceed 6 inches. Each layer shall be spread evenly and shall be thoroughly mixed to obtain near uniform moisture content and uniform blend of materials.

E. Moisture Content: Fill materials shall be placed at or above the optimum moisture content or as recommended by the geotechnical report. Where the moisture content of the engineered fill is less than recommended, water shall be added, and the fill materials shall be blended so that near uniform moisture content is achieved. If the moisture content is above the limits specified by the Geotechnical Consultant, the fill materials shall be aerated by discing, blading, or other methods until the moisture content is acceptable.

F. Each layer of fill shall be compacted to the project standards in accordance to the project specifications and recommendations of the Geotechnical Consultant. Unless otherwise specified by the Geotechnical Consultant, the fill shall be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method: D1557-09.

G. Benching: Where placing fill on a slope exceeding a ratio of 5 to 1 (horizontal to vertical), the ground should be keyed or benched. The keyways and benches shall extend through all unsuitable materials into suitable materials such as firm materials or sound bedrock or as recommended by the Geotechnical Consultant. The minimum keyway width shall be 15 feet and extend into suitable materials, or as recommended by the geotechnical report and approved by the Geotechnical Consultant. The minimum keyway width for fill over cut slopes is also 15 feet, or as recommended by the geotechnical report and approved by the Geotechnical Consultant. As a general rule, unless otherwise recommended by the Geotechnical Consultant, the minimum width of the keyway shall be equal to 1/2 the height of the fill slope.

H. Slope Face: The specified minimum relative compaction shall be maintained out to the finish face of fill and stabilization fill slopes. Generally, this may be achieved by overbuilding the slope and cutting back to the compacted core. The actual amount of overbuilding may vary as field conditions dictate. Alternately, this may be achieved by back rolling the slope face with suitable equipment or other methods that produce the designated result. Loose soil should not be allowed to build up on the slope face. If present, loose soils shall be trimmed to expose the compacted slope face.

I. Slope Ratio: Unless otherwise approved by the Geotechnical Consultant and governing agencies, permanent fill slopes shall be designed and constructed no steeper than 2 to 1 (horizontal to vertical).

J. Natural Ground and Cut Areas: Design grades that are in natural ground or in cuts should be evaluated by the Geotechnical Consultant to determine whether scarification and processing of the ground and/or overexcavation is needed.

K. Fill materials shall not be placed, spread, or compacted during unfavorable weather conditions. When grading is interrupted by rain, filing operations shall not resume until the Geotechnical Consultant approves the moisture and density of the previously placed compacted fill.

## IV. Cut Slopes

A. The Geotechnical Consultant shall inspect all cut slopes, including fill over cut slopes, and shall be notified by the contractor when cut slopes are started.

B. If adverse or potentially adverse conditions are encountered during grading; the Geotechnical Consultant shall investigate, evaluate, and make recommendations to mitigate the adverse conditions.

C. Unless otherwise stated in the geotechnical report, cut slopes shall not be excavated higher or steeper than the requirements of the local governing agencies. Short-term stability of the cut slopes and other excavations is the contractor's responsibility.

## V. Drainage

A. Back drains and Subdrains: Back drains and subdrains shall be provided in fill as recommended by the Geotechnical Consultant and shall be constructed in accordance with the governing agency and/or recommendations of the Geotechnical Consultant. The location of subdrains, especially outlets, shall be surveyed and recorded by the Civil Engineer.

B. Top-of-slope Drainage: Positive drainage shall be established away from the top of slope. Site drainage shall not be permitted to flow over the tops of slopes.

C. Drainage terraces shall be constructed in compliance with the governing agency requirements and/or in accordance with the recommendations of the Geotechnical Consultant.

D. Non-erodible interceptor swales shall be placed at the top of cut slopes that face the same direction as the prevailing drainage.

**VI. Erosion Control**

A. All finish cut and fill slopes shall be protected from erosion and/or planted in accordance with the project specifications and/or landscape architect's recommendations. Such measures to protect the slope face shall be undertaken as soon as practical after completion of grading.

B. During construction, the contractor shall maintain proper drainage and prevent the ponding of water. The contractor shall take remedial measures to prevent the erosion of graded areas until permanent drainage and erosion control measures have been installed.

**VII. Trench Excavation and Backfill**

A. Safety: The contractor shall follow all OSHA requirements for safety of trench excavations. Knowing and following these requirements is the contractor's responsibility. All trench excavations or open cuts in excess of 5 feet in depth shall be shored or laid back. Trench excavations and open cuts exposing adverse geologic conditions may require further evaluation by the Geotechnical Consultant. If a contractor fails to provide safe access for compaction testing, backfill not tested due to safety concerns may be subject to removal.

B. Bedding: Bedding materials shall be non-expansive and have a Sand Equivalent greater than 30. Where permitted by the Geotechnical Consultant, the bedding materials can be densified by jetting.

C. Backfill: Jetting of backfill materials is generally not acceptable. Where permitted by the Geotechnical Consultant, the bedding materials can be densified by jetting provided the backfill materials are granular, free-draining and have a Sand Equivalent greater than 30.

**VIII. Geotechnical Observation and Testing During Grading**

A. Compaction Testing: Fill shall be tested by the Geotechnical Consultant for evaluation of general compliance with the recommended compaction and moisture conditions. The tests shall be taken in the compacted soils beneath the surface if the surficial materials are disturbed. The contractor shall assist the Geotechnical Consultant by excavating suitable test pits for testing of compacted fill.

B. Where tests indicate that the density of a layer of fill is less than required, or the moisture content not within specifications, the Geotechnical Consultant shall notify the contractor of the unsatisfactory conditions of the fill. The portions of the fill that are not within specifications shall be reworked until the required density and/or moisture content has been attained. No additional fill shall be placed until the last lift of fill is tested and found to meet the project specifications and approved by the Geotechnical Consultant.

C. If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as adverse weather, excessive rock or deleterious materials being placed in the fill, insufficient equipment, excessive rate of fill placement, results in a quality of work that is unacceptable, the consultant shall notify the contractor, and the contractor shall rectify the conditions, and if necessary, stop work until conditions are satisfactory.

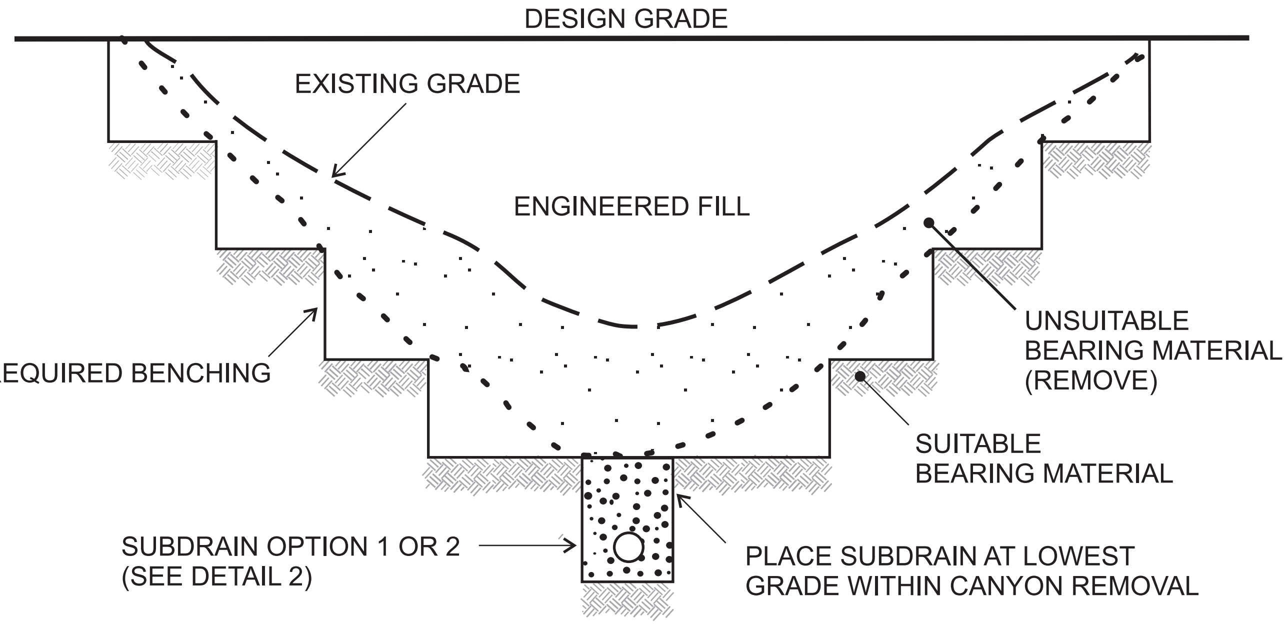
D. Frequency of Compaction Testing: The location and frequency of tests shall be at the Geotechnical Consultant's discretion. Generally, compaction tests shall be taken at intervals not exceeding two feet in fill height and 1,000 cubic yards of fill materials placed.

E. Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of the compaction test locations. The contractor shall coordinate with the surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations. Alternately, the test locations can be surveyed and the results provided to the Geotechnical Consultant.

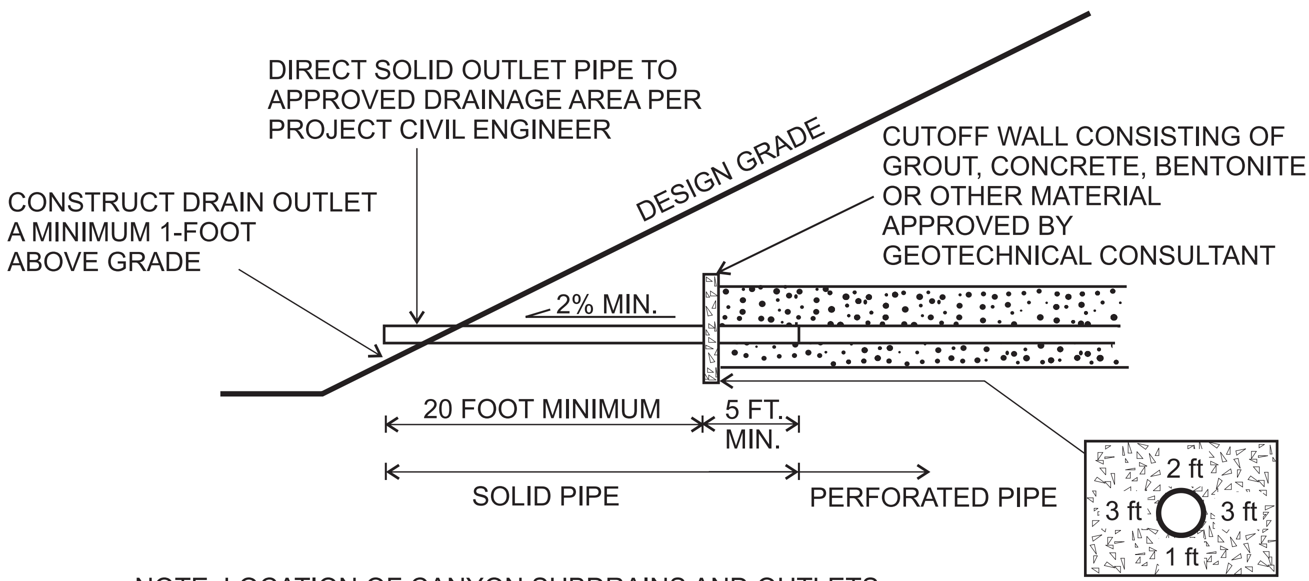
F. Areas of fill that have not been observed or tested by the Geotechnical Consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removals will be determined by the Geotechnical Consultant.

G. Observation and testing by the Geotechnical Consultant shall be conducted during grading in order for the Geotechnical Consultant to state that, in his opinion, grading has been completed in accordance with the approved geotechnical report and project specifications.

H. Reporting of Test Results: After completion of grading operations, the Geotechnical Consultant shall submit reports documenting their observations during construction and test results. These reports may be subject to review by the local governing agencies.




## CANYON SUBDRAIN PROFILE




NOTE: LOCATION OF CANYON SUBDRAINS AND OUTLETS SHOULD BE DOCUMENTED BY PROJECT CIVIL ENGINEER. OUTLETS MUST BE KEPT UNOBSTRUCTED AT ALL TIMES.

CUTOFF WALL DIMENSIONS

## CANYON SUBDRAIN TERMINUS

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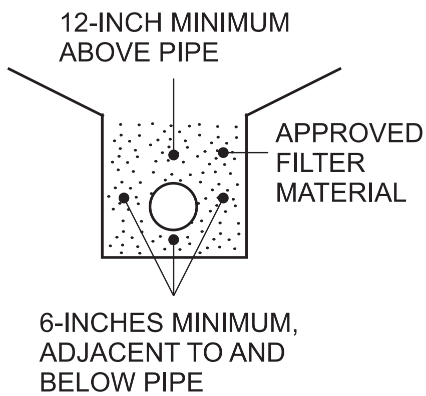
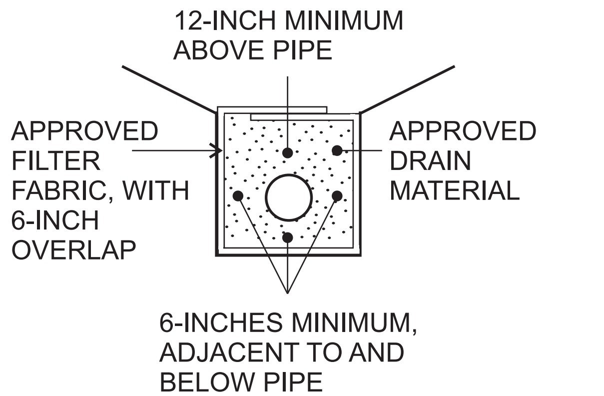
|---|---|---|
| AGS ADVANCED GEOTECHNICAL SOLUTIONS | CANYON SUBDRAIN | DETAIL 1 |




## OPTION 1

FILTER MATERIAL: MINIMUM VOLUME OF 9 CUBIC FEET PER LINEAL FOOT OF CALTRANS CLASS 2 PERMEABLE MATERIAL




## OPTION 2

DRAIN MATERIAL: MINIMUM VOLUME OF 9 CUBIC FEET PER LINEAL FOOT OF 3/4-INCH MAX ROCK OR APPROVED EQUIVALENT SUBSTITUTE

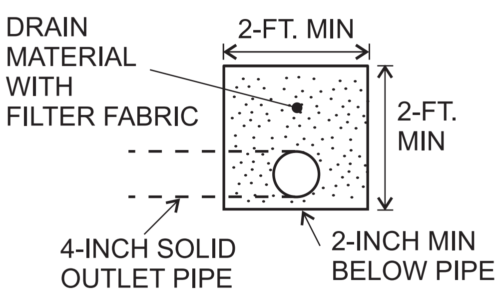
FILTER FABRIC: MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

PIPE: 6 OR 8-INCH ABS OR PVC PIPE OR APPROVED SUBSTITUTE WITH A MINIMUM OF 8 PERFORATIONS (1/4-INCH DIAMETER) PER LINEAL FOOT IN BOTTOM HALF OF PIPE

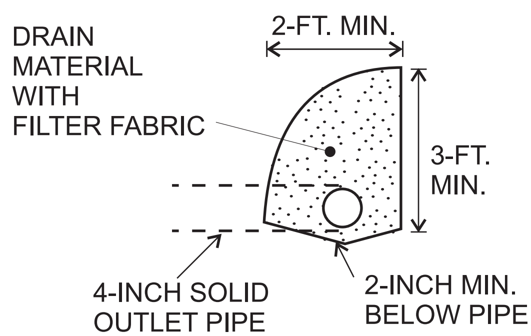
(ASTM D2751, SDR-35 OR ASTM D3034, SDR-35
ASTM D1527, SCHD. 40 OR ASTM D1785, SCHD. 40)

NOTE: CONTINUOUS RUN IN EXCESS OF 5OO FEET REQUIRES 8-INCH DIAMETER PIPE (ASTM D3034, SDR-35, OR ASTM D1785, SCHD. 40)

# CANYON SUBDRAIN




## OPTION 1




## OPTION 2

DRAIN MATERIAL: GRAVEL TRENCH TO BE FILLED WITH 3/4-INCH MAX ROCK OR APPROVED EQUIVALENT SUBSTITUTE

FILTER FABRIC: MIRAFI 140 FILTER FABRIC OR EQUIVALENT SUBSTITUTE WITH A MINIMUM 6-INCH OVERLAP

PIPE: 4-INCH ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE WITH A MINIMUM OF 8 PERFORATIONS (1/4-INCH DIAMETER) PER LINEAL FOOT IN BOTTOM HALF OF PIPE

(ASTM D2751, SDR-35 OR ASTM D3034, SDR-35
ASTM D1527, SCHD. 40 OR ASTM D1785, SCHD. 40)

# BUTTRESS/STABILIZATION DRAIN

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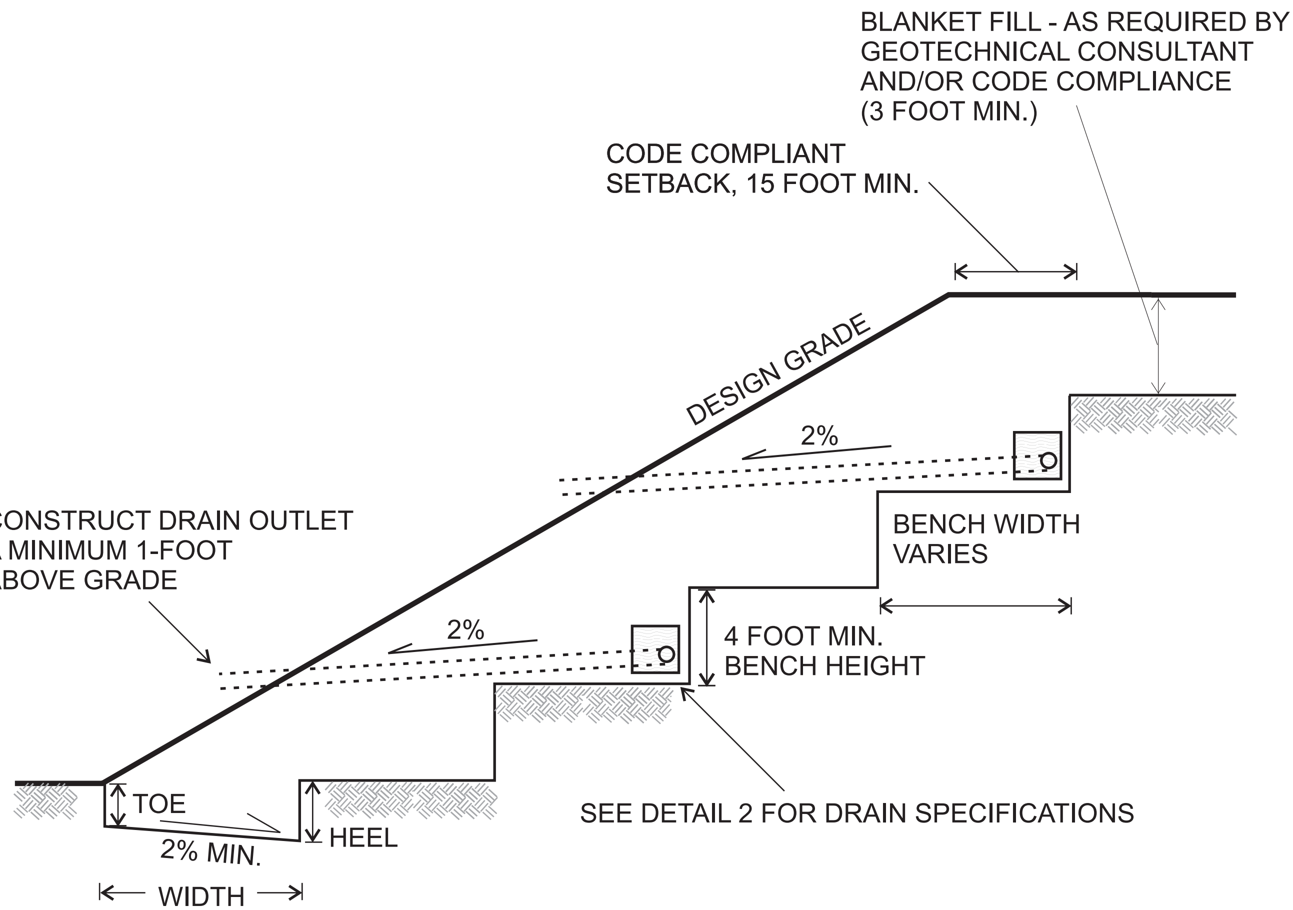
ADVANCED GEOTECHNICAL SOLUTIONS

DRAIN SPECIFICATIONS

DETAIL 2




CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE 2 FOOT MIN.
HEEL 3 FOOT MIN.
WIDTH 15 FOOT MIN.

NOTES:

1. DRAIN OUTLETS TO BE PROVIDED EVERY 100 FEET CONNECT TO PERFORATED DRAIN PIPE BY “L” OR “T” AT A MINIMUM 2% GRADIENT.

2. THE NECESSITY AND LOCATION OF ADDITIONAL DRAINS SHALL BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL CONSULTANT. UPPER STAGE OUTLETS SHOULD BE EMPTIED ONTO CONCRETE TERRACE DRAINS.

3. DRAIN PIPE TO EXTEND FULL LENGTH OF STABILIZATION/BUTTRESS WITH A MINIMUM GRADIENT OF 2% TO SOLID OUTLET PIPES.

4. LOCATION OF DRAINS AND OUTLETS SHOULD BE DOCUMENTED BY PROJECT CIVIL ENGINEER. OUTLETS MUST BE KEPT UNOBSTRUCTED AT ALL TIMES.

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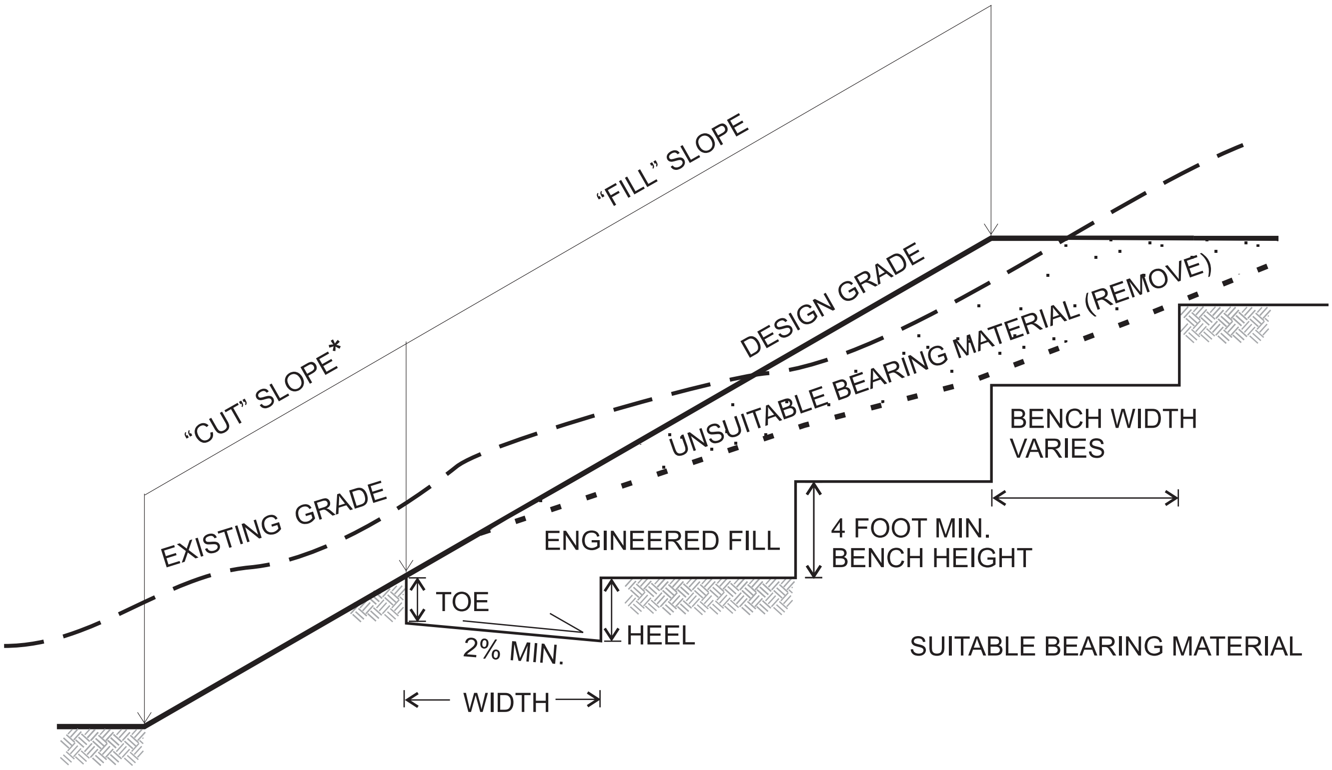
| AGS ADVANCED GEOTECHNICAL SOLUTIONS | STABILIZATION/BUTTRESS FILL | DETAIL 3 |

* THE “CUT” PORTION OF THE SLOPE SHALL BE EXCAVATED AND EVALUATED BY THE GEOTECHNICAL CONSULTANT PRIOR TO CONSTRUCTING THE “FILL” PORTION




SUITABLE
BEARING MATERIAL

CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

1. THE NECESSITY AND LOCATION OF DRAINS SHALL BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL CONSULTANT

2. SEE DETAIL 2 FOR DRAIN SPECIFICATIONS

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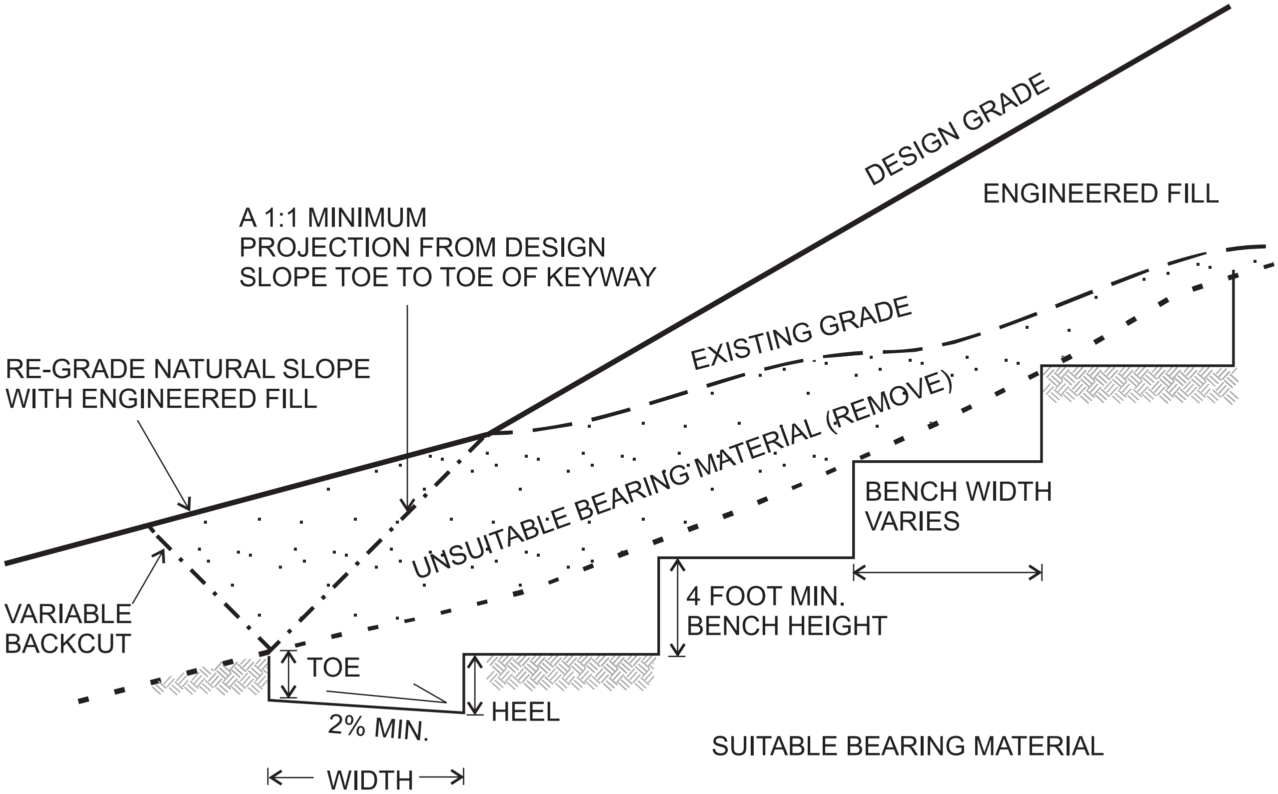
ADVANCED GEOTECHNICAL SOLUTIONS

FILL OVER CUT SLOPE

DETAIL 4




CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

1. WHEN THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN GRADE SLOPE RATIO, SPECIAL RECOMMENDATIONS ARE NECESSARY BY THE GEOTECHNICAL CONSULTANT

2. THE GEOTECHNICAL CONSULTANT WILL DETERMINE THE REQUIREMENT FOR AND LOCATION OF SUBSURFACE DRAINAGE SYSTEMS.

3. MAINTAIN MINIMUM 15 FOOT HORIZONTAL WIDTH FROM FACE OF SLOPE TO BENCH/BACKCUT

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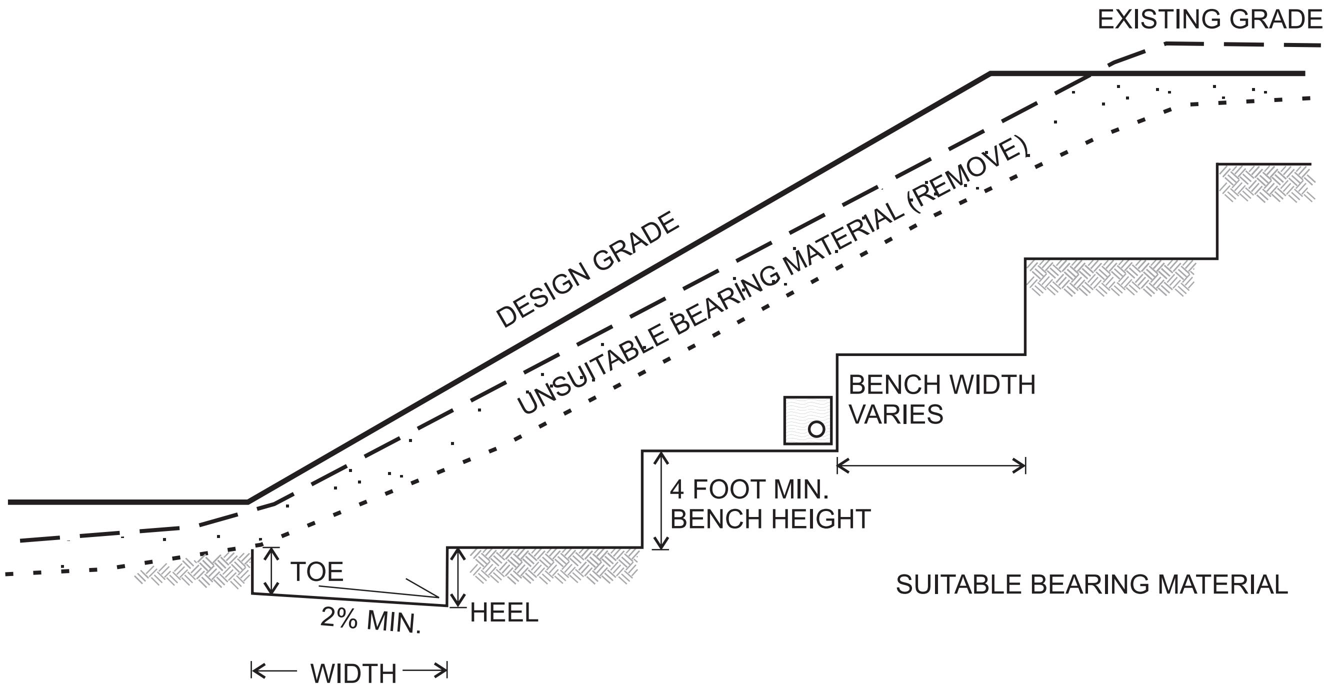
FILL OVER NATURAL SLOPE

DETAIL 5




CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

1. MAINTAIN MINIMUM 15 FOOT HORIZONTAL WIDTH FROM FACE OF SLOPE TO BENCH/BACKCUT
2. SEE DETAIL 2 FOR DRAIN SPECIFICATIONS

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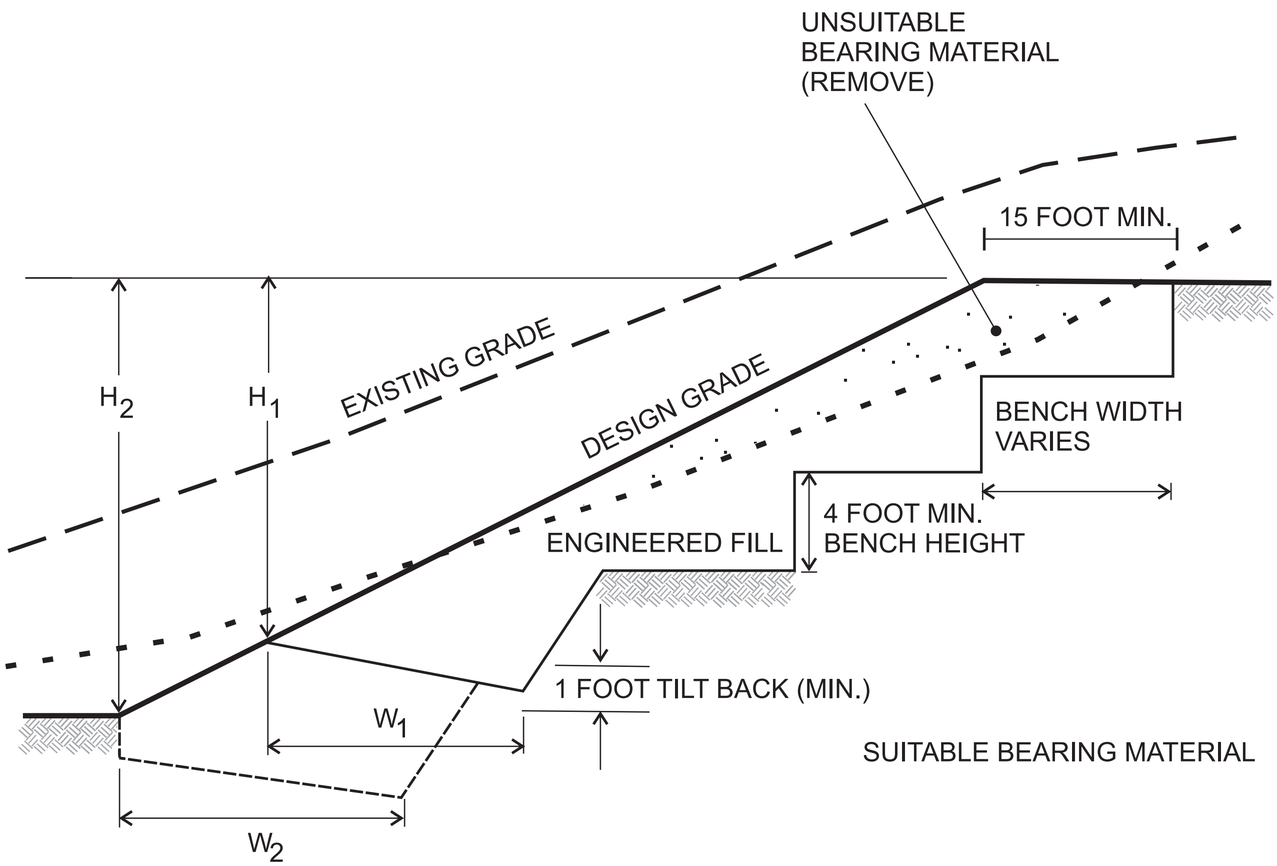
ADVANCED GEOTECHNICAL SOLUTIONS

# SKIN FILL CONDITION

DETAIL 6




NOTES:

1. IF RECOMMENDED BY THE GEOTECHNICAL CONSULTANT, THE REMAINING CUT PORTION OF THE SLOPE MAY REQUIRE REMOVAL AND REPLACEMENT WITH AN ENGINEERED FILL

2. “W” SHALL BE EQUIPMENT WIDTH (15 FEET) FOR SLOPE HEIGHT LESS THAN 25 FEET. FOR SLOPES GREATER THAN 25 FEET, “W” SHALL BE DETERMINED BY THE GEOTECHNICAL CONSULTANT. AT NO TIME SHALL “W” BE LESS THAN H/2

3. DRAINS WILL BE REQUIRED (SEE DETAIL 2)

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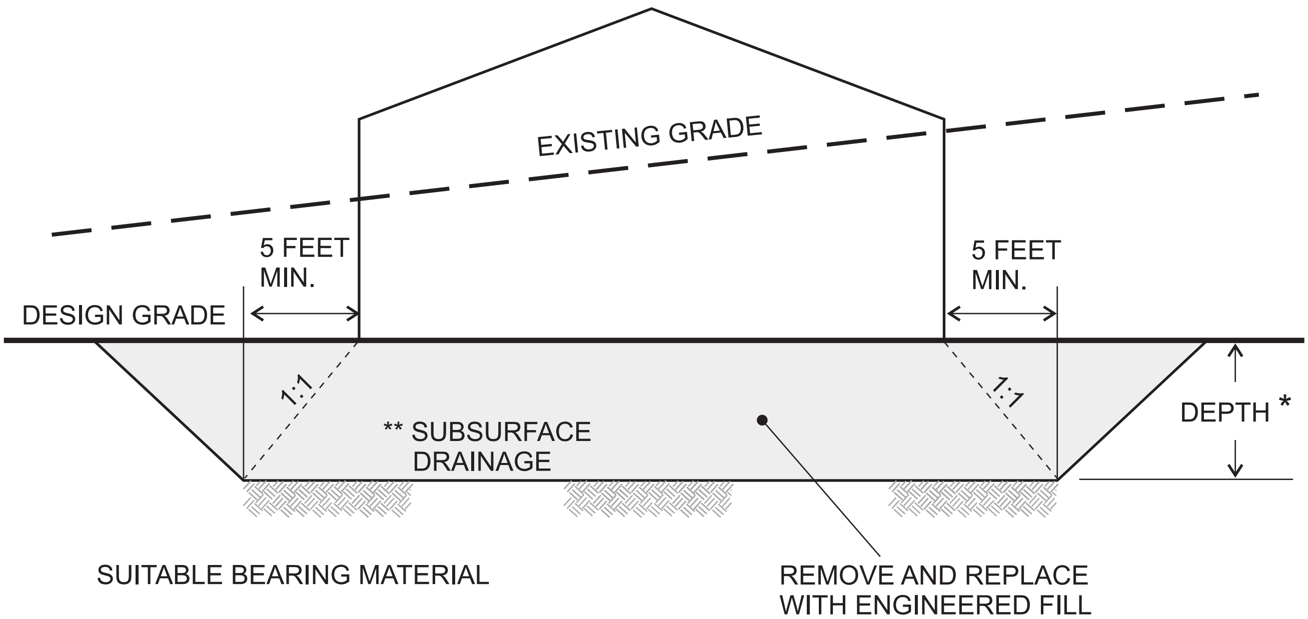
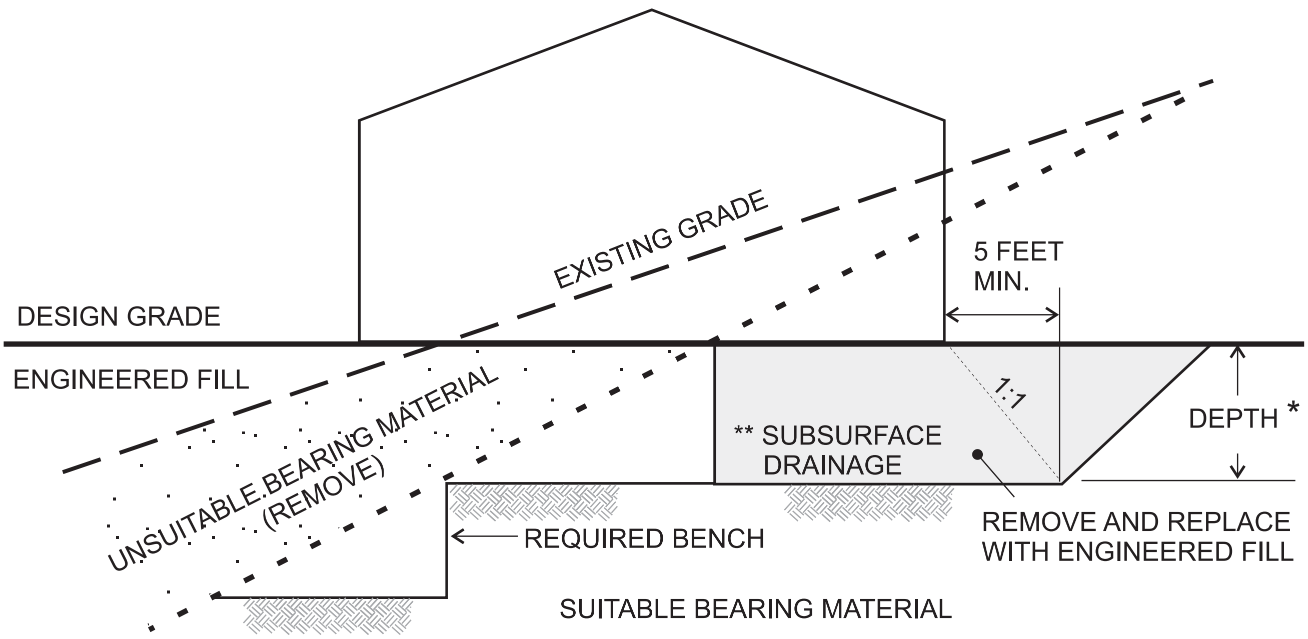
| AGS<br>ADVANCED GEOTECHNICAL SOLUTIONS | PARTIAL CUT SLOPE STABILIZATION | DETAIL 7 |




## CUT LOT OVEREXCAVATION




## CUT-FILL LOT OVEREXCAVATION

NOTES:

* SEE REPORT FOR RECOMMENDED DEPTHS, DEEPER OVEREXCAVATION MAY BE REQUIRED BY THE GEOTECHNICAL CONSULTANT BASED ON EXPOSED FIELD CONDITIONS

** CONSTRUCT EXCAVATION TO PROVIDE FOR POSITIVE DRAINAGE TOWARDS STREETS, DEEPER FILL AREAS OR APPROVED DRAINAGE DEVICES BASED ON FIELD CONDITIONS

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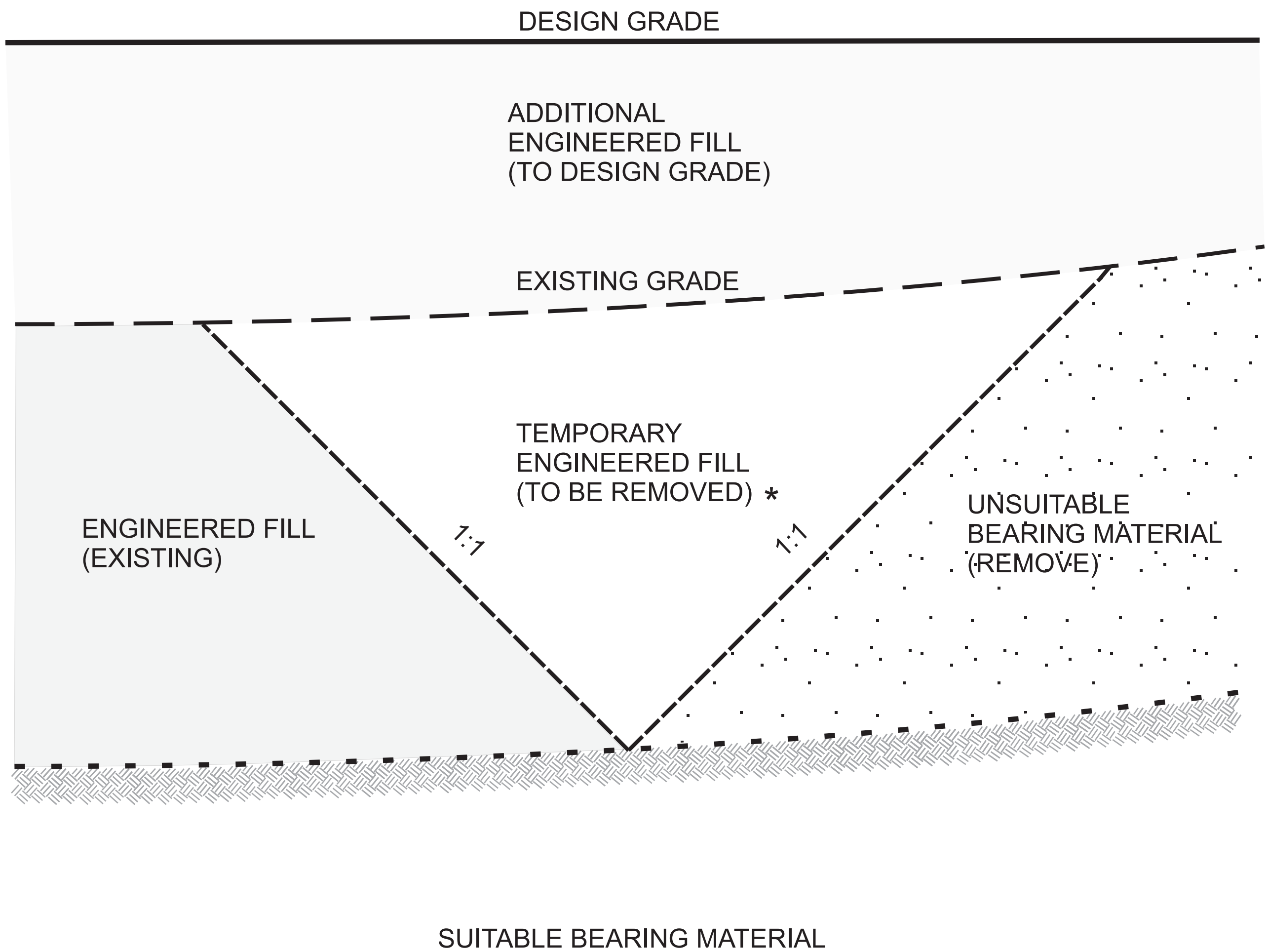
| AGS<br>ADVANCED GEOTECHNICAL SOLUTIONS | CUT & CUT-FILL LOT OVEREXCAVATION | DETAIL 8 |




* REMOVE BEFORE PLACING ADDITIONAL ENGINEERED FILL

# TYPICAL UP-CANYON PROFILE

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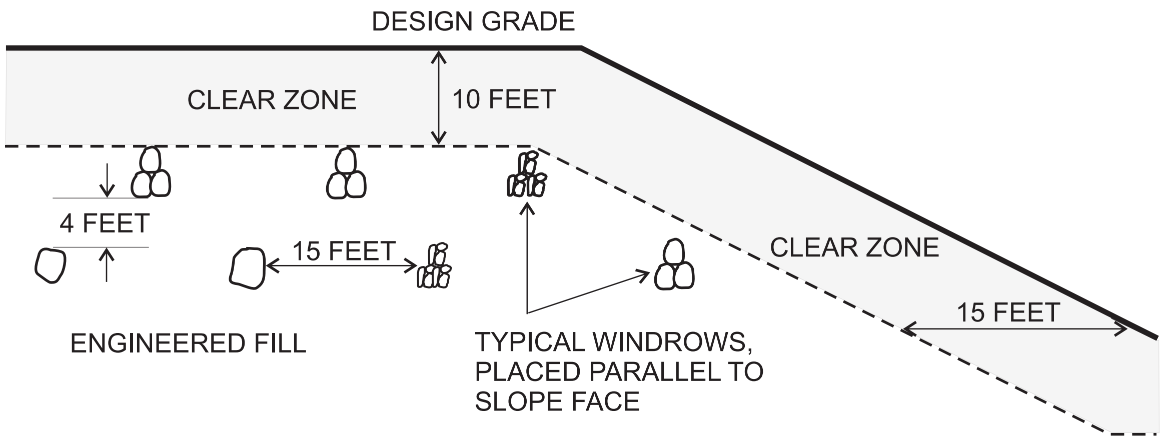
ADVANCED GEOTECHNICAL SOLUTIONS

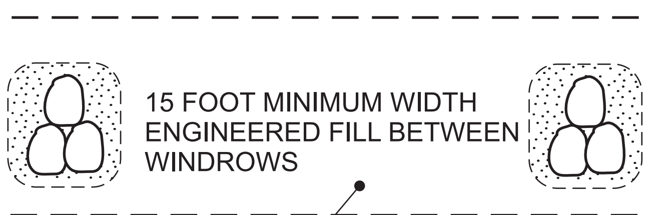
REMOVAL ADJACENT TO EXISTING FILL

DETAIL 9




CLEAR ZONE DIMENSIONS FOR REFERENCE ONLY, ACTUAL DEPTH, WIDTH, WINDROW LENGTH, ETC. TO BE BASED ON ELEVATIONS OF FOUNDATIONS, UTILITIES OR OTHER STRUCTURES PER THE GEOTECHNICAL CONSULTANT OR GOVERNING AGENCY APPROVAL

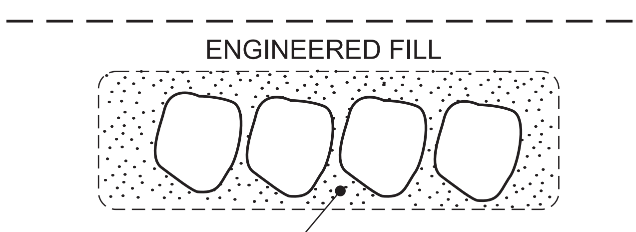
## OVERSIZED MATERIAL DISPOSAL PROFILE




HORIZONTALLY PLACED ENGINEERED FILL, FREE OF OVERSIZED MATERIALS AND COMPACTED TO MINIMUM PROJECT STANDARDS

COMPACT ENGINEERED FILL ABOVE OVERSIZED MATERIALS TO FACILITATE "TRENCH" CONDITION PRIOR TO FLOODING GRANULAR MATERIALS

## WINDROW CROSS-SECTION




GRANULAR MATERIAL APPROVED BY THE GEOTECHNICAL CONSULTANT AND CONSOLIDATED IN-PLACE BY FLOODING

## WINDROW PROFILE

VER 1.0

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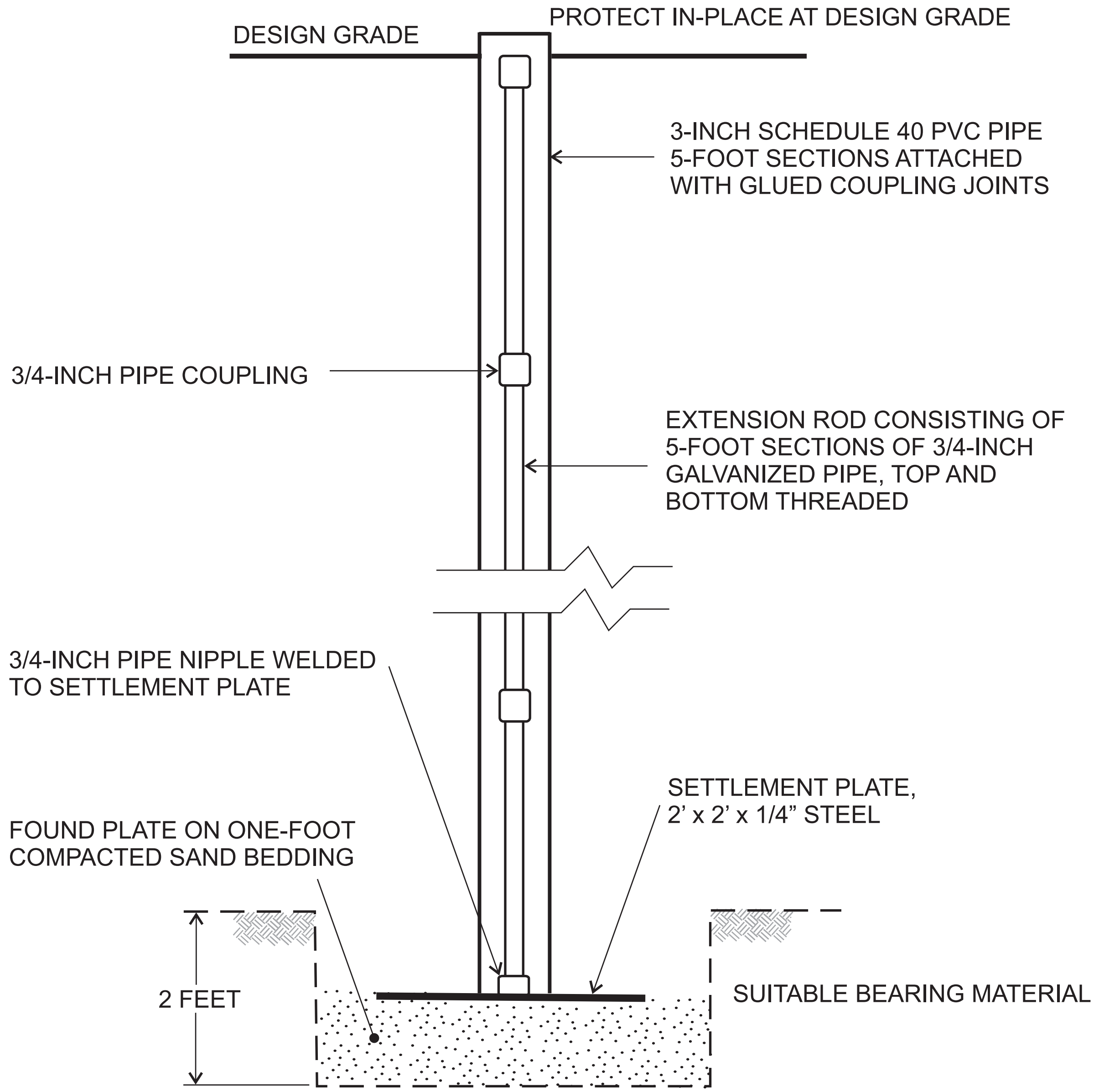
ADVANCED GEOTECHNICAL SOLUTIONS

OVERSIZED MATERIAL DISPOSAL CRITERIA

DETAIL 10




NOTES:

1. SETTLEMENT PLATE LOCATIONS SHALL BE SUFFICIENTLY IDENTIFIED BY THE CONTRACTOR AND BE READILY VISIBLE TO EQUIPMENT OPERATORS.

2. CONTRACTOR SHALL MAINTAIN ADEQUATE HORIZONTAL CLEARANCE FOR EQUIPMENT OPERATION AND SHALL BE RESPONSIBLE FOR REPAIRING ANY DAMAGE TO SETTLEMENT PLATE DURING SITE CONSTRUCTION.

3. A MINIMUM 5-FOOT ZONE ADJACENT TO SETTLEMENT PLATE/EXTENSION RODS SHALL BE ESTABLISHED FOR HAND-HELD MECHANICAL COMPACTION OF ENGINEERED FILL. ENGINEERED FILL SHALL BE COMPACTED TO MINIMUM PROJECT STANDARD.

4. ELEVATIONS OF SETTLEMENT PLATE AND ALL EXTENSION ROD PLACEMENT SHALL BE DOCUMENTED BY PROJECT CIVIL ENGINEER OR SURVEYOR.

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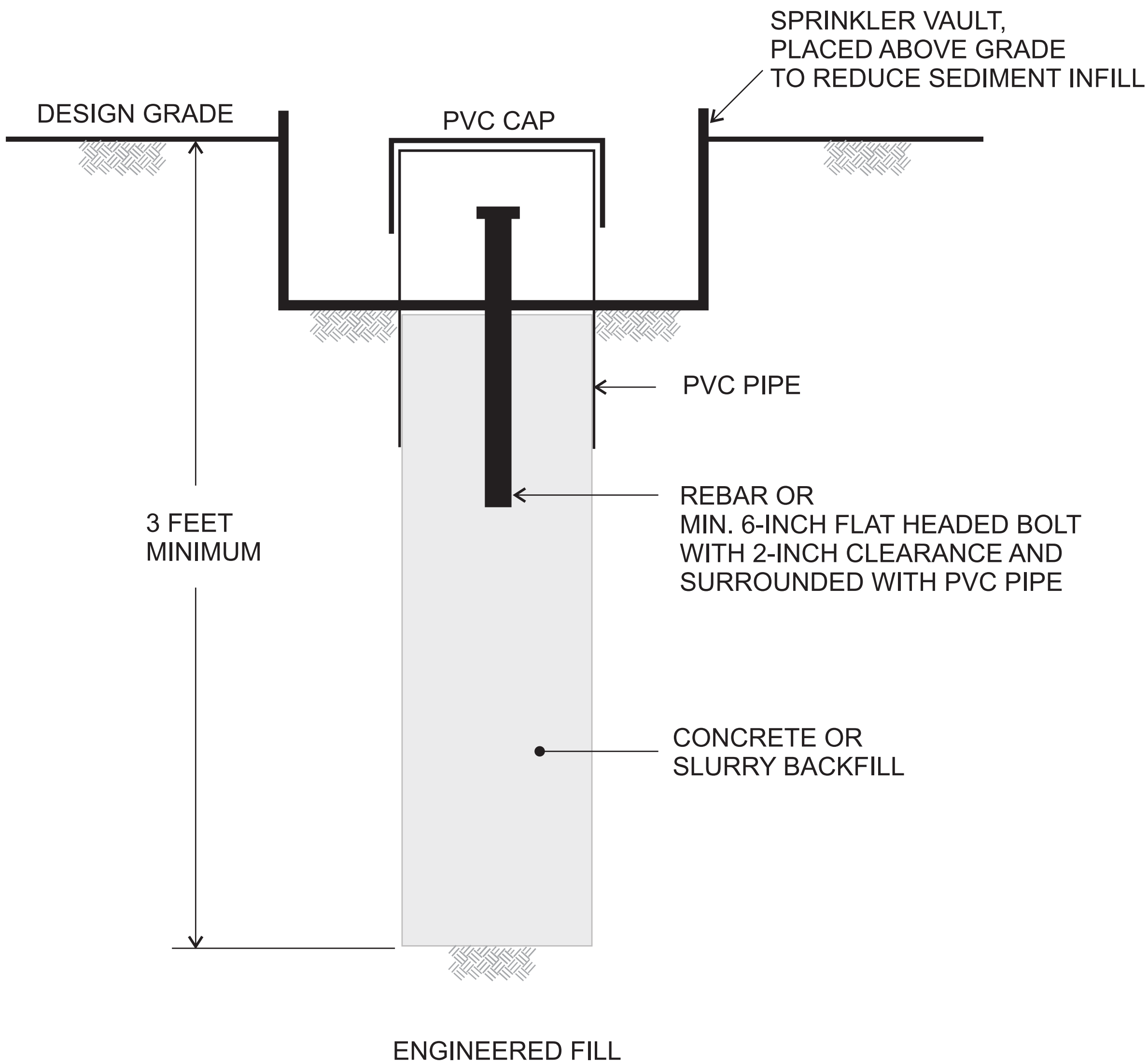
|---|---|---|
| AGS<br>ADVANCED GEOTECHNICAL SOLUTIONS | SETTLEMENT PLATE | DETAIL 11 |




NOTES:

1. SETTLEMENT MONUMENT LOCATIONS SHALL BE SUFFICIENTLY IDENTIFIED AND BE READILY VISIBLE TO EQUIPMENT OPERATORS.

2. ELEVATIONS OF SURFACE MONUMENTS SHALL BE DOCUMENTED BY PROJECT CIVIL ENGINEER OR SURVEYOR.

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|---|---|---|
| AGS<br>ADVANCED GEOTECHNICAL SOLUTIONS | SETTLEMENT MONUMENT | DETAIL 12 |

# Appendix F

## Homeowner Maintenance Recommendations

# HOMEOWNER MAINTENANCE AND IMPROVEMENT CONSIDERATIONS

Homeowners are accustomed to maintaining their homes. They expect to paint their houses periodically, replace wiring, clean out clogged plumbing, and repair roofs. Maintenance of the home site, particularly on hillsides, should be considered on the same basis or even on a more serious basis because neglect can result in serious consequences. In most cases, lot and site maintenance can be taken care of along with landscaping, and can be carried out more economically than repair after neglect.

Most slope and hillside lot problems are associated with water. Uncontrolled water from a broken pipe, cesspool, or wet weather causes most damage. Wet weather is the largest cause of slope problems, particularly in California where rain is intermittent, but may be torrential. Therefore, drainage and erosion control are the most important aspects of home site stability; these provisions must not be altered without competent professional advice. Further, maintenance must be carried out to assure their continued operation.

As geotechnical engineers concerned with the problems of building sites in hillside developments, we offer the following list of recommended home protection measures as a guide to homeowners.

## Expansive Soils

Some of the earth materials on site have been identified as being expansive in nature. As such, these materials are susceptible to volume changes with 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, patios, and other lot improvements. In recognition of this, the project developer has constructed homes on these lots on post-tensioned or mat slabs with pier and grade beam foundation systems, intended to help reduce the potential adverse effects of these expansive materials on the residential structures within the project. Such foundation systems are not intended to offset the forces (and associated movement) related to expansive soil, but are intended to help soften their effects on the structures constructed thereon.

Homeowners purchasing property and living in an area containing expansive soils must assume a certain degree of responsibility for homeowner improvements as well as 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 onsite soils material. Lot maintenance and landscaping should also be conducted in consideration of the expansive soil characteristics. Of primary importance is minimizing the moisture variation below all lot improvements. Such design, construction and homeowner maintenance provisions should include:

- Employing contractors for homeowner improvements who design and build in recognition of local building code and site specific soils conditions.
- Establishing and maintaining positive drainage away from all foundations, walkways, driveways, patios, and other hardscape improvements.
- Avoiding the construction of 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 and providing roof gutters on all structures with downspouts installed to carry roof runoff directly into area drains or discharged well away from the structures.
- ❖ Avoiding the placement of trees closer to the proposed structures than a distance of one-half the mature height of the tree.
- ❖ 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 should be cautioned against the import and use of certain 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 "moderate" or "severe" levels. In some cases, concrete improvements constructed in soils containing high levels of soluble sulfates may be affected by deterioration and loss of strength.

## Water - Natural and Man Induced

Water in concert with the reaction of various natural and man-made elements, can cause detrimental effects to your structure and surrounding property. Rain water and flowing water erodes and saturates the ground and changes the engineering characteristics of the underlying earth materials upon saturation. Excessive irrigation in concert with a rainy period is commonly associated with shallow slope failures and deep seated landslides, saturation of near structure soils, local ponding of water, and transportation of water soluble substances that are deleterious to building materials including concrete, steel, wood, and stucco.

Water interacting with the near surface and subsurface soils can initiate several other potentially detrimental phenomena other than slope stability issues. These may include expansion/contraction cycles, liquefaction potential increase, hydro-collapse of soils, ground surface settlement, earth material consolidation, and introduction of deleterious substances.

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

- ❖ Before the rainy season arrives, check and clear roof drains, gutters and down spouts of all accumulated debris. Roof gutters are an important element in your arsenal against rain damage. If you do not have roof gutters and down spouts, you may elect to install them. Roofs, with their, wide, flat area can shed tremendous quantities of water. Without gutters or other adequate drainage, water falling from the eaves collects against foundation and basement walls.
- ❖ Make sure to clear surface and terrace drainage ditches, and check them frequently during the rainy season. This task is a community responsibility.
- ❖ Test all drainage ditches for functioning outlet drains. This should be tested with a hose and done before the rainy season. All blockages should be removed.

- ❖ Check all drains at top of slopes to be sure they are clear and that water will not overflow the slope itself, causing erosion.
- ❖ Keep subsurface drain openings (weep-holes) clear of debris and other material which could block them in a storm.
- ❖ Check for loose fill above and below your property if you live on a slope or terrace.
- ❖ Monitor hoses and sprinklers. During the rainy season, little, if any, irrigation is required. Oversaturation of the ground is unnecessary, increases watering costs, and can cause subsurface drainage.
- ❖ Watch for water backup of drains inside the house and toilets during the rainy season, as this may indicate drain or sewer blockage.
- ❖ Never block terrace drains and brow ditches on slopes or at the tops of cut or fill slopes. These are designed to carry away runoff to a place where it can be safely distributed.
- ❖ Maintain the ground surface upslope of lined ditches to ensure that surface water is collected in the ditch and is not permitted to be trapped behind or under the lining.
- ❖ Do not permit water to collect or pond on your home site. Water gathering here will tend to either seep into the ground (loosening or expanding fill or natural ground), or will overflow into the slope and begin erosion. Once erosion is started, it is difficult to control and severe damage may result rather quickly.
- ❖ Never connect roof drains, gutters, or down spouts to subsurface drains. Rather, arrange them so that water either flows off your property in a specially designed pipe or flows out into a paved driveway or street. The water then may be dissipated over a wide surface or, preferably, may be carried away in a paved gutter or storm drain. Subdrains are constructed to take care of ordinary subsurface water and cannot handle the overload from roofs during a heavy rain.
- ❖ Never permit water to spill over slopes, even where this may seem to be a good way to prevent ponding. This tends to cause erosion and, in the case of fill slopes, can eat away carefully designed and constructed sites.
- ❖ Do not cast loose soil or debris over slopes. Loose soil soaks up water more readily than compacted fill. It is not compacted to the same strength as the slope itself and will tend to slide when laden with water; this may even affect the soil beneath the loose soil. The sliding may clog terrace drains below or may cause additional damage in weakening the slope. If you live below a slope, try to be sure that loose fill is not dumped above your property.
- ❖ Never discharge water into subsurface blanket drains close to slopes. Trench drains are sometimes used to get rid of excess water when other means of disposing of water are not readily available. Overloading these drains saturates the ground and, if located close to slopes, may cause slope failure in their vicinity.
- ❖ Do not discharge surface water into septic tanks or leaching fields. Not only are septic tanks constructed for a different purpose, but they will tend, because of their construction, to naturally accumulate additional water from the ground during a heavy rain. Overloading them artificially during the rainy season is bad for the same reason as subsurface subdrains, and is doubly dangerous since their overflow can pose a serious health hazard. In many areas, the use of septic tanks should be discontinued as soon as sewers are made available.
- ❖ Practice responsible irrigation practices and do not over-irrigate slopes. Naturally, ground cover of ice plant and other vegetation will require some moisture during the hot summer months, but during the wet season, irrigation can cause ice plant and other heavy ground cover to pull loose. This not only destroys the cover, but also starts serious erosion. In some areas, ice plant and other heavy cover can cause surface sloughing when saturated due to the increase in weight and weakening of the near-surface soil. Planted slopes should be planned where possible to acquire sufficient moisture when it rains.
- ❖ Do not let water gather against foundations, retaining walls, and basement walls. These walls are built to withstand the ordinary moisture in the ground and are, where necessary, accompanied by

subdrains to carry off the excess. If water is permitted to pond against them, it may seep through the wall, causing dampness and leakage inside the basement. Further, it may cause the foundation to swell up, or the water pressure could cause structural damage to walls.

- ❖ Do not try to compact soil behind walls or in trenches by flooding with water. Not only is flooding the least efficient way of compacting fine-grained soil, but it could damage the wall foundation or saturate the subsoil.
- ❖ Never leave a hose and sprinkler running on or near a slope, particularly during the rainy season. This will enhance ground saturation which may cause damage.
- ❖ Never block ditches which have been graded around your house or the lot pad. These shallow ditches have been put there for the purpose of quickly removing water toward the driveway, street or other positive outlet. By all means, do not let water become ponded above slopes by blocked ditches.
- ❖ 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 resident is responsible for proper irrigation and for maintenance and repair of properly installed irrigation systems. Leaks should be fixed immediately. Residents must undertake a program to eliminate burrowing animals. This must be an ongoing program in order to promote slope stability. The burrowing animal control program should be conducted by a licensed exterminator and/or landscape professional with expertise in hill side maintenance.

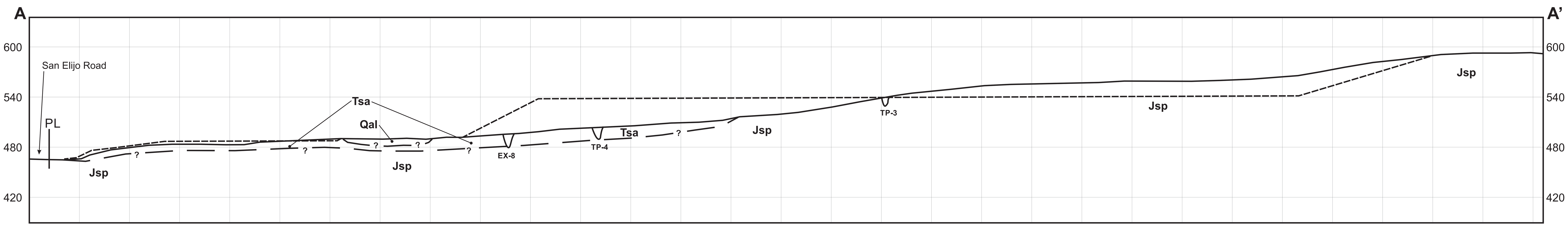
## Geotechnical Review

Due to the fact that soil types may vary with depth, it is recommended that plans for the construction of rear yard improvements (swimming pools, spas, barbecue pits, patios, etc.), be reviewed by a geotechnical engineer who is familiar with local conditions and the current standard of practice in the vicinity of your home.

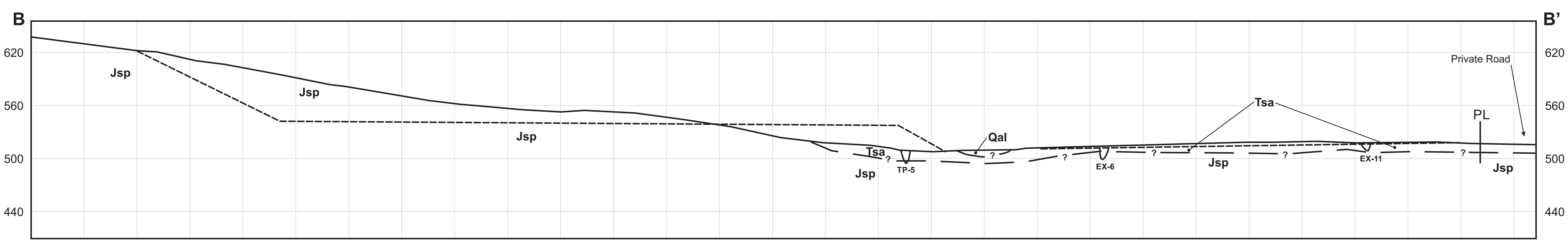
In conclusion, your neighbor’s slope, above or below your property, is as important to you as the slope that is within your property lines. For this reason, it is desirable to develop a cooperative attitude regarding hillside maintenance, and we recommend developing a “good neighbor” policy. Should conditions develop off your property, which are undesirable from indications given above, necessary action should be taken by you to insure that prompt remedial measures are taken. Landscaping of your property is important to enhance slope and foundation stability and to prevent erosion of the near surface soils. In addition, landscape improvements should provide for efficient drainage to a controlled discharge location downhill of residential improvements and soil slopes.

Additionally, recommendations contained in the Geotechnical Engineering Study report apply to all future residential site improvements, and we advise that you include consultation with a qualified professional in planning, design, and construction of any improvements. Such improvements include patios, swimming pools, decks, etc., as well as building structures and all changes in the site configuration requiring earth cut or fill construction.

SITE CONCEPT
SCALE: 1"=60'
A
A'
B
B'
NORTHWEST PARCEL
BUILDABLE PAD AREA=66,283.5 SQ. FT.
AVERAGE PAD ELEVATION=489.0
NORTHEAST PARCEL A
BUILDABLE PAD AREA=232,589.3 SQ. FT.
AVERAGE PAD ELEVATION=515.0
NORTHEAST PARCEL B
BUILDABLE PAD AREA=248,777.0 SQ. FT.
AVERAGE PAD ELEVATION=514.0
SOUTH PARCEL
BUILDABLE PAD AREA=739,461.7 SQ. FT.
AVERAGE PAD ELEVATION=538.0
150' FIRE BUFFER
EX-9 0-6' Qcol 6-9' Tsa 9-13' Jsp TD=13' No GW
EX-10 0-6' Qcol 6-9' Tsa 9-13.5' Jsp TD=13.5' No GW
EX-8 0-8' Qcol 8-14' Tsa 14-14.5' Jsp TD=14.5' No GW
T-4 Tsa 0-11' Jsp@11' TD=11.0' NW/NC
EX-11 0-3' Qcol 3-11' Tsa TD=11' No GW
TP-4 Qcol 0-4.0' Tsa 4.0-16.0' Ref.@16.0' NW/NC
EX-7 0-3' Qcol 3-11' Tsa 11-15' Jsp TD=15' No GW
EX-6 0-1.5' Qcol 1.5-3.5' Tsa 3.5-15' Jsp TD=15' No GW
EX-12 0-3' Qcol 3-14' Tsa TD=14' No GW
TP-5 Qcol 0-3.5' Tsa 3.5-12.5' Jsp 12.5-16.5' Ref.@16.5' NW/NC
T-2 Qcol 0-1' Tsa 1-12.0' Jsp @12' Ref.@12' NW/NC
TP-3 Qcol 0-3.5' Jsp 3.5-9.0' Ref.@9.0' NW/NC
T-1 Soil 0-1' Tsa 1-9.5' Jsp 9.5-10' Ref.@10' NW/NC
TP-6 afu 0-8.0' Tsa 8.0-19.0' TD=19.0' NW/NC
EX-5 0-1.5' Qcol 1.5-24.5' Tsa TD=24.5' No GW
TP-8 Qcol 0-3.0' Tsa 3.0-14.0' Ref.@14.0' NW/NC
EX-4 0-1.5' Soil 1.5-20.5' Tsa 20.5-21.5' Jsp TD=21.5' No GW
T-3 Qal 0-9' Tsa 9-12.5' TD=12.5' NW/NC
TP-7 Qcol 0-3.0' Jsp 3.0-6.5' Ref.@6.5' NW/NC
TP-2 Qcol 0-4.0' Jsp 4.0-17.5' Ref.@17.5' NW/NC
TP-1 Qcol 0-3.0' Jsp 3.0-14.0' Ref.@14.0' NW/NC
EX-2 0-3' Qal 3-9' Jsp TD=9' No GW
EX-3 0-1' Soil 1-4' Qcol 4-7' Jsp TD=7' No GW
EX-1 0-5.5' Qal 5.5-9' Tsa TD=9' No GW
SL-1
SL-2
SL-3
SL-4
S-1
S-2
S-3
S-4
S-5
B-1
B-2
B-3
B-4
B-5
B-6
Tsa (Jsp)
Qal (Tsa) ((Jsp))
afu (Tsa) ((Jsp))
Jsp
Prismoidal Volume Results
Original Surface Model: EXIST_AERIAL-TOPO
Final Surface Model: MS-FINAL
Cut Compaction Factor: 0.00 %
Fill Compaction Factor: 0.00 %
Raw Cut Volume: 345467.80 cu yd
Compacted Cut Volume: 0.00 cu yd
Total Cut Volume: 345467.80 cu yd
Raw Fill Volume: 368622.10 cu yd
Compacted Fill Volume: 0.00 cu yd
Total Fill Volume: 368622.10 cu yd
EXCEL ENGINEERING
LAND PLANNING • ENGINEERING • GIS • SURVEYING
440 STATE PLACE, ESCONDIDO, CA 92029
PH (760)745-8118 FX (760)745-1890
0 60 120 180 240
LEGEND:
EX-1 Approximate location of exploratory excavator pits (AGS, 2016)
SL-5 Approximate location of seismic refraction survey (AGS, 2016)
T-1 Approximate location of exploratory backhoe test pit (AGS, 2015)
B-1 Approximate location of infiltration test boring (AGS, 2015)
TP-1 Approximate location of exploratory test pits (AGS, 2014)
S-5 Approximate location of seismic refraction line (Geotechnics, 2003)
afu Artificial Fill - Undocumented
Qal Alluvium
Qcol Colluvium
Tsa Santiago Formation
Jsp Santiago Peak Volcanics
Approximate Location of Geologic Contact; Queried where Uncertain, Dotted where buried
PLATE 1
Geologic Map and Exploration Location Plan
AGS
ADVANCED GEOTECHNICAL SOLUTIONS, INC.
Project: P/W 1406-06
Report: 1406-06-B-6
Date: March 2016




**Cross Section A-A'**
**Scale: 1"=60'**




**Cross Section B-B'**
**Scale: 1"=60'**

**LEGEND:**

| | |
|---|---|
| | Existing Grade |
| | Proposed Grade |
| | Approximate Location of Geologic Contact; Queried where Uncertain |
| afu | Artificial Fill - Undocumented |
| Qal | Alluvium |
| Qcol | Colluvium |
| Tsa | Santiago Formation |
| Jsp | Santiago Peak Volcanics |

PLATE 2
Geologic Cross Sections

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| Project: | Report: | Date: |
|---|---|---|
| P/W 1406-06 | 1406-06-B-6 | March 2016 |