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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 AGS accepts the content, conclusions and recommendations presented in the referenced PSE documents except where superseded herein.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.

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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. <u>Previous Geotechnical Investigations</u>

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. <u>Current Investigation</u>

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. <u>Geologic Analysis</u>

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. 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). 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. <u>Analytical Methods</u>

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. <u>Slope Stability and Remediation</u>

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.

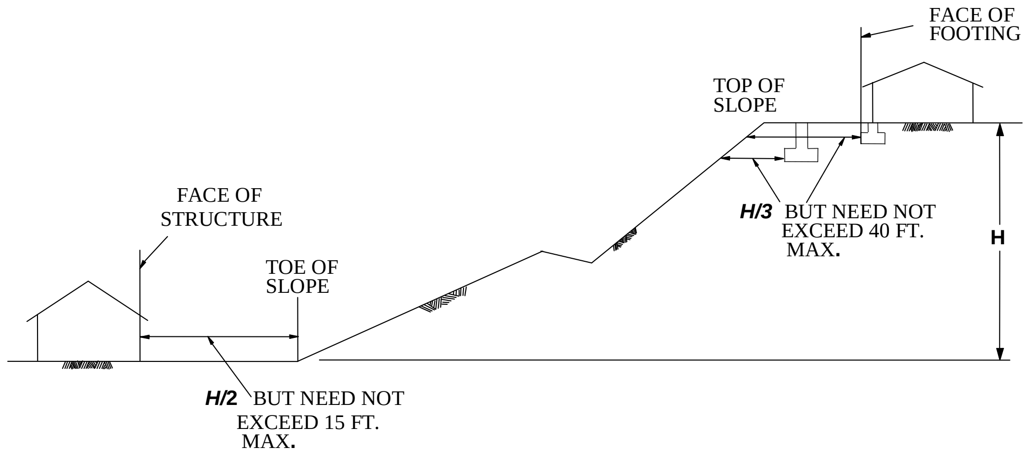
7.1.1.2. Deepened Footings and Setbacks

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. <u>Civil Design Recommendations</u>

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.

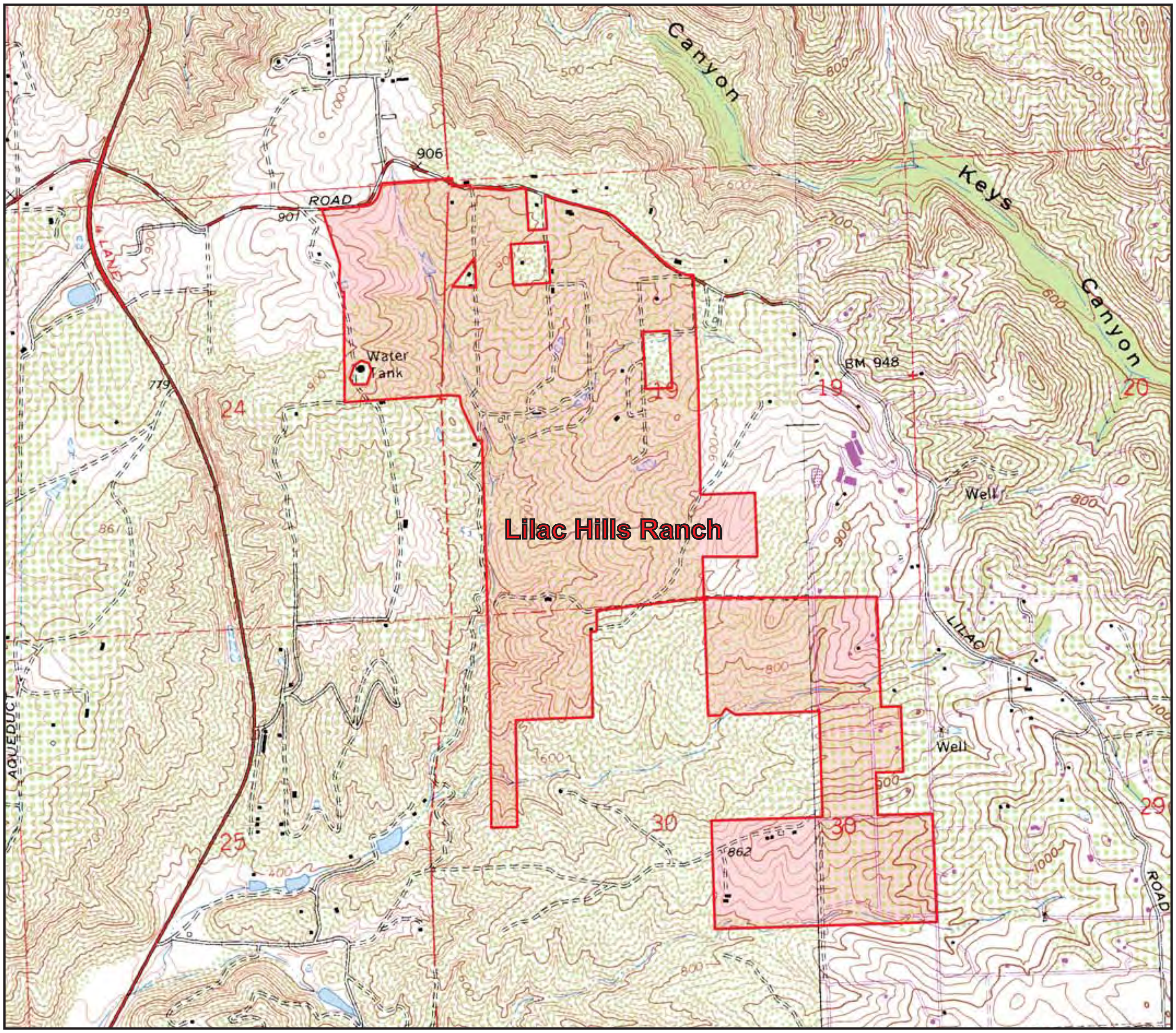
9.2. Limitations

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.

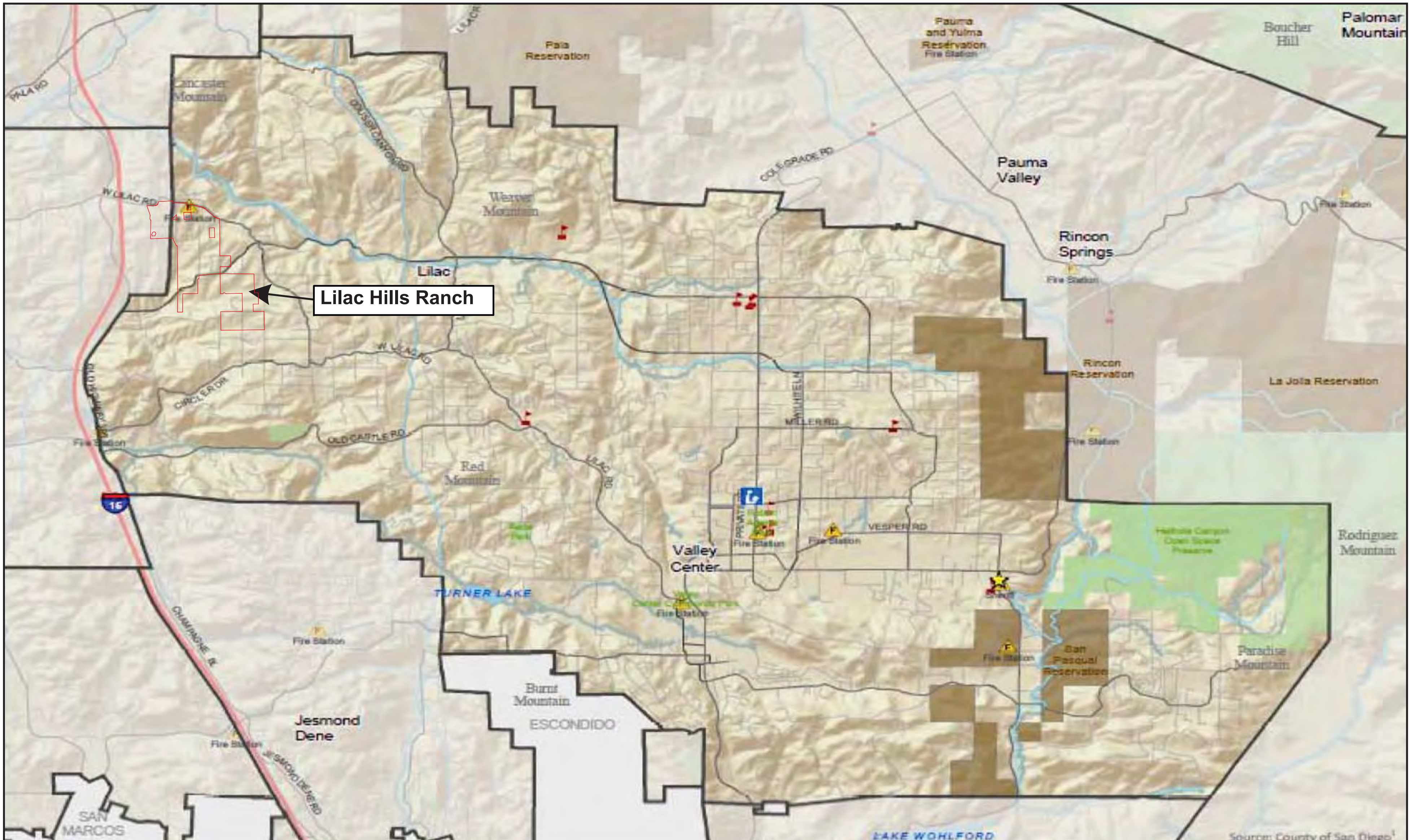
U.S.G.S. SITE LOCATION MAP
LILAC HILLS RANCH DEVELOPMENT
NORTHWEST CORNER VALLEY CENTER
COUNTY OF SAN DIEGO, CALIFORNIA

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**



VALLEY CENTER CONTEXT MAP

San Diego County General Plan

Figure 2

N

Valley Center Context Map

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

FIGURE 2

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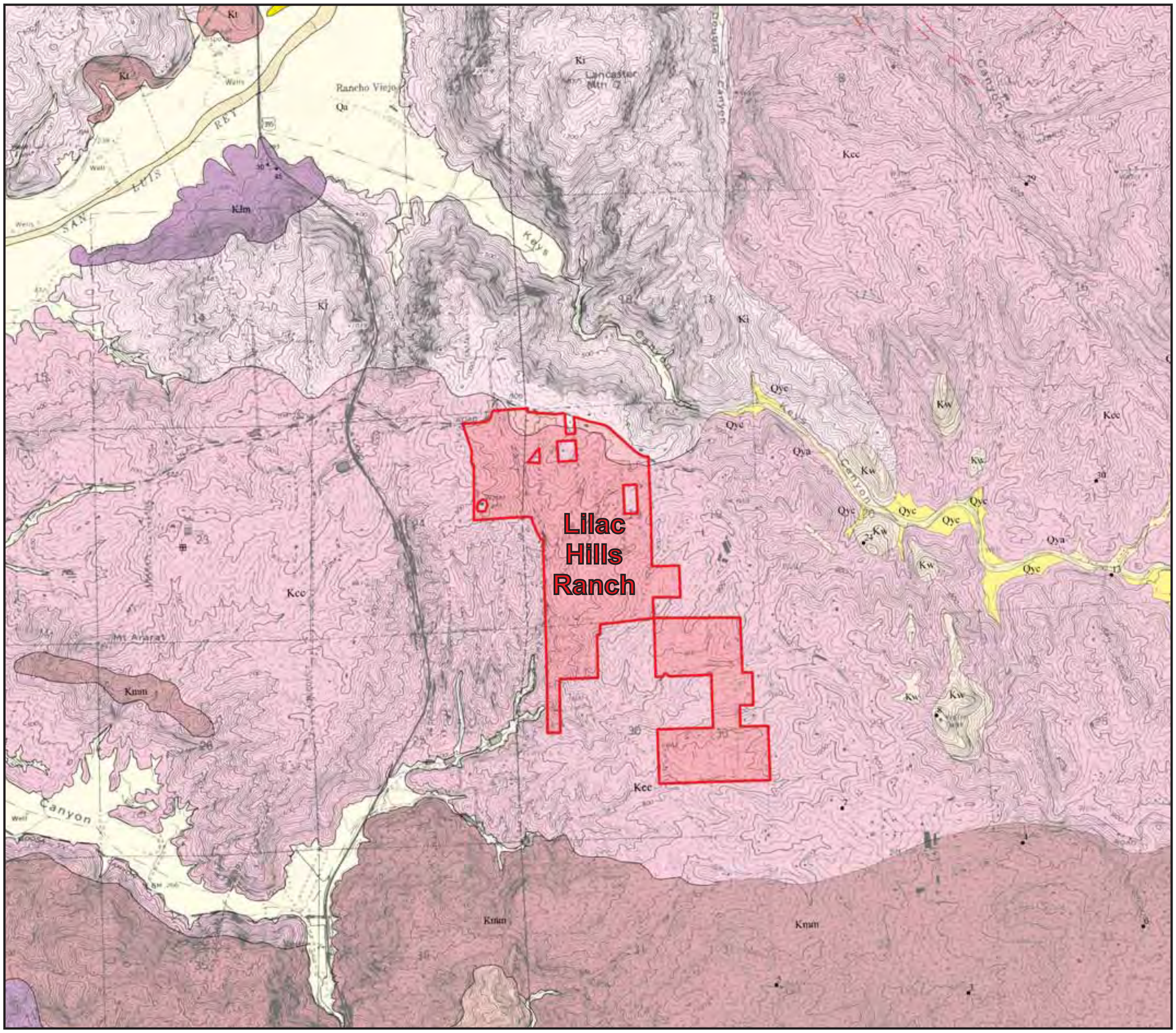
P/W 1102-01 **Report No. 1102-01-B-9**



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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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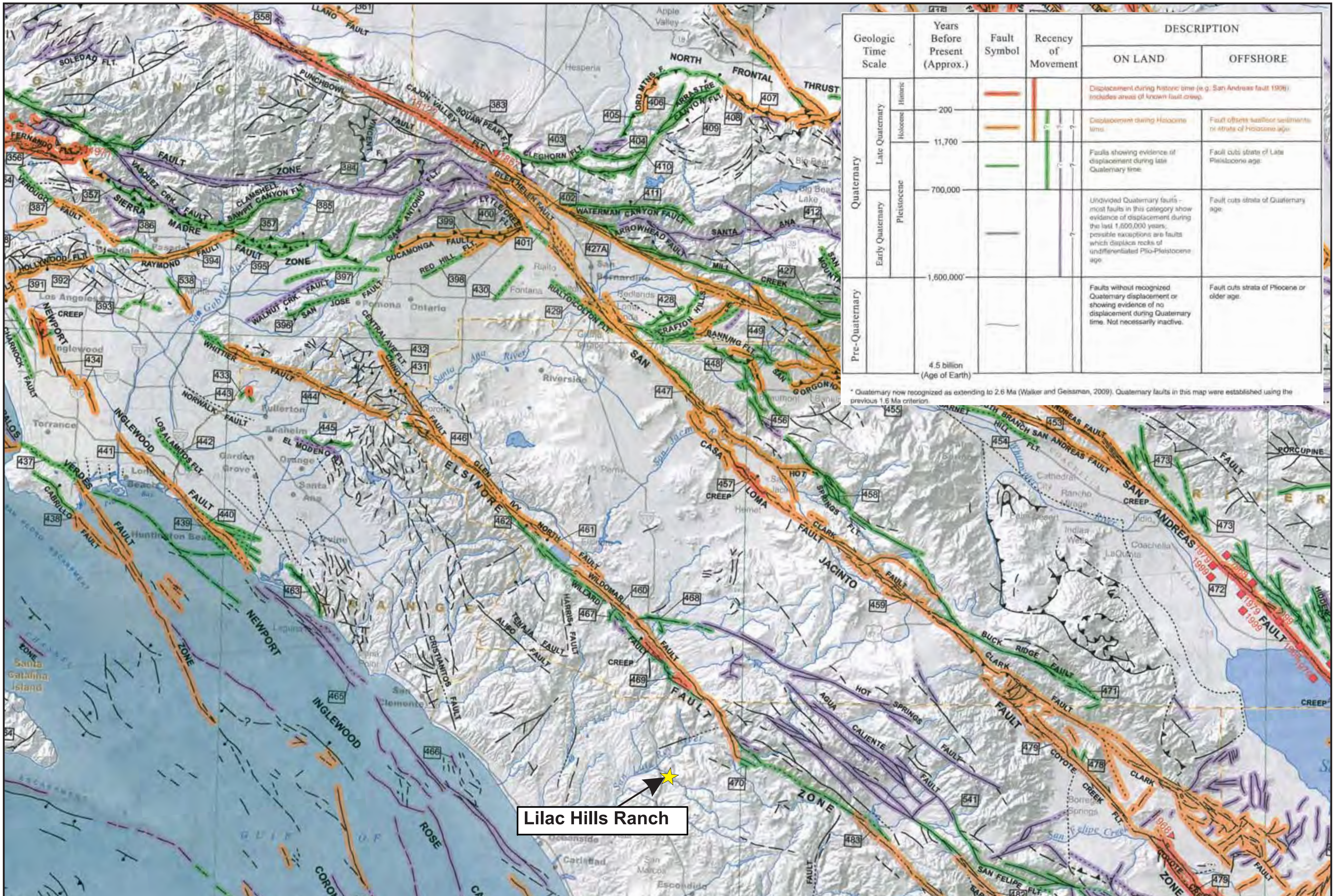

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			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.
Pre-Quaternary			1,600,000*			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

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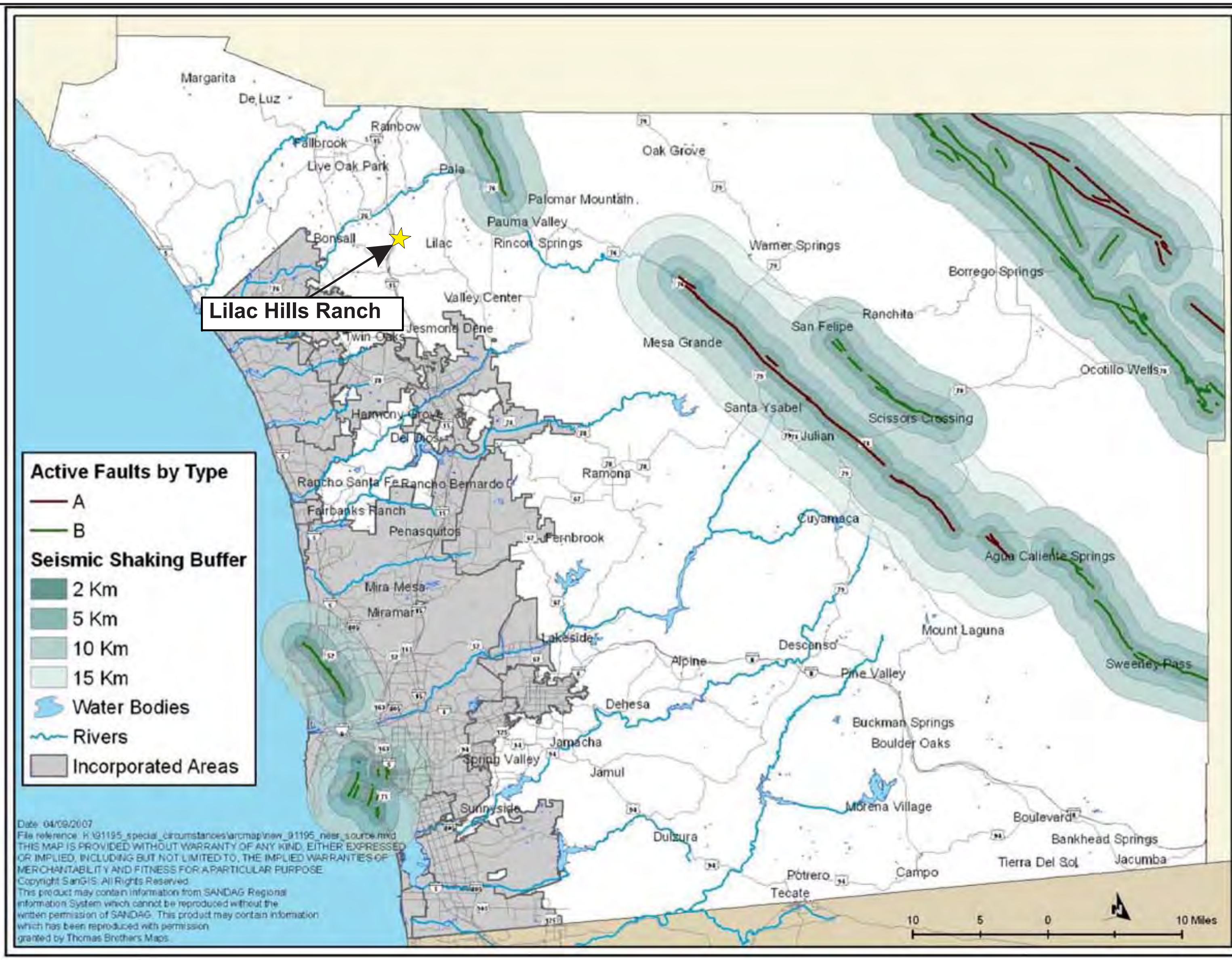

Near Source Shaking Zones

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

FIGURE 5

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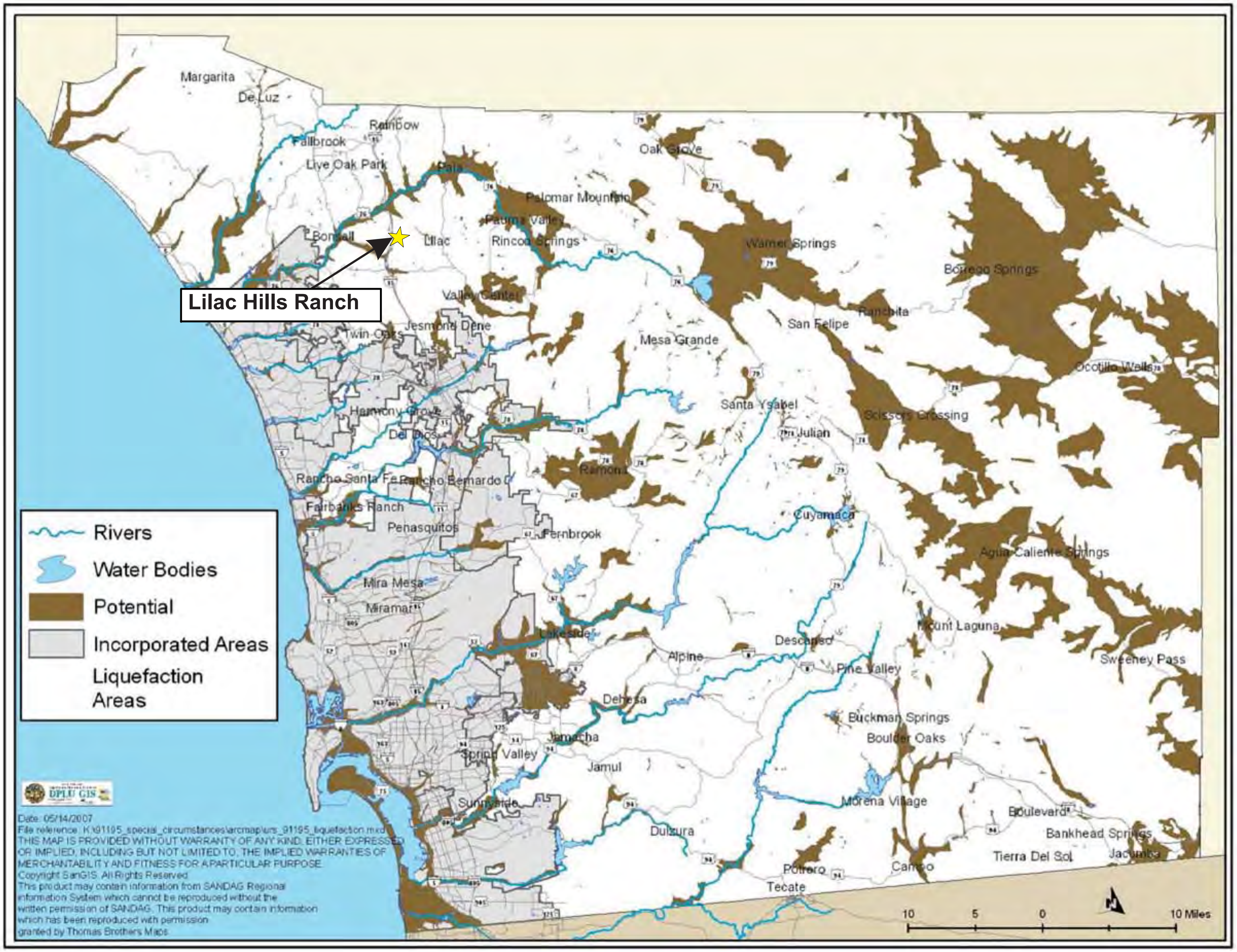

Potential Liquefaction Areas

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

FIGURE 6

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N

SCALE: 1"= 200

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.
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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

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SEE SHEET 1 FOR LEGEND
SHEET 2

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

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

COUNTY OF SAN DIEGO
TENTATIVE MAP

LILAC HILLS RANCH
MASTER TENTATIVE MAP