ADVANCED GEOTECHNICAL SOLUTIONS, INC.
5333 Mission Center Road, Suite 390
San Diego, California 92108
Telephone: (619) 708-1649 Fax: (714) 409-3287

Accretive Investments, Inc.
12275 El Camino Real, Suite 220
San Diego CA 92130

April 2, 2012
P/W 1102-01
Report No. 1102-01-B-9

Attention: Mr. Jon Rilling

Subject: ***EIR Level Geotechnical Review of Tentative Tract Map, Lilac Hills Ranch Community, Escondido, California***

References: See Appendix A

Gentlemen:

Pursuant to your request, presented herein are the results of Advanced Geotechnical Solutions, Inc.'s, (AGS) EIR Level Geotechnical Review of Tentative Tract Map for Lilac Hills Ranch Community, Escondido, California. AGS has been retained by Accretive Investments, Inc. to complete the geotechnical services supporting the tentative tract approval process for this project.

AGS has reviewed the referenced geotechnical documents prepared by Pacific Soils Engineering, Inc. (PSE), conducted additional field mapping, performed additional subsurface exploration and laboratory testing, performed additional engineering and geologic analysis, and reviewed the latest Tentative Tract Map for Lilac Hills Ranch. AGS accepts the content, conclusions and recommendations presented in the referenced PSE documents except where superseded herein. AGS is now assuming the role of Geotechnical Consultant of Record for Tentative Tract Map for Lilac Hills Ranch.

The purpose of this geotechnical review is to evaluate the proposed Master Tentative Tract Map conceptual grading plans relative to the near-site and on-site geologic and geotechnical conditions and provide conclusions and recommendations to aid in the development of the project. Master Tentative Tract Map proposed grading prepared by Landmark Consulting was provided to AGS for preparation of this report. These maps are included in this document with appurtenant geologic and geotechnical data superimposed upon it.

Feasibility level geotechnical studies that addressed a large portion of the present tentative tract map were conducted by PSE in 2006 and 2007 and reported on May 23, 2007. Information generated from that report along with additional data collected during recent geotechnical studies conducted by AGS form the database utilized in addressing the tentative tract map.

Advanced Geotechnical Solutions, Inc., appreciates the opportunity to provide you with geotechnical consulting services and professional opinions. If you have any questions, please contact the undersigned at (619) 708-1649.

Respectfully Submitted,
Advanced Geotechnical Solutions, Inc.

JEFFREY A. CHANEY, Vice President
GE 2314, Reg. Exp. 6-30-13

PAUL DE RISI, Vice President
CEG 2536, Reg. Exp. 5-31-13

Distribution: (6) Addressee
(1) Landmark Consulting Attn: Mark Brencik

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ATTACHMENTS:

VOLUME II

EIR Level Geotechnical Review of Tentative Tract for Lilac Hills Ranch Community, Escondido, California

1.0 INTRODUCTION

1.1. Background and Purpose

The purpose of this report is to provide a "Tentative Tract Map" (TTM) level geotechnical study that may be utilized to support the EIR submittal for the proposed Tentative Tract Map for Lilac Hills Ranch Community located in Escondido, California. This report has been prepared to address the most current TTM conceptual design prepared by Landmark Consulting in a manner consistent with County of San Diego geotechnical report guidelines and current standard of practice. Geotechnical conclusions and recommendations are presented herein and the items addressed include: 1) Unsuitable soil removals and remedial grading; 2) Cut, fill and natural slope stability; 3) Potential geologic hazards which may be onsite and general mitigation measures for these hazards; 4) Buttress/Stabilization fill requirements; 5) Cut/fill pad over excavation criteria; 6) Remedial and design grading recommendations; 7) Rippability of the onsite granitic rock; 8) Disposal of oversize hard earth materials; and 9) General foundation design recommendations based upon anticipated as graded soil conditions.

1.2. Scope of Study

This study is aimed at providing geotechnical/geologic conclusions and recommendations for development of TTM for residential and commercial uses, attendant streets, parks, schools, community facilities, trash transfer station, sewerage treatment facility and open space areas.

The scope of this study included the following tasks:

- Review of pertinent published and unpublished geologic and geotechnical literature, maps, and aerial photographs readily available to this firm (Appendix A).
- Review and compile previous subsurface data from PSE (2007) including 43 backhoe test pits, 21 air track borings, and 15 seismic refraction traverses.
- Transfer selected geologic and geotechnical information generated from this and previous investigations onto the Master Tentative Tract Map grading prepared by Landmark Consulting, included as Sheets 1 thru 3, included herewith. These plans depict existing grades and proposed sheet grading. AGS has added geologic and geotechnical information including: the approximate limits of surface geologic units; locations of air-track borings, test pits (backhoe and excavator) with abbreviated logs, and shallow seismic refraction survey traverses.
- Coordinate field studies on the various parcels with the land owners and underground utility location identifiers.
- Perform confirmatory geologic mapping on previously studied areas and conduct additional geologic mapping on newly accessible parcels within the proposed tentative tract map boundaries.
- Excavate, sample, and log 28 backhoe test pits T-1 thru T-28 (Appendix B).

- Excavate and record drilling rates for 19 air-hammer borings AT-1 thru AT-19 utilizing an ECM 370 tracked drill rig (Appendix B).
- Conduct a shallow seismic refraction survey study with our sub-consultant Southwest Geophysics, Inc. at selected locations. This study consisted of eight (8) traverses (SLB-1 thru SLB-8) approximately 240 feet long. These studies also were evaluated utilizing tomographic modeling methods to further define the relative velocities within the bedrock and model the potential corestones which are commonly found within weathered granitic rock (Appendix B).
- Laboratory testing of bulk samples obtained during this study (Appendix C).
- Prepare geologic/geotechnical cross-sections A-A' thru E-E' as shown on Sheets 4 and 5.
- Conduct a geotechnical engineering and geologic hazard analysis of the site.
- Conduct a limited seismicity analysis.
- Define remedial grading requirements.
- Slope stability analysis of both the highest cut and fill slopes (Appendix D).
- Data analyses in relation to the site specific proposed improvements.
- Analysis of the excavation characteristics (i.e. rippability) of onsite bedrock materials.
- Discussion of pertinent geologic and geotechnical topics.
- Prepare general foundation design parameters which can be used for preliminary design.
- Prepare this geotechnical tentative tract map review report with exhibits summarizing our findings. This report is suitable for design support and regulatory review.

1.3. Geotechnical Study Limitations

The conclusions and recommendations in this report are professional opinions based on the data developed during this and previous investigations. The conclusions presented herein are based upon the current design as reflected on the included Tentative Tract Map. Changes to the plan would necessitate further review.

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.

2.0 SITE LOCATION AND DESCRIPTION

2.1. Site Location

The Lilac Hills Ranch Community is located in northern unincorporated San Diego County, ¼-mile east of the Interstate 15 corridor on with freeway access off the Old Highway 395 Interchange (Figure 1 - Site Location Map). The project site is located to the south and west of West Lilac Road with State Route 76 to the north, downtown Valley Center 10 miles to the east, downtown Escondido 16 miles to the south, and Interstate 15 and Old Highway 395 to the west.

The Lilac Hills Ranch Community project is located entirely in the Escondido zip code (92026) and occurs primarily within the westernmost portion of the Valley Center Community Planning Area (CPA) although a small portion is within the Bonsall Sub-regional Plan Area (Figure 2). From the northwest project corner, West Lilac Road serves as the northern and eastern boundary of the project site, while Circle R Drive is less than a 1/2 mile south of the project boundary. From the southwest project corner, the western boundary of the project runs along Shirey Road and extends to Standel Lane, which serves as the northwestern project boundary. The project is within Township 10 South, Range 3 West, Section 24, and Township 10 South, Range 2 West, Sections 19 and 30, on the USGS 7.5' Pala and Bonsall quadrangles.

2.2. Site Description

The irregularly shaped project consists of an assemblage of individual parcels with a total acreage of approximately 611 acres. Roughly half of the project area presently supports agricultural operations including citrus, avocado, assorted fruit trees, flowers and nursery plants. The remainder of the project is in a natural state and is covered with a light to moderate growth of annuals and some chaparral. Numerous structures are scattered throughout the 611-acre site, primarily consisting of residential structures of varying sizes and composition along with some agricultural structures (barns, packing houses, and storage structures). A network of improved and unimproved roads provides access throughout the site. Several of the parcels are fenced and as a result ingress and egress to theses parcels are provided by the main roads and secondary driveways. Several waterlines are present onsite and are part of the Valley Center Municipal Water District.

In general, the site can be described as rolling hills that vary from gentle to moderate slopes. Elevations within the project limits range from 650 MSL to 950 MSL, rendering a total relief of approximately 300 feet. The northern portions of the site drain towards the south via a primary drainage located along the west side of the project. Several smaller drainage areas are fed by sheet flow that then drain into the primary drainage on the west side of the project ultimately collecting in Moosa Canyon, and eventually draining into San Luis Rey River.

2.3. Proposed Development

The Lilac Ranch Hills project will be a new mixed use master planned community that will be multi-phased. It is anticipated that conventional cut and fill grading techniques will be utilized to develop the project.

The current grading depicted on the tentative tract map reflects sheet graded pads which will support a variety of uses including but not limited to : multi-family; single-family; live/work; retail; commercial; school; YMCA; church; municipal; park sites; trash transfer station; sewer treatment facility; and open space areas.

Both cut and fill slopes are designed at a slope ratio of 2:1 (horizontal: vertical) or flatter. The highest proposed cut slope is approximately 70 feet at a slope ratio of 2:1. The highest proposed fill slope is approximately 70 feet. The maximum depths of cut and fill to achieve design grade is approximately 50 to 60 feet. However, fills may approach 60 to 70 feet after unsuitable soil removals have been accomplished.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1. Previous Geotechnical Investigations

PSE conducted and reported a geotechnical investigation addressing a significant portion of the site in early 2007 (PSE, 2007). At that project planning stage various parcels were in process of acquisition and access to some parcels was not possible. For that study PSE had excavated and logged twenty-one (21) air-hammer borings utilizing an ECM 590 drill rig; excavated, logged and sampled forty-three (43) backhoe pits with a John Deere 310G backhoe; and had fifteen (15) shallow seismic refraction traverses performed by Southwest Geophysics, Inc. (SG). Appendix B presents the logs from the backhoe test pits, drilling rates for the air-hammer borings, seismic traverse profiles and geophysical survey report by SG. Appendix C presents laboratory test results.

3.2. Current Investigation

For this Tentative Tract Map level investigation AGS has performed additional geologic mapping, conducted additional subsurface exploration and laboratory testing, as well as reviewed and utilized the results of the subsurface investigation conducted by PSE in preparing this study. AGS excavated, logged, and sampled 28 backhoe test pits (T-1 thru T-28) utilizing a Case 580M Extend-a-hoe w/24" bucket. 19 air- hammer borings (AT-1 thru AT-19) were excavated with an ECM 370 track mounted drill rig utilizing a 4-inch diameter bit with the drilling rates plotted verse depth. Eight shallow seismic refraction traverses (SLB-1 thru SLB-8) were performed by Southwest Geophysics, Inc. to evaluate the hardness of the bedrock. All of this data is presented herein in Appendix B. Selected bulk samples obtained from the backhoe test pits were transported to our approved laboratory for testing and analysis, with results of that testing presented in Appendix C

As part of our services AGS has integrated appurtenant information from this and previous investigations on the Mater Tentative Tract Map grading prepared by Landmark Consulting (Sheets 1 thru 3), prepared cross-sections A-A’ through E-E’ (Sheets 4 and 5) and prepared this report with our findings and recommendations.

4.0 ENGINEERING GEOLOGY

4.1. Geologic Analysis

4.1.1. Literature Review

AGS has reviewed the referenced geologic documents in preparing this study. Where deemed appropriate, this information has been included with this document. Of particular use are the maps by Kennedy (2000), Tan (2000), and PSE (2007).

4.1.2. Aerial Photograph Review

AGS has re-visited the aerial photographs reviewed during previous studies and has taken advantage of recent web content aerial photographs. No features in addition to those identified by previous studies were noted.

4.1.3. Field Mapping

The geologic contacts mapped by PSE were either verified or modified by AGS during this investigation based upon additional surface and subsurface information obtained.

4.2. Geologic and Geomorphic Setting

The Lilac Hills Ranch Community is located in the lower Peninsular Range Region of San Diego County, a subset of the greater Peninsular Ranges Geomorphic Province of California. This portion of the Peninsular Ranges is underlain by the intrusive southern California Batholith. Approximately two (2) miles northwest of the project lay the major drainage of the area, the San Luis Rey River, meandering to empty into the Pacific Ocean in Oceanside. Agua Tibia Mountain lies north of the river.

This portion of San Diego County is made up of foothills that span elevations from 600 to 2000 feet above mean sea level (MSL). It is characterized by rolling and hilly uplands that contain frequent narrow and winding valleys. The Lilac Hills Ranch Community project is in the lower rolling hills area.

The rolling hills are predominately composed of Tonalite of the Couser Canyon geologic formation with a minor amount of the Granodiorite of Indian Mountain exposed at the northern boundary of the project (Kennedy, 2000; Tan, 2000). Tonalite is an igneous, plutonic (intrusive) rock, of felsic composition, with phaneritic texture and a granodiorite is an intrusive igneous rock similar to granite, but containing more plagioclase than orthoclase-type feldspar. These two bedrock types will be referred to with the more common term “granite” throughout this document. These igneous rocks are deeply (five to forty feet) weathered within the proposed Lilac Hills Ranch Community. A regional geology map is shown on Figure 3.

4.3. Stratigraphy

The geologic units underlying the project are characterized by weathered and decomposed granitic rocks with a very minor amount of exposed outcrops of hard granitic boulder corestones. A relatively thin veneer of surficial units including undocumented artificial fill, topsoil, alluvium and older alluvium cap the granitic rocks. The enclosed geologic maps (Sheets 1 through 3) show the presently mapped location of the units. A brief description of the units is described below:

4.3.1. Surficial Units

Surficial units onsite include undocumented artificial fill (afu), Topsoil (unmapped), Alluvial Deposits (map symbol Qal), and Older Alluvium (map symbol Qoal). Detailed descriptions of these units are presented below.

4.3.1.1. Artificial Fill, Undocumented (afu)

Undocumented artificial fills are located throughout the Lilac Hills Ranch Community associated with past and present land use including residential construction, farming operations, private roadway construction, local water retention embankments, utility construction, and pad areas, among other minor land uses. The mapped locations of the most prominent fills are shown on the accompanying plates however; due to the map scale numerous lesser fills are present but unmapped. Future studies may determine documentation regarding the engineering of fills and how present site development plans would impact the function of these fills.

The vast majorities of the fills are locally derived and consist of light reddish brown, clayey and silty sands that are commonly dry to slightly moist and loose to moderately dense.

4.3.1.2. Topsoil (no map symbol)

Surficial weathering over the majority of the site has resulted in a thin veneer of topsoil throughout the project. The topsoil is composed of medium brown to reddish brown clayey to silty sands that are dry to slightly moist and loose to moderately dense.

4.3.1.3. Alluvium (Qal)

Alluvial deposits occupy the canyon areas and active drainage courses throughout the project and the mapped locations are shown on Sheets 1 and 2. The Holocene-aged alluvium varies from a light orange brown to light to medium brown silty and clayey sand to sandy silt that is damp to locally wet, loose and soft to moderately dense and firm. The thickness of the alluvium logged in the borings and trenches reached maximum depths of 13 to 14 feet and are likely deeper in unexplored areas such as portions of the dominant drainage on the southwest portion of the project.

4.3.1.4. Older Alluvium (Qoal)

Early Holocene to Pleistocene Older Alluvium has been mapped onsite and in areas is evident as a distinct geomorphic surface. It has also been observed in some areas below the younger alluvial deposits where it was not removed by erosion by the two distinct depositional episodes. The Older Alluvium has distinctly well-developed reddish to orange-brown color due to its age and exposure to weathering elements since its deposition. Composed of silty to clayey sands that are moderately hard to hard and slightly moist to moist, the moderately oxidized earth material is well consolidated.

4.3.2. Bedrock Units

4.3.2.1. "Granitic Rocks" (Kgr)

Identified and discussed as "granite" in this document, the Tonalite of Couser Canyon is a "granitic-type" rock that underlies the entire Tentative Tract with a small exception of some Granodiorite of Indian Mountain mapped by Kennedy (2000) and Tan (2000) along the northern boundary of the project. In most areas this unit is deeply weathered and hard boulder corestones were observed at ground surface in only a few areas area. These outcrops are shown on Sheets 1 and 2.

4.4. <u>Geologic Structure and Tectonic Setting</u>

4.4.1. Regional Faulting

The San Andreas fault zone is the dominant and controlling tectonic stress regime of southern California (Figure 4). As the boundary between the Pacific and North American structural plates, this northwest trending right lateral, strike–slip, active fault has controlled the crustal structural regimes of southern California since Miocene time. Numerous related active fault zones with a regular spacing, including the Elsinore-Whittier-Chino, Newport-Inglewood-Rose Canyon, and San Jacinto fault zones characterize the stress regime and also trend to the northwest as do the Santa Ana Mountains and the Peninsular Ranges.

The Temecula section (Wildomar Fault) of the Elsinore fault zone is closest to the project and is located 7.8 miles to the northeast. The next closest fault zone to the site is the Oceanside section of the Newport-Rose Canyon fault zone at approximately 20 miles to the southwest. The Anza section of the San Jacinto fault zone is approximately 32 miles to the northeast and the San Bernardino section of the San Andreas fault zone is about 55 miles to the northeast.

4.4.2. Local Faulting

Alquist-Priolo County Special Studies Fault Zones and San Diego County Fault Zones are not located onsite (Figure 4). The most influential geologic faults potentially affecting the property are the active and potentially active Williard, Wildomar, Wolf Valley and Temecula segments of the Elsinore Fault System. No faults have been mapped onsite on published geologic maps and none were observed during this and previous geologic studies.

4.4.3. Geologic Structure

Dominant foliations, fracture patterns or other structural features common to granitic rocks were not mapped or observed during this or previous studies. Geologic maps by Kennedy and Tan are also void of any such mapped features. The highly weathered nature of the granitic rock apparently has contributed significantly to this lack of observable features. Dike patterns offsite indicate a northwest trend that is typical of rocks in the Peninsular Ranges province.

4.5. Groundwater

Shallow groundwater was not observed during this or previous studies. Localized springs and seeps were observed within the active lager drainages. For the most part these areas will not be developed as they are considered to be wetlands. A few small man made ponds were observed and all appear to be related to agricultural needs and or the construction of embankment fills to collect water within the drainages.

4.6. Non-seismic Geologic Hazards

4.6.1. Mass Wasting and Debris Flows

The majority of the site is sloping to the southwest at shallow to moderate slope ratios and is capped by a relatively thin veneer of surficial earth material underlain by granitic rocks and is considered not susceptible to mass wasting. No evidence of past landsliding or debris flows has been mapped within the limits of the project. Since there is no steep terrain offsite or onsite, the potential for debris flows emanating from the mouths of the up-gradient drainages are considered to be remote.

4.6.2. Rock Fall

The potential for rock fall is considered to be very low given the lack of rock outcrops within the proposed limits of the development.

4.6.3. Flooding

The site is not located within a County of San Diego Flood Plain Zone. Hydrology studies should be provided by the Civil Engineer.

4.6.4. Subsidence and Ground Fissuring

Owing to the very shallow granitic bedrock underlying the site, subsidence and ground fissuring potential at the site is considered nil.

4.7. Seismic Hazards

The site is located in the tectonically active Southern California area, and will therefore likely experience shaking effects from earthquakes. The Near Source Shaking Zones of the County of San Diego (Figure 5) shows the distance of the site from near source shaking zones. 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, the direction of propagation of the seismic wave and the underlying soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction, seismically induced slope failure 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. The following seismic hazards discussion is guided by the California Building Code (2010), CDMG (2008), and Martin and Lew (1998).

4.7.1. Surface Fault Rupture

Surface rupture is a break in the ground surface during or as a consequence of seismic activity. To a large part, research supports the conclusion that active faults tend to rupture at or near pre-existing fault planes. No faults much less active faults have been mapped within or near the project. As such, it is appropriate to conclude that the potential for surface fault rupture is very low.

4.7.2. Ground Motions

As noted, the site is within the tectonically active southern California area, with segments of the Elsinore Fault system within 8 miles of the site. The potential exists for strong ground motion that may affect future improvements. As part of this assessment, AGS utilized the California Geologic Survey Probabilistic Seismic Hazards Seismic Hazards Mapping Ground Motion Page. A site location with latitude of 33.2905˚N and longitude -117.1333˚W was utilized. Ground motions (10% probability of being exceeded in 50 years) are expressed as a fraction of the acceleration due to gravity (g). Three values of ground motion are shown, peak ground acceleration (Pga), spectral acceleration (Sa) at short (0.2 second) and moderately long (1.0 second) periods. Ground motion values are also modified by the local site soil conditions. Ground motion values are shown for two different site conditions: granitic rock (site category B) and Stiff soil (Older Alluvium and artificial fill) (site category D).

TABLE 5.7.2
SELECTED GROUND MOTIONS*

	Rock	Stiff Soil
Pga (g)	0.349	0.395
Sa 0.2 sec	0.835	0.951
Sa 1.0 sec.	0.314	0.479

*NEHRP Soil Corrections were used to calculate Soft Rock and Alluvium. Ground Motion values were interpolated from a grid (0.05 degree spacing) of calculated values. Interpolated ground motion may not equal values calculated for a specific site, therefore these values are not intended for design or analysis.

At this point in time, non-critical structures (commercial, residential, and industrial) are usually designed according to the 2010 California Building Code and that of the controlling local agency. However, liquefaction/seismic slope stability analyses, critical structures, water tanks and unusual structural designs will likely require site specific ground motion input.

4.7.3. Liquefaction

Liquefaction is the phenomenon in which the buildup of excess pore pressures, in saturated granular soils due to seismic agitation, results in a temporary “quick” or “liquefied” condition. The site is not within an area zoned by the County of San Diego as a Potential Liquefaction Area (Figure 6). After remedial grading, saturated alluvium will be entirely removed within the projects development footprint. The remedial grading as recommended herein will render the potential for liquefaction to be nil.

4.7.4. Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. Due to the anticipated removals proposed herein the potential for lateral spreading is considered to be very low.

4.7.5. Seismically Induced Dynamic Settlement

Seismically induced dynamic settlement occurs in response to seismic shaking of loose sandy earth materials. The source of settlement is volumetric strain associated with liquefaction of saturated soils strata, and/or, the rearrangement of sandy particles in dry, relatively loose layers of sandy soils (cohesionless). These two sources of settlement potential are mutually exclusive, as such, if the groundwater rises, the liquefaction potential and its adverse effects increase, while dry sand settlement potential decreases, and vice-versa.

Due to the anticipated removals proposed herein, the density and cementation of older alluvium to be left in-place and the hardness of the underlying granitic rock, the potential for seismically induced settlement is considered nil.

4.7.6. Seismically Induced Landsliding

Seismically induced landsliding is considered to be very low for, engineered fill slopes. For cut slopes excavated in the granitic rock, or on the remaining shallow natural slopes the potential for seismically induced landsliding is considered to be very low.

4.7.7. Earthquake Induced Flooding

Earthquake induced flooding can be caused by tsunamis, dam failures, or seiches. Also, earthquakes can cause landslides that dam rivers and streams, and flooding can occur upstream above the dam and also downstream when these dams are breached. A seiche is a free or standing-wave oscillation on the surface of water in an enclosed or semi-enclosed basin. The wave can be initiated by an earthquake and can vary in height from several centimeters to a few meters. Due to the lack of a freestanding body of water nearby, the potential for a seiche impacting the site is considered to be non-existent.

Considering the lack of any dams or permanent water sources upstream, earthquake induced flooding caused by a dam failure is considered to be non-existent.

Considering the distance of the site from the coastline, the potential for flooding due to tsunamis is nil.

5.0 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.

5.1. Material Properties

5.1.1. Excavation Characteristics

Based on our previous experience with similar projects near the subject site and review of the information gathered during this and previous investigations, it is AGS's opinion that the shallow and surficial earth materials above the weathered and un-weathered granitic rock onsite can be readily excavated with conventional grading equipment however deeper cuts within the granitic rock potentially require moderate to heavy ripping in the weathered portions and potentially heavy ripping to blasting of throughout much of the un-weathered granitics.

AGS performed a preliminary rippability evaluation for the subject site supplementing the previous study. This evaluation included analyses of data from 19 air-hammer borings (AT-1 through AT-19, Appendix B) and eight seismic refraction survey lines (SLB-1 through SLB-8, Appendix B) conducted under AGS's direction. Additionally, data from 21 air-hammer borings (AT-101 through AT-121, Appendix B) and nineteen (19) seismic refraction survey lines (SL-1 through SL-19, Appendix B) reported by PSE (2007) were utilized in the evaluation. This rippability evaluation is also based upon the performance capabilities of a Caterpillar D9N bulldozer and our experience with similar projects in the region.

In general, the ease of rock rippability depends upon factors such as the rock type, rock hardness and density, the amount of weathering, and the existence and characteristics of discontinuities such as joint spacing, foliation, or random fractures. For example, a rock mass that is weathered and exhibits well-developed discontinuities, such as joints, will be easier to excavate than a compositionally similar rock mass that lacks discontinuities and significant weathering. This is because weathering typically decreases cohesive rock strength, and discontinuities typically provide a mechanism that allows the rock mass to readily part upon stress (Hoek and Bray, 1981).

For the subject site, the main controls on rippability are joints, fractures and foliations, the degree of weathering at depth, and the depth and size of the cut areas. Based upon our seismic traverses, the bedrock generally shows a weathered halo that ranges from approximately 5 to greater than 5 feet in depth below the surface.

In general, it has been AGS's experience that drilling rates for ECM 590 with a 4.5 inch diameter bit above 10 seconds per foot indicates marginally rippable. For the ECM 370 with a 4-inch bit drilling rates of 17 seconds per foot indicates marginally rippable. Once rates reach 12 seconds per foot for the ECM 590 and 20 seconds per foot for the ECM 370 blasting will be likely be required for efficient excavation of the granitic rock.

Detailed interpretation of the seismic information is presented in the report by Southwest Geophysics, Inc. (Appendix B). It is AGS's experience, that when velocities are higher than 5,000 to 5,500 feet/sec., blasting potentially will be required for efficient excavation utilizing a D-9 bulldozer equipped with a single-shank ripper. Although it is possible that in certain instances velocities approaching 6,500 feet/sec. can be ripped, production rate would be such that drilling and shooting is typically preferred in order to increase production. Velocities less than 5,000 to 5,500 feet/sec. may potentially require localized blasting and will probably contain common boulders that will require special handling and this can also result in slow production rates.

Additionally, numerous other factors can affect the necessity to use blasting, including: 1) considerations of overburden; 2) fracture spacing and pattern; 3) the experience of the equipment operator; 4) the equipment type; 5) the size and depth of the cuts; and 6) cost/contractual issues. Based upon our preliminary evaluation, slopes, hills and ridges underlain by the granitic rock will generally be difficult to excavate but rippable (with local heavy ripping) to approximately five to 50+ feet below ground surface, however, some of these areas will potentially require blasting from the surface for dislodgement.

For the Lilac Ranch Hills Community, the granitic bedrock unit appears to have a highly weathered profile whose thickness is highly variable, ranging from about five (5) to fifty plus (50+) feet. Below that horizon, boulder corestones become more common and the materials surrounding the boulders becomes harder such that the conditions during excavation are expected to be difficult and will probably require blasting for efficient excavation. Blasting techniques may require an overburden of material to be left in place in order to control the blast debris and size of material produced. Therefore, some areas that are rippable may be left in place in order to provide adequate overburden for effective blasting. A grading and blasting logistics program should be developed for the subject site. Techniques for potential blasting of hard-rock at the site should be evaluated by a blasting specialist during the grading plan review stage of the project.

5.1.2. Oversized Material

Oversized rock (> 24 inches) will be generated in the deeper cuts and over excavations within the granitic bedrock. This rock may be incorporated into the compacted fill section to within ten (10) feet of finish grade or within two (2) feet of the deepest utility (if utility is greater than ten (10) feet). Oversize rock is not to be placed within areas of proposed drainage structures and should be kept minimally five (5) feet outside and below proposed culverts, pipes, etc.

Maximum rock size between three (3) feet and ten (10) feet of finished grade is restricted to 24 inches and in the upper three (3) feet from finish grade is restricted to a maximum rock size of eight (8) inches. Variances to the above rock hold-down must be approved by the owner, geotechnical consultant and governing agencies.

5.1.3. Compressibility

The onsite materials that are compressible include undocumented artificial fills, alluvium, weathered older alluvium, and weathered bedrock. Highly compressible materials will require removal from fill areas prior to placement of fill and where exposed at grade in cut areas.

5.1.4. Collapse Potential/Hydro-Consolidation

The hydro-consolidation process is a singular response to the introduction of water into collapse-prone alluvial soils. Upon initial wetting, the soil structure and apparent strength are altered and a virtually immediate settlement response occurs. Recommended measures to mitigate potential for differential settlement due to hydro-collapse include removal/recompaction and/or foundation design, such as described in Sections 6.1 and 7.1 of this report.

5.1.5. Expansion Potential

Based upon the sampling and associated laboratory testing conducted by AGS and PSE the near surface soils are considered to exhibit “Very Low” to “Moderately” expansive potential ($0 \leq EI \leq 90$), with the majority of the onsite soils falling into the “Very Low “to “Low” expansion potential. Typical mitigation measures for expansive soils include: structural design, pre-saturation and overexcavation where the higher expansion characteristics are present.

5.1.6. Shear Strength

Shear strength testing was conducted by AGS and by PSE on remolded samples that were collected during this and past studies onsite (see Appendix C). Within the onsite bedrock units, the in-situ shear strength and fracture patterns are the most significant factors in cut slope and natural slope stability. Typically, the granitic rock possesses considerable shear strength and can stand unsupported at relatively steep slope ratios. The "older alluvium" generally possesses good in-situ shear strength except where weathered such as the upper five feet. Alluvium generally can be characterized as possessing fair to poor strength characteristics. The shear strength of the fill soils created during grading generally will exhibit good shear strength for fill slopes and for support of structures. The shear strengths recommended by AGS for design are presented in Table 5.1.6.

TABLE 5.1.6
RECOMMENDED SHEAR STRENGTHS FOR DESIGN

Material	Cohesion (psf)	Friction Angle (degrees)	Density (pcf)
Artificial Fill Compacted (afc) & Older Alluvium (Qoal)	150	35	125
Granitic Bedrock (Kgr)	500	40	140

5.1.7. Chemical and Resistivity Test Results

The test results from AGS’s and PSE’s previous investigation in the general area indicate that sulfate concentrations for the onsite soils will be below 0.1 percent, which corresponds to a “very low” sulfate exposure when classified in accordance with ACI 318-05 Table 4.3.1 (per 2010 CBC). Testing should be conducted during and upon completion of grading operations to further evaluate the sulfate content and potential corrosivity on the onsite soils.

5.1.8. Earthwork Adjustments

The following average earthwork adjustment factors are presented for use in evaluating earthwork quantities. These numbers are considered approximate and should be refined during grading when actual conditions are better defined. Contingencies should be made to adjust the earthwork balance during grading if these numbers are adjusted.

TABLE 5.1.8
EARTHWORK ADJUSTMENTS

Geologic Unit	Approximate Range
Artificial Fill Undocumented (Afu)	8% to 12% Shrink
Topsoil & Alluvium (Qal)	8% to 12% Shrink
Older Alluvium (Qoal)	0% to 5% Bulk
Granitic Bedrock (Kgr) - rippable	10% to 18% Bulk
Granitic Bedrock (Kgr) - non-rippable	18% to 25% Bulk

5.1.9. Pavement Support Characteristics

Compacted fill derived from onsite soils and cuts within the older alluvium and granitic rock is expected to possess good to very good pavement support characteristics. Testing should be completed once subgrade elevations are reached for the onsite roadways. For preliminary planning purposes, AGS has used an R-Value of 40 for the preliminary design of roadway pavement sections.

5.2. Analytical Methods

5.2.1. Slope Stability Analysis

Stability analyses were performed for both static and seismic (pseudo-static) conditions using the GSTABL7 computer program. The Modified Bishop method was used to analyze circular type failures. The critical failure surface determined in the static analysis was used in the pseudo-static analysis. A horizontal destabilizing seismic coefficient (kh) of 0.15g was selected for the site and used in the pseudo-static analyses. Peak shear strengths have been utilized in the pseudo-static analysis.

Surficial stability analyses were conducted using an infinite height slope method assuming seepage parallel to the slope surface.

5.2.2. Pavement Design

Asphalt concrete pavement sections have been designed using the recommendations and methods presented in the Caltrans Highway Design Manual. Portland cement concrete pavement for onsite roads and driveways has been designed in accordance with the recommendations presented in the "Design of Concrete Pavement for City Streets" by the American Concrete Pavement Association.

5.2.3. Bearing Capacity and Lateral Pressure

Ultimate bearing capacity values were obtained using the graphs and formula presented in NAVFAC DM-7.1. Allowable bearing was determined by applying a factor of safety of at least 3 to the ultimate bearing capacity. Static lateral earth pressures were calculated using Rankine methods for active and passive cases.

6.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Based on the information presented herein and our experience in the vicinity of the subject site, it is AGS's opinion that the proposed development of Lilac Ranch Hills Community is feasible, from a geotechnical point of view, provided that the constraints discussed in this report are addressed in the design and construction of each proposed residential structure. Presented below are issues identified by this study or previous studies as possibly impacting site development. Recommendations to mitigate these issues and geotechnical recommendations for use in planning and design are presented in the following sections of this report.

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).

6.1. Site Preparation and Removals/Overexcavation

Guidelines to determine the depth of removals are presented below; however, the exact extent of the removals must be determined in the field during grading, when observation and evaluation of the greater detail afforded by those exposures can be performed by the Geotechnical Consultant. In general, removed soils will be suitable for reuse as compacted fill when free of deleterious materials and after moisture conditioning.

Removal of unsuitable soils typically should be established at a 1:1 projection to suitable materials outside the proposed engineered fills. Front cuts should be made no steeper than 1:1, except where constrained by other factors such as property lines and protected structures. Removals should be initiated at approximately twice the distance of the anticipated removal depth, outside the engineered fills. The bottoms of all removal areas should be observed, mapped, and approved by the Geotechnical Consultant prior to fill placement. It is recommended the bottoms of removals be surveyed and documented.

6.1.1. 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.

6.1.2. **Topsoil** (no map symbol)

All topsoil should be removed before placement of compacted fill.

6.1.3. **Artificial Fill Undocumented** (map symbol afu)

All undocumented fill material in designed fill areas and/or where exposed in cuts should be removed. Undocumented fill removals are anticipated to range in depth from two to fifteen, with possibly deeper localized areas. It is anticipated that these materials will be suitable for re-use provided that all deleterious materials (brush, roots, ect.) is removed prior to incorporation into fill.

6.1.4. **Alluvium** (map symbol Qal)

All alluvium should be removed within a 1:1 projection of the designed fill and cut areas. Alluvium removals are anticipated to range from a few feet to as deep as twenty feet, with possibly deeper localized areas.

6.1.5. **Older Alluvium** (map symbol Qoal)

The upper three to four feet of older alluvium should be removed within a 1:1 projection of the designed fill areas and cut areas.

6.1.6. **Granitic Rock** (map symbol Kgr)

The upper one to three feet of highly weathered granitic rock should be removed within a 1:1 projection of the designed fill and cut areas.

6.1.7. **Overexcavation**

6.1.7.1. Cut Lot Overexcavation

Cut lots exposing older alluvium and granitic rock should be overexcavated such that a minimum of three feet of compacted fill is placed below the building pad and deeper overexcavation may be considered for structures planned with deeper footings, swimming pools, etc. The undercut overexcavation should maintain a minimum one (1) percent gradient to the front of the lot. In addition, where steep cut/fill transitions are created, additional overexcavation and flattening of the transitions may be required.

6.1.7.2. Cut/Fill Transition Lot Overexcavation

Where design or remedial grading activities create a cut/fill transition on the "structural" lots excavation of the cut or shallow fill portion should be performed such that at least three (3) feet of compacted fill exits over the pad. The undercut overexcavation should maintain a minimum one (1) percent gradient to the front of the lot. In addition, where steep cut/fill transitions are created, additional overexcavation and flattening of the transitions may be recommended.

6.1.7.3. Street Overexcavation

Streets that are cut into older alluvium and granitic rock could potentially pose excavation difficulties during utility and street installation. The granitic rock may potentially require heavy ripping and/or blasting in deeper cut areas in order to get to utility excavation depth. During mass grading, where such materials are exposed, consideration should be given to undercutting the street/utility areas during mass grading to minimize this condition. The undercut should extend at least one foot below the deepest utility. The undercut zone should be replaced with compacted fill in accordance with project standards as outlined herein.

6.1.8. Removals Along Grading Limits and Property Lines

Removals of unsuitable soils will be required prior to fill placement along the project grading limits. A 1:1 projection, from toe of slope or grading limit, outward to competent materials should be established, when possible.

6.2. Slope Stability and Remediation

Proposed maximum slope heights to be created during grading are on the order of 70 feet or less.

6.2.1. Cut Slopes

The highest proposed cut slope is approximately 70 feet at a slope ratio of 2:1 (horizontal: vertical). Based upon the currently available information, we anticipate that proposed cut slopes in Older Alluvium and Granitic Rock will be grossly stable as designed. Calculations supporting AGS's conclusions and recommendations relative to cut slopes are represented in Appendix D (Plates D-1 and D-2).

Cut slopes should be observed by the Geotechnical Consultant during grading. Where cut slopes expose unfavorable geology such as daylighted joints, loose or raveling weathered granitic rock or where boulders may pose a rock fall problem, replacement of the unsuitable portions of the cut with stabilization fill will be recommended.

Terrace and downdrains should be constructed on all cuts slopes in conformance to the San Diego County Grading Ordinance.

6.2.2. Fill Slopes

Fill slopes on the project are designed at 2:1 ratios (horizontal to vertical). The highest anticipated fill slope is approximately 70 feet high. Fill slopes, when properly constructed with onsite materials, are expected to be grossly stable as designed. Stability calculations supporting this conclusion are presented in Appendix D (Plates D-4 and D-5). Fill slopes will be subject to surficial erosion and should be landscaped as quickly as possible.

Keys should be constructed at the toe of all fill slopes "toeing" on existing or cut grade. Fill keys should have a minimum width equal to one-half the height of ascending slope, and not less than 15 feet. Unsuitable soil removals below the toe of proposed fill slopes should extend from the catch point of the design toe outward at a minimum 1:1 projection into approved material to establish the location of the key. Backcuts to establish that removal geometry should be cut no steeper than 1:1 or as recommended by the Geotechnical Consultant.

Terrace and downdrains should be constructed on all cuts slopes in conformance to the San Diego County Grading Ordinance.

6.2.3. Skin Cut and Skin Fill Slopes

A review of the Tract Map did not indicate any significant design skin fill and skin cut conditions, however, skin cut or thin fill sections may be created during grading. For all such conditions, it is recommended that a backcut and keyway be established such that a minimum fill thickness equal to one-half the remaining slope height, and not less than 15 feet, is provided. Where the design cut is insufficient to remove all unsuitable materials, overexcavation and replacement with a stabilization fill will be required, as shown on Grading Detail 6 in Appendix E.

6.2.4. Fill Over Cut Slopes

Fill over cut slopes should be constructed such that the cut portion is excavated first for geologic mapping and stability determination. If deemed stable then a “tilt-back” keyway half the remaining slope height or minimally twenty (20) feet wide should be established. Drains will be required for this condition with the locations determined based upon exposed field conditions.

6.2.5. Surficial Stability

The surficial stability of 2:1 fill and cut slopes, constructed in accordance with the recommendations presented herein, have been analyzed, and the analyses presented in Appendix D (Plates D-3 and D-6, respectively) indicates factors-of-safety in excess of code minimums. When fill and cut slopes are properly constructed and maintained, satisfactory performance can be anticipated although slopes will be subject to erosion, particularly before landscaping is fully established.

6.2.6. Temporary Backcut Stability

During grading operations, temporary backcuts may occur due to grading logistics and during retaining wall construction. Backcuts should be made no steeper than 1:1 (horizontal to vertical) to heights of up to 20 feet, and 1½:1 (horizontal: vertical) for heights greater than 20 feet. Flatter backcuts may be necessary where geologic conditions dictate, and where minimum width dimensions are to be maintained.

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

6.2.7. Observation During Grading

All temporary slope excavations, including front, side and backcuts, and all cut slopes should be mapped to verify the geologic conditions that were modeled prior to grading.

6.3. Survey Control During Grading

Removal bottoms fill keys, stabilization fill keys, and backdrains should be surveyed prior to final observation and approval by the geotechnical engineer/engineering geologist in order to verify locations and gradients.

6.4. Subsurface Drainage

Canyon subdrains should be constructed within the major drainages which will ultimately be filled as part of the mass grading of the site. Canyon subdrains will range in diameter from 6 to 8 inches in diameter and should be constructed in accordance with Grading Detail 1 and 2, Appendix E. Final determination as to the location and the size of these subdrain systems will be dependent upon the final finished design grades. Accordingly, once more detailed plans become available site specific recommendations will be prepared regarding the size, location and extant of the subdrain system for the project.

Due to the lack of a significant backcuts and the anticipated depth of fill in the toe areas after remedial grading, the need for backdrain systems are not anticipated at the toes of constructed fill slopes or fill over cut slopes. This should be further evaluated during future grading plan reviews and during grading. Backdrains, where required, should be constructed in accordance with Grading Detail 2.

Drains should be installed behind all retaining walls.

6.5. 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.

6.6. Earthwork Considerations

6.6.1. Compaction Standards

All fills should be compacted at least 90 percent of the maximum dry density as determined by ASTM D1557-09. All loose and or deleterious soils should be removed to expose firm native soils or bedrock. 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 D1557-09). Fill should be placed in thin (6 to 8-inch) lifts, moisture conditioned to optimum moisture or slightly above, and compacted to 90 percent of the maximum dry density (ASTM D1557-09) until the desired grade is achieved.

6.6.2. Benching

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

6.6.3. 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.

6.6.4. Haul Roads

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

6.6.5. Import Soils

The project is proposed to balance on site. If this changes the Geotechnical Consultant should be contacted.

6.6.6. Rock Excavation Considerations and Potential Grading Impacts

The impacts of grading and potential blasting with regard to dust control, noise, etc. is generally under the purview of others and the conditions of the regulating agency. Potential impacts to the surrounding community environment during grading, blasting and rock crushing should be evaluated by licensed, experienced grading and blasting contractors. The grading, blasting and rock crushing operations should be coordinated by the contractors to minimize the impact of the grading operation on the surrounding community environment and improvements. The grading and blasting contractors should follow the guidelines and permit conditions provided by the regulating agency.

6.6.7. Oversize Rock

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 two (2) 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 property owners 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

6.6.7.1. Rock Blankets

Rock blankets consisting of a mixture of fines, sand, gravel, and rock to a maximum dimension of 2 feet may be constructed. The construction of rock fill shall be continuously observed by the geotechnical consultant. The rocks should be placed on a 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, is without significant voids, and forms a dense, compact fill matrix. Adequate water shall be provided continuously during these operations.

Rock blankets may be extended to the slope face provided the following additional conditions are met: 1) no rocks greater than 12 inches in diameter are allowed within 6 horizontal feet of the slope face; 2) 50 percent of the material is to be three-quarters (3/4) inch minus by volume; and 3) back-rolling or track walking of the slope face is conducted at 4-foot verticals to meet project compaction specifications.

6.6.7.2. Rock Windrows

Rocks to maximum dimension of 4 feet may be placed in windrows in deeper soil fill areas in accordance with Grading Detail 10. The construction of rock fill shall be continuously observed by the geotechnical consultant. 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 by at least 15 feet of compacted fill, be staggered vertically and separated by at least 4 vertical feet of compacted fill. Windrows should not be placed within 10 feet of finish grade within structural fill areas,–within 2 vertical feet of the lowest buried utility conduit in structural fills, or within 15 feet of the finish slope surface unless specifically approved by the owner, geotechnical consultant and governing agency.

6.6.7.3. 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.

6.6.7.4. *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.

6.6.8. Fill Slope Construction

Fill slopes may be constructed by preferably overbuilding and cutting back to the compacted core or by back-rolling and compacting the slope face. The following recommendations should be incorporated into construction of the proposed fill slopes.

Care should be taken to avoid spillage of loose materials down the face of any slopes during grading. Spill fill will require complete removal before compaction, shaping and grid rolling.

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.

6.6.8.1. *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.

6.6.8.2. *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.

6.6.9. Utility Trench Excavation and Backfill

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. The geotechnical consultant should be consulted on these issues during construction.

Mainline and lateral utility trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557-09. 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.

To reduce moisture penetration beneath the slab-on-grade areas, shallow utility trenches should be backfilled with lean concrete or concrete slurry where they intercept the foundation perimeter, or such excavations can be backfilled with native soils, moisture-conditioned to over optimum, and compacted to a minimum of 90 percent relative compaction.

7.0 DESIGN RECOMMENDATIONS

From a geotechnical perspective, the proposed development is feasible provided the following recommendations are incorporated into the design and construction. Preliminary design recommendations are presented herein and are based on some of the general soils conditions encountered during the recent investigation and described in the referenced geotechnical investigations. As such, recommendations provided herein are considered preliminary and subject to change based on the results of additional observation and testing that will occur during grading operations. Final design recommendations should be provided in a final rough/precise grading report.

7.1. Structural Design Recommendations

It's our understanding that the site will be graded and lots will be ultimately sold to merchant builders; thus precise building products, loading conditions, and locations are not currently available. It is expected that for typical one to three story residential/commercial products and loading conditions (1 ksf to 4 ksf for spread and continuous footings), conventional shallow slab-on-grade foundations will be utilized in areas with low expansive and shallow fill areas (<50 feet). Post-tensioned slab/foundations may also be used for the residential lots. Typically post-tensioned slab/foundations will be used for lots which exhibit expansion potentials ranging from "moderate" to "very high" and for lots in areas where the fill depth exceeds fifty (50) feet.

Upon the completion of rough grading, finish grade samples should be collected and tested to develop specific recommendations as they relate to final foundation design recommendations for individual lots. These test results and corresponding design recommendations should be presented in a Final Rough Grading Report.

7.1.1. Foundation Design

Residential/Commercial structures can be supported on conventional shallow foundations and slab-on-grade or post-tensioned slab/foundation systems, as discussed above. The design of foundation systems should be based on as-graded conditions as determined after grading completion. The following values may be used in preliminary foundation design:

Allowable Bearing: 2000 psf.

Lateral Bearing: 250 psf. per foot of depth to a maximum of 2000 psf. for level conditions. Reduced values may be appropriate for descending slope conditions.

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 and should be evaluated by a qualified engineer.

7.1.1.1. Post Tensioned Foundations

Preliminary geotechnical engineering design and construction parameters for post-tensioned slab foundations are as follows:

- Post-tensioned slabs should incorporate a perimeter-thickened edge to reduce the potential for moisture infiltration, seasonal moisture fluctuation and associated differential movement around the slab perimeter. The minimum depth of the thickened edge could vary from 12-inches for “low” expansion to 24-inches for “high” expansion potential.
- Design and construction of the post-tensioned foundations should be undertaken by firms experienced in the field. It is the responsibility of the foundation design engineer to select the design methodology and properly design the foundation system for the onsite soils conditions. The slab designer should provide deflection potential to the project architect/structural engineer for incorporation into the design of the structure.
- The project foundation design engineer should use the Post-Tensioning Institute (PTI) foundation design procedures as described in UBC, based upon appropriate soil design parameters relating to edge moisture variation and differential swell provided by the geotechnical consultant at the completion of rough grading operations.
- A vapor/moisture barrier is recommended below all moisture sensitive areas.

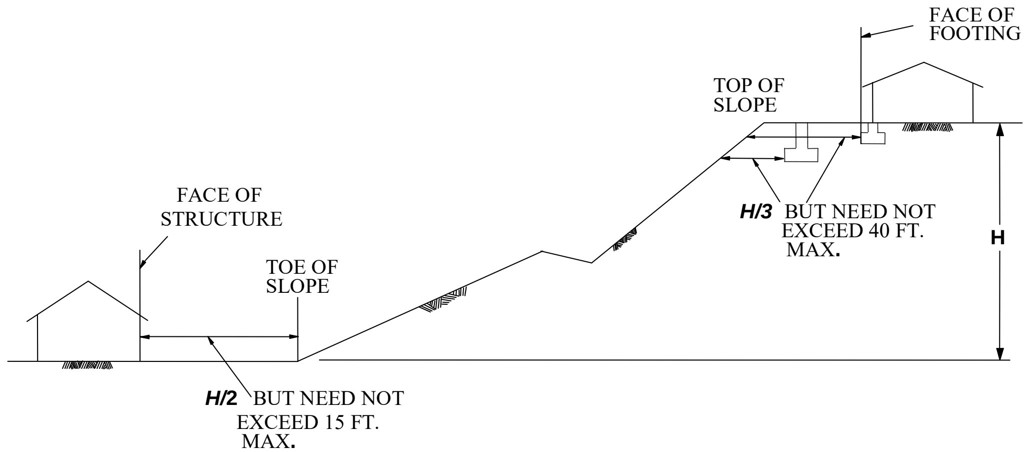
7.1.1.2. Deepened Footings and Setbacks

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, require that structures be set back or footings deepened where subject to the influence of these natural processes.

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

FIGURE 7.1.1.2

Setback Dimensions (CBC, 2010)



7.1.1.3. Moisture and Vapor Barrier

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.

7.1.2. Retaining Wall Design

The foundations for retaining walls of appurtenant structures structurally separated from the building structure may bear on properly compacted fill. The foundations may be designed in accordance with the recommendations provided in Table 7.1.2, Conventional Foundation Design Parameters. When calculating the lateral resistance, the upper 12 inches of soil cover should be ignored in areas that are not covered with hardscape. 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.

Retaining walls should be designed to resist earth pressures presented in the following table. These values assume that the retaining walls will be backfilled with select materials as shown in Detail RTW-A or native soils as shown in Detail RTW-B. The type of backfill ("select" or "native") should be specified by the wall designer and shown on the plans. Retaining walls should be designed to resist additional loads such as construction loads, temporary loads, and other surcharges as evaluated by the structural engineer.

TABLE 7.1.2
RETAINING WALL EARTH PRESSURES

"Native"* Backfill Materials (γ=125pcf, EI$\leq$50)

	Level Backfill		Sloping (2:1) Backfill	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	$K_a = 0.33$	42	$K_a = 0.54$	67
Passive Pressure	$K_p = 3.00$	375	$K_p = 1.12$	140
At Rest Pressure	$K_o = 0.50$	63	$K_o = 0.81$	101

"Select"* Backfill Materials (γ=120pcf, EI$\leq$20, SE$\geq$20)

	Level Backfill		Sloping (2:1) Backfill	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	$K_a = 0.28$	34	$K_a = 0.44$	53
Passive Pressure	$K_p = 3.54$	420	$K_p = 1.33$	160
At Rest Pressure	$K_o = 0.44$	53	$K_o = 0.75$	90

Notes: "Select" backfill materials should be granular, structural quality backfill with a Sand Equivalent of 20 or better and an Expansion Index of 20 or less. The "select" backfill must extend at least one-half the wall height behind the wall; otherwise, the values presented in the "Native" backfill materials columns must be used for the design. "Native" backfill materials should have an Expansion Index of 50 or less. The upper one-foot of backfill should be comprised of native on-site soils.

In addition to the above static pressures, unrestrained retaining walls located should be designed to resist seismic loading as required by the 2010 CBC. 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:

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

Where:

- Pe = Seismic thrust load
- H = Height of the wall (feet)
- γ = soil density = 125 pounds per cubic foot (pcf)
- k_h = seismic pseudostatic coefficient = 0.5 * peak horizontal ground acceleration / g

The peak horizontal ground accelerations are provided in Section 5.7.2. Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces as shown in Details RTW-A and RTW-B in Appendix E. Otherwise, the retaining walls should be designed to resist hydrostatic forces. Proper drainage devices should be installed along the top of the wall backfill and 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 of the maximum dry density as determined by ASTM D1557-09. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. No backfill should be placed against concrete until minimum design strengths are achieved as verified by compression tests of cylinders. The geotechnical consultant should observe the retaining wall footings, back drain installation, and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

7.1.3. Seismic Design

In general, the site has been identified to be a “D” site class in accordance with Table 1613.5.2 of the 2010 CBC. Utilizing this information, the computer program Seismic Hazard Curves, Response Parameters and Design Parameters, v5.1.0, provided by the United States Geological Survey, and 2005 ASCE 7 criterion, the seismic design category for 0.20 second (Ss) and 1.0 second (S1) period response accelerations have been determined (2010 CBC, Section 1613.5.1) along with the design spectral response accelerations (2010 CBC, Sections 1613.5.3 and 1613.5.4). Results are presented in Table 7.1.3.

TABLE 7.1.3
SEISMIC DESIGN PARAMETERS

Site Class	SD_S (g)	SD_I (g)	SM_S (g)	SM_I (g)
A (Hard Rock)	0.744	0.286	1.117	0.429
B (Rock)	0.930	0.358	1.396	0.536
D (Artificial Fill)	0.930	0.536	1.396	0.804

7.2. Civil Design Recommendations

7.2.1. Rear and Side Yard Walls and Fences

Block wall footings should be founded a minimum of 24-inches below the lowest adjacent grade. To reduce the potential for uncontrolled, unsightly cracks, it is recommended that a construction joint be incorporated at regular intervals. Spacing of the joints should be between 10 and 20 feet. Side yard walls should be structurally.

7.2.2. Drainage

Final site grading should assure positive drainage away from structures. Planter areas should be provided with area drains to transmit irrigation and rain water away from structures. The use of gutters and down spouts to carry roof drainage well away from structures is recommended. Raised planters should be provided with a positive means to remove water through the face of the containment wall.

7.2.3. Pavement Design

Final pavement design should be made based upon sampling and testing of post-grading conditions. For preliminary design and estimating purposes the pavement structural sections presented in Table 7.2.3 can be used for the range of likely traffic indices. The structural sections are based upon an assumed R - Value of 40.

TABLE 7.2.3
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	3	4
6.0	3	5
7.0	4	8
8.0	4	9.5

Pavement subgrade soils should be at or near optimum moisture content and should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557-09. Aggregate base should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557-09 and should conform with the specifications listed in Section 26 of the *Standard Specifications for the State of California Department of Transportation* (Caltrans) or Section 200-2 of the *Standard Specifications for Public Works Construction* (Green Book). The asphalt concrete should conform to Section 26 of the Caltrans *Standard Specifications* or Section 203-6 of the Green Book.

7.2.4. Water Quality Basins/ Drainage

All water should be diverted along a relatively impervious channel away from the top of the slope as to not impact the stability of the slope nor erode the slope face.

8.0 FUTURE STUDY NEEDS

This report represents an EIR level Tentative Tract Map review of the Lilac Hills Ranch Community project. As the project design progresses, additional site specific geologic and geotechnical issues will need to be considered in the ultimate design and construction of the project. Consequently, future geotechnical reviews are necessary. These reviews may include reviews of:

- Rough grading plans.
- Precise grading plans.
- Foundation plans.
- Retaining wall plans.

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

9.0 CLOSURE

9.1. 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.

AGS should review the pertinent plans and sections of the project specifications, to evaluate conformance with the intent of the recommendations contained in this report.

If the project description or final design varies from that described in this report, AGS must be consulted regarding the applicability of, and the necessity for, any revisions to the recommendations presented herein. AGS accepts no liability for any use of its recommendations if the project description or final design varies and AGS is not consulted regarding the changes.

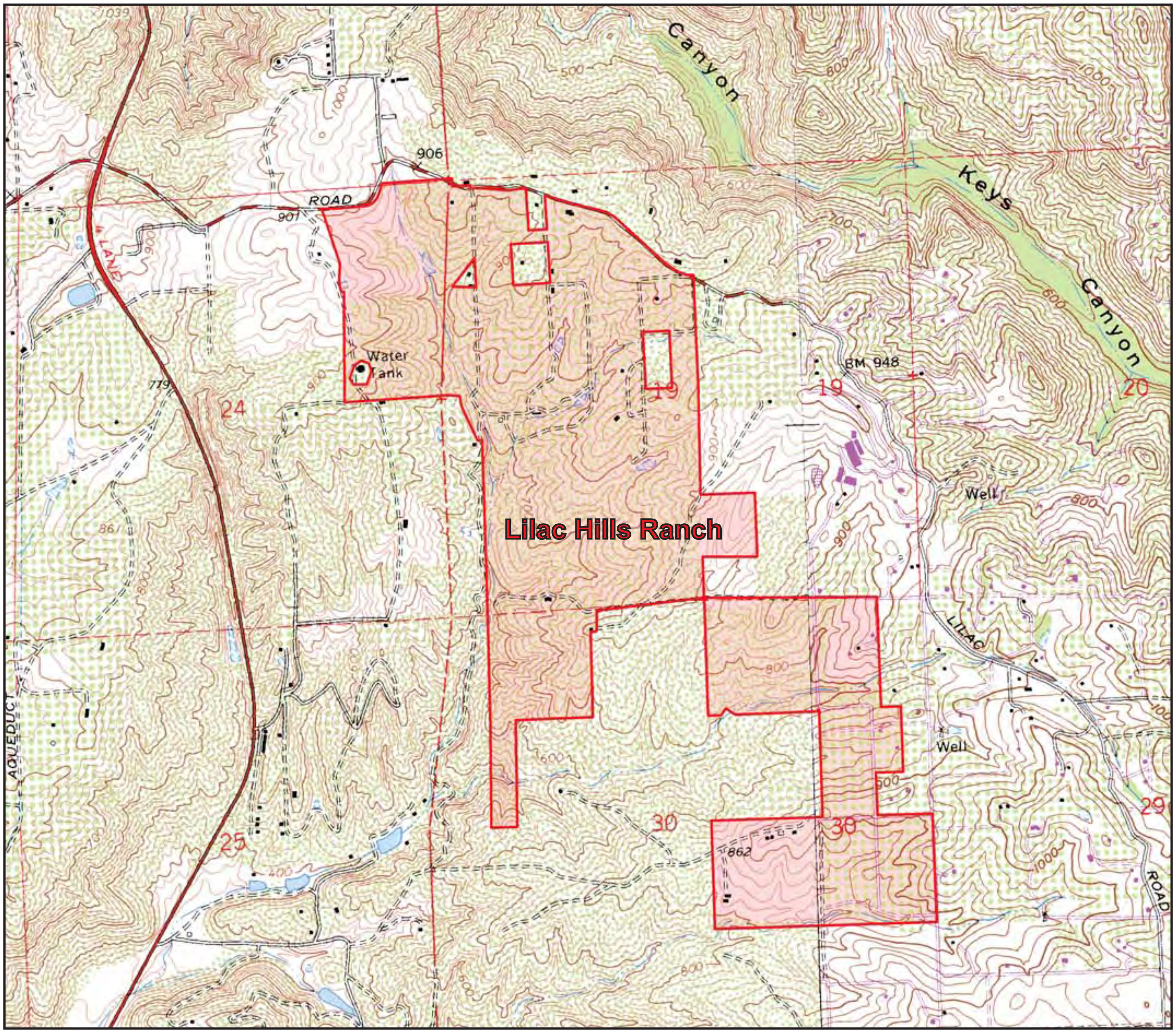
9.2. Limitations

This report is based on the project as described and the information obtained from referenced reports and the borings at the locations indicated on the plan. The findings are based on the review of the field and laboratory data combined with an interpolation and extrapolation of conditions between and beyond the exploratory excavations. 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.



N

SCALE: 1 in. = 2000 ft.

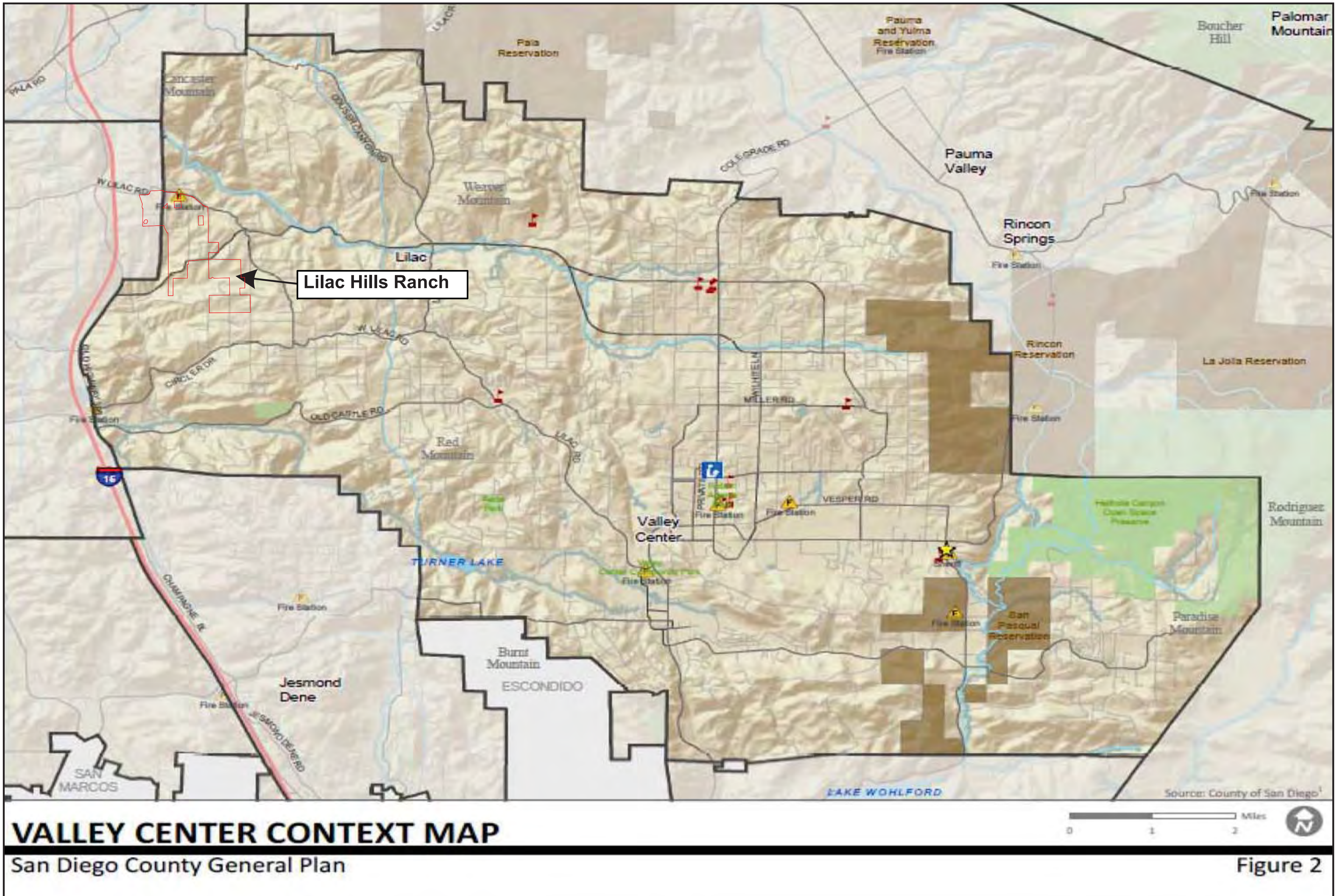
U.S.G.S. SITE LOCATION MAP
LILAC HILLS RANCH DEVELOPMENT
NORTHWEST CORNER VALLEY CENTER
COUNTY OF SAN DIEGO, CALIFORNIA

Latitude: 33.2905° N
Longitude: -117.1333° W

FIGURE 1

SOURCE MAPS - PALA AND BONSALL U.S.G.S.
7.5-MINUTE QUADRANGLES

AGS
ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Ste 220, Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287
P/W 1102-01 **Report No. 1102-01-B-9**



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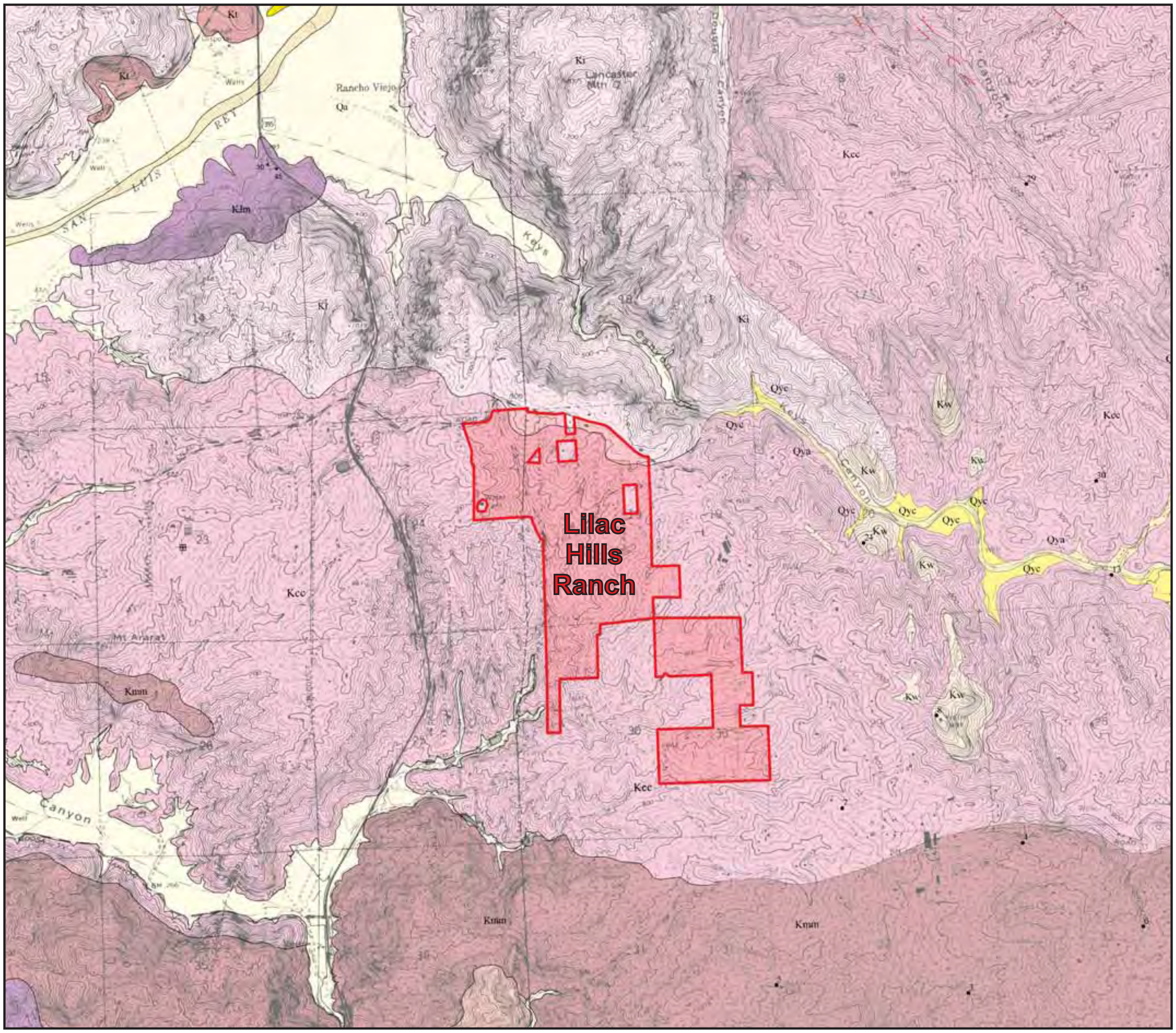
Valley Center Context Map

SOURCE MAP - Valley Center Community Plan, San Diego County General Plan, August 2011

FIGURE 2

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Ste 220, Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

P/W 1102-01 Report No. 1102-01-B-9



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SCALE: 1 in. = 4000 ft.

REGIONAL GEOLOGY MAP
LILAC HILLS RANCH DEVELOPMENT
NORTHWEST CORNER VALLEY CENTER
COUNTY OF SAN DIEGO, CALIFORNIA

FIGURE 3

SOURCE MAPS - GEOLOGY OF THE PALA AND BONSALL 7.5 MINUTE QUADRANGLES

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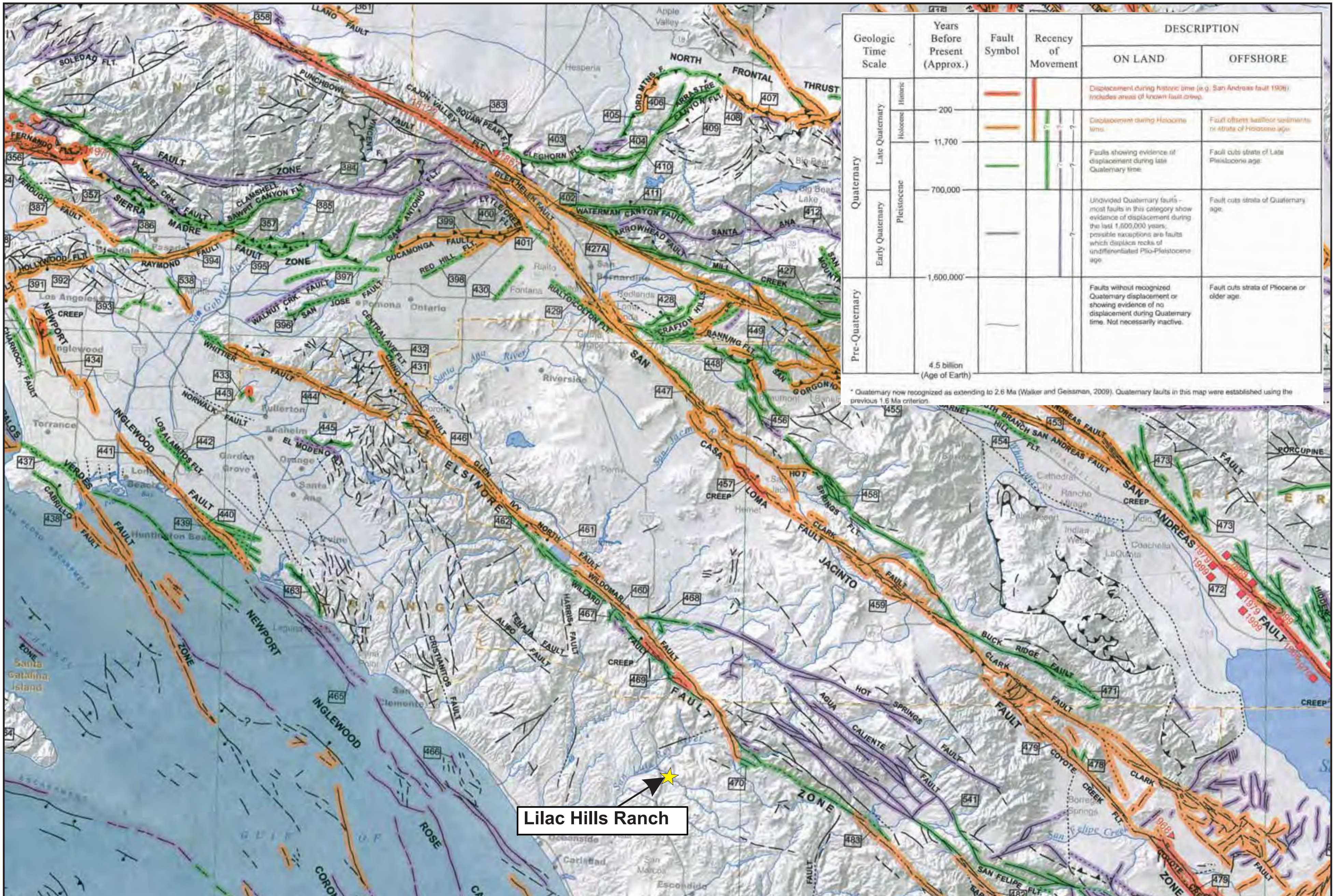
Telephone: (619) 708-1649 Fax: (714) 409-3287
P/W 1102-01 **Report No. 1102-01-B-9**



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary					Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000*				
Pre-Quaternary						Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
			4.5 billion (Age of Earth)				

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

N

SCALE: 1:750,000

FAULT ACTIVITY MAP OF CALIFORNIA

SOURCE MAP - 2010, California Geological Survey, 150th Anniversary Fault Activity Map of California

FIGURE 4

AGS
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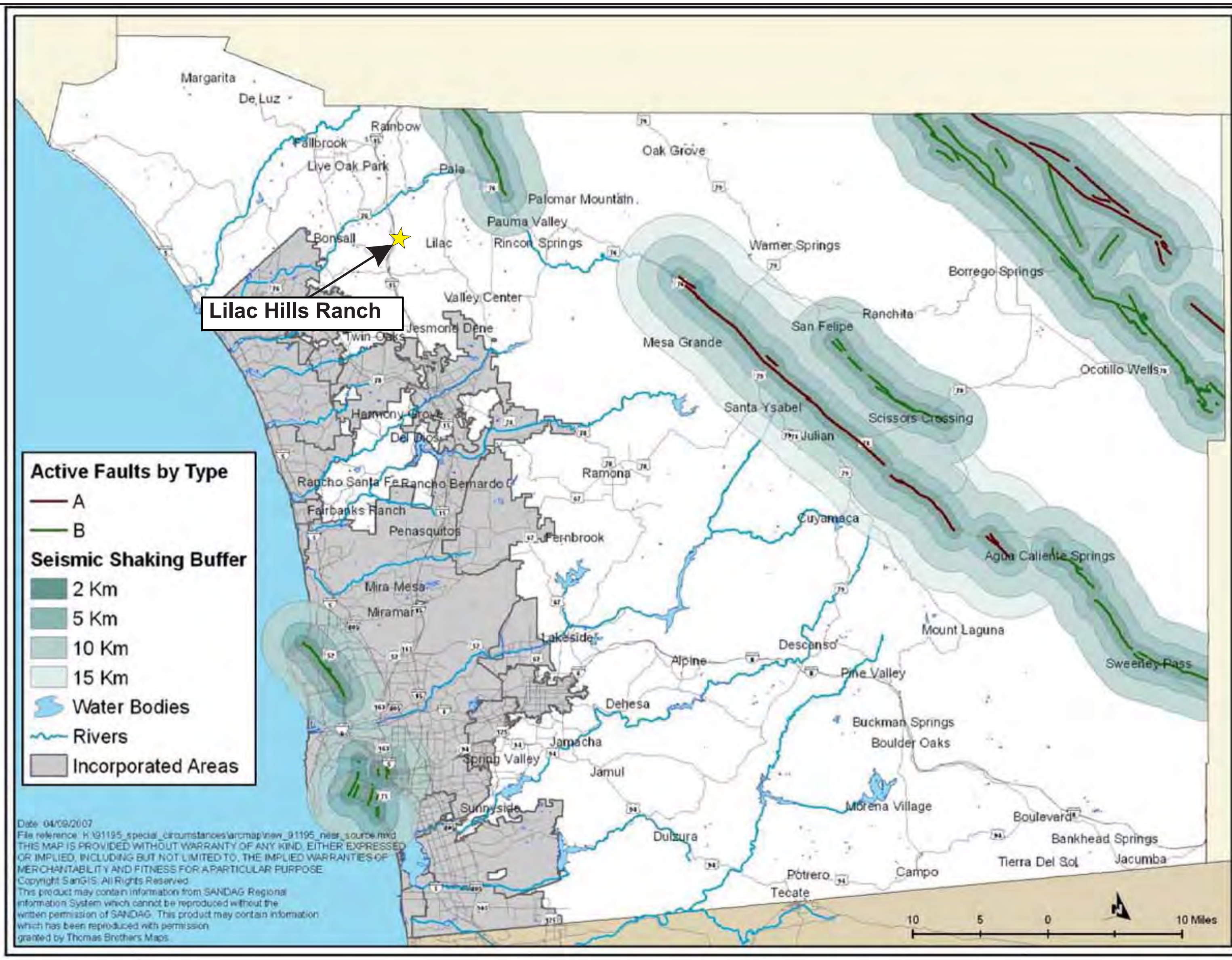
P/W 1102-01 **Report No. 1102-01-B-9**



Guidelines for Determining Significance
Geologic Hazards 25

Near Source Shaking Zones

SOURCE MAP - County of San Diego Guidelines for Determining Significance, Geologic Hazards; July 30, 2007

FIGURE 5

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
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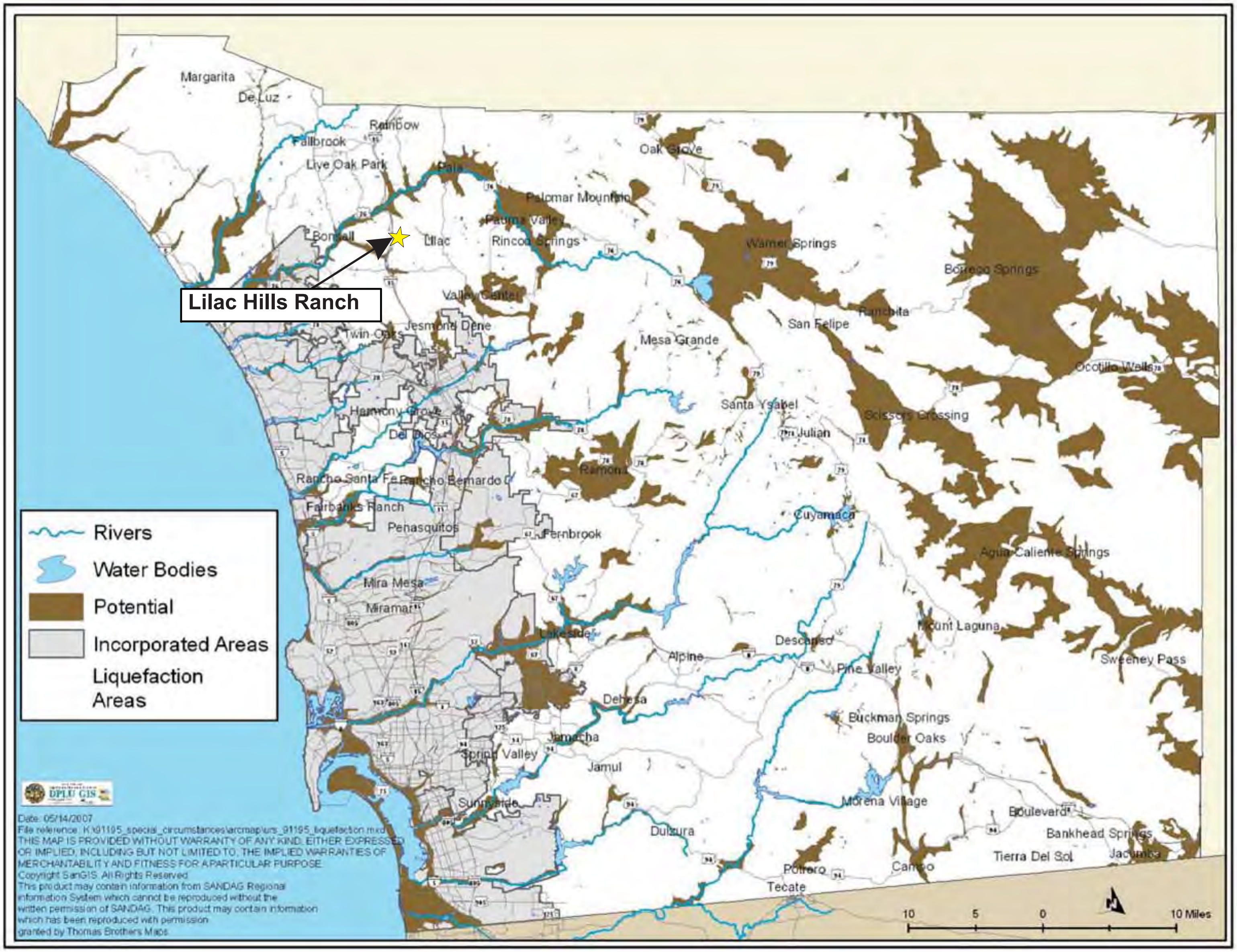
P/W 1102-01 Report No. 1102-01-B-9



Guidelines for Determining Significance
Geologic Hazards

26

Potential Liquefaction Areas

SOURCE MAP - County of San Diego Guidelines for Determining Significance, Geologic Hazards; July 30, 2007

FIGURE 6

AGS
ADVANCED GEOTECHNICAL SOLUTIONS, INC.
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P/W 1102-01 Report No. 1102-01-B-9

SCALE: 1"=200'

LEGEND

af	ARTIFICIAL FILL
Qal	ALLUVIUM
Qoal	OLDER ALLUVIUM
Kgr	GRANITIC ROCK
	APPROXIMATE LOCATION OF GEOLOGIC CONTACT (DOTTED WHERE BURRIED, QUERIED WHERE UNKNOWN)
	APPROXIMATE PROJECT BOUNDARY
T-28	APPROXIMATE LOCATION OF EXPLORATORY TRENCH, AGS, INC. 2012
TP-43	APPROXIMATE LOCATION OF BACKHOE TEST PIT, PSE, INC. 2007
SLB-8	APPROXIMATE LOCATION OF SEISMIC REFRACTION LINE, AGS, INC. 2012
SL-19	APPROXIMATE LOCATION OF SEISMIC REFRACTION LINE, PSE, INC. 2007
AT-19	APPROXIMATE LOCATION OF AIR TRACK BORING AGS, INC. 2012
AT-121	APPROXIMATE LOCATION OF AIR TRACK BORING PSE, INC. 2007
X X X X X	GRANITIC ROCK OUTCROPS
A A'	APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
N.A.P.	NOT A PART

SEE SHEET 2

SHEET 1

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220,
Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#	Report#	Date:
P/W 1102-01	1102-01	APRIL 2, 2012

COUNTY OF SAN DIEGO
TENTATIVE MAP

LILAC HILLS RANCH
MASTER TENTATIVE MAP

Dwg: c:\drafting\advanced geotechnical solutions\jobs\project 1102-01\rprt-of-3-29-12\plate-1.dwg Plot'd: 4/2/2012 8:29:11 PM

SEE SHEET 1 FOR LEGEND

SHEET 2

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave. Suite 220,
Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#	Date:
P/W 1102-01	APRIL 2, 2012

COUNTY OF SAN DIEGO

TENTATIVE MAP

LILAC HILLS RANCH

MASTER TENTATIVE MAP

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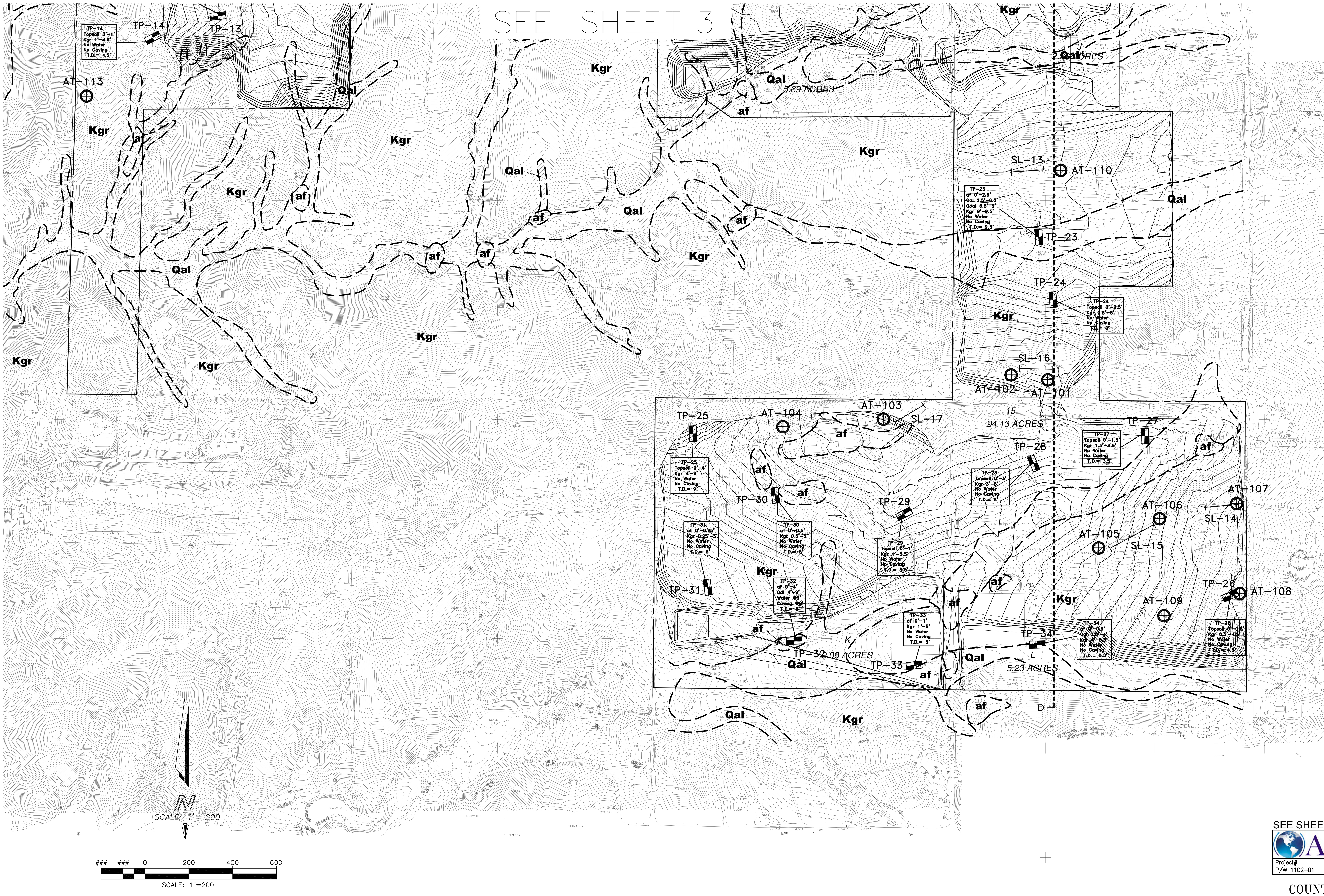

SEE SHEET 1 FOR LEGEND SHEET 3

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220,
Murrieta, California 92562
Telephone (619)708-1649 Fax: (714)409-3287

Project#	Date:
P/W 1102-01	APRIL 2, 2012

COUNTY OF SAN DIEGO
TENTATIVE MAP

LILAC HILLS RANCH

MASTER TENTATIVE MAP

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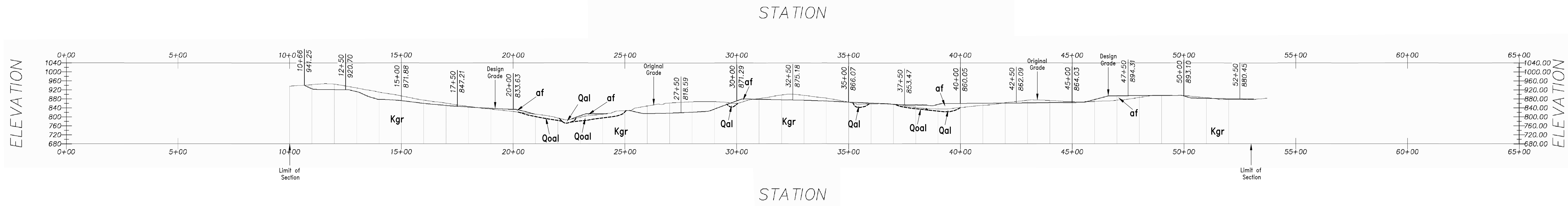
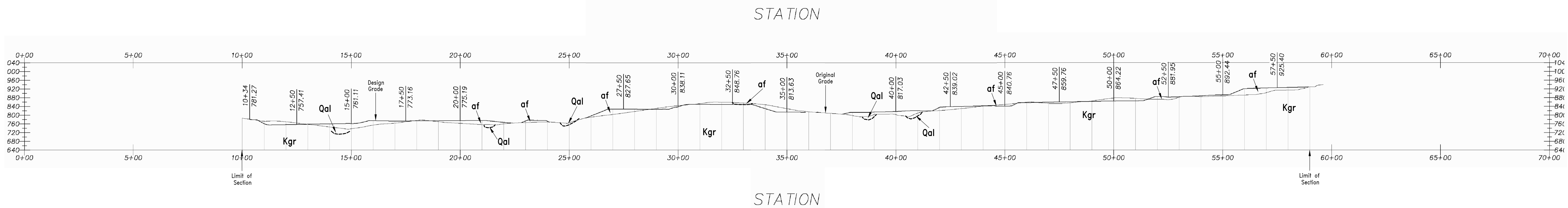
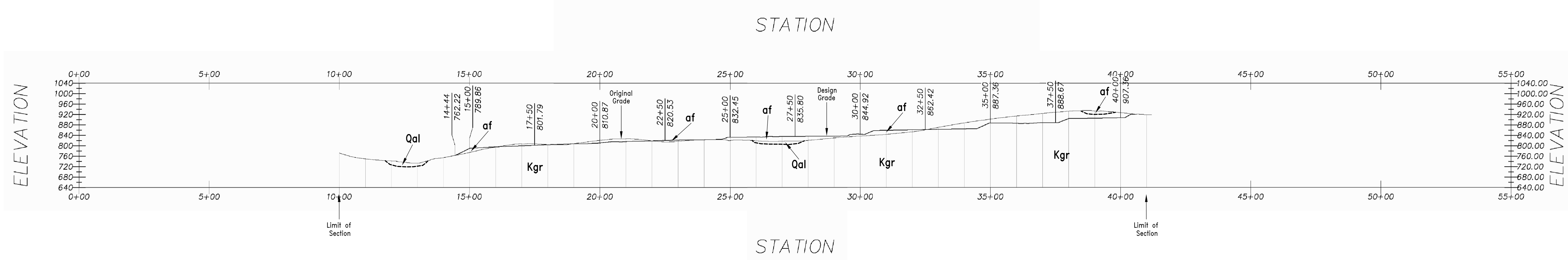
PROFILE: SECTION A–A



PROFILE: SECTION B–B



PROFILE: SECTION C–C



SCALE: 1"=200'

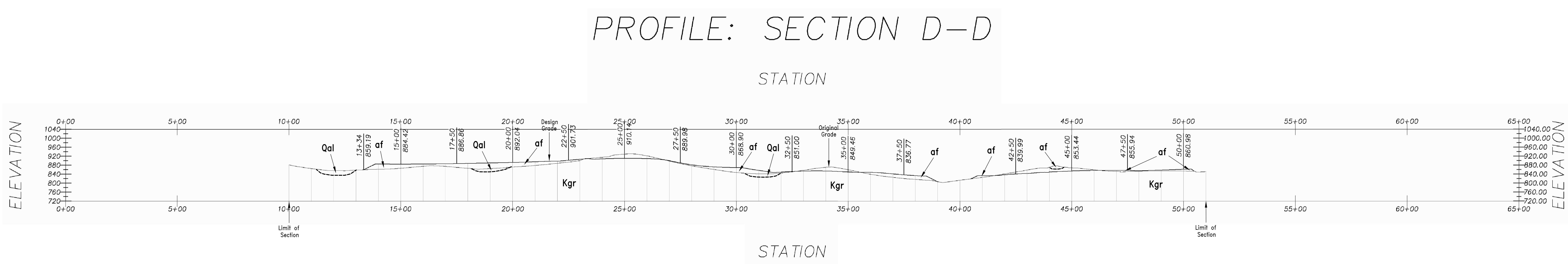
SHEET 4

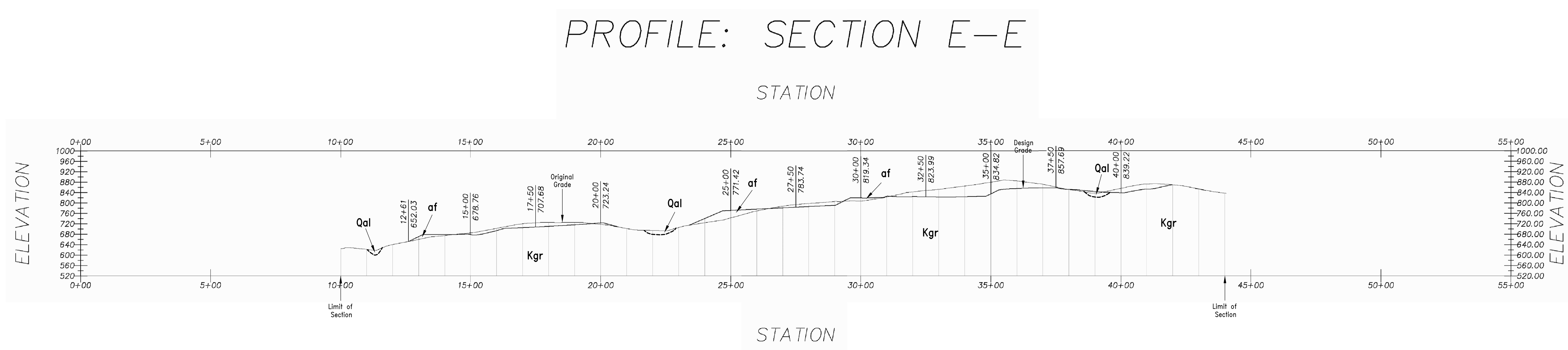
AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220, Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project# P/W 1102-01

Date: APRIL 2, 2012

CROSS–SECTIONS A–A', B–B', C–C'





CROSS-SECTIONS D-D', E-E'

SCALE: 1"=200'

SHEET 5

AGS
ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220,
Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#
P/W 1102-01

Date:
APRIL 2, 2012

APPENDIX A

CITED REFERENCES

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APPENDIX B

SUBSURFACE INVESTIGATION

TEST PIT LOGS

T-1 THROUGH T-28

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
MARCH 2012

Project Las Lilas
Date Excavated 12/21/11 & 2/21/12
Logged by PJD
Equipment Case 580M Extend-a-hoe w/24" bucket

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-1	0.0 – 6.5	SW	**ALLUVIUM** (Qal): WELL GRADED SAND, fine- to coarse-grained, reddish brown, slightly moist, loose.
		SM	@ 1 ft. becomes SILTY SAND with clay, dark brown, moist, loose to medium dense; root hairs, occasional gravel and small cobble.
			@ 2.5 ft. becomes reddish brown.
		SC	@ 4.5 ft. becomes CLAYEY SAND, fine-grained, yellow brown to olive brown, medium dense.
	6.5 – 8.0		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard, breaks into fine- to coarse-grained sand, reddish brown to olive brown.
			@ 7.0 ft. becomes highly weathered, hard.
			@ 8.0 ft. becomes very hard; refusal.
			TOTAL DEPTH 8.0 FT. NO WATER, NO CAVING
T-2	0.0 – 1.5	SM/SC	**TOPSOIL** (No Map Symbol): SILTY to CLAYEY SAND, dark reddish brown, moist, loose.
	1.5 – 4.5		**GRANITIC ROCK** (Kgr): SILTY SAND with CLAY (Residual Soil), fine- to coarse-grained, reddish brown, slightly moist, medium dense.
			@ 2.5 ft. becomes dense; trace clay.
			@ 3.5 ft. highly weathered, moderately hard, breaks into silty fine- to coarse-grained sand.
			@ 4.0 ft. becomes hard to very hard.
			@ 4.5 ft. refusal.
			TOTAL DEPTH 4.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-3	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND with CLAY, fine- to coarse-grained, dark brown, moist, loose.
	1.0 – 8.5		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard to hard, olive brown, breaks into clayey fine- to coarse-grained sand. @ 3.0 ft. becomes highly weathered, hard to very hard. @ 8.5 ft. practical refusal. TOTAL DEPTH 8.5 FT. NO WATER, NO CAVING
T-4	0.0 – 3.0	SM/SC	**ALLUVIUM** (Qal): SILTY to CLAYEY SAND, fine- to coarse-grained, dark brown to dark reddish brown, moist, loose. @ 2 ft. becomes medium dense.
	3.0 – 8.0		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard, breaks into fine- to coarse-grained sand, orangish brown. @ 4.0 ft. becomes hard. @ 5.0 ft. becomes highly weathered, very hard. @ 8.0 ft. practical refusal TOTAL DEPTH 8.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-5	0.0 – 1.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND with CLAY, fine- to coarse-grained, dark red brown, moist, loose; roots to 3 ft. deep.
	1.5 – 12.0	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND with CLAY, fine- to medium-grained, dark brown, slightly moist, medium dense; porous.
		SC	@ 3.0 ft. becomes CLAYEY SAND, medium dense to dense.
			@ 6.0 ft. weakly cemented, pinhole porosity.
			TOTAL DEPTH 12.0 FT. NO WATER, NO CAVING
T-6	0.0 – 1.5	SM/SC	**TOPSOIL** (No Map Symbol): SILTY to CLAYEY SAND, fine- to medium-grained, dark red brown, moist to wet, loose.
	1.5 – 13.0	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND with CLAY, fine- to coarse-grained, dark brown to dark grayish brown, slightly moist, medium dense to dense; weakly cemented, micaceous.
		SC	@ 6.0 ft. becomes CLAYEY SAND with SILT.
		SM/SC	@ 11.0 ft. SILTY to CLAYEY SAND, fine-grained, mottled reddish brown to olive gray.
		SM	@ 12.0 ft. SILTY SAND, fine- to coarse-grained.
			TOTAL DEPTH 13.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-7	0.0 – 10.0		**GRANITIC ROCK** (Kgr): SILTY SAND with CLAY (Residual Soil), fine- to coarse-grained, reddish brown, slightly moist to moist, loose. @ 1.0 ft. becomes medium dense. @ 2.0 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand, orangish brown; trace silt. @ 4.0 ft. becomes olive brown with abundant iron oxide. @ 6.0 ft. becomes highly weathered, hard. @ 8.0 ft. becomes very hard. @ 10.0 ft. practical refusal. TOTAL DEPTH 10.0 FT. NO WATER, NO CAVING
T-8	0.0 – 11.0		**GRANITIC ROCK** (Kgr): SILTY SAND with CLAY (Residual Soil), fine- to coarse-grained, reddish brown, slightly moist, loose to medium dense. @ 2.5 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown; abundant iron oxide. @ 4.5 ft. becomes gray to olive brown. @ 6.0 ft. becomes highly weathered, hard. @ 8.5 ft. becomes very hard. @ 11.0 ft. practical refusal. TOTAL DEPTH 11.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-9	0.0 – 1.0	SC	**TOPSOIL** (No Map Symbol): CLAYEY SAND, fine- to coarse-grained, reddish brown, moist, loose.
	1.0 – 6.0	SC	**OLDER ALLUVIUM** (Qoal): CLAYEY SAND, fine- to coarse-grained, dark reddish brown to dark grayish brown, slightly moist, medium dense to dense; moderately cemented. @ 6.0 ft. practical refusal. TOTAL DEPTH 6.0 FT. NO WATER, NO CAVING
T-10	0.0 – 9.0		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard, breaks into fine- to coarse-grained sand, orangish brown. @ 4.0 ft. becomes hard. @ 6.0 ft. becomes highly weathered, very hard. @ 9.0 ft. practical refusal. TOTAL DEPTH 9.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-11	0.0 – 1.5	SM/SC	**ALLUVIUM** (Qal): SILTY to CLAYEY SAND, fine- to coarse- grained, dark brown, moist to wet, loose.
	1.5 – 7.5	SC	**OLDER ALLUVIUM** (Qoal): CLAYEY SAND, fine- to coarse-grained, dark reddish brown to dark brown, slightly moist to moist, medium dense to dense; weakly cemented. @ 3.0 ft. becomes dense, moderately cemented.
			TOTAL DEPTH 7.5 FT. NO WATER, NO CAVING
T-12	0.0 – 1.5	SM	**TOPSOIL** (No Map Symbol) SILTY SAND, fine- to coarse-grained, dark reddish brown, slightly moist, loose.
	1.5 – 4.0	SC	**OLDER ALLUVIUM** (Qoal): CLAYEY SAND, fine- to coarse-grained, dark brown, slightly moist, medium dense to dense.
	4.0 – 6.0		**GRANITIC ROCK** (Kgr): highly to moderately weathered, hard to very hard, breaks into olive gray, fine-grained silty sand. @ 6.0 ft. very hard, slightly weathered; refusal.
			TOTAL DEPTH 6.0 FT. NO WATER, NO CAVING

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-13	0.0 – 3.5	SM/SC	**TOPSOIL** (No Map Symbol): SILTY to CLAYEY SAND, fine- to coarse-grained, reddish brown, moist, loose to medium dense.
	3.5 – 5.5	SM/SC	**OLDER ALLUVIUM** (Qoal): SILTY to CLAYEY SAND, fine- to coarse-grained, dark brown, slightly moist, dense; moderately cemented. @ 4.5 ft. becomes very dense. @ 5.5 ft. practical refusal. TOTAL DEPTH 5.5 FT. NO WATER, NO CAVING
T-14	0.0 – 2.0	SM	**TOPSOIL** (No Map Symbol) SILTY SAND with CLAY, fine- to coarse-grained, reddish brown, moist, loose.
	2.0 – 7.5		**GRANITIC ROCK** (Kgr): SILTY SAND with CLAY (Residual Soil), fine- to coarse-grained, reddish brown, slightly moist to moist, medium dense. @ 4.0 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown; abundant iron oxide @ 6.0 ft. becomes highly weathered, hard to very hard. @ 7.5 ft. practical refusal. TOTAL DEPTH 7.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-15	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to coarse-grained, brown to dark reddish brown, moist, loose.
	1.0 – 10.0		**GRANITIC ROCK** (Kgr): SILTY SAND (Residual Soil), fine- to coarse-grained, reddish brown, moist, loose to medium dense; trace clay. @ 4.5 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown. @ 6.0 ft. becomes highly weathered, hard, grayish brown with iron oxide. @ 8.0 ft. becomes very hard. @ 10.0 ft practical refusal. TOTAL DEPTH 10.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-16	0.0 – 1.0	SM/SC	**TOPSOIL** (No Map Symbol): SILTY to CLAYEY SAND, fine- to coarse-grained, brown to dark reddish brown, moist, loose; abundant roots.
	1.0 – 9.5		**GRANITIC ROCK** (Kgr): SILTY SAND (Residual Soil), fine- to coarse-grained, reddish brown, moist, loose to medium dense; trace clay. @ 4.0 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown. @ 6.0 ft. becomes highly weathered, hard. @ 8.0 ft. becomes very hard. @ 9.5 ft. practical refusal. TOTAL DEPTH 9.5 FT. NO WATER, NO CAVING
T-17	0.0 – 5.5		**GRANITIC ROCK** (Kgr): SILTY SAND (Residual Soil), fine- to coarse-grained, reddish brown, slightly moist, loose to medium dense; trace clay. @ 0.5 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand, orangish brown to reddish brown; abundant iron oxide @ 2.5 ft. becomes highly weathered, hard to very hard. @ 5.5 ft. practical refusal. TOTAL DEPTH 5.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-18	0.0 – 8.0	SM	**ALLUVIUM** (Qal): SILTY SAND, fine- to coarse- grained, reddish brown, moist, loose; trace clay.
	8.0 – 13.0	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND with CLAY, fine- to coarse-grained, yellowish brown to dark brown, slightly moist, medium dense; weakly cemented; abundant iron oxide. @ 10.0 ft. becomes dense, moderately cemented. TOTAL DEPTH 13.0 FT. NO WATER, NO CAVING

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T-19	0.0 – 2.0	SM/SC	**TOPSOIL** (No Map Symbol): SILTY SAND with CLAY, fine- to coarse-grained, dark brown to dark reddish brown, moist, loose.
	2.0 – 11.0		**GRANITIC ROCK** (Kgr): CLAYEY SAND with SILT (Residual Soil), fine- to coarse-grained, reddish brown, moist, medium dense. @ 4.0 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown; trace clay. @ 6.5 ft. becomes highly weathered, hard, grayish brown with iron oxide; some small gravel. @ 9.0 ft. becomes very hard. @ 11.0 ft. practical refusal. TOTAL DEPTH 11.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-20	0.0 – 8.0		**GRANITIC ROCK** (Kgr): CLAYEY SAND (Residual Soil), fine- to coarse-grained, reddish brown, moist, loose to medium dense; large roots in upper 1 ft. @ 1.0 ft. completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, orangish brown; trace clay. @ 3.5 ft. becomes highly weathered, hard. @ 5.5 ft. becomes very hard. @ 8.0 ft. practical refusal. TOTAL DEPTH 8.0 FT. NO WATER, NO CAVING
T-21	0.0 – 3.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, dark brown to dark reddish brown, moist, loose; roots.
	3.0 – 9.0	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND, fine- to coarse-grained, reddish brown, slightly moist, medium dense. @ 6.0 ft. becomes reddish brown to dark grayish brown, weakly to moderately cemented, moderately hard to hard. @ 9.0 ft. practical refusal. TOTAL DEPTH 9.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-22	0.0 – 1.5	SM	**TOPSOIL** (No Map Symbol) SILTY SAND, fine- to coarse-grained, dark brown, slightly moist to moist, loose; trace clay, abundant roots.
	1.5 – 6.0	SM/SC	**OLDER ALLUVIUM** (Qoal): SILT to CLAYEY SAND, fine- to coarse-grained, grayish brown to reddish brown, slightly moist, medium dense; fine roots, porous. @ 2.5 ft. becomes reddish brown, weakly cemented, moderately hard.
	4.0 – 8.5		**GRANITIC ROCK** (Kgr): completely to highly weathered, moderately hard, breaks into orange brown, fine- to coarse-grained sand. @ 7.0 ft. becomes moderately weathered, hard. @ 8.5 ft. practical refusal. TOTAL DEPTH 8.5 FT. NO WATER, NO CAVING
T-23	0.0 – 3.0	SM/SC	**TOPSOIL** (No Map Symbol) SILTY to CLAYEY SAND, fine- to coarse-grained, dark reddish brown to dark brown, moist, loose; roots.
	3.0 – 7.5	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND with CLAY, fine- to coarse-grained, reddish brown, slightly moist, medium dense. @ 4.0 ft. becomes dark grayish brown, slightly moist, moderately hard to hard; porous.
	7.5 – 10.0		**GRANITIC ROCK** (Kgr): highly to completely weathered, moderately hard, breaks into olive gray, fine- to coarse-grained clayey sand; some clay alteration. @ 9.0 ft. hard, slightly to moderately weathered; @ 10.0 ft. very hard, practical refusal. TOTAL DEPTH 10.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-24	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to coarse-grained, brown to dark reddish brown, moist, loose.
	1.0 – 5.0		**GRANITIC ROCK** (Kgr): SILTY SAND (Residual Soil), fine- to coarse-grained, reddish brown, moist, loose to medium dense; trace clay. @ 3.0 ft. completely weathered, moderately hard to hard, breaks into fine- to coarse-grained sand with silt, yellowish brown to reddish brown. TOTAL DEPTH 5.0 FT. NO WATER, NO CAVING
T-25	0.0 – 0.5	SM	**TOPSOIL** (No Map Symbol) SILTY SAND, fine- to medium-grained, brown, moist, loose; trace clay.
	0.5 – 5.0	SC	**OLDER ALLUVIUM** (Qoal): CLAYEY SAND, fine- to coarse-grained, dark brown, slightly moist to moist, dense; fine roots, porous. @ 4.0 ft. becomes SILTY to CLAYEY SAND, weakly cemented, moderately hard.
	5.0 – 12.0		**GRANITIC ROCK** (Kgr): completely to highly weathered, moderately hard, breaks into dark grayish to reddish gray, clayey fine- to coarse-grained sand; micaceous, abundant iron oxide development, clay seams. @ 8.0 ft. highly weathered, less clay. @ 10.0 ft. becomes moderately weathered, hard. @ 12.0 ft. practical refusal. TOTAL DEPTH 12.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-26	0.0 – 8.5		**GRANITIC ROCK** (Kgr): completely weathered, soft to moderately hard, breaks into fine- to coarse-grained sand with clay, dark olive gray. @ 3.0 ft. becomes hard. @ 6.5 ft. becomes highly weathered, very hard. @ 8.5 ft. practical refusal. TOTAL DEPTH 8.5 FT. NO WATER, NO CAVING
T-27	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol) SILTY SAND, fine- to coarse-grained, brown to reddish brown, moist, loose.
	1.0 – 5.5	SM/SC	**OLDER ALLUVIUM** (Qoal): SILTY to CLAYEY SAND, fine- to coarse-grained, grayish brown to reddish brown, slightly moist, medium dense to dense.
	5.5 – 7.5		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard, breaks into olive gray, fine- to coarse-grained clayey sand. @ 6.0 ft moderately weathered, hard. @ 7.0 ft. very hard, slightly weathered. @ 7.5 ft. practical refusal. TOTAL DEPTH 7.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
T-28	0.0 – 0.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to coarse-grained, brown to dark reddish brown, moist, loose.
	0.5 – 7.5		**GRANITIC ROCK** (Kgr): completely weathered, moderately hard, breaks into fine- to coarse-grained sand with silt, yellowish brown to reddish brown. @ 5.5' highly weathered, hard, olive gray. @ 7.0 ft. very hard. @ 7.5 ft. practical refusal.
			TOTAL DEPTH 7.5 FT. NO WATER, NO CAVING

TEST PIT LOGS

TP-1 THROUGH TP-14

AND TP-23 THROUGH TP-43

PACIFIC SOILS ENGINEERING, INC.
MAY 2007

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

Work Order 401120
Date Excavated 4/2/07
Excavated by CI
Equipment John Deere 310G Backhoe w/24" bucket

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-1	0.0 – 1.5	ML	**ARTIFICIAL FILL** (af): SANDY SILT, fine- to coarse-grained, reddish brown, moist, soft.
	1.5 – 3.8		**OLDER ALLUVIUM** (Qoal): SANDY SILT, fine- to medium-grained, dark brownish red, moist, moderately hard to hard; difficult digging.
			TOTAL DEPTH 3.8 FT. NO WATER, NO CAVING
TP-2	0.0 – 10.0	ML	**ARTIFICIAL FILL** (af): CLAYEY SILT, reddish brown to grayish brown, slightly moist to wet, soft. @ 7.5 ft. becoming dark gray. @ 8.0 ft. some sandy zones, fine- to coarse-grained. @ 9.5 ft. strong organic odor. @ 10.0 ft. trace fine gravel; wet.
			TOTAL DEPTH 10.0 FT. NO WATER, NO CAVING

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LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-3	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to coarse-grained, light reddish brown, dry, loose.
	1.0 – 2.0		**GRANITIC ROCK** (Kgr): brownish red, slightly moist, moderately hard to very hard; decomposed. @ 2.0 ft. becomes highly weathered, very hard; refusal. TOTAL DEPTH 2.0 FT. NO WATER, NO CAVING
TP-4	0.0 – 0.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to coarse-grained, slightly moist, loose to moderately dense.
	0.5 – 6.0		**GRANITIC ROCK** (Kgr): grayish brown to reddish brown, slightly moist, moderately hard to hard; decomposed. @ 3.5 ft. hard; highly weathered. @ 6.0 ft. very hard; refusal. TOTAL DEPTH 6.0 FT. NO WATER, NO CAVING

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-5	0.0 – 8.5		**OLDER ALLUVIUM** (Qoal): SANDY SILT, fine- to medium-grained, light reddish brown, slightly moist, soft to moderately hard; porous. @ 1.5 ft. becoming moderately hard to hard; slightly porous. @ 2.0 ft. dark grayish brown; micaceous. @ 4.0 ft. becomes CLAYEY SILT, grayish brown to red brown, moist, stiff to very stiff. @ 7.5 ft. trace fine gravel; micaceous; difficult digging. TOTAL DEPTH 8.5 FT. NO WATER, NO CAVING
TP-6	0.0 – 6.0		**OLDER ALLUVIUM** (Qoal): SANDY SILT, fine- to medium-grained, light brown, slightly moist, soft to moderately hard. @ 2.5 ft. hard. @ 4.0 ft. difficult digging. @ 5.5 ft. becomes light reddish brown. TOTAL DEPTH 6.0 FT. NO WATER, NO CAVING

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TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-7	0.0 – 5.5		**OLDER ALLUVIUM** (Qoal): CLAYEY SAND, fine- to medium-grained, light brown, slightly moist, soft to moderately hard. @ 3.0 ft. becoming moderately hard, moist.
	5.5 – 7.0		**GRANITIC ROCK** (Kgr): brownish red to gray brown, moist, hard; decomposed. @ 6.0 ft. becomes moderately weathered. @ 7.0 ft. refusal. TOTAL DEPTH 7.0 FT. NO WATER, NO CAVING
TP-8	0.0 – 9.0	SM	**ARTIFICIAL FILL** (af): SILTY SAND to SANDY SILT, fine- to coarse-grained, slightly moist to moist, loose. @ 4.5 ft. becomes soft to moderately hard; common fine to coarse gravel. TOTAL DEPTH 9.0 FT. NO WATER, NO CAVING

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TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-9	0.0 – 1.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND to CLAYEY SAND, fine- to coarse-grained, brownish red, slightly moist, loose.
	1.5 – 6.5		**GRANITIC ROCK** (Kgr): reddish brown, slightly moist, moderately hard; decomposed. @ 3.5 ft. brownish gray, slightly moist, hard; highly weathered. @ 6.5 ft. moderately weathered; refusal.
			TOTAL DEPTH 6.5 FT. NO WATER, NO CAVING
TP-10	0.0 – 2.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, light reddish brown, slightly moist, loose.
	2.5 – 4.5		**GRANITIC ROCK** (Kgr): light grayish brown, slightly moist, moderately hard; decomposed. @ 4.5 ft. becomes hard to very hard; moderately weathered; refusal.
			TOTAL DEPTH 4.5 FT. NO WATER, NO CAVING

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TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-11	0.0 – 2.5	ML	**TOPSOIL** (No Map Symbol): SANDY SILT, fine- to medium-grained, light reddish brown, dry to slightly moist, soft.
	2.5 – 3.0		**GRANITIC ROCK** (Kgr): reddish orange, slightly moist, hard; moderately weathered. @ 3.0 ft. very hard; slightly weathered; refusal.
			TOTAL DEPTH 3.0 FT. NO WATER, NO CAVING
TP-12	0.0 – 1.5	ML	**TOPSOIL** (No Map Symbol) SANDY SILT, fine- to coarse-grained, light reddish brown, slightly moist, soft.
	1.5 – 3.5		**GRANITIC ROCK** (Kgr): reddish orange, slightly moist, hard; moderately weathered. @ 3.5 ft. very hard, slightly weathered; refusal.
			TOTAL DEPTH 3.5 FT. NO WATER, NO CAVING

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-13	0.0 – 1.5	ML	**TOPSOIL** (No Map Symbol): SANDY SILT, fine- to medium-grained, light reddish brown, dry to slightly moist, soft to firm.
	1.5 – 2.5		**GRANITIC ROCK** (Kgr): red orange, slightly moist, hard; highly weathered. @ 2.5 ft. becomes very hard; moderately weathered; refusal. TOTAL DEPTH 2.5 FT. NO WATER, NO CAVING
TP-14	0.0 – 1.0	ML	**TOPSOIL** (No Map Symbol) SANDY SILT, fine- to medium-grained, light reddish brown, dry, soft.
	1.0 – 4.5		**GRANITIC ROCK** (Kgr): reddish orange, slightly moist, moderately hard; decomposed. @ 2.0 ft. becomes hard; highly weathered. @ 4.5 ft. becomes very hard, moderately weathered; refusal. TOTAL DEPTH 4.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-23 (~El. 840)	0.0 – 2.5	SM	**ARTIFICIAL FILL** (af): SILTY SAND, light reddish brown, moist, loose to moderately dense. @ 2.0 ft. bulk.
	2.5 – 6.5	SM	**ALLUVIUM** (Qal): SILTY SAND, dark reddish brown, moist, moderately dense; porous; faint organic smell. @ 5.0 ft. bulk.
	6.5 – 9.0	SM	**OLDER ALLUVIUM** (Qoal): SILTY SAND, reddish brown, moist, moderately dense.
	9.0 – 9.5		**GRANITIC ROCK** (Kgr): reddish brown to light olive brown, moist, hard; highly weathered; friable. @ 9.0 ft. bulk. @ 9.5 ft. refusal.
			TOTAL DEPTH 9.5 FT. NO WATER, NO CAVING

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TP-23 continued

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TP-23 continued

Work Order: 401120
Date Excavated: 04/27/07
Excavated by: AB
Equipment: John Deere 310G rubber-tired backhoe

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-24 (~El. 870)	0.0 – 2.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, red brown, slightly moist, loose to moderately dense.
	2.5 – 6.0		**GRANITIC ROCK** (Kgr): red brown with black and white, moist to wet; highly weathered; friable; medium to coarse-grained; micaceous. @ 6.0 ft. moderately hard to hard; refusal.
			TOTAL DEPTH 6.0 FT. NO WATER, NO CAVING

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TP-24 continued

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TP-24 continued

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-25 (~El. 850)	0.0 – 4.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine-grained, red brown, slightly moist to moist, loose.
	4.0 – 9.0	SM	**GRANITIC ROCK** (Kgr): dark red brown, moist, soft; micaceous; highly weathered. @ 8.0 ft. light red brown with black and white, moist, moderately hard; freshening; friable, medium- to coarse-grained; micaceous. @ 9.0 ft. hard, refusal.
			TOTAL DEPTH 9.0 FT. NO WATER, NO CAVING

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TP-25 continued

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-26 (~El. 934)	0.0 – 0.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, light red brown, moist to moist; micaceous.
	0.5 – 4.5		**GRANITIC ROCK** (Kgr): light reddish brown to reddish yellow with black and white, moist; moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 4.5 ft. hard; refusal.
			TOTAL DEPTH 4.5 FT. NO WATER, NO CAVING

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TP-26 continued

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-27 (~El. 894)	0.0 – 1.5	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, light reddish brown, slightly moist to moist, loose to moderately dense.
		SM - CL	@ 0.5 ft. SILTY SAND some CLAY, reddish brown, moist, moderately dense.
	1.5 – 3.5		**GRANITIC ROCK** (Kgr): light reddish yellow with black and white, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; some cemented fragments to 4"; micaceous. @ 3.5 ft. hard; refusal.
			TOTAL DEPTH 3.5 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-28 (~El. 880)	0.0 – 3.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, light red brown, slightly moist to moist, moderately dense.
		SM	@ 3.0 ft.: SILTY SAND, reddish brown, moist, moderately dense.
	3.0 – 8.0		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 8.0 ft. hard.
			TOTAL DEPTH 8.0 FT. NO WATER, NO CAVING

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TP-28 continued

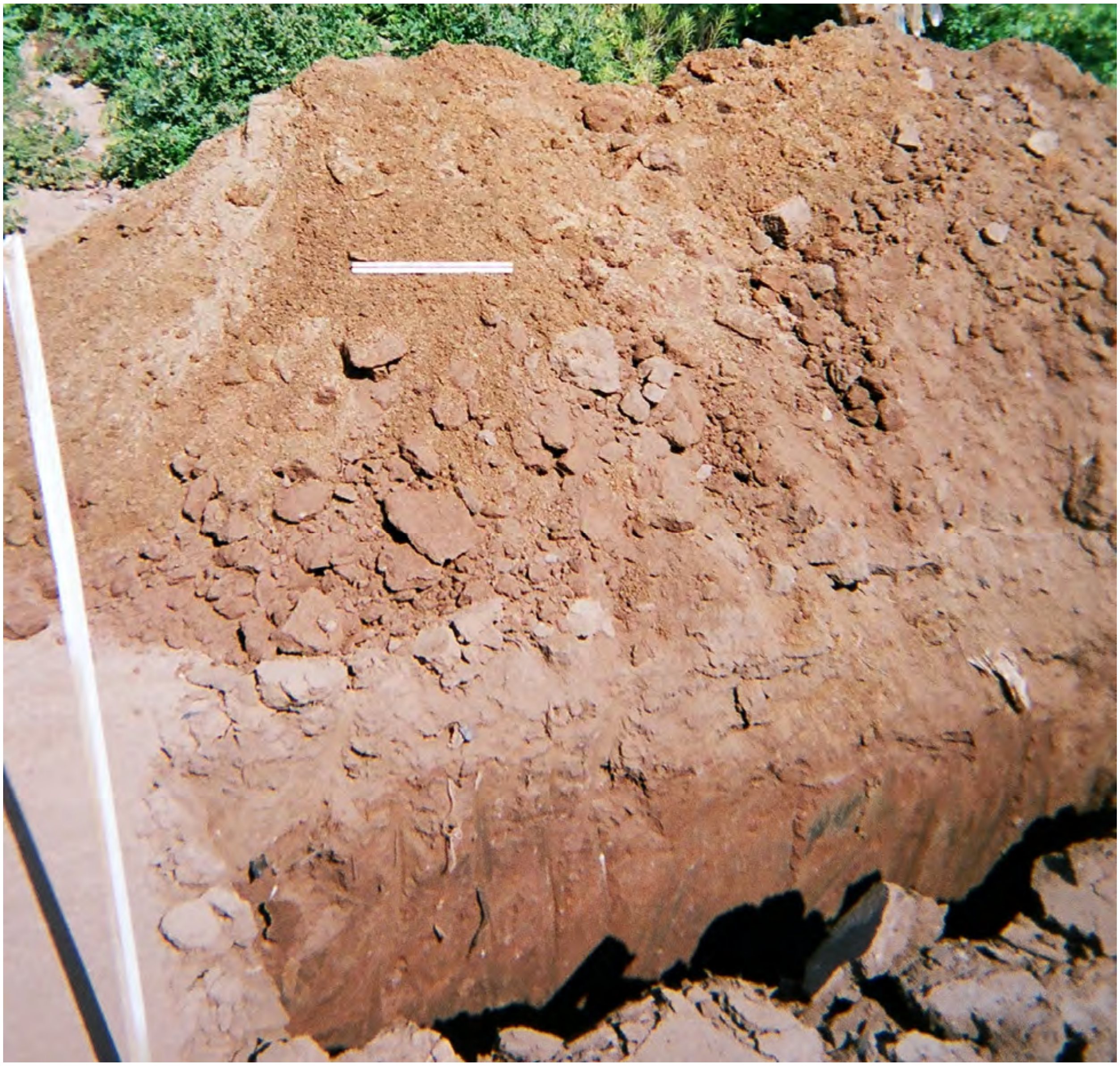

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-29 (~El. 888)	0.0 – 1.0	SM	**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, light red brown, slightly moist to moist, loose to moderately dense.
	1.0 – 5.5		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous.. @ 5.5 ft. hard; friable.
			TOTAL DEPTH 5.5 FT. NO WATER, NO CAVING

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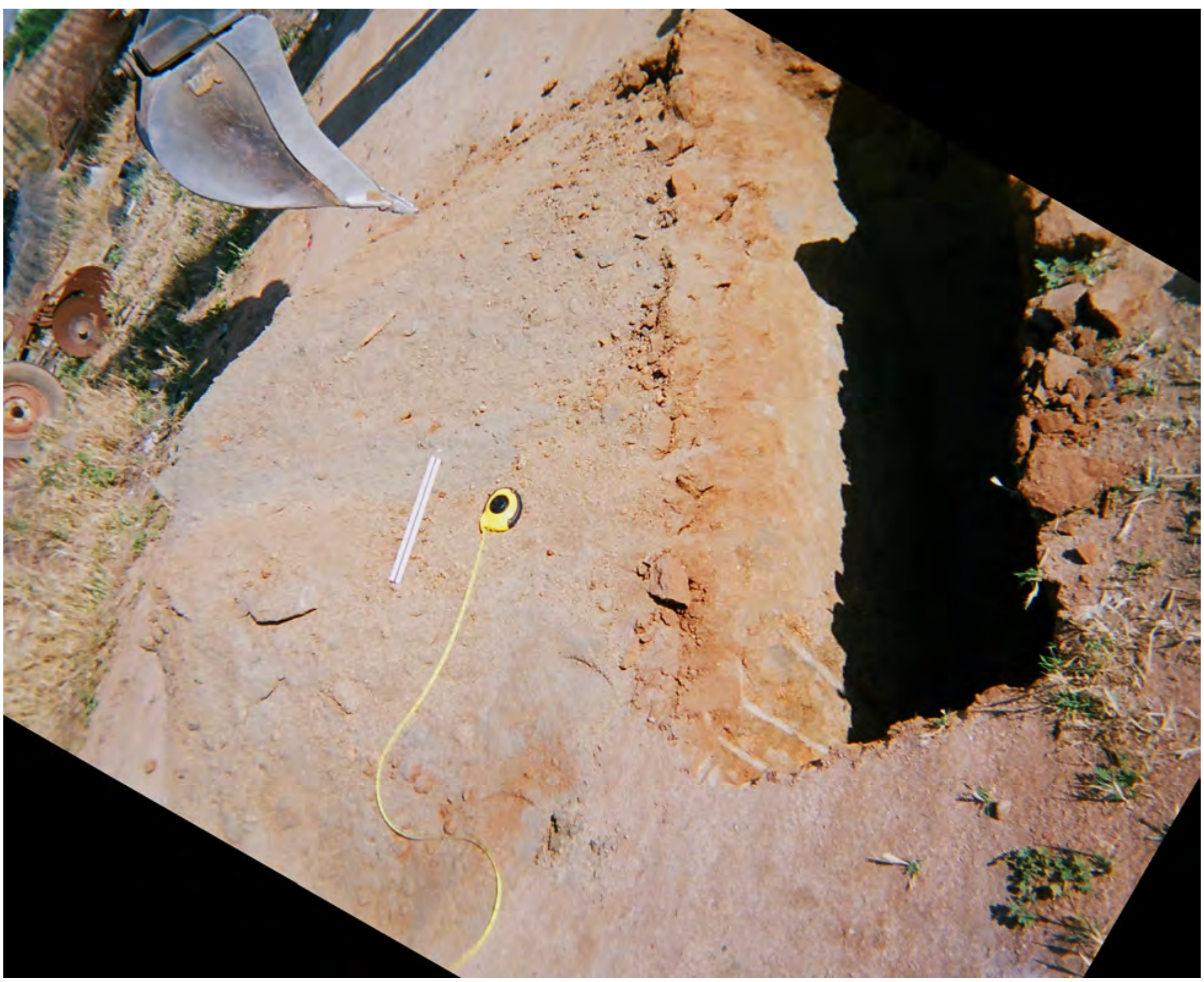
TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-30 (~El. 902)	0.0 – 0.5	SM	**ARTIFICIAL FILL** (Af): SILTY SAND, fine- to medium-grained, light red brown, slightly moist, loose to moderately dense.
	0.5 – 5.0		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 5.0 ft. hard; refusal.
			TOTAL DEPTH 5.0 FT. NO WATER, NO CAVING

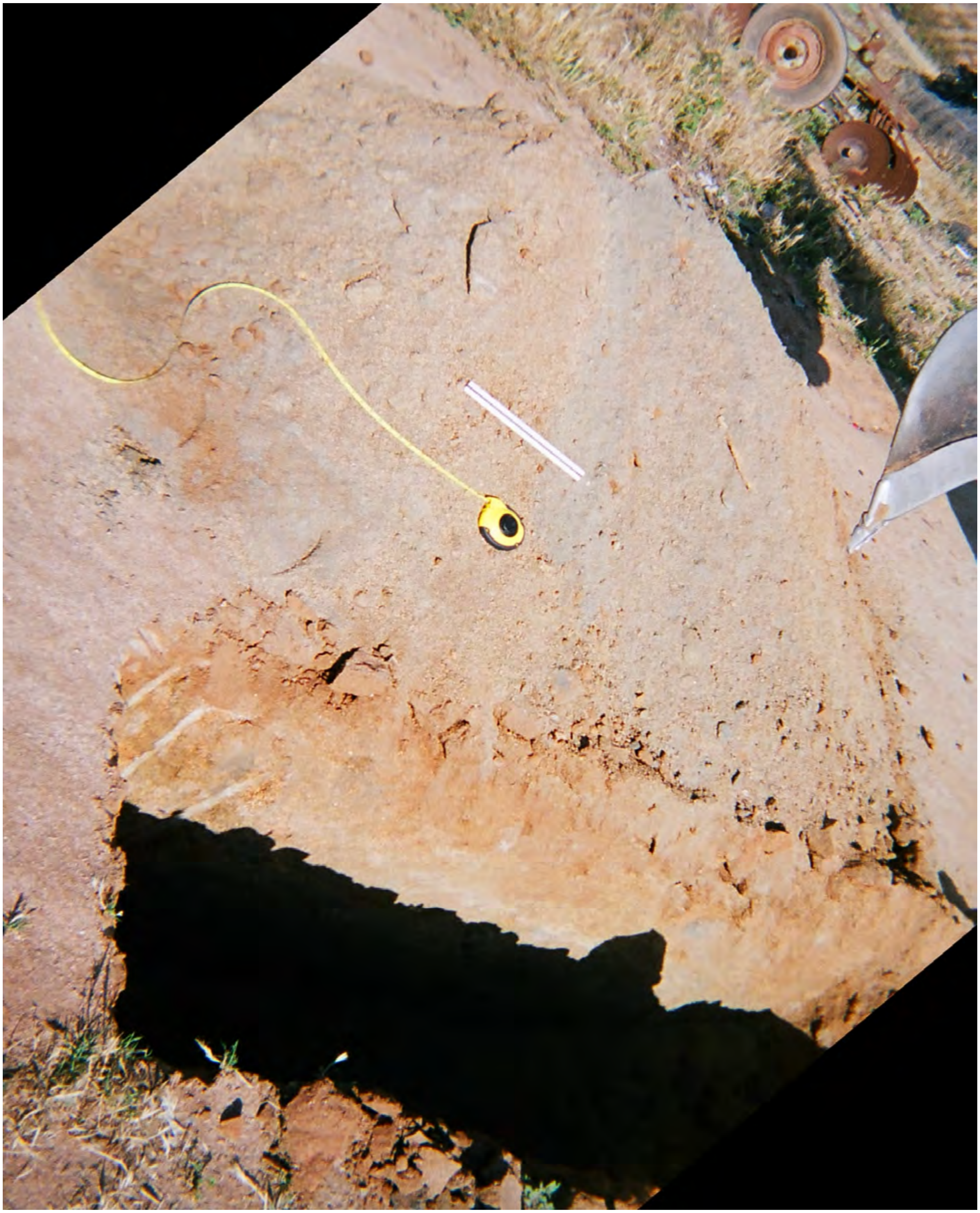


see next page

Work Order 401120

May 23, 2007

TP-30 continued



Work Order 401120

May 23, 2007

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-31 (~El. 831)	0.0 – 0.25	SM	**ARTIFICIAL FILL** (af): SILTY SAND, fine- to medium-grained, light red brown, slightly moist, loose to moderately dense.
	0.25 – 3.0		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 3.0 ft. hard; refusal.
			TOTAL DEPTH 3.0 FT. NO WATER, NO CAVING

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-32 (~El. 824)	0.0 – 4.0	SM	**ARTIFICIAL FILL** (af): SILTY SAND, fine- to medium-grained, red brown to dark brown, moist to wet, loose to medium dense; roots; plastic pipe.
	4.0 – 9.0	SM	**ALLUVIUM** (Qal): SILTY SAND some CLAY, dark brown to gray, moist to wet; granitic rock fragments; friable. @ 5.0 ft. bulk. @ 8.0 ft. caving, saturated. @ 9.0 ft. groundwater (seepage and accumulating).
			TOTAL DEPTH 9.0 FT. WATER AS NOTED, CAVING AS NOTED

see next two pages

Work Order 401120

May 23, 2007

TP-32 continued

Work Order 401120

May 23, 2007

TP-32 continued

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-33 (~El. 841)	0.0 – 1.0	SM	**ARTIFICIAL FILL** (af): SILTY SAND, fine- to medium-grained, light red brown, slightly moist, loose to moderately dense.
	1.0 – 5.0		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 3.0 ft. 10% fragments to 4" size. @ 5.0 ft. hard; refusal.
			TOTAL DEPTH 5.0 FT. NO WATER, NO CAVING

Work Order 401120

May 23, 2007

TABLE I continued

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-34 (~El. 852)	0.0 – 0.5	SM	**ARTIFICIAL FILL** (af): SILTY SAND, fine- to medium-grained, light red brown, slightly moist, loose to moderately dense.
	0.5 – 4.0	SM	**ALLUVIUM** (Qal): SILTY SAND, dark reddish brown, moist, moderately dense; porous; faint organic smell. @ 3.0 ft. bulk.
	4.0 – 5.5		**GRANITIC ROCK** (Kgr): light reddish brown, moist, moderately hard; highly weathered; friable; medium- to coarse-grained; micaceous. @ 5.5 ft. hard; refusal.
			TOTAL DEPTH 5.5 FT. NO WATER, NO CAVING

Work Order 401120
Date Excavated 5/10/07 – 5/11/07
Excavated by DTW
Equipment: John Deere 310SG Backhoe w/24"bucket

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-35	0.0 – 0.5		**TOPSOIL** (No Map Symbol): SILTY SAND, fine- to medium-grained, brown to reddish brown, dry, loose; trace pinhole porosity; trace roots.
	0.5 – 9.0		**GRANITIC ROCK** (Kgr): grayish brown to brownish gray; dry to slightly moist, soft to moderately hard; friable ; highly weathered; freshens with depth; trace roots to ~3.0 ft.; excavates to SILTY SAND, medium- to coarse-grained with some gravel. @ 2.0 ft. brownish gray to gray; slightly moist to moist, moderately hard; moderate digging. @ 4.0 ft. gray to dark gray, moist; hard digging. @ 9.0 ft. practical refusal. TOTAL DEPTH 9.0 FT. NO WATER, NO CAVING

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Work Order 401120

May 23, 2007

TP-35 continued

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-36	0.0 – 3.0		**ARTIFICIAL FILL** (af): SILTY SAND, dark brown to brownish black, moist, loose; trace boulders; trace gravel; abundant trash (metal/bottles, etc.). @ 2.5 ft. minor seepage.
	3.0 – 3.0		**GRANITIC ROCK** (Kgr): brownish gray to gray, moist, very hard; refusal. TOTAL DEPTH 3.0 FT. WATER AS NOTED, NO CAVING
TP-37	0.0 – 4.0		**ARTIFICIAL FILL** (af): SILTY SAND, dark brown to brownish black, moist, loose; trace boulders; trace gravel; abundant trash (metal/bottles, etc.). @ 3.5 ft. water.
	4.0 – 6.0		**GRANITIC ROCK** (Kgr): brownish gray with orange hues, moist, hard; weathered; excavates to SILTY SAND, medium- to coarse-grained. @ 6.0 ft. refusal. TOTAL DEPTH 6.0 FT. WATER AS NOTED, NO CAVING

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-38	0.0 – 3.0		**ARTIFICIAL FILL** (af): SILTY SAND, dark brown to brownish black, moist, loose; trace boulders; trace gravel; abundant trash (metal/bottles, etc.).
	3.0 – 3.0		**GRANITIC ROCK** (Kgr): gray, moist, hard to very hard; refusal.
			TOTAL DEPTH 3.0 FT. NO WATER, NO CAVING

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-39	0.0 – 3.0		**ARTIFICIAL FILL** (af): SILTY SAND, dark brown to brownish black, moist, loose; trace boulders; trace gravel; abundant trash (metal/bottles, etc.). @ 2.0 ft. minor caving.
	3.0 – 7.0		**ALLUVIUM** (Qal): SILTY SAND, fine- to medium-grained, dark brown to brown, slightly moist, loose; trace porosity; trace roots; trace 12" minus granitic boulders.
	7.0 – 8.5		**GRANITIC ROCK** (Kgr): grayish brown with orange hues, moist, hard; weathered; excavates to SILTY SAND, medium- to coarse-grained, minor seepage. @ 8.5 ft. refusal. TOTAL DEPTH 8.5 FT. WATER AS NOTED, CAVING AS NOTED

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Work Order 401120

May 23, 2007

TP-39 continued

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-40	0.0 – 2.0		**TOPSOIL** (No Map Symbol): SILTY SAND with cobbles; medium- to coarse-grained, dark grayish brown (10YR 4/2) to brown (10YR 4/3), slightly moist to moist, loose; trace 12" minus granitic boulders.
	0.2 – 7.0		**GRANITIC ROCK** (Kgr): orange brown, slightly moist; weathered; friable; freshens with depth; easy digging; trace 12" minus corestones; excavates to GRAVELLY SAND, medium- to coarse-grained. @ 5.0 ft. brownish gray with trace orange hues, moist, hard; hard digging; excavates to SILTY SAND due to scratching with teeth. @ 7.0 ft. practical refusal.
			TOTAL DEPTH 7.0 FT. NO WATER, NO CAVING

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Work Order 401120

May 23, 2007

TP-40 continued

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-41	0.0 – 3.0		**ALLUVIUM** (Qal): CLAYEY SAND, fine- to medium-grained with some coarse-grained, very dark grayish brown (10YR 3/2) to dark brown (10YR 3/3), slightly moist to moist, soft.
	3.0 – 5.0		**GRANITIC ROCK** (Kgr): reddish brown with orange hues, medium- to coarse phenocrysts; slightly moist, soft to moderately hard; highly weathered; freshens with depth; trace gravel; friable; easy digging; excavates to SAND, medium- to coarse-grained. @ 4.0 ft. moderately hard digging. @ 5.0 ft. practical refusal. TOTAL DEPTH 5.0 FT. NO WATER, NO CAVING

see next page

Work Order 401120

May 23, 2007

<u>TP-41 continued</u>

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-42	0.0 – 2.0		**ARTIFICIAL FILL** (af): SILTY SAND, medium- to coarse-grained, grayish brown (10YR 5/2) to pale brown (10YR 6/3), dry to slightly moist, loose.
	2.0 – 9.0		**ALLUVIUM** (Qal): SILTY SAND, medium- to coarse-grained, very dark brown (10YR 2/2) to reddish brown (5YR 4/4), slightly moist to moist, loose; trace CLAYEY overbank deposits; cross-bedded; repeated fining up sequences; very easy digging. @ 5.0 ft. caving. @ 6.0 ft. moist. @ 7.5 ft. moist to wet. @ 8.5 ft. saturated; seepage.
	9.0 – 11.0		**GRANITIC ROCK** (Kgr): dark gray, moist, moderately hard to hard; hard digging; excavates to SILTY SAND with gravel, fine- to medium-grained.
			TOTAL DEPTH 11.0 FT. WATER AS NOTED, CAVING AS NOTED

See next page

Work Order 401120

May 23, 2007

<u>TP-42 continued</u>

Work Order 401120

May 23, 2007

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-43	0.0 – 4.0		**GRANITIC ROCK** (Kgr): reddish brown (5YR 4/4) to strong brown (7.5 YR 5/6), dry; highly weathered; abundant porosity; trace roots; excavates to CLAYEY SAND, medium- to coarse-grained. @ 1.0 ft. reddish brown to orange brown, dry to slightly moist, moderately hard to hard; hard digging; excavates to coarse SAND with trace gravel. @ 3.0 ft. brownish gray to gray. @ 4.0 ft. practical refusal. TOTAL DEPTH 4.0 FT. NO WATER, NO CAVING

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Work Order 401120

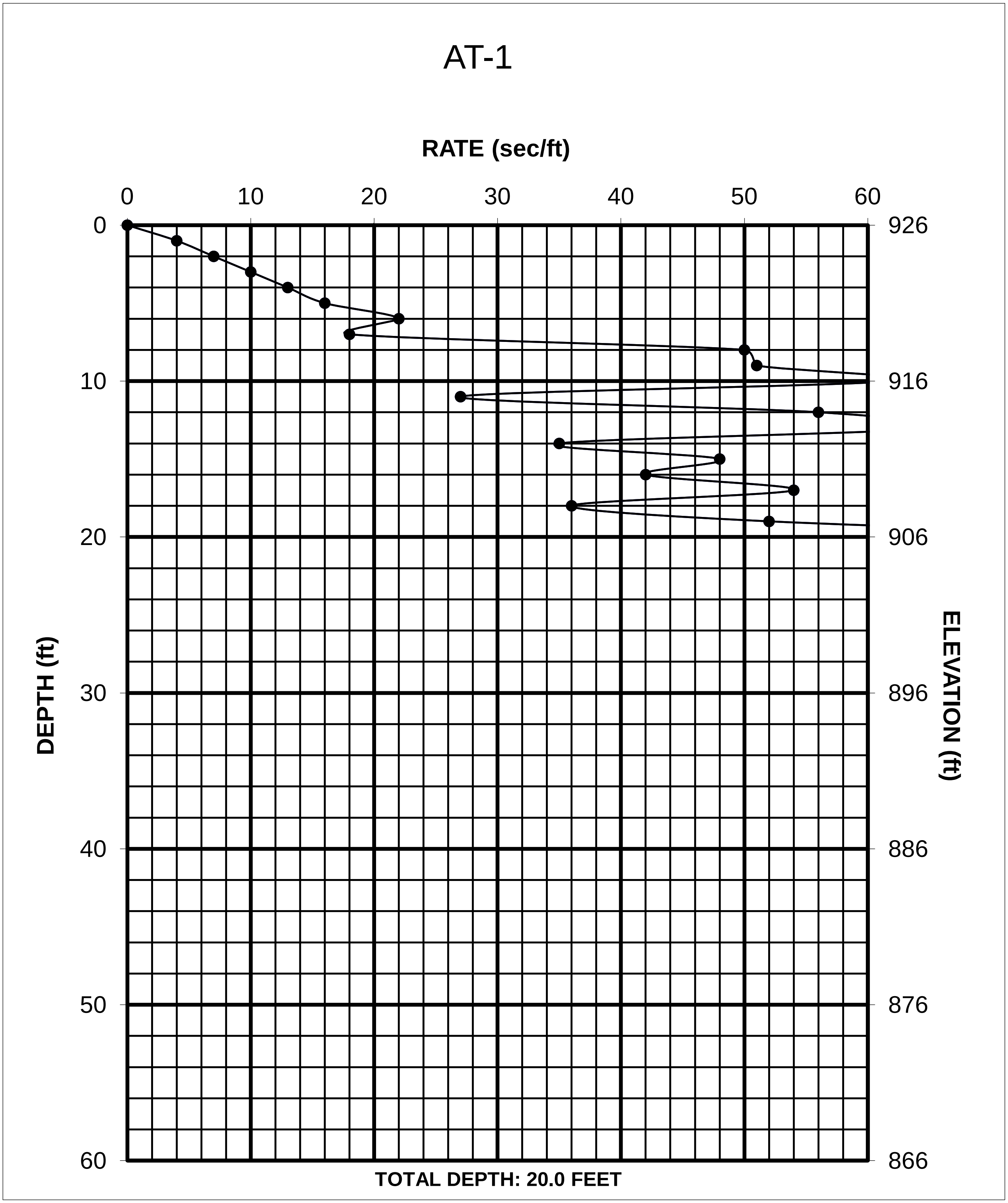
May 23, 2007

TP-43 continued

AIR TRACK BORING

AT-1 THROUGH AT-19

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
MARCH 2012



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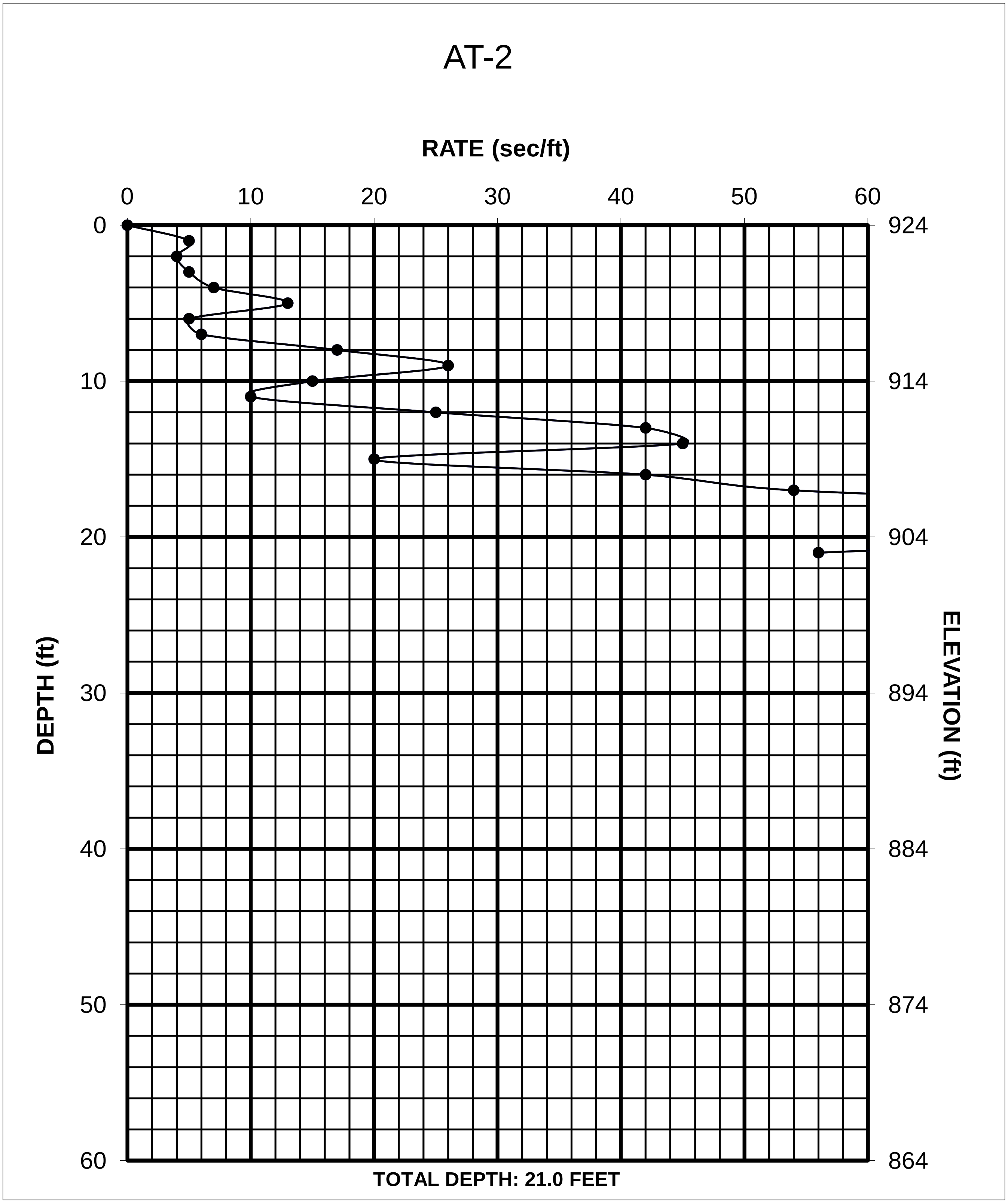
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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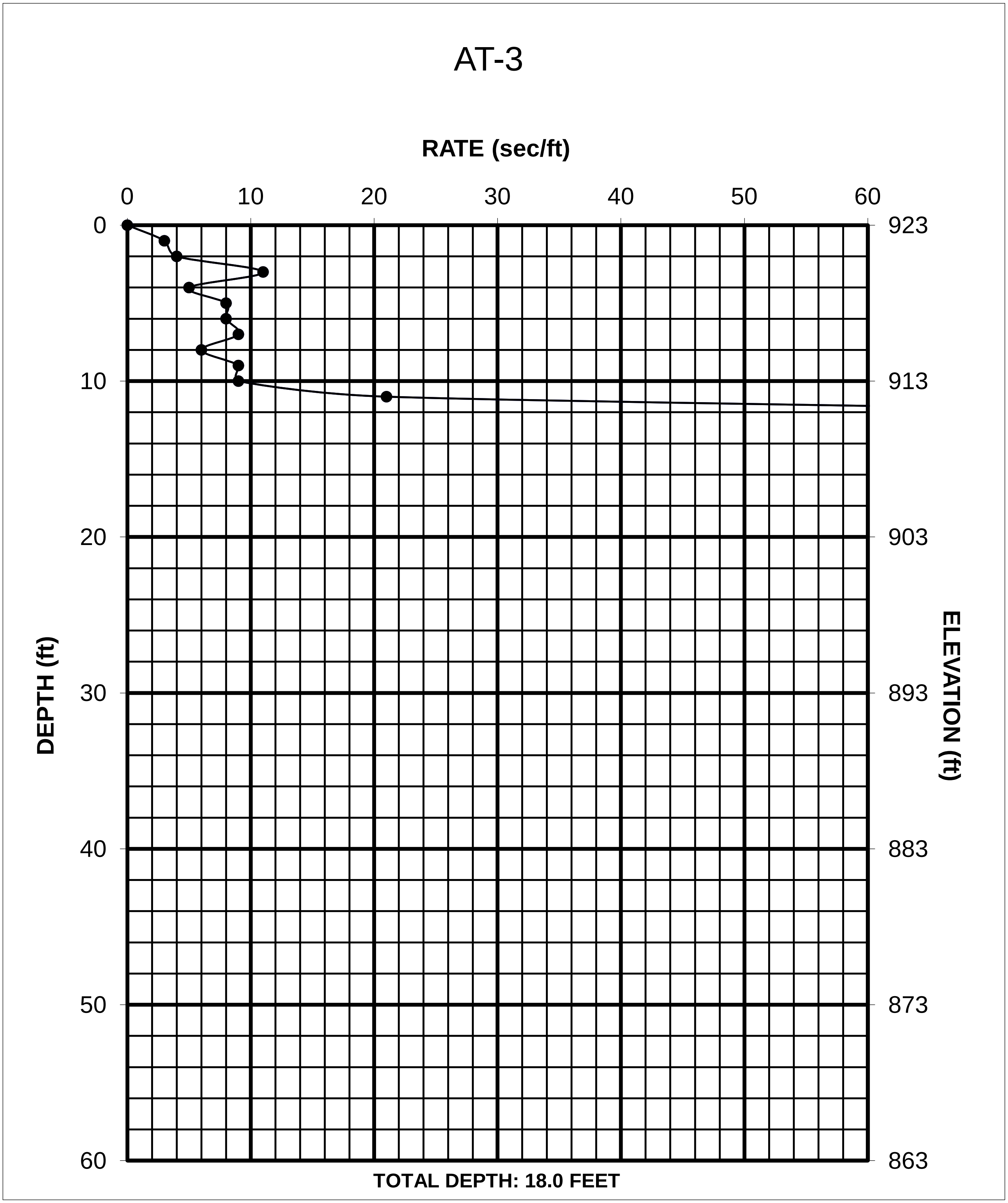

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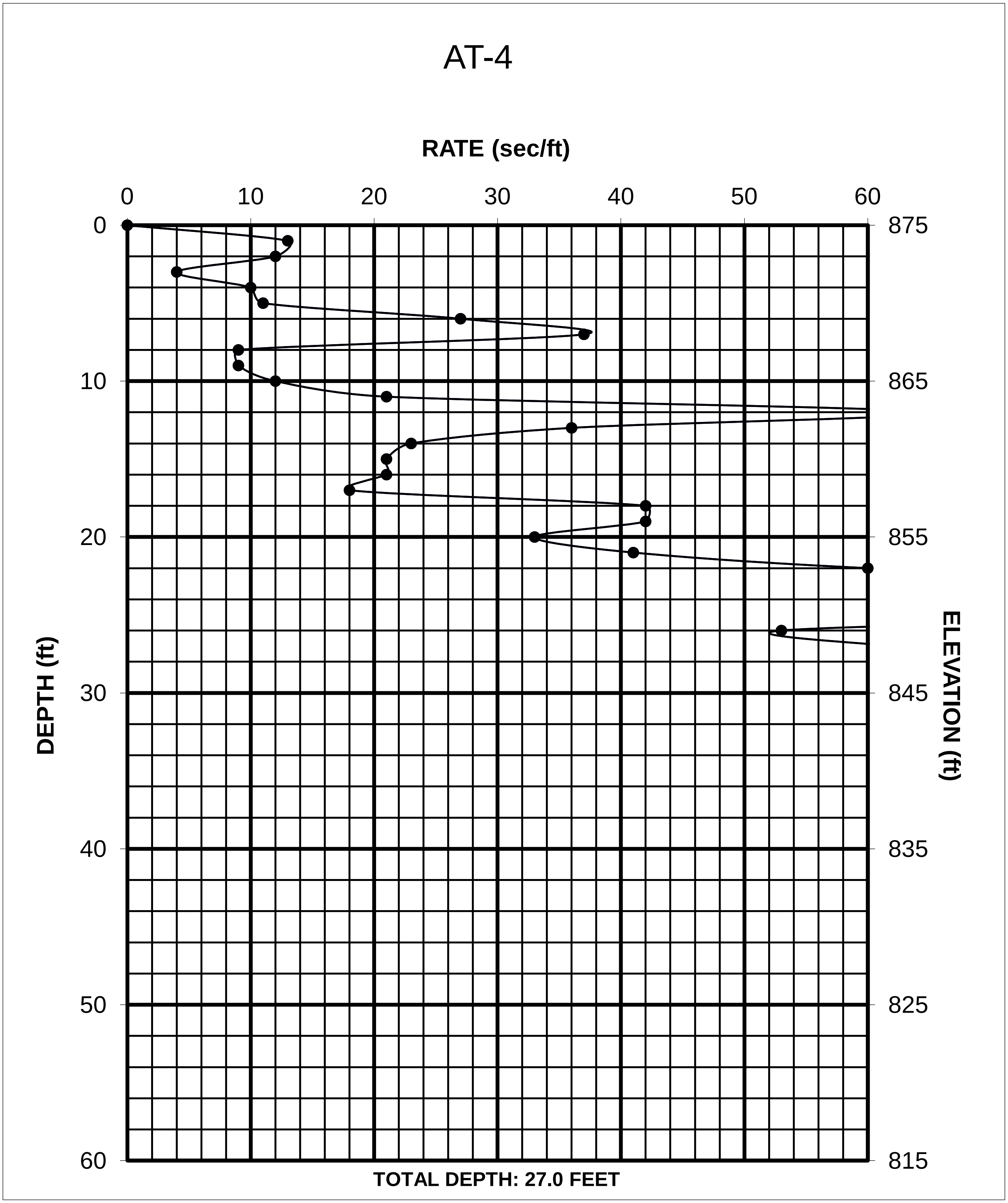

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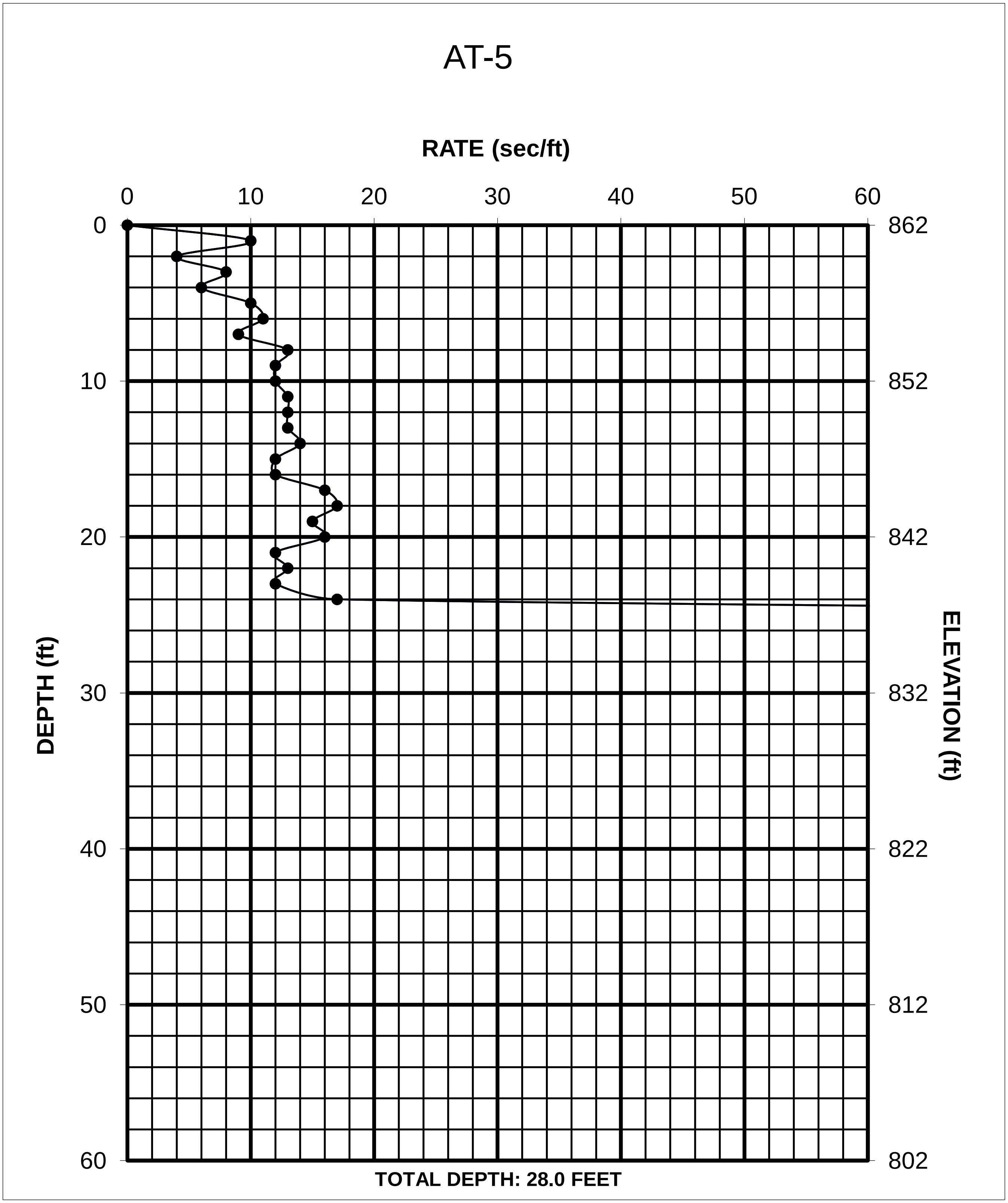

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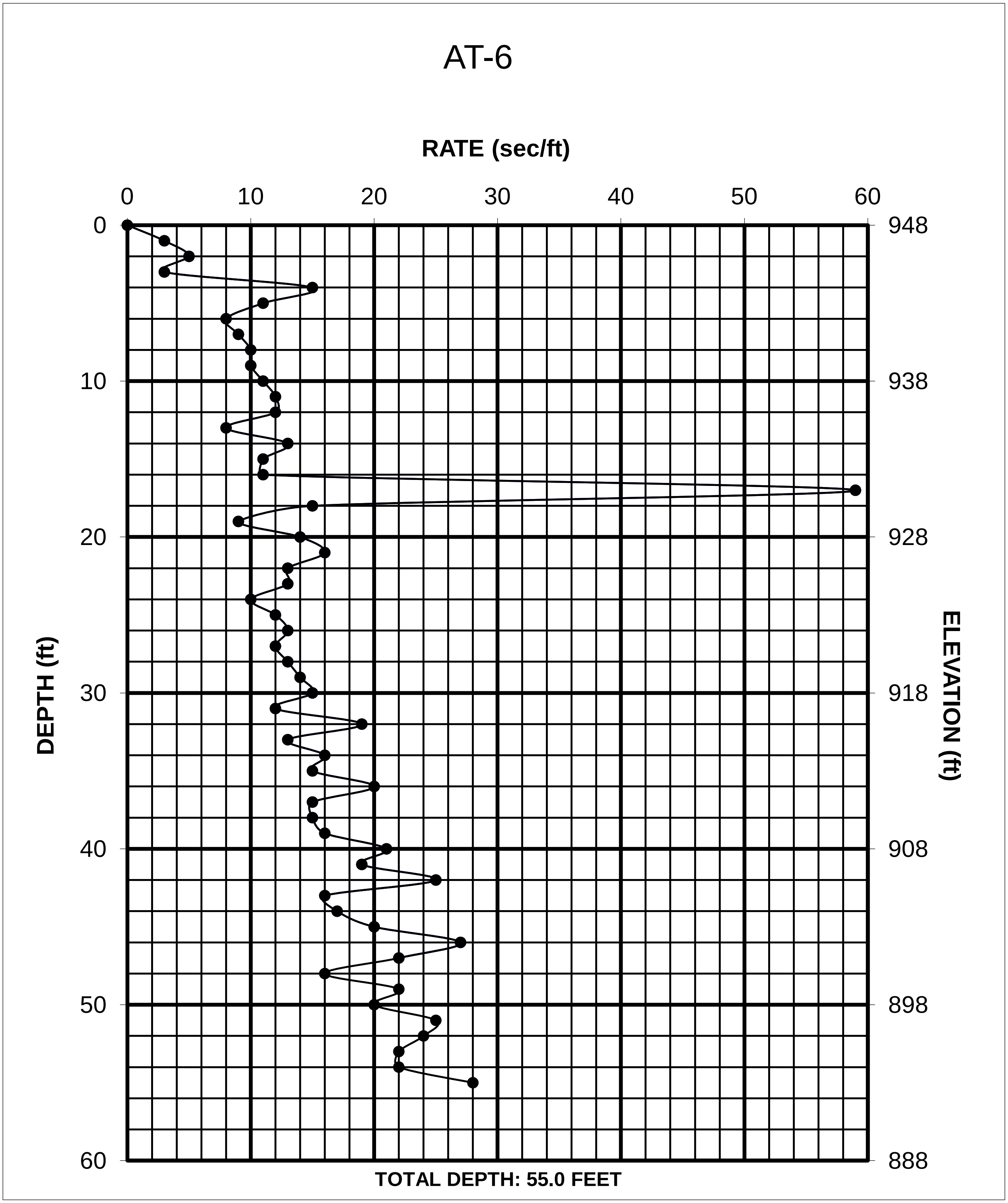

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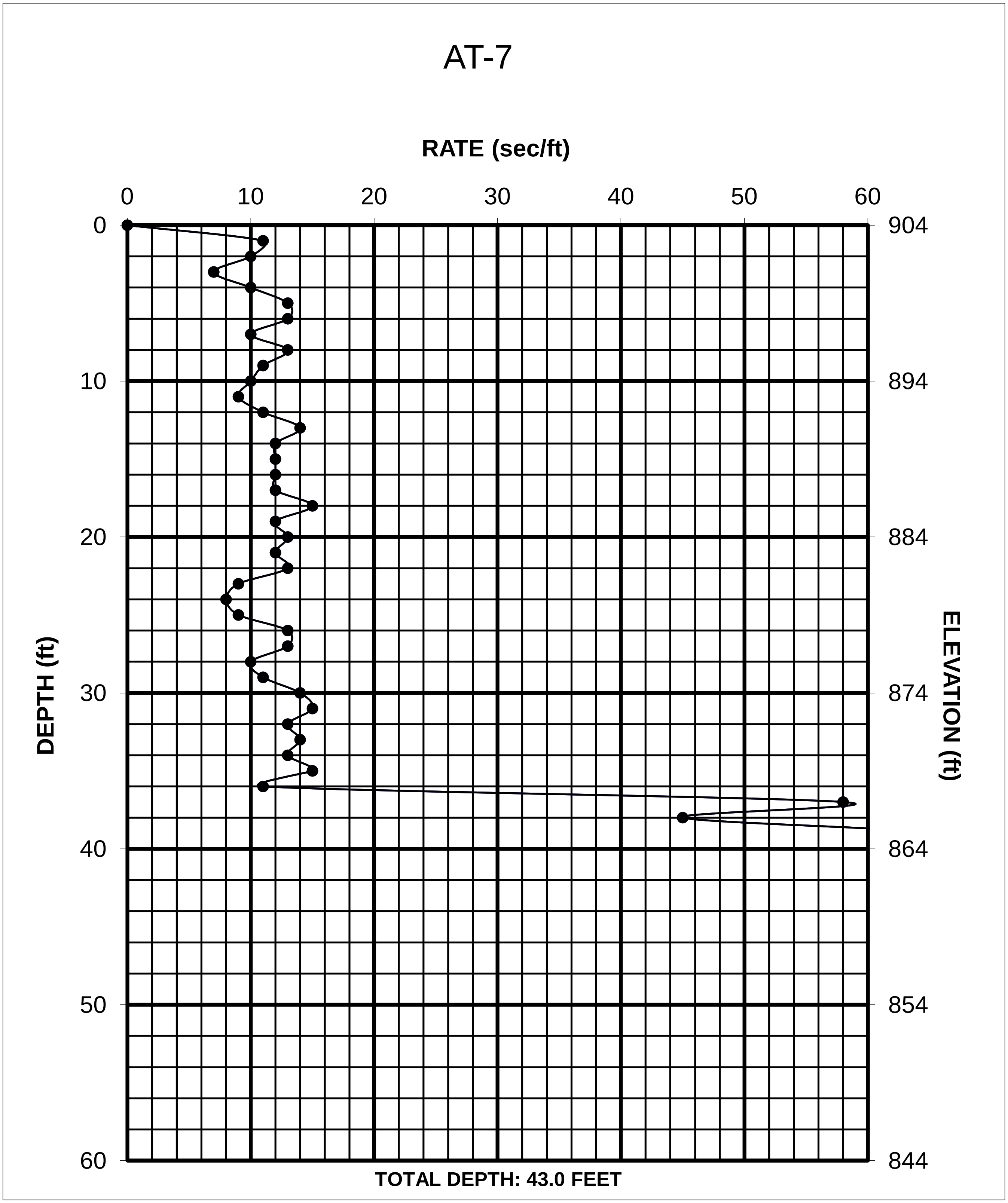
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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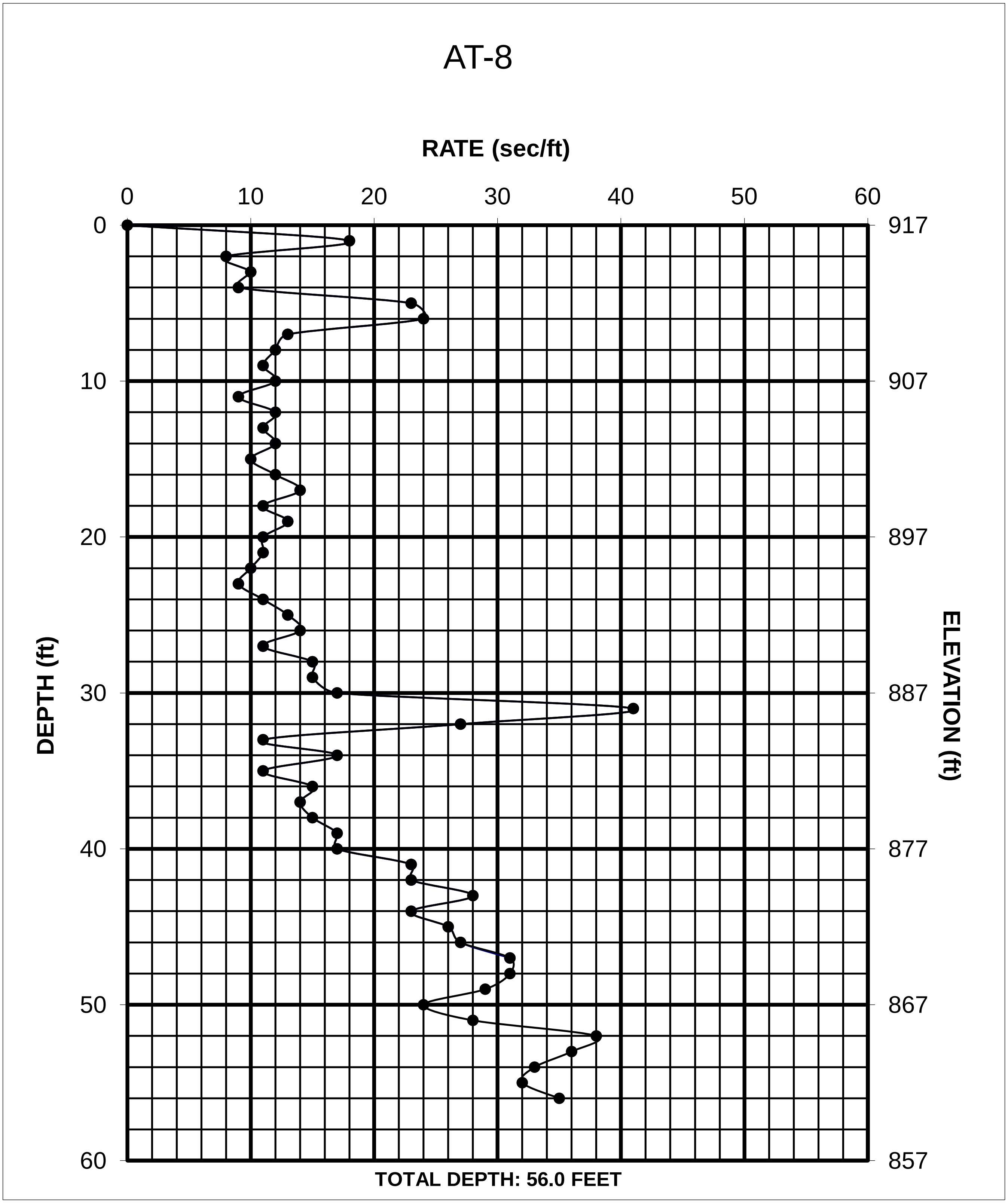

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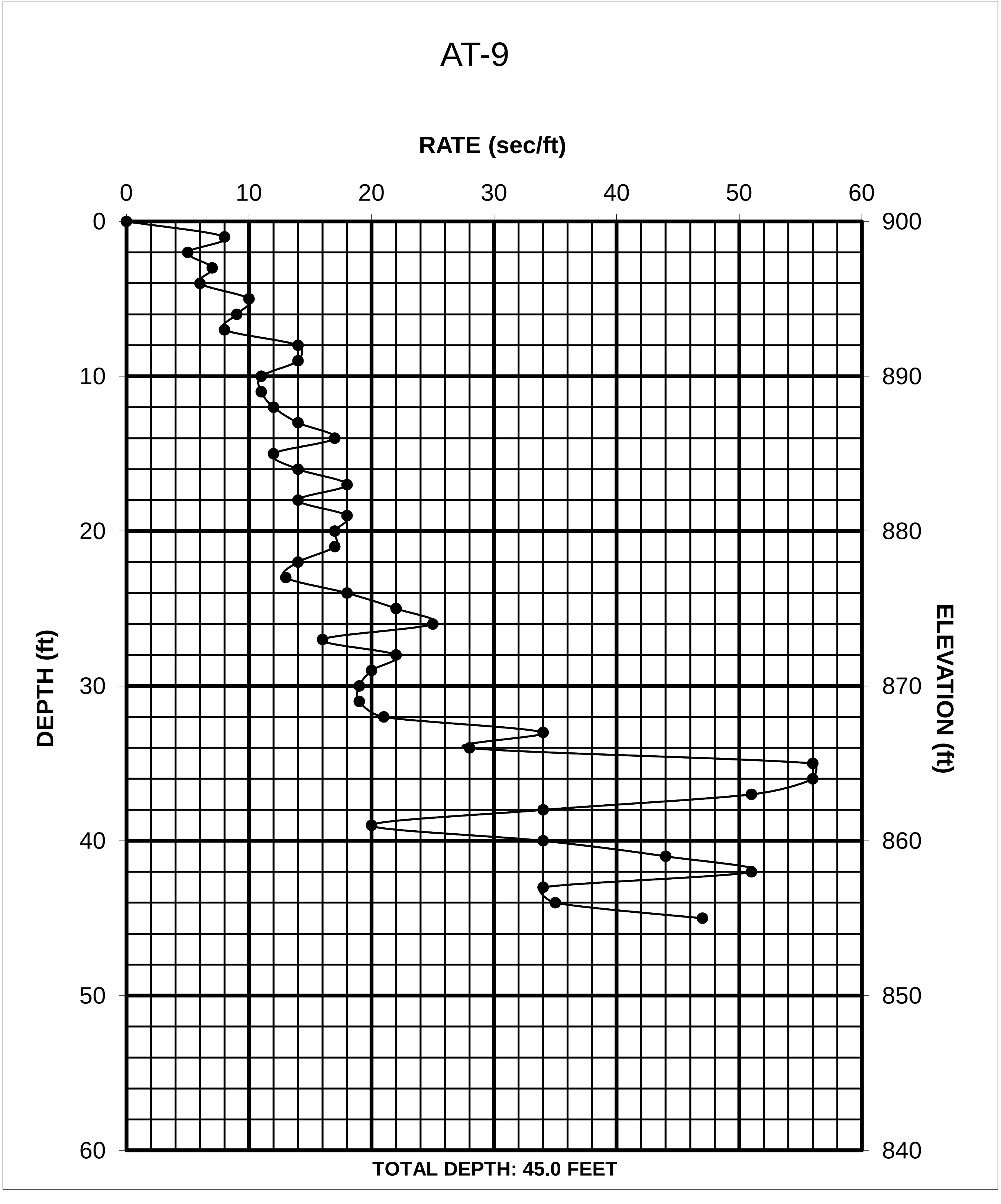

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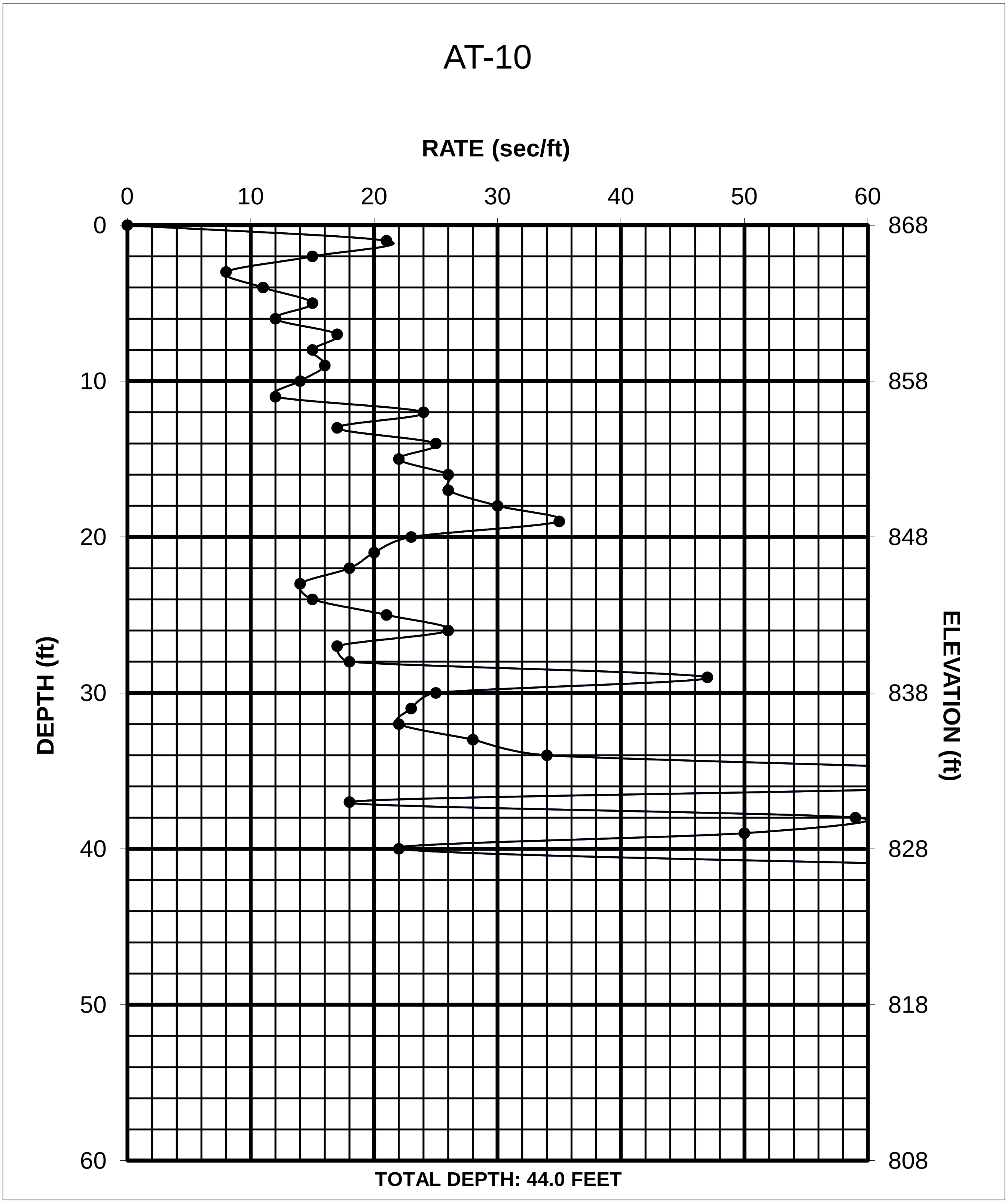
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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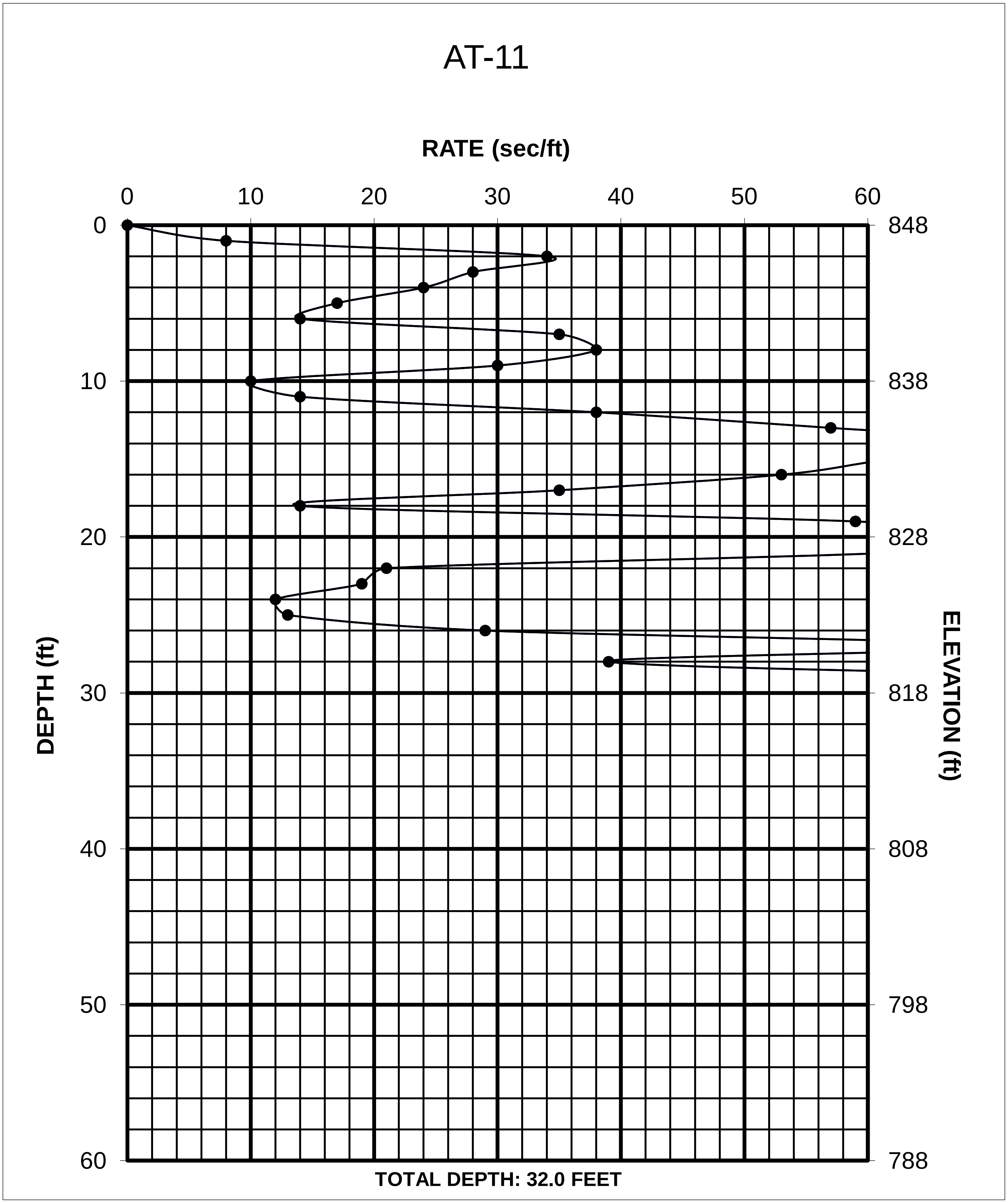
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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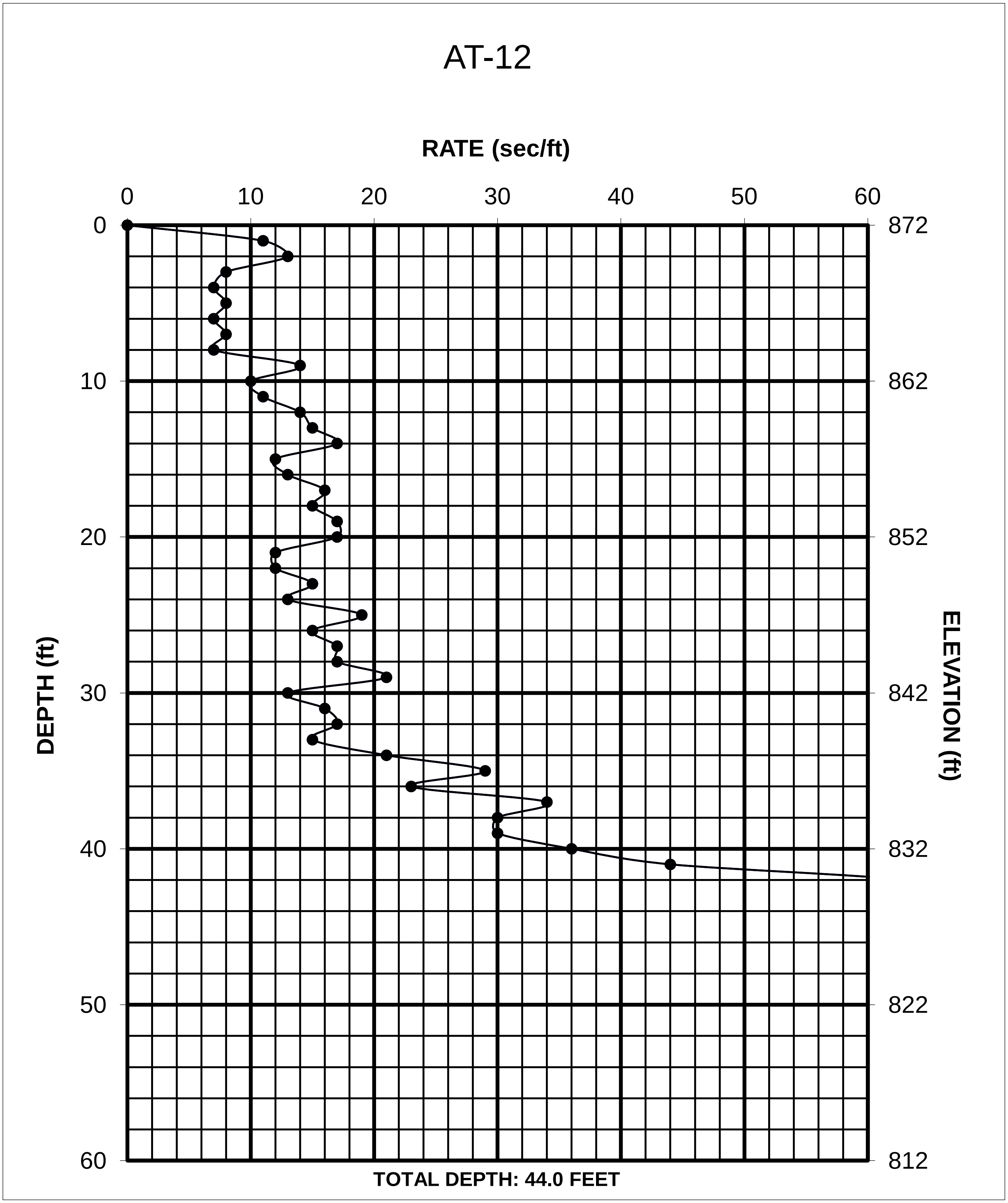
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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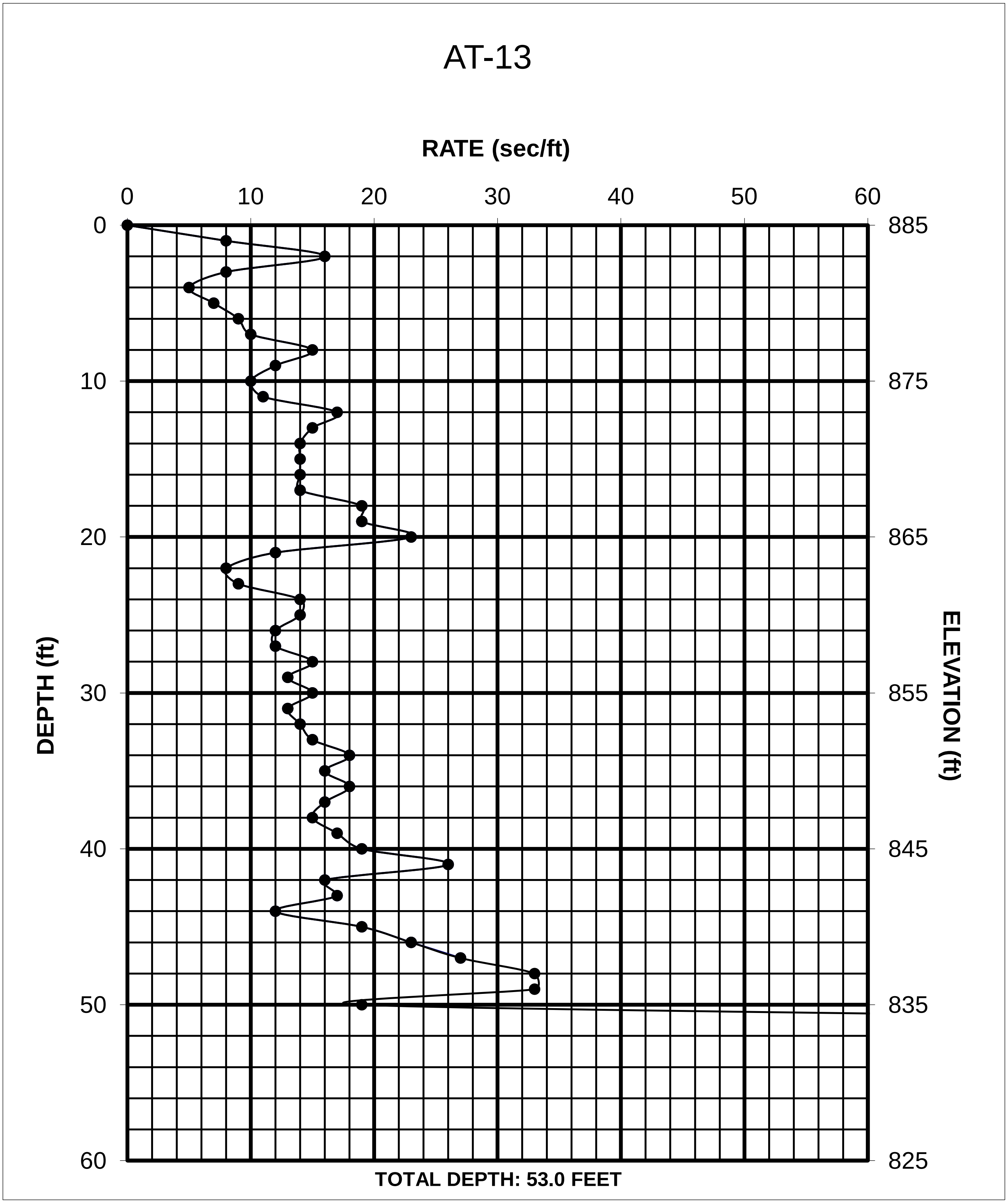
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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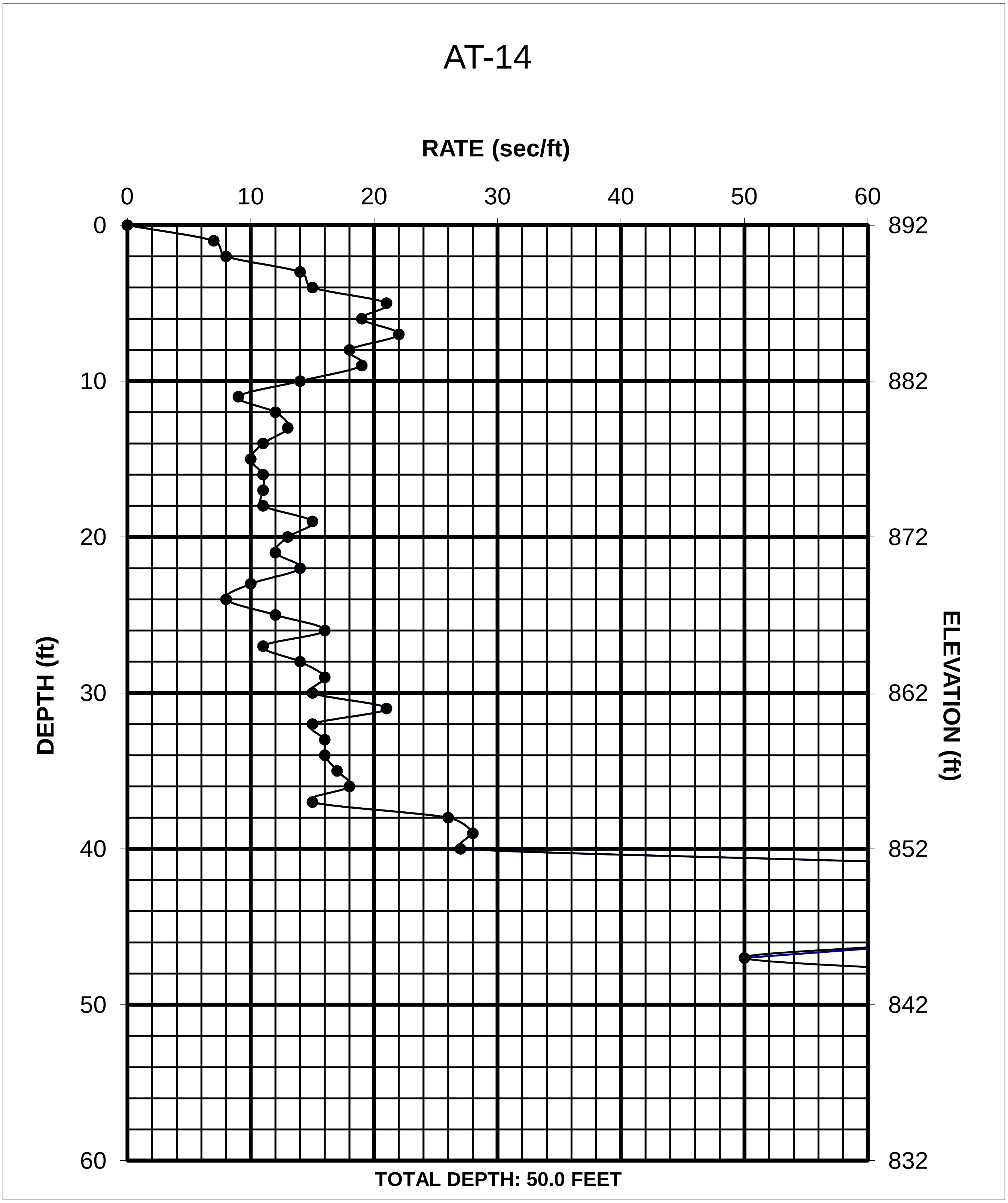

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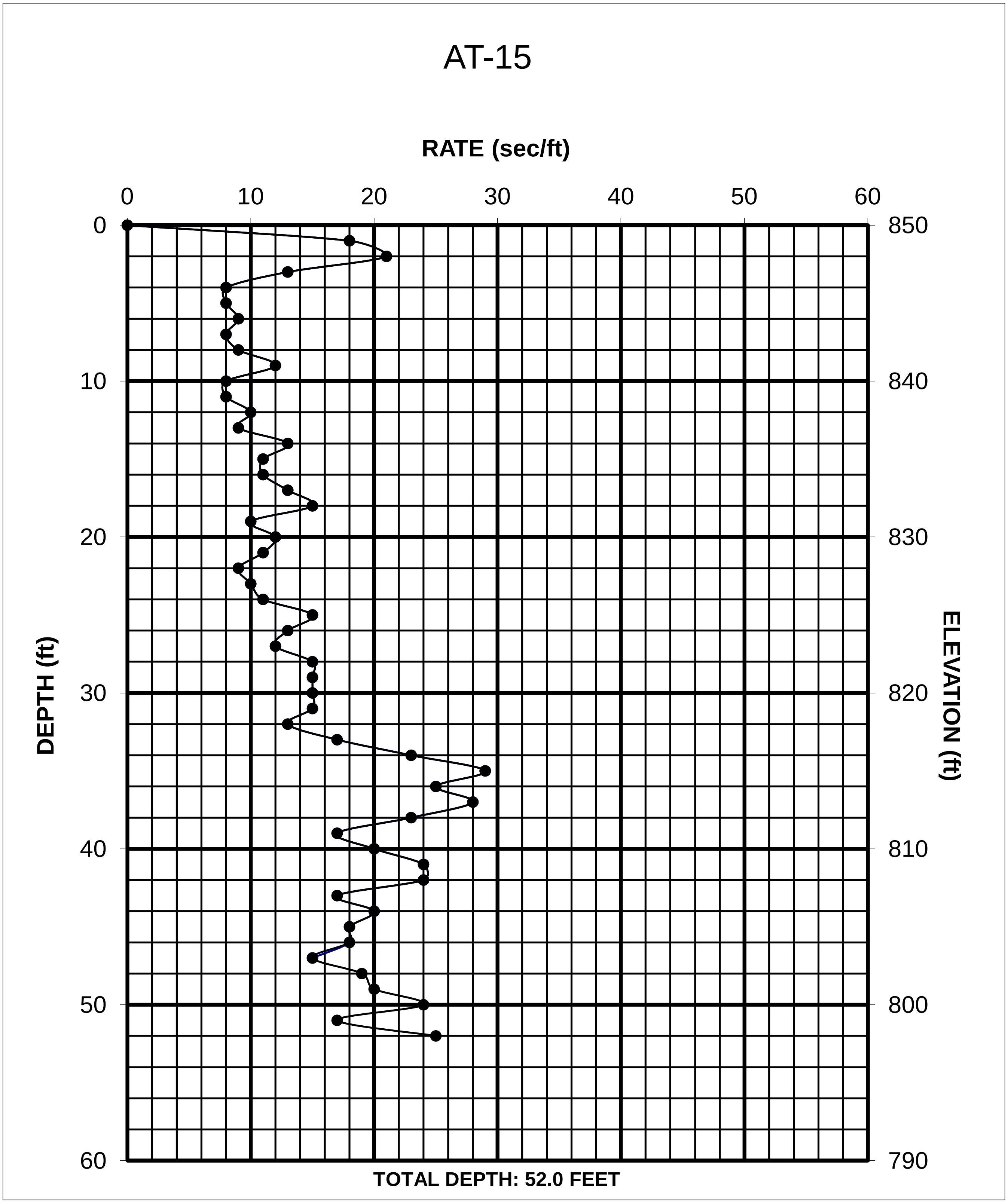
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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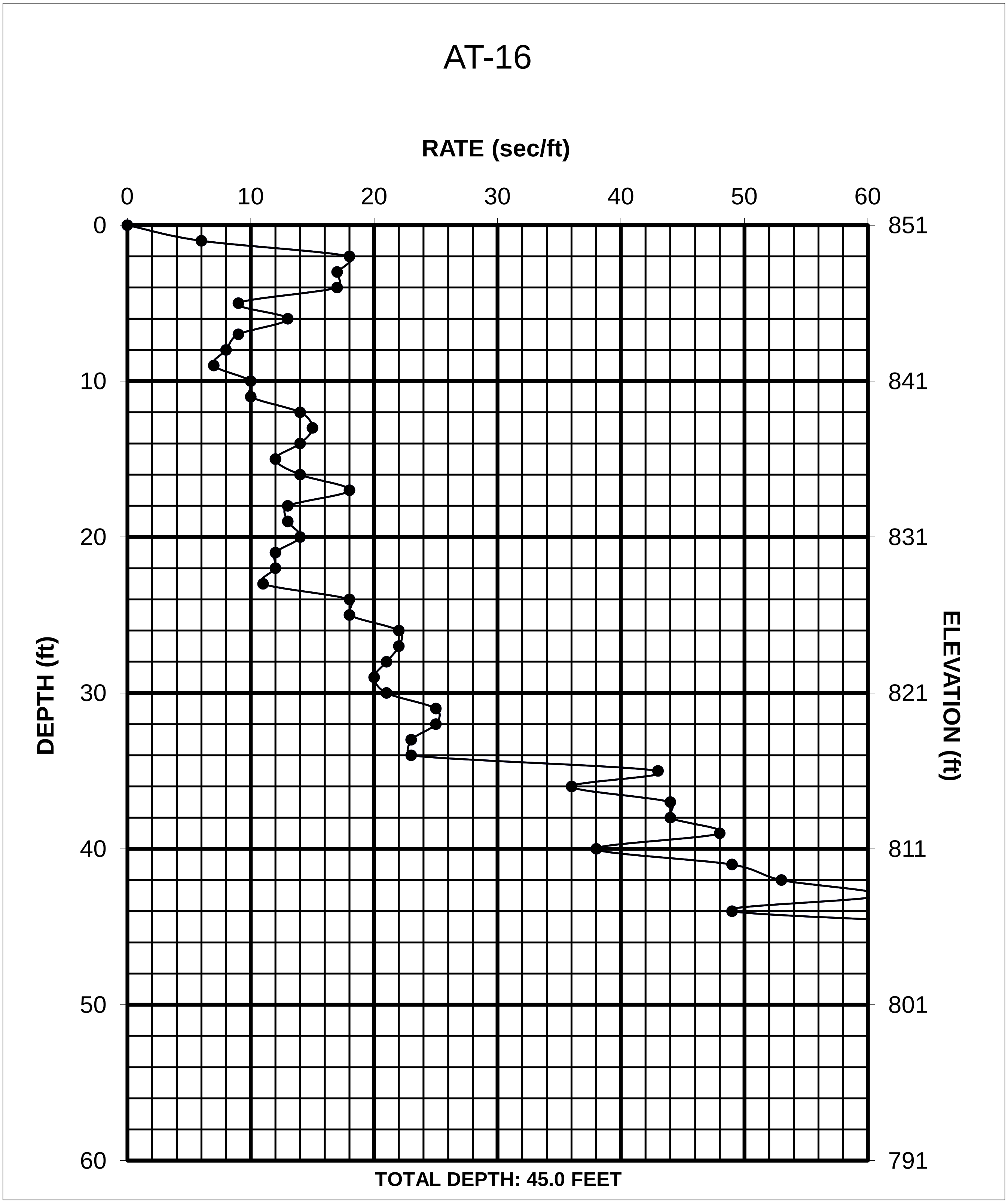
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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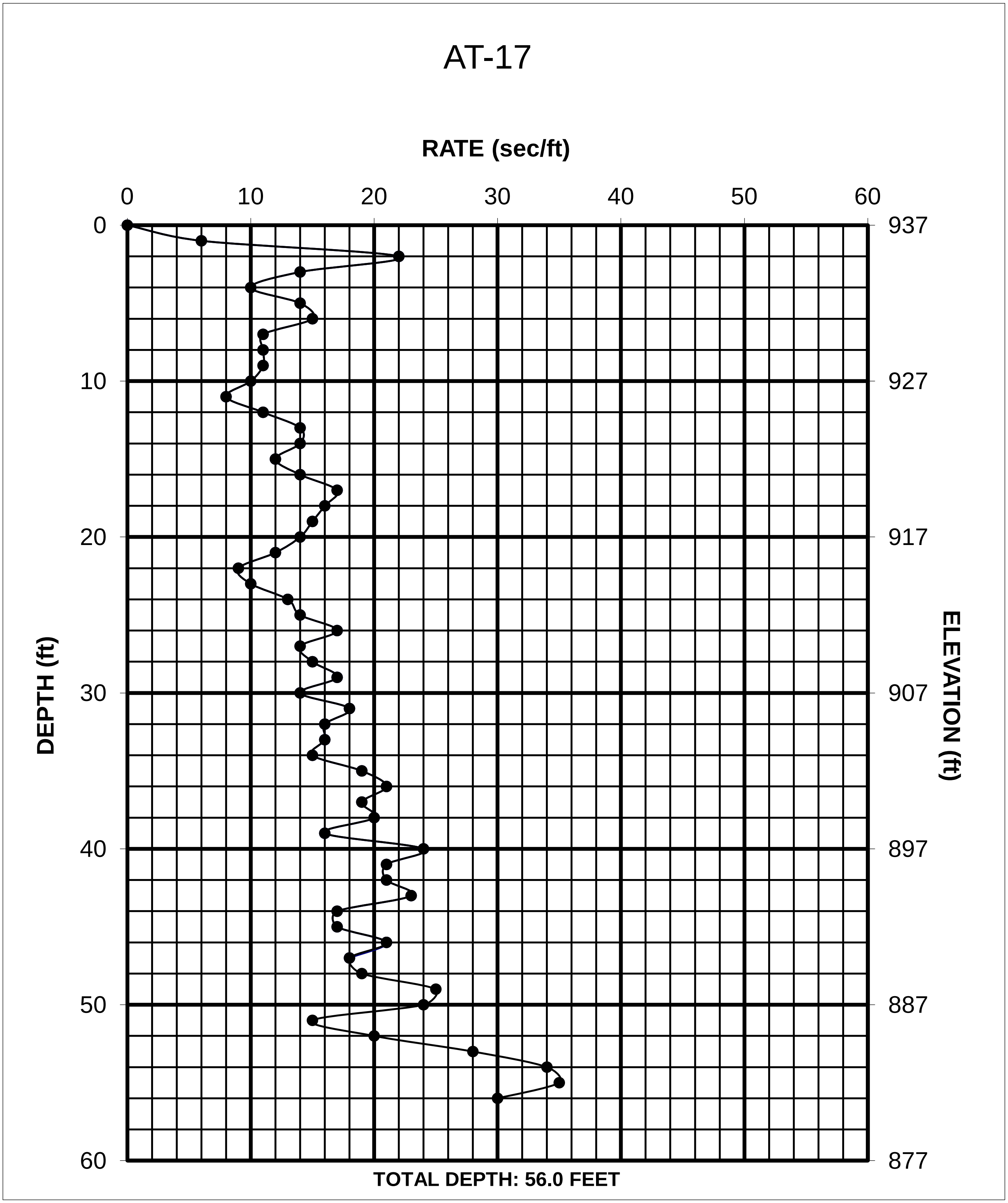
ADVANCED GEOTECHNICAL SOLUTIONS, INC.



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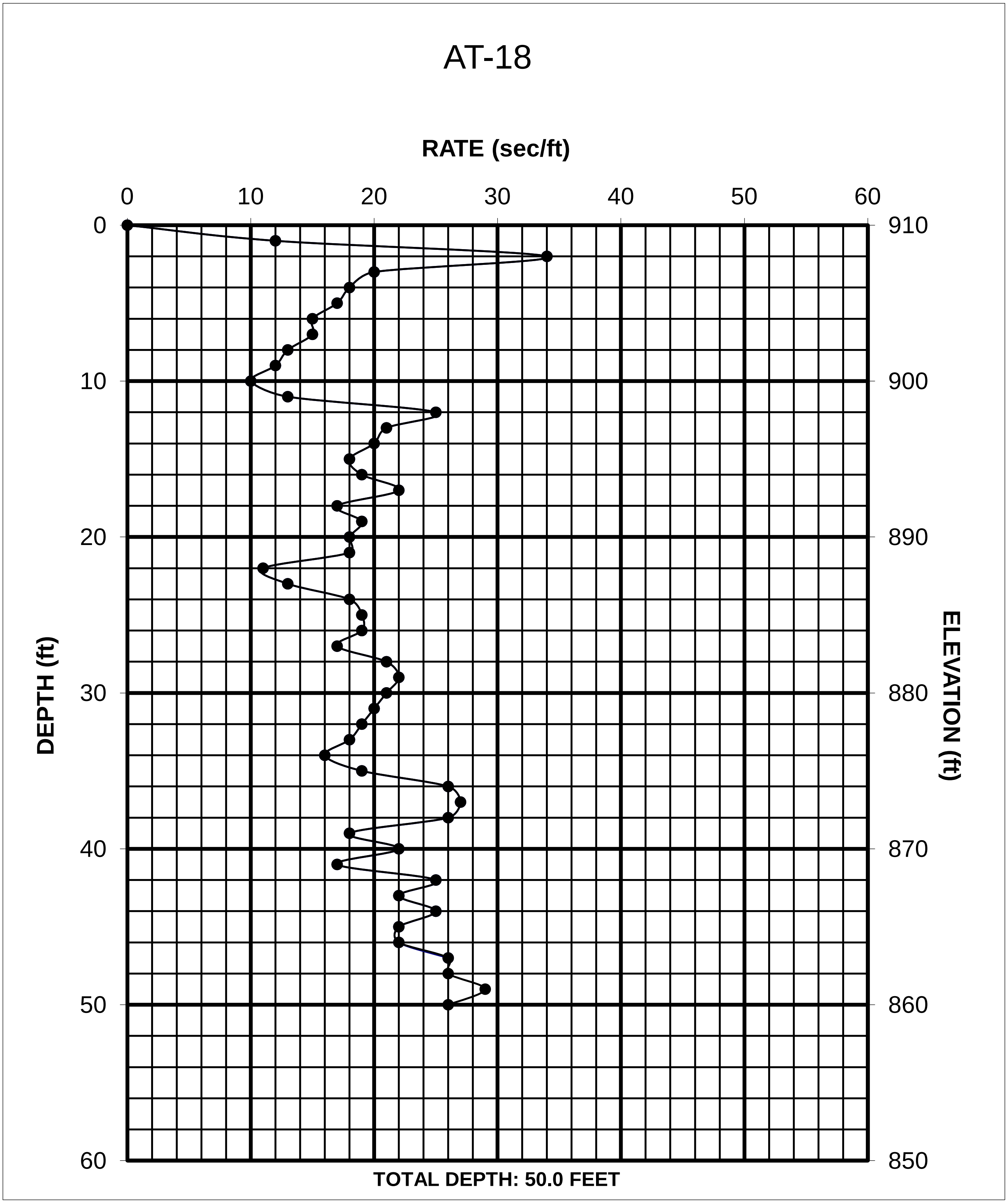

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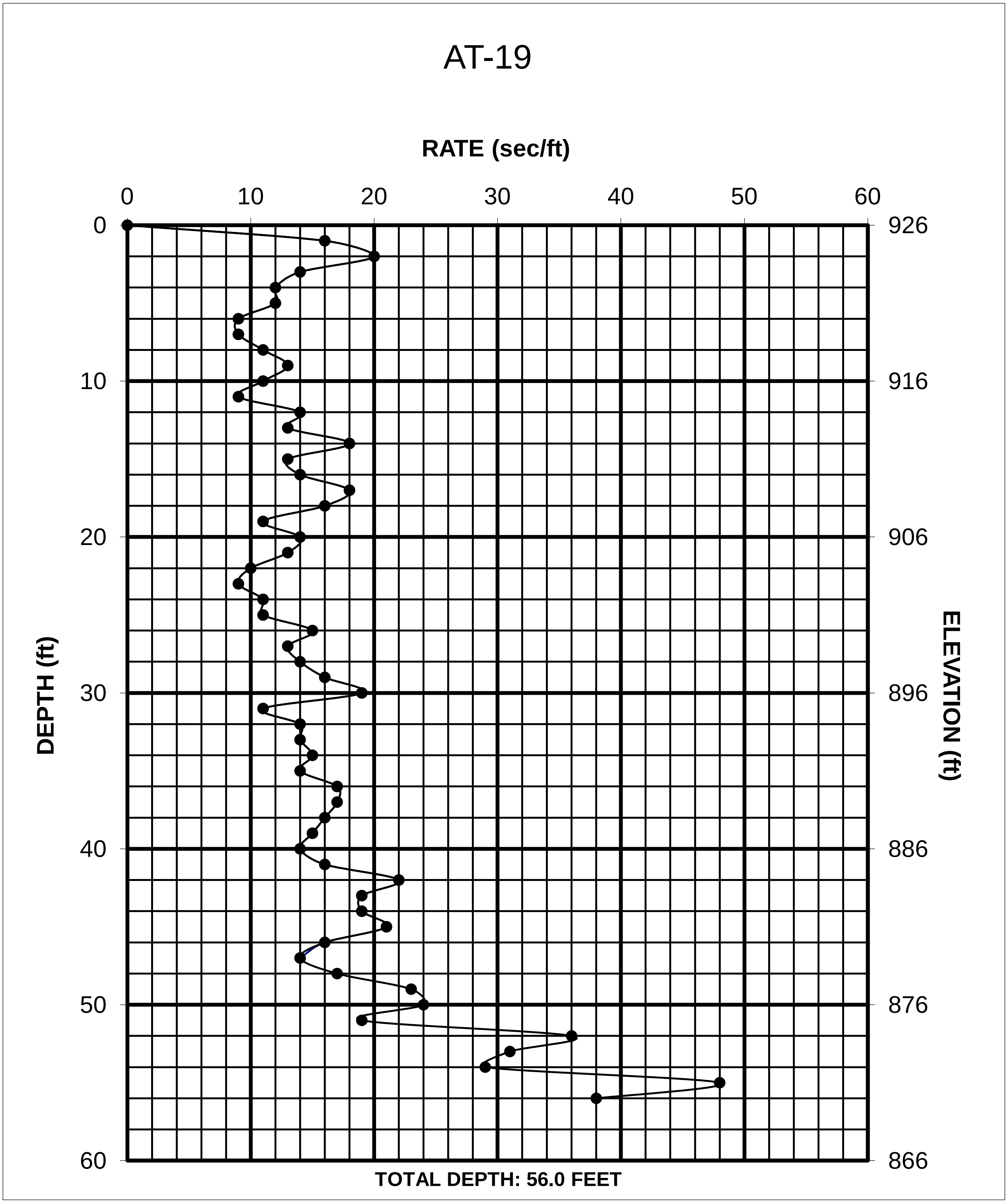
BIT SIZE: 4"

ADVANCED GEOTECHNICAL SOLUTIONS, INC.



EQUIPMENT: INGERSOLL RAND ECM-370

BIT SIZE: 4"

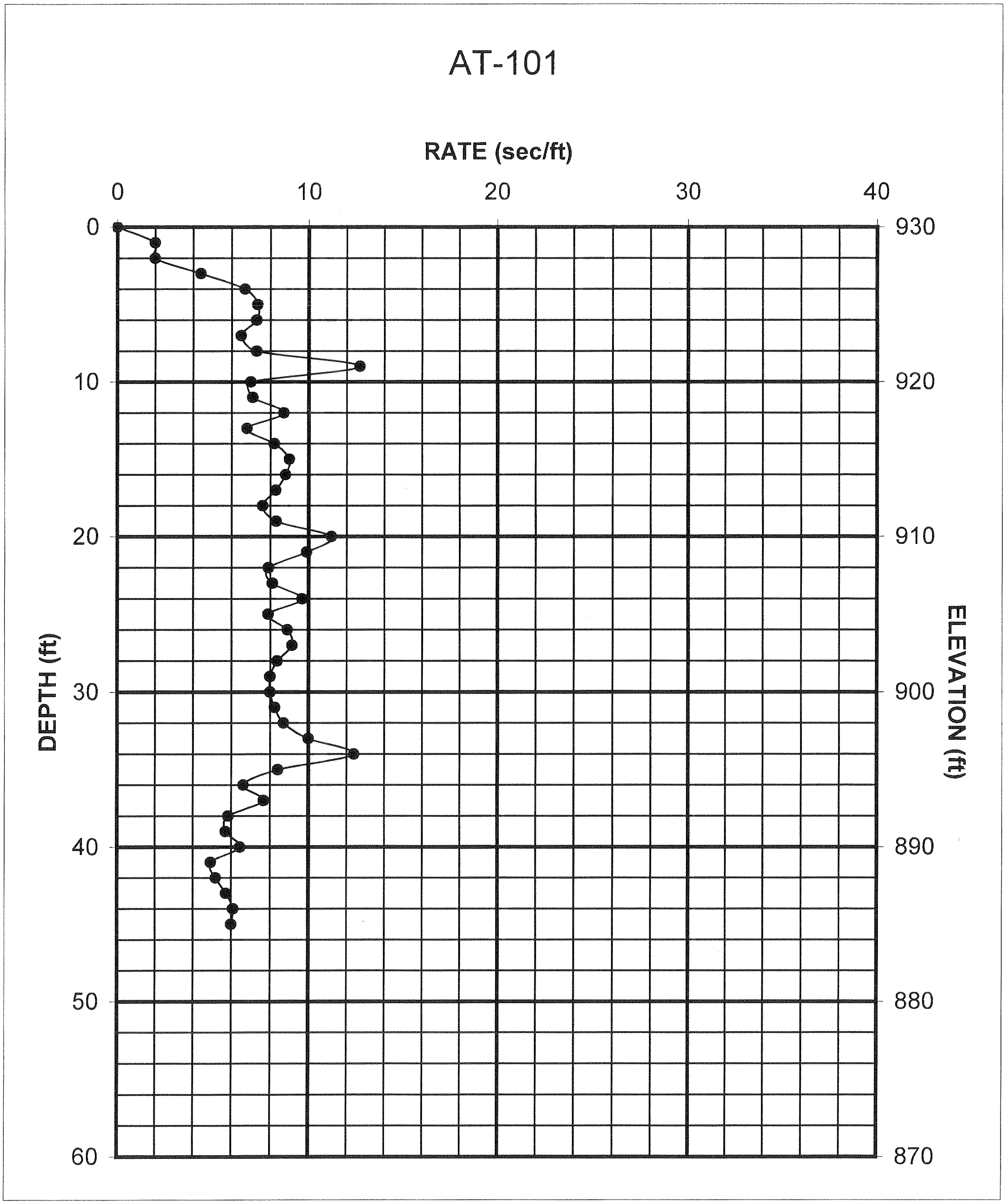
ADVANCED GEOTECHNICAL SOLUTIONS, INC.

AIR TRACK BORINGS

AT-101 THROUGH AT-121

PACIFIC SOILS ENGINEERING, INC.
MAY 2007

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

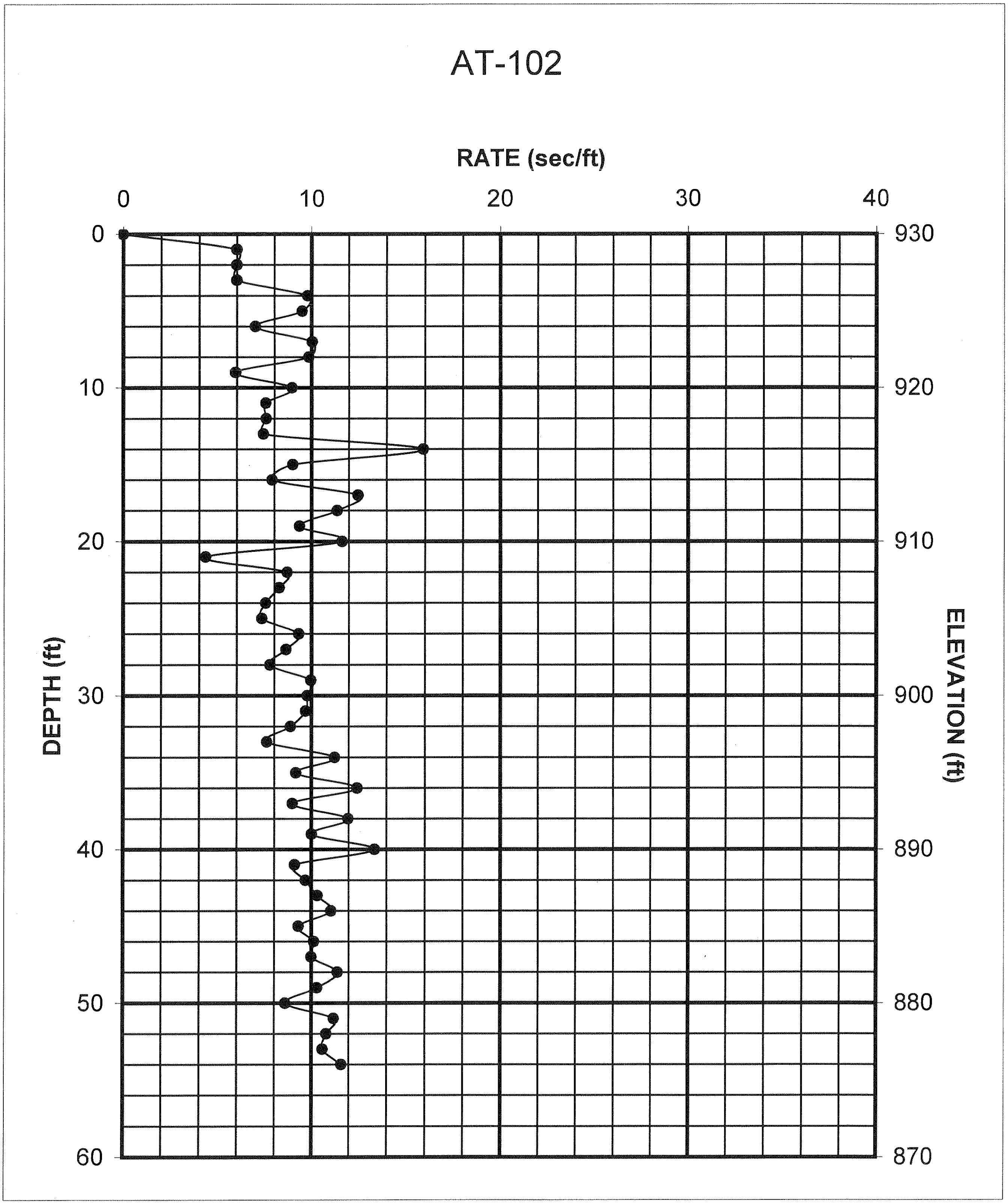


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 1

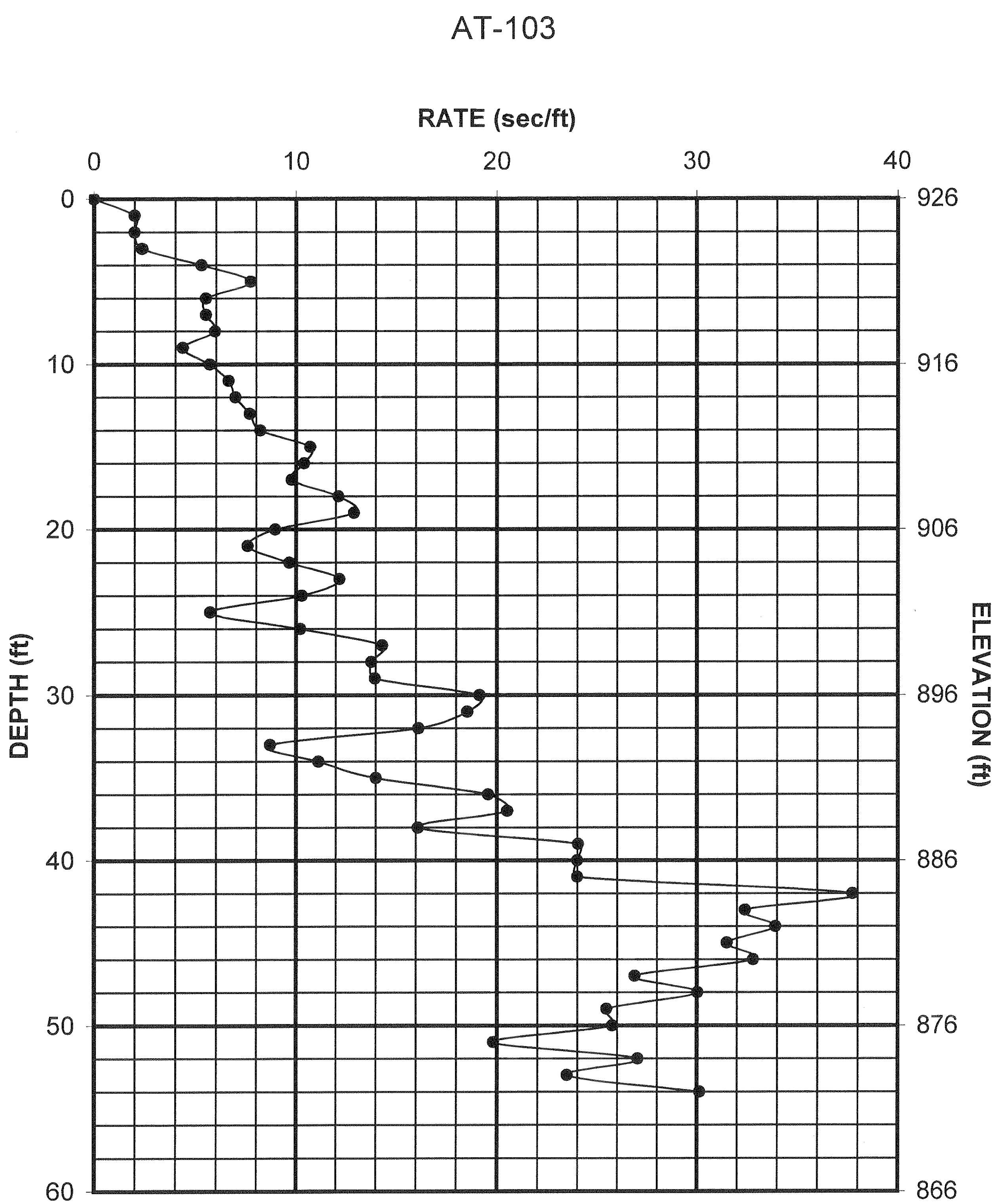


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 2

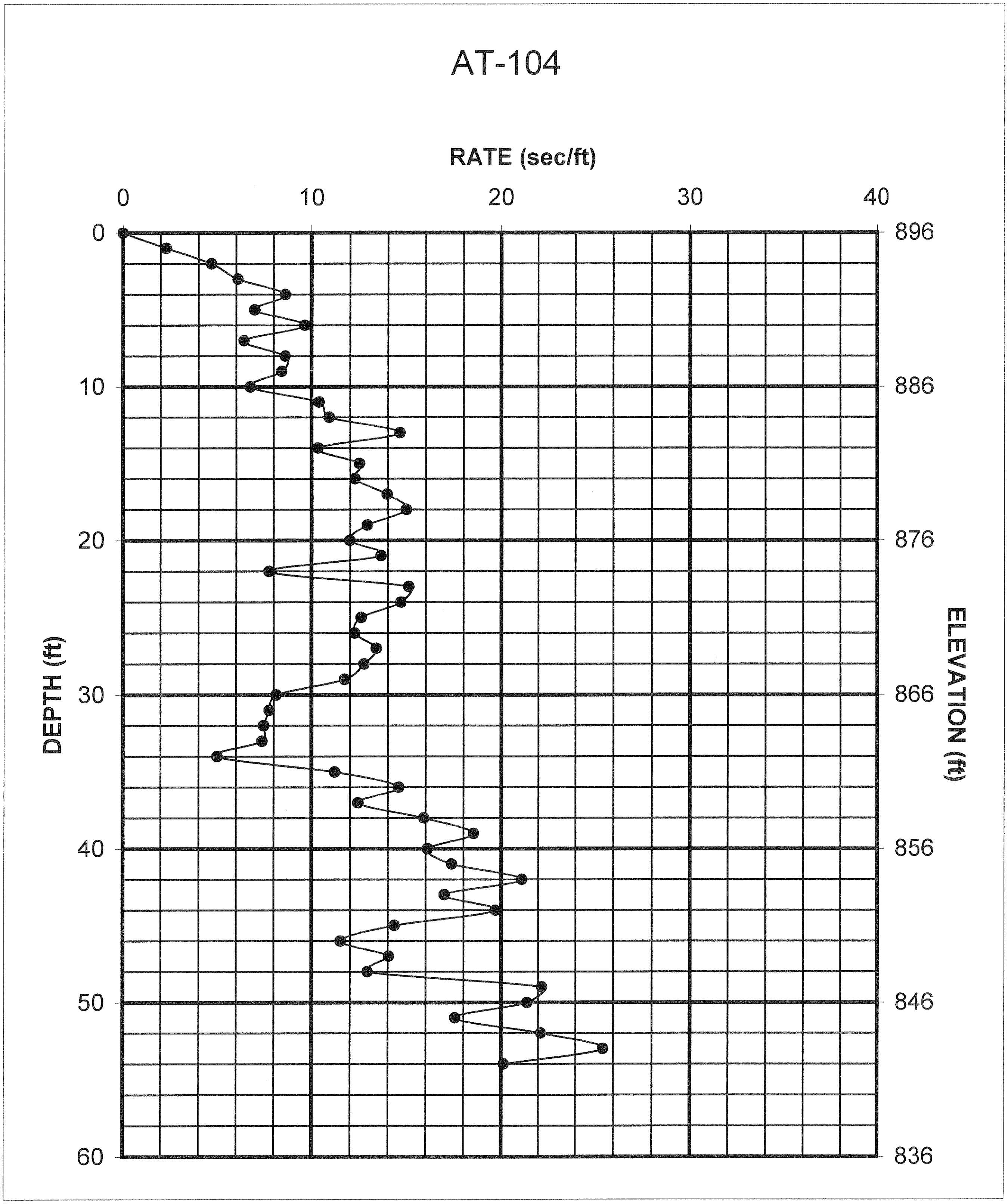


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 3

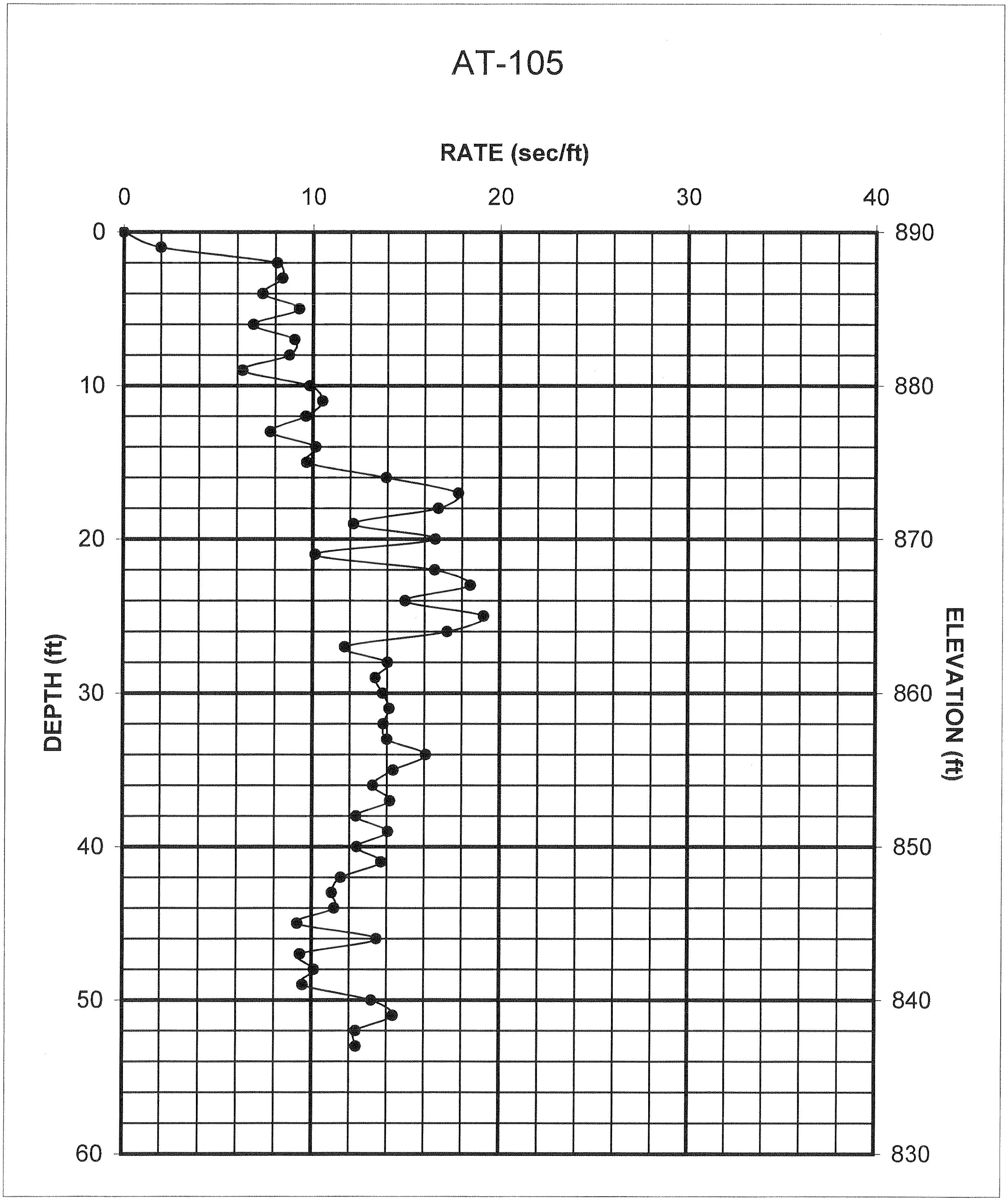


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 4

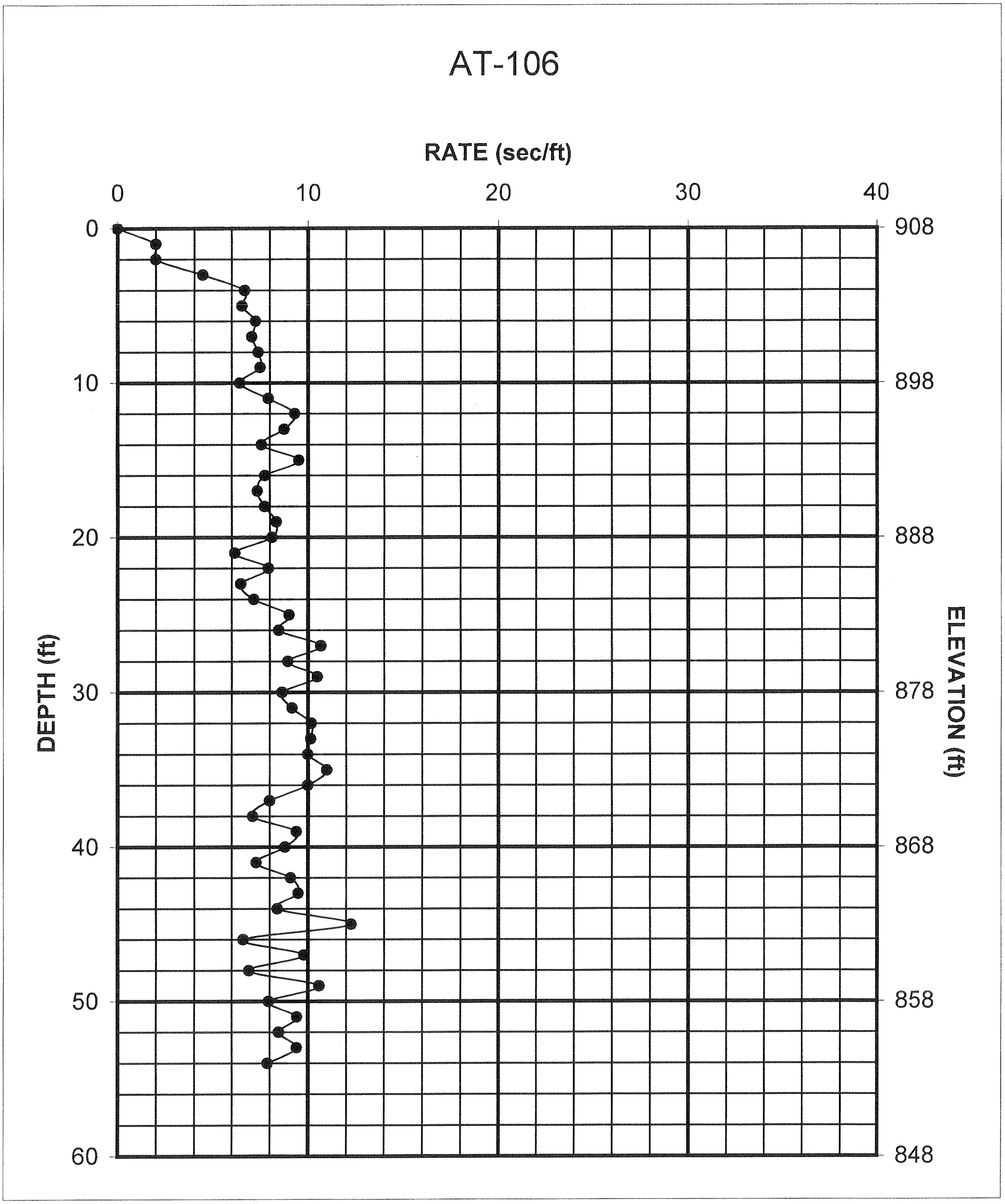


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 5

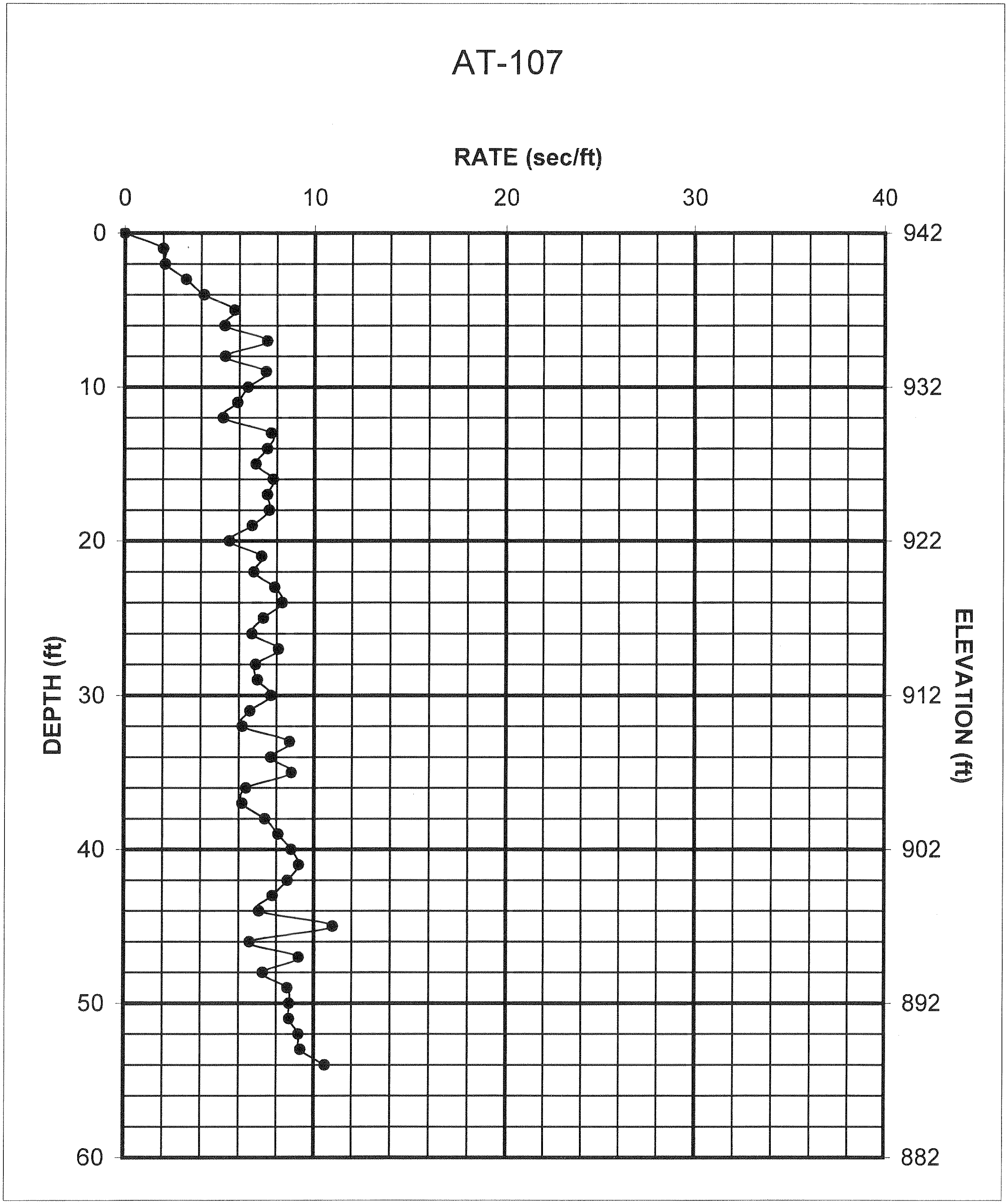


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 6

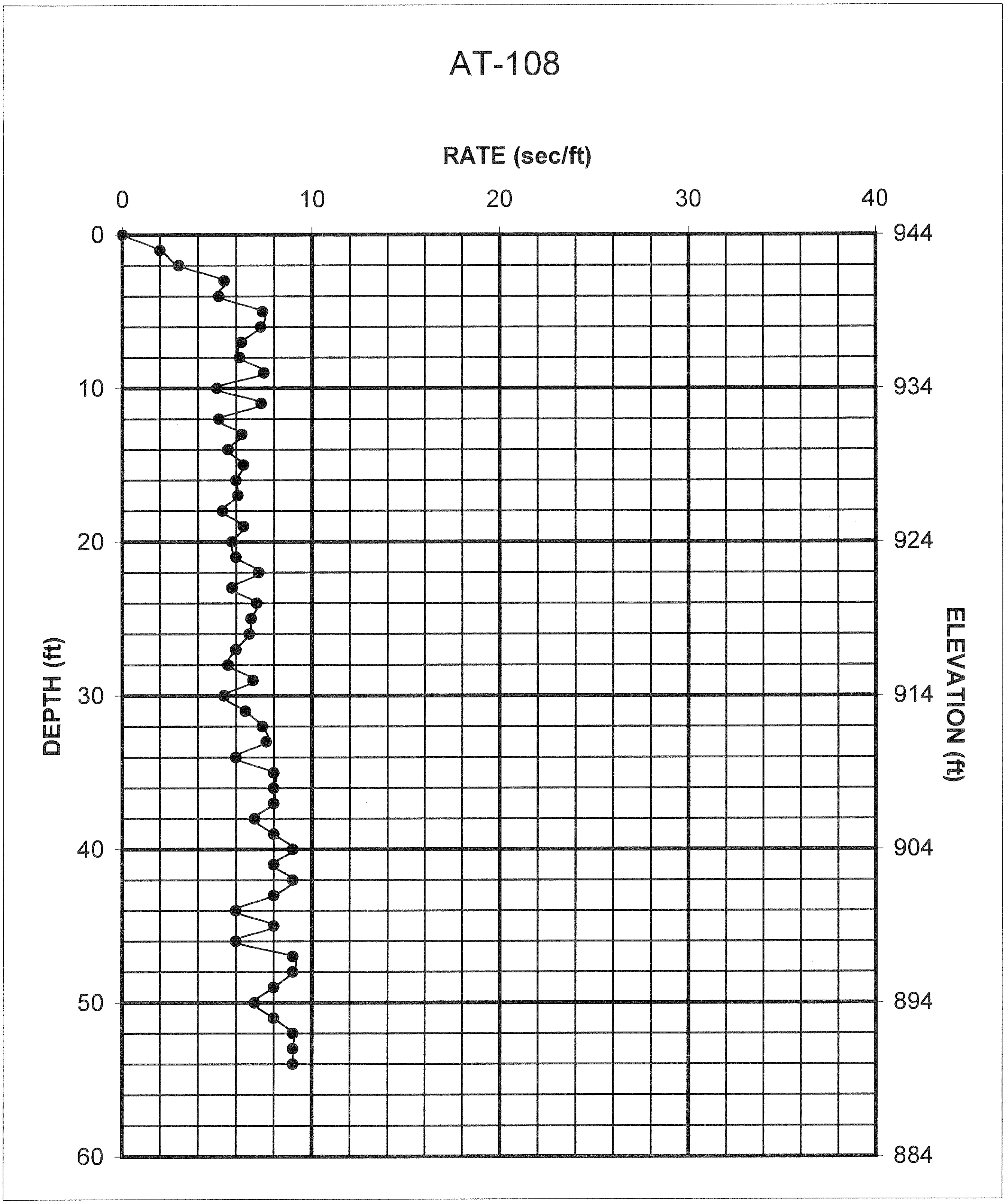


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 7

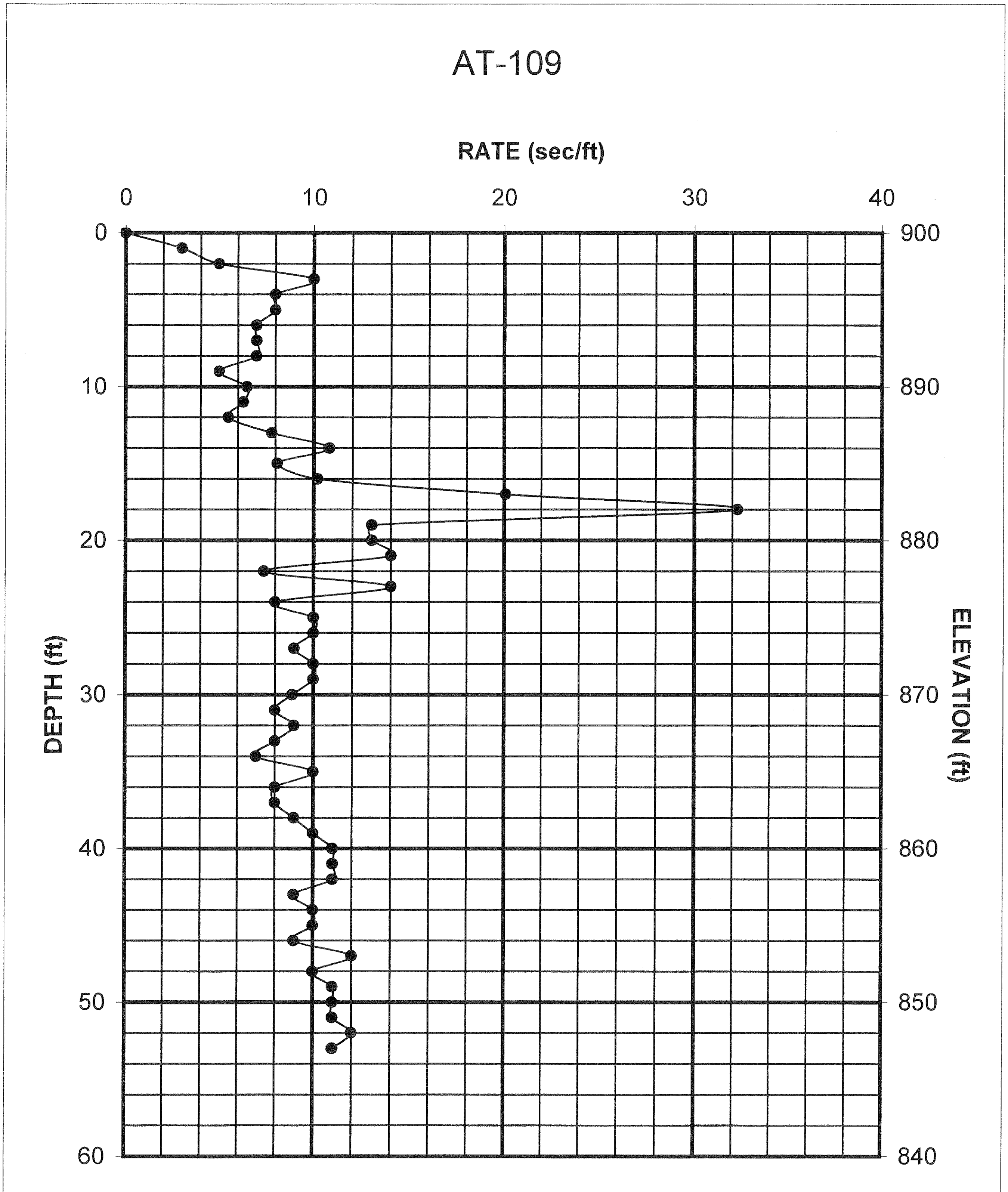


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 8

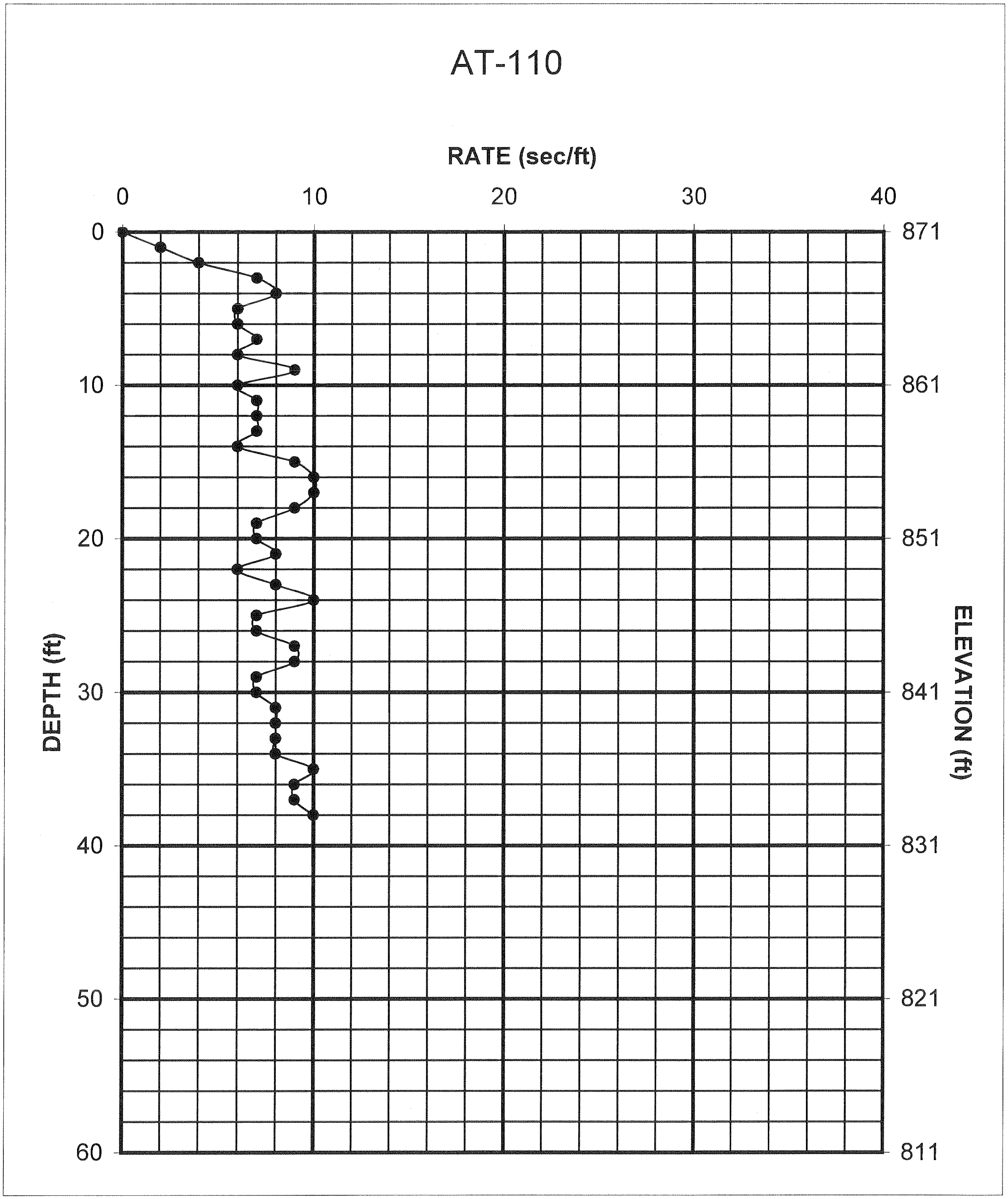


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 9

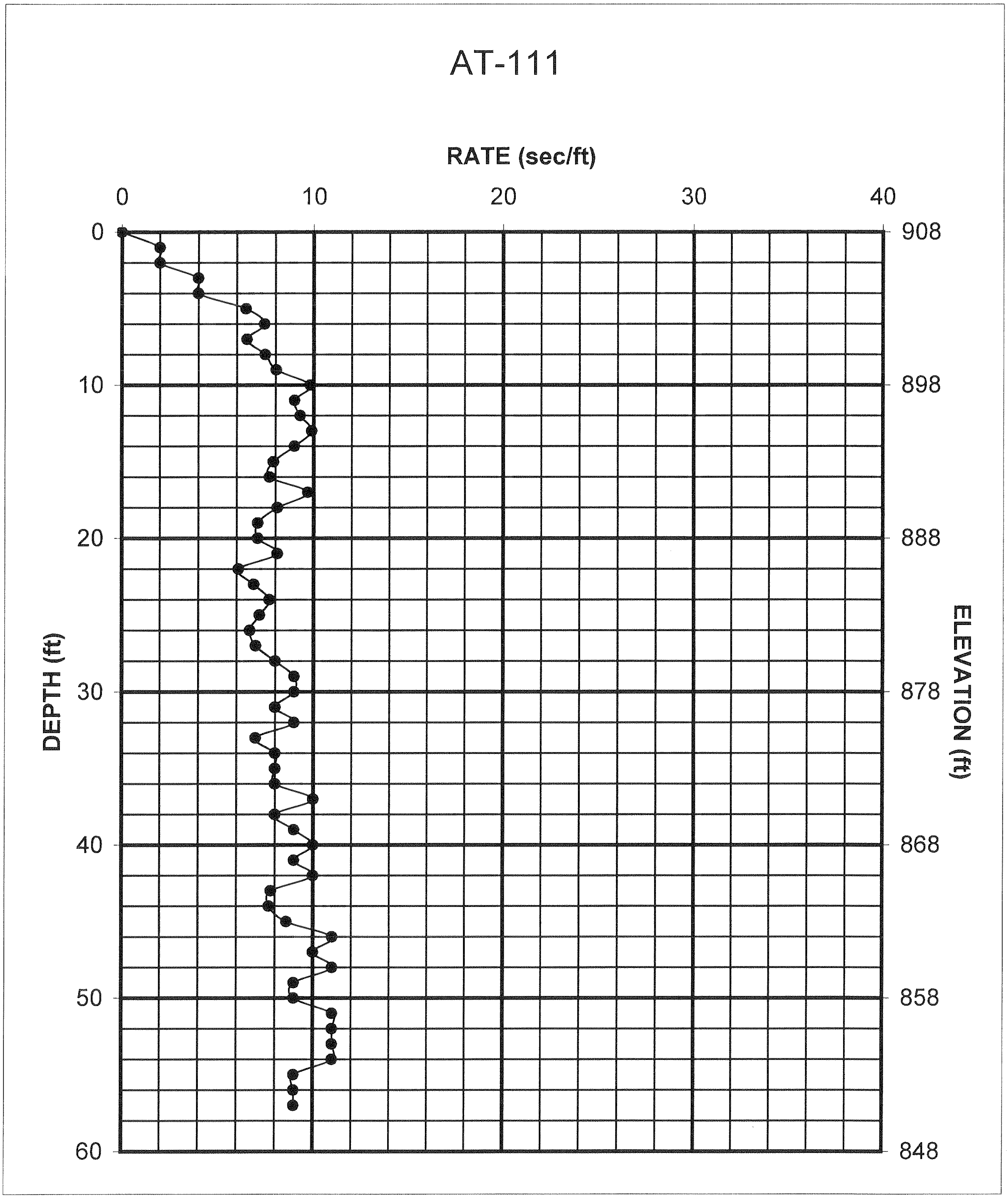


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 10

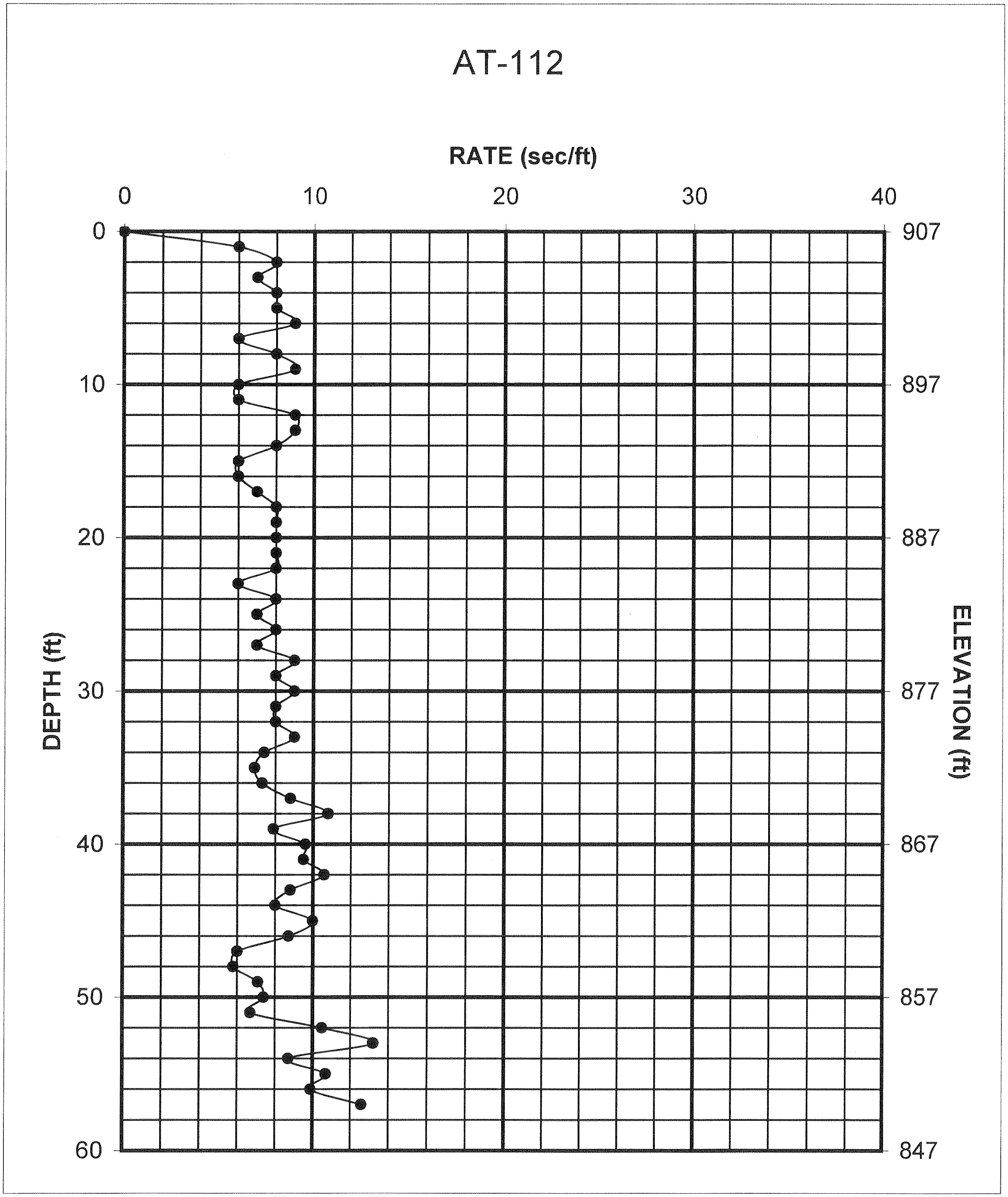


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 11

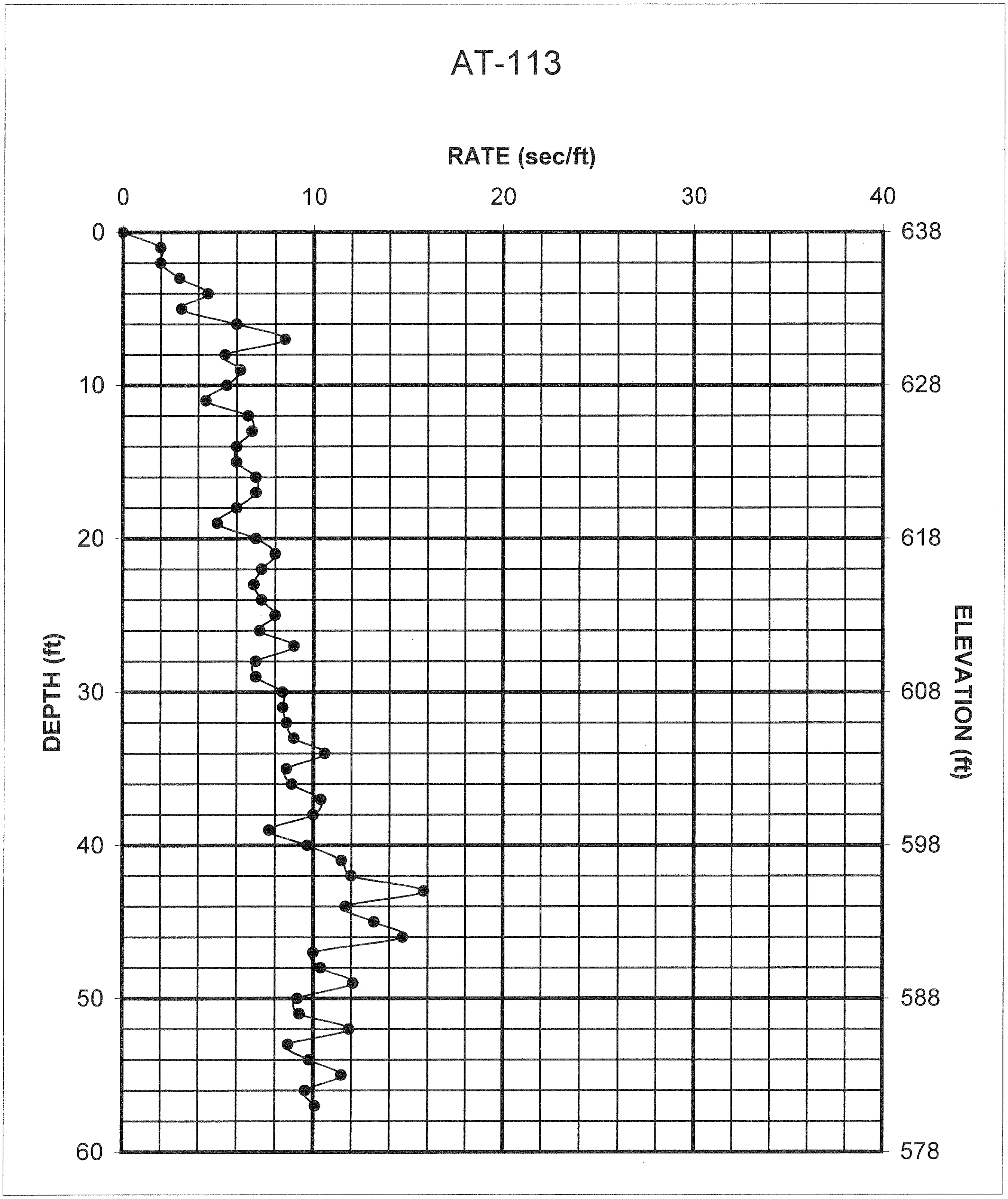


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 12

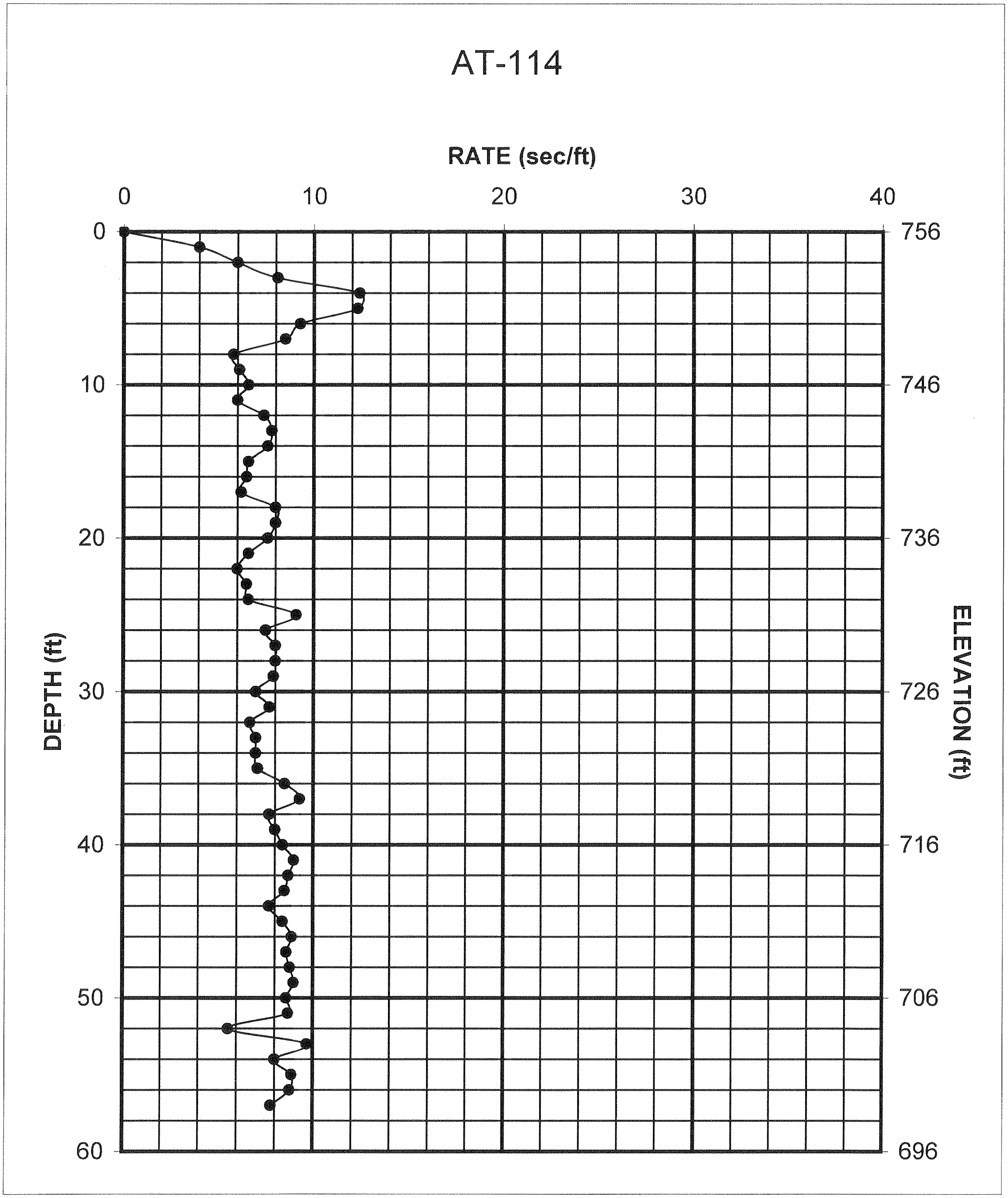


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 13

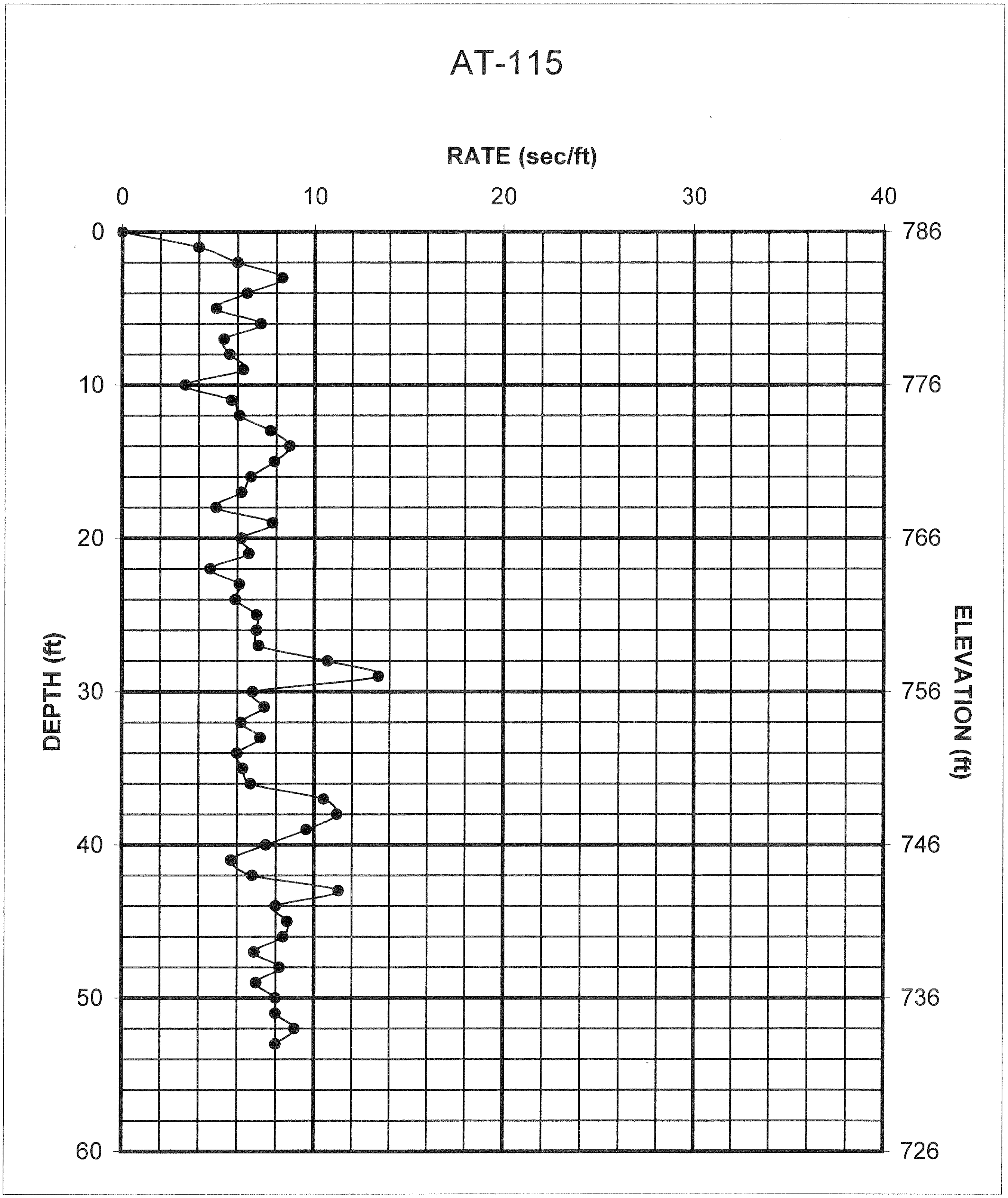


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 14

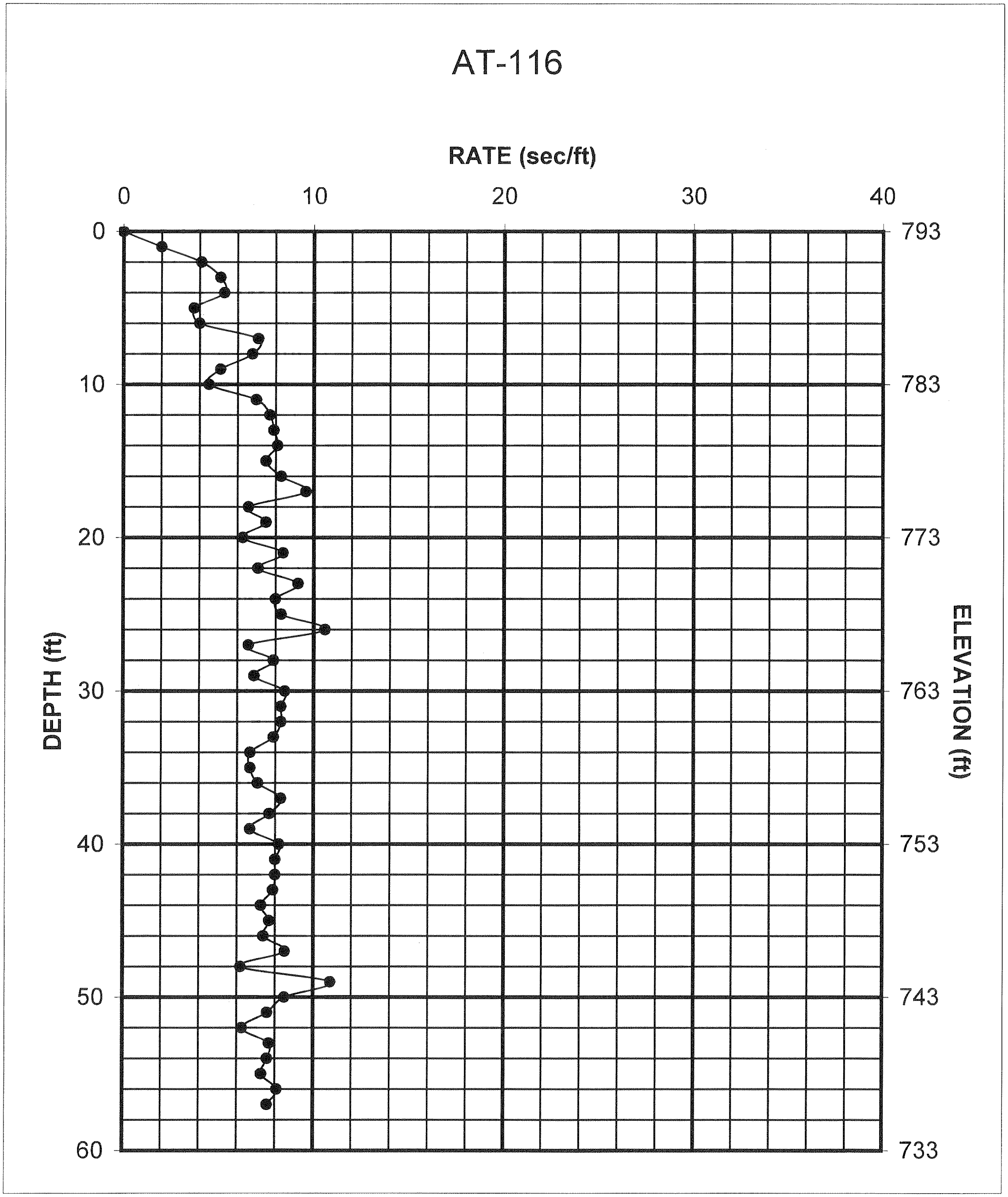


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 15

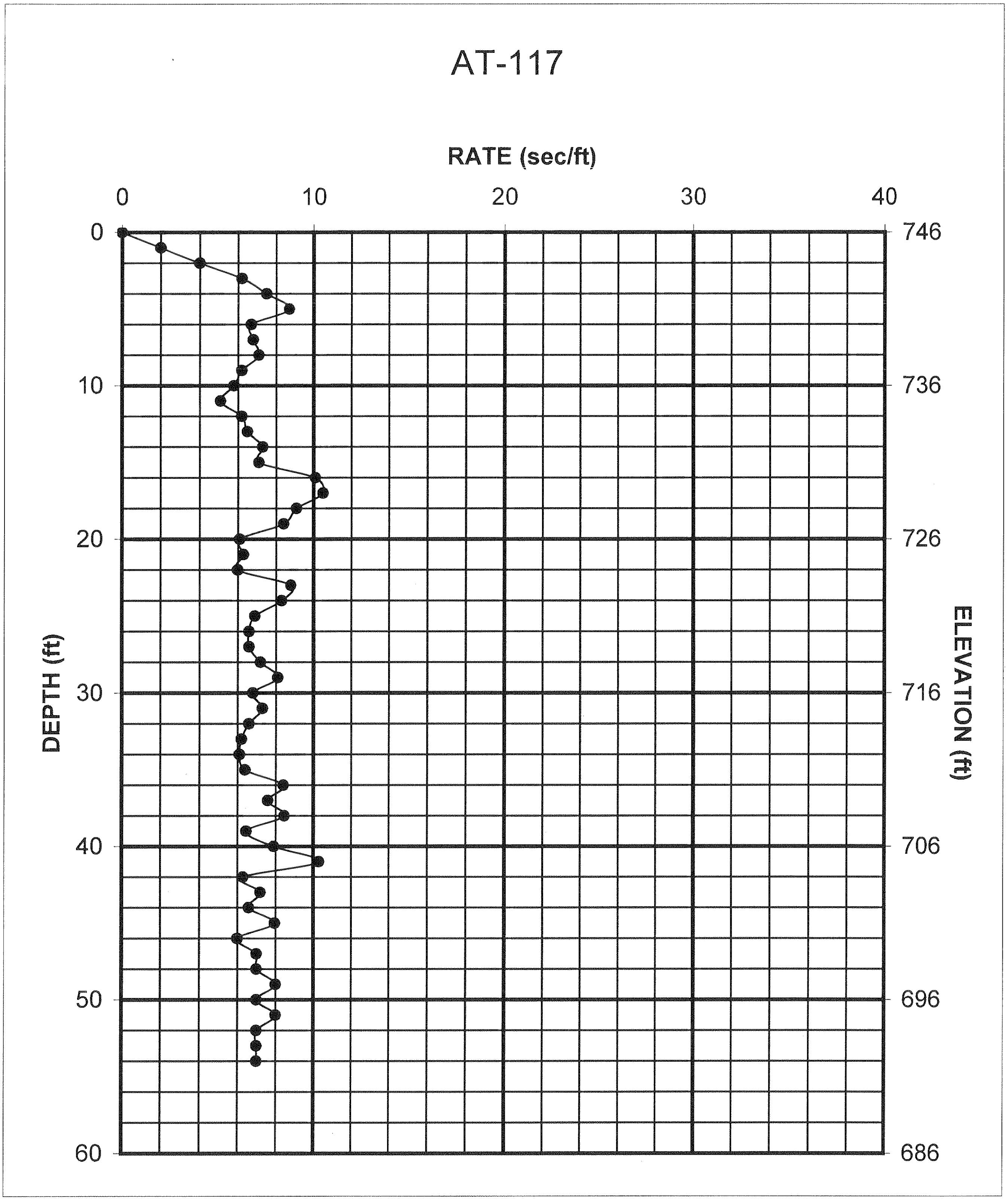


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 16



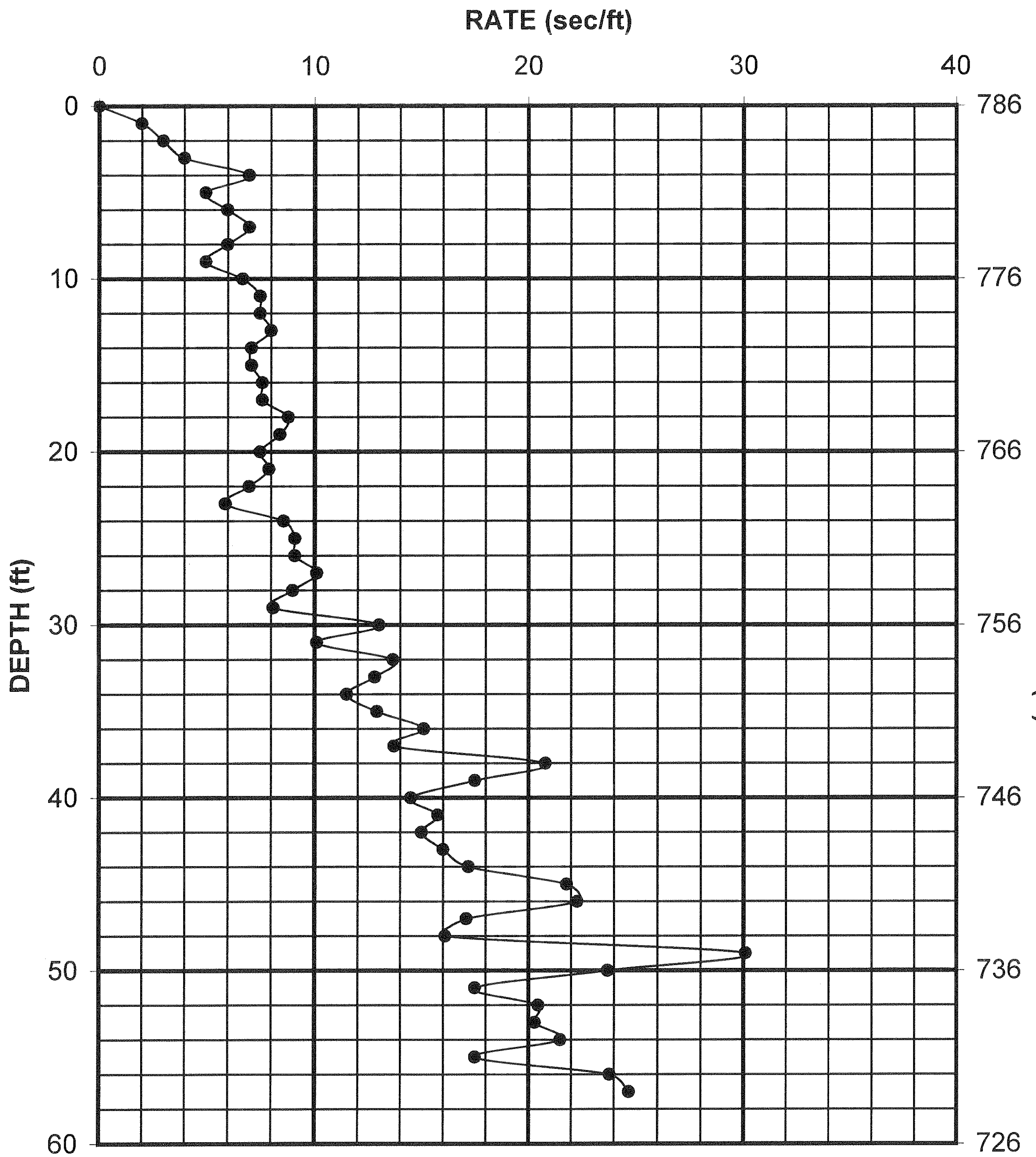
EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 17

AT-118

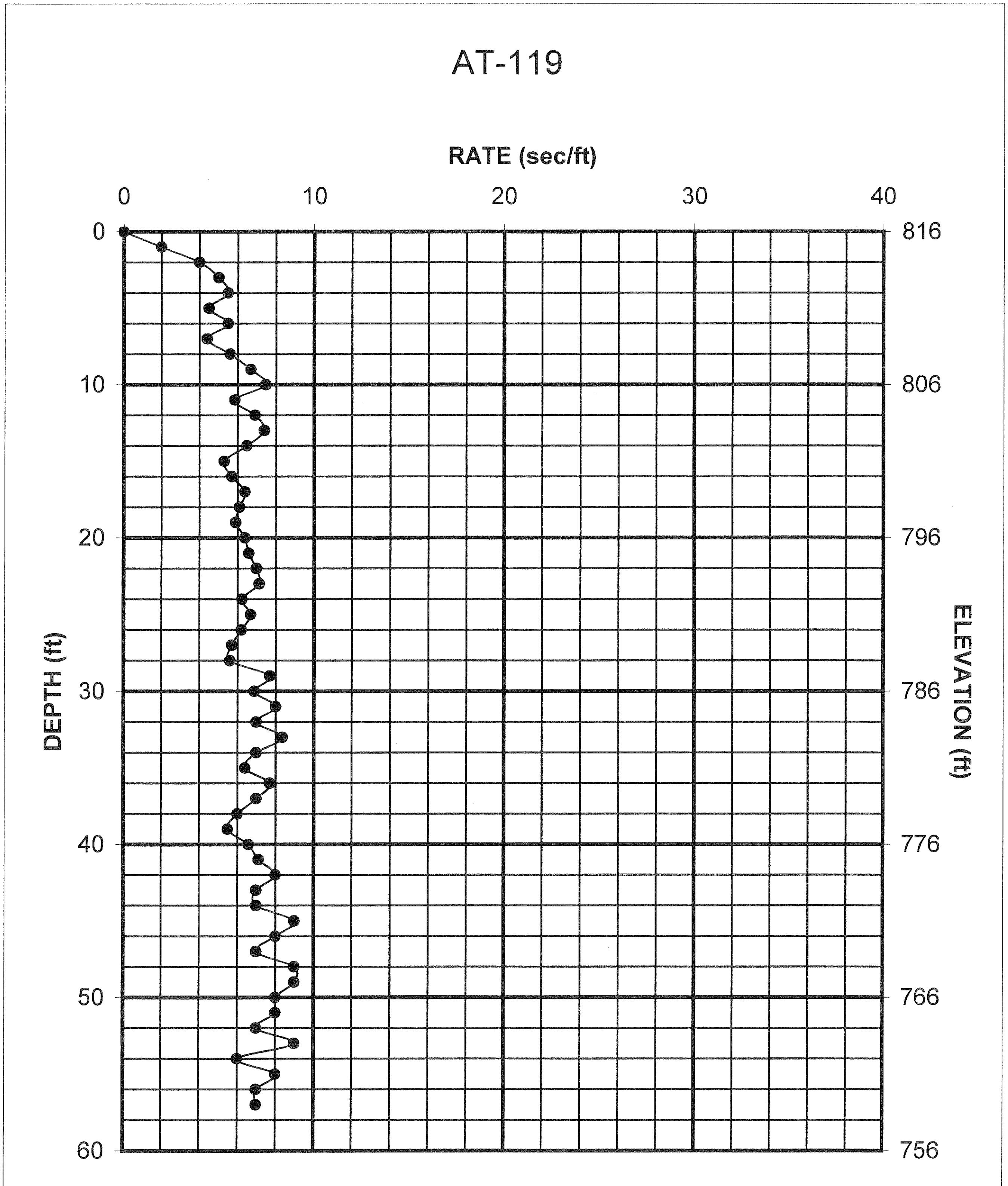


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 18

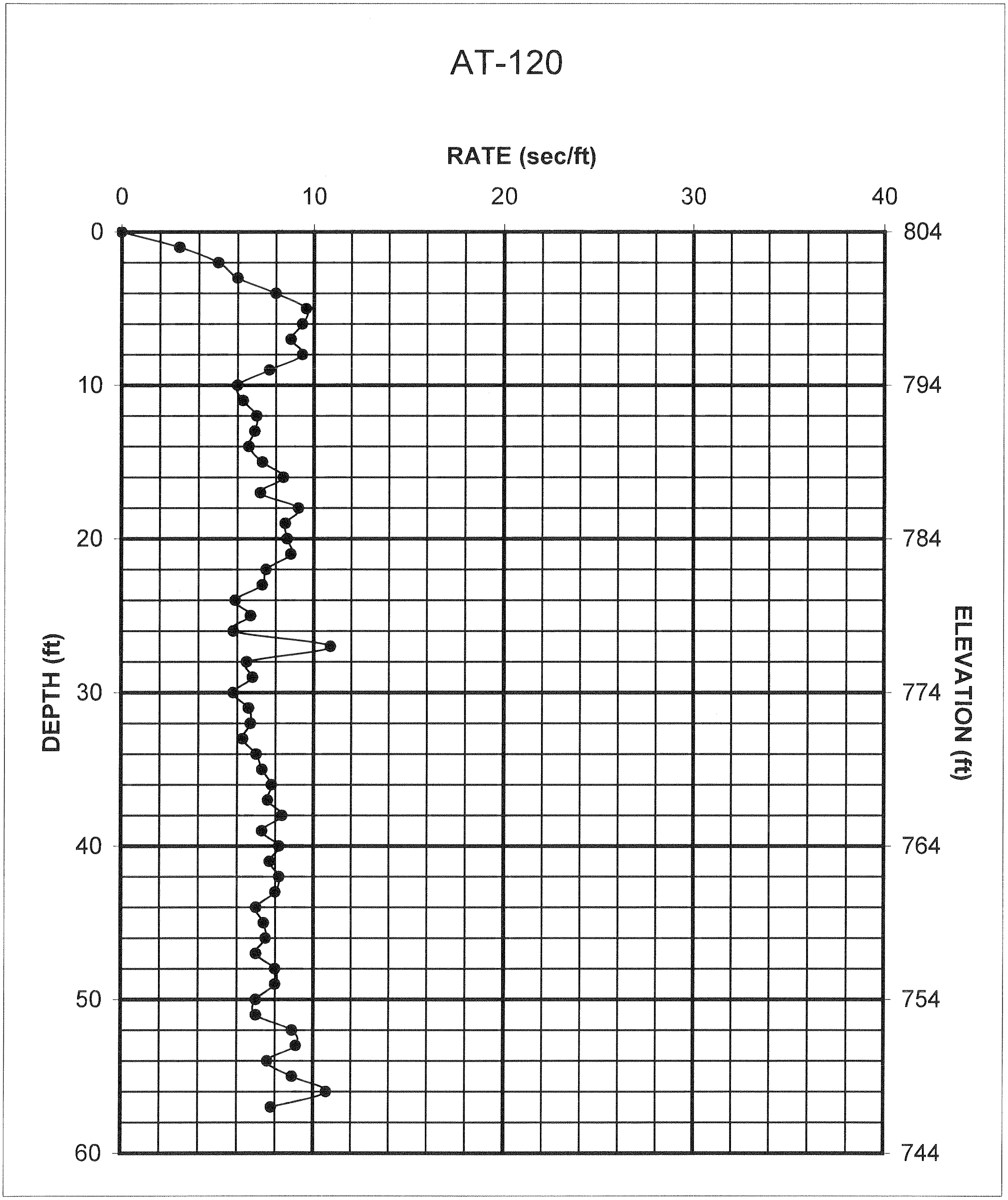


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 19

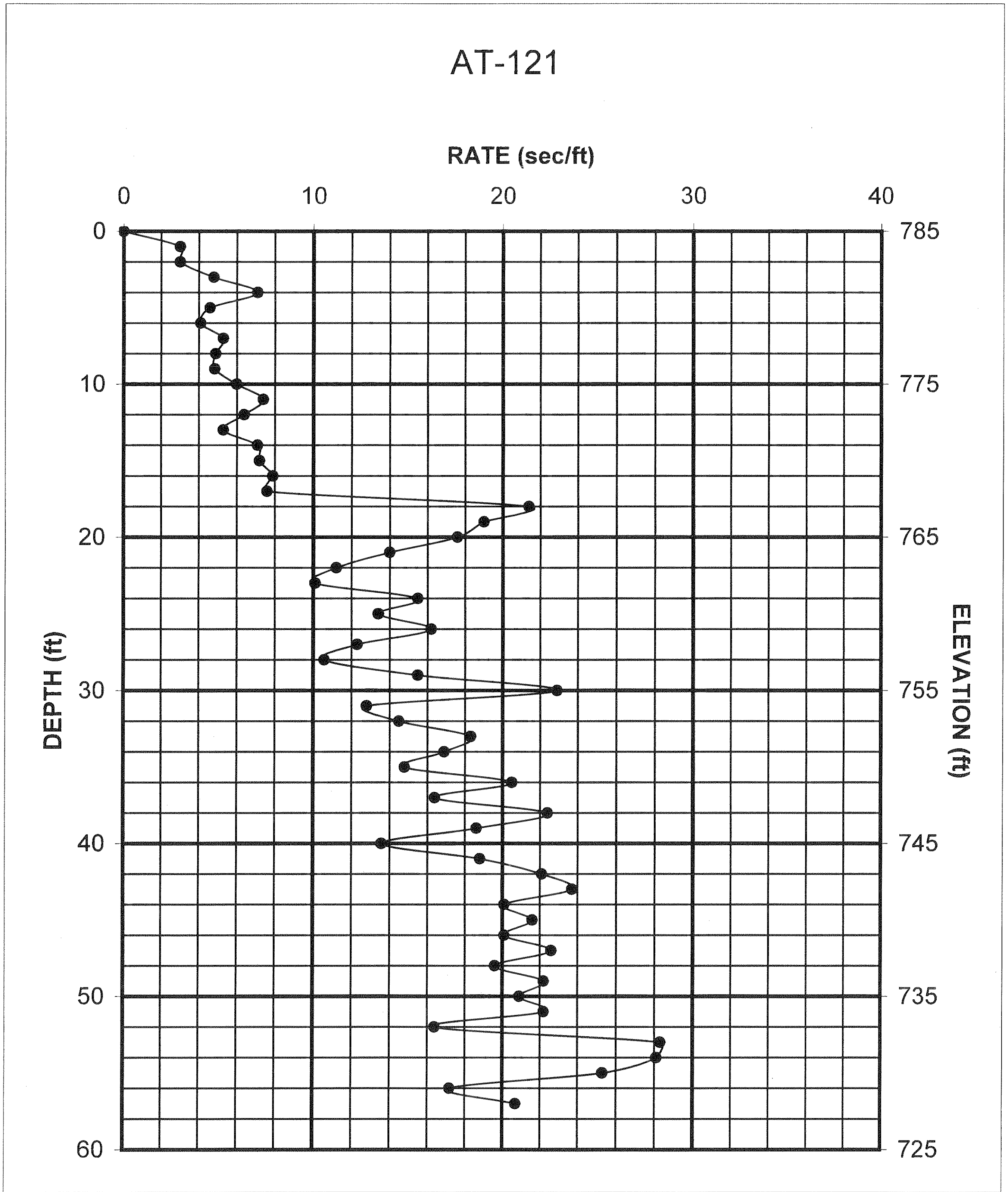


EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 20



EQUIPMENT - INGERSOLL RAND ECM-590

BIT SIZE = 4.5"

PACIFIC SOILS ENGINEERING, INC.
7715 Convoy Ct., San Diego, CA 92111
Work Order: 401120

PLATE A - 21

SEISMIC REFRACTION STUDY

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
MARCH 2012

SEISMIC REFRACTION SURVEY
LAS LILAS
SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR:

Advanced Geotechnical Solutions, Inc.
529 West 4th Avenue, Suite B
San Diego, CA 92025

PREPARED BY:

Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

February 17, 2012
Project No. 111415

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

February 17, 2012
Project No. 111415

Mr. Jeff Chaney
Advanced Geotechnical Solutions, Inc.
529 West 4th Avenue, Suite B
San Diego, CA 92025

Subject: Seismic Refraction Survey
Las Lilas
San Diego County, California

Dear Mr. Chaney:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Las Lilas project located in San Diego County, California. Specifically, our survey consisted of performing eight 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 related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

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

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

HV/PFL/hv

Distribution: (1) Electronic

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/29/12
STATE OF CALIFORNIA

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Las Lilas project located in San Diego County, California (Figure 1). Specifically, our survey consisted of performing eight 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 eight seismic 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 AND PROJECT DESCRIPTION

The project site is located just to the east of Interstate 15 and south of Lilac Road in San Diego County (Figure 1). The project area consists residential properties and includes single family custom homes, groves, and natural vegetation. Scattered exposures of crystalline rock were observed at the site. Figures 2a, 2b, 3a and 3b depict the general site conditions.

Based on our discussions with you, it is our understanding that the proposed project will include the construction of single family homes and associated infrastructure. Grading at the site may include cuts and fills with cuts up to 50 feet deep.

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 materi-

als of contrasting velocities. These refracted seismic waves are then detected by a series of 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.
Eight seismic lines (SL-1 through SL-8) 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.

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, 2004). 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. RESULTS

As previously indicated, eight 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 both SIPwin and SeisOpt Pro (Optim, 2008). Both programs use first arrival picks and elevation data to produce subsurface velocity models. SIPwin uses layer-based modeling techniques to produce a layered velocity model, where changes in velocities are depicted as discrete contacts. SeisOpt Pro uses 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.

Table 2 lists the approximate P-wave velocities and depths calculated from the seismic refraction traverse using the layered modeling method. The approximate locations of the seismic refraction traverses are shown on the Line Location Maps (Figures 2a and 2b). The velocity models are included in Figures 4a through 4h. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

Table 2 – Seismic Traverse Results[1]

Traverse No. And Length	P-wave Velocity feet/second	Approximate Depth to Bottom of Layer in feet	Apparent Rippability[2]
SL-1 240 feet	V1 = 1,290 V2 = 4,040	3 – 11 ---	Easy Difficult, Possible Blasting
SL-2 240 feet	V1 = 1,180 V2 = 3,380 V3 >10,000	2 – 6 34 – 50 ---	Easy Moderate Blasting Generally Required
SL-3 240 feet	V1 = 1,270 V2 = 3,790 V3 = 5,780	4 – 8 4 – 51 ---	Easy Moderate Very Difficult, Probable Blasting
SL-4 240 feet	V1 = 1,180 V2 = 3,620 V3 = 4,770	5 – 8 26 – 46 ---	Easy Moderate Difficult, Possible Blasting
SL-5 240 feet	V1 = 1,230 V2 = 3,690 V3 >10,000	2 – 7 60 – 67 ---	Easy Moderate Blasting Generally Required
SL-6 240 feet	V1 = 1,420 V2 = 4,680 V3 >10,000	7 – 9 36 – 74 ---	Easy Difficult, Possible Blasting Blasting Generally Required
SL-7 240 feet	V1 = 1,290 V2 = 3,730 V3 = 8,120	1 – 8 28 – 62 ---	Easy Moderate Blasting Generally Required
SL-8 240 feet	V1 = 1,240 V2 = 3,200	4 – 9 ---	Easy Moderate

1 Results based on the model generated using SIPwin, 2003
2 Rippability criteria based on the use of a Caterpillar D-9 dozer ripping with a single shank

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil (colluvium and topsoil) overlying crystalline bedrock with varying degrees of weathering. Figures 4a through 4h provide the velocity models calculated from both SIPwin and SeisOpt Pro. Distinct vertical and lateral variations between the two models are evident. In general the tomography results better characterize the onsite conditions than the layer models.

The cause of the velocity variations revealed in the data are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. Therefore, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area.

Based on our results, very difficult conditions where blasting may be required will likely be encountered depending on the excavation depth, location, 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, expressed 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., 2004, Caterpillar Performance Handbook, Edition 35, Caterpillar, Inc., Peoria, Illinois.

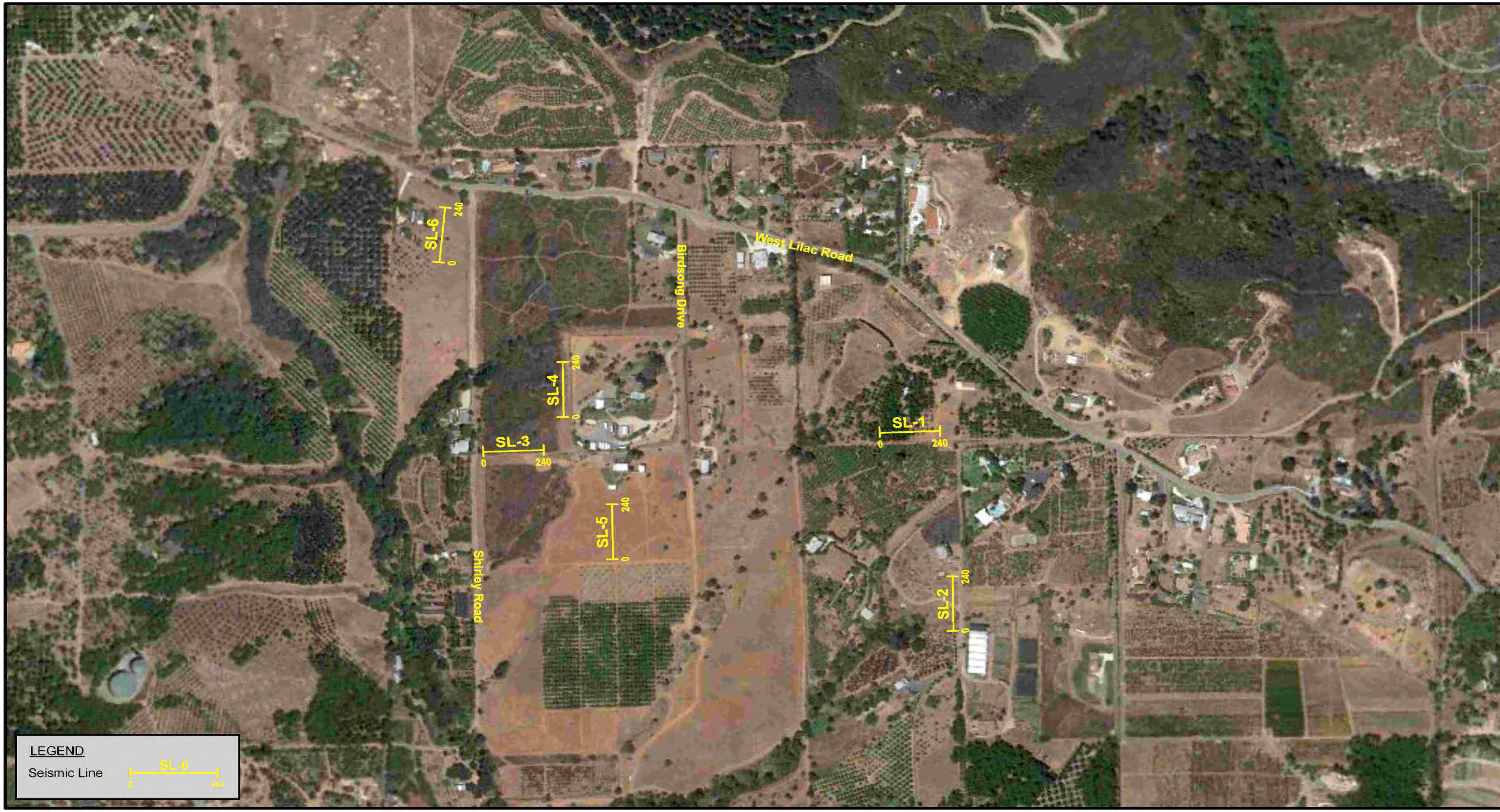
Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

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

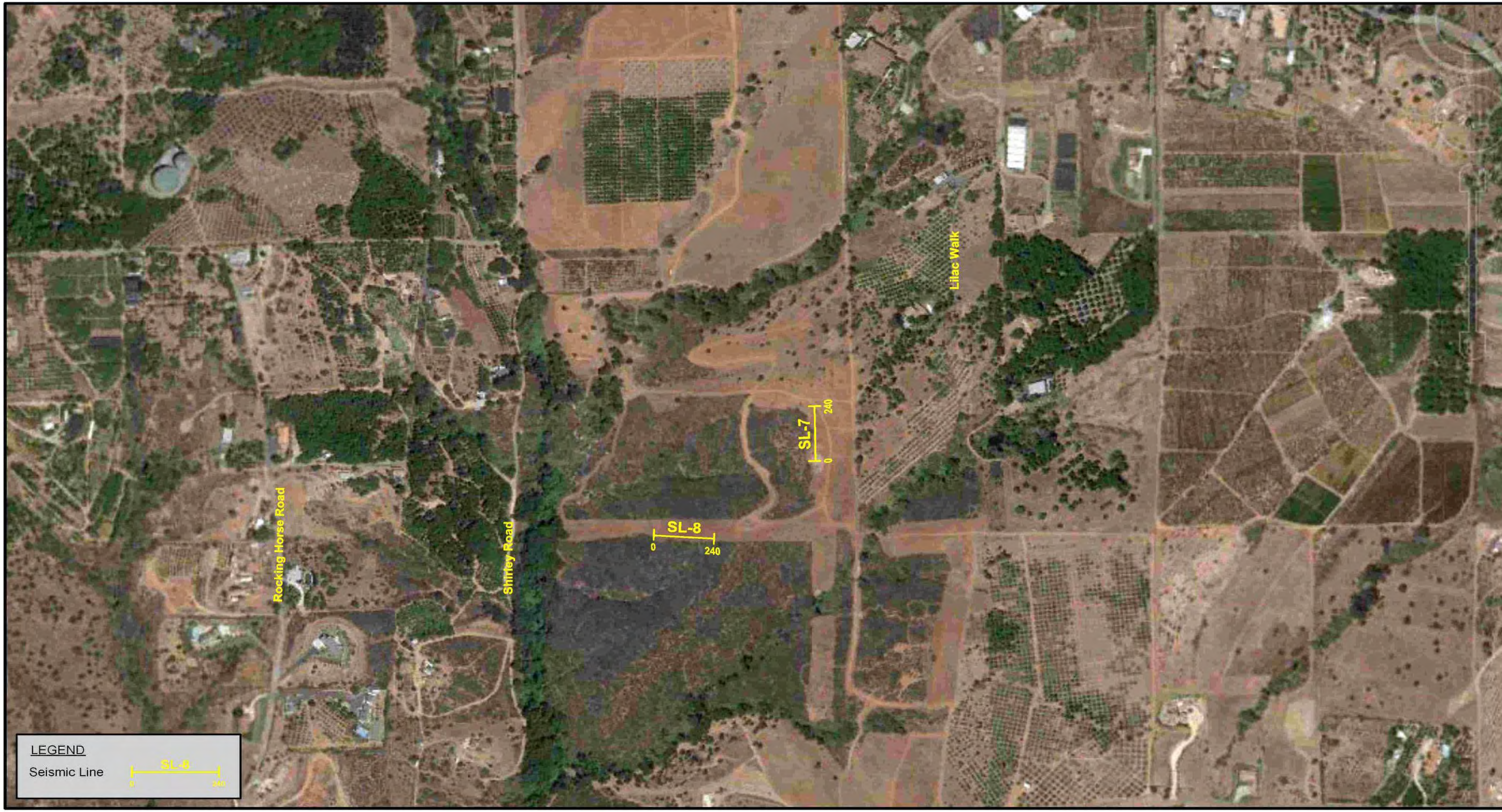
Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

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

SITE LOCATION MAP	N	Las Lilas San Diego County, California		SOUTHWEST GEOPHYSICS INC.
		Project No.: 111415	Date: 02/12	Figure 1

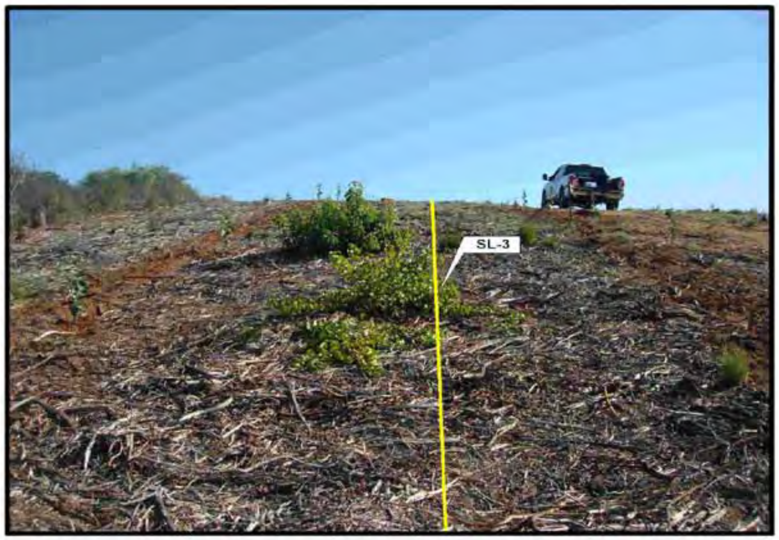


LINE LOCATION MAP (SL-1 through SL-6)	N	Las Lilas San Diego County, California Project No.: 111415 Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 2a

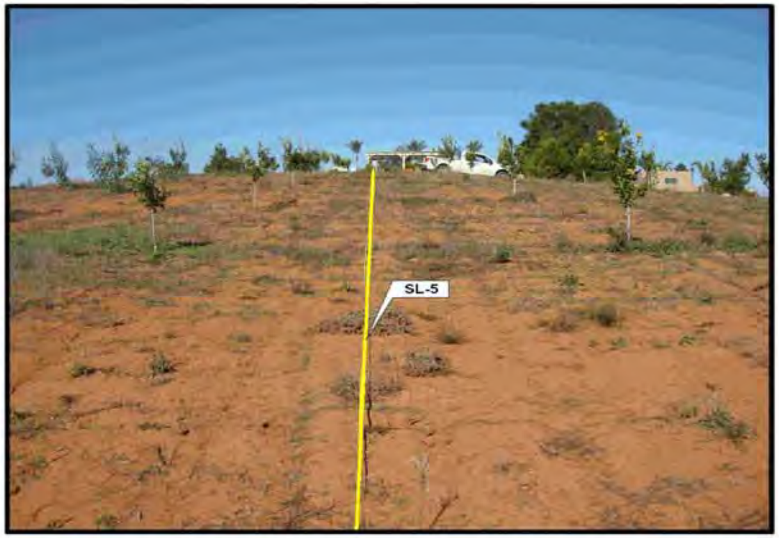


LINE LOCATION MAP (SL-7 and SL-8)	N	Las Lilas San Diego County, California Project No.: 111415 Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 2b

0 400 800
approximate scale in feet

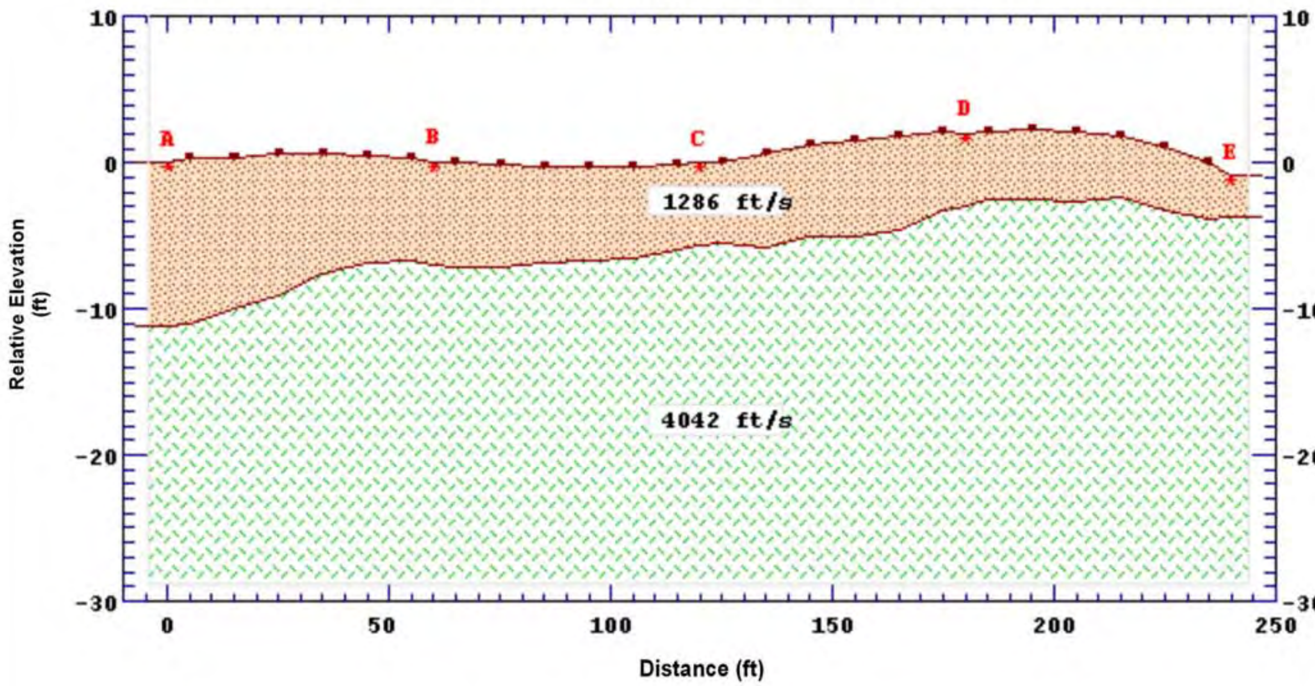


SITE PHOTOGRAPHS (SL-1 to SL-4)	Las Lilas San Diego County, California		SOUTHWEST GEOPHYSICS INC. Figure 3a
	Project No.: 111415	Date: 02/12	



SITE PHOTOGRAPHS (SL-5 to SL-8)	Las Lilas San Diego County, California Project No.: 111415 — Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 3b

Layer Model



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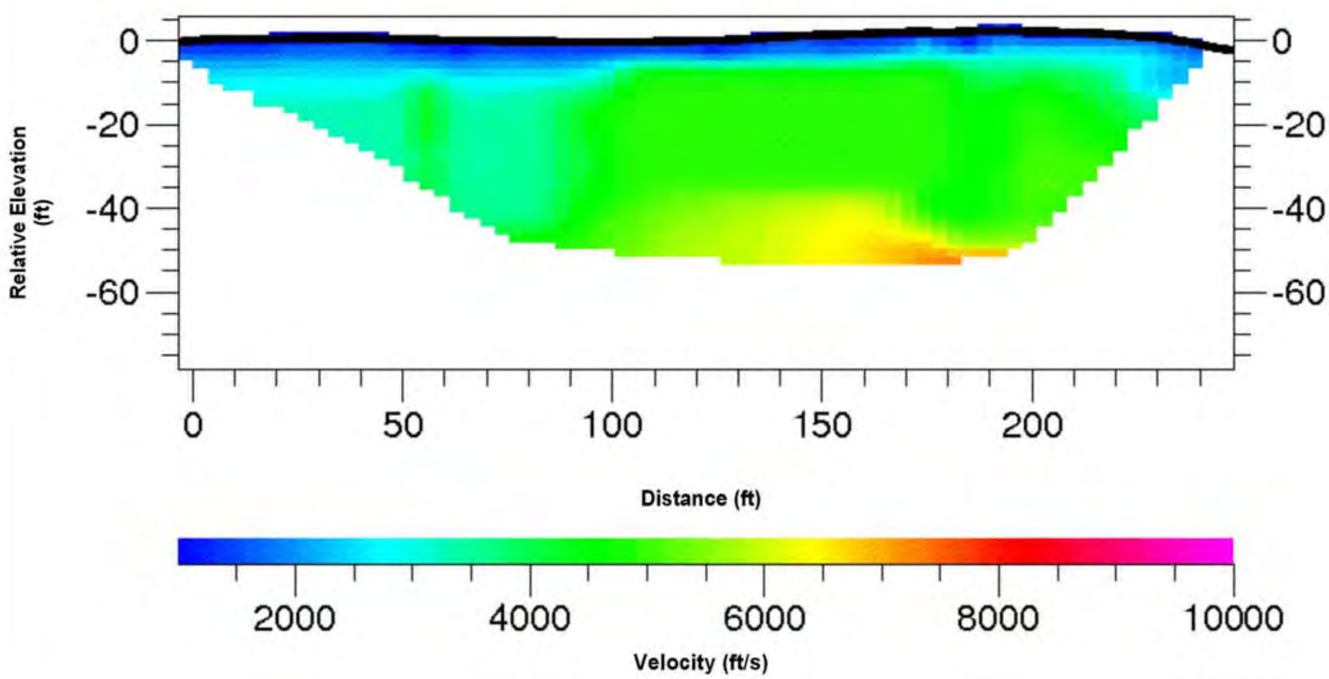
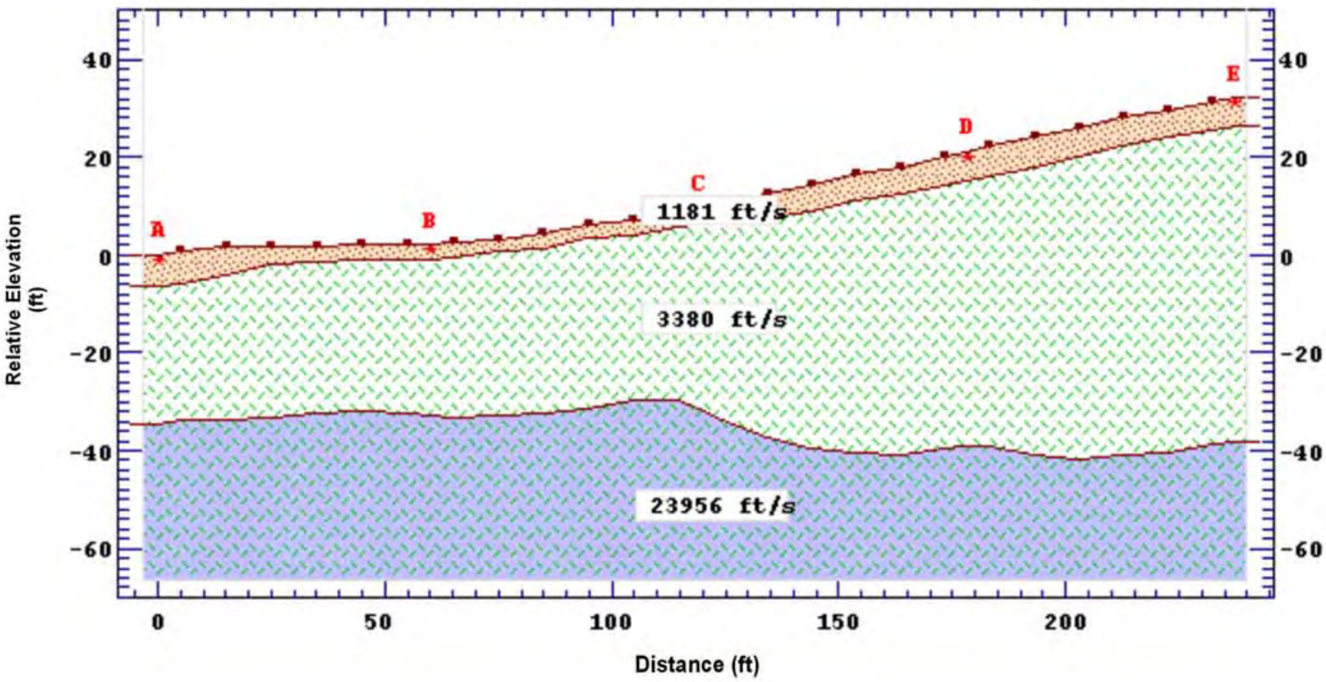


SEISMIC PROFILE SL-1	Las Lilas San Diego County, California Project No.: 111415 Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 4a

Layer Model



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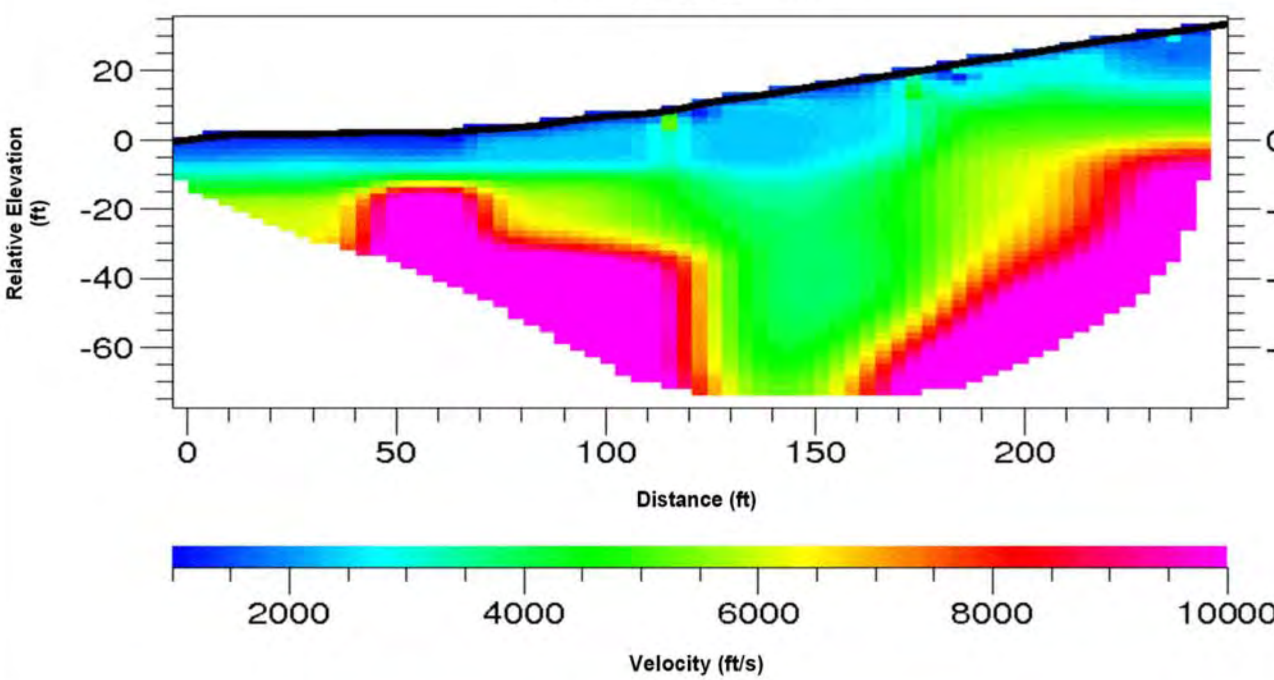
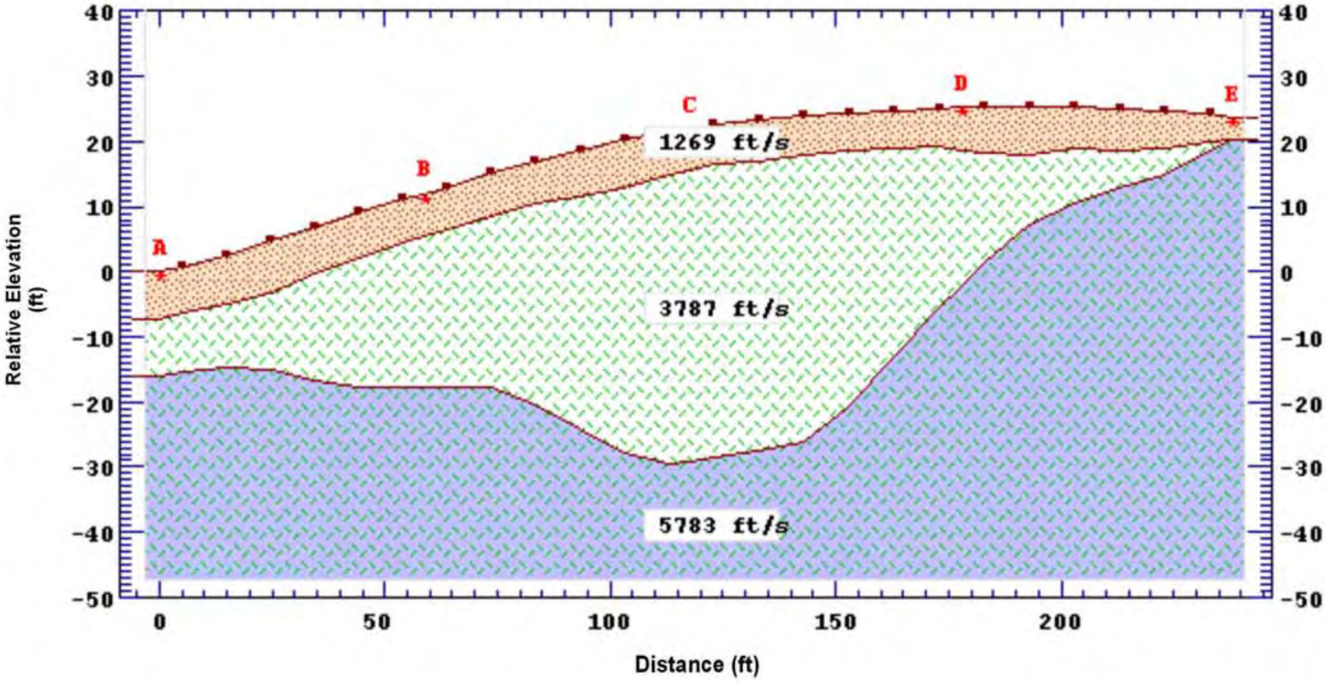


SEISMIC PROFILE SL-2	Las Lilas San Diego County, California		SOUTHWEST GEOPHYSICS INC. Figure 4b
	Project No.: 111415	Date: 02/12	

Layer Model



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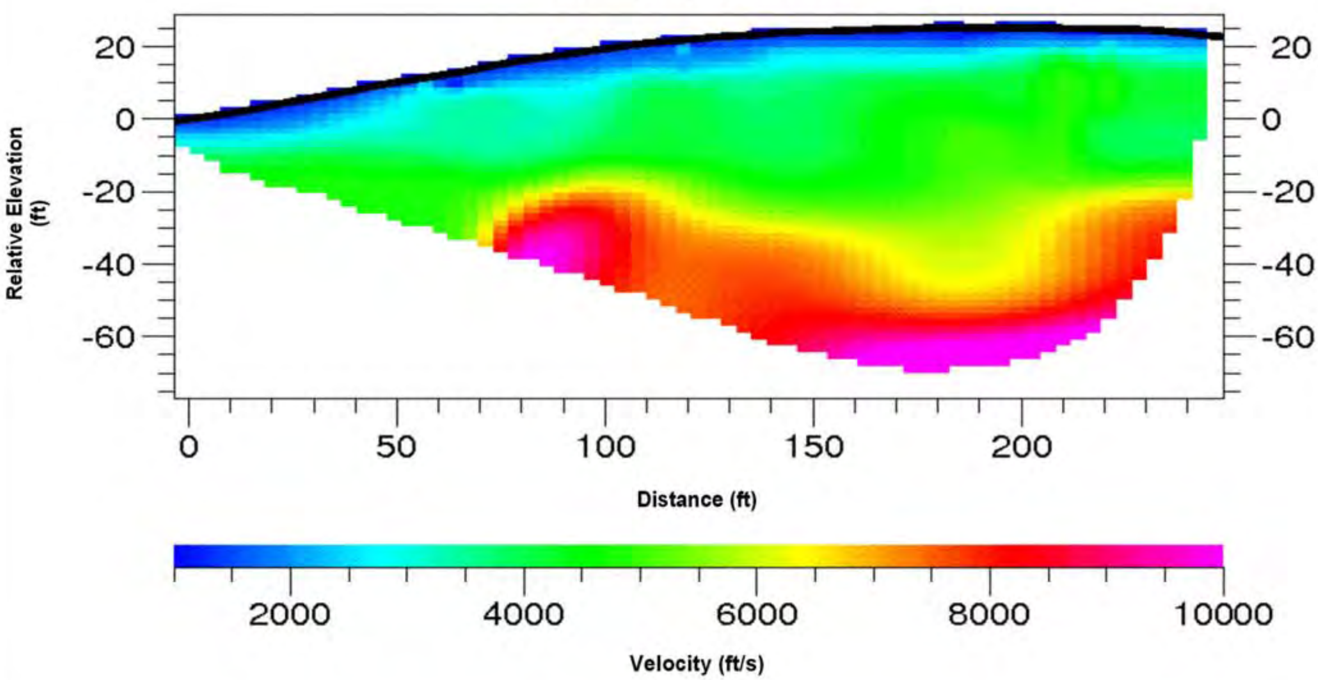

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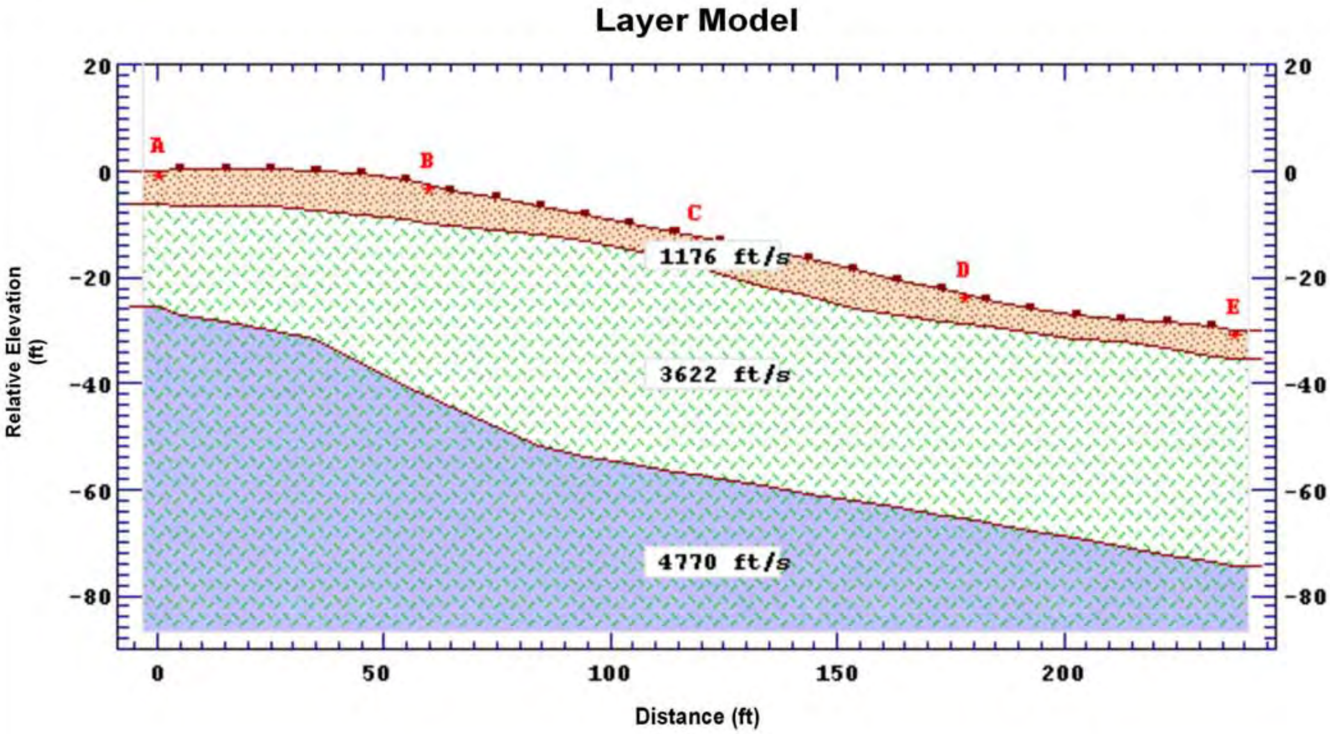
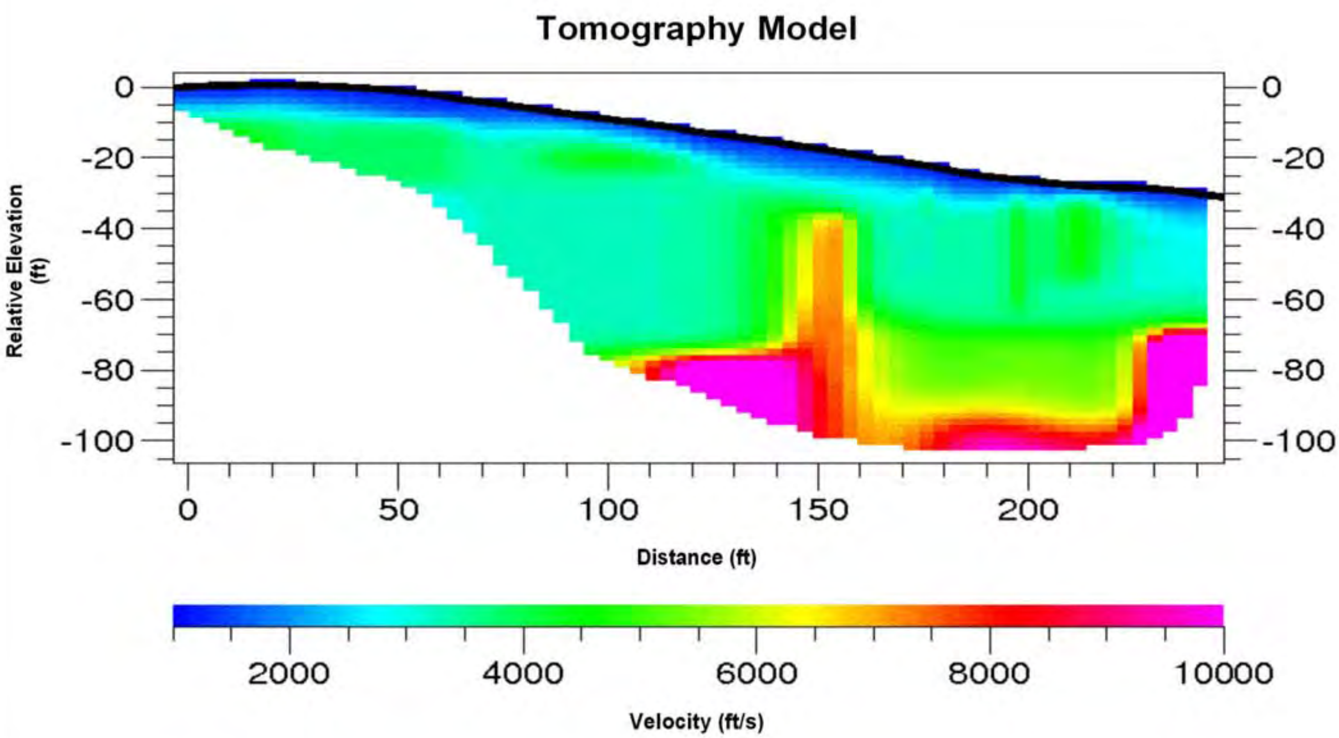




SEISMIC PROFILE SL-4

Las Lilas
San Diego County, California

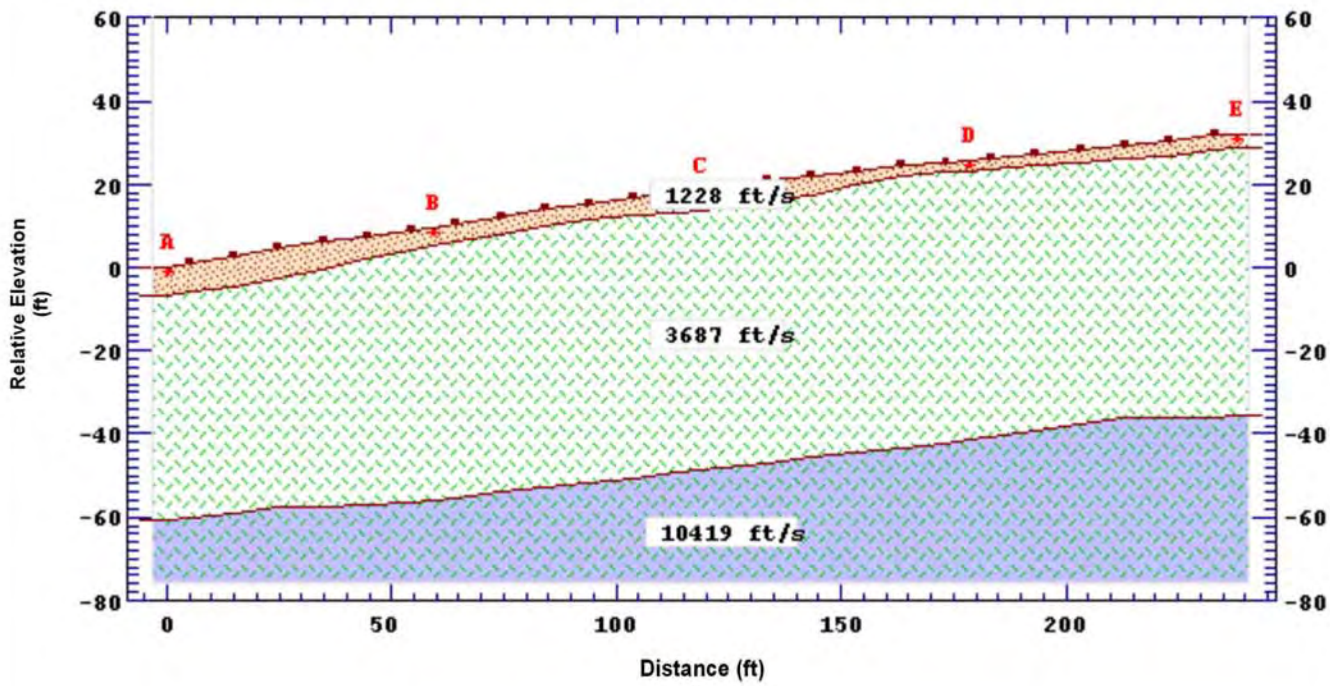
Project No.: 111415

Date: 02/12

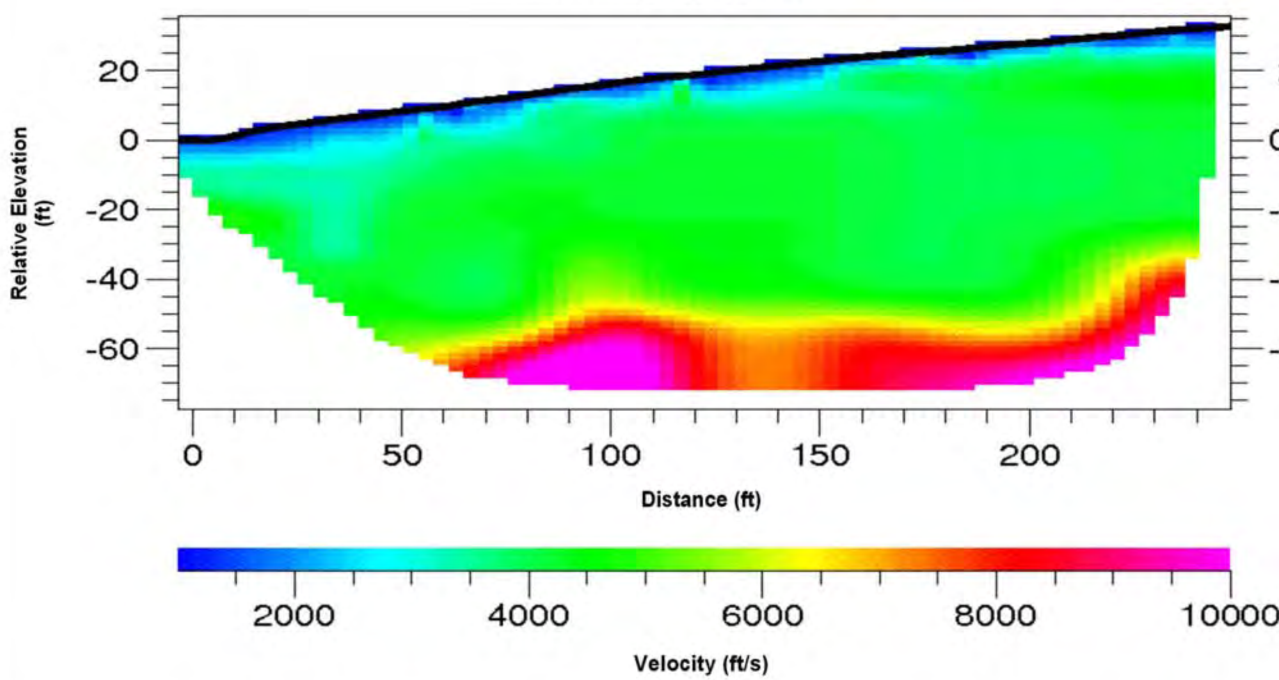
SOUTHWEST GEOPHYSICS INC.

Figure 4d

Layer Model

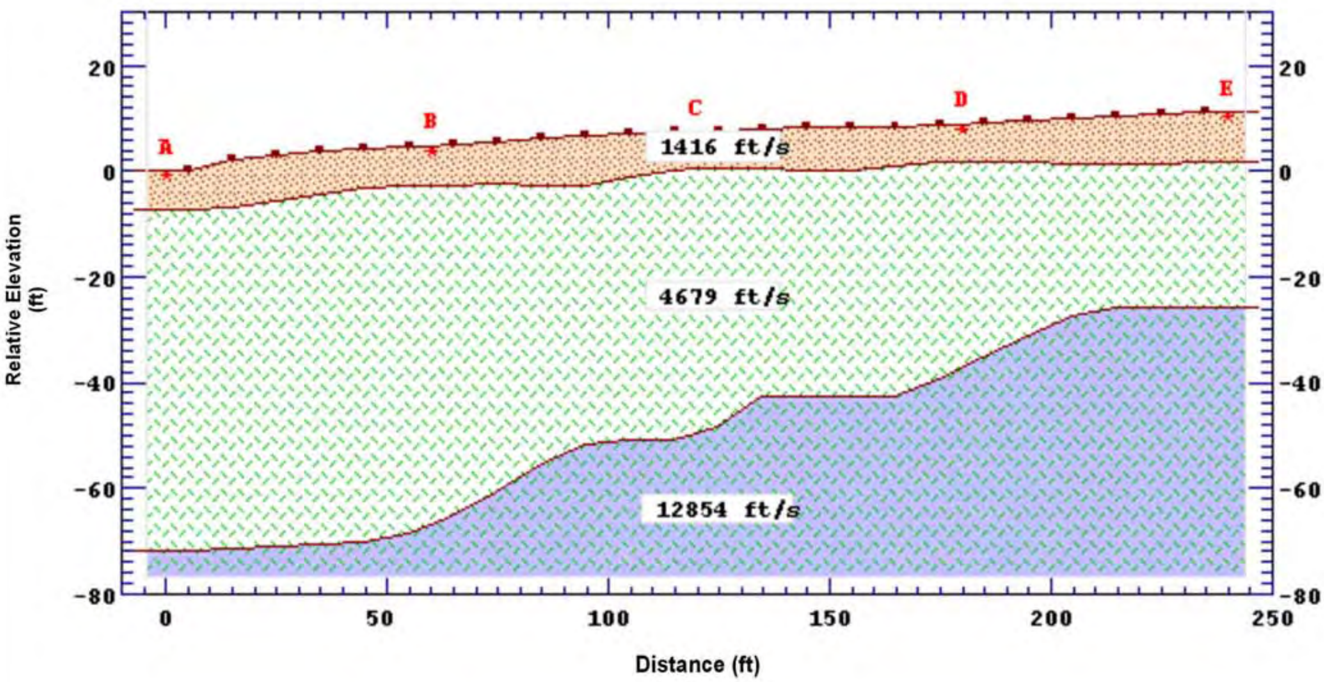


Tomography Model



SEISMIC PROFILE SL-5	Las Lilas San Diego County, California Project No.: 111415 · Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 4e

Layer Model



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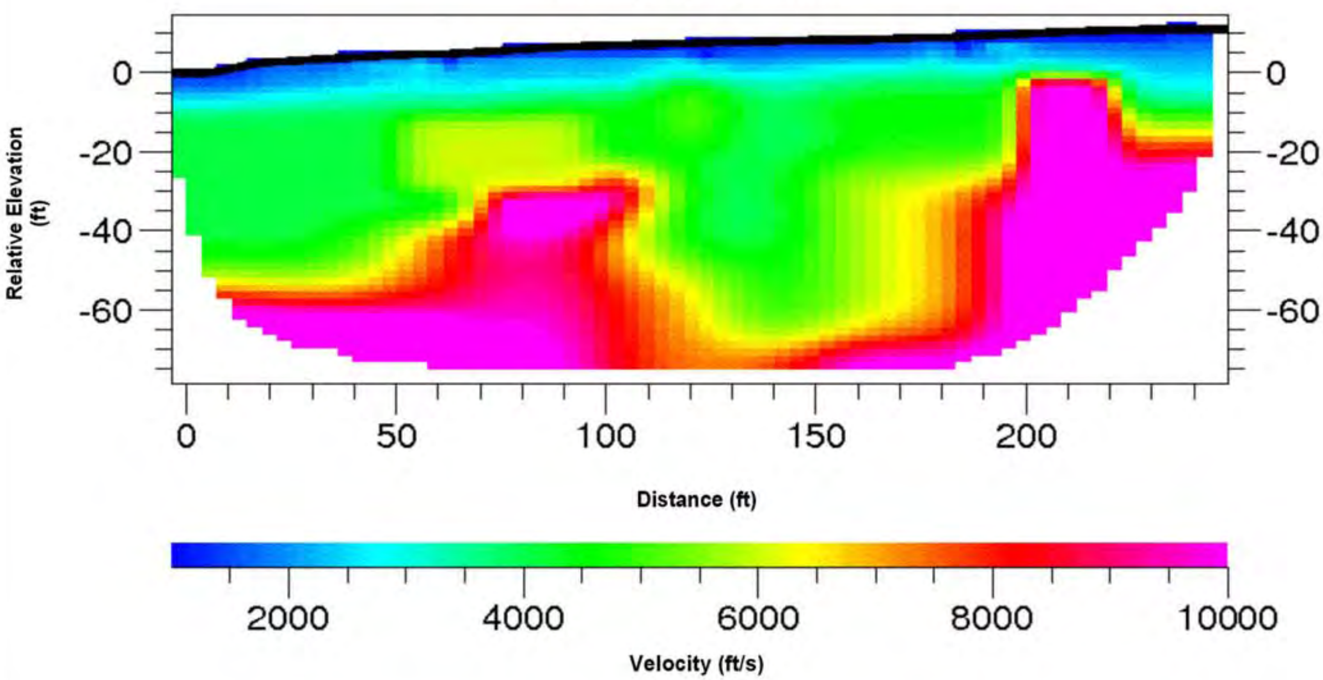
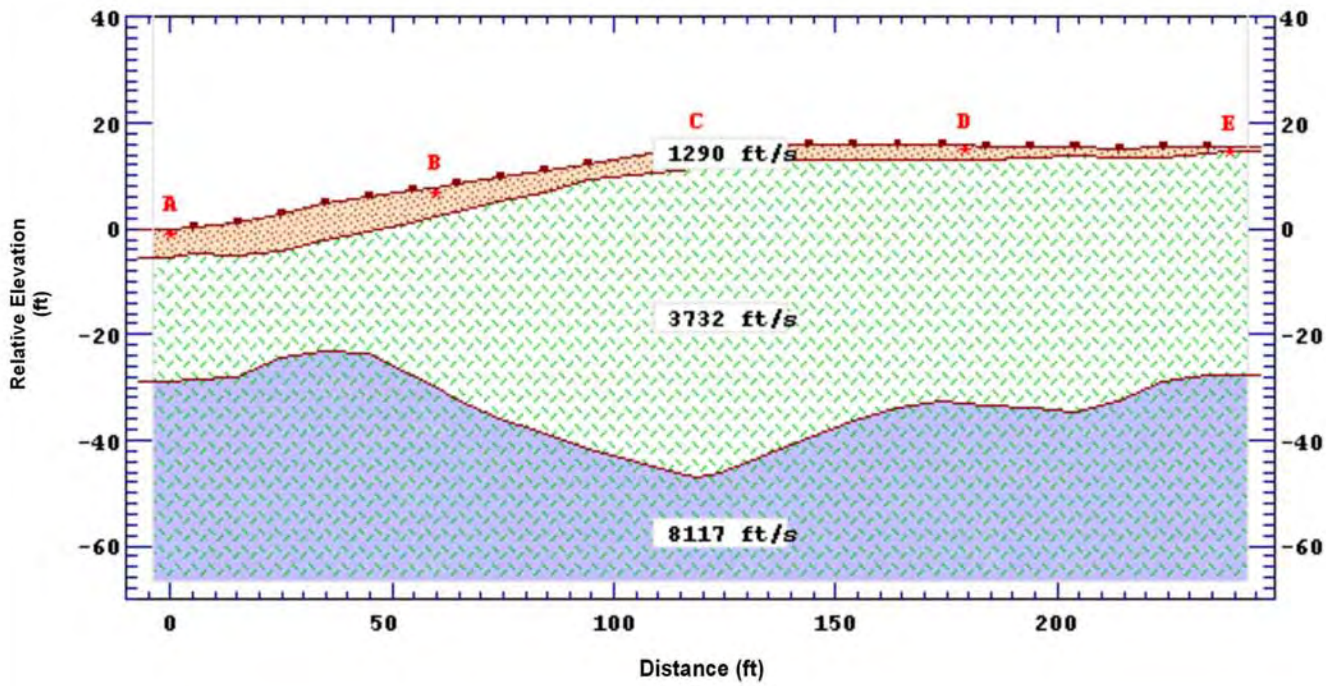


SEISMIC PROFILE SL-6	Las Lilas San Diego County, California Project No.: 111415 — Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 4f

Layer Model



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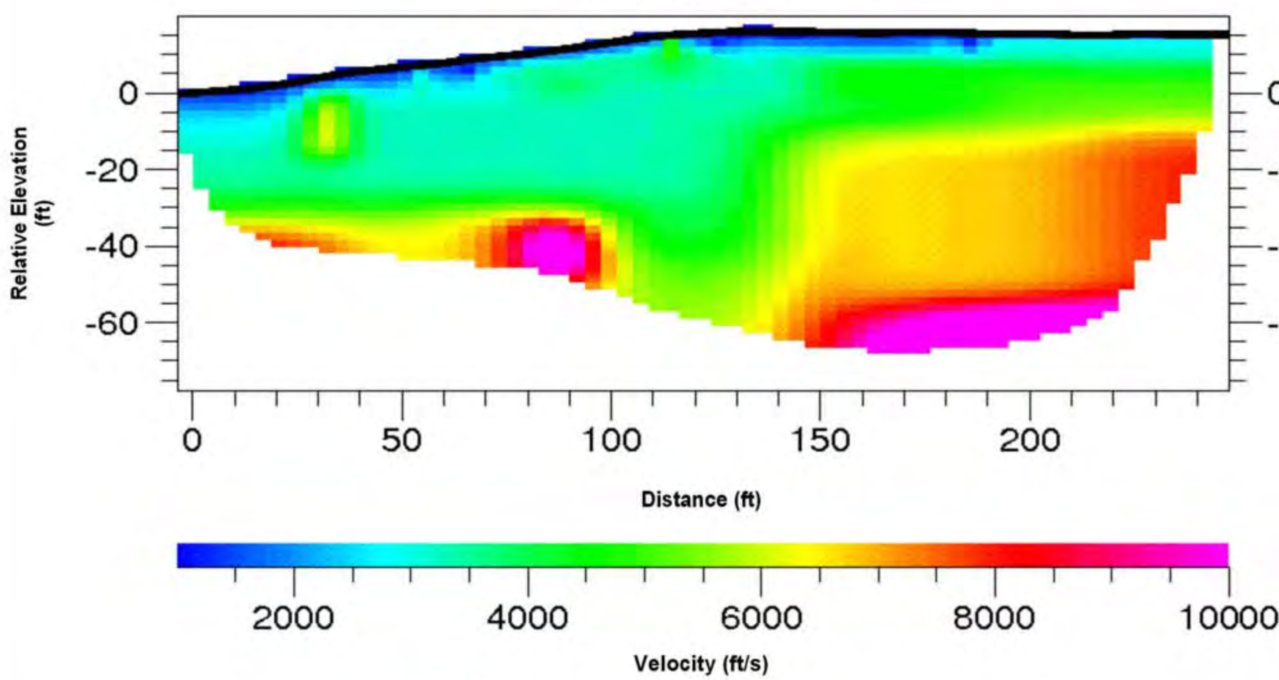
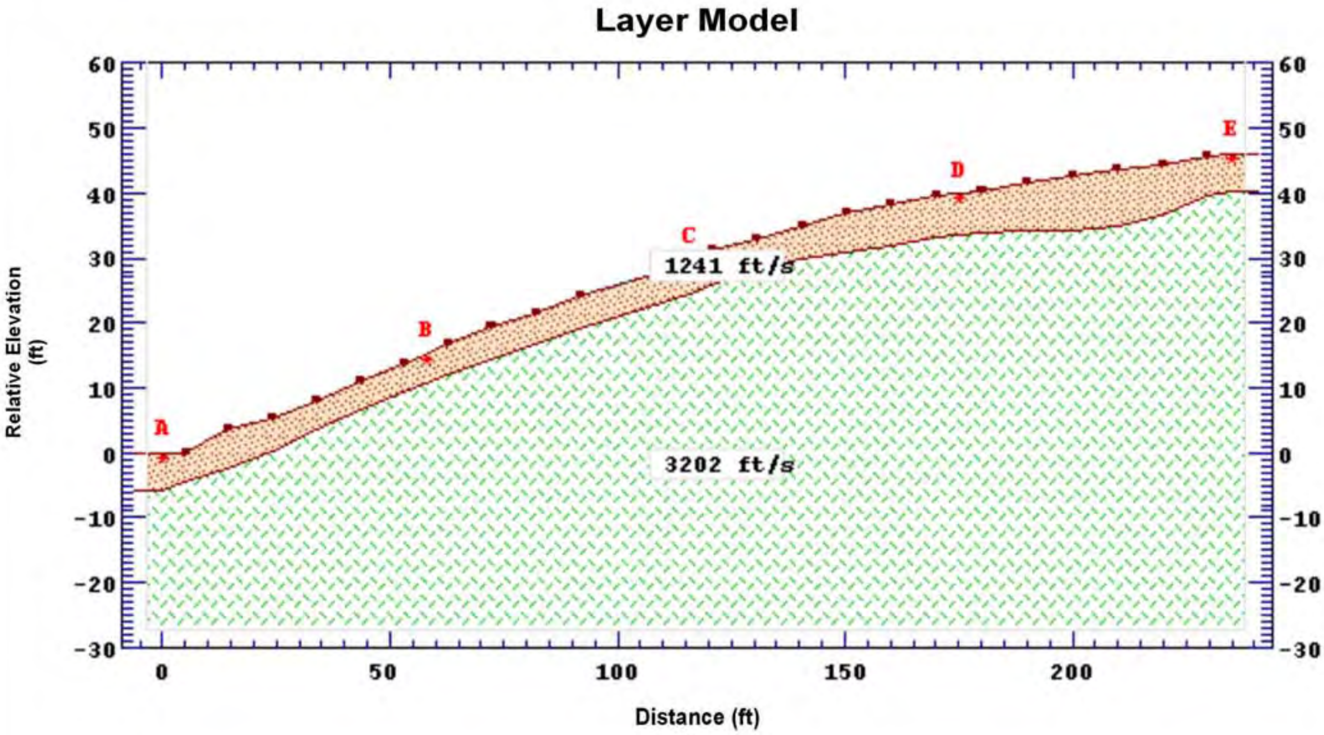
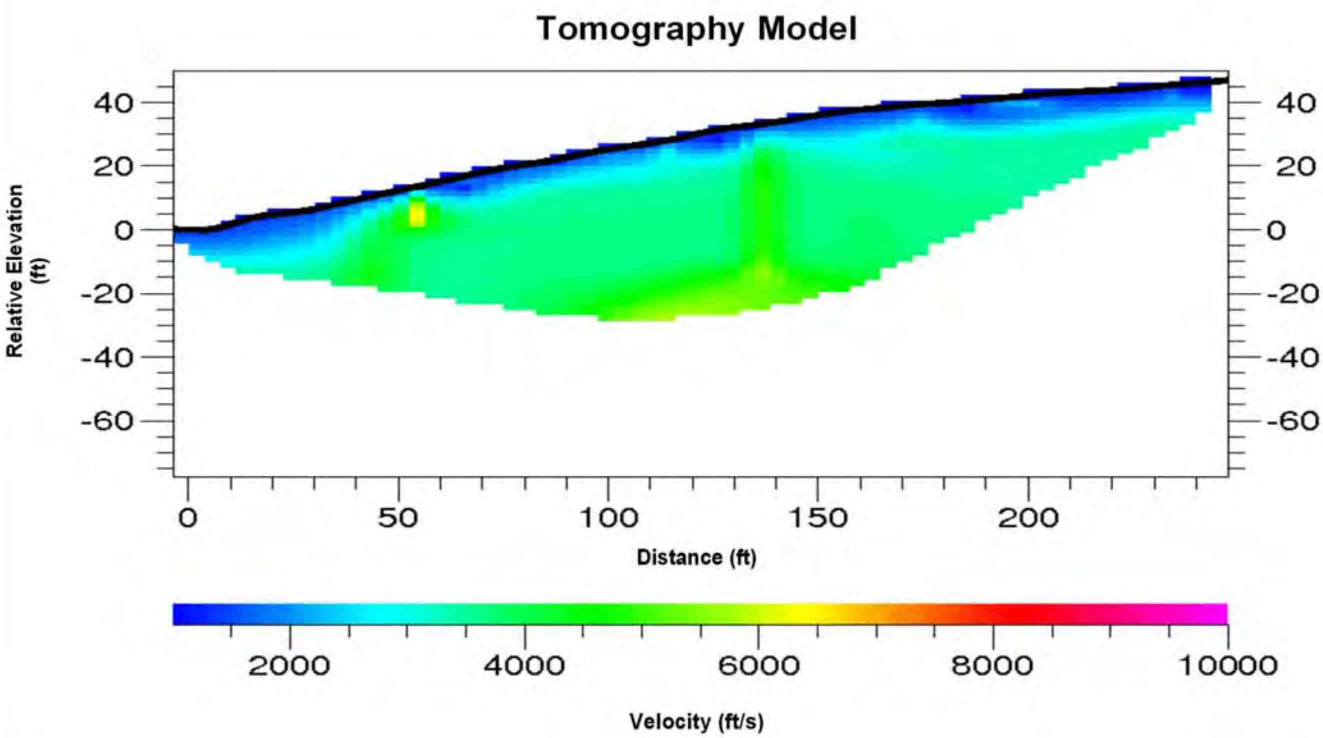


SEISMIC PROFILE SL-7	Las Lilas San Diego County, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 111415 / Date: 02/12	Figure 4g





SEISMIC PROFILE SL-8	Las Lilas San Diego County, California Project No.: 111415 — Date: 02/12	SOUTHWEST GEOPHYSICS INC. Figure 4h

SEISMIC REFRACTION STUDY

PACIFIC SOILS ENGINEERING, INC.
MAY 2007

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

SEISMIC REFRACTION SURVEY
WEST LILAC ROAD AND COVEY LANE
ESCONDIDO, CALIFORNIA

PREPARED FOR:
Pacific Soils Engineering, Inc.
7715 Convoy Court
San Diego, California 92111

PREPARED BY:
Southwest Geophysics, Inc.
7438 Trade Street
San Diego, California 92121

May 17, 2007
Project No. 107060

SOUTHWEST GEOPHYSICS, INC.

YOUR SUBSURFACE SOLUTION

May 17, 2007
Project No. 107060

Mr. Charles Ince
Pacific Soils Engineering, Inc.
7715 Convoy Court
San Diego, California 92111

Subject: Seismic Refraction Survey
West Lilac Road and Covey Lane
Escondido, California

Dear Mr. Ince:

In accordance with your authorization, we have performed a seismic refraction survey for the proposed West Lilac Road and Covey Lane project located in Escondido, California. Specifically, our survey consisted of performing 19 seismic refraction lines at the project site. The purpose of our study was to develop a subsurface velocity profile of the areas surveyed, and to assess the apparent rippability of near surface 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 related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Patrick Lehrmann

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

Hans van de Vrugt

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

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/29/08
STATE OF CALIFORNIA

HV/PFL/hv

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TABLE OF CONTENTS

1. INTRODUCTION

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- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is located to the south of West Lilac Road, east of Interstate 15 in Escondido (Figure 1). In general, the study area included farm land and residential properties located in the vicinity of West Lilac Road and Covey Lane (Figure 2). Terrain in the study area consists of gentle to moderately steep hills and associated drainages. Vegetation varied significantly across the project area and included avocado trees, cacti, annual grass, sage brush, and miscellaneous bushes and trees. Several outcrops of granitic rock and piles of displaced boulders were observed in the study area. Figures 3a through 3e provide a general view of the site conditions along the seismic lines.

Based on our discussions with you, we understand that the project will include the construction of single family homes and associated improvements. Although grading plans were not available for our review, we understand that cuts up to 100 feet are planned.

4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the depth to bedrock and apparent rippability characteristics of the subsurface materials, and to develop a subsurface velocity profile 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 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.

Nineteen seismic lines/profiles (SL-1 through SL-19) were conducted at the project site. The general location and length of the lines were selected by your office prior to our study (Figure 2). Shot points were conducted at each end of the line and at the midpoint. In addition, intermediate shots between the midpoint and the end of the line were conducted along SL-18. The lines were up to 270 feet long.

The refraction method requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not 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/outcrops, 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 Local Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Local to General 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, 2004). 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. RESULTS

Table 2 lists the approximate P-wave velocities and depths calculated from the seismic refraction traverses conducted during the evaluation. The approximate locations of the seismic refraction traverses are shown on the Seismic Line Location Maps (Figures 2a – 2c). The layer velocity profiles are included in Figures 4a through 4j. It should also be noted that, as a general rule, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line.

Table 2 – Seismic Traverse Results

Traverse No. And Length	P-wave Velocity feet/second	Approximate Depth to Bottom of Layer in feet	Apparent Rippability*
SL-1 150 feet	V1 = 3,650 V2 = 4,450	1 – 9 ---	Moderate Difficult, Possible Blasting
SL-2 150 feet	V1 = 2,350 V2 = 3,400	2 – 7 ---	Moderate Moderate
SL-3 150 feet	V1 = 1,500 V2 = 3,150	0 – 3 ---	Easy Moderate
SL-4 150 feet	V1 = 3,300 V2 = 3,800	0 – 12 ---	Moderate Moderate
SL-5 270 feet	V1 = 3,000 V2 = 5,250	6 – 22 ---	Moderate Difficult, Possible Blasting
SL-6 200 feet	V1 = 2,250 V2 = 4,300	1 – 8 ---	Moderate Difficult, Possible Blasting
SL-7 150 feet	V1 = 3,150 V2 = 5,650	2 – 7 ---	Moderate Very Difficult, Probable Blasting
~~SL-8~~ ~~200 feet~~	~~V1 = 2,250~~ ~~V2 = 3,050~~	~~1 – 11~~ ~~---~~	~~Moderate~~ ~~Moderate~~
~~SL-9~~ ~~200 feet~~	~~V1 = 3,000~~ ~~V2 = 3,700~~ ~~V3 = 4,900~~	~~0 – 6~~ ~~36 – 42~~ ~~---~~	~~Moderate~~ ~~Moderate~~ ~~Difficult, Possible Blasting~~
SL-10 150 feet	V1 = 3,300 V2 = 4,850	1 – 5 ---	Moderate Difficult, Possible Blasting
SL-11 200 feet	V1 = 2,100 V2 = 3,150 V3 = 4,300	1 – 4 15 – 36 ---	Moderate Moderate Difficult, Possible Blasting
~~SL-12~~ ~~150 feet~~	~~V1 = 2,300~~ ~~V2 = 3,700~~	~~4 – 8~~ ~~---~~	~~Moderate~~ ~~Moderate~~
SL-13 150 feet	V1 = 2,050 V2 = 3,050 V3 = 4,300	0 – 6 10 – 20 ---	Moderate Moderate Difficult, Possible Blasting
SL-14 150 feet	V1 = 2,500 V2 = 3,850	4 – 7 ---	Moderate Moderate
SL-15 240 feet	V1 = 1,850 V2 = 3,200 V3 = 5,050	1 – 9 24 – 46 ---	Easy Moderate Difficult, Possible Blasting
SL-16 150 feet	V1 = 1,550 V2 = 3,000	0 – 5 ---	Easy Moderate
SL-17 150 feet	V1 = 2,050 V2 = 3,300	2 – 5 ---	Moderate Moderate

Table 2 – Seismic Traverse Results			
SL-18 270 feet	V1 = 1,450 V2 = 2,950	2 – 6 ---	Easy Moderate
SL-19 200 feet	V1 = 1,500 V2 = 3,150	2 – 8 ---	Easy Moderate

* Rippability criteria based on the use of a Caterpillar D-9 dozer ripping with a single shank

6. CONCLUSIONS

The results from our seismic survey revealed two to three distinct geologic layers at the locations surveyed. Based on our site observations and discussions with you, the onsite materials include fill, colluvium/topsoil, residuum, decomposed granitic rock, and weathered granitic rock.

During our site visit, we noted the presence of granitic rock outcrops, which indicate the presence of lateral variations in the subsurface materials. Furthermore, some scatter was noted in the first-arrivals, which also indicates the presence of inhomogeneities in the subsurface materials. Accordingly, variability in the excavatability (including excavation depth) of the subsurface materials should be expected across the project area. It should also be noted that our general depth of exploration is on the order of 40 to 60 feet; therefore, higher velocity material should be expected beyond these depths.

Based on our results, difficult conditions where blasting may be required to obtain proposed excavation depths may be encountered depending on the location, excavation depth, and desired rate of production. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment, production rate, and oversized materials.

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, expressed 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.

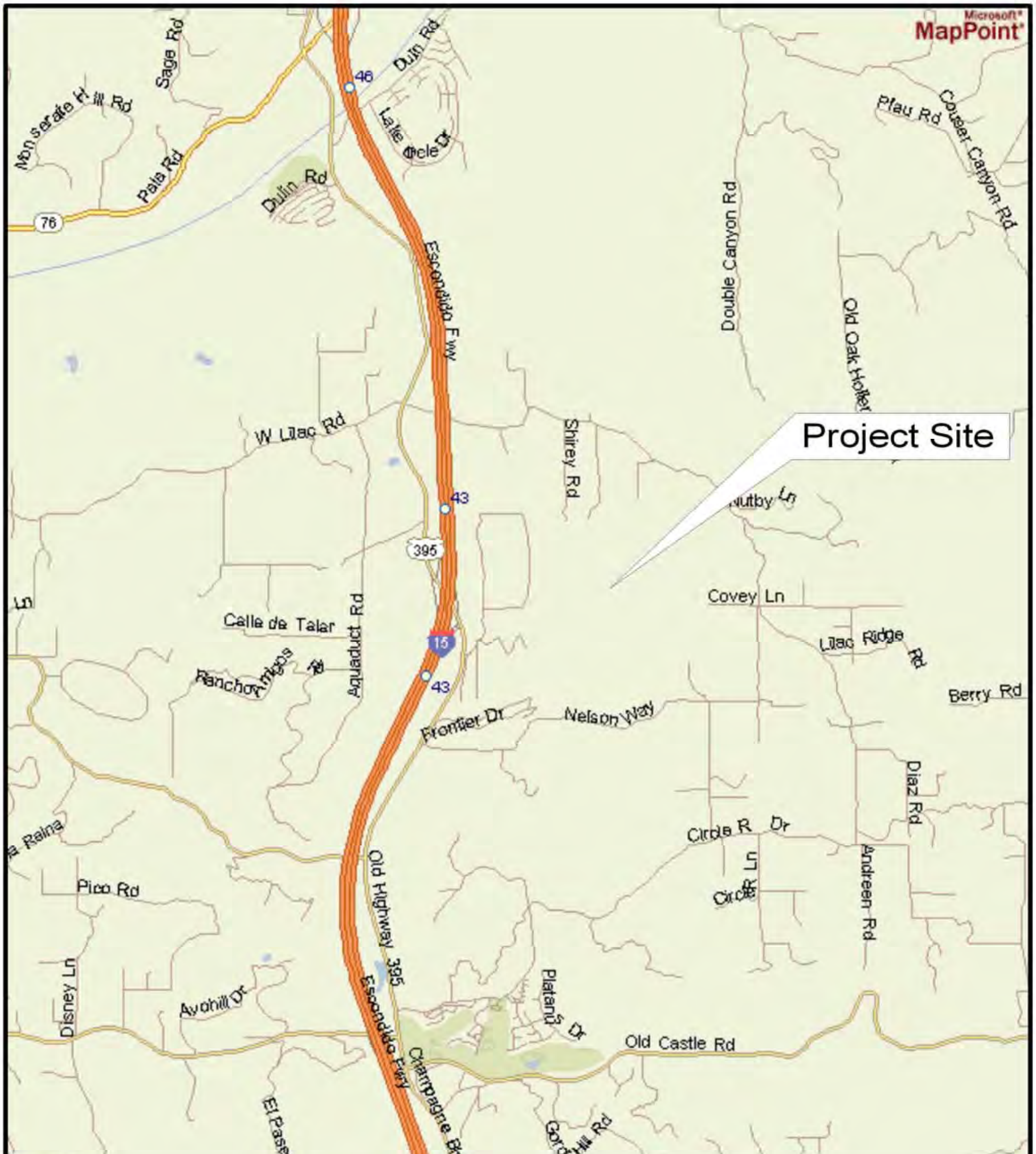
8. SELECTED REFERENCES

Caterpillar, Inc., 2004, Caterpillar Performance Handbook, Edition 35, Caterpillar, Inc., Peoria, Illinois.

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

Rimrock Geophysics, 2003, Seismic Refraction Interpretation Programs (SIPwin), V-2.76.

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



SITE LOCATION MAP

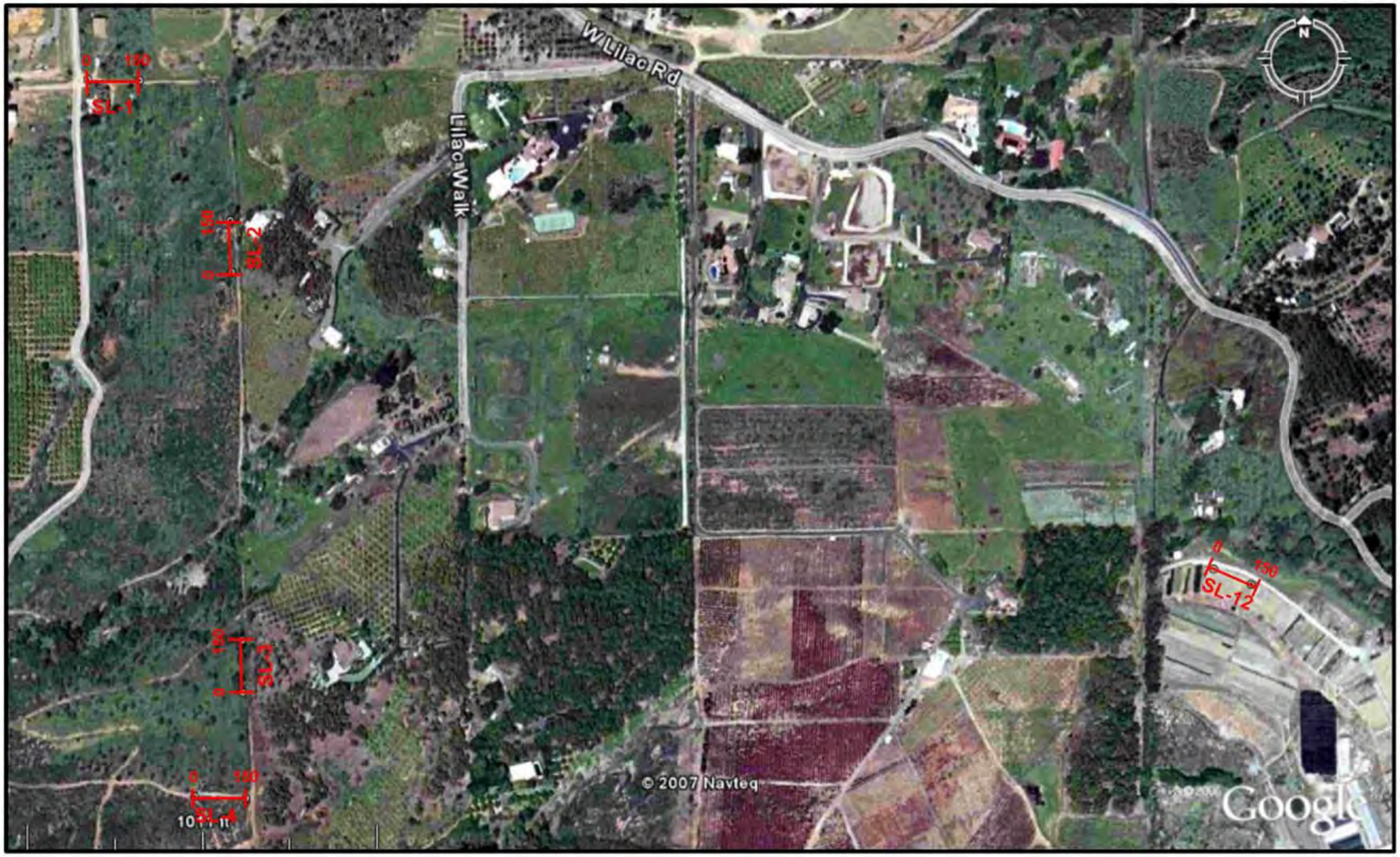
N

West Lilac Road and Covey Lane
Escondido, California

Project No.: 107060 | Date: 05/07

SOUTHWEST GEOPHYSICS INC.

Figure 1



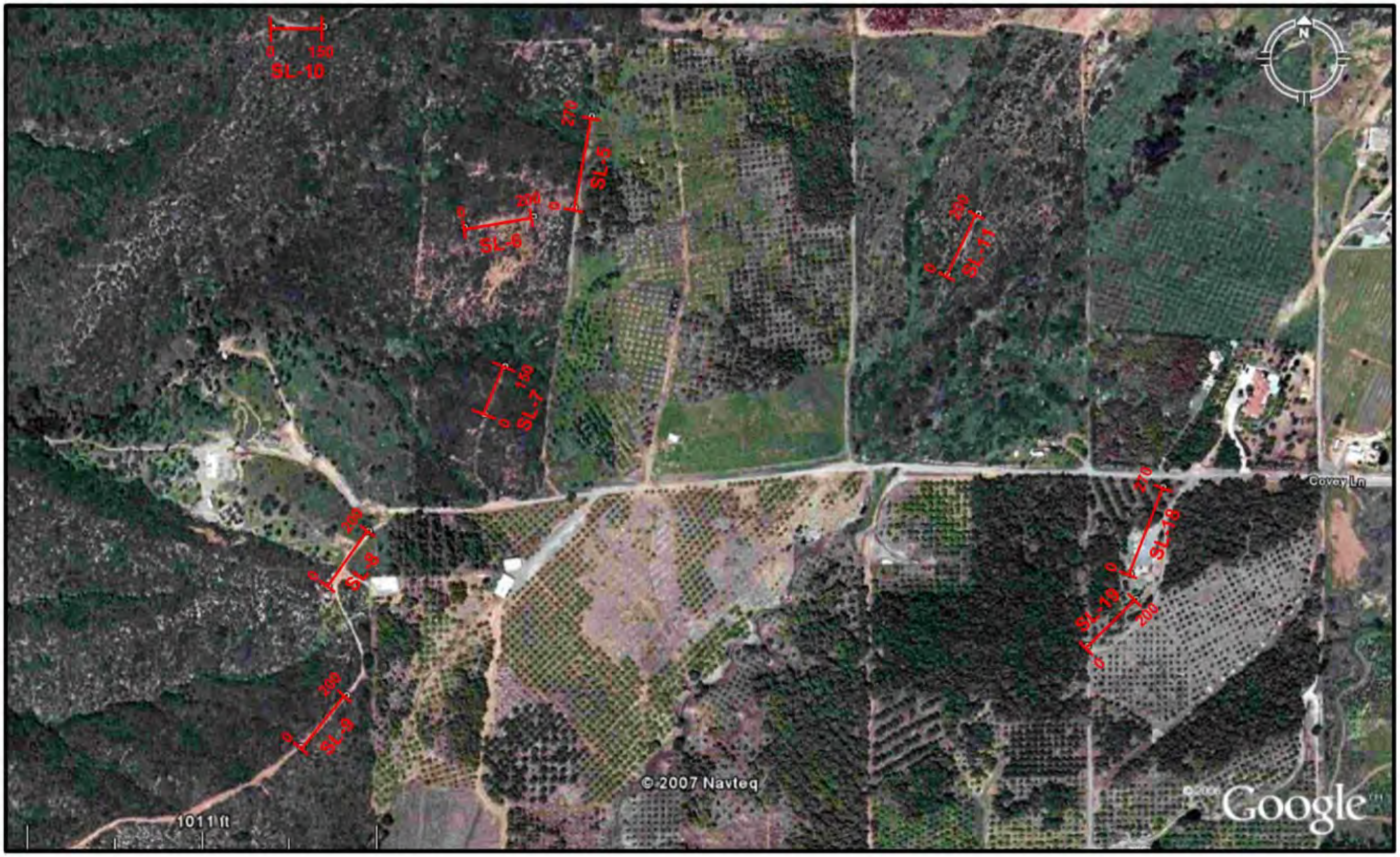
LEGEND

Seismic Line 0 SL-12 150

0 400 800

approximate scale in feet

SEISMIC LINE LOCATION MAP	N	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 2a



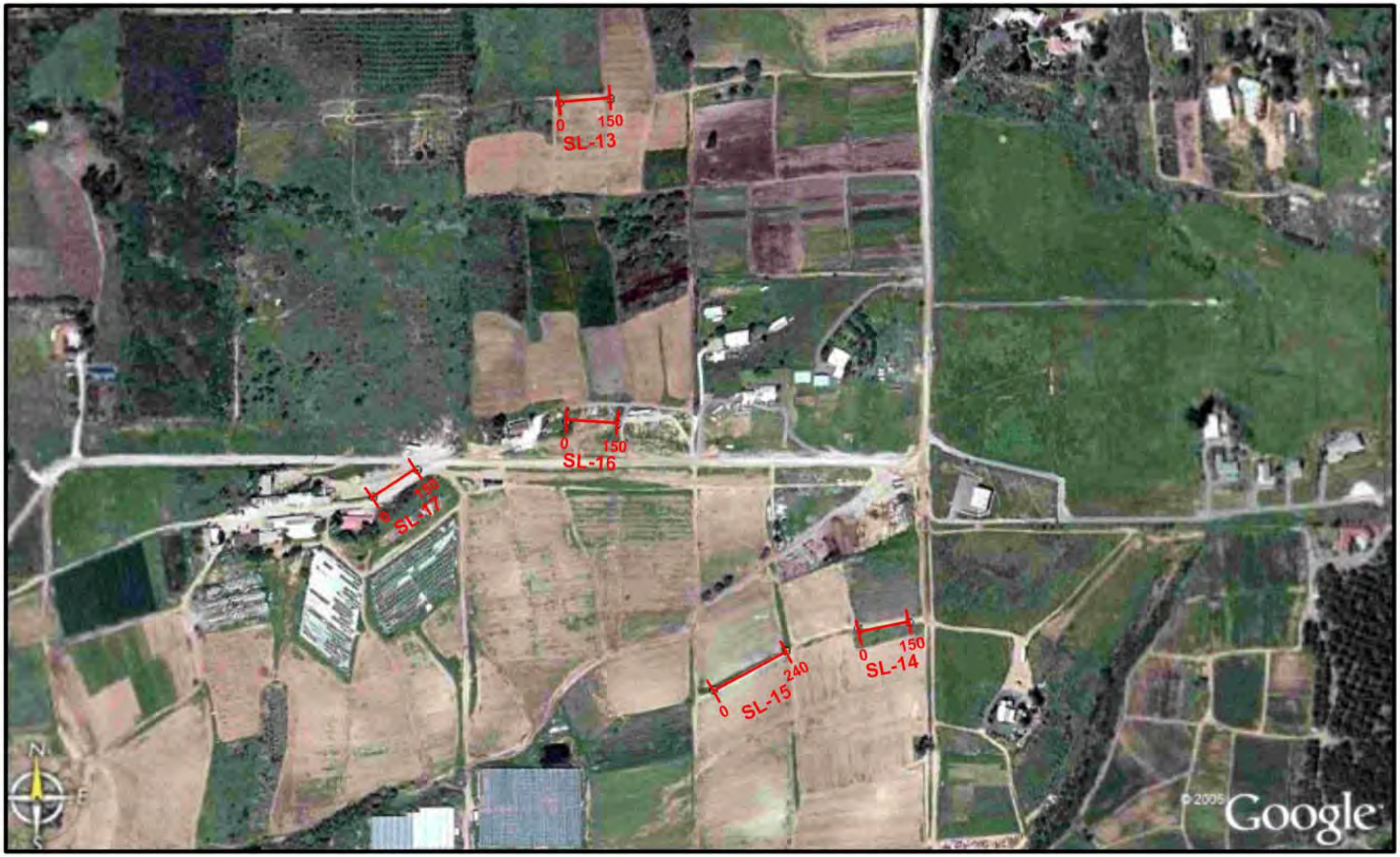
LEGEND

Seismic Line — 0 SL-19 150

0 400 800

approximate scale in feet

SEISMIC LINE LOCATION MAP	N	West Lilac Road and Covey Lane Escondido, California	SOUTHWEST GEOPHYSICS INC. Figure 2b
		Project No.: 107060 / Date: 05/07	



LEGEND

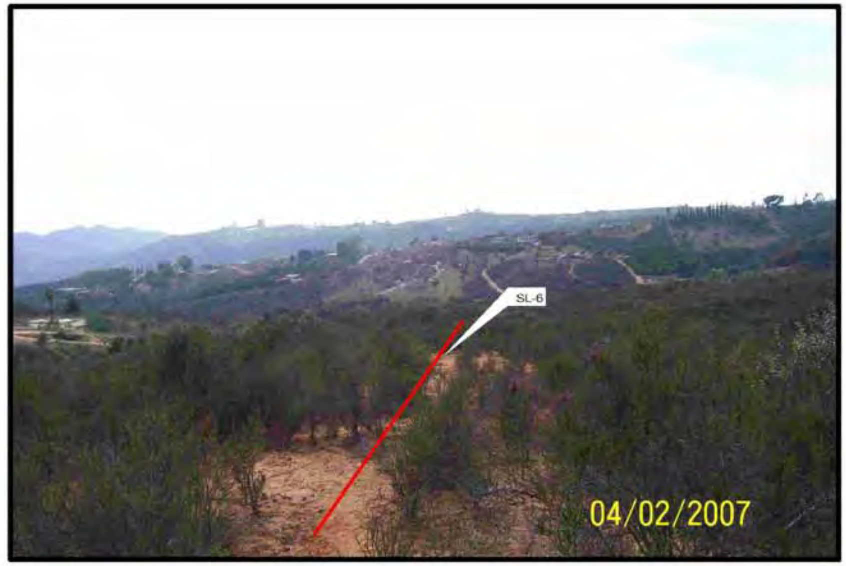
Seismic Line 0 SL-17 270

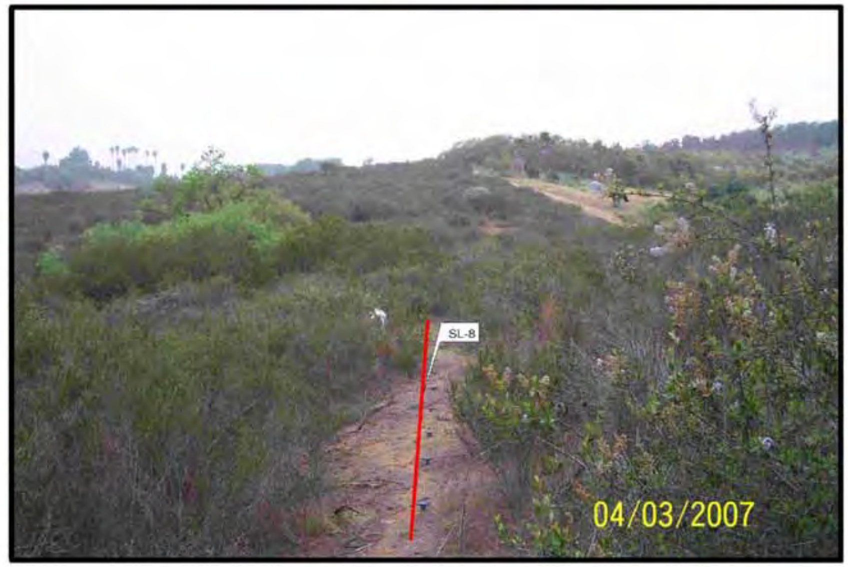
0 400 800

approximate scale in feet

SEISMIC LINE LOCATION MAP	N	West Lilac Road and Covey Lane Escondido, California	SOUTHWEST GEOPHYSICS INC. Figure 2c
		Project No.: 107060 Date: 05/07	

SITE PHOTOGRAPHS	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 3a

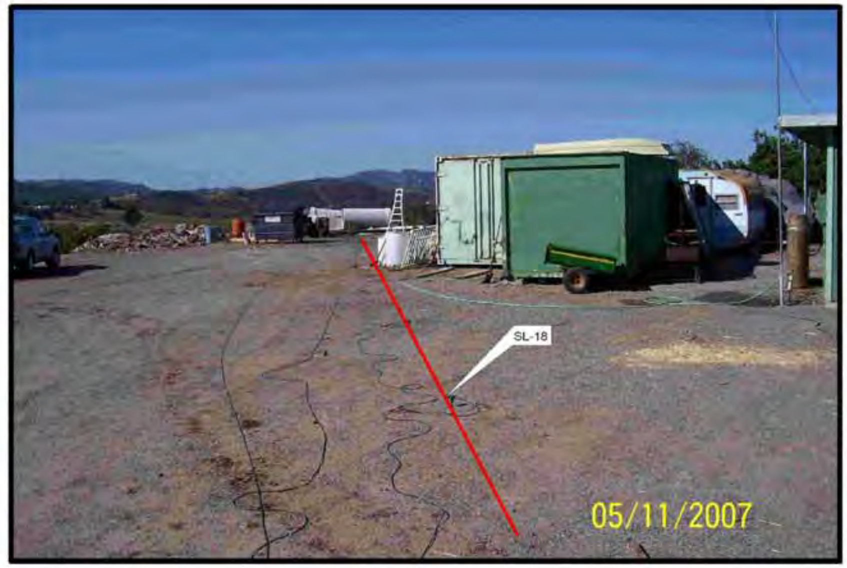




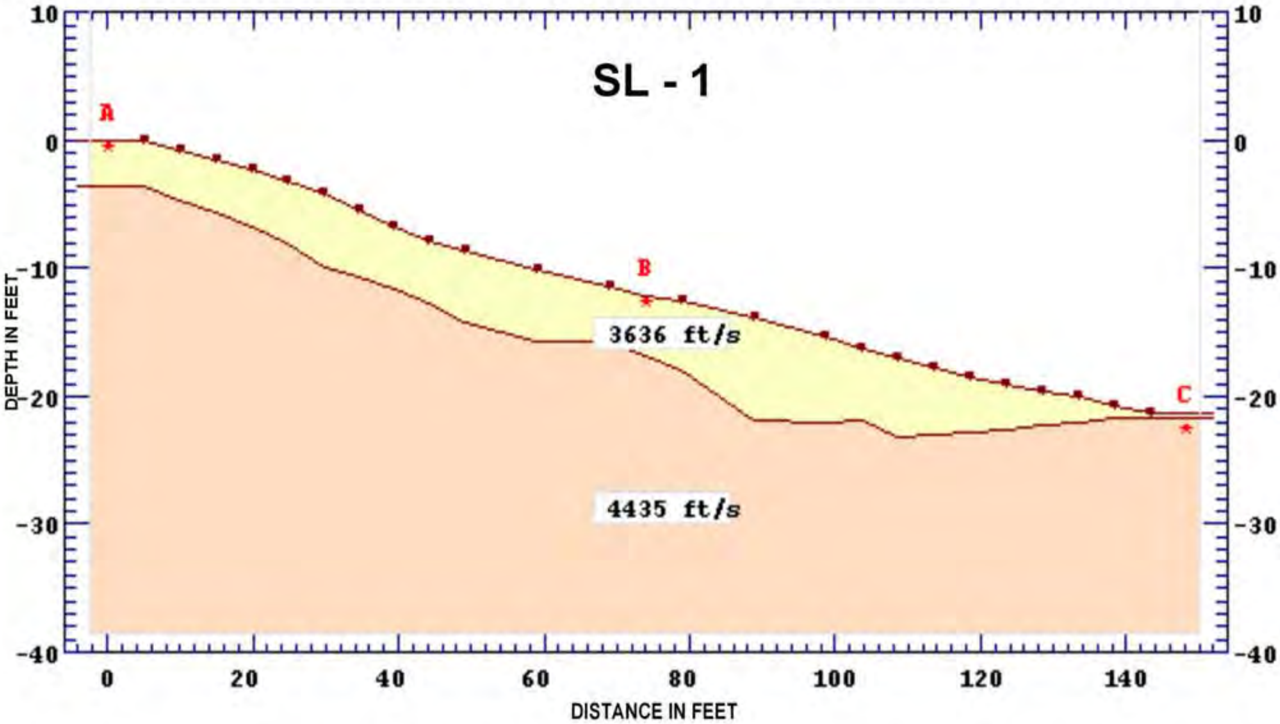
SITE PHOTOGRAPHS	West Lilac Road and Covey Lane Escondido, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 107060 — Date: 05/07	Figure 3b

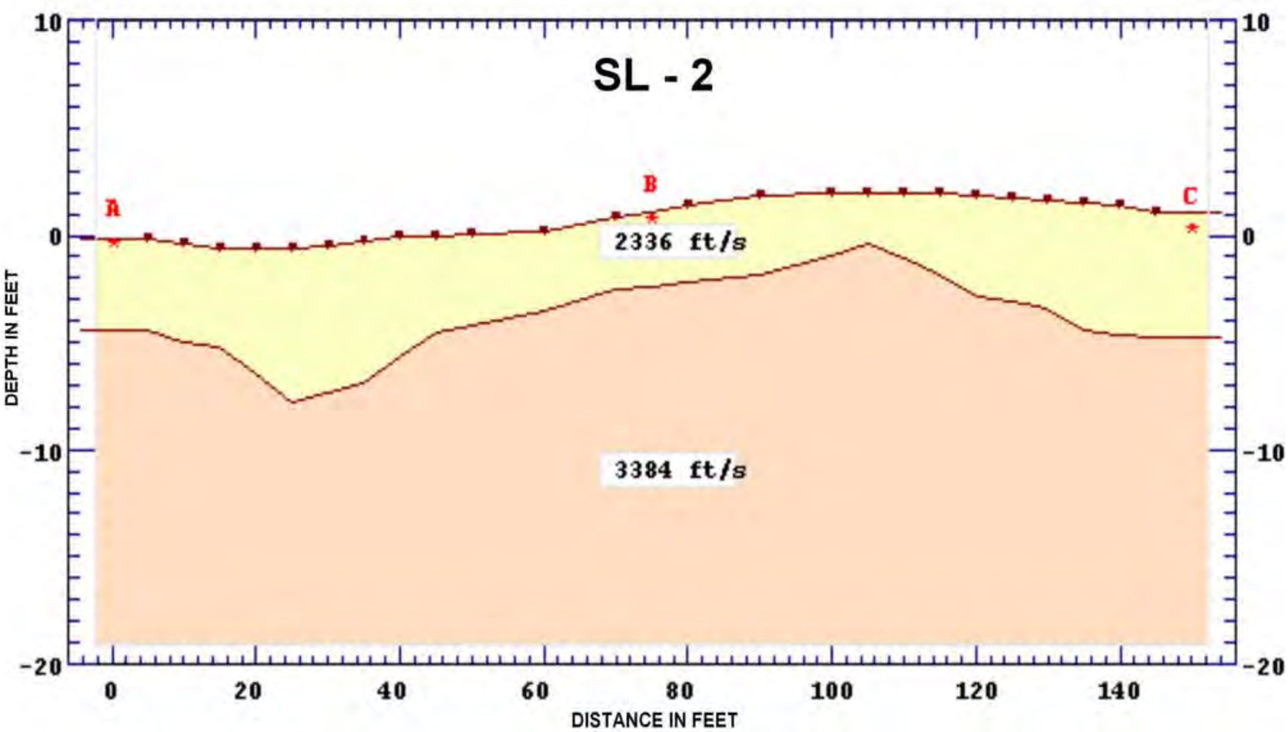
SITE PHOTOGRAPHS	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 3c

SITE PHOTOGRAPHS	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 3d

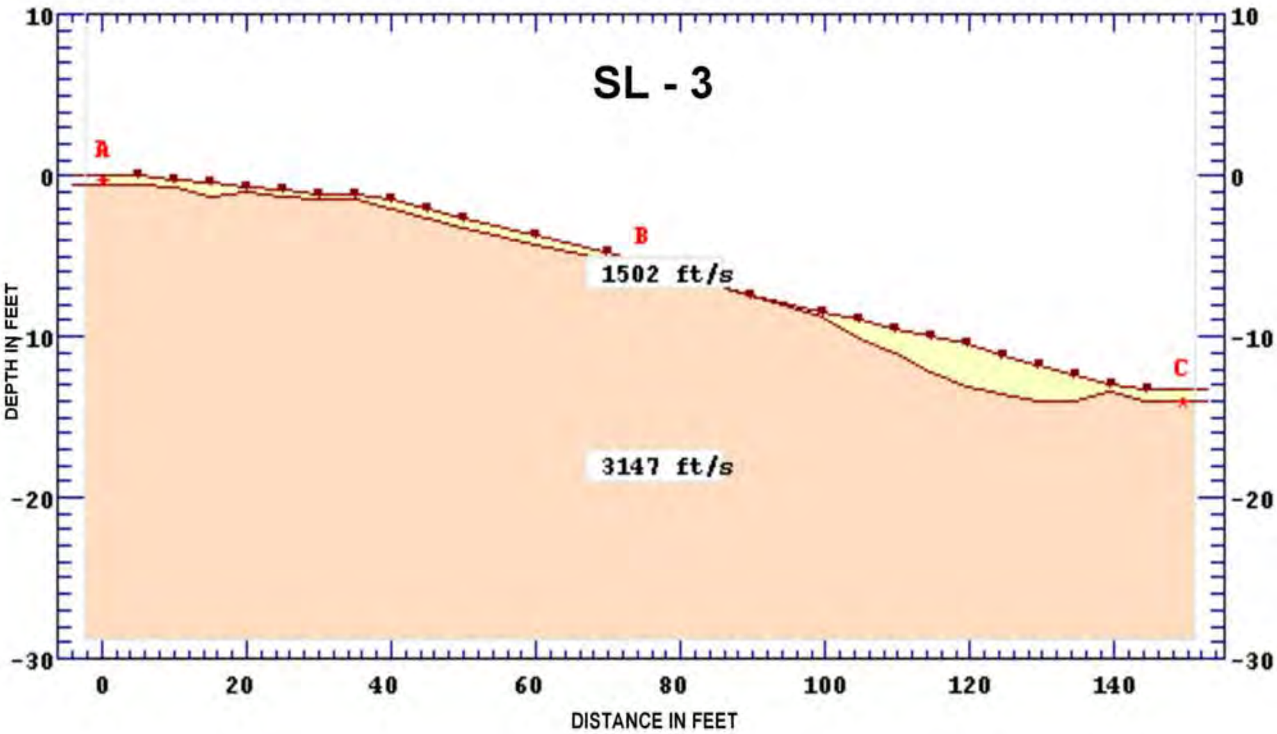


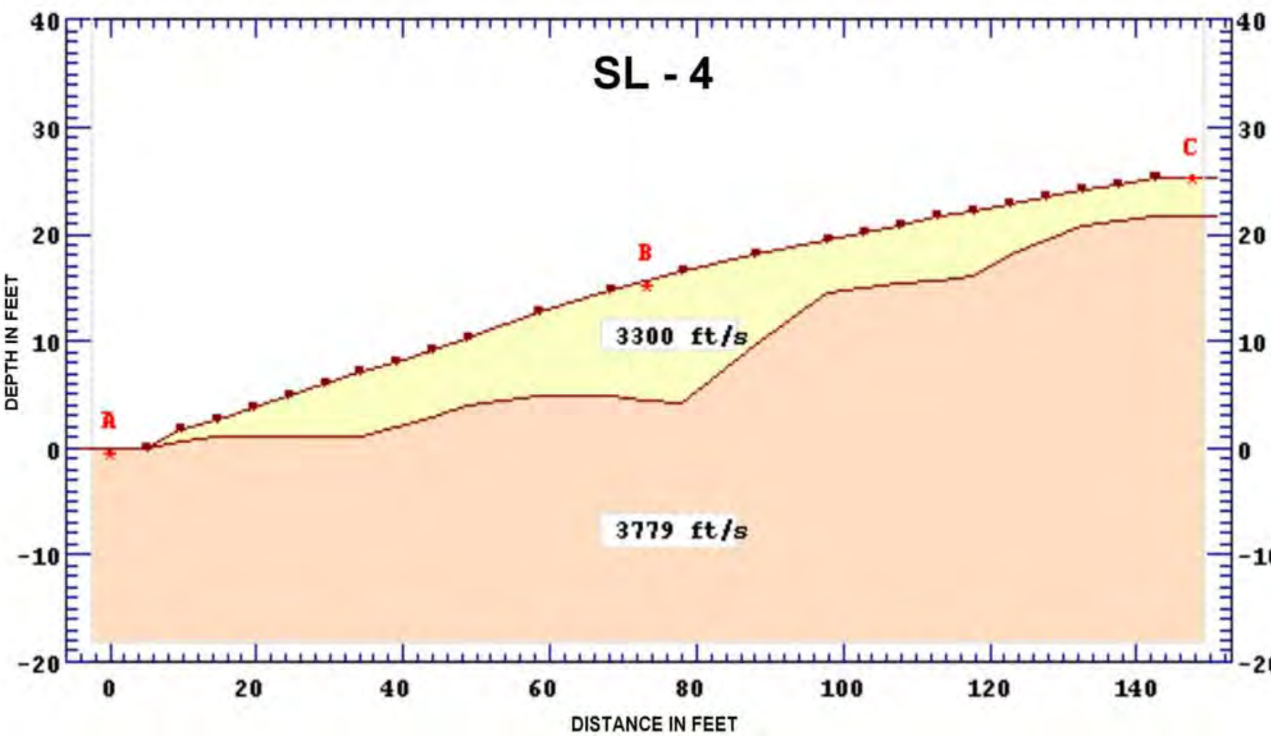
SITE PHOTOGRAPHS	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 3e



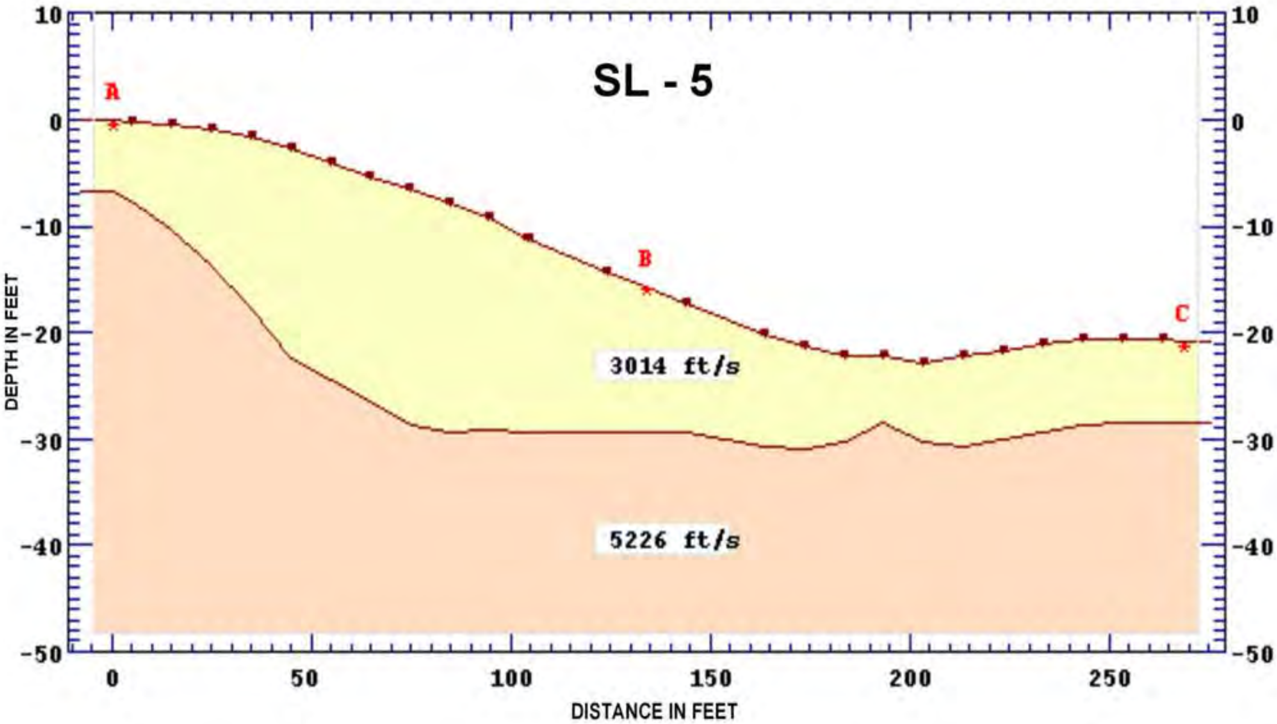


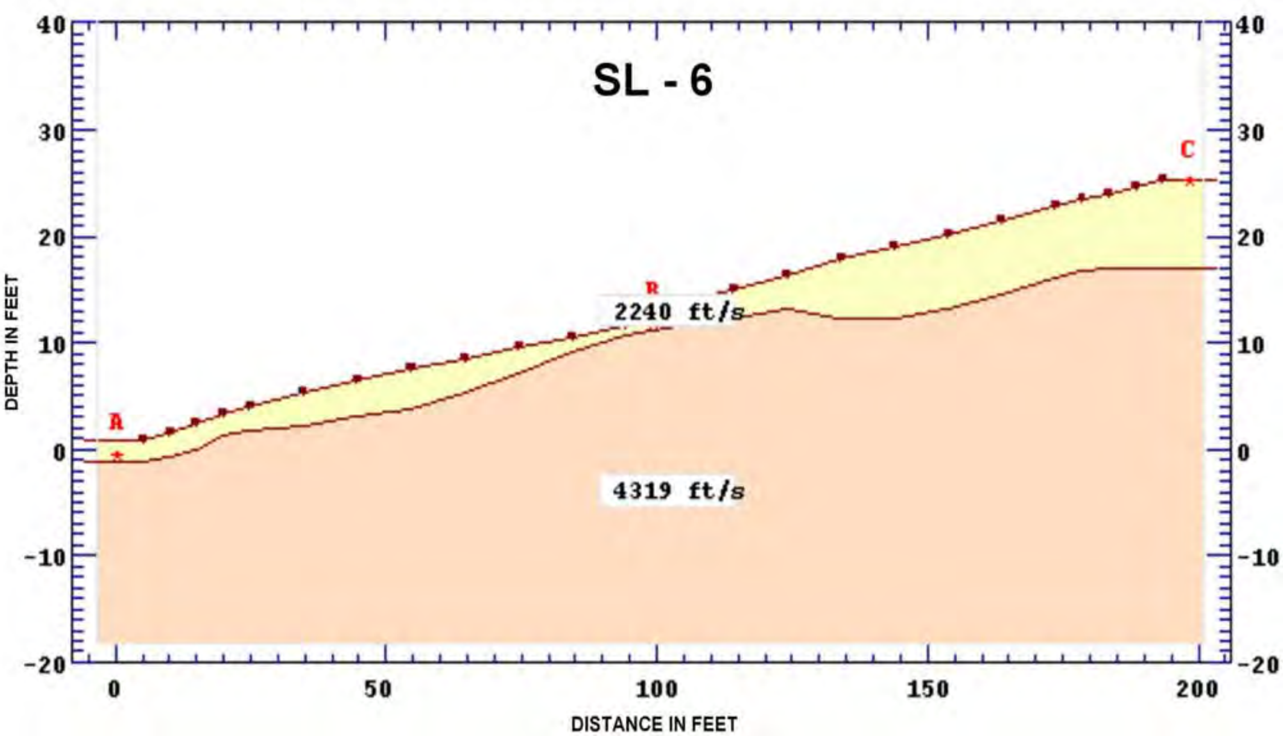
SEISMIC PROFILES, SL-1 AND SL-2	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 4a



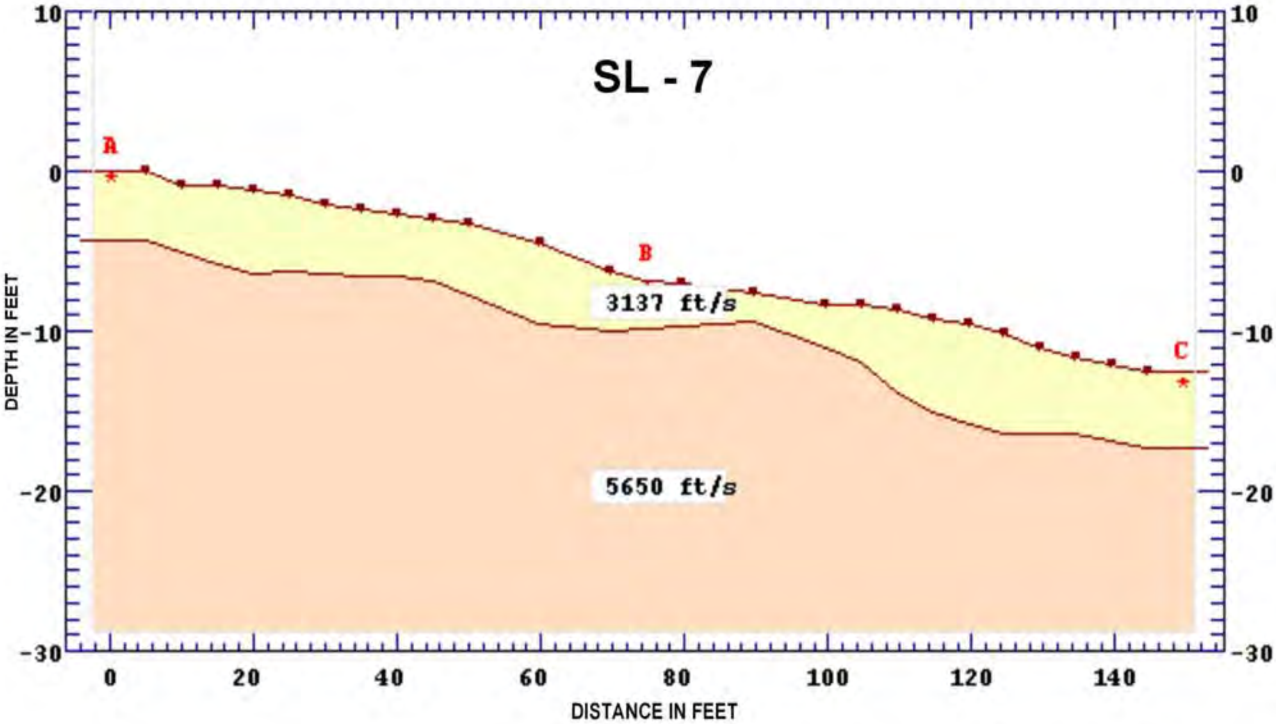


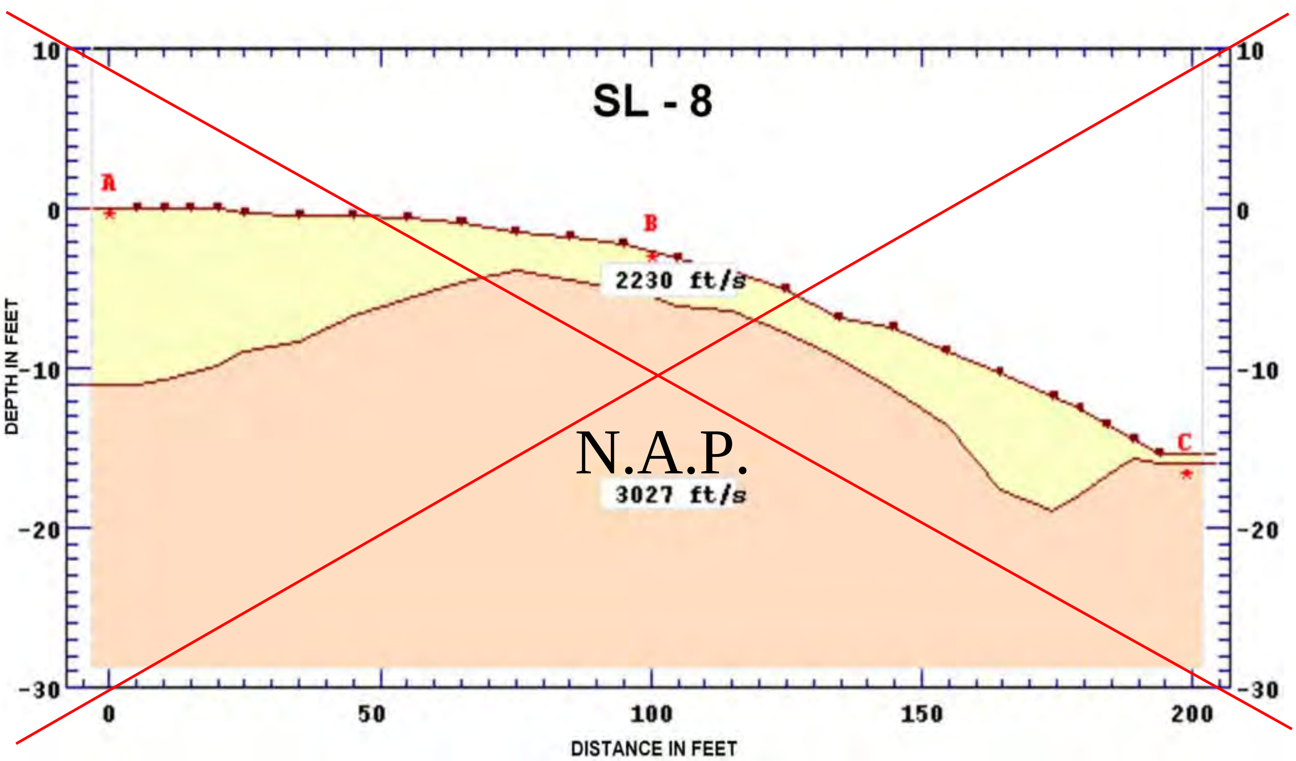
SEISMIC PROFILES, SL-3 AND SL-4	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 4b



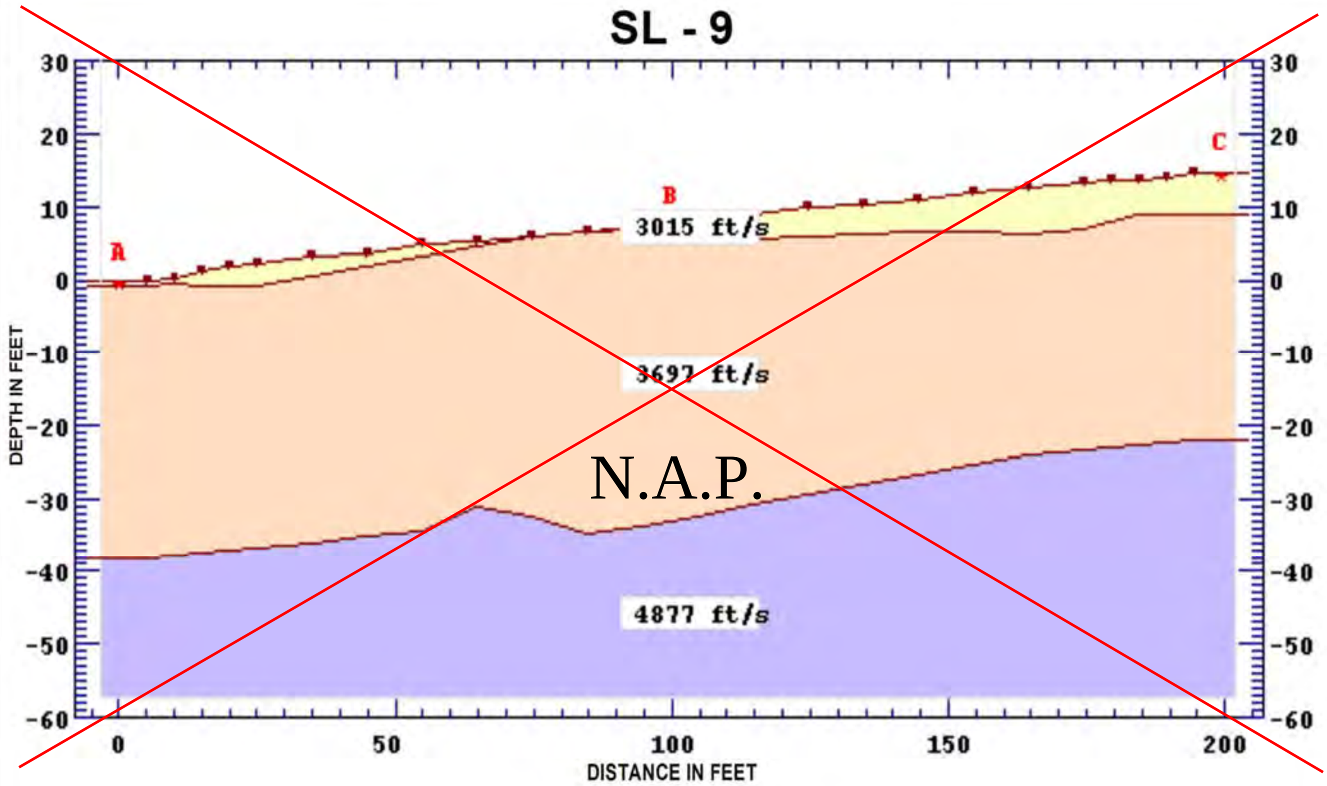


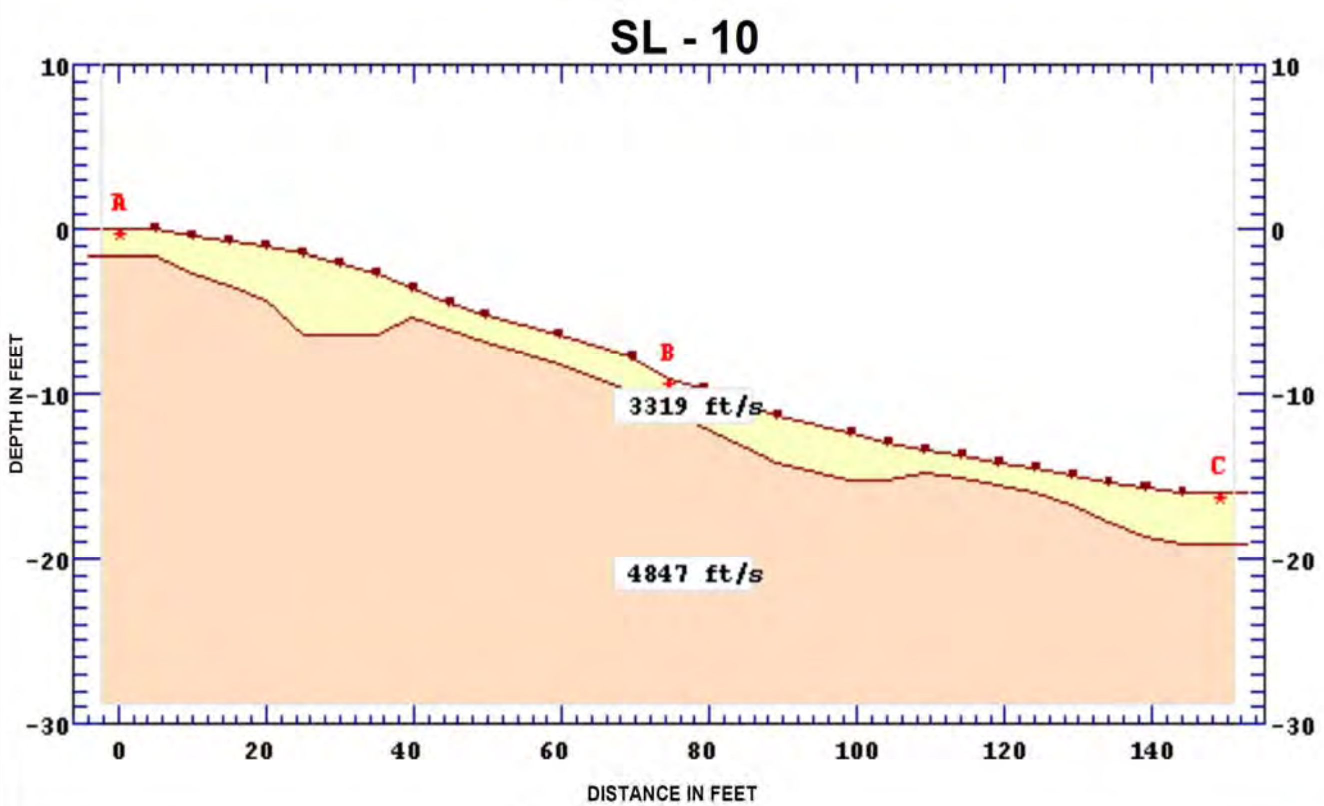
SEISMIC PROFILES, SL-5 AND SL-6	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 4c



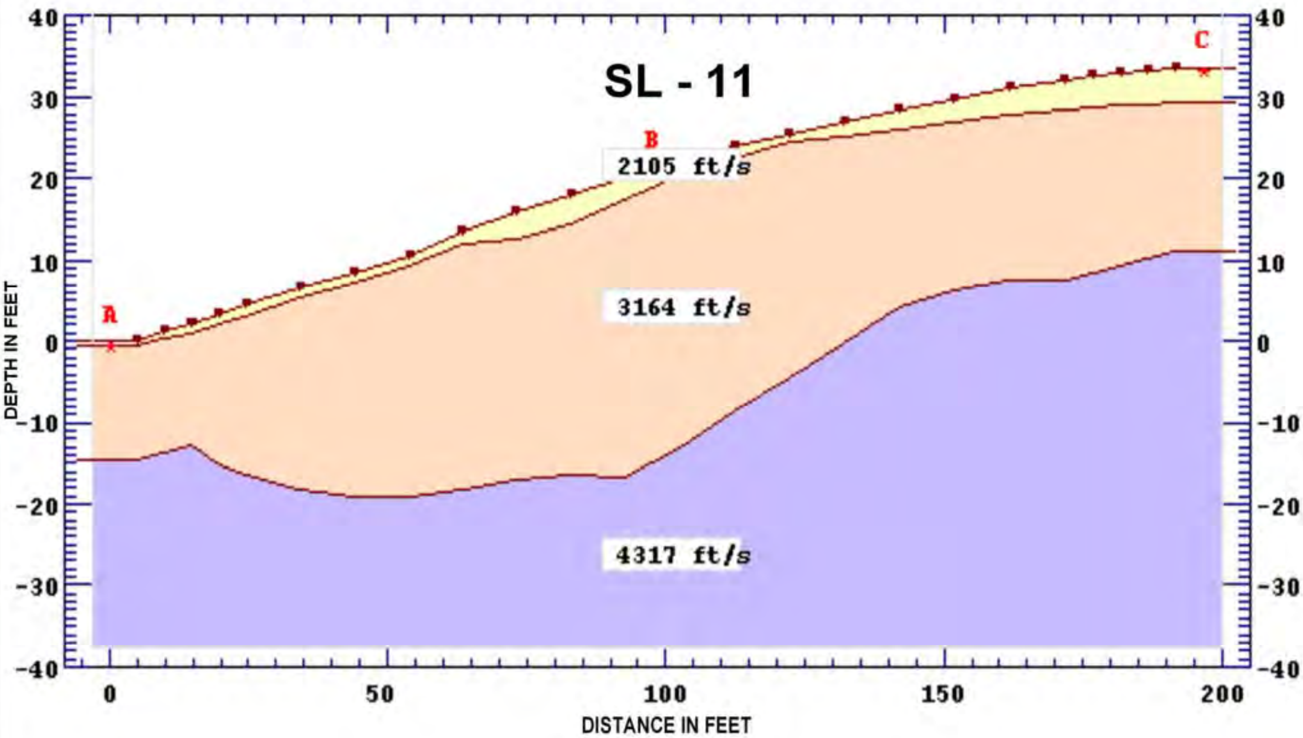


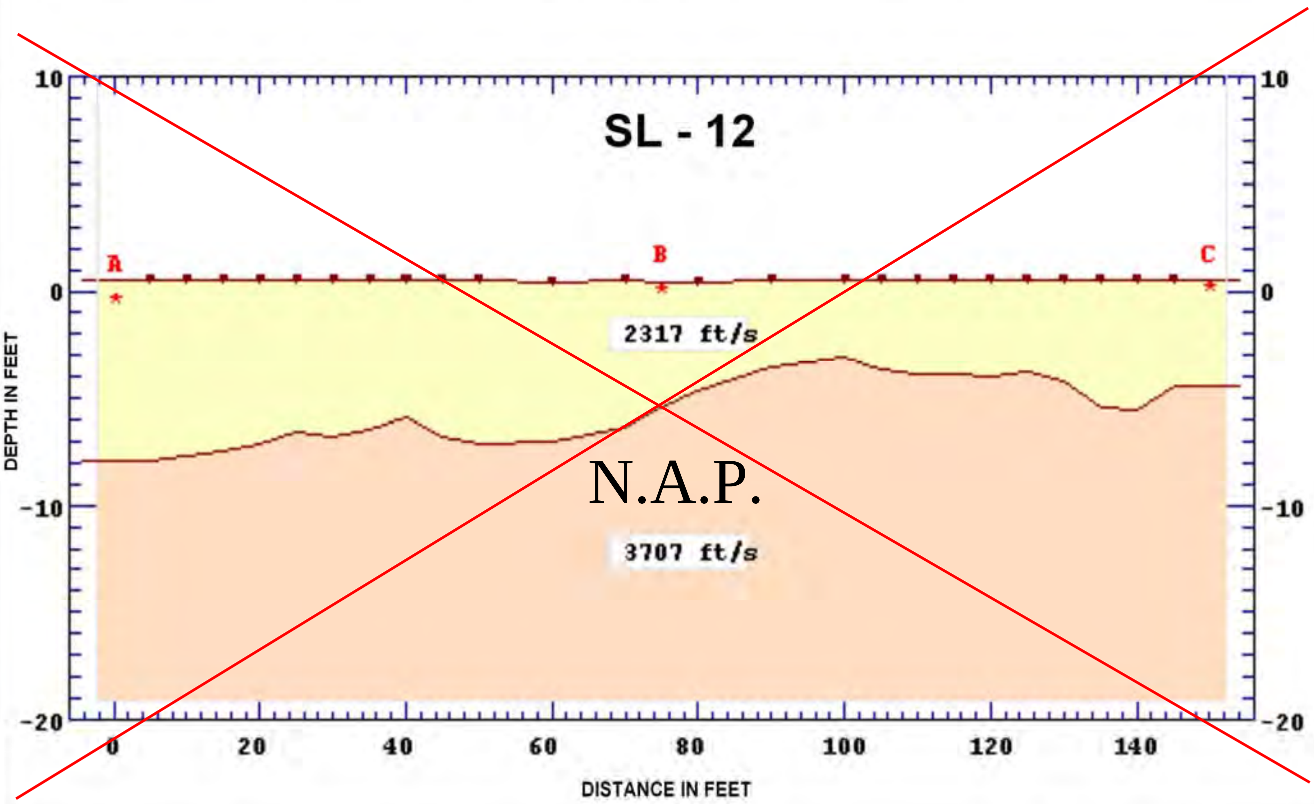
SEISMIC PROFILES, SL-7 AND SL-8	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 4d





SEISMIC PROFILES, SL-9 AND SL-10	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 Date: 05/07	SOUTHWEST GEOPHYSICS INC Figure 4e





SEISMIC PROFILES, SL-11 AND SL-12

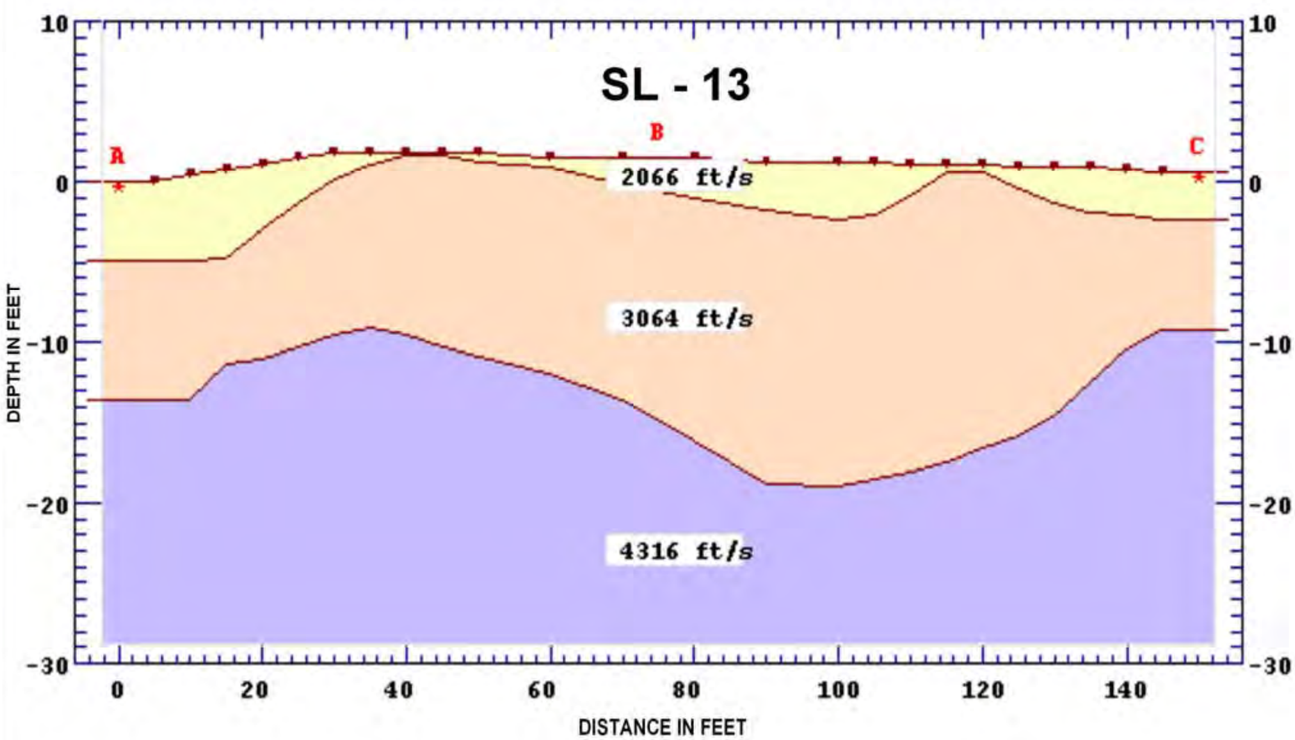
West Lilac Road and Covey Lane
Escondido, California

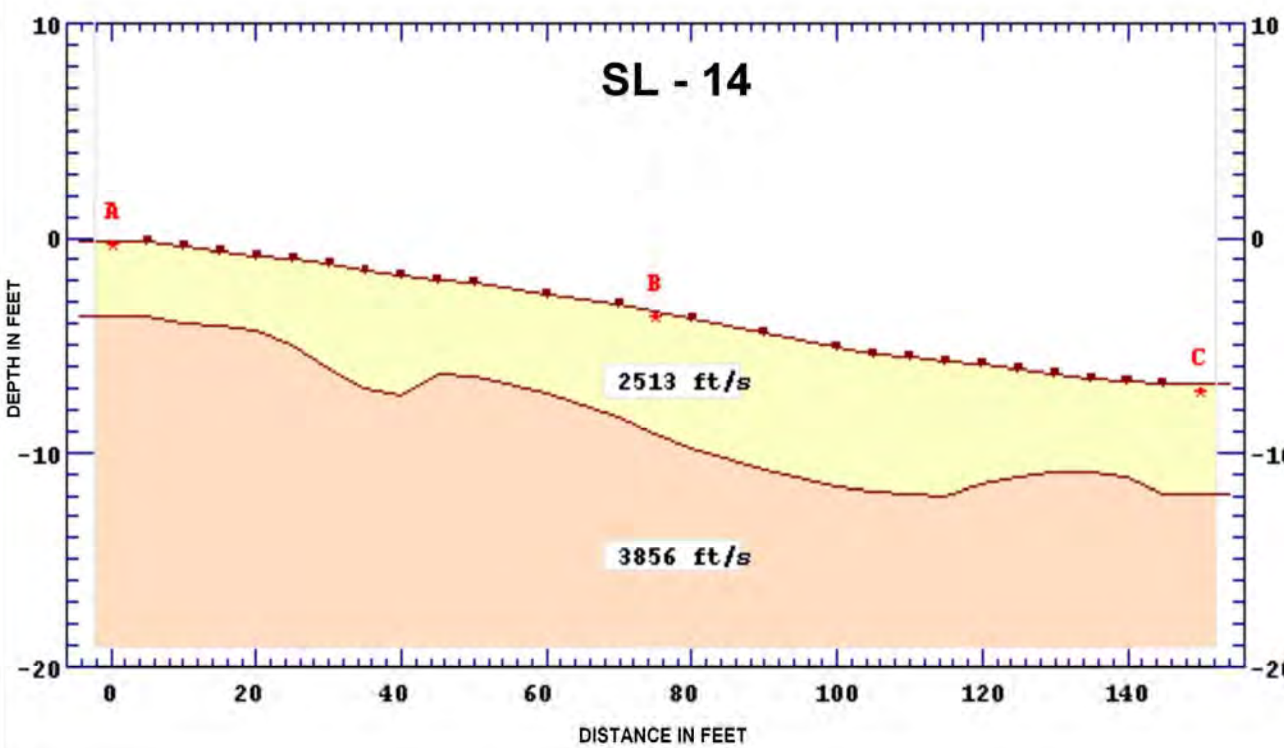
Project No.: 107060

Date: 05/07

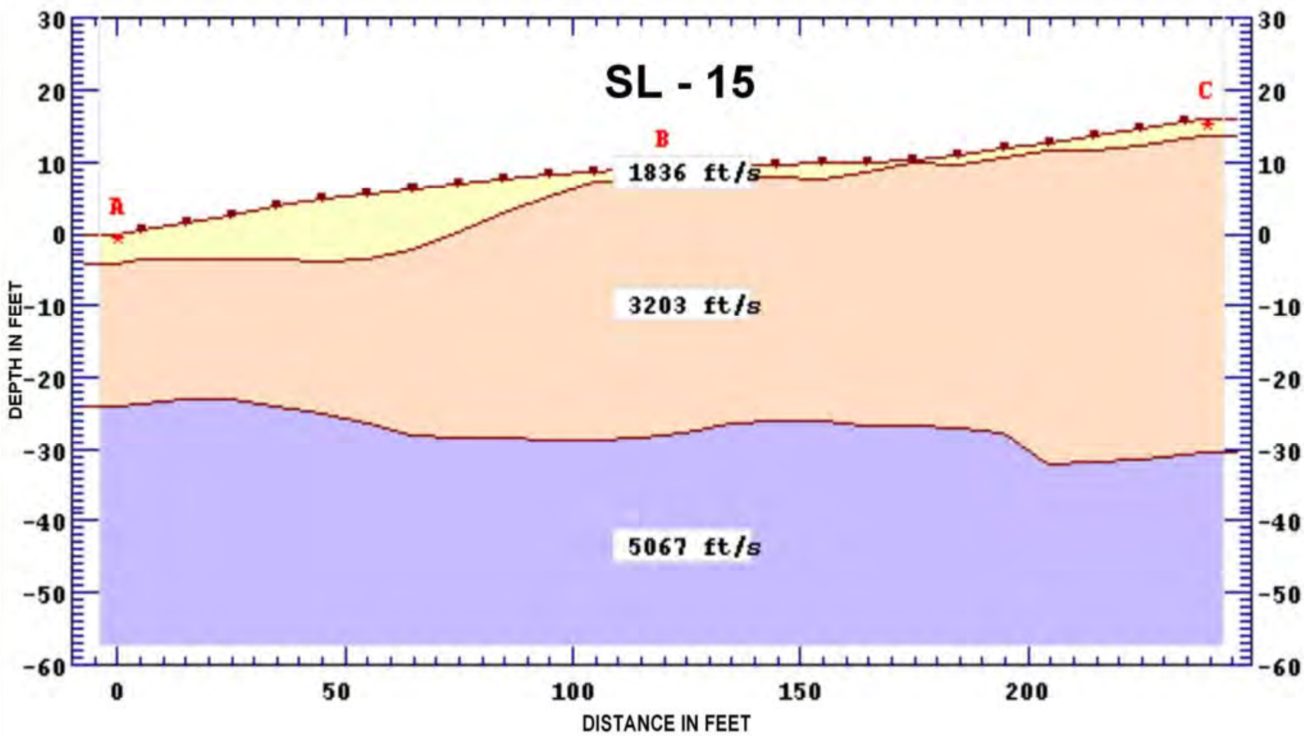
SOUTHWEST GEOPHYSICS INC.

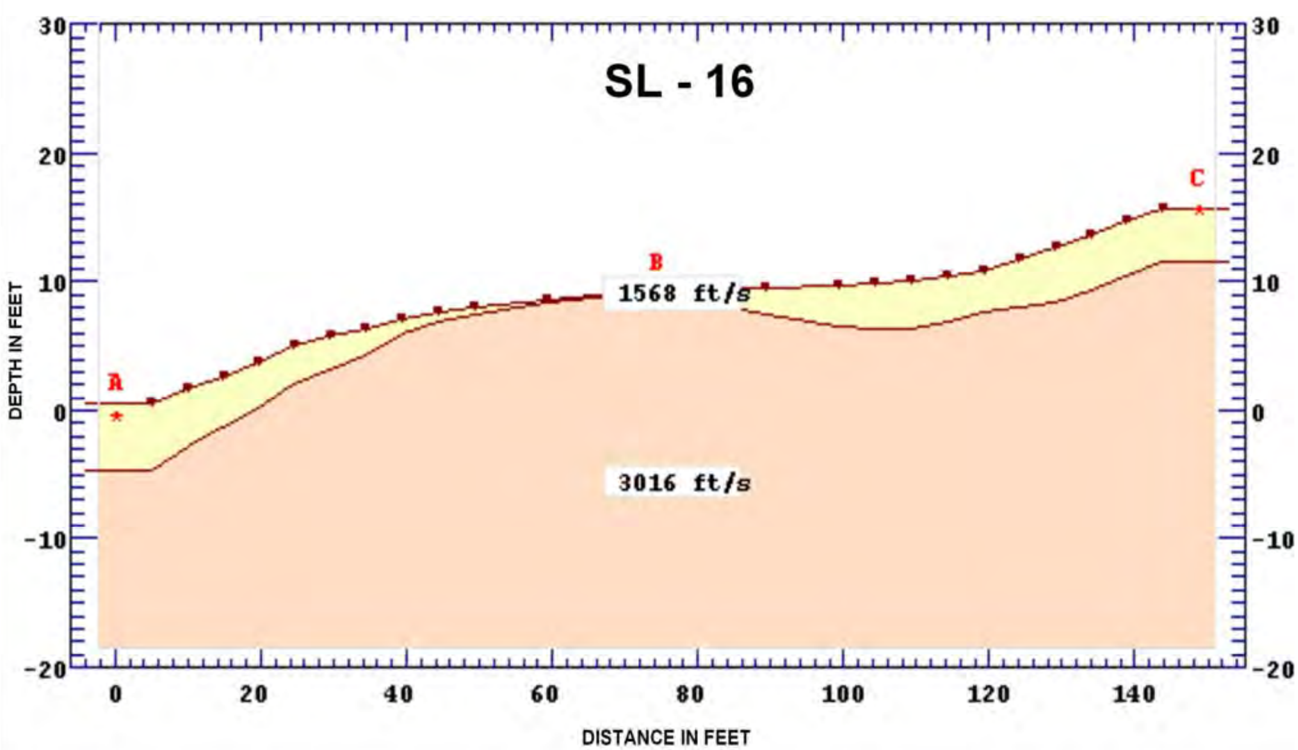
Figure 4f





SEISMIC PROFILES, SL-13 AND SL-14	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 Date: 05/07	SOUTHWEST GEOPHYSICS INC Figure 4g





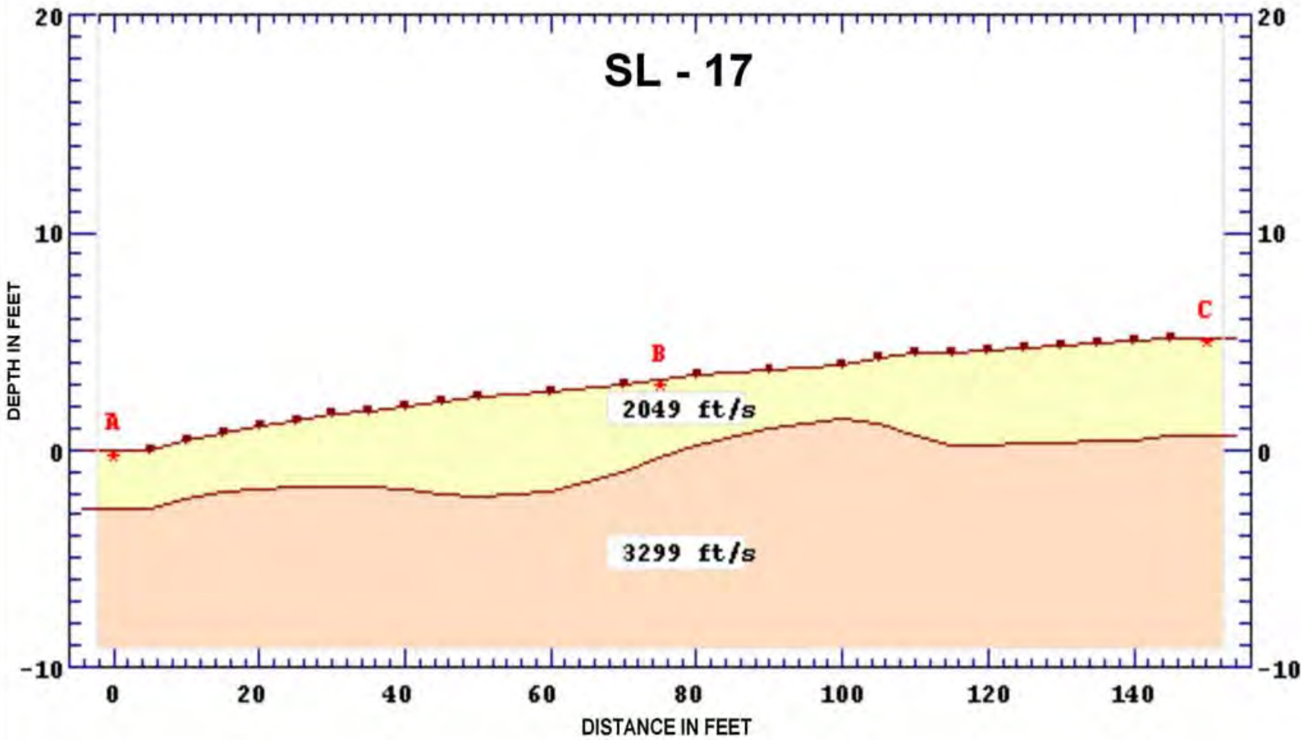
SEISMIC PROFILES, SL-15 AND SL-16

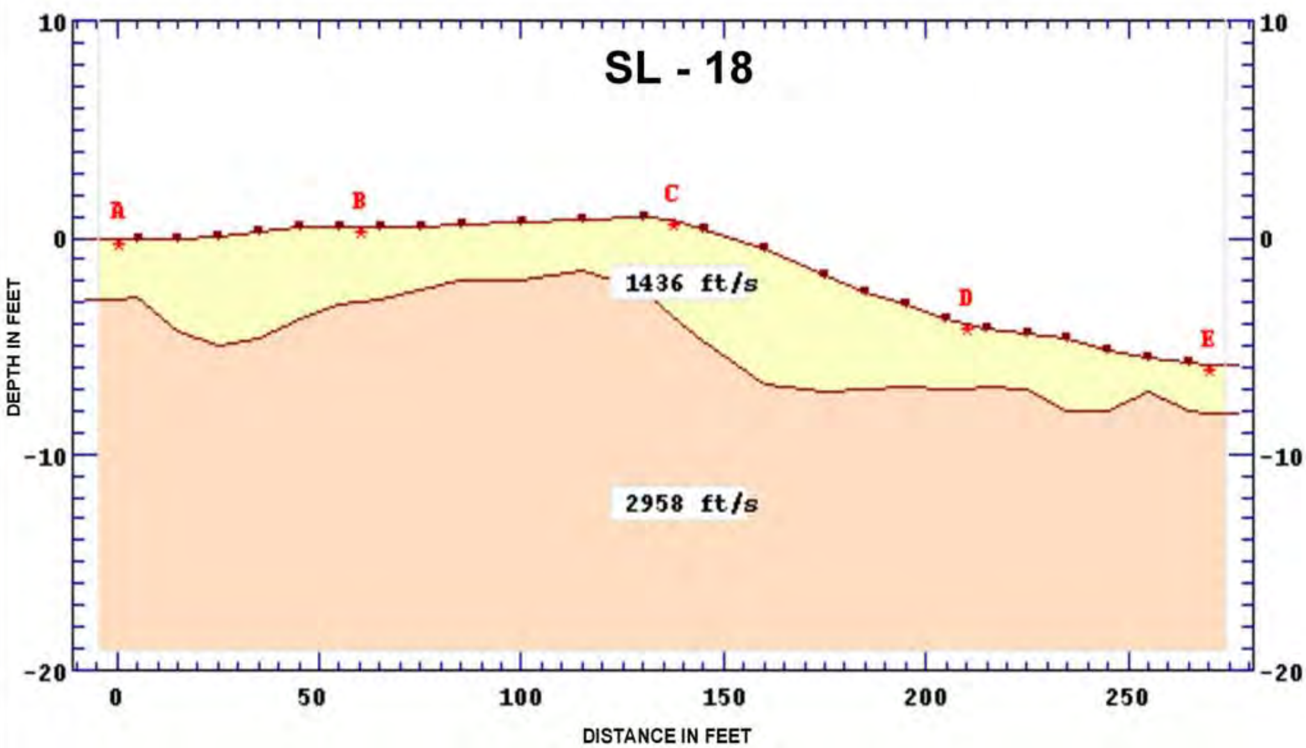
West Lilac Road and Covey Lane
Escondido, California

Project No.: 107060 | Date: 05/07

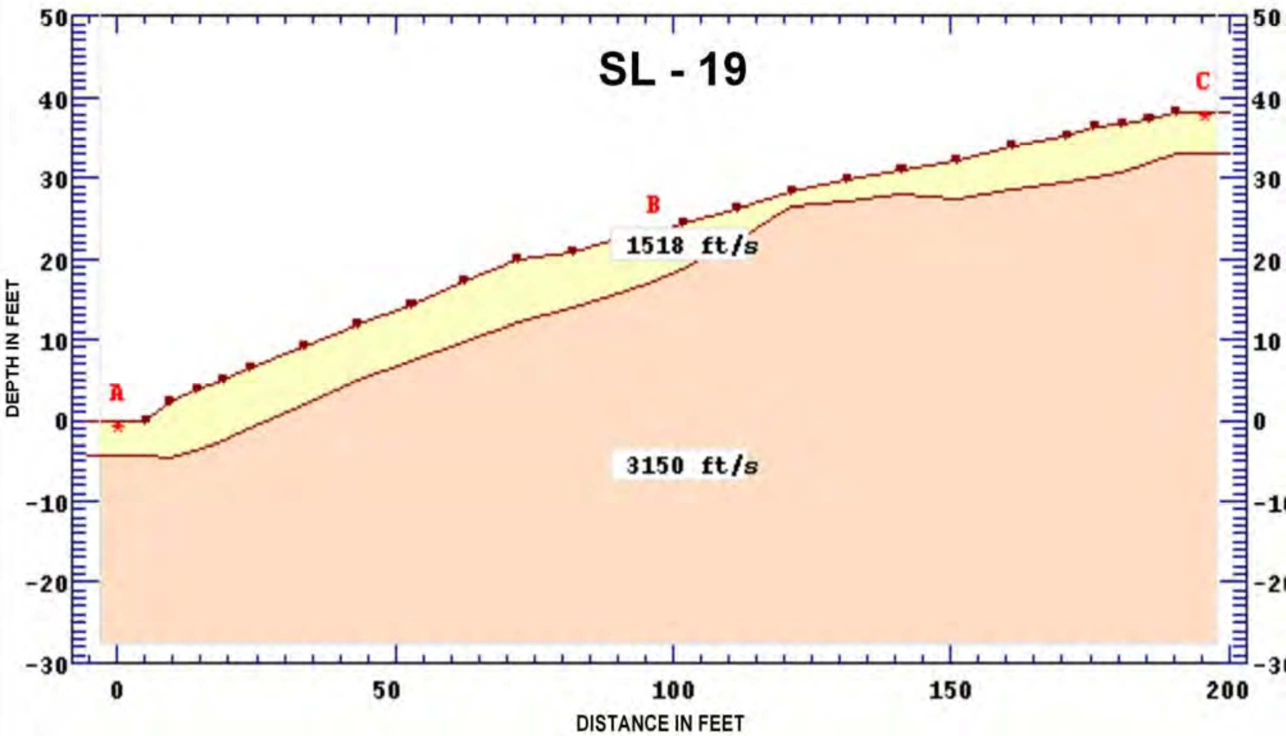
SOUTHWEST GEOPHYSICS INC.

Figure 4h





SEISMIC PROFILES, SL-17 AND SL-18	West Lilac Road and Covey Lane Escondido, California Project No.: 107060 — Date: 05/07	SOUTHWEST GEOPHYSICS INC. Figure 4i



SEISMIC PROFILE, SL-19	West Lilac Road and Covey Lane Escondido, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 107060 | Date: 05/07	Figure 4j

APPENDIX C

LABORATORY TEST RESULTS

LABORATORY DATA

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
MARCH 2012

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

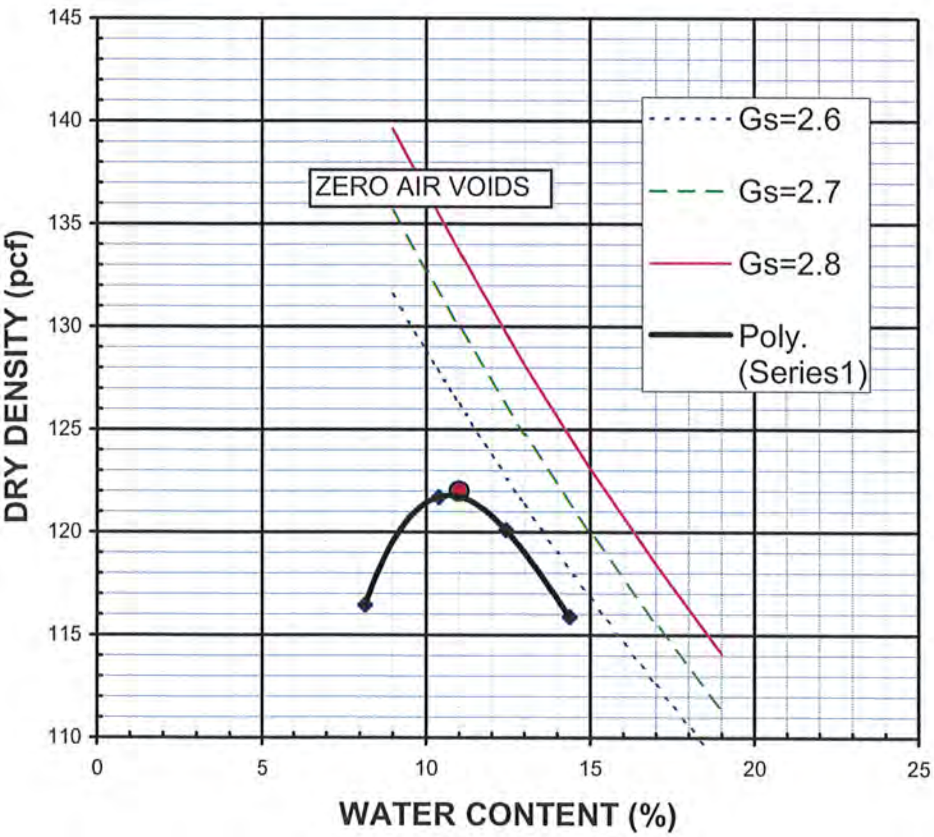
STANDARD TEST METHOD FOR MOISTURE - DENSITY RELATIONSHIP (ASTM D1557)

REV Dec-11

PROJECT: AGS
PROJECT NO.: SD-248
TESTED BY: RJP
SAMPLE DESCRIPTION: Dark yellowish brown clayey sand (SC)
SAMPLE ID: TP5 @ 6-8 ft
DATE: January 10, 2011
CHECKED BY: Chad Davis

A) WATER ADDED	50	100	150	200			milliliters
B) MOLD TARE WEIGHT	1966.4	1966.4	1966.4	1966.4			grams
C) WEIGHT OF WET SOIL AND MOLD	3871.0	3997.6	4008.3	3970.5			grams
D) WET SOIL WEIGHT (C - B)	1904.6	2031.2	2041.9	2004.1			grams
E) WET DENSITY (D / V)	126.0	134.3	135.0	132.5			pcf
F) DRY DENSITY (E / [(L/100) + 1])	116.5	121.7	120.1	115.9			pcf

G) TARE WEIGHT	139.2	153.7	139.9	150.1			grams
H) WEIGHT OF WET SOIL AND TARE	869.3	880.3	879.3	802.2			grams
I) WEIGHT OF DRY SOIL AND TARE	814.2	812.0	797.5	720.2			grams
J) WEIGHT OF WATER (H - I)	55.1	68.3	81.8	82.0			grams
K) DRY WEIGHT OF SOIL (I - G)	675.0	658.3	657.6	570.1			grams
L) MOISTURE CONTENT (J / K * 100)	8.2	10.4	12.4	14.4			percent



VOLUME CORRECTION (V):
4 INCH (4-1): V=15.120 pcf/gm
4 INCH (4-3): V=15.026 pcf/gm
4 INCH (4-4): V=15.090 pcf/gm

6 INCH: V=34.020 pcf/gm)

A	METHOD USED (A,B or C)

4-1	MOLD USED
15.120	MOLD VOLUME CORRECTION (V)
No. 4	SIEVE NUMBER
0.0%	PERCENT RETAINED

122	**MAXIMUM DENSITY [PCF]**
11	**OPTIMUM MOISTURE [%]**

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

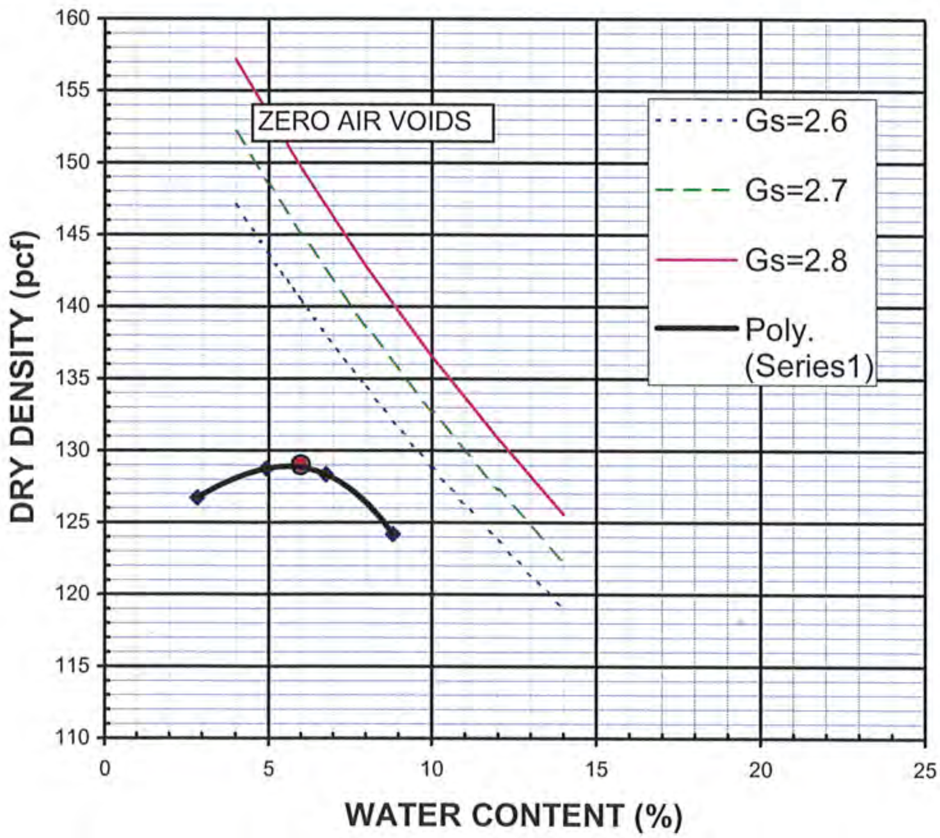
STANDARD TEST METHOD FOR MOISTURE - DENSITY RELATIONSHIP (ASTM D1557)

REV Dec-11

PROJECT: AGS
PROJECT NO.: SD-248
TESTED BY: RJP
SAMPLE DESCRIPTION: Dark yellowish brown silty sand / DG (SM)
SAMPLE ID: TP8 @ 7-9 ft
DATE: January 10, 2011
CHECKED BY: Chad Davis

A) WATER ADDED	50	100	150	200			milliliters
B) MOLD TARE WEIGHT	1966.3	1966.3	1966.3	1966.3			grams
C) WEIGHT OF WET SOIL AND MOLD	3936.9	4008.6	4038.3	4010.1			grams
D) WET SOIL WEIGHT (C - B)	1970.6	2042.3	2072.0	2043.8			grams
E) WET DENSITY (D / V)	130.3	135.1	137.0	135.2			pcf
F) DRY DENSITY (E / [(L/100) + 1])	126.7	128.7	128.3	124.2			pcf

G) TARE WEIGHT	138.9	140.0	153.7	150.0			grams
H) WEIGHT OF WET SOIL AND TARE	961.2	1046.4	1088.1	1176.0			grams
I) WEIGHT OF DRY SOIL AND TARE	938.4	1003.9	1028.8	1092.8			grams
J) WEIGHT OF WATER (H - I)	22.8	42.5	59.3	83.2			grams
K) DRY WEIGHT OF SOIL (I - G)	799.5	863.9	875.1	942.8			grams
L) MOISTURE CONTENT (J / K * 100)	2.9	4.9	6.8	8.8			percent



VOLUME CORRECTION (V):
4 INCH (4-1): V=15.120 pcf/gm
4 INCH (4-3): V=15.026 pcf/gm
4 INCH (4-4): V=15.090 pcf/gm

6 INCH: V=34.020 pcf/gm)

B	METHOD USED (A,B or C)

4-1	MOLD USED
15.120	MOLD VOLUME CORRECTION (V)
3/8	SIEVE NUMBER
<1	PERCENT RETAINED

129	**MAXIMUM DENSITY [PCF]**
6	**OPTIMUM MOISTURE [%]**

GROUP DELTA CONSULTANTS

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

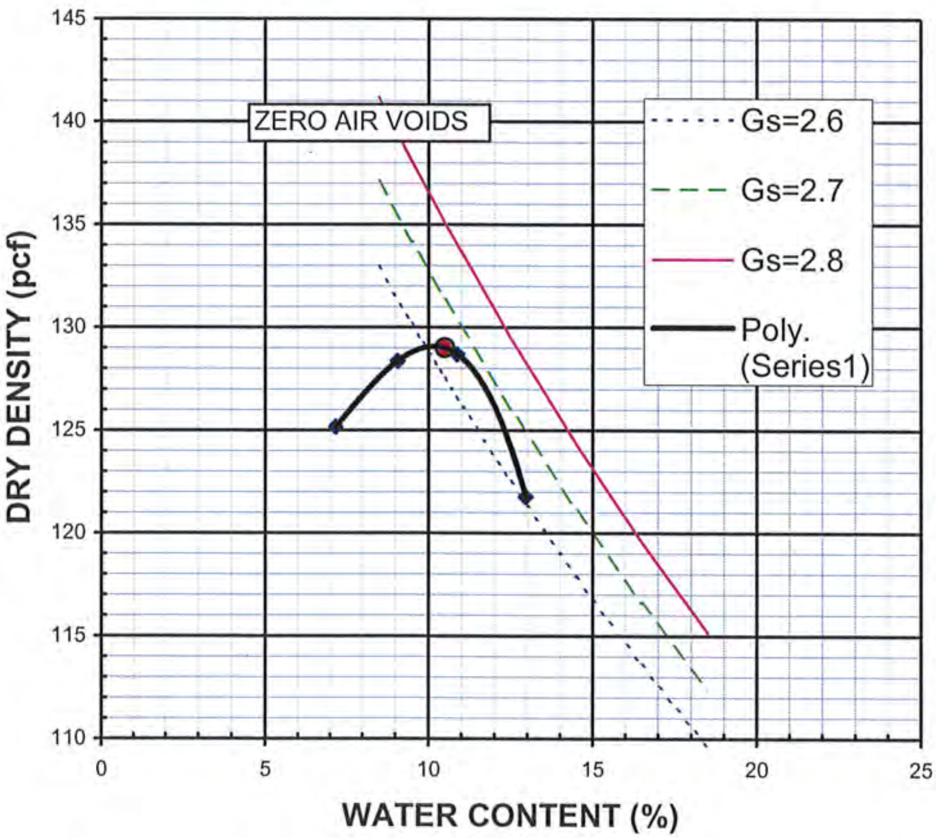
STANDARD TEST METHOD FOR MOISTURE - DENSITY RELATIONSHIP (ASTM D1557)

REV Dec-11

PROJECT: AGS
SAMPLE ID: TP19 @ 2-4 ft
PROJECT NO.: SD-248
DATE: January 10, 2011
TESTED BY: RJP
CHECKED BY: Chad Davis
SAMPLE DESCRIPTION: Dark reddish brown silty clayey sand (SM / SC)

A) WATER ADDED	100	150	200	250			milliliters
B) MOLD TARE WEIGHT	1966.3	1966.3	1966.3	1966.3			grams
C) WEIGHT OF WET SOIL AND MOLD	3994.7	4083.5	4123.7	4045.9			grams
D) WET SOIL WEIGHT (C - B)	2028.4	2117.2	2157.4	2079.6			grams
E) WET DENSITY (D / V)	134.2	140.0	142.7	137.5			pcf
F) DRY DENSITY (E / [(L/100) + 1])	125.2	128.4	128.7	121.8			pcf

G) TARE WEIGHT	154.8	143.8	135.8	132.5			grams
H) WEIGHT OF WET SOIL AND TARE	837.2	847.8	842.4	870.2			grams
I) WEIGHT OF DRY SOIL AND TARE	791.5	789.2	773.1	785.6			grams
J) WEIGHT OF WATER (H - I)	45.7	58.6	69.3	84.6			grams
K) DRY WEIGHT OF SOIL (I - G)	636.7	645.4	637.3	653.1			grams
L) MOISTURE CONTENT (J / K * 100)	7.2	9.1	10.9	13.0			percent



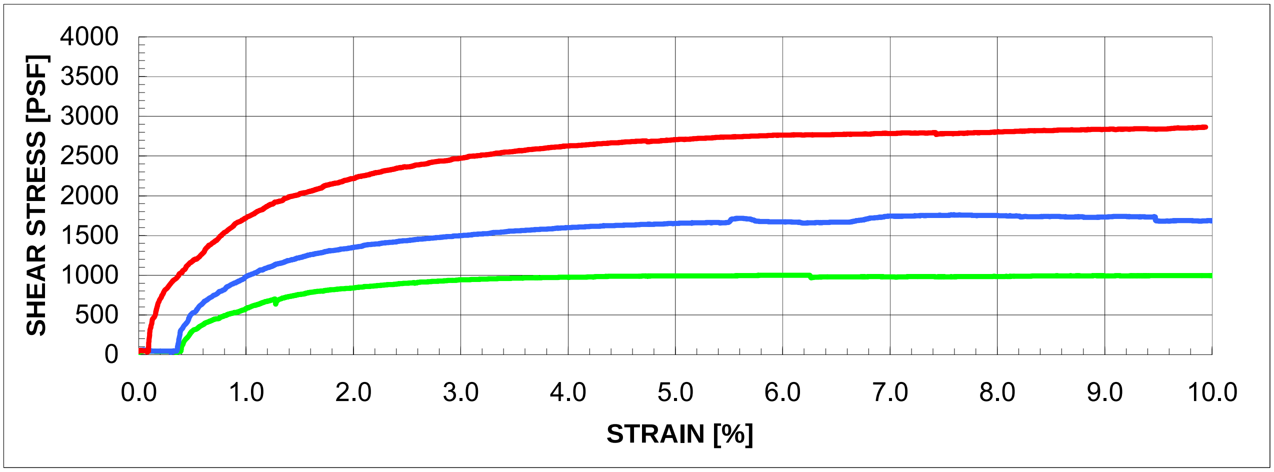
VOLUME CORRECTION (V):
4 INCH (4-1): V=15.120 pcf/gm
4 INCH (4-3): V=15.026 pcf/gm
4 INCH (4-4): V=15.090 pcf/gm

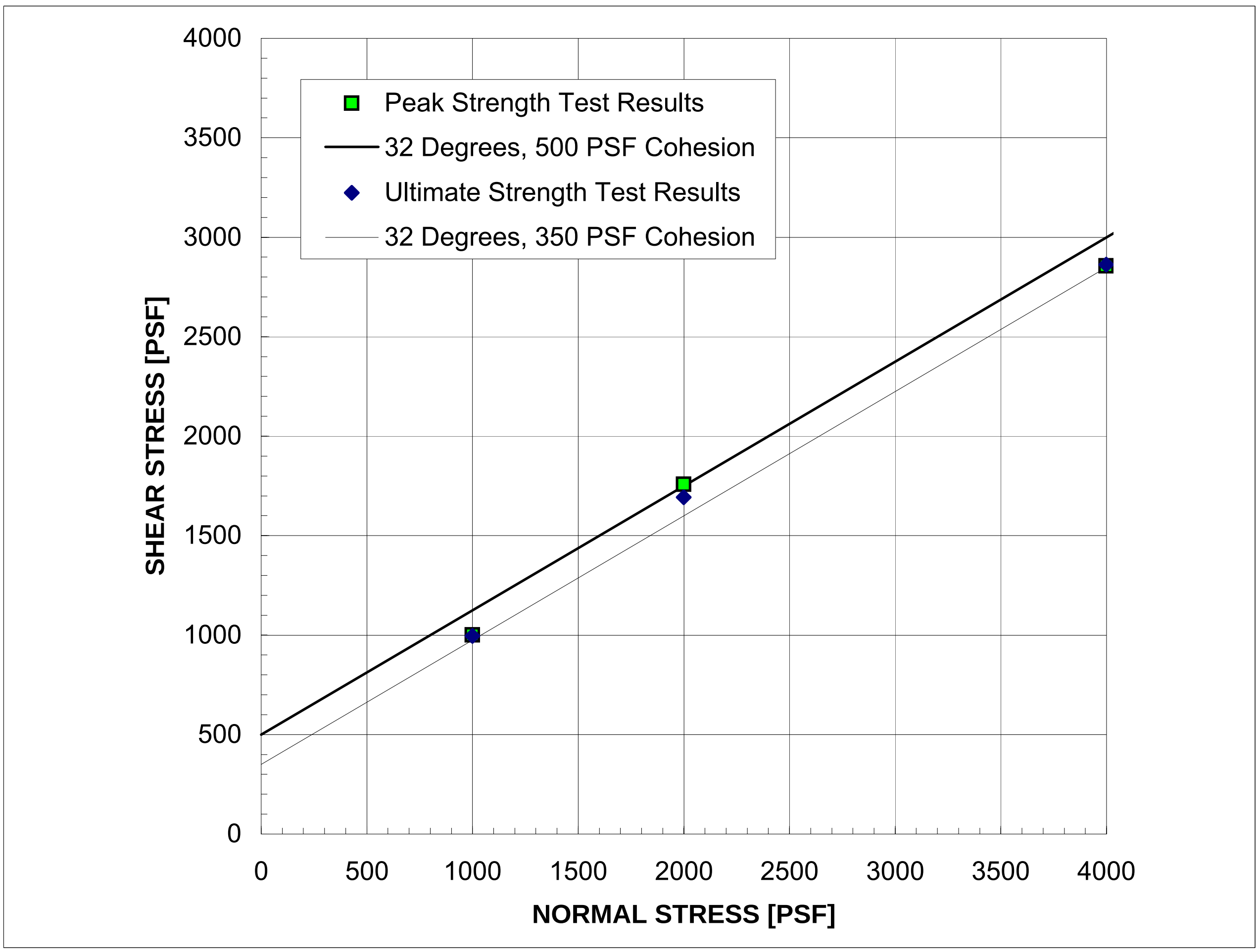
6 INCH: V=34.020 pcf/gm)

A	METHOD USED (A,B or C)

4-1	MOLD USED
15.120	MOLD VOLUME CORRECTION (V)
#4	SIEVE NUMBER
0.0%	PERCENT RETAINED

129	**MAXIMUM DENSITY [PCF]**
10 1/2	**OPTIMUM MOISTURE [%]**





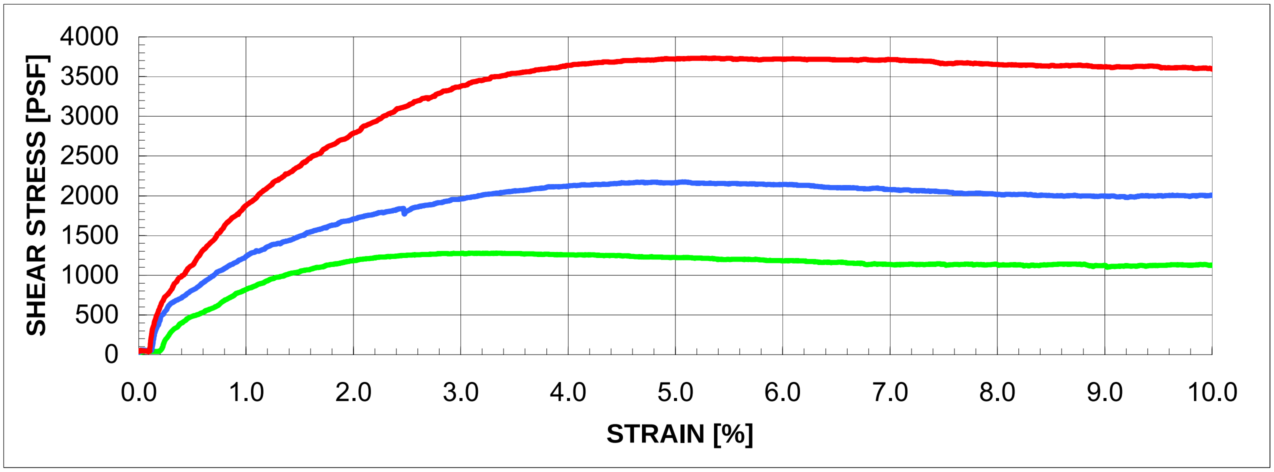
SAMPLE: TP5 @ 6' - 8'

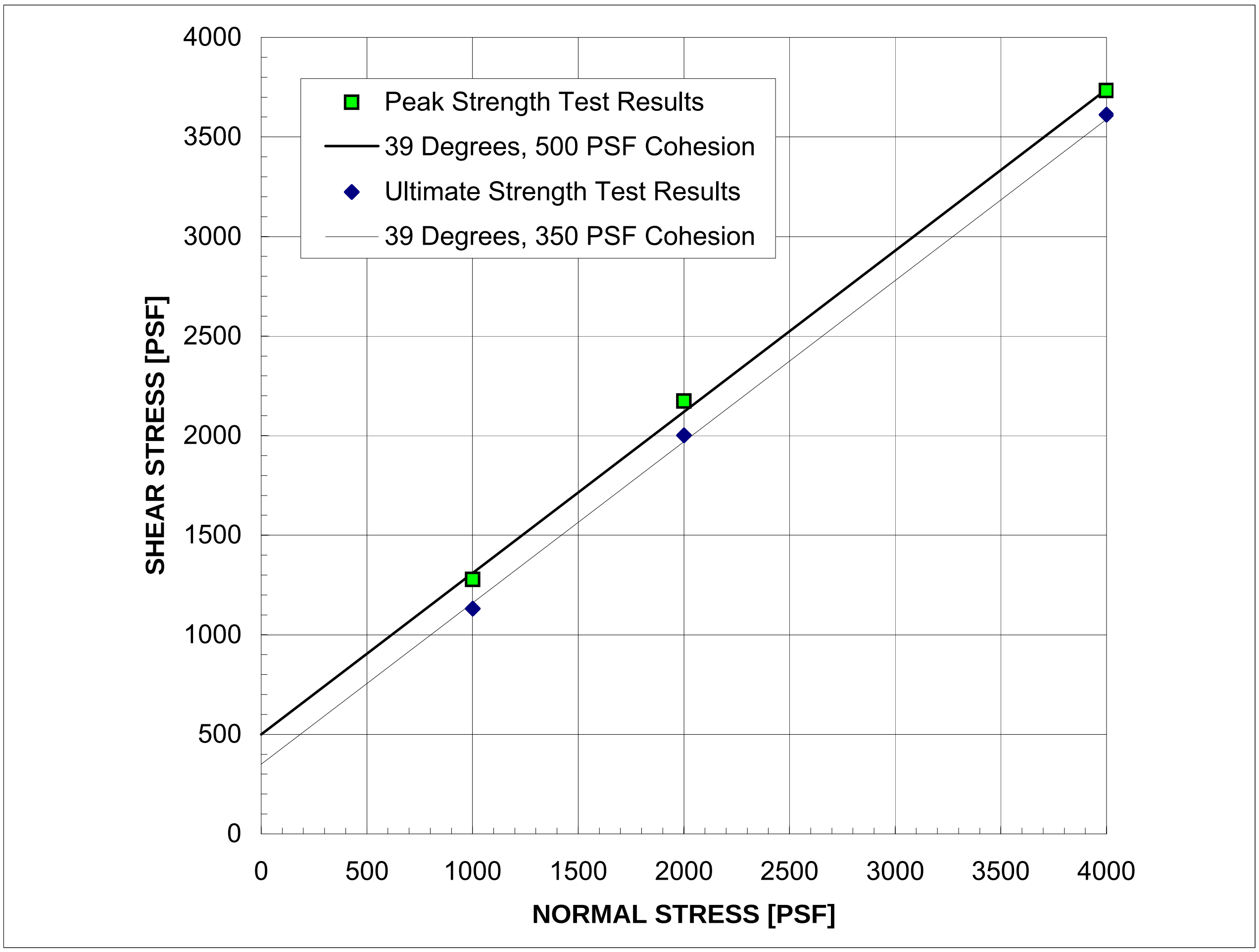
Description: Dark brown clayey sand (SC)
(Remolded to ~90% Maximum @ Optimum)

	PEAK	ULTIMATE
ϕ'	32 °	32 °
C'	500 PSF	350 PSF

STRAIN RATE: 0.0020 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	110.7 PCF	110.7 PCF
w_c	10.8 %	20.0 %





SAMPLE: TP8-2 @ 7' - 9'

Description: Dark brown silty sand (SM).
(Remolded to ~90% Maximum @ Optimum)

	PEAK	ULTIMATE
ϕ'	39 °	39 °
C'	500 PSF	350 PSF

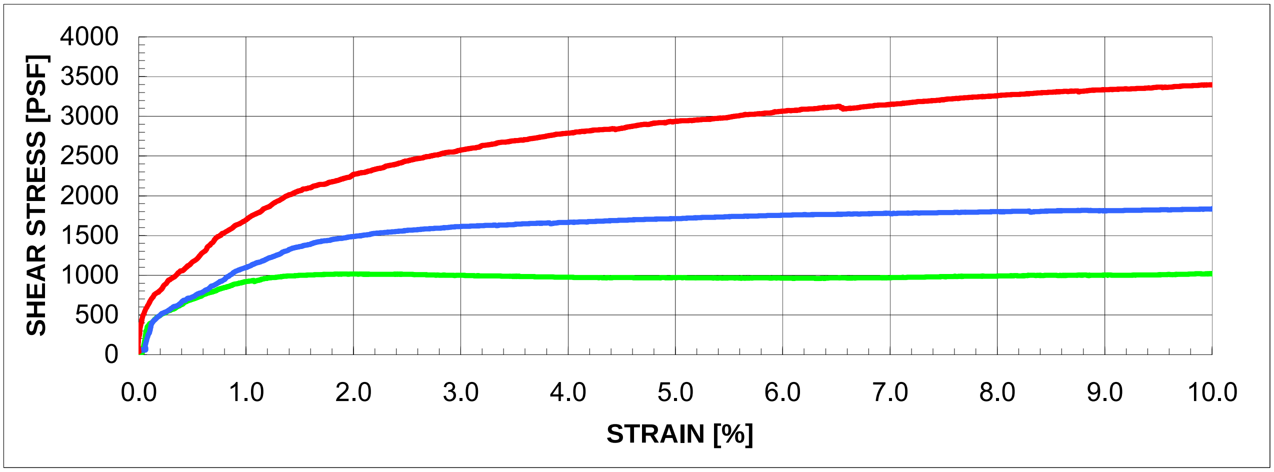
STRAIN RATE: 0.0040 IN/MIN
(Sample was consolidated and drained)

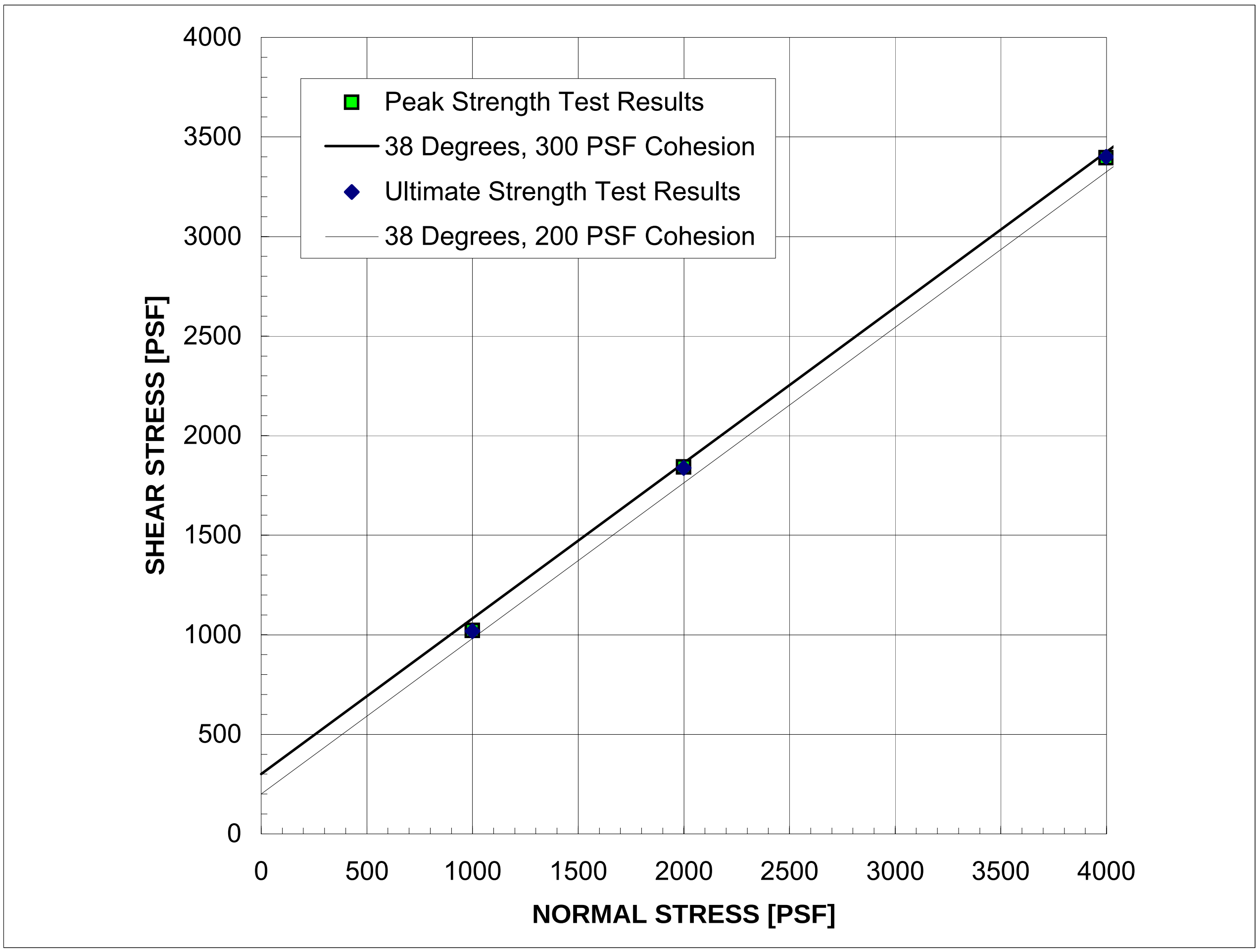
	IN-SITU	AS-TESTED
γ_d	117.1 PCF	117.1 PCF
w_c	5.7 %	16.9 %

GROUP DELTA CONSULTANTS
370 Amapola Ave., Suite 212, Torrance, CA 90501
32 Mauchly, Suite B, Irvine, CA 92618
4201 Santa Ana St., Suite F, Ontario, CA 91761
9245 Activity Road, Suite 103, San Diego, CA 92126

DIRECT SHEAR TEST RESULTS

Document No. 12-0026
Project No. SD-248





SAMPLE: TP19-1 @ 2' - 4'

Description: Drk brn silty/clayey sand (SC/SM).
(Remolded to ~90% Maximum @ Optimum)

	PEAK	ULTIMATE
ϕ'	38 °	38 °
C'	300 PSF	200 PSF

STRAIN RATE: 0.0030 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	116.8 PCF	116.8 PCF
w_c	10.2 %	16.2 %

CORROSIVITY TEST RESULTS
(ASTM D516, CTM 643)

SAMPLE	pH	RESISTIVITY [OHM-CM]	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
TP9-1 @ 3' – 5'	n/t	n/t	<0.01	<0.01

SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	-
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

CHLORIDE (Cl) CONTENT [%]	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive

Sand Equivalent

(ASTM D2419)

G Force Lab No. 8415
Date Sampled: 3/6/2012 By: Client
Date Submitted: 3/6/2012 By: Client
Sample Location: T-28 @ 0' - 2'
Sample Description: Brown Silty Sand (SM)

Clay Reading	Sand Reading	SE
14	2.9	21
14	3.0	22
14	2.9	21
Average Sand Equivalent		22

Reviewed by: NW Clements
Neal W. Clements, P.E., C54902

AGS
Las Lilas

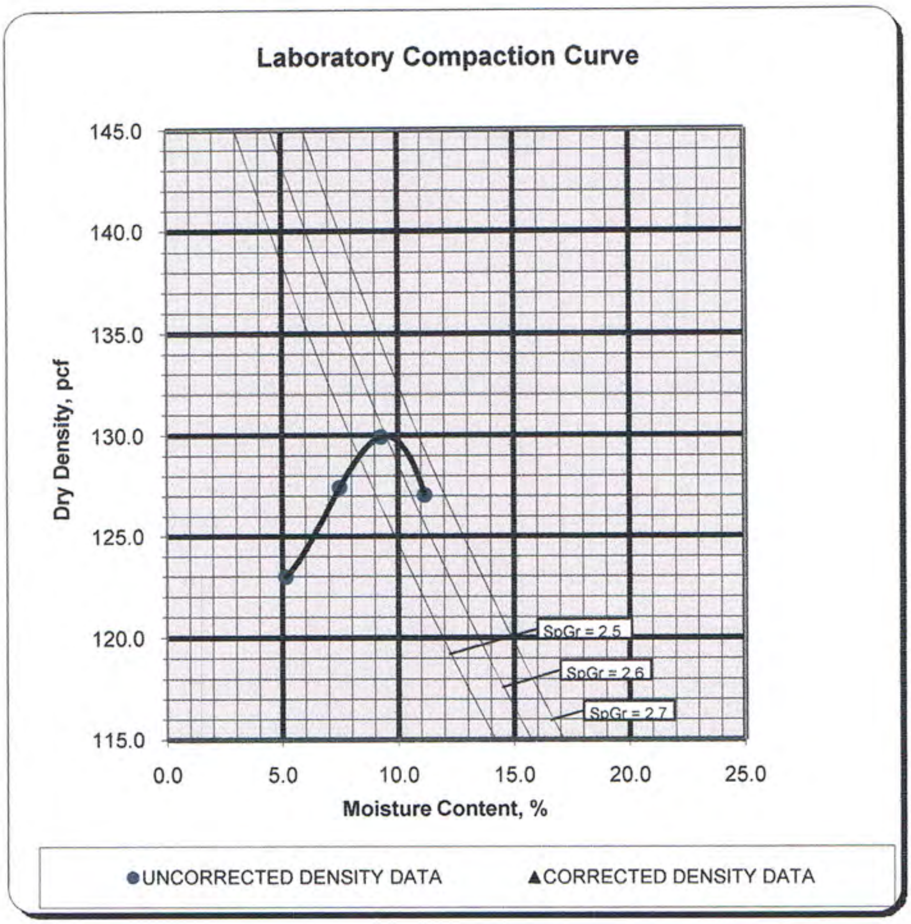
March 12, 2012
G13375L

LABORATORY COMPACTION CURVE

G Force Lab No.:	**8415**		
Sample Location:	**T-28**	Depth, ft.:	**0 - 2**
Soil Description:	**Brown Silty Sand (SM)**		
Source of Soil:	**Native**		

Test Designation: **ASTM D1557** Method **A**
% +3/4" **0** % +3/8" **0** % +#4 **0**
Oversize Correction Applied? **No**
Method of Sample Preparation: **Wet**
Type of Rammer Used: **Manual**

M/D Curve No. 1



Test Results

Maximum Density, pcf	130.0
Optimum Moisture, %	9.5

Oversize Corrected Results

Maximum Density, pcf	
Optimum Moisture, %	

Reviewed by: *NW Clements*
Neal W. Clements, P.E., C54902

G FORCE DVBE · SDVOSB · SBE
♦ *8788 Balboa Avenue, San Diego, CA 92123* ♦ *Tel: 619-583-6633* ♦
www.gforceca.com

Expansion Index

(ASTM D4829)

G Force Lab No.	8413		
Date Sampled:	03/06/12	By:	Client
Date Submitted:	03/06/12	By:	Client
Sample Location:	T-25 @ 1' - 4'		
Sample Description:	Brown Silty Sand (SM)		

Initial Water Content, %	9.8
Dry Density, pcf	120.8
Saturation, %	49.8
Final Water Content, %	18.3
Expansion Index	**42.2**
Potential Expansion	**Low**

Checked by: Ricardo Hernandez
Ricardo Hernandez, Lab Manger

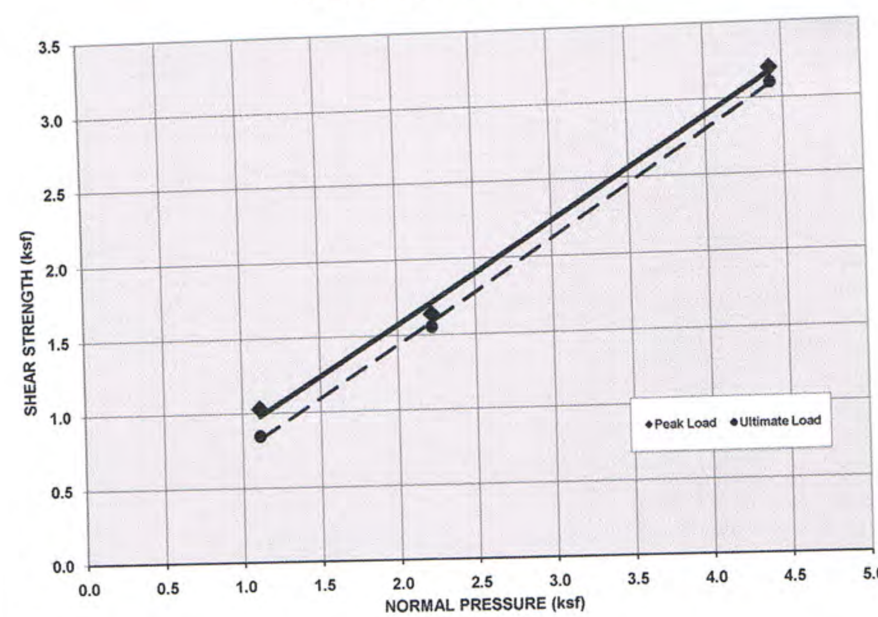
Reviewed by: NW Clements
Neal W. Clements, P.E., C54902

March 12, 2012
Project No. G13375L

AGS
Las Lilas

DIRECT SHEAR TEST REPORT

G-FORCE LAB NO.:	8415
SAMPLE LOCATION:	T-28 @ 0' - 2'
SOIL TYPE:	Brown Silty Sand (Remolded to 90%)



CALCULATED DATA

INITIAL				
WET DENSITY	pcf	126.6	126.1	126.7
DRY DENSITY	pcf	115.6	115.2	115.7
MOISTURE	%	9.5	9.5	9.5
AFTER SATURATION				
COMPRESSION(-) or EXPANSION (+)	%	0.7%	1.0%	2.4%
FINAL, at failure				
WET DENSITY	pcf	135.8	132.2	130.2
DRY DENSITY	pcf	116.7	114.0	113.1
MOISTURE	%	16.5	16.0	15.1
COMPRESSION(-) or EXPANSION (+)	%	-0.9%	1.0%	1.7%

	Peak			Ultimate		
NORMAL PRESSURE, ksf	1.11	2.22	4.42	1.11	2.22	4.42
SHEAR STRENGTH, ksf	1.03	1.64	3.20	0.85	1.55	3.10
FRICTION ANGLE, degrees	33.5			34.4		
COHESION, ksf	0.25			0.07		

Checked by: Ricardo Hernandez
Ricardo Hernandez, Lab Manager

Reviewed by: NW Clements
Neal W. Clements, P.E., C54902

LABORATORY DATA

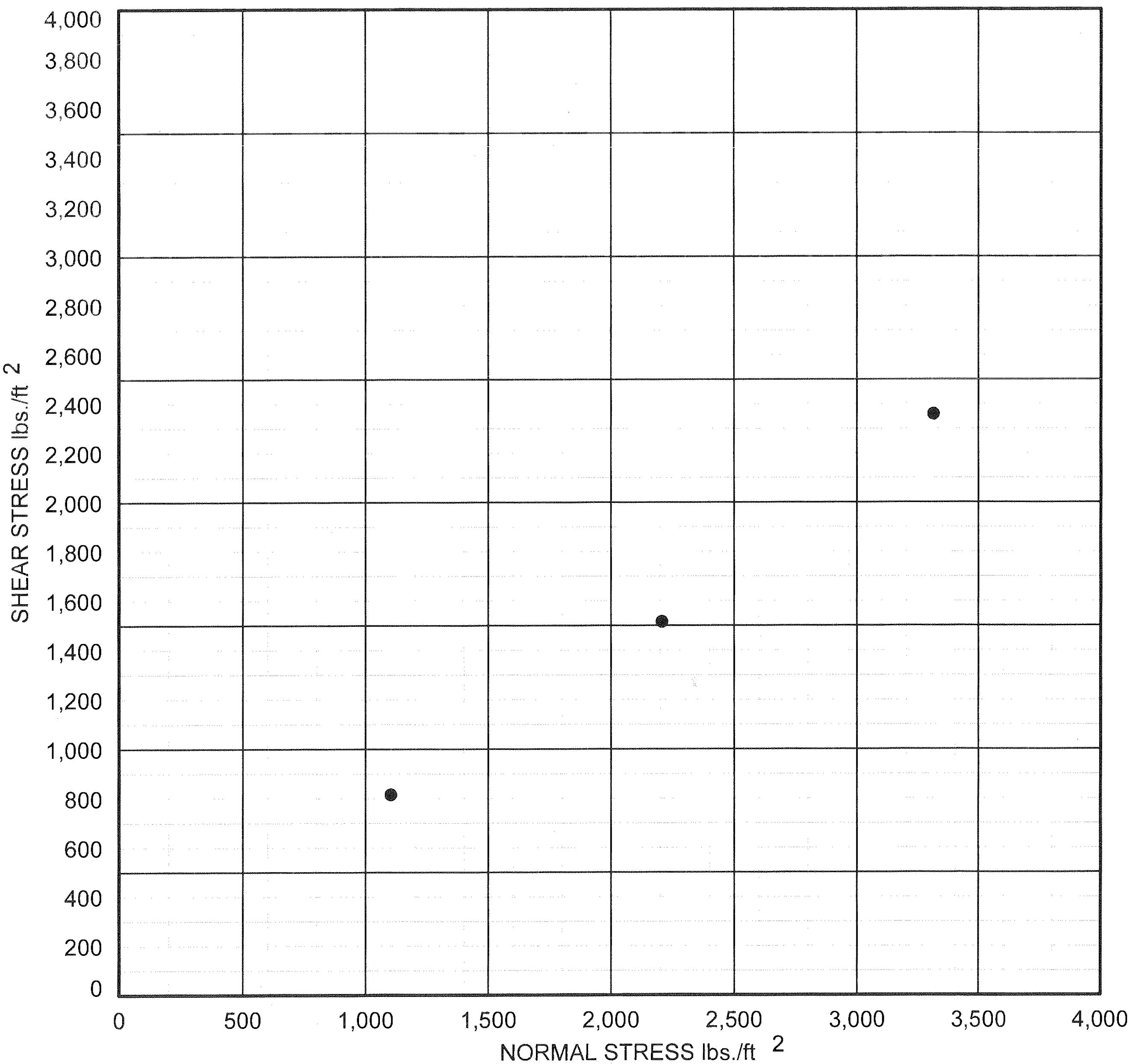
PACIFIC SOILS ENGINEERING, INC.
MAY 2007

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

TABLE II
SUMMARY OF LABORATORY TEST DATA
W.O. 401120

SAMPLE	DEPTH (FEET)	SOIL DESCRIPTION	GROUP SYMBOL	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)	IN-SITU DRY DENSITY (PCF)	IN-SITU MOISTURE CONTENT (%)	DEGREE OF SATURATION (%)	PLUS NO.4 SIEVE (%)	COARSE SAND (%)	MEDIUM TO FINE SAND (%)	SILT (0.075mm - 0.005mm) (%)	CLAY (minus 0.005mm) (%)	EXPANSION INDEX UBC 18-2	COHESION (PSF)	FRICTION ANGLE (DEGREES)	OTHER TESTS REMARKS
TP-18	2	Granitic Rock (Kgr)		124.1	10.3				0	20	73	4	3		0	35	
TP-21	2	Artificial Fill (Af)	SM	131.7	9.3				0	12	58	17	13				
TP-34	3	Alluvium (Qal)	SM	128.9	9.6				0	3	52	28	17	6	0	35	

DIRECT SHEAR TEST
Remolded at 90% Relative Compaction



boring	depth (ft.)	dry density (pcf)	in situ moist. (%)	-200 sieve (%)	group symbol	typical names
TP-34	3.0			45	SM	Alluvium (Qal)

COHESION	0	psf.
FRICTION ANGLE	35	degrees

DIRECT SHEAR TEST

PACIFIC SOILS ENGINEERING, INC.

W.O. 401120 PLATE D-2

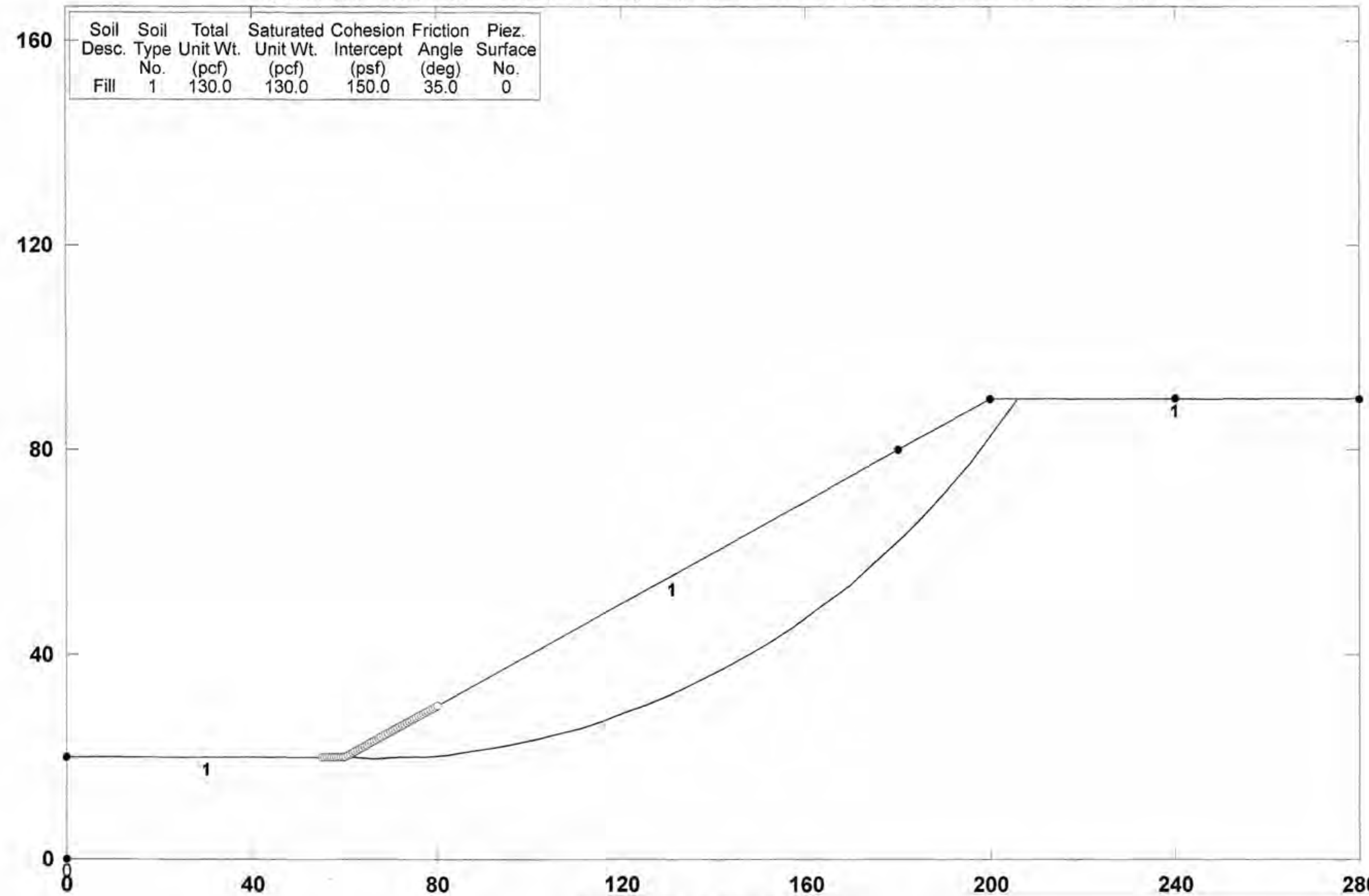
APPENDIX D

SLOPE STABILITY ANALYSES

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

P/W 1102-01 2:1 70 foot Fill Slope LHR

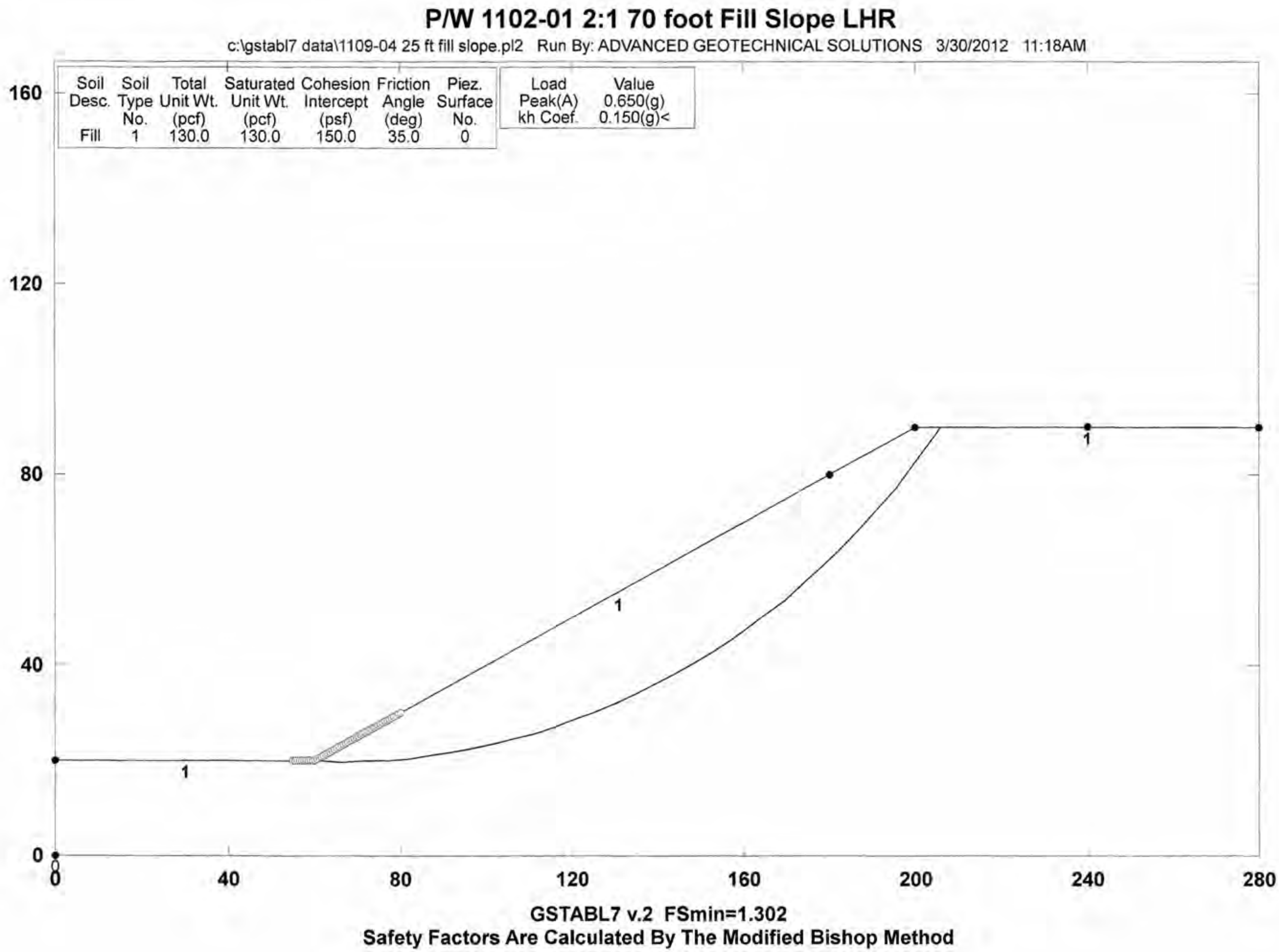
c:\gstabl7 data\1109-04 25 ft fill slope.pl2 Run By: ADVANCED GEOTECHNICAL SOLUTIONS 3/30/2012 11:18AM



GSTABL7 v.2 FSmin=1.793

Safety Factors Are Calculated By The Modified Bishop Method

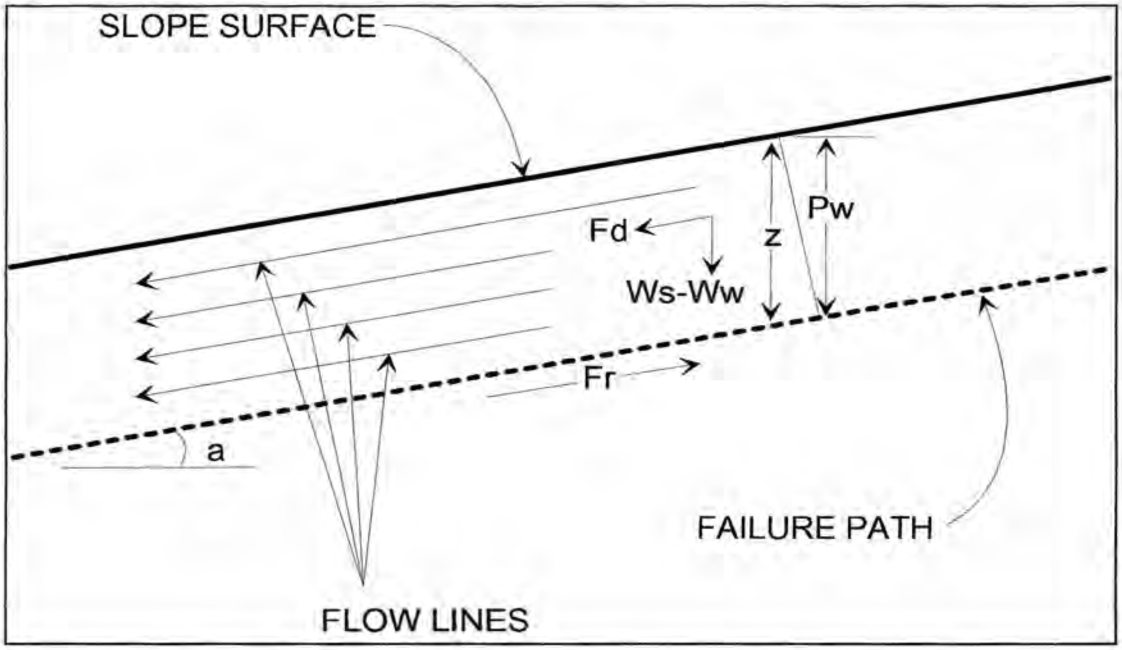
Plate D-1



P/W 1102-01 2:1 70 foot Fill Slope LHR
c:\gstabl7 data\1109-04 25 ft fill slope.pl2 Run By: ADVANCED GEOTECHNICAL SOLUTIONS 3/30/2012 11:18AM
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Fill 1 130.0 130.0 150.0 35.0 0
Load Value
Peak(A) 0.650(g)
kh Coef. 0.150(g)<
160
120
80
40
0
0
40
80
120
160
200
240
280
1
1
1
GSTABL7 v.2 FSmin=1.302
Safety Factors Are Calculated By The Modified Bishop Method

Plate D-2

SURFICIAL SLOPE STABILITY



(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)	(radians)	phi (degrees)	(radians)	c (psf)
130	3	26.56505	0.4636476	35	0.6108652	150

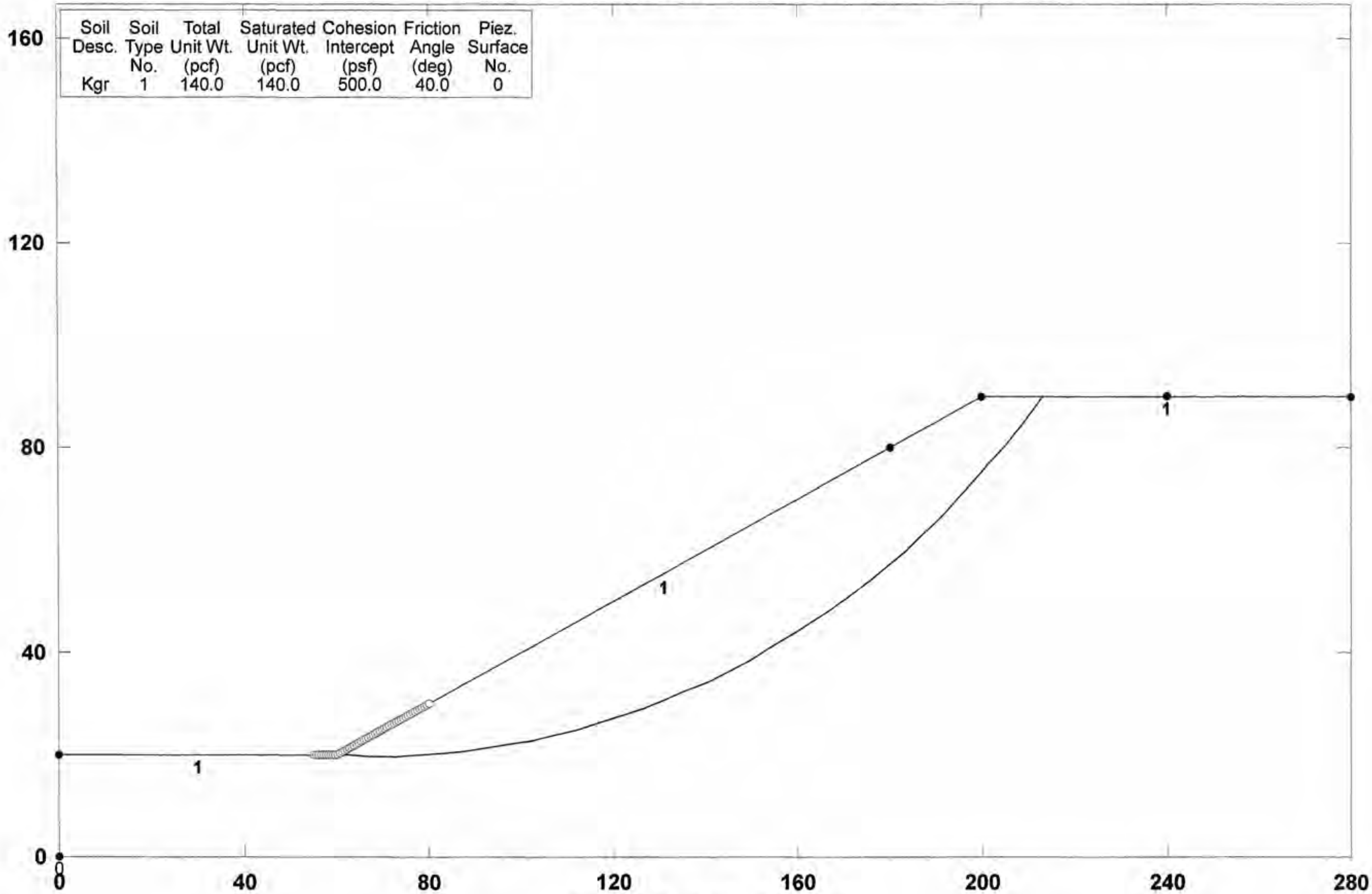
Calculations:

Pw	u	Fd	Fr	FS
2.40	149.76	156.00	263.60	**1.69**

P/W 1102-01 2:1 70 foot Cut Slope LHR

c:\gstabl7 data\1109-04 25 ft fill slope.pl2 Run By: ADVANCED GEOTECHNICAL SOLUTIONS 3/30/2012 11:12AM

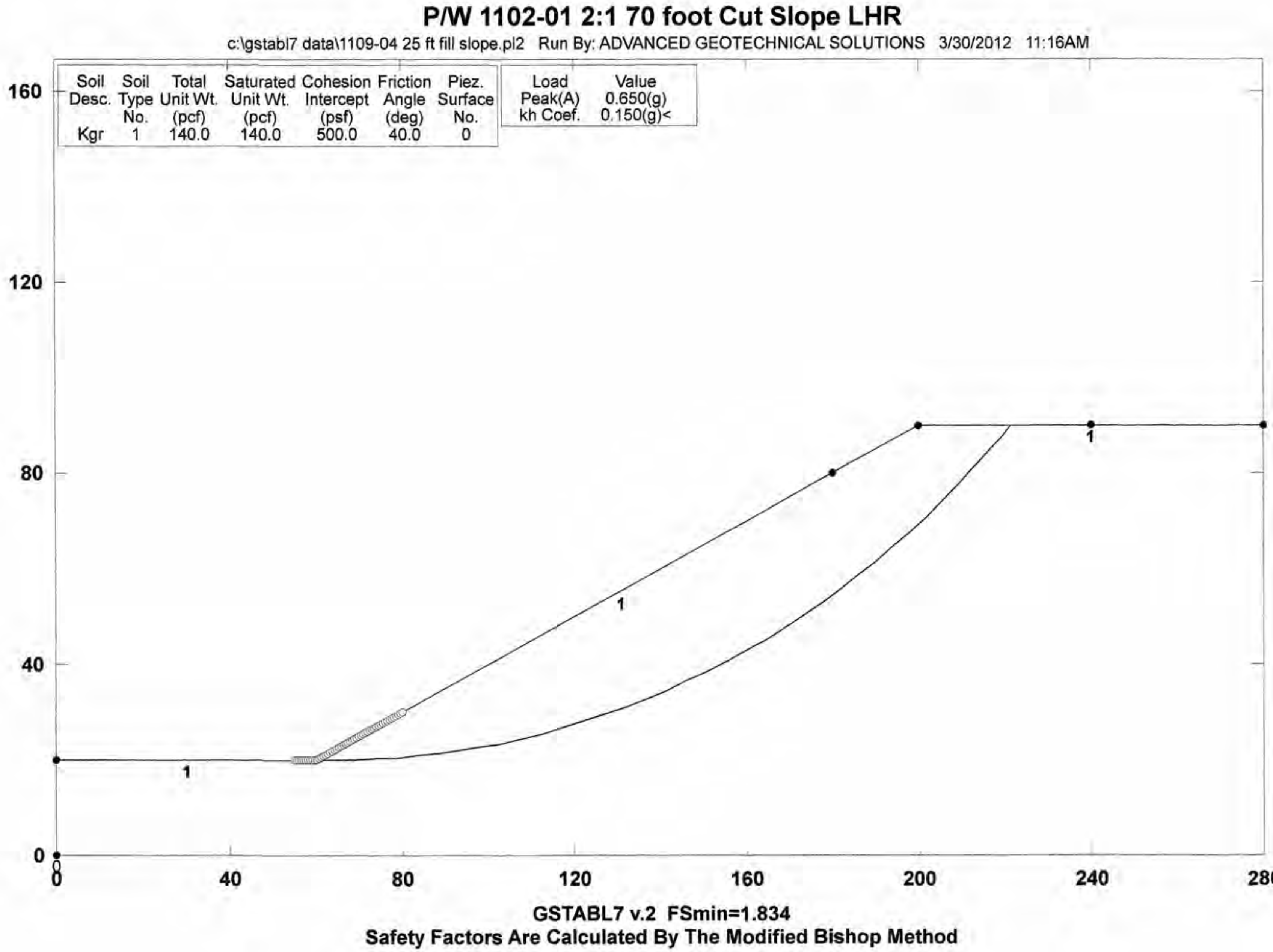
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Kgr	1	140.0	140.0	500.0	40.0	0



GSTABL7 v.2 FSmin=2.517

Safety Factors Are Calculated By The Modified Bishop Method

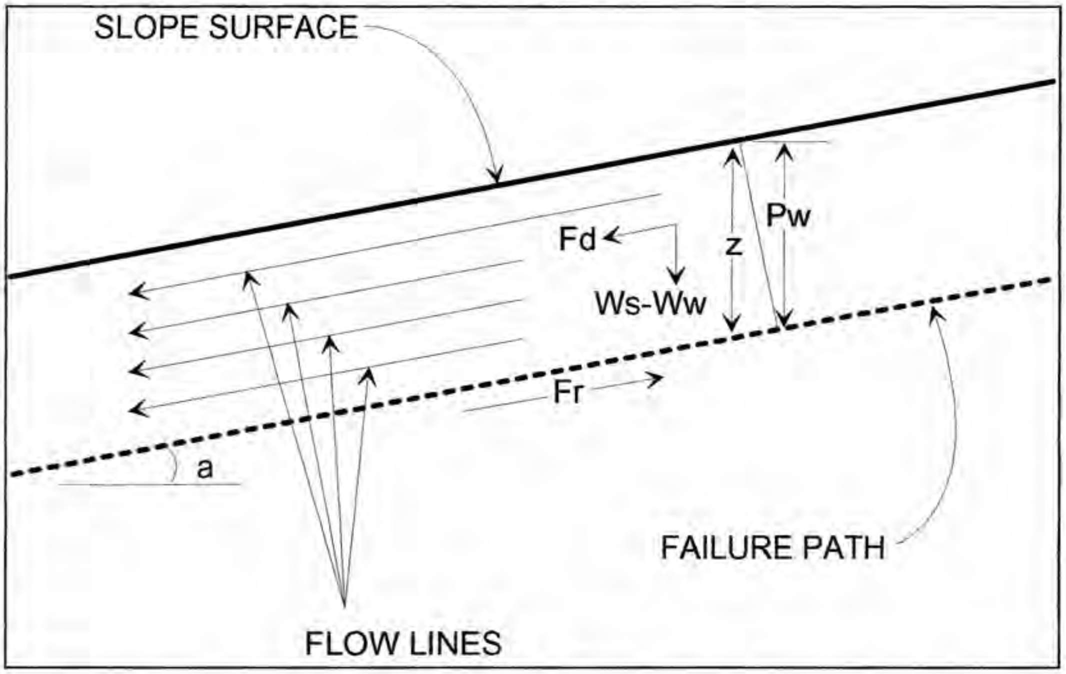
Plate D-4



P/W 1102-01 2:1 70 foot Cut Slope LHR
c:\gstabl7 data\1109-04 25 ft fill slope.pl2 Run By: ADVANCED GEOTECHNICAL SOLUTIONS 3/30/2012 11:16AM
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Kgr | 1 | 140.0 | 140.0 | 500.0 | 40.0 | 0
Load | Value
Peak(A) | 0.650(g)
kh Coef. | 0.150(g)<
160
120
80
40
0
0
40
80
120
160
200
240
280
1
1
1
GSTABL7 v.2 FSmin=1.834
Safety Factors Are Calculated By The Modified Bishop Method

Plate D-5

SURFICIAL SLOPE STABILITY



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)	(radians)	phi (degrees)	(radians)	c (psf)
140	3	33.690068	0.5880026	35	0.6108652	500

Calculations:

Pw	u	Fd	Fr	FS
2.08	129.60	193.85	612.85	**3.16**

APPENDIX E

EARTHWORK SPECIFICATIONS AND GRADING DETAILS

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 depict 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 appraise 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 cesspoles, 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 throughly 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 throughly mixed to obtain a 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 a 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 backrolling 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. Backdrains and Subdrains: Backdrains 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 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.

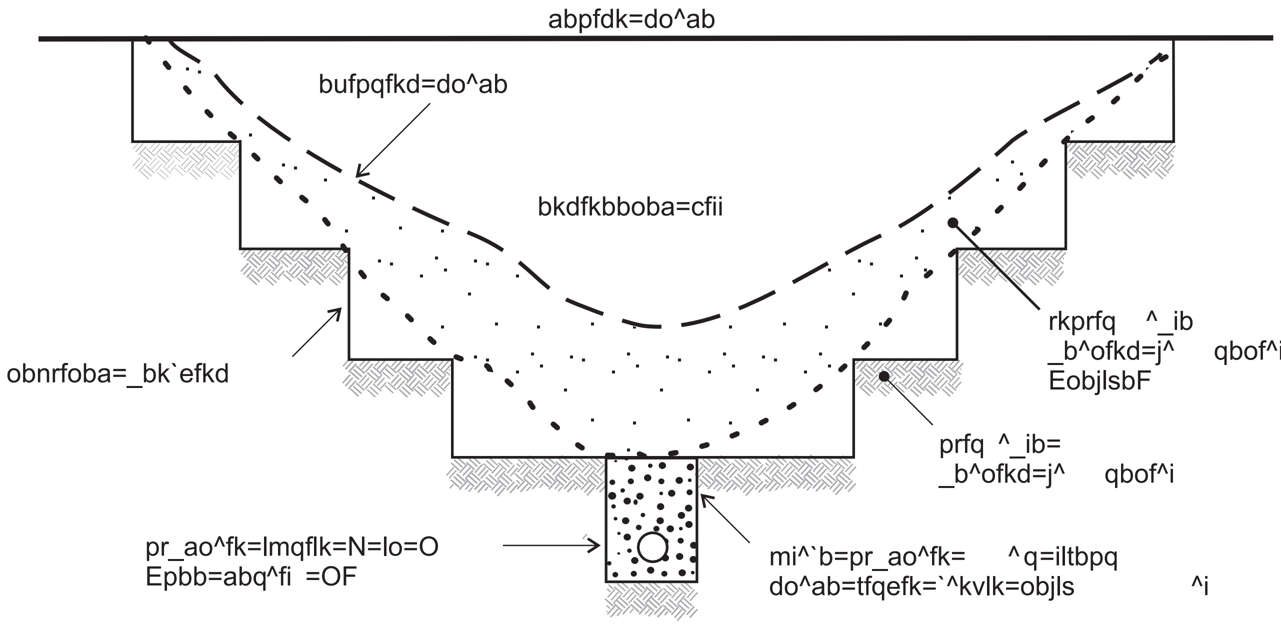
D. Frequency of Compaction Testing: The location and frequency of tests shall be at the Geotechnical Consultant's discretion. Generally, compaction tests shall be taken at intervals not exceeding two feet in fill height and 1,000 cubic yards of fill materials placed.

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.

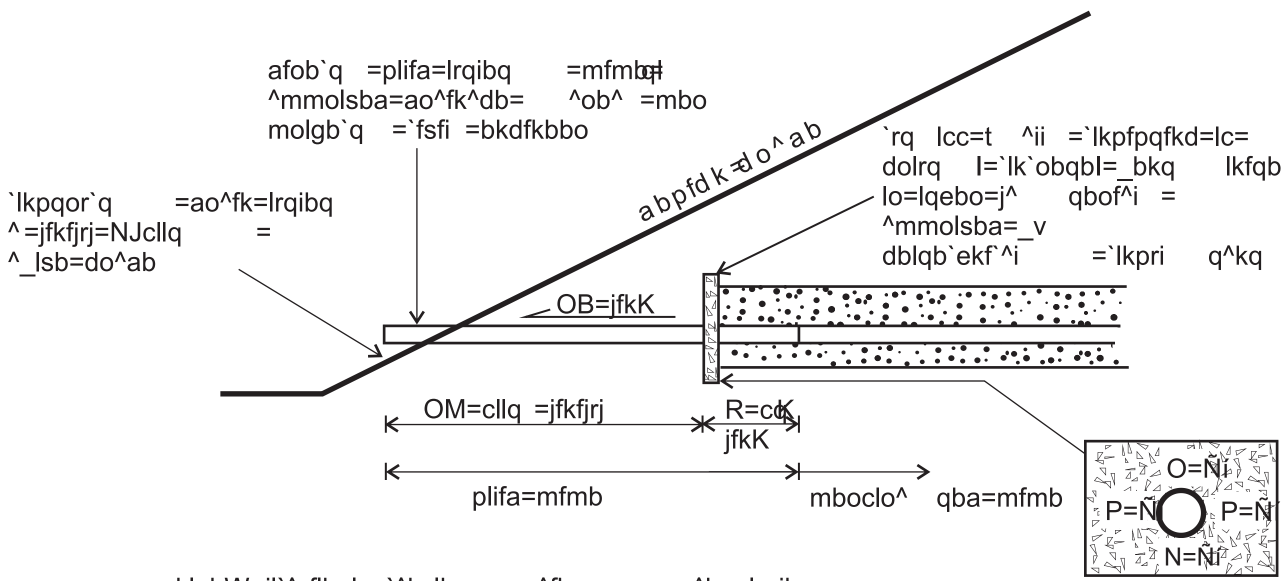
F. Areas of fill that have not been observed or tested by the Geotechnical Consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removals will be determined by the Geotechnical Consultant.

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



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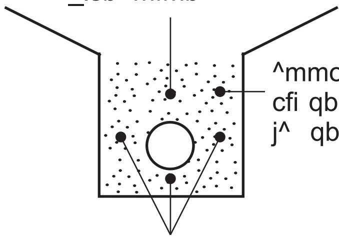

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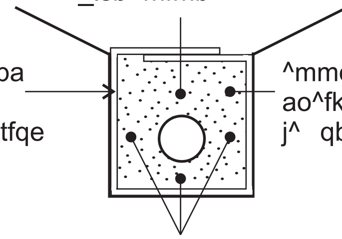

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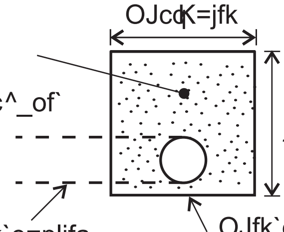

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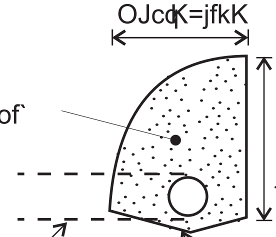

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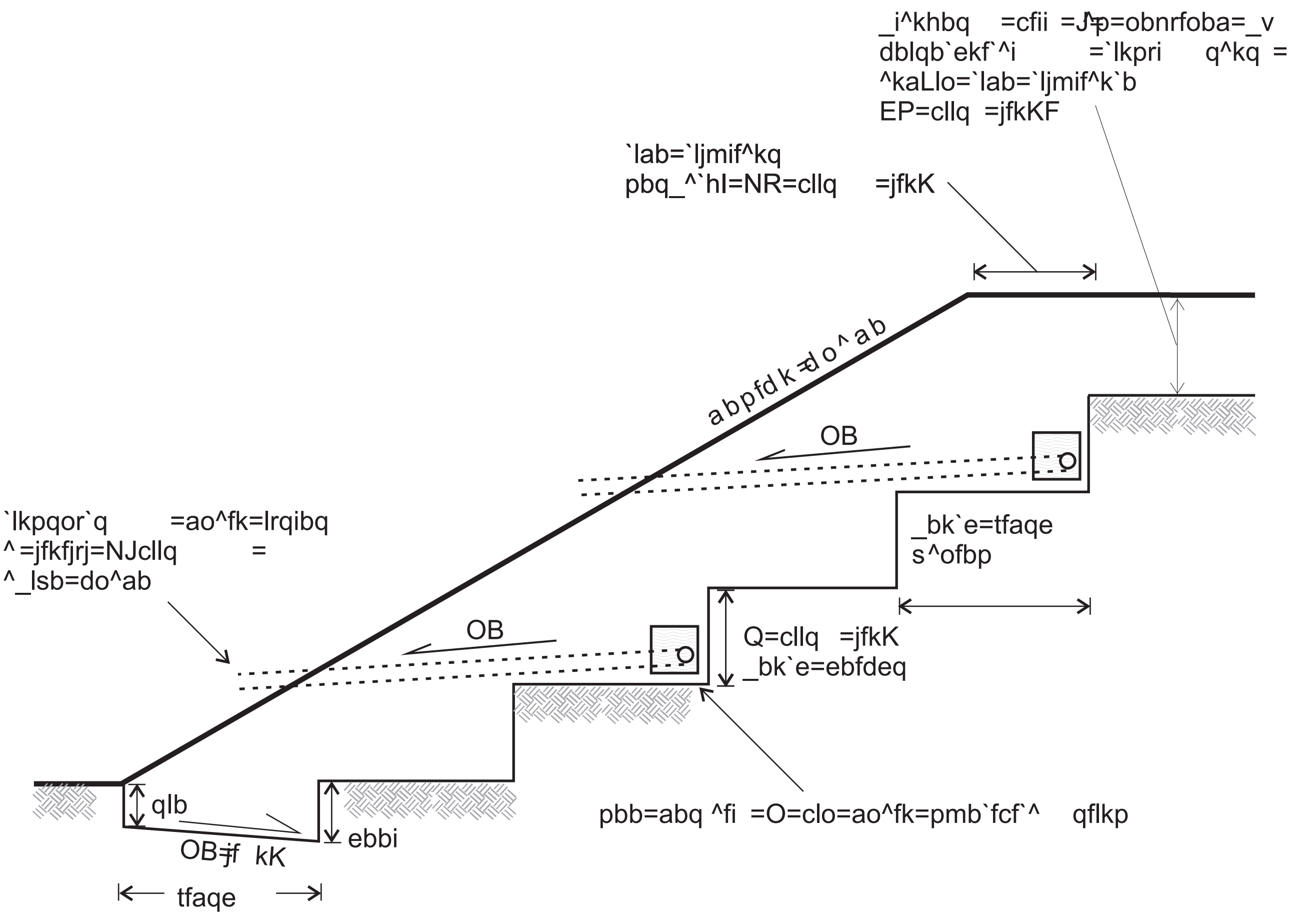

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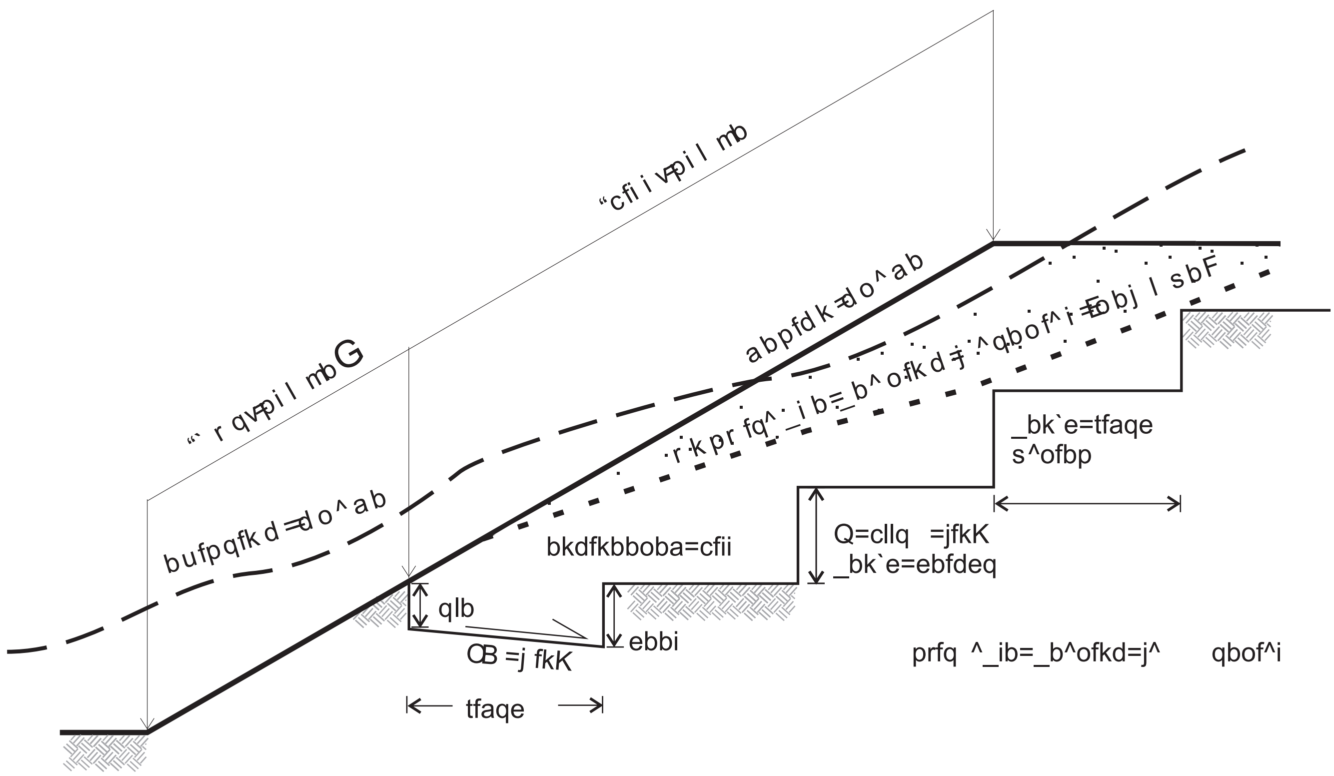

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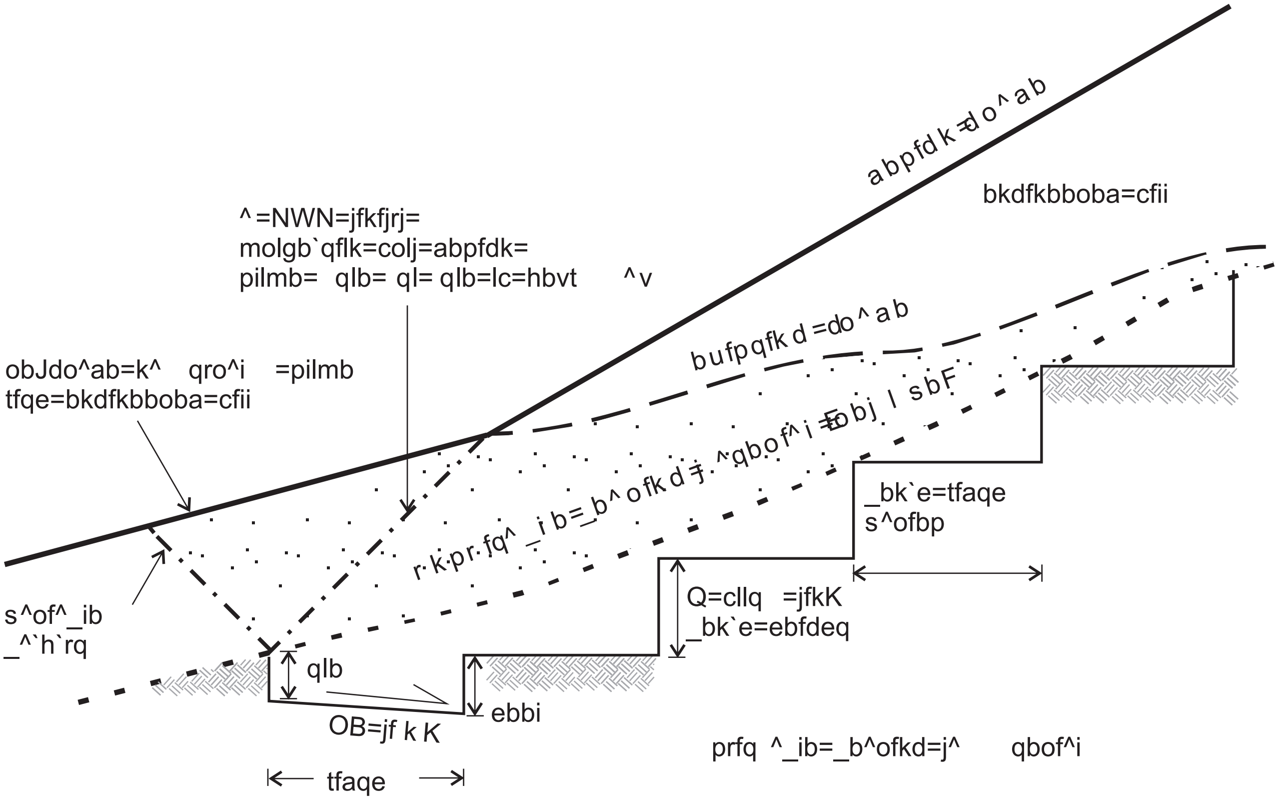

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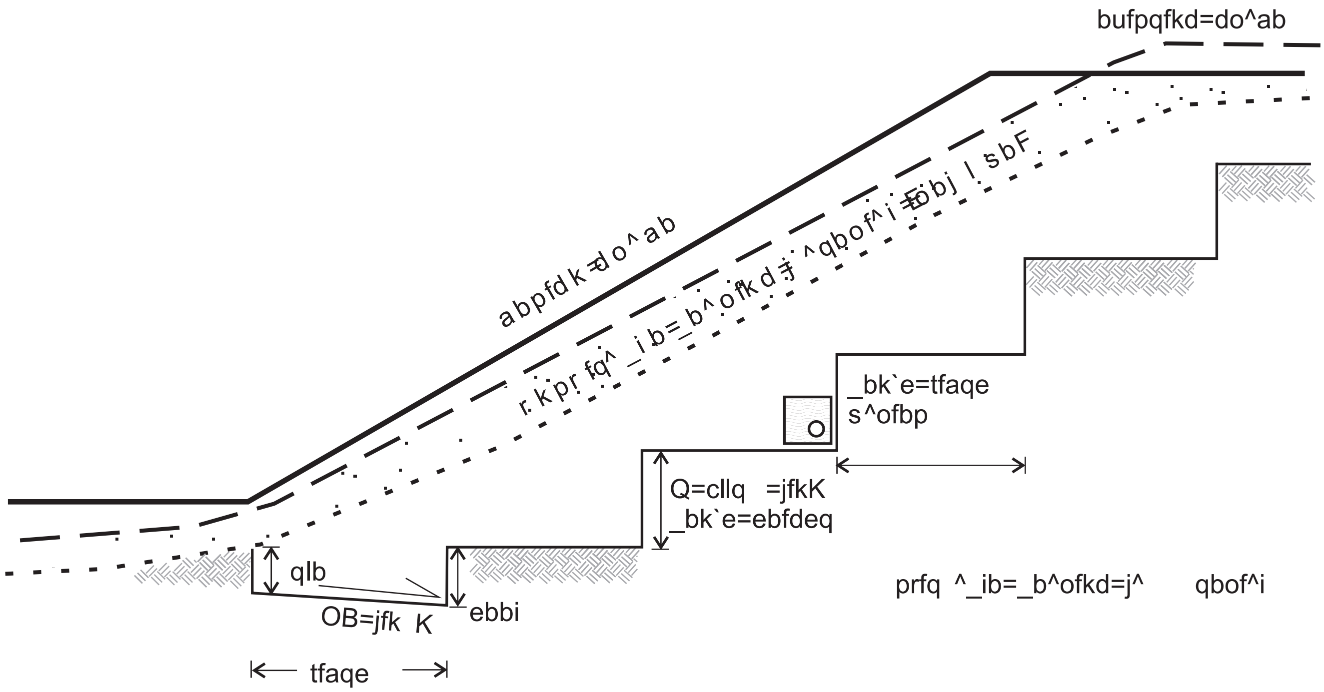

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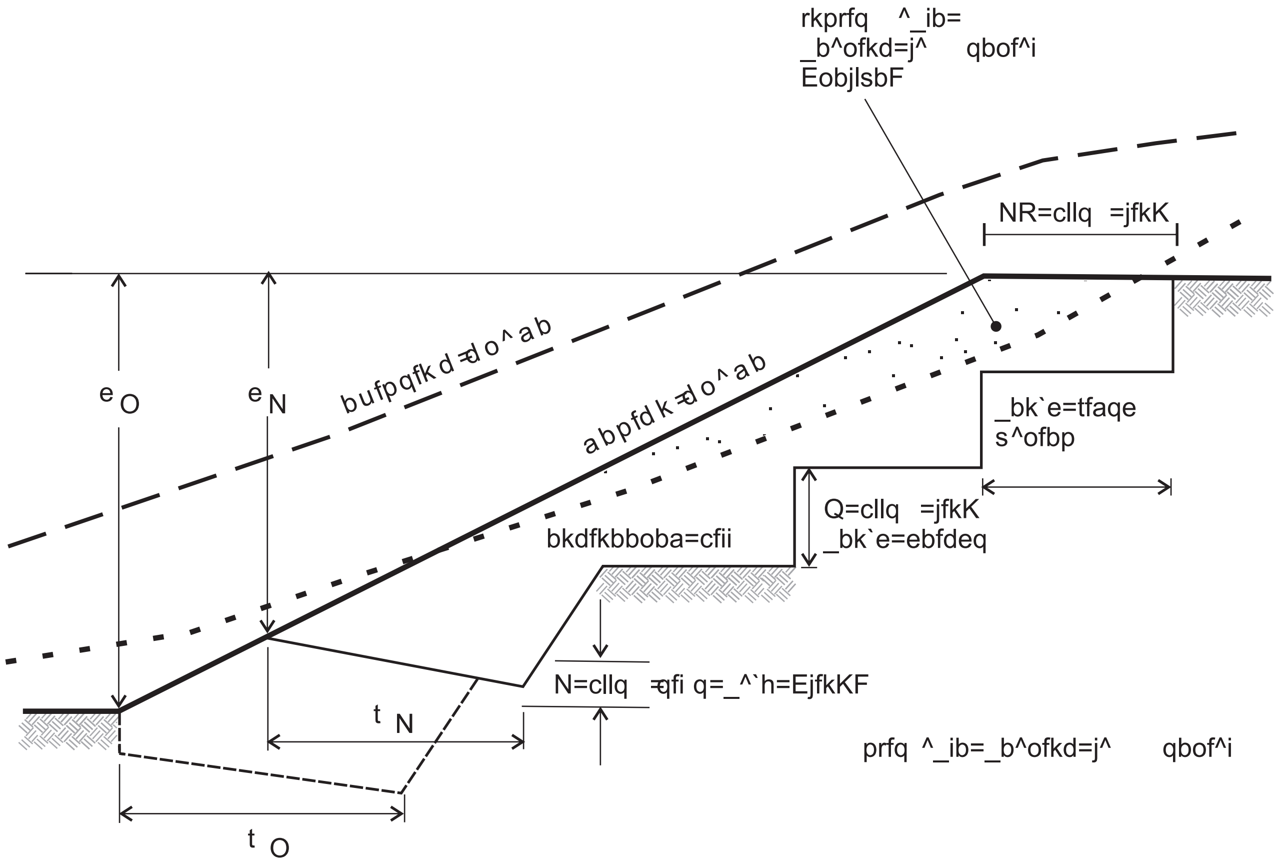

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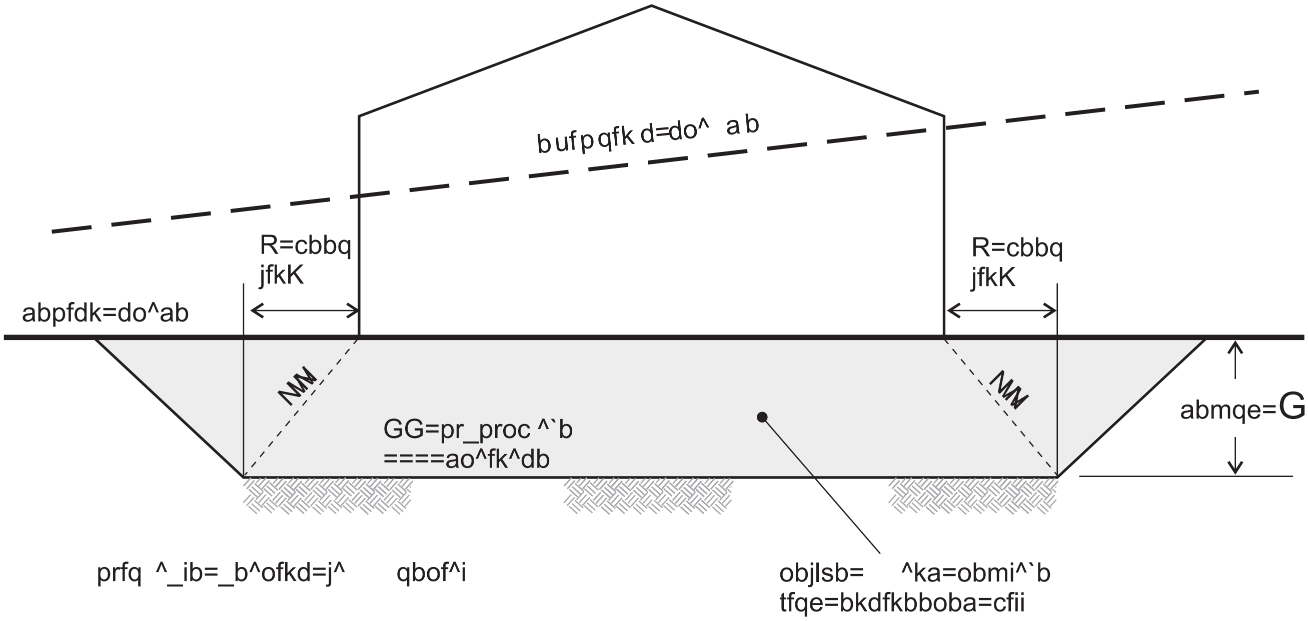

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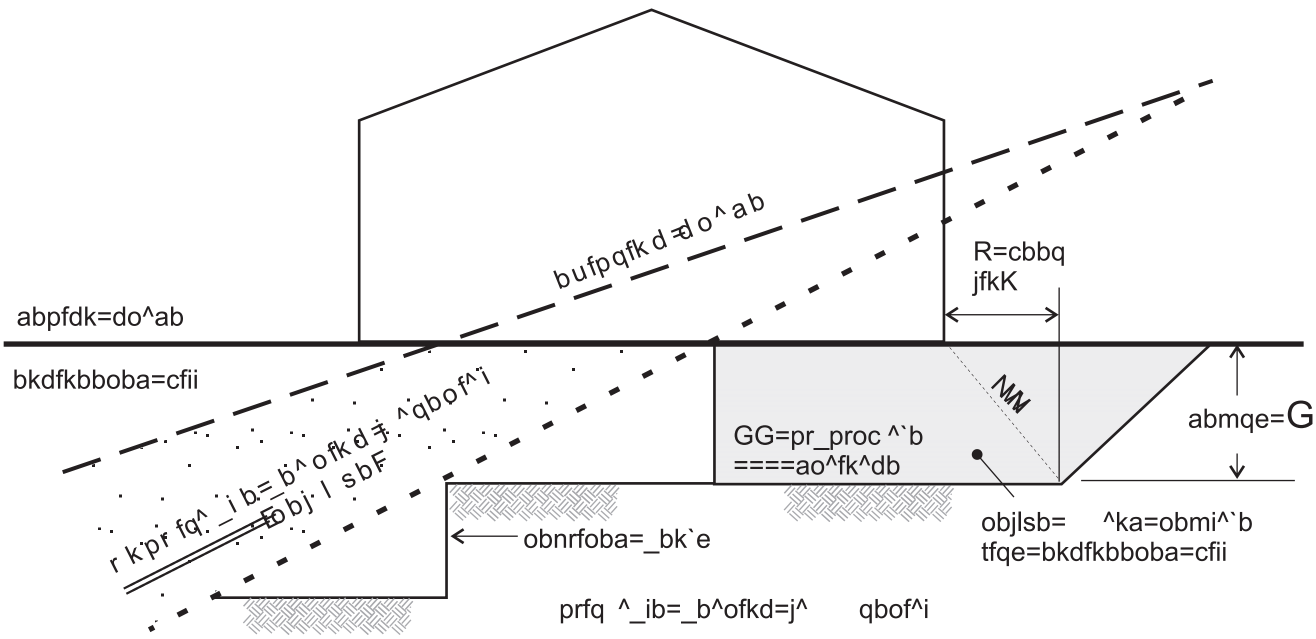

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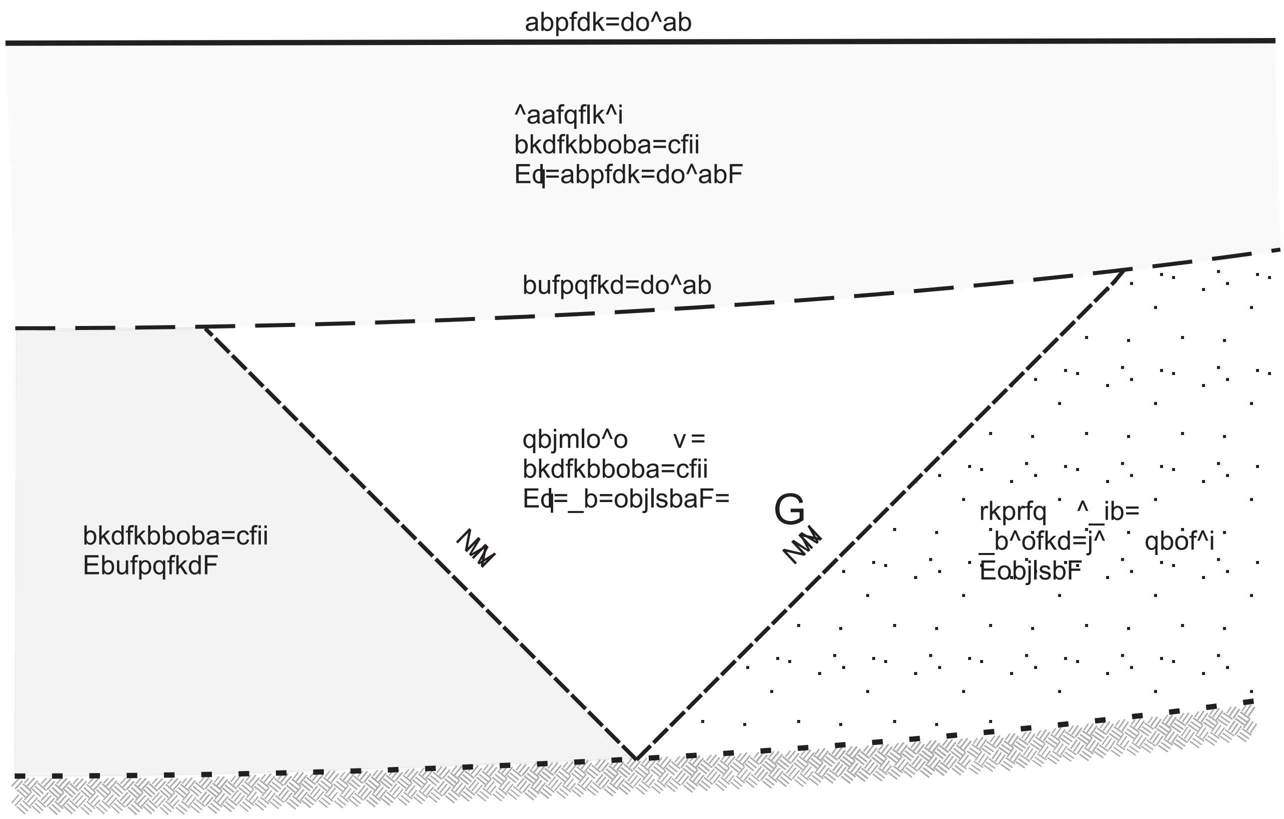

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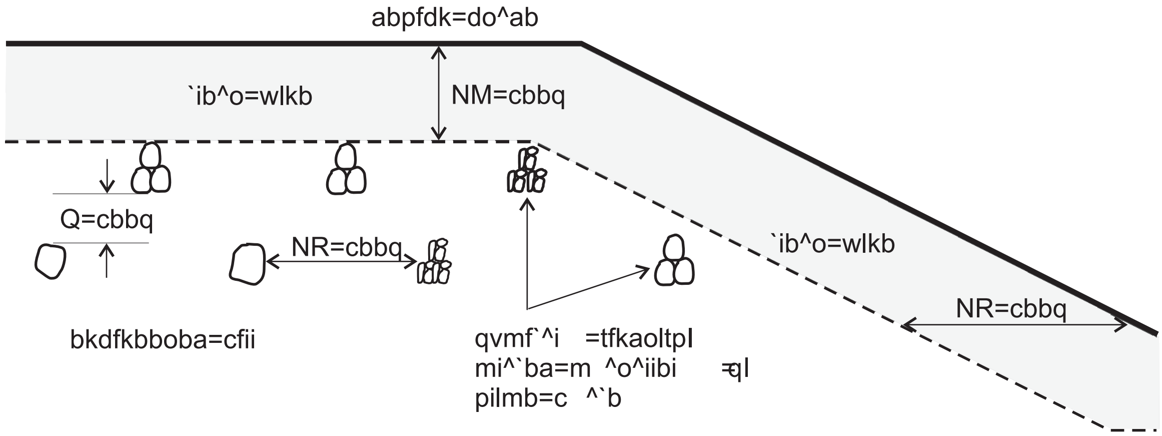

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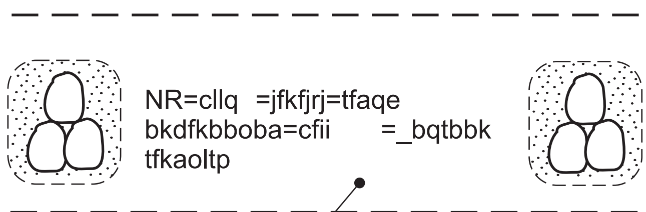

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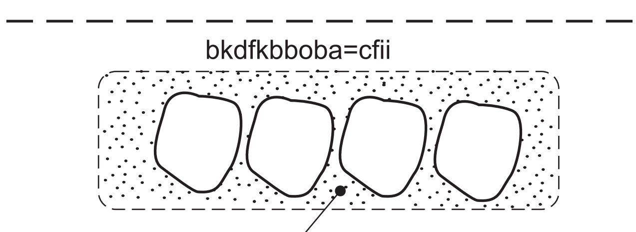

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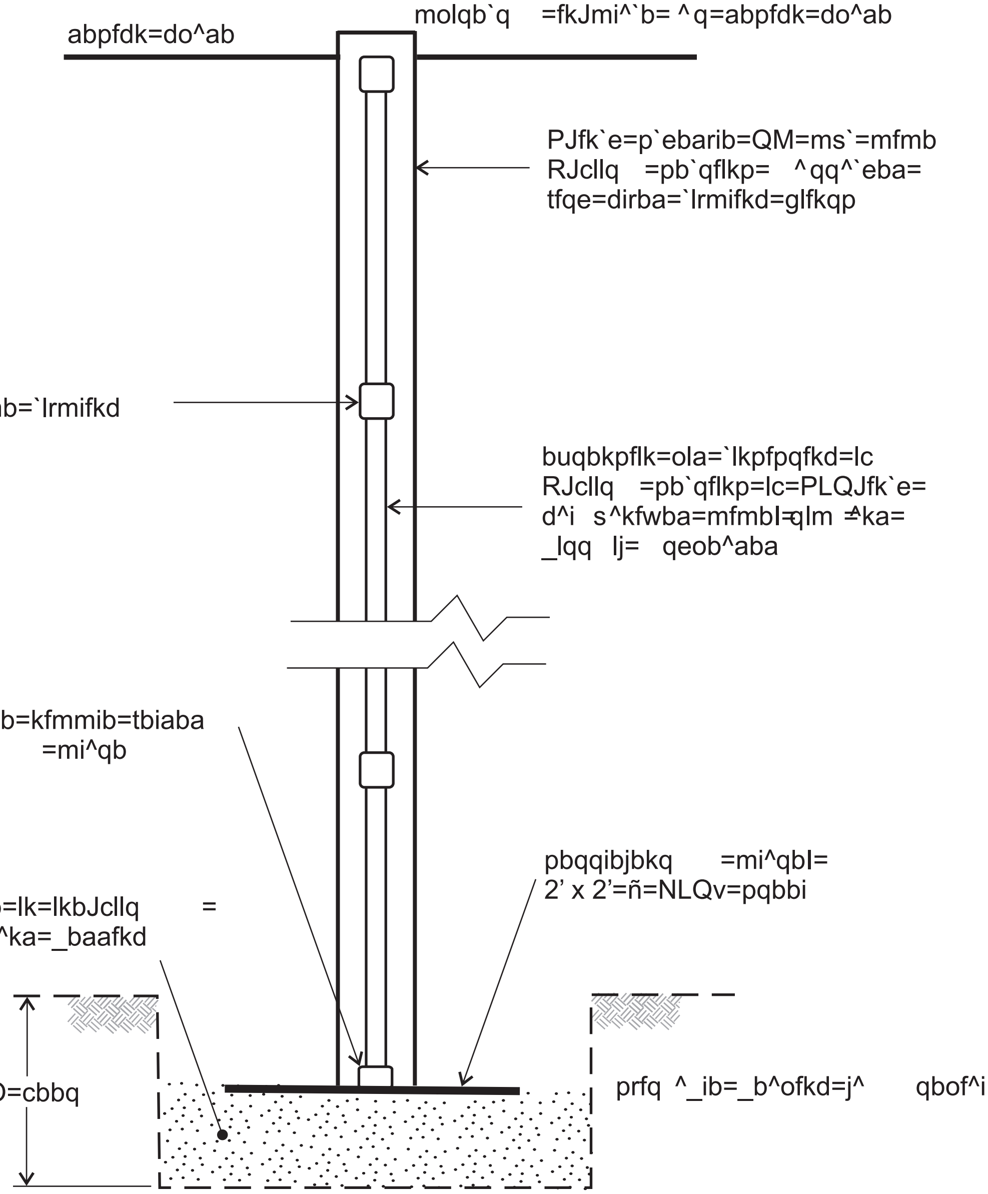

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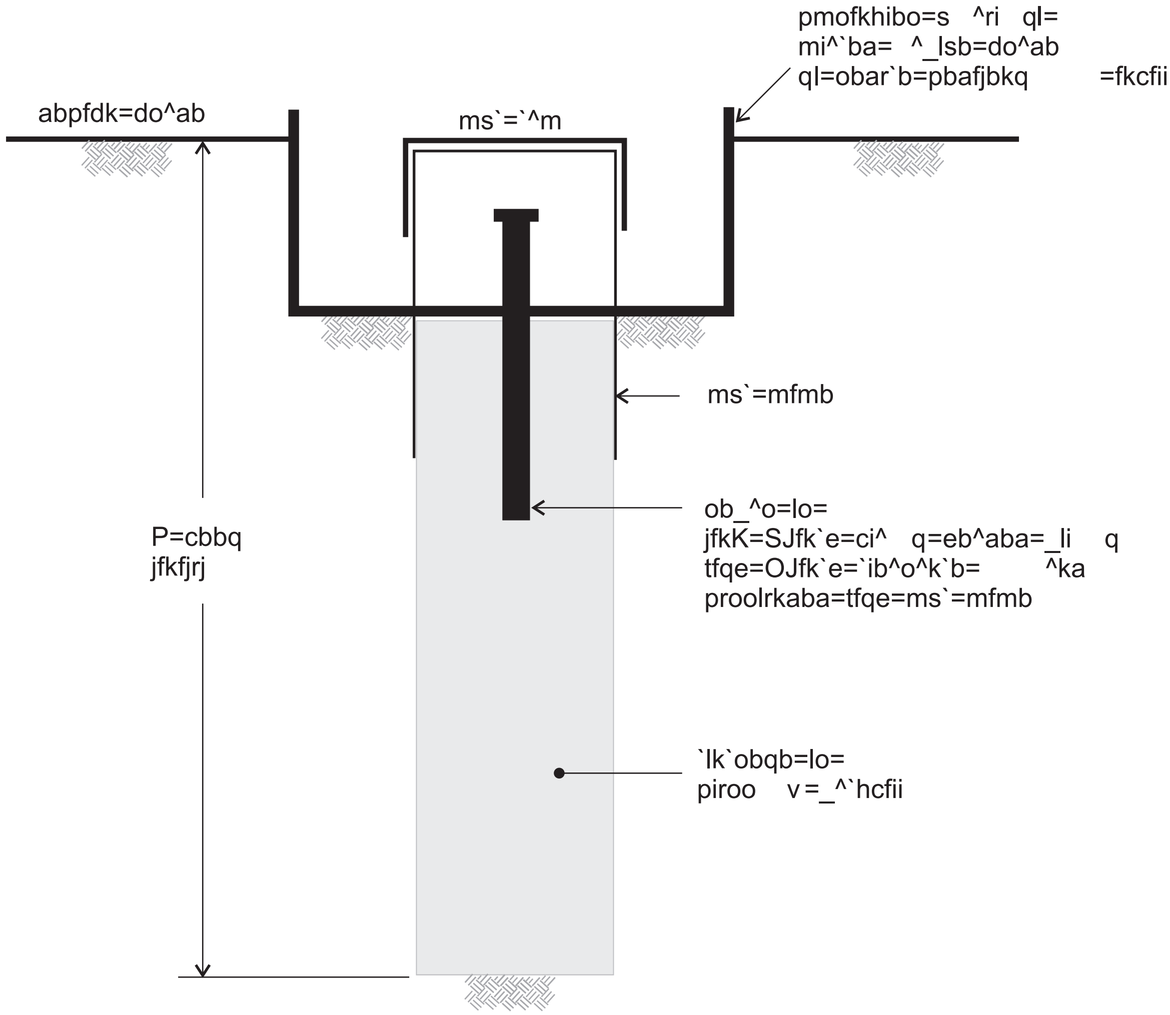

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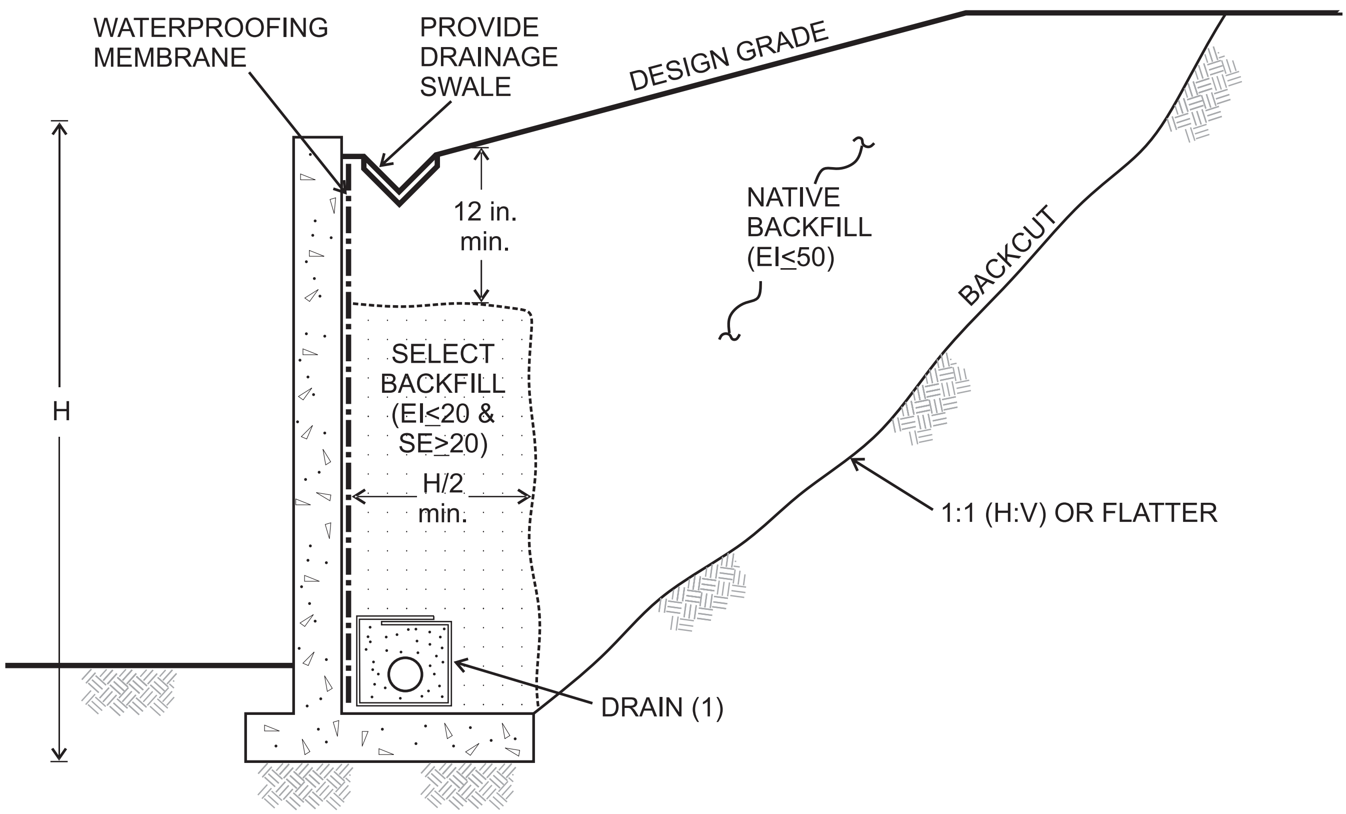
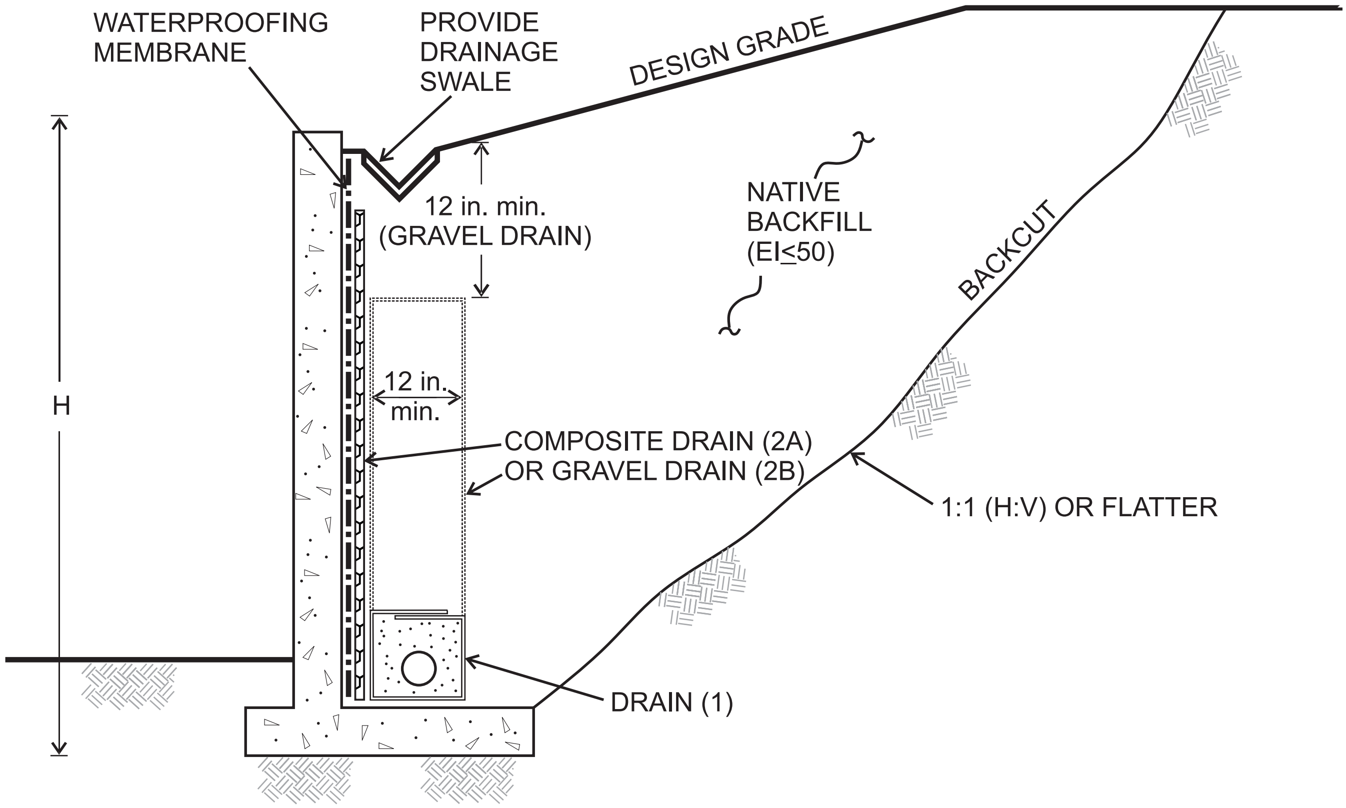


NOTES: (1) DRAIN: 4-INCH PERFORATED ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE PLACED PERFORATIONS DOWN AND SURROUNDED BY A MINIMUM OF 1 CUBIC FEET OF 3/4 INCH ROCK OR APPROVED EQUIVALENT SUBSTITUTE AND WRAPPED IN MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

VER 1.0

NTS

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.	RETAINING WALL ALT. A - SELECT BACKFILL	DETAIL RTW-A



NOTES: (1) DRAIN: 4-INCH PERFORATED ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE PLACED PERFORATIONS DOWN AND SURROUNDED BY A MINIMUM OF 1 CUBIC FEET OF 3/4 INCH ROCK OR APPROVED EQUIVALENT SUBSTITUTE AND WRAPPED IN MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

(2A) COMPOSITE DRAIN SYSTEM: MIRAFI G200N, DELTA DRAIN 2000/6000/6200 OR APPROVED EQUIVALENT SUBSTITUTE CONNECTED TO DRAIN (1)

(2B) GRAVEL DRAIN: MINIMUM 12-INCH WIDE 3/4-INCH GRAVEL BLANKET WRAPPED IN MIRAFI FILTER FABRIC (140 OR APPROVED EQUIVALENT SUBSTITUTE)

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AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.	RETAINING WALL ALT.B - NATIVE BACKFILL	DETAIL RTW-B

APPENDIX F

HOMEOWNER MAINTENANCE GUIDELINES

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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 then 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.

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
5333 Mission Center Road, Suite 390
San Diego, California 92108
Telephone: (619) 708-1649 Fax: (714) 409-3287

Accretive Investments, Inc.
12275 El Camino Real, Suite 220
San Diego CA 92130

September 11, 2012
P/W 1102-01.01
Report No. 1102-01.01-B-2

Attention: Mr. Jon Rilling

Subject: ***Supplemental EIR Level Geotechnical Review of Proposed Offsite Improvements, Tentative Tract Map, Lilac Hills Ranch Community, Escondido, California***

References: See Appendix A

Gentlemen:

Pursuant to your request, presented herein are the results of Advanced Geotechnical Solutions, Inc.'s (AGS), Supplemental EIR Level Geotechnical Review of Proposed Offsite Improvements, Tentative Tract Map for Lilac Hills Ranch Community, Escondido, California. AGS has been retained by Accretive Investments, Inc. to complete the geotechnical services supporting the tentative tract approval process for this project.

AGS has reviewed the referenced geotechnical documents prepared by AGS and Pacific Soils Engineering, Inc. (PSE), conducted additional field mapping, performed additional engineering and geologic analyses, and reviewed the latest Offsite Improvements Plans – Master Tentative Map, Lilac Hills Ranch, Sheets 6 through 8, prepared by Landmark Consulting.

The purpose of this geotechnical review is to evaluate the proposed offsite improvements associated with processing of the Tentative Tract Map relative to the near-site and on-site geologic and geotechnical conditions and provide conclusions and recommendations to aid in the development of the project. The offsite improvement plans prepared by Landmark Consulting were provided to AGS for preparation of this report. These maps are included in this document with appurtenant geologic and geotechnical data superimposed upon them.

Advanced Geotechnical Solutions, Inc., appreciates the opportunity to provide you with geotechnical consulting services and professional opinions. If you have any questions, please contact the undersigned at (619) 708-1649.

Respectfully Submitted,
Advanced Geotechnical Solutions, Inc.

PROFESSIONAL ENGINEER REGISTERED JEFFREY A. CHANEY No. 2314 Exp. 6/30/13 GEOTECHNICAL STATE OF CALIFORNIA

JEFFREY A. CHANEY, Vice President
GE 2314, Reg. Exp. 6-30-13

PROFESSIONAL GEOLOGIST PAUL J. DERISI No. 2536 CERTIFIED ENGINEERING GEOLOGIST Exp. 5/31/13 STATE OF CALIFORNIA

PAUL DE RISI, Vice President
CEG 2536, Reg. Exp. 5-31-13

Distribution: (2) Addressee
(1) Landmark Consulting Attn: Mark Brencik

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ATTACHMENTS:

Supplemental EIR Level Geotechnical Review of Offsite Improvements Tentative Tract for Lilac Hills Ranch Community, Escondido, California

1.0 INTRODUCTION

1.1. Background and Purpose

The purpose of this report is to provide a "Tentative Tract Map" (TTM) level geotechnical study that may be utilized to support the EIR submittal for the proposed Offsite Improvements for the Lilac Hills Ranch Community Tentative Tract Map located in Escondido, California. This report has been prepared to address the most current offsite improvement plans prepared by Landmark Consulting in a manner consistent with County of San Diego geotechnical report guidelines and current standard of practice. Geotechnical conclusions and recommendations are presented herein and the items addressed include: 1) Unsuitable soil removals and remedial grading; 2) Cut, fill and natural slope stability; 3) Potential geologic hazards and general mitigation measures for these hazards; 4) Remedial and design grading recommendations; 5) Rippability of the granitic rock in the vicinity of the improvements; and 6) General foundation design recommendations based upon anticipated as graded soil conditions.

1.2. Scope of Study

This study is aimed at providing geotechnical/geologic conclusions and recommendations for development of offsite roadway and associated infrastructure improvements (sewer and water).

The scope of this study included the following tasks:

- Review of pertinent published and unpublished geologic and geotechnical literature, maps, and aerial photographs readily available to this firm (Appendix A).
- Review and compile previous subsurface data from PSE (2007) and AGS (2012).
- Perform confirmatory geologic mapping on previously studied areas and conduct additional geologic mapping on the proposed offsite areas.
- Transfer selected geologic and geotechnical information generated from this and previous investigations onto the offsite improvement plans prepared by Landmark Consulting, included as Plates 1 thru 3. These plans depict existing grades and the approximate limits of the proposed improvements. AGS has added the approximate limits of surface geologic units based upon field mapping conducted during this and previous studies.
- Conduct a geotechnical engineering and geologic hazard analysis of the site.
- Conduct a limited seismicity analysis.
- Define remedial grading requirements.
- Discussion of slope stability for cut and fill slopes.
- Data analyses in relation to the site specific proposed improvements.

- Analysis of the excavation characteristics (i.e. rippability) of onsite bedrock materials.
- Discussion of pertinent geologic and geotechnical topics.
- Prepare general foundation design parameters which can be used for preliminary design.
- Prepare this supplemental geotechnical offsite improvement review report with exhibits summarizing our findings. This report is suitable for design support and regulatory review.

1.3. Geotechnical Study Limitations

The conclusions and recommendations in this report are professional opinions based on the data developed during this and previous investigations. The conclusions presented herein are based upon the current design as reflected on the included offsite improvement maps. Changes to the plans would necessitate further review.

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.

2.0 SITE LOCATION, DESCRIPTION AND PROPOSED IMPROVEMENTS

The Lilac Hills Ranch Community is located in northern unincorporated San Diego County, ¼-mile east of the Interstate 15 corridor with freeway access off the Old Highway 395 Interchange (Figure 1 - Site Location Map in AGS 2012). The project site is located to the south and west of West Lilac Road with State Route 76 to the north, downtown Valley Center 10 miles to the east, downtown Escondido 16 miles to the south, and Interstate 15 and Old Highway 395 to the west. The Lilac Hills Ranch Community project is located entirely in the Escondido zip code (92026) and occurs primarily within the westernmost portion of the Valley Center Community Planning Area (CPA) although a small portion is within the Bonsall Sub-regional Plan Area (see Figure 2 in AGS 2012). The proposed offsite improvements consist of the following:

- **West Lilac Road Widening** - The proposed widening will begin at the intersection of West Lilac Road and old Highway 395 and extend westerly approximately 700 feet near the western edge of the concrete Rainbow Bridge. These improvements will consist of widening the roadway from approximately 35 feet to approximately 60 feet. Cuts and fills of up to 15 feet are proposed at slope ratios of 2:1 (horizontal to vertical). No widening or structural changes are proposed for the Rainbow Bridge structure. Plate 1 depicts the proposed improvement limits and tentative design of these proposed offsite improvements.
- **Old Highway 395 between Gopher Canyon Road and Circle "R" Road -** Minor improvements are proposed on Old Highway 395 in the general vicinity of the intersections of Circle "R" Road and Gopher Canyon Road. Primarily these improvements consist of signalization and minor surface modifications. Plate 2 depicts

the proposed improvement limits and tentative design of these proposed offsite improvements.

- **Mountain Ridge Road Sewer Force Main –** The proposed sewer force main is approximately 2,570 lineal feet long and will be begin at the southerly end of Phase 5 extending south within the existing alignment of Mountain Ridge Road to the intersection of Circle "R" Drive. Preliminary design indicates that the force main will require cuts of 5 to 10 feet below existing grade to the ultimate invert elevation of the pipeline. It is anticipated that the proposed pipeline will consist of CML & C pipe. Minor grading adjacent to portions of Mountain Ridge Road are also anticipated with cuts and fills on the order of 10 feet or less with slope ratios of 2:1 (horizontal to vertical). Plate 3 depicts the proposed alignment of the sewer force main.
- **Covey Lane Widening -** Covey Lane will be re-aligned and widened to approximately 36 feet from the intersection of West Lilac Road extending westerly approximately 240 feet to accommodate a right-hand turn lane to Rodriguez Road and West Lilac Road. Cuts and fills of approximately 5 feet are proposed at slope ratios of 2:1 (horizontal to vertical) on the northerly and southerly sides of Covey Lane.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1. Current Investigation

AGS has performed additional geologic mapping for this supplemental investigation of the proposed offsite improvements, and utilized the results of our previous subsurface investigation (AGS 2012) and those conducted by PSE (2007) in preparing this study. As part of our services AGS has integrated appurtenant information from this and previous investigations on the conceptual design sheets for offsite improvements prepared by Landmark Consulting (Plates 1 thru 3) and prepared this report with our findings and recommendations.

4.0 ENGINEERING GEOLOGY

4.1. Geologic Analysis

4.1.1. Literature Review

AGS has reviewed the referenced geologic documents in preparing this study. Where deemed appropriate, this information has been included with this document. Of particular use are the maps by Kennedy (2000), Tan (2000), PSE (2007), and AGS (2012).

4.1.2. Aerial Photograph Review

AGS has re-visited the aerial photographs reviewed during previous studies and has taken advantage of recent web content aerial photographs. No features in addition to those identified by previous studies were noted.

4.1.3. Field Mapping

The geologic contacts mapped by AGS during this investigation are based upon additional field mapping, our familiarity with the site and from the previously conducted surface and subsurface information obtained during the Tentative Tract level study (AGS 2012).

4.2. Geologic and Geomorphic Setting

The Lilac Hills Ranch Community offsite improvements are located in the lower Peninsular Range Region of San Diego County, a subset of the greater Peninsular Ranges Geomorphic Province of California. This portion of the Peninsular Ranges is underlain by the intrusive southern California Batholith. Approximately two (2) miles northwest of the project lay the major drainage of the area, the San Luis Rey River, meandering to empty into the Pacific Ocean in Oceanside. Agua Tibia Mountain lies north of the river.

This portion of San Diego County is made up of foothills that span elevations from 600 to 2000 feet above mean sea level (MSL). It is characterized by rolling and hilly uplands that contain frequent narrow and winding valleys. The Lilac Hills Ranch Community offsite improvements are in the lower rolling hills area.

The rolling hills are predominately composed of Tonalite of the Couser Canyon geologic formation with a minor amount of the Granodiorite of Indian Mountain exposed at the northern boundary of the project (Kennedy, 2000; Tan, 2000). Tonalite is an igneous, plutonic (intrusive) rock, of felsic composition, with phaneritic texture and a granodiorite is an intrusive igneous rock similar to granite, but containing more plagioclase than orthoclase-type feldspar. These two bedrock types will be referred to with the more common term “granite” throughout this document. These igneous rocks are deeply (five to forty feet) weathered within the proposed Lilac Hills Ranch Community.

4.3. Stratigraphy

The geologic units underlying the project are characterized by weathered and decomposed granitic rocks with a very minor amount of exposed outcrops of hard granitic boulder corestones. A relatively thin veneer of surficial units including undocumented artificial fill, topsoil, alluvium and older alluvium cap the granitic rocks. The enclosed geologic maps (Plates 1 through 4) show the presently mapped location of the units. A brief description of the units is described below:

4.3.1. Surficial Units

Surficial units onsite include artificial fill (af), Topsoil (unmapped), Alluvial Deposits (Qal), and Older Alluvium (Qoal). Detailed descriptions of these units are presented below.

4.3.1.1. Artificial Fill (af)

Undocumented artificial fills are located throughout the Lilac Hills Ranch Community associated with past and present land use including residential construction, farming operations, private roadway construction, local water retention embankments, utility construction, and pad areas, among other minor land uses. Previously placed compacted fill soils will likely exist within and immediately adjacent to highway off ramps and County constructed roadways. The mapped locations of the most prominent fills are shown on the accompanying plates however; due to the map scale numerous lesser fills are present but unmapped. Future studies may determine documentation regarding the engineering of fills and how present site development plans would impact the function of these fills.

The vast majority of the fill is locally derived and consists of light reddish brown, clayey and silty sands that are commonly dry to slightly moist and loose to moderately dense.

4.3.1.2. Topsoil (no map symbol)

Surficial weathering over the majority of the overall project site has resulted in a thin veneer of topsoil throughout the project. The topsoil is composed of medium brown to reddish brown clayey to silty sands that are dry to slightly moist and loose to moderately dense.

4.3.1.3. Alluvium (Qal)

Alluvial deposits occupy the canyon areas and active drainage courses throughout the improvements on the southern portions of Highway 395 and crossing the Mountain Ridge Road alignment. The Holocene-aged alluvium varies from light orange brown to brown silty and clayey sand to sandy silt that is damp to locally wet, loose and soft to moderately dense and firm. The thickness of the alluvium is anticipated to range from a few feet to greater than 15 feet. These deeper deposits will be found in the drainages in the lower portions of Highway 395 in vicinity of Circle “R” Road.

4.3.1.4. Older Alluvium (Qoal)

Early Holocene to Pleistocene Older Alluvium has been mapped onsite and in areas is evident as a distinct geomorphic surface. It has also been observed in some areas below the younger alluvial deposits where it was not removed by erosion between the two distinct depositional episodes. The Older Alluvium has distinctly well-developed reddish to orange-brown color due to its age and exposure to weathering elements since its deposition. Composed of silty to clayey sands that are moderately hard to hard and slightly moist to moist, the moderately oxidized earth material is well consolidated.

4.3.2. Bedrock Units

4.3.2.1. “Granitic Rocks” (Kgr)

The majority of the project is underlain by undivided Tonalite with lesser exposures of Monzogranite of Merriam Mountain in the south and Granodiorite of Indian Mountain in the north. These bedrock units are identified and discussed as "granite" in this document based on their similar plutonic origin and physical properties. In most areas the bedrock materials are deeply weathered. Localized hard boulder corestones were observed at ground surface in only a few areas area.

4.4. Geologic Structure and Tectonic Setting

4.4.1. Regional Faulting

The San Andreas fault zone is the dominant and controlling tectonic stress regime of southern California (Figure 4 in AGS 2012). As the boundary between the Pacific and North American structural plates, this northwest trending right lateral, strike–slip, active fault has controlled the crustal structural regimes of southern California since Miocene time. Numerous related active fault zones with a regular spacing, including the Elsinore-Whittier-Chino, Newport-Inglewood-Rose Canyon, and San Jacinto fault zones characterize the stress regime and also trend to the northwest as do the Santa Ana Mountains and the Peninsular Ranges.

The Temecula section (Wildomar Fault) of the Elsinore fault zone is closest to the project and is located 7.8 miles to the northeast. The next closest fault zone to the site is the Oceanside section of the Newport-Rose Canyon fault zone at approximately 20 miles to the southwest. The Anza section of the San Jacinto fault zone is approximately 32 miles to the northeast and the San Bernardino section of the San Andreas fault zone is about 55 miles to the northeast.

4.4.2. Local Faulting

Alquist-Priolo County Special Studies Fault Zones and San Diego County Fault Zones are not located onsite (Figure 4 in AGS (2012). The most influential geologic faults potentially affecting the property are the active and potentially active Williard, Wildomar, Wolf Valley and Temecula segments of the Elsinore Fault System. No faults have been mapped onsite or within the proposed offsite improvement areas on published geologic maps and none were observed during this and previous geologic studies.

4.4.3. Geologic Structure

Dominant foliations, fracture patterns or other structural features common to granitic rocks were not mapped or observed during this or previous studies. Geologic maps by Kennedy and Tan are also void of any such mapped features. The highly weathered nature of the granitic rock apparently has contributed significantly to this lack of observable features. Dike patterns offsite indicate a northwest trend that is typical of rocks in the Peninsular range province.

4.5. Groundwater

Shallow groundwater was not observed during this or previous studies. Localized springs and seeps were observed within the active lager drainages. For the most part the proposed offsite improvements in these areas are not considered to be wetlands, excepting the Sewer force main following Mountain Ridge Road at the southern end of Lilac Hills Ranch Community. Groundwater may be encountered during construction of the sewer force main within the active drainage.

4.6. Non-seismic Geologic Hazards

4.6.1. Mass Wasting and Debris Flows

The majority of the site is sloping to the southwest at shallow to moderate slope ratios and is capped by a relatively thin veneer of surficial earth material underlain by granitic rocks and is considered not susceptible to mass wasting. No evidence of past landsliding or debris flows has been mapped within the limits of the proposed offsite improvements. Since there is no steep terrain offsite or onsite, the potential for debris flows emanating from the mouths of the up-gradient drainages are considered to be remote.

4.6.2. Rock Fall

The potential for rock fall in the areas of the proposed offsite improvements is considered to be very low to low given the lack of rock outcrops and the topography within the proposed limits of the improvements.

4.6.3. Flooding

The site is not located within a County of San Diego Flood Plain Zone. Hydrology studies should be provided by the Civil Engineer.

4.6.4. Subsidence and Ground Fissuring

Owing to the very shallow granitic bedrock underlying the site, subsidence and ground fissuring potential at the site is considered nil.

4.7. Seismic Hazards

The site is located in the tectonically active Southern California area, and will therefore likely experience shaking effects from earthquakes. The Near Source Shaking Zones of the County of San Diego (Figure 5 in AGS, 2012) shows the distance of the site from near source shaking zones. 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, the direction of propagation of the seismic wave and the underlying soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction, seismically induced slope failure 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. The following seismic hazards discussion is guided by the California Building Code (2010), CDMG (2008), and Martin and Lew (1998).

4.7.1. Surface Fault Rupture

Surface rupture is a break in the ground surface during or as a consequence of seismic activity. To a large part, research supports the conclusion that active faults tend to rupture at or near pre-existing fault planes. No faults much less active faults have been mapped within or near the project. As such, it is appropriate to conclude that the potential for surface fault rupture is very low.

4.7.2. Ground Motions

As noted, the site is within the tectonically active southern California area, with segments of the Elsinore Fault system within 8 miles of the site. The potential exists for strong ground motion that may affect future improvements. As part of this assessment, AGS utilized the California Geologic Survey Probabilistic Seismic Hazards Seismic Hazards Mapping Ground Motion Page. A site location with latitude of 33.2905˚N and longitude -117.1333˚W was utilized. Ground motions (10% probability of being exceeded in 50 years) are expressed as a fraction of the acceleration due to gravity (g). Three values of ground motion are shown, peak ground acceleration (Pga), spectral acceleration (Sa) at short (0.2 second) and moderately long (1.0 second) periods. Ground motion values are also modified by the local site soil conditions. Ground motion values are shown for two different site conditions: granitic rock (site category B) and Stiff soil (Older Alluvium and artificial fill) (site category D).

TABLE 5.7.2
SELECTED GROUND MOTIONS*

	Rock	Stiff Soil
Pga (g)	0.349	0.395
Sa 0.2 sec	0.835	0.951
Sa 1.0 sec.	0.314	0.479

*NEHRP Soil Corrections were used to calculate Soft Rock and Alluvium. Ground Motion values were interpolated from a grid (0.05 degree spacing) of calculated values. Interpolated ground motion may not equal values calculated for a specific site, therefore these values are not intended for design or analysis.

At this point in time, non-critical structures (commercial, residential, and industrial) are usually designed according to the 2010 California Building Code and that of the controlling local agency. However, liquefaction/seismic slope stability analyses, critical structures, water tanks and unusual structural designs will likely require site specific ground motion input.

4.7.3. Liquefaction

Liquefaction is the phenomenon in which the buildup of excess pore pressures, in saturated granular soils due to seismic agitation, results in a temporary “quick” or “liquefied” condition. The majority of the offsite improvements are not within an area zoned by the County of San Diego as a Potential Liquefaction Area (Figure 6, AGS 2012) and the potential for liquefaction is considered to be nil. However, portions of the offsite roadway improvements along Highway 395 and Gopher Canyon Road may be located in potentially liquefiable areas. The potential for liquefaction and seismically induced settlement will be reduced to "low to very low" in the southern portions of the improvements in the vicinity of Highway 395 and Gopher Canyon Road after the proposed remedial grading recommendations outlined herein are conducted.

4.7.4. Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. Due to the anticipated removals proposed herein the potential for lateral spreading is considered to be very low.

4.7.5. Seismically Induced Dynamic Settlement

Seismically induced dynamic settlement occurs in response to seismic shaking of loose sandy earth materials. The source of settlement is volumetric strain associated with liquefaction of saturated soils strata, and/or, the rearrangement of sandy particles in dry, relatively loose layers of sandy soils (cohesionless). These two sources of settlement potential are mutually exclusive, as such, if the groundwater rises, the liquefaction potential and its adverse effects increase, while dry sand settlement potential decreases, and vice-versa.

Due to the anticipated removals proposed herein, the density and cementation of the alluvium to be left in-place and the hardness of the underlying granitic rock, the potential for seismically induced settlement is considered low.

4.7.6. Seismically Induced Landsliding

Seismically induced landsliding is considered to be very low for engineered fill slopes. For cut slopes excavated in the granitic rock, or on the remaining shallow natural slopes the potential for seismically induced landsliding is considered to be very low.

4.7.7. Earthquake Induced Flooding

Earthquake induced flooding can be caused by tsunamis, dam failures, or seiches. Also, earthquakes can cause landslides that dam rivers and streams, and flooding can occur upstream above the dam and also downstream when these dams are breached. A seiche is a free or standing-wave oscillation on the surface of water in an enclosed or semi-enclosed basin. The wave can be initiated by an earthquake and can vary in height from several centimeters to a few meters. Due to the lack of a freestanding body of water nearby, the potential for a seiche impacting the site is considered to be non-existent.

Considering the lack of any dams or permanent water sources upstream, earthquake induced flooding caused by a dam failure is considered to be non-existent.

Considering the distance of the site from the coastline, the potential for flooding due to tsunamis is nil.

5.0 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.

5.1. Material Properties

5.1.1. Excavation Characteristics

Based on our previous experience with similar projects near the subject site and review of the information gathered during this and previous investigations, it is AGS's opinion that the shallow and surficial earth materials above the weathered and un-weathered granitic rock onsite can be readily excavated with conventional grading equipment however deeper cuts within the granitic rock potentially require moderate to heavy ripping in the weathered portions and potentially heavy ripping to blasting of throughout much of the un-weathered granitics.

AGS performed a preliminary rippability evaluation for the Lilac Hills Ranch Community earlier this year (AGS 2012) and reviewed previous rippability information from the previous consultant (PSE). This rippability evaluation was based upon the performance capabilities of a Caterpillar D9N bulldozer and our experience with similar projects in the region.

In general, the ease of rock rippability depends upon factors such as the rock type, rock hardness and density, the amount of weathering, and the existence and characteristics of discontinuities such as joint spacing, foliation, or random fractures. For example, a rock mass that is weathered and exhibits well-developed discontinuities, such as joints, will be easier to excavate than a compositionally similar rock mass that lacks discontinuities and significant weathering. This is because weathering typically decreases cohesive rock strength, and discontinuities typically provide a mechanism that allows the rock mass to readily part upon stress (Hoek and Bray, 1981).

For the subject offsite improvements, the main controls on rippability are joints, fractures and foliations, the degree of weathering at depth, and the depth and size of the cut areas and whether the cuts will be excavated in a trench with excavators (sewer force main) or in a grading operation (road widening). Based upon our field mapping and previous studies on the tentative tract, the bedrock generally shows a weathered halo that ranges from approximately 5 to greater than 50 feet in depth below the surface.

In general, given the relatively shallow cuts (less than 15 feet) it is AGS's opinion that the majority of the proposed improvements can be graded with conventional grading equipment. However, specialized grading techniques (heavy ripping with D-9 Bull dozers and the use of large excavators with "Hoe-Rams", and possibly limited areas of blasting) will likely be required throughout the cuts situated in the granitic rock.

5.1.2. Oversized Materials

Oversized rock (> 24 inches) will be generated in the deeper cuts and over excavations within the granitic bedrock. This rock may be incorporated into the compacted fill section to within ten (10) feet of finish grade or within two (2) feet of the deepest utility (if utility is greater than ten (10) feet). Oversize rock is not to be placed within areas of proposed drainage structures and should be kept minimally five (5) feet outside and below proposed culverts, pipes, etc.

Maximum rock size between three (3) feet and ten (10) feet of finished grade is restricted to 24 inches and in the upper three (3) feet from finish grade is restricted to a maximum rock size of eight (8) inches. Variances to the above rock hold-down must be approved by the owner, geotechnical consultant and governing agencies.

5.1.3. Compressibility

The onsite materials that are compressible include undocumented artificial fills, alluvium, weathered older alluvium, and weathered bedrock. Highly compressible materials will require removal from settlement sensitive areas prior to placement of fill and where exposed at grade in cut areas.

5.1.4. Collapse Potential/Hydro-Consolidation

The hydro-consolidation process is a singular response to the introduction of water into collapse-prone alluvial soils. Upon initial wetting, the soil structure and apparent strength are altered and a virtually immediate settlement response occurs. Recommended measures to mitigate potential for differential settlement due to hydro-collapse include removal/recompaction and/or foundation design, such as described in Sections 6.1 and 7.1 of this report.

5.1.5. Expansion Potential

Based upon the sampling and associated laboratory testing conducted by AGS and PSE the near surface soils are considered to exhibit “Very Low” to “Moderately” expansion potential ($0 \leq EI \leq 90$), with the majority of the onsite soils falling into the “Very Low “to “Low” expansion potential range. Typical mitigation measures for expansive soils include: structural design, pre-saturation and overexcavation where the higher expansion characteristics are present.

5.1.6. Shear Strength

Shear strength testing was conducted by AGS and by PSE on remolded samples that were collected during past studies onsite (see Appendix C, AGS 2012). Within the onsite bedrock units, the in-situ shear strength and fracture patterns are the most significant factors in cut slope and natural slope stability. Typically, the granitic rock possesses considerable shear strength and can stand unsupported at relatively steep slope ratios. The "older alluvium" generally possesses good in-situ shear strength except where weathered such as the upper five feet. Alluvium generally can be characterized as possessing fair to poor strength characteristics. The shear strength of the fill soils created during grading generally will exhibit good shear strength for fill slopes and for support of structures. The shear strengths recommended by AGS for design are presented in Table 5.1.6.

TABLE 5.1.6
RECOMMENDED SHEAR STRENGTHS FOR DESIGN

Material	Cohesion (psf)	Friction Angle (degrees)	Density (pcf)
Artificial Fill Compacted (afc) & Older Alluvium (Qoal)	150	35	125
Granitic Bedrock (Kgr)	500	40	140

5.1.7. Chemical and Resistivity Test Results

The test results from AGS's and PSE's previous investigations in the general area indicate that sulfate concentrations for the onsite soils will be below 0.1 percent, which corresponds to a "very low" sulfate exposure when classified in accordance with ACI 318-05 Table 4.3.1 (per 2010 CBC). Testing should be conducted during and upon completion of grading operations to further evaluate the sulfate content and potential corrosivity on the onsite soils.

5.1.8. Earthwork Adjustments

The following average earthwork adjustment factors are presented for use in evaluating earthwork quantities. These numbers are considered approximate and should be refined during grading when actual conditions are better defined. Contingencies should be made to adjust the earthwork balance during grading if these numbers are adjusted.

TABLE 5.1.8
EARTHWORK ADJUSTMENTS

Geologic Unit	Approximate Range
Artificial Fill Undocumented (Afu)	8% to 12% Shrink
Artificial Fill Compacted (Afc)	0% to 2% Shrink
Topsoil & Alluvium (Qal)	8% to 12% Shrink
Older Alluvium (Qoal)	0% to 5% Bulk
Granitic Bedrock (Kgr) - rippable	10% to 18% Bulk
Granitic Bedrock (Kgr) - non-rippable	18% to 25% Bulk

5.1.9. Pavement Support Characteristics

Compacted fill derived from onsite soils and cuts within the older alluvium and granitic rock is expected to possess good to very good pavement support characteristics. Testing should be completed once subgrade elevations are reached for the onsite roadways. For preliminary planning purposes, AGS has used an R-Value of 40 for the design of pavement sections for the proposed roadway improvements.

5.2. Analytical Methods

5.2.1. Slope Stability Analysis

Stability analyses were performed in our previous study (Appendix D, AGS 2012) for both static and seismic (pseudo-static) conditions using the GSTABL7 computer program. The Modified Bishop method was used to analyze circular type failures. The critical failure surface determined in the static analysis was used in the pseudo-static analysis. A horizontal destabilizing seismic coefficient (kh) of 0.15g was selected for the site and used in the pseudo-static analyses. Peak shear strengths have been utilized in the pseudo-static analysis.

Surficial stability analyses were conducted using an infinite height slope method assuming seepage parallel to the slope surface.

5.2.2. Pavement Design

Asphalt concrete pavement sections have been designed using the recommendations and methods presented in the Caltrans Highway Design Manual. Portland cement concrete pavement for onsite roads and driveways has been designed in accordance with the recommendations presented in the "Design of Concrete Pavement for City Streets" by the American Concrete Pavement Association.

5.2.3. Bearing Capacity and Lateral Pressure

Ultimate bearing capacity values were obtained using the graphs and formula presented in NAVFAC DM-7.1. Allowable bearing was determined by applying a factor of safety of at least 3 to the ultimate bearing capacity. Static lateral earth pressures were calculated using Rankine methods for active and passive cases.

6.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Based on the information presented herein and our experience in the vicinity of the subject site, it is AGS's opinion that the proposed offsite improvements for Lilac Ranch Hills Community are feasible, from a geotechnical point of view, provided that the constraints discussed in this report are addressed in the design and construction of each proposed improvement. Presented below are issues identified by this study or previous studies as possibly impacting site development. Recommendations to mitigate these issues and geotechnical recommendations for use in planning and design are presented in the following sections of this report.

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, AGS 2012).

6.1. Site Preparation and Removals/Overexcavation

Guidelines to determine the depth of removals are presented below; however, the exact extent of the removals must be determined in the field during grading, when observation and evaluation of the greater detail afforded by those exposures can be performed by the Geotechnical Consultant. In general, removed soils will be suitable for reuse as compacted fill when free of deleterious materials and after moisture conditioning.

Removal of unsuitable soils typically should be established at a 1:1 projection to suitable materials outside the proposed engineered fills. Front cuts should be made no steeper than 1:1, except where constrained by other factors such as property lines and protected structures. Removals should be initiated at approximately twice the distance of the anticipated removal depth, outside the engineered fills. The bottoms of all removal areas should be observed, mapped, and approved by the Geotechnical Consultant prior to fill placement. It is recommended the bottoms of removals be surveyed and documented.

6.1.1. 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.

6.1.2. Topsoil (no map symbol)

All topsoil should be removed before placement of compacted fill.

6.1.3. Artificial Fill - Undocumented (map symbol af)

All undocumented fill material in designed fill areas and/or where exposed in cuts should be removed. Undocumented fill removals are anticipated to range in depth from two to fifteen, with possibly deeper localized areas. It is anticipated that these materials will be suitable for re-use provided that all deleterious materials (brush, roots, ect.) is removed prior to incorporation into fill.

6.1.4. Artificial Fill - Compacted (map symbol afc)

Previously placed compacted fill soils will likely be encountered within and immediately adjacent to highway off ramps and County constructed and maintained roadways. It is anticipated that the upper 1 to 3 feet of compacted fill soils may be weathered and unsuitable support of settlement sensitive improvements or placement of additional fill in their current condition and should be removed. The resulting removal bottoms should be observed by the Geotechnical Consultant to verify that adequate removal of unsuitable materials have been conducted prior to fill placement or construction of settlement sensitive improvements. It is anticipated that the removed materials will be suitable for re-use provided that all deleterious materials (brush, roots, ect.) is removed prior to incorporation into fill.

6.1.5. Alluvium (map symbol Qal)

All alluvium should be removed within a 1:1 projection of the designed fill and cut areas. Alluvium removals are anticipated to range from a few feet to as deep as twenty feet, with possibly deeper localized areas.

6.1.6. Older Alluvium (map symbol Qoal)

The upper three to four feet of older alluvium should be removed within a 1:1 projection of the designed fill areas and cut areas.

6.1.7. Granitic Rock (map symbol Kgr)

The upper one to three feet of highly weathered granitic rock should be removed within a 1:1 projection of the designed fill and cut areas.

6.1.8. Street Overexcavation

Streets that are cut into older alluvium and granitic rock could potentially pose excavation difficulties during utility and street installation. The granitic rock may potentially require heavy ripping, large excavators with hoe-rams and/or blasting in deeper cut areas in order to get to utility excavation depth. During mass grading, where such materials are exposed, consideration should be given to undercutting the street/utility areas during mass grading to minimize this condition. The undercut should extend at least one foot below the deepest utility. The undercut zone should be replaced with compacted fill in accordance with project standards as outlined herein.

6.1.9. Removals Along Grading Limits and Property Lines

Removals of unsuitable soils will be required prior to fill placement along the project grading limits. A where possible a 1:1 projection, from toe of slope or grading limit, outward to competent materials should be established. If due to property line constraints or the location of adjacent improvements site specific recommendations can be determined during grading.

6.2. Slope Stability and Remediation

Proposed maximum slope heights to be created during grading are on the order of 20 feet or less.

6.2.1. Cut Slopes

The highest proposed cut slope is less than 20 feet in height at a slope ratio of 2:1 (horizontal: vertical). Based upon the currently available information, we anticipate that proposed cut slopes in Older Alluvium and Granitic Rock will be grossly stable as designed. Calculations supporting AGS's conclusions and recommendations relative to cut slopes are represented in Appendix D of the Tentative Tract level geotechnical investigation prepared by AGS (2012).

Cut slopes should be observed by the Geotechnical Consultant during grading. Where cut slopes expose unfavorable geology such as day lighted joints, loose or raveling weathered granitic rock or where boulders may pose a rock fall problem, replacement of the unsuitable portions of the cut with stabilization fill will be recommended.

Terrace and down drains should be constructed on all cuts slopes in conformance to the San Diego County Grading Ordinance.

6.2.2. Fill Slopes

Fill slopes on the project are designed at 2:1 ratios (horizontal to vertical). The highest anticipated fill slope is approximately 20 feet high. Fill slopes, when properly constructed with onsite materials, are expected to be grossly stable as designed. Stability calculations supporting this conclusion are presented in Appendix D of the Tentative Tract level geotechnical investigation prepared by AGS (2012). Fill slopes will be subject to surficial erosion and should be landscaped as quickly as possible.

Keys should be constructed at the toe of all fill slopes “toeing” on existing or cut grade. Fill keys should have a minimum width equal to one-half the height of ascending slope, and not less than 15 feet. Unsuitable soil removals below the toe of proposed fill slopes should extend from the catch point of the design toe outward at a minimum 1:1 projection into approved material to establish the location of the key. Backcuts to establish that removal geometry should be cut no steeper than 1:1 or as recommended by the Geotechnical Consultant.

Terrace and down drains should be constructed on all cuts slopes in conformance to the San Diego County Grading Ordinance.

6.2.3. Skin Cut and Skin Fill Slopes

A review of the improvement plans did not indicate any significant design skin fill and skin cut conditions, however, skin cut or thin fill sections may be created during grading. For all such conditions, it is recommended that a backcut and keyway be established such that a minimum fill thickness equal to one-half the remaining slope height, and not less than 15 feet, is provided. Where the design cut is insufficient to remove all unsuitable materials, overexcavation and replacement with a stabilization fill will be required, as shown on Grading Detail 6 in Appendix E (AGS 2012).

6.2.4. Fill Over Cut Slopes

Fill over cut slopes should be constructed such that the cut portion is excavated first for geologic mapping and stability determination. If deemed stable then a “tilt-back” keyway half the remaining slope height or minimally twenty (20) feet wide should be established. Drains will be required for this condition with the locations determined based upon exposed field conditions.

6.2.5. Surficial Stability

The surficial stability of 2:1 fill and cut slopes, constructed in accordance with the recommendations presented herein, have been analyzed, and the analyses presented in Appendix D in AGS (2012) indicates factors-of-safety in excess of code minimums. When fill and cut slopes are properly constructed and maintained, satisfactory performance can be anticipated although slopes will be subject to erosion, particularly before landscaping is fully established.

6.2.6. Temporary Backcut Stability

During grading operations, temporary backcuts may occur due to grading logistics and during retaining wall construction. Backcuts should be made no steeper than 1:1 (horizontal to vertical) to heights of up to 20 feet, and 1½:1 (horizontal: vertical) for heights greater than 20 feet. Flatter backcuts may be necessary where geologic conditions dictate, and where minimum width dimensions are to be maintained.

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

6.2.7. Observation During Grading

All temporary slope excavations, including front, side and backcuts, and all cut slopes should be mapped to verify the geologic conditions that were modeled prior to grading.

6.3. Subsurface Drainage

Canyon subdrains should be constructed within the major drainages which will ultimately be filled as part of the mass grading of the site. Canyon subdrains will range in diameter from 6 to 8 inches in diameter and should be constructed in accordance with Grading Detail 1 and 2 (Appendix E, AGS 2012). Final determination as to the location and the size of these subdrain systems will be dependent upon the final finished design grades. Accordingly, once more detailed plans become available site specific recommendations will be prepared regarding the size, location and extant of the subdrain system for the project.

Due to the lack of a significant backcuts and the anticipated depth of fill in the toe areas after remedial grading, the need for backdrain systems are not anticipated at the toes of constructed fill slopes or fill over cut slopes. This should be further evaluated during future grading plan reviews and during grading. Backdrains, where required, should be constructed in accordance with Grading Detail 2 (Appendix E, AGS 2012).

Drains should be installed behind all retaining walls.

6.4. 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.

6.5. Earthwork Considerations

6.5.1. Compaction Standards

All fills should be compacted at least 90 percent of the maximum dry density as determined by ASTM D1557-09. All loose and or deleterious soils should be removed to expose competent compacted fill soils, firm native soils or bedrock. 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 D1557-09). Fill should be placed in thin (6 to 8-inch) lifts, moisture conditioned to optimum moisture or slightly above, and compacted to 90 percent of the maximum dry density (ASTM D1557-09) until the desired grade is achieved.

6.5.2. Benching

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

6.5.3. 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.

6.5.4. Haul Roads

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

6.5.5. Import Soils

The project is proposed to balance on site. If this changes the Geotechnical Consultant should be contacted.

6.5.6. Rock Excavation Considerations and Potential Grading Impacts

The impacts of grading and potential blasting with regard to dust control, noise, etc. is generally under the purview of others and the conditions of the regulating agency. Potential impacts to the surrounding community environment during grading, blasting and rock crushing should be evaluated by licensed, experienced grading and blasting contractors. The grading, blasting and rock crushing operations should be coordinated by the contractors to minimize the impact of the grading operation on the surrounding community environment and improvements. The grading and blasting contractors should follow the guidelines and permit conditions provided by the regulating agency.

6.5.7. Oversize Rock

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 two (2) foot below the deepest utility in street areas. Maximum rock size in the upper portion of the hold-down zone is restricted to eight (8) inches. Rock disposal details are presented on Detail 10, Appendix E (AGS 2012). 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

6.5.7.1. Rock Blankets

Rock blankets consisting of a mixture of fines, sand, gravel, and rock to a maximum dimension of 2 feet may be constructed. The construction of rock fill shall be continuously observed by the geotechnical consultant. The rocks should be placed on a 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, is without significant voids, and forms a dense, compact fill matrix. Adequate water shall be provided continuously during these operations.

Rock blankets may be extended to the slope face provided the following additional conditions are met: 1) no rocks greater than 12 inches in diameter are allowed within 6 horizontal feet of the slope face; 2) 50 percent of the material is to be three-quarters (3/4) inch minus by volume; and 3) back-rolling or track walking of the slope face is conducted at 4-foot verticals to meet project compaction specifications.

6.5.7.2. Rock Windrows

Rocks to maximum dimension of 4 feet may be placed in windrows in deeper soil fill areas in accordance with Grading Detail 10 (AGS 2012). The construction of rock fill shall be continuously observed by the geotechnical consultant. 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 by at least 15 feet of compacted fill, be staggered vertically and separated by at least 4 vertical feet of compacted fill. Windrows should not be placed within 10 feet of finish grade within structural fill areas, within 2 vertical feet of the lowest buried utility conduit in structural fills, or within 15 feet of the finish slope surface unless specifically approved by the owner, geotechnical consultant and governing agency.

6.5.7.3. 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.

6.5.7.4. 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.

6.5.8. Fill Slope Construction

Fill slopes may be constructed by preferably overbuilding and cutting back to the compacted core or by back-rolling and compacting the slope face. The following recommendations should be incorporated into construction of the proposed fill slopes.

Care should be taken to avoid spillage of loose materials down the face of any slopes during grading. Spill fill will require complete removal before compaction, shaping and grid rolling.

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.

6.5.8.1. 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.

6.5.8.2. 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.

6.5.9. Utility Trench Excavation and Backfill

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. The geotechnical consultant should be consulted on these issues during construction.

Mainline and lateral utility trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557-09. 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.

To reduce moisture penetration beneath the slab-on-grade areas, shallow utility trenches should be backfilled with lean concrete or concrete slurry where they intercept the foundation perimeter, or such excavations can be backfilled with native soils, moisture-conditioned to over optimum, and compacted to a minimum of 90 percent relative compaction.

7.0 DESIGN RECOMMENDATIONS

From a geotechnical perspective, the proposed development of the offsite improvements is feasible provided the following recommendations are incorporated into the design and construction. Preliminary design recommendations are presented herein and are based on some of the general soils conditions encountered during the recent investigation and described in the referenced geotechnical investigations. As such, recommendations provided herein are considered preliminary and subject to change based on the results of additional observation and testing that will occur during grading operations. Final design recommendations should be provided in a final rough/precise grading report.

7.1. Structural Design Recommendations

For the preliminary design of retaining walls, thrust blocks for the sewer force main and other similar structural improvements the following preliminary design parameters are presented.

7.1.1. Foundation Design

The following values may be used in preliminary foundation design:

Allowable Bearing:	2500 psf.
Lateral Bearing:	250 psf. per foot of depth to a maximum of 2500 psf. for level conditions.
Sliding Coefficient:	0.37

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 and should be evaluated by a qualified engineer.

7.1.2. Retaining Wall Design

The foundations for retaining walls of appurtenant structures structurally separated from the building structure may bear on properly compacted fill. The foundations may be designed in accordance with the recommendations provided in Table 7.1.2, Conventional Foundation Design Parameters. When calculating the lateral resistance, the upper 12 inches of soil cover should be ignored in areas that are not covered with hardscape. 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.

Retaining walls should be designed to resist earth pressures presented in the following table. These values assume that the retaining walls will be backfilled with select materials as shown in Detail RTW-A or native soils as shown in Detail RTW-B. The type of backfill ("select" or "native") should be specified by the wall designer and shown on the plans. Retaining walls should be designed to resist additional loads such as construction loads, temporary loads, and other surcharges as evaluated by the structural engineer.

TABLE 7.1.2
RETAINING WALL EARTH PRESSURES

“Native”* Backfill Materials (γ=125pcf, EI≤50)

	Level Backfill		Sloping (2:1) Backfill	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	K_a = 0.33	42	K_a = 0.54	67
Passive Pressure	K_p = 3.00	375	K_p = 1.12	140
At Rest Pressure	K_o = 0.50	63	K_o = 0.81	101

“Select”* Backfill Materials (γ=120pcf, EI≤20, SE≥20)

	Level Backfill		Sloping (2:1) Backfill	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	K_a = 0.28	34	K_a = 0.44	53
Passive Pressure	K_p = 3.54	420	K_p = 1.33	160
At Rest Pressure	K_o = 0.44	53	K_o = 0.75	90

Notes: “Select” backfill materials should be granular, structural quality backfill with a Sand Equivalent of 20 or better and an Expansion Index of 20 or less. The “select” backfill must extend at least one-half the wall height behind the wall; otherwise, the values presented in the “Native” backfill materials columns must be used for the design. “Native” backfill materials should have an Expansion Index of 50 or less. The upper one-foot of backfill should be comprised of native on-site soils.

In addition to the above static pressures, unrestrained retaining walls located should be designed to resist seismic loading as required by the 2010 CBC. 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:

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

Where:

- Pe = Seismic thrust load
- H = Height of the wall (feet)
- γ = soil density = 125 pounds per cubic foot (pcf)
- k_h = seismic pseudostatic coefficient = 0.5 * peak horizontal ground acceleration / g

The peak horizontal ground accelerations are provided in Section 5.7.2. Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces as shown in Details RTW-A and RTW-B in Appendix E (AGS 2012). Otherwise, the retaining walls should be designed to resist hydrostatic forces. Proper drainage devices should be installed along the top of the wall backfill and 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 of the maximum dry density as determined by ASTM D1557-09. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. No backfill should be placed against concrete until minimum design strengths are achieved as verified by compression tests of cylinders. The geotechnical consultant should observe the retaining wall footings, back drain installation, and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

7.1.3. Seismic Design

In general, the site has been identified to be a "D" site class in accordance with Table 1613.5.2 of the 2010 CBC. Utilizing this information, the computer program Seismic Hazard Curves, Response Parameters and Design Parameters, v5.1.0, provided by the United States Geological Survey, and 2005 ASCE 7 criterion, the seismic design category for 0.20 second (Ss) and 1.0 second (S1) period response accelerations have been determined (2010 CBC, Section 1613.5.1) along with the design spectral response accelerations (2010 CBC, Sections 1613.5.3 and 1613.5.4). Results are presented in Table 7.1.3.

TABLE 7.1.3
SEISMIC DESIGN PARAMETERS

Site Class	**SD_S (g)**	**SD_I (g)**	**SM_S (g)**	**SM_I (g)**
A (Hard Rock)	0.744	0.286	1.117	0.429
B (Rock)	0.930	0.358	1.396	0.536
D (Artificial Fill)	0.930	0.536	1.396	0.804

7.1.4. Pavement Design

Final pavement design should be made based upon sampling and testing of post-grading conditions. For preliminary design and estimating purposes the pavement structural sections presented in Table 7.2.3 can be used for the range of likely traffic indices. The structural sections are based upon an assumed R - Value of 40.

TABLE 7.2.3
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	3	4
6.0	3	5
7.0	4	8
8.0	4	9.5

The upper one (1) foot of pavement subgrade soils should be at or near optimum moisture content and should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557-09. Aggregate base should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557-09 and should conform with the specifications listed in Section 26 of the *Standard Specifications for the State of California Department of Transportation* (Caltrans) or Section 200-2 of the *Standard Specifications for Public Works Construction* (Green Book). The asphalt concrete should conform to Section 26 of the Caltrans *Standard Specifications* or Section 203-6 of the Green Book.

8.0 FUTURE STUDY NEEDS

This report represents a supplemental EIR level Tentative Tract Map review of the offsite sewer and roadway improvements associated with Lilac Hills Ranch Community project. As the project design progresses, additional site specific geologic and geotechnical issues will need to be considered in the ultimate design and construction of these improvements. Consequently, future geotechnical reviews are necessary. These reviews may include reviews of:

- Rough grading plans.
- Improvement plans.
- Retaining wall plans.

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

9.0 CLOSURE

9.1. 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.

AGS should review the pertinent plans and sections of the project specifications, to evaluate conformance with the intent of the recommendations contained in this report.

If the project description or final design varies from that described in this report, AGS must be consulted regarding the applicability of, and the necessity for, any revisions to the recommendations presented herein. AGS accepts no liability for any use of its recommendations if the project description or final design varies and AGS is not consulted regarding the changes.

9.2. Limitations

This report is based on the project as described and the information obtained from referenced reports and the borings at the locations indicated on the plan. The findings are based on the review of the field and laboratory data combined with an interpolation and extrapolation of conditions between and beyond the exploratory excavations. 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.

LEGEND

af	ARTIFICIAL FILL
Qal	ALLUVIUM
Qoal	OLDER ALLUVIUM
Kgr	GRANITIC ROCK
— — —	APPROXIMATE LOCATION OF GEOLOGIC CONTACT (DOTTED WHERE BURRIED, QUERIED WHERE UNKNOWN)
— — — — —	APPROXIMATE PROJECT BOUNDARY
X X X X X	GRANITIC ROCK OUTCROPS
N.A.P.	NOT A PART

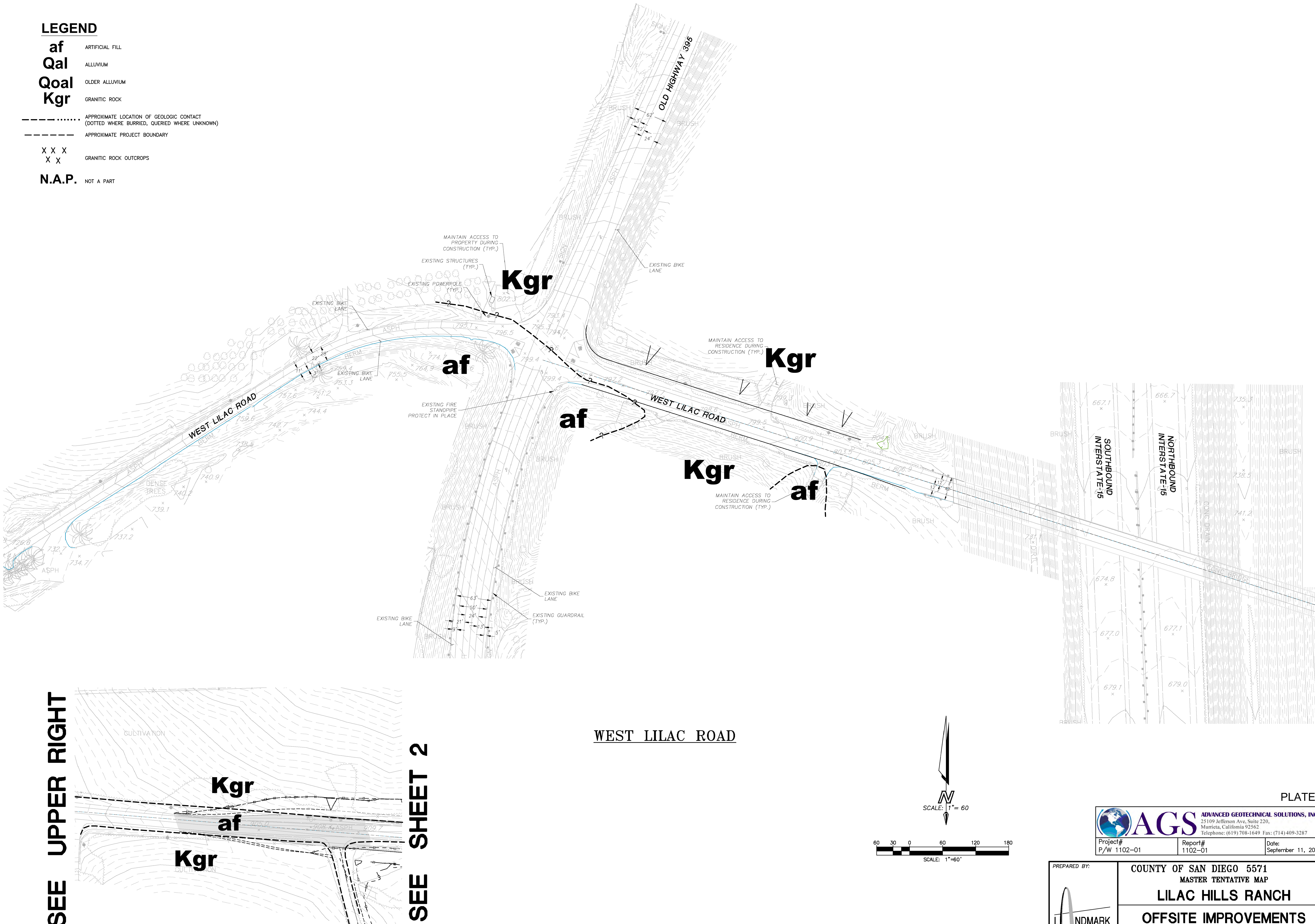


PLATE 1

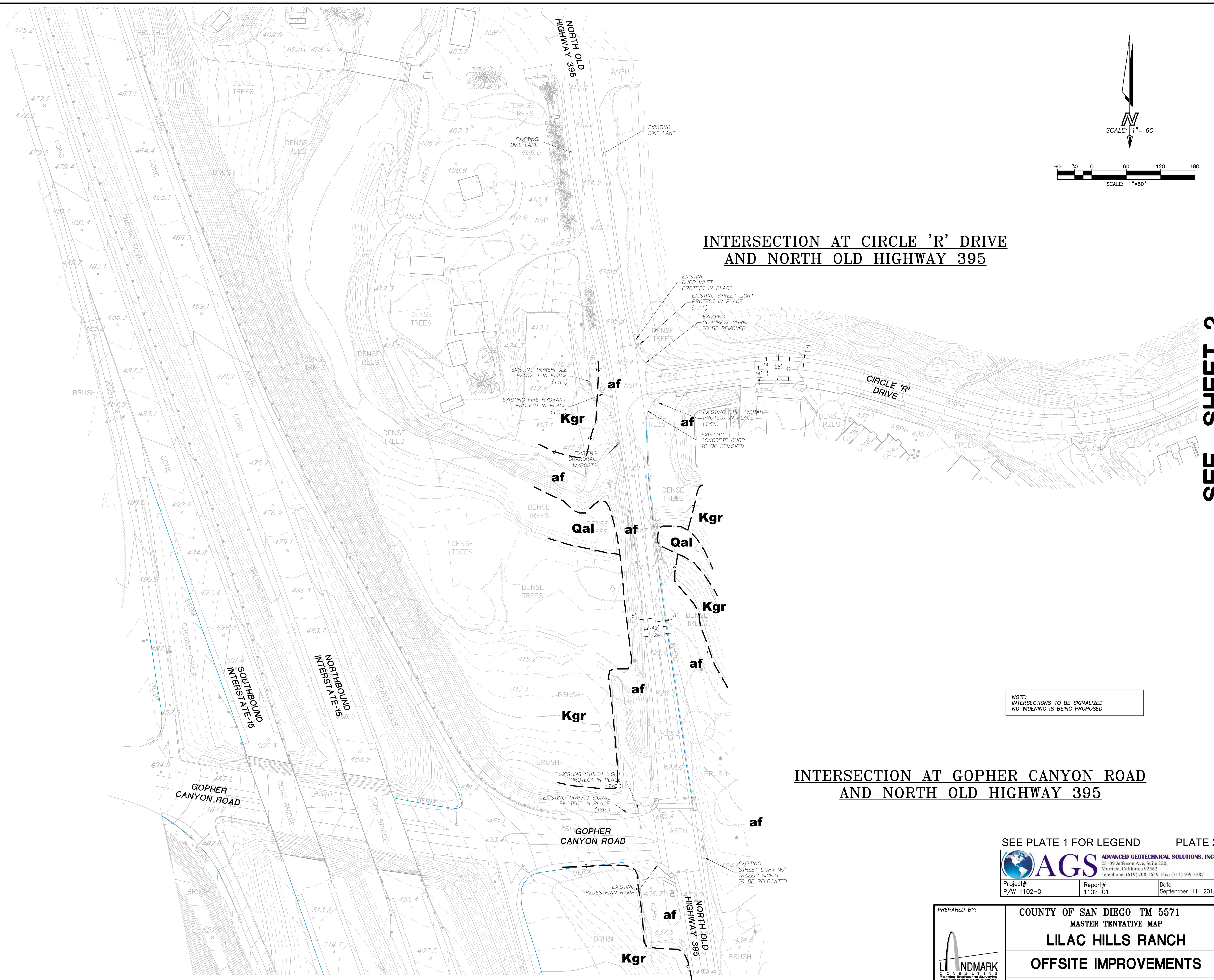
AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220,
Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#	Report#	Date:
P/W 1102-01	1102-01	September 11, 2012

PREPARED BY:
LANDMARK CONSULTING

COUNTY OF SAN DIEGO 5571
MASTER TENTATIVE MAP
LILAC HILLS RANCH
OFFSITE IMPROVEMENTS
SHEET NO. 6 OF 8

DRAFT



SEE SHEET 2

SEE PLATE 1 FOR LEGEND

PLATE 2

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220, Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#	Report#	Date:
P/W 1102-01	1102-01	September 11, 2012

PREPARED BY:

LANDMARK CONSULTING

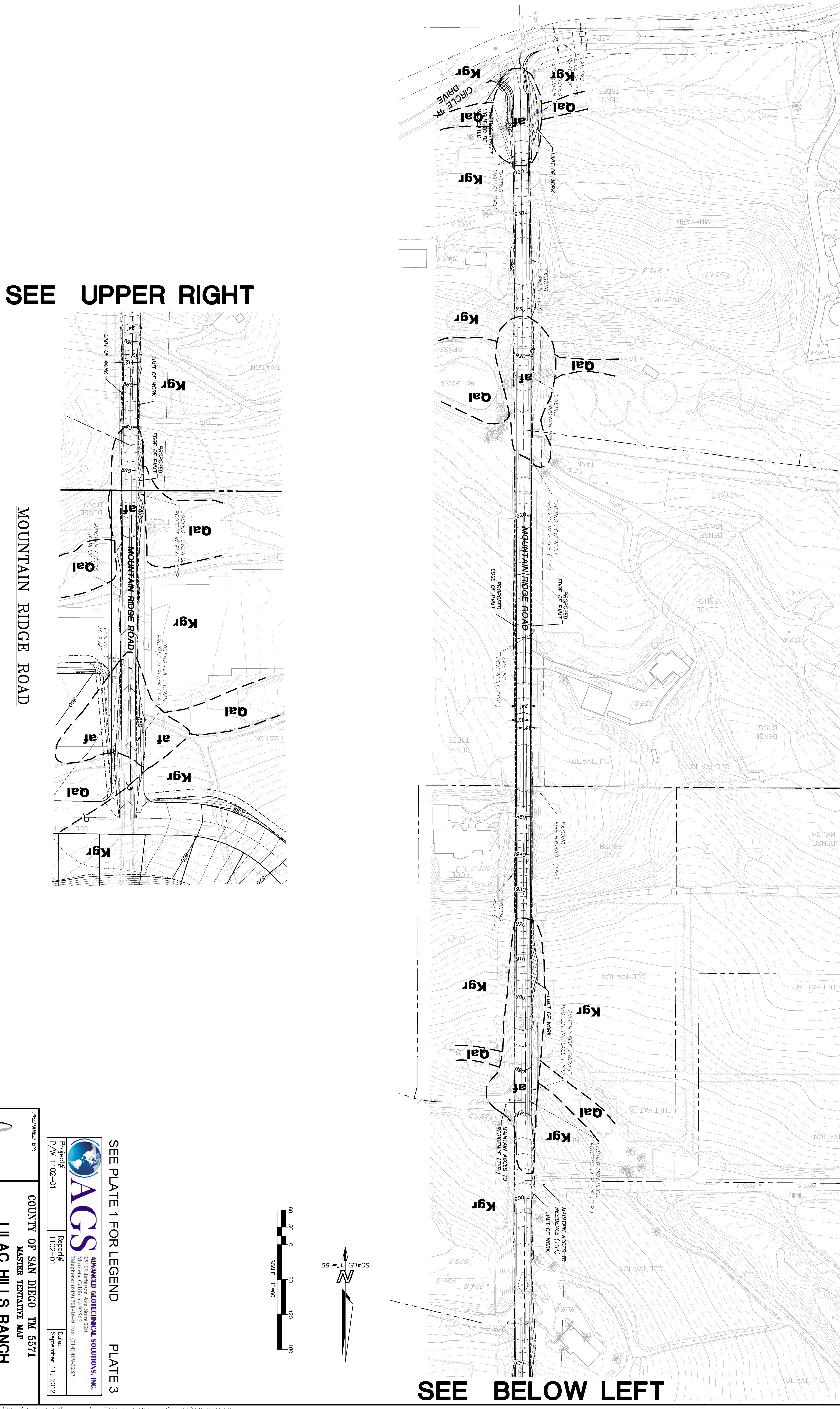
COUNTY OF SAN DIEGO TM 5571
MASTER TENTATIVE MAP

LILAC HILLS RANCH

OFFSITE IMPROVEMENTS

SHEET NO. 7 OF 8

DRAFT



SEE PLATE 1 FOR LEGEND

PLATE 3

ADVANCED GEOTECHNICAL SOLUTIONS, INC.
25109 Jefferson Ave, Suite 220,
Murrieta, California 92562
Telephone: (619) 708-1649 Fax: (714) 409-3287

Project#	Report#	Date:
P/W 1102-01	1102-01	September 11, 2012

PREPARED BY:

LANDMARK CONSULTING
Planning Engineering Surveying
9555 Genesee Avenue, Suite 200
San Diego, CA 92121, (858) 587-8070

COUNTY OF SAN DIEGO TM 5571
MASTER TENTATIVE MAP

LILAC HILLS RANCH

OFFSITE IMPROVEMENTS

SHEET NO. 8 OF 8

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APPENDIX A

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