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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.

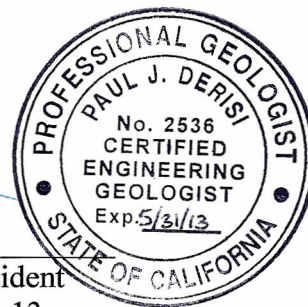
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PLATES 1 THRU 3- GEOLOGIC MAPS (OFFSITE IMPROVEMENTS TENTATIVE TRACT MAP)

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 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 larger 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 (k_h) 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 extent 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				
<u>“Native”* Backfill Materials</u> ($\gamma=125\text{pcf}$, $EI\leq 50$)				
	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
<u>“Select”* Backfill Materials</u> ($\gamma=120\text{pcf}$, $EI\leq 20$, $SE\geq 20$)				
	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:

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

Where:

P_e = Seismic thrust load

H = Height of the wall (feet)

γ = soil density = 125 pounds per cubic foot (pcf)

k_h = seismic pseudostatic coefficient = $0.5 * \text{peak horizontal ground acceleration} / g$

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 (S_s) and 1.0 second (S_1) 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_1 (g)	SM_s (g)	SM_1 (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

ALUMINUM

Qoal

OLDER ALUMINUM

Kgr

GRANITIC ROCK

.....

APPROXIMATE LOCATION OF GEOLOGIC CONTACT
(DOTTED WHERE BURIED, QUEERED WHERE UNKNOWN)

APPROXIMATE PROJECT BOUNDARY

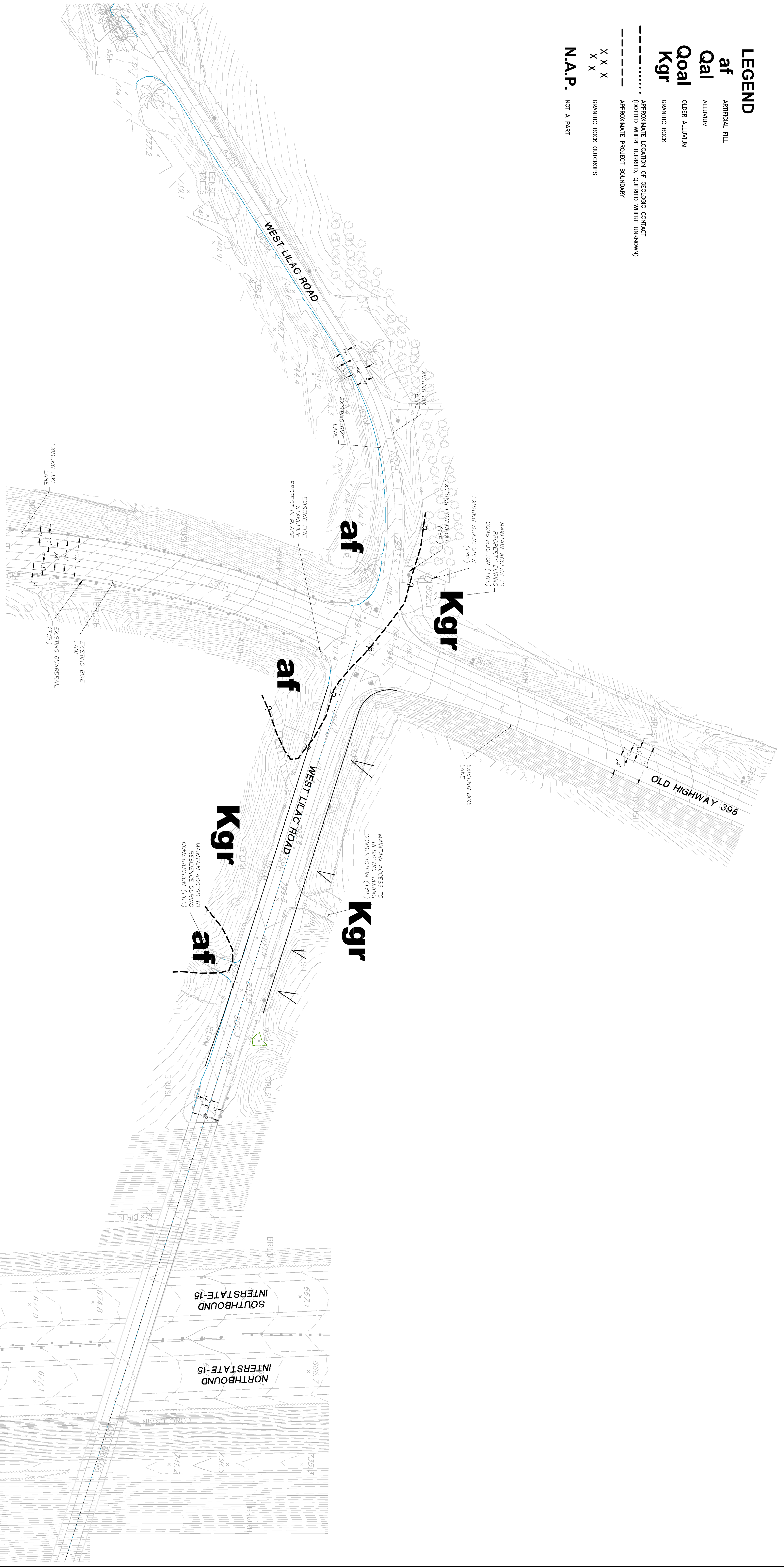
X X X

GRANITIC ROCK OUTCROPS

X X

N.A.P.

NOT A PART



SEE UPPER RIGHT

SEE SHEET 2

WEST LILAC ROAD

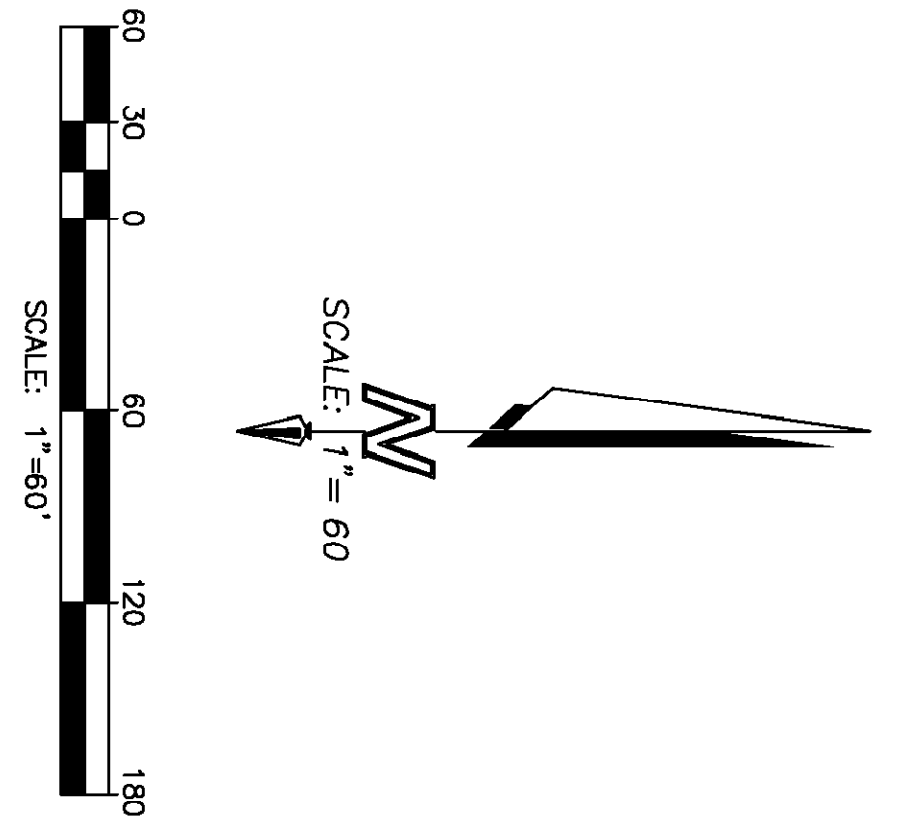
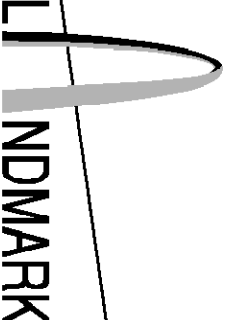


PLATE 1

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COUNTY OF SAN DIEGO 5571

MASTER TENTATIVE MAP

LILAC HILLS RANCH

OFFSITE IMPROVEMENTS

SHEET NO. 8 OF 8

APPENDIX A

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