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December 24, 2019
P/W 1312-02
Report No. 1312-02-B-10

Attention: **Ms. Liz Jackson**

Subject: ***Geotechnical Review of Proposed Project Amendment, Otay Ranch Village 14 and Planning Areas 16 and 19, County of San Diego, California***

References: See Appendix A

Gentleperson:

Pursuant to your request, presented herein are the results of Advanced Geotechnical Solutions, Inc.'s, (AGS) Geotechnical Review of the Proposed Project Amendment for Otay Ranch Village 14 and Planning Areas 16 and 19, County of San Diego, California. AGS has been retained to complete the geotechnical services supporting the Revised Tentative Map approval process for this project.

As part of our services, AGS has conducted field mapping, subsurface exploration, laboratory testing, geotechnical engineering and geologic analysis related to the Proposed Project Amendment's Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16 and 19.

The purpose of this geotechnical review is to evaluate the proposed Revised Tentative Map/Preliminary Grading Plans relative to the near-Project Area and on-Project Area geologic and geotechnical conditions and provide conclusions and recommendations to aid in the development of the Proposed Project Amendment. The Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16 and 19 prepared by Hunsaker & Associates was 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) 867-0487.

Respectfully Submitted,
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Geotechnical Review of Proposed Project Amendment for Otay Ranch Village 14 and Planning Areas 16 and 19, County of San Diego, California

1.0 INTRODUCTION

This technical report addresses proposed changes to the Otay Ranch Village 14 and Planning Areas 16 and 19 Project that was approved by the San Diego County Board of Supervisors on June 26, 2019 (Approved Project). The changes to the Approved Project would reconfigure the development footprint to consolidate development in Village 14; add 147 units, for a total of 1,266 residential units; and reduce impacts by approximately 230 acres, to 579 acres (the Proposed Project Amendment). Figure 1, Regional Location Map, shows the regional location of the Approved Project Area and the Proposed Project Amendment Project Area, in the County of San Diego (County).

An Environmental Impact Report (EIR) was prepared for the Approved Project and was certified by the Board on June 26, 2019. The Final EIR analyzed the potential impacts of the Approved Project. The Final EIR also analyzed potential impacts associated with the EIR Land Exchange Alternative, which was one of the alternatives to the Approved Project. The Final EIR is incorporated herein by reference and the results are summarized in Section 3 and mitigation measures are described in Section 5.

The Final EIR analyzed the Approved Project's potential impacts related to potential geologic and geotechnical hazards in Section 2.6 Geology, Soils, and Seismology. This technical report was prepared for the Proposed Project Amendment. The purpose of this analysis is to evaluate whether, and to what extent, the potential impacts of the Proposed Project Amendment to geologic and geotechnical hazards differ from those of the Approved Project and the EIR Land Exchange Alternative.

For additional context, the following terminology is used in this technical report.

Approved Project: The project described in Otay Ranch Village 14 and Planning Areas 16/19 Tentative Map 5616, Specific Plan 16-002, and the certified EIR, which the County of San Diego (County) approved on June 26, 2019. The Approved Project permits 1,119 residential units within a Project Area of approximately 1,369 acres. The Development Footprint of the Approved Project is 809 acres.

Conserved Open Space: Conserved Open Space refers to 24.5 acres of land within the Project Area, which, while designated in the Otay Ranch General Development Plan/Otay Subregional Plan (Otay Ranch GDP/SRP) for residential uses within Village 14 and Planning Areas 16/19, will not be developed as part of the Proposed Project Amendment. Instead, the Conserved Open Space will be preserved on site and be (a) added to the Otay Ranch Resource Management Plan (RMP) Preserve (through a future RMP Amendment), (b) managed under a separate RMP, or (c) utilized to mitigate impacts to the City of San Diego Multiple Species Conservation Program Cornerstone Lands. The Conserved Open Space areas are located adjacent to Otay Ranch RMP Preserve and will be conserved by recording a biological open space easement over the land.

Development Footprint: The areas where a given project will cause permanent or temporary ground disturbance. The Development Footprint includes all on-site development, off-site improvements, and impacts resulting from infrastructure and other allowable uses within the Otay Ranch Resource Management Plan (RMP) Preserve.

EIR Land Exchange Alternative: The project alternative identified as the "Land Exchange Alternative" in Chapter 4 of the certified Final EIR. This Land Exchange Alternative contemplated a land exchange with the California Department of Fish and Wildlife (CDFW) and would develop 1,530 residential units within a Project Area of approximately 2,387.6 acres, with a Development Footprint of 658.3 acres.

Off-Site Improvements: Off-site improvements total approximately 40.1 acres and include Proctor Valley Road, wet and dry utilities, drainage facilities, trails, an off-site sewer pump station in the southern reach of Proctor Valley Road, and off-site sewer facilities to connect to the Salt Creek Interceptor as planned since 1994.

Project Area: The total land area for the Proposed Project Amendment as contemplated in the proposed land exchange between applicant and CDFW.[1] The Project Area consists of approximately 1,283.6 acres currently owned by GDCI Proctor Valley, L.P., the owner/applicant, 219.4 acres currently owned by CDFW, and approximately 40.1 acres of off-site improvements, for a total of 1,543 acres.

Proposed Project Amendment: The Proposed Project Amendment reflects proposed changes to the Approved Project, which would add 147 units for a total of 1,266 residential units and would reduce the Development Footprint by approximately 230 acres, to a total of 579 acres, within a Project Area of 1,543 acres, as shown on Figure 2 and more fully described below in Section 2. The Proposed Project Amendment includes a Revised Tentative Map, and Specific Plan. As contemplated in the Dispute Resolution Agreement, the Proposed Project Amendment assumes and will require a County-initiated amendment to the MSCP County Subarea Plan. Because the amendment to the MSCP County Subarea Plan will be initiated by the County, it is not part of the Proposed Project Amendment.

PV1 and PV3: PV1 and PV3 are areas of the Approved Project located in Village 14 as shown on Figure 3 attached.

Revised Tentative Map: The Revised Tentative Map reflects revisions to the June 26, 2019, Approved Tentative Map #5616 that are necessary to process and implement the land exchange with CDFW and the Proposed Project Amendment in the County.

Specific Plan Amendment: The Specific Plan Amendment reflects revisions to the June 26, 2019, Approved Specific Plan #16-002 that are necessary to process and implement the land exchange with CDFW and the Proposed Project Amendment in the County.

1.1. Background

The Proposed Project Amendment Project Area comprises approximately 1,543 acres of undeveloped land within the Proctor Valley area of the 23,000-acre Otay Ranch master planned community (see Figure 4, Proposed Project Amendment Project Area). The Proposed Project Amendment reflects proposed changes to the Approved Project, including a proposed land exchange with the California Department of Fish and Wildlife (CDFW).

On June 27, 2019, the owner/applicant of the Approved Project entered into a Dispute Resolution Agreement with CDFW, the U.S. Fish and Wildlife Service, and the County. Pursuant to this agreement, the owner/applicant would seek a land exchange with CDFW through a process

[1] As indicated above, the land exchange necessary to implement the Proposed Project Amendment must be approved by the California Wildlife Conservation Board.

overseen by the California Wildlife Conservation Board. The proposed land exchange, if approved by the Wildlife Conservation Board, would require the owner/applicant to (i) transfer 147.3 acres in Village 14 and 192.4 acres in Planning Area 16 to CDFW, and (ii) record a conservation easement over 191.5 acres in Planning Area 16. In exchange, CDFW would transfer 219.4 acres in Village 14 to the owner/applicant. The Proposed Project Amendment would then be implemented upon the lands within the applicant's ownership, including those received via the Wildlife Conservation Board land exchange. Because the Proposed Project Amendment assumes the above-described land exchange, it would result in a different development footprint than the Approved Project's development footprint. Therefore, a Specific Plan Amendment to the approved Village 14 and Planning Areas 16/19 Specific Plan and a Revised Tentative Map are required processes for the Proposed Project Amendment.

While the Proposed Project Amendment and EIR Land Exchange Alternative both contemplate exchanges of land with the CDFW, the Development Footprints and other aspects differ. It is important to note that the Development Footprint of the Proposed Project Amendment was assessed in the certified Final EIR as part of the Approved Project Development Footprint and as part of the EIR Land Exchange Alternative Development Footprint.

1.2. Scope of Study

This report has been prepared to address the conceptual design for the Proposed Project Amendment as shown on the Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16/19 prepared by Hunsaker & Associates in a manner consistent with County of San Diego geotechnical report guidelines and current standard of practice. This study provides geotechnical/geologic conclusions and recommendations for development of the Proposed Project Amendment for residential and commercial uses, attendant streets, parks, schools, community facilities, water storage distribution facilities, and open space areas. Geotechnical conclusions and recommendations are presented herein, and the items addressed include, without limitation: 1) unsuitable soil removals and remedial grading; 2) cut, fill and natural slope stability; 3) potential geologic hazards and general mitigation measures for these potential hazards; 4) buttress/stabilization fill criteria; 5) cut/fill pad overexcavation criteria; 6) remedial and design grading recommendations; 7) rippability of the onsite bedrock; 8) disposal of oversize hard earth materials; and 9) general foundation design recommendations based upon anticipated as-graded soil conditions.

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).
- Perform geologic field mapping within the Proposed Project Amendment boundaries.
- Transfer selected geologic and geotechnical information generated from this investigation onto the 100-scale Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16/19 prepared by Hunsaker & Associates, included as Plates 1 through 11 (attached). These plans depict existing grades and proposed rough grading. AGS added geologic and geotechnical information to the plans, including: the approximate limits of

surficial geologic units; locations of seismic refraction lines, soil borings, and test pits (backhoe and excavator) with abbreviated logs.

- Excavate, sample, and log seventeen (17) backhoe test pits (TP-1 through TP-17), with a Caterpillar 416F and 430F (Appendix B) and fifteen (15) backhoe test pits (T-1 through T-15), with a Case 580L.
- Excavate, sample, and log six (6) excavator test pits (EX-1A through EX-6A), with a Caterpillar 328D (Appendix B).
- Excavate, sample, and log twenty-three (23) excavator test pits (EX-1 through EX-23), with a Caterpillar 349E (Appendix B).
- Excavate, sample, and log nine (9) excavator test pits (ST-1 through ST-9), with a Hitachi 350 (Appendix B).
- Excavate, sample, and log eight (8) 30-inch diameter borings (BA-1 through BA-8), with a truck mounted drill rig (Appendix B).
- Excavate, sample, and log four (4) 24-inch diameter borings (LD-1 through LD-4), with a tracked excavator mounted LoDril drill rig (Appendix B).
- Excavate and log eight (8) percolation test borings (P-1 through P-8), and eight (8) associated exploratory trenches (PT-1 through PT-8), with a Caterpillar 420F backhoe (Appendix E).
- Conduct preliminary percolation/infiltration testing in general conformance with the County of San Diego's Best Management Practices Design Manual.
- Perform thirteen (13) seismic refraction traverses (SL-1 through SL-13) and development of two-dimensional tomographic models (Appendix B).
- Laboratory testing of representative "undisturbed" and bulk samples obtained during the various phases of this study (Appendix C).
- Conduct a geotechnical engineering and geologic hazard analysis of the Project Area.
- Conduct a limited seismicity analysis.
- Define remedial grading requirements.
- Evaluate the stability of the highest cut, fill, and natural slopes within the limits of the proposed development (Appendix D).
- Data analyses in relation to the proposed improvements within the Proposed Project Amendment Area.
- Preliminary analysis of the excavation characteristics (i.e. rippability) of onsite bedrock materials.
- Limited evaluation of liquefaction potential based upon field investigation and familiarity with the existing soils conditions.
- Discussion of pertinent geologic and geotechnical topics.

- Prepare general foundation design parameters which can be used for preliminary design.
- Prepare this geotechnical review report of the current grading plans and tentative tract map with the associated exhibits summarizing our findings. This report is suitable for preliminary design 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 investigation. The conclusions presented herein are based upon the current design as reflected on the included Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16/19. 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 materials is beyond the scope of this firm's services.

2.0 PROJECT AREA LOCATION AND DESCRIPTION

2.1. Project Area Location and Description

The Proposed Project Amendment is located within Township 17 South, Range 1 East on the USGS 7.5' Jamul Mountains quadrangle, generally along Proctor Valley Road, between the City of Chula Vista and Jamul, California. The Proposed Project Amendment area is more specifically located within Otay Ranch Village 14 and Planning Area 19 as depicted in Figure 1 (Project Area).

The Project Area is in a natural state and is covered with a light to dense growth of annuals and some chaparral. A network of improved and unimproved roads provides access throughout the Project Area. The existing elevations within the proposed development ranges from a high of approximately 1,070 feet above mean sea level (AMSL) southeasterly superjacent to the water tank to a low of approximately 550 feet AMSL within an active drainage near the southern limit of the proposed development. Topography within the Project Area ranges from gently sloping terraces to moderately steep existing natural slopes approaching 1:1 (horizontal to vertical) slope inclinations. Two southerly flowing active drainages transect the Project Area ultimately converging into a broad drainage adjacent to the existing Proctor Valley Road which drains into Upper Otay Reservoir (Figure 2).

2.2. Proposed Project Amendment Description and Summary of Amendments/Revisions

The Proposed Project Amendment proposes 1,266 residential units within a Project Area of 1,543 acres. The Proposed Project Amendment Development Footprint would be approximately 578.6 acres, which would consistent of approximately 513.1 acres within Otay Ranch Village 14, 25.2 acres within Otay Ranch Planning Area 19, and 40.1 acres of off-site improvements (i.e., Proctor Valley Road). Of the 1,266 residential units, 1,253 units would be located in Village 14 (consistent with the Otay Ranch GDP/SRP) and 13 units would be located in Planning Area 19 (consistent with the Otay Ranch GDP/SRP). The Proposed Project Amendment is depicted in Figure 3.

As described above, the Proposed Project Amendment would also include a land exchange between the owner/applicant and CDFW, which would require the owner/applicant to transfer 339.7 acres to CDFW and record a conservation easement over 191.5 acres, and, in exchange, CDFW would transfer 219.4 acres in Village 14 to the owner/applicant to create a consolidated Development Footprint. The proposed land exchange between the applicant and CDFW is depicted in Figure 4. As defined above, the Proposed Project Amendment requires a Specific Plan Amendment and Revised Tentative Map, which must be approved by the County. The Revised Tentative Map would replace that certain approved Tentative Map TM #5616, approved by the County on June 26, 2019. The Specific Plan Amendment would amend the Specific Plan 16-002 to reflect the Proposed Project Amendment, including the Revised Tentative Map and the land exchange with CDFW.

Additional improvements include several water quality retention/infiltration basins; water and sewer pump stations; a water tank; and associated roadways and utilities. It is anticipated that conventional cut and fill grading techniques will be utilized to develop the Proposed Project Amendment. Current plans prepared by Hunsaker and Associates show maximum cuts and fill on the order of 80 to 100 feet, with proposed cut and fill slope ratios on the order of 2:1.

Offsite Improvements include: Proctor Valley Road; wet and dry utilities; drainage facilities; trails; an off-site sewer pump station in the southern reach of Proctor Valley Road; and off-site sewer facilities to connect to the Salt Creek Interceptor as planned since 1994. As part of the proposed roadway improvements, culverts and a bridge structure will likely be needed to cross existing active drainages.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1. Field Investigation

For this and previous studies, AGS performed geologic mapping and conducted subsurface exploration and laboratory testing. Our scope of work included the following:

- Seventeen (17) backhoe test pits (TP-1 through TP-17), with a Caterpillar 416F and 430F, and fifteen (15) backhoe test pits (T-1 through T-15), with a Case 580L.
- Six (6) excavator test pits (EX-1A through EX-6A), with a Caterpillar 328D.
- Eight (8) 30-inch diameter bucket auger borings (BA-1 through BA-8).
- Twenty-three (23) excavator test pits (EX-1 through EX-23) with a Caterpillar 349E.
- Eight (8) percolation boreholes (P-1 through P-8), and eight (8) associated backhoe test pits (PT-1 through PT-8).
- Thirteen (13) seismic refraction traverses (SL-1 through SL-13) and associated tomographic modeling.
- Four (4) 24-inch diameter flight auger borings (LD-1 through LD-4).
- Nine (9) excavator test pits (ST-1 through ST-9) with a Hitachi 350.

The data for the field investigation is presented in Appendix B. Selected bulk samples and ring samples obtained during our field investigation were transported to our approved laboratory for testing and analysis; results of that testing are presented in Appendix C.

As part of our services, AGS integrated pertinent information from our field mapping and subsurface exploration on the 100-scale Revised Preliminary Grading Plan for Otay Ranch Village 14 and Planning Areas 16/19 prepared by Hunsaker & Associates (Plates 1 through 11) and prepared this report with our findings and recommendations.

4.0 ENGINEERING GEOLOGY

4.1. Geologic Analysis

4.1.1. Literature Review

AGS reviewed the referenced geologic documents in preparing this study, and where appropriate, that information was included in this document. Of particular use, are the maps by Tan (1992 and 2002) and Todd (2004).

4.1.2. Aerial Photograph Review

AGS reviewed historic aerial photographs and satellite imagery during this investigation. The photographs AGS reviewed are presented in the References section. Notable features observed include possible landslides and lineaments. These features are discussed in further detail in the following sections.

4.1.3. Field Mapping

The geologic contacts mapped on the TM are based on our observations of the Project Area and subsurface data collected from our test pits and soil borings.

4.2. Geologic and Geomorphic Setting

The Proposed Project Amendment is located in the lower Peninsular Range Region of San Diego County, a subset of the greater Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges Geomorphic province is approximately bounded to the east by Elsinore Fault Zone, to the north by the Transverse Ranges, the south by Baja California, and to the west by the Pacific Ocean. This portion of the Peninsular Ranges is underlain by Jurassic and Cretaceous plutonic rocks of the Peninsular Ranges Batholith, which contains screens of variably metamorphosed Mesozoic supracrustal rocks. Late Jurassic and Early Cretaceous volcanic and volcaniclastic rocks exposed southwest of the Elsinore Fault Zone represent an older superjacent part of the Peninsular Ranges magmatic arc. These basement rocks are non-conformably overlain by a thick sequence of relatively undisturbed sedimentary rocks ranging from upper Cretaceous to Pleistocene in age.

The Project Area is located near the eastern edge of the coastal plain at the contact with the metavolcanic rocks of the Jamul Mountains. Geologically, the site is underlain by two principle rock types, the Late Jurassic to early Cretaceous aged metavolcanic rocks of the Santiago Peak

Volcanics and the Tertiary aged sedimentary rocks of the Otay Formation. The Otay Formation is informally subdivided into three subunits: an upper sandstone-claystone member; a middle gritstone member; and a basal angular-clast fanglomerate member. Minor exposures of upper Pleistocene older alluvium exist locally as relatively flat lying river terraces and unconsolidated alluvium of Holocene age occupies the active drainages onsite.

A regional geology map is shown on Figure 5.

4.3. Stratigraphy

A basement complex consisting of Mesozoic-aged prebatholithic volcanic and metavolcanic rocks underlies the Proposed Project Amendment at depth and are exposed at the surface at higher elevations in the easterly and northerly portions of the Proposed Project Amendment. The basement rocks are non-conformably overlain by Tertiary-aged sedimentary bedrock which are subsequently mantled by Quaternary-aged surficial soil units. Approximate geologic contacts are shown on Plates 1 through 11. A brief description of the earth materials encountered during our investigation is presented in the following sections. More detailed description of these materials is provided in the boring and test pit logs included in Appendix B.

4.3.1. Surficial Units

Surficial units onsite and close proximity offsite include undocumented artificial fill (afu), topsoil/colluvium (unmapped), young alluvium (map symbol Qal), older alluvium (map symbol Qoal), and landslide debris (map symbol Qls). More detailed descriptions of these units are presented below.

4.3.1.1. Artificial Fill (afu)

Although not encountered during our subsurface exploration, undocumented artificial fill soils were observed locally within the Project Area. The undocumented fills are primarily located along the current alignment of Proctor Valley Road as embankment fills for the road and associated culverts. Based on limited observed exposures, these materials can generally be described as clayey to gravelly sands with abundant rock fragments in a dry to slightly moist and loose to moderately dense condition. In addition, minor undocumented fills exist locally across the Project Area as Jeep trails. In consideration of the limited extent of the material and the plan scale, these fills are not mapped.

4.3.1.2. Topsoil/Colluvium (no map symbol)

Undifferentiated topsoil and colluvium exist throughout the Project Area as a thin soil veneer. Thicker accumulations commonly occur near the base of slopes and natural topographic swales. As encountered, these materials ranged from less than one foot to four feet in thickness and are generally composed of silty to clayey sand and sandy clay in a dry to slightly moist and loose to moderately dense condition. Roots and minor to moderate porosity are common.

4.3.1.3. Alluvium (Qal)

Young alluvial deposits occupy the bottoms of the primary and tributary drainages onsite. These materials can generally be described as silty to clayey sand with gravel and small rock fragments in a dry to moist and loose condition and sandy clay in a moist and soft condition.

4.3.1.4. Older Alluvium (Qoal)

Older alluvium occurs onsite as moderately dissected terraces that flank modern drainage channels/valleys. The older alluvium consists of poorly bedded, poorly to moderately well consolidated sand to boulder sized sediment in a clayey sand matrix. Clasts are generally subangular to subrounded. Matrix soils are commonly rubified and locally exhibit weak cementation.

4.3.1.5. Ancient Landslide Debris (Qls)

Ancient landslide debris was encountered during recent subsurface exploration conducted during August 2019. The landslide debris is mapped in the central portion of Village 14 in the vicinity of neighborhood R-8, southeasterly adjacent to the proposed new alignment of Proctor Valley Road. As encountered, the landslide debris varied from silty to sandy clay and clayey sand with common gravel near the toe of the landslide to breccia with sandy clay to clayey sand matrix in the upper portion of the landslide. The landslide debris is generally in a soft to moderately hard condition with abundant carbonate development. Thin, weakly developed and discontinuous shears were encountered at depths of 12 feet and 15 feet below ground surface in boring LD-1. The shears were observed to dip shallowly in a southeasterly direction and minor seepage/perched groundwater flowed into the excavation from above each shear.

4.3.2. Bedrock Units

4.3.2.1. Otay Formation – Fanglomerate (Tof)

Regional mapping of the El Cajon 30’ x 60’ quadrangle by Todd (2004) identifies the unit as an ‘unnamed’ fanglomerate of Pliocene and Miocene age. Tan (2002) identifies the unit in the Jamul Mountains 7.5’ geologic map as Otay Formation - Fanglomerate of Oligocene and Miocene age. In consideration of the non-conformable contact with the underlying Santiago Peak Volcanics, the presence of

intertongued lenses of bentonitic sandstone and claystone common to the Otay Formation, and the presence of Otay Formation overlying the fanglomerate facies west of the Project Area, we consider the fanglomerate unit to be roughly coeval with the Otay Formation and are following Tan's (2002) designation.

The Otay Formation - Fanglomerate underlies much of the Project Area and occupies the lower flanks and valleys of the highlands to the east and north of the Project Area. The fanglomerate has a more subdued topography and is moderately to highly dissected. This unit is typified by thickly to massively bedded breccia intertongued with a finer grained subunit consisting of claystone and sandstone. The breccia subunit is generally in a slightly moist to moist and moderately hard to hard condition. The breccia subunit is composed of subangular to angular, gravel to cobble size clasts in a clayey sand matrix. Occasional to common boulder sized clasts were encountered in our borings and excavator test pits. Rock clasts appear to be locally derived from the Santiago Peak Volcanics. The clay matrix is commonly waxy, highly expansive, and is likely bentonitic. The finer grained subunit is generally comprised of olive gray to pale brownish yellow, sandy claystone and clayey sandstone in slightly moist to moist and soft to hard condition.

4.3.2.2. Santiago Peak Volcanics (Jsp)

The Project Area is underlain by Jurassic-aged Santiago Peak Volcanics at depth and outcrops at the surface primarily in the eastern and northern portions of the Project Area. The contact between the Santiago Peak Volcanics and the overlying younger geologic units represents a significant geologic hiatus. This contact is irregular and reflects a relatively high relief Mesozoic landscape. Subsequent erosion has exhumed portions of this ancient landscape, creating modern topographic highs including San Miguel Mountain to the north and the Jamul Mountains to the east.

The Santiago Peak Volcanics are generally dense and mildly metamorphosed volcanic rocks. Composition of the volcanic rocks varies from basalt to rhyolite but is predominantly dacite and andesite (Kennedy and Tan, 1977). Typically the meta-volcanics display crude to moderate bedding and foliation. Fracturing is poorly to moderately well developed. In general, outside of boulder areas, a weathered halo of only a few feet thick exists. Below this, the rock is very dense and hard.

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 6). 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 Silver Strand section of the Newport-Inglewood-Rose Canyon fault zone is closest known active fault to the project and is located approximately 15 miles to the west.

4.4.2. Local Faulting

Alquist-Priolo County Special Studies Fault Zones and San Diego County Fault Zones are not located onsite. The most influential geologic fault potentially affecting the property is the active Silver Strand section of the Newport-Inglewood-Rose Canyon fault zone. The La Nacion Fault is located approximately 6 miles west of the Project Area. The La Nacion Fault is currently considered to be a potentially active fault, having evidence of displacement within the Quaternary Period. Some recent, independent reports indicate there is evidence to support classification as an active fault (movement within the last ~11,000 years).

No faults have been mapped onsite on published geologic maps and none were observed during this geologic studies. Offsite to the northwest, lineaments were observed both in the field and on aerial photos. These lineaments occur in the Santiago Peak Metavolcanics and are considered to be related to foliation in the metamorphic rock and are not related to faulting. Foliation is discussed in further detail in Section 4.4.3 below.

4.4.3. Geologic Structure

The Otay Formation rests nonconformably over metavolcanic basement rock of the Santiago Peak Volcanics. Geologic structure within the sedimentary Otay Formation is typically characterized by regional westerly to southwesterly dipping beds with inclinations on the order of 3 to 7 degrees from horizontal. However, geologic mapping by Tan (2002) shows a bedding attitude within the fanglomerate having a northeasterly strike with a 30-degree dip to northwest. Crude bedding observed within the Otay Formation as encountered during our subsurface exploration was generally flat lying to very slightly dipping to the west and east. Contacts between the between breccia (fanglomerate) and finer grained subunits are conformable and typically near horizontal to undulatory and indicative of scour and fill type successions.

Dominant foliations, fracture patterns or other structural features common to bedrocks were not mapped during this study. Geologic mapping by Tan (2002) shows foliation in the Santiago Peak Volcanics as predominantly striking northwest to north-northeast and dipping steeply to the west and east. Lineaments identified by air photo analysis are also presented on this map and strongly correlate with the local foliation.

4.5. Groundwater

Shallow groundwater was not observed during this study. Groundwater seepage was encountered above two thin clay seams in boring LD-1 at depths of 12 and 15 feet below ground surface.

Intermittent flows within the active primary and tributary drainages should be anticipated during rain events.

4.6. Non-seismic Geologic Hazards

4.6.1. Mass Wasting and Debris Flows

Tan (1992) identified the majority of the Proposed Project Area as being generally susceptible to landsliding, hypothesizing that due to the presence of weaker materials (fine-grained fanglomerate subunit) many slopes within the Project Area are at or near their stability limits. Based on our review of aerial photographs and observations of the Project Area, there is geomorphic evidence that suggests the presence of landslide(s) locally onsite and offsite. Recent subsurface exploration confirmed the presence of an ancient landslide within the Project Area in the vicinity of neighborhood R-8. Current plans indicate a large portion of the landslide debris will be removed to achieve design grades. The remaining portions will be removed and the surrounding slopes will be constructed as drained buttress fills. The presence of an ancient landslide is anticipated to have a less than significant effect on the proposed development.

Based on our field investigation, the majority of the Proposed Project Area is sloping at shallow to moderate slope ratios and are underlain by bedrock that is not considered to be generally susceptible to mass wasting. In addition, bedding attitudes mapped in the field are predominantly favorable with respect to slope stability. In general, it is our opinion that the potential for landsliding is low to moderate in its current state. The potentially adverse effects of landsliding can be mitigated during development through the use of design avoidance or through typical remedial grading measures (removal and recompaction and/or construction of stabilization and buttress fills). Accordingly, it is our opinion that the susceptibility to landsliding onsite after development is considered to be less than significant.

Due to the presence of the steeper offsite terrain composed of highly fractured/jointed Santiago Peak Volcanics, the potential for debris flows emanating from the mouths of the up-gradient drainages is feasible but the likelihood is considered to be less than significant.

4.6.2. Rock Fall

The potential for rock fall is generally considered to be low given the limited rock outcrops and subdued topography within a majority of the proposed development. Mitigation for rock fall is considered to be unlikely. Mapping and evaluation of hard rock slopes should be performed by an engineering geologist prior to and during the development of the Project Area. Rock fall hazards could become potentially significant if unforeseen conditions are encountered during the development of the Project Area. If rock fall hazards are encountered during grading mitigation measures during development may be warranted to reduce the potential risks. Potential mitigation measures could include scaling of the slope faces, construction of catchment areas or debris fences, and removal of precariously situated boulders. It is our opinion that the potential risk for rock fall hazards within the

Project Area currently is less than significant, and if any rock fall hazards are encountered during grading, these mitigation measures also would reduce the potential risk to less than significant.

4.6.3. Flooding

Detailed FEMA flood maps are not currently available for the Proposed Project Area. The San Diego County Hazard Mitigation plan indicates the Project Area is located outside designated 100- and 500-year floodplain areas. The potential for flooding is considered to be low.

4.6.4. Subsidence and Ground Fissuring

Owing to the presence of shallow bedrock and dense formational materials underlying the Proposed Project, subsidence and ground fissuring potential within the Project Area is considered very low.

4.7. Seismic Hazards

The Proposed Project 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 7) shows the distance of the Project Area from near source shaking zones. The type and severity of seismic hazards affecting the Project Area 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 a less than significant level of risk. The following seismic hazards discussion is guided by the California Building Code (2016), CGS (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. In large part, research supports the conclusion that active faults tend to rupture at or near pre-existing fault planes. No 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 Project Area is within the tectonically active southern California area, with segments of the Newport-Inglewood-Rose Canyon fault zone within 15 miles of the Project Area. 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 Ground Motion Interpolator Page. A site location with latitude of 32.6756˚N and longitude 116.9161˚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 soil conditions. Ground motion values are shown for two different soil conditions: Rock (site category B, V_s30=760m/s) and Stiff Soil (site category D, V_s30=270m/s).

TABLE 4.7.2
SELECTED GROUND MOTIONS*

	Bedrock	**Stiff Soil**
Pga (g)	0.171g	0.224g
Sa 0.2 sec	0.396g	0.508g
Sa 1.0 sec.	0.142g	0.274g

*Ground Motion values were interpolated from a grid (0.05 degree spacing) of values calculated using the 2008 PSHA model. Interpolated ground motion may not equal values calculated for a specific site, therefore these values are not intended for design or analysis.

Currently, non-critical structures (commercial, residential, and industrial) are usually designed according to the 2016 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 where seismic agitation of loose, saturated sands and silty sands can result in a buildup of pore pressures that, if sufficient to overcome overburden stresses, can produce a temporary quick condition. Localized, loose lenses/layers of sandy soils may be subject to liquefaction when a large, prolonged, seismic event affects the Project Area. As the excess pore water pressure dissipates, the liquefied zones/lenses can consolidate causing settlement. Post liquefaction effects within the Project Area can manifest in several ways and may include: 1) ground deformations; 2) loss of shear strength; 3) lateral spread; 4) dynamic settlement; and 5) flow failure.

In general, the more recently sediment has been deposited, the more likely it is to be susceptible to liquefaction. Further, liquefaction potential is greatest in loose, poorly graded sands and silty sands with mean grain size in the range of 0.1 to 0.2 mm. Other factors that must be considered are groundwater, confining stresses, relative density, intensity and duration of ground shaking. It is generally held that soils possessing clay content (particle size < 0.005mm) greater than fifteen (15) to twenty (20) percent may be considered non-liquefiable (Southern California Earthquake Center, 1999).

The State of California has mandated that the California Geological Survey (2008) identify areas that may be susceptible to liquefaction and provide *USHMA* quadrangle maps showing these zones establish procedures for investigating same and ensure that local agencies require such studies prior to project approval. Similarly, the County of San Diego has established its study zones. The Project Area is not in a State liquefaction susceptibility zone; however, localized areas of the Proposed Project are situated within an area zoned

by the County of San Diego as a Potential Liquefaction Area (Figure 8). The boundary of the Potential Liquefaction Area in proximity to the Project Area, utilizing County of San Diego GIS data, is presented on Plates 12 through 15. The area identified by the County of San Diego as being potentially susceptible to liquefaction appears to be related to young alluvial soils contained within the primary drainage north of Upper Otay Reservoir but, likely do to scale, the County's mapped boundary extends beyond the limits of the drainage and includes areas not susceptible to liquefaction. Based on geologic mapping, subsurface exploration, and laboratory testing specific to the Project Area, it is our opinion that a large portion of the area identified by County of San Diego as being susceptible to liquefaction is not liquefiable due to the soils age, density, and fines content.

In an effort to better evaluate the potential for liquefiable soils to impact the Proposed Project Amendment, the area within the County's boundary has been further delineated to show the limits of areas considered not liquefiable and those that are potentially susceptible to liquefaction. The refined boundaries are presented on Plates 12 through 15. The areas identified as not susceptible to liquefaction are delineated with purple cross-hatching. The yellow cross-hatching delineates areas we consider to be potentially susceptible to liquefaction. Areas potentially susceptible to liquefaction include a portion of the proposed school site (S-1), mixed use areas (MU-1 & 2), and a private park (PP-2) in Center Village 14 and portions of proposed Proctor Valley Road south and west of South Village 14.

The potentially liquefiable soils in the area of the proposed school site are shallow and will be mitigated through complete removal and replacement with compacted fill during grading operations. Proposed offsite improvements located in areas susceptible to liquefaction can also be mitigated through complete removal and replacement with compacted fill. If complete removal and replacement cannot be accomplished, mitigation in the form of ground improvement (e.g. stone columns) and/or deepened foundation elements will be required to mitigate the liquefaction potential to an acceptable level of risk.

In consideration of the recommended remedial grading, and dense nature of the formational materials and proposed fills within the limits of the Proposed Project Amendment, the potential for liquefaction or seismically induced settlement is considered remote.

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 cohesionless sand soils. 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 cohesionless sandy soils. These two sources of settlement

potential are mutually exclusive. As a result, 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 bedrock, the potential for seismically induced settlement is considered very low.

4.7.6. Seismically Induced Landsliding

Seismically induced landsliding of engineered fill slopes is considered to be very low. For cut slopes excavated in the metavolcanic bedrock and remaining shallow natural slopes, the potential for seismically induced landsliding is considered to be very low. Cut slopes excavated in the fine-grained subunit of the fanglomerate has low to moderate potential for seismically induced landsliding. However, as discussed in Section 6.2, cut slopes within this unit will likely be removed and replaced with engineered fill in the form of buttresses or stability fills. Pseudo-static slope stability analyses were performed using a horizontal destabilizing seismic coefficient (kh) of 0.15g for the highest design cut and fill slopes and were determined to be grossly stable. A more detailed discussion of slope stability is presented in Section 6.2.

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 an up gradient freestanding body of water nearby, the potential for a seiche impacting the Project Area 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 remote.

Considering the distance of the Project Area 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

It is anticipated that excavations within the alluvium, older alluvium, colluvium/residual soil, fanglomerate and highly weathered portions of the Santiago Peak Volcanics can be

accomplished with conventional grading equipment (D-9 or equivalent). It is likely that oversized "float" will be encountered in surface outcrops and will require special handling.

As part of our current study, AGS subcontracted Southwest Geophysics, Inc. (SGI) to perform 13 seismic refraction survey lines (SL-1 through SL-13) within the Project Area and provide two-dimensional tomographic models for each traverse. The report by SGI is presented in Appendix B. Approximate locations of the survey lines are shown on Plates 1 through 5. Generally, it has been AGS's experience that when velocities are higher than 6,500 to 7,000 feet/sec., blasting will be required for efficient excavation. Although it is possible that in certain instances seismic velocities in excess of 6,500 feet/sec. can be ripped, production rates are typically low and drilling and shooting may be preferred in order to increase production. Velocities in areas greater than 5,000 to 5,500 feet/sec. may require localized blasting for efficiency during grading and will probably contain common boulders that will require special handling. It should be anticipated that oversized materials will be generated from cuts in the bedrock. These oversized materials should be handled as discussed in Section 6.6.6. Recommended undercuts to remove hard rock from the near pad grade and within utility alignments are presented in Section 6.1.2.

In addition, 23 test pit excavations (EX-1 through EX-23) utilizing a Caterpillar 349 excavator, which is comparable in size and breakout power to a Caterpillar D9 bulldozer, were performed at accessible locations throughout the Project Area. Refusal was encountered in several of the exploratory test pits excavated with the Caterpillar 349 excavator. Depth of refusal provides a reasonable estimation of the depth of rippable materials in the area. Logs of the excavator pits are presented in Appendix B.

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. 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 Proposed Project Amendment, 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. Additionally, numerous other factors can affect whether 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, areas underlain by the breccia subunit of the fanglomerate and cemented portions of the older alluvium will generally be difficult to excavate but rippable with larger Bulldozers (Caterpillar D-9 or equivalent) to currently proposed cut depths. Excavations within the Santiago Peak Volcanics below the upper weathered surface (approximately 5 feet) will require blasting for efficient excavation in order to achieve design grade as well as the undercuts to accommodate footings, utilities and other subsurface improvements. Recently, heavier

equipment has become available including large Bull Dozers (Caterpillar D-10 and D-11) 190,000 to 230,000lb machines and Self Propelled Shovels (Caterpillar 5130) 400,000lb machines which have greater down forces resulting the ability to excavate in fractured rock with higher velocities. As a result, this may reduce the volume of hard rock requiring blasting. It is likely that the blasting and excavation operations will generate oversized rock fragments requiring specialized handling and grading techniques.

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 will likely be left in place in order to provide adequate overburden for effective blasting. Techniques for potential blasting of hard-rock within the Project Area should be evaluated by a blasting specialist during the grading plan review stage of the Proposed Project Amendment. Further, it is recommended that a grading and blasting logistics program should be developed to allow for efficient excavation and to reduce the potential for adverse effects of blasting.

5.1.2. Oversized Materials

Oversized rock greater than 24 inches will be generated in design cuts and over excavations within the Santiago Peak Volcanics. Portions of the Older Alluvium and fanglomerate will locally generate oversized rock. 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.

It is recommended that the maximum rock size between three (3) feet and ten (10) feet of finished grade is restricted to twenty-four (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 topsoil, alluvium, colluvium, and highly 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 sandy 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. Typical mitigation measures consist of removal and recompaction of these soils where these soils are found within structural areas.

5.1.5. Expansion Potential

Based upon the sampling and associated laboratory testing conducted by AGS the onsite soils are considered to exhibit "Very Low" to "Very High" expansion potential, with the majority of the onsite soils possessing "Low" to "High" expansion potential. Typical mitigation measures for expansive soils include: structural design; pre-saturation; and overexcavation where the higher expansion characteristics are present and replacement with lower expansive soils (selective grading).

5.1.6. Shear Strength

Shear strength testing was conducted by AGS on undisturbed and remolded samples that were collected during this study (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 metavolcanic bedrock possesses relatively high shear strength and can stand unsupported at relatively steep slope ratios. The breccia subunit of the fanglomerate and older alluvium generally possess "good" in-situ shear strength except within the upper weathered horizon (upper five feet). The fine-grained subunit of the fanglomerate contains bentonitic claystone and clayey sandstones. This subunit possesses "poor to moderate" in situ shear strength. The alluvium generally can be characterized as possessing "poor to fair" strength characteristics. The shear strength of the fill soils created during grading generally will exhibit "moderate to good" shear strength for fill slopes and for support of structures. The shear strengths recommended by AGS for use in preliminary 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 @90% (afc_{90})	200	32	125
Artificial Fill Compacted @93% (afc_{93})	250	33	125
Alluvium	100	25	120
Fanglomerate	300	33	130
Santiago Peak Volcanics (Jsp)	500	40	140

5.1.7. Chemical and Resistivity Test Results

The initial test results from AGS's investigation in the general area indicate that the water soluble sulfate concentrations for the onsite soils tested ranged from 0.01% to 0.603% which corresponds to a "negligible" to "severe" sulfate exposure when classified in accordance with ACI 318-05 Table 4.3.1 (per 2016 CBC). Resistivity testing of the onsite soils ranged from 223 ohm-cm to 776 ohm-cm which corresponds to "low" to "very low" resistivity. Based upon these initial test results higher concrete strength, low water to cement ratios (0.5 to 0.45) and specialized cement types (Type V) could be required to

mitigate the adverse effects these aggressive soils could have on concrete and buried metallic pipes.

As the majority of this testing was conducted in the Fanglomerate deposits it is anticipated that further chemical and resistivity testing will indicate that the other geologic deposits found onsite (Santiago Peak Metavolcanics, alluvium, and older alluvium) will have more favorable characteristics. Based upon our past experience in the general area these other deposits will likely have significantly lower water soluble sulfate concentrations ("negligible" to "moderate") and will likely exhibit higher resistivity.

5.1.8. Earthwork Adjustments

The following average earthwork adjustment factors are presented for use in evaluating earthwork quantities. The numbers for earthwork adjustments are considered approximate and should be refined during grading when actual conditions are better defined. Contingencies should be made during grading, to modify the earthwork adjustment factors, if necessary.

TABLE 5.1.8
EARTHWORK ADJUSTMENTS

Geologic Unit	Approximate Range
Topsoil/Colluvium & Alluvium (Qal)	10% to 12% Shrink
Older Alluvium (Qoal)	0% to 5% Bulk
Landslide Debris (Qls)	6% to 12% Shrink
Fanglomerate (Tof)	10% to 15% Bulk
Santiago Peak Volcanics (Jsp): (*Rippable*)	15% to 18% Bulk
Santiago Peak Volcanics (Jsp): (*Blasting*)	18% to 25% Bulk

5.1.9. Permeability/Infiltration Potential

AGS conducted eight (8) borehole percolation tests (P-1 through P-8) at various locations throughout the Proposed Project Area. Approximate locations of the percolation test holes are presented on the Geologic Map and Exploration Location Plan, Plates 1 through 11, included herewith. Testing was performed in accordance with the methods described in Appendix D of the 2016 County of San Diego BMP Design Manual. Field percolation rates were converted to infiltration rates using the using the Porchet method. Based on the results of our subsurface investigation and percolation testing within the Project Area, it is anticipated that the onsite soils and bedrock possess low infiltration rates. Preliminary infiltration rates ranged between 0.09 in/hr and 0.36 in/hr. A more detailed discussion of test methods and findings are presented in Appendix E – Infiltration Feasibility Study. Table 5.1.9 presents estimated infiltration rates for the various onsite soil and geologic

units. Dependent upon proposed BMP type and location, additional infiltration may be warranted.

TABLE 5.1.9
ESTIMATED INFILTRATION RATES

Geologic Unit	Estimated Infiltration Rate (Inches per Hour)
Topsoil/Colluvium & Alluvium (Qal)	0.10 – 0.50
Older Alluvium (Qoal)	0.05 – 0.35
Fanglomerate (Tof)	0.05 – 0.20
Santiago Peak Volcanics (Jsp)	0.00 – 0.10

5.1.10. Pavement Support Characteristics

Compacted fill derived from onsite soils and cuts within the older alluvium and fanglomerate are expected to possess "poor to moderate" pavement support characteristics. Cuts within the Santiago Peak Volcanic rock are anticipated to exhibit "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 20 for the preliminary design of roadway pavement sections.

5.2. Analytical Methods

5.2.1. Slope Stability Analysis

Stability analyses were performed for both static and seismic (pseudo-static) conditions using the GSTABL7 computer program. The Modified Bishop method was used to analyze circular-type failures. The critical failure surface determined in the static analysis was used in the pseudo-static analysis. A horizontal destabilizing seismic coefficient (kh) of 0.15g was selected for the Project Area 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 Project Area, it is AGS's opinion that the proposed development of Otay Ranch Village 14 and Planning Area 19 Proposed Project Amendment is feasible, from a geotechnical point of view, provided that the constraints discussed in this report are addressed in the design and construction phases. Key issues within the development footprint are discussed and associated 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 F).

6.1. Project Area Preparation and Removals/Over excavation

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 in 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 adequate moisture conditioning and mixing.

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. During grading, 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. Project Area Preparation and Removals

Grading should be accomplished under the observation and testing of the project soils engineer and engineering geologist or their authorized representative in accordance with the recommendations contained herein, the current San Diego County Grading Ordinance, and AGS's Earthwork Specifications (Appendix F). Existing vegetation, trash, debris and other deleterious materials should be removed and wasted from the Project Area prior to removal of unsuitable soils and placement of compacted fill. Artificial fill, topsoil/colluvium, alluvium, landslide debris, highly weathered older alluvium and highly weathered Otay Formation - Fanglomerate and Santiago Peak Volcanics should be removed in areas planned to receive fill or where exposed at final grade. The resulting undercuts should be replaced with engineered fill. Estimated depths of removals based upon the geologic unit are presented in Table 6.1, it should be noted that local variations can be expected requiring an increase in the depth of removal for unsuitable and weathered deposits. The extent of removals can best be determined in the field during grading when

observation and evaluation can be performed by the soil engineer and/or engineering geologist. Removals should expose competent formational materials and be observed and mapped by the engineering geologist prior to fill placement. In general, soils removed during remedial grading will be suitable for reuse in compacted fills provided they are properly moisture conditioned, mixed, and do not contain deleterious materials.

Table 6.1

Estimated Depth of Removal

Geologic Unit (map symbol)	Estimated Removal Depth
Undocumented Artificial Fill (afu)	3-15 feet
Topsoil/Colluvium (No Map Symbol)	2-5 feet
Alluvium (Qal)	4-10 feet
Older Alluvium (Qoal)	1-4 feet
Landslide Debris (Qls)	10-40 feet
Otay Formation – Fanglomerate (Tof)	1-3 feet
Santiago Peak Volcanics (Jsp)	1-3 feet

6.1.2. Overexcavation

6.1.2.1. Cut Lot Overexcavation

Cut lots exposing older alluvium, fanglomerate, and Santiago Peak Volcanics 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.

6.1.2.2. 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 exists 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.

6.1.2.3. Street Overexcavation

Streets that are cut into older alluvium, fanglomerate and metavolcanic bedrock could potentially pose excavation difficulties during utility and street installation. The older alluvium and fanglomerate may require heavy ripping and the

metavolcanic bedrock will likely 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.

6.1.3. Removals Along Grading Limits and Property Lines

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

6.2. Slope Stability and Remediation

Based on current geologic information, it is generally anticipated that the proposed permanent graded cut and fill slopes will be grossly and surficially stable as designed. The majority of the Project Area is underlain by Metavolcanic bedrock and Tertiary age Otay Formation – Fanglomerate which are locally mantled by Quaternary age surficial soil units. It should be noted that weak and/or sheared zones can be found within claystone beds in the Otay Formation – Fanglomerate. If encountered during grading operations in sloped areas, these slopes may require construction of stabilization fills to provide adequate long-term slope stability.

As discussed in our Response to County Review Comments (AGS, 2016) it is AGS's opinion that six foot wide fill and cut slope terrace benches for every 40 feet are not required for geotechnical stability. Historically, these benches have been constructed to provide maintenance access and are not a geotechnical consideration for the long term stability of the slope. In some instances these benches have created instability resulting in long term maintenance issues.

6.2.1. Cut Slopes

The highest proposed cut slopes are approximately 120 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 Otay Formation - Fanglomerate will be grossly stable at slope ratios of 2:1 (horizontal: vertical) or flatter to maximum proposed height and that proposed cut slopes in Metavolcanic bedrock will be grossly stable at slope ratios of 1.5:1 or flatter to maximum proposed height. Calculations supporting AGS's conclusions and recommendations relative to cut slopes are represented in Appendix D (Plates D-1 thru D-3).

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

TABLE 6.2.1 CUT SLOPE STABILITY			
Slope Height (Slope Ratio)	Geologic Unit	Static F.S.	Pseudostatic F.S.
120 ft. (2:1)	Tof	1.90	1.34

6.2.2. Fill Slopes

Fill slopes on the project are designed at 2:1 ratios (horizontal to vertical) or flatter. The highest anticipated fill slope is approximately 150 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 through D-6). 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.

TABLE 6.2.2 FILL SLOPE STABILITY		
Slope Height	Static F.S.	Pseudostatic F.S.
150 ft. (2:1 Fill Slope)	1.65	1.17

6.2.3. Skin Cut and Skin Fill Slopes

A review of the preliminary grading 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. If these conditions occur, 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 F.

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 proposed 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) 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 Project Area. Canyon subdrains will range in diameter from 6 to 8 inches in diameter and should be constructed in accordance with Grading Details 1 and 2. Final determination as to the location and the size of these subdrain systems will be dependent upon the

final design grades and length of drain sections. Accordingly, once more detailed plans become available, specific recommendations will be prepared regarding the size, location, and extent of the subdrain system for the project. Actual subdrain locations will be determined in the field, after completion of remedial grading.

Backdrains, where required, should be constructed in accordance with Grading Detail 2. Drains should be installed behind all retaining walls. Drainage devices behind retaining walls are discussed in further detail in section 7.1.2.

6.5. Seepage

Seepage, if 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. 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). 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) until the desired grade is achieved. For fills greater than 50 feet, AGS recommends a minimum compaction standard of 93 percent of the maximum dry density (ASTM D1557). For fills deeper than 75 feet, AGS recommends a minimum compaction standard of 95 percent of the maximum dry density (ASTM D1557).

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.

6.6.2. 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.

6.6.3. Haul Roads

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

6.6.4. Import Soils

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

6.6.5. 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 established by the County of San Diego's grading and blasting ordinances.

6.6.6. 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

6.6.6.1. 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 rock 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) of an 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.

6.6.6.2. Rock Windrows

Rocks up to a 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 the width of the equipment and 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.

6.6.6.3. Individual Rock Burial

Rocks in excess of four (4) feet, but not 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.

6.6.6.4. Rock Disposal Logistics

The grading contractor should consider the volume of rock disposal 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.

6.6.7. 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.

6.6.7.1. Overbuilding Fill Slopes

Fill slopes should be overfilled as determined by the grading contractor, but not less than 2 feet measured perpendicular to the slope face, so that when trimmed back to the compacted core, 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 slope should be back-rolled at fill intervals not exceeding 4 feet in height, unless a more extensive overfilling is undertaken.

6.6.7.2. 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 lift 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.

6.6.8. 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, and 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. 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, and 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 Project 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 is our understanding that the Proposed Project Amendment will be graded and at least some portion of the 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 6 ksf for spread and continuous footings), conventional or post-tensioned shallow slab-on-grade foundation systems will be utilized

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.

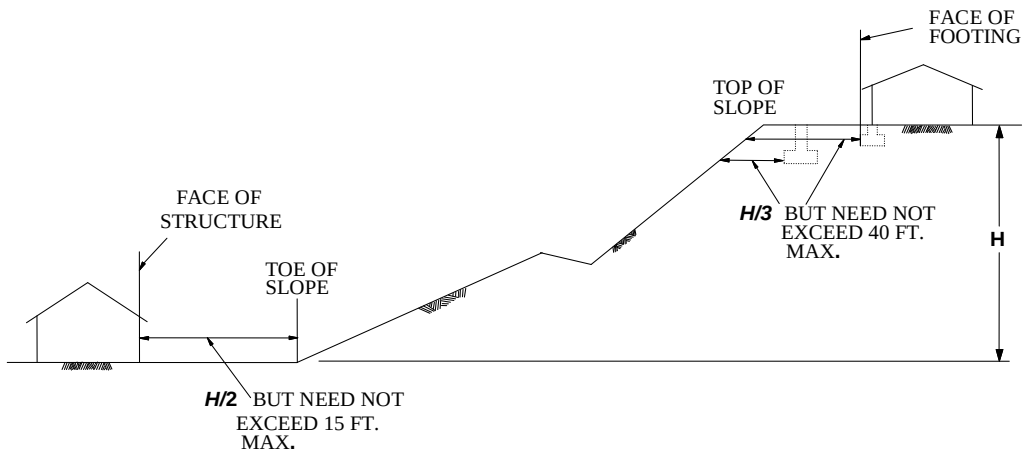
7.1.1.1. Deepened Footings and Setbacks

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 development footprint, 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, 2016)



7.1.1.2. 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≤50)				
	Level Backfill		**Sloping (2:1) Backfill**	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	K_a = 0.33	42	K_a = 0.54	67
Passive Pressure	K_p = 3.00	375	K_p = 1.12	140
At Rest Pressure	K_o = 0.50	63	K_o = 0.81	101
"Select"* Backfill Materials (γ=120pcf, EI≤20, SE≥20)				
	Level Backfill		**Sloping (2:1) Backfill**	
	Rankine Coefficients	**Equivalent Fluid Pressure (psf / lineal foot)**	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf / lineal foot)**
Active Pressure	K_a = 0.28	34	K_a = 0.44	53
Passive Pressure	K_p = 3.54	420	K_p = 1.33	160
At Rest Pressure	K_o = 0.44	53	K_o = 0.75	90

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

In addition to the above static pressures, unrestrained retaining walls should be designed to resist seismic loading as required by the 2016 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 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. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and 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 backfill to confirm that the walls are properly backfilled and compacted.

7.1.3. Seismic Design

The Project Area has been identified to have multiple site classes (Site Class B, C, and D) in accordance with CBC, 2016, Section 1613.3.2 and ASCE 7, Chapter 20. Utilizing this information, the United States Geological Survey (USGS) web tool (http://earthquake.usgs.gov/hazards/designmaps/) and ASCE 7 criterion, the mapped seismic acceleration parameters S_S, for 0.2 seconds and S_1, for 1.0 second period (CBC, 2016, 1613.3.1) for Risk-Targeted Maximum Considered Earthquake (MCE_R) can be determined. The mapped acceleration parameters are provided for Site Class "B". Adjustments for other Site Classes are made, as needed, by utilizing Site Coefficients F_a and F_v for determination of MCE_R spectral response acceleration parameters S_{MS} for short periods and S_{M1} for 1.0 second period (CBC, 2016 1613.3.3). Five-percent damped design spectral response acceleration parameters S_{DS} for short periods and S_{D1} for 1.0 second periods can be determined from the equations in CBC, 2016, Section 1613.3.4. Results are presented in Table 7.1.3.

TABLE 7.1.3
SEISMIC DESIGN CRITERIA

PARAMETER	SITE CLASS B (Hard rock-Jsp)	SITE CLASS C (Soft Rock-Tof/Qoa)	SITE CLASS D (Compacted Fill-afc)
Mapped Spectral Acceleration (0.2 sec Period), S_S	0.818g	0.818g	0.818g
Mapped Spectral Acceleration (1.0 sec Period), S_1	0.318g	0.318g	0.318g
Site Coefficient, F_a	1.000	1.073	1.173
Site Coefficient, F_v	1.000	1.482	1.764
MCE Spectral Response Acceleration (0.2 sec Period), SM_S	0.818g	0.878g	0.960g
MCE Spectral Response Acceleration (1.0 sec Period), SM_1	0.318g	0.471g	0.561g
Design Spectral Response Acceleration (0.2 sec Period), SD_S	0.546g	0.585g	0.640g
Design Spectral Response Acceleration (1.0 sec Period), SD_1	0.212g	0.314g	0.374g

7.2. Civil Design Recommendations

7.2.1. Rear and Side Yard Walls and Fences

Block wall footings should be founded a minimum of 24-inches below the lowest adjacent grade. To reduce the potential for uncontrolled, unsightly cracks, it is recommended that a construction joint be incorporated at regular intervals. For side yard walls situated perpendicular to the top of slopes a joint should be constructed at approximately 10 feet from the slope hinge point. Spacing of the joints should be between 10 and 20 feet.

7.2.2. Drainage

Final 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. Water Quality/BMP Basins

AGS conducted percolation testing within the Project Area to evaluate feasibility for storm water infiltration within the Project Area and to determine preliminary infiltration rates for the proposed BMP basins. Testing was completed in general accordance with the 2016 County of San Diego BMP Design Manual. A copy of our Infiltration Feasibility Study is included herewith in Appendix E.

Based on the results of our testing, Partial Infiltration design for the proposed BMPs is potentially feasible. This is dependent on final basin size and location, and verification of the soil/geologic conditions.

7.2.4. 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 7.2.4 can be used for the range of likely traffic indices. The structural sections are based upon an assumed R - Value of 20.

TABLE 7.2.4
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	3	7.5
6.0	4	8.5
7.0	4	10.5
8.0	5	14.5

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. Aggregate base should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557 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 SUMMARY OF SIGNIFICANCE FOR GEOLOGIC HAZARDS

The potential for adverse effects related to soil and geologic hazards, both onsite and offsite, has been evaluated based on guidance presented in the County of San Diego Guidelines for Determining Significance – Geologic Hazards (2007). Soil and geologic hazards addressed in this report include fault rupture, ground shaking, liquefaction, landslides, rock fall, and expansive soils. It is our opinion that the risk associated with these hazards is less than significant or less than significant with mitigation. Where appropriate, mitigation measures are presented in the text of this report.

Because the Proposed Project Amendment would not result in any new significant geologic or geotechnical impacts beyond those disclosed in the Approved Project's Certified EIR, no new mitigation measures beyond those included in the Approved Project's Mitigation Monitoring and Reporting Plan (MMRP) are necessary.

9.0 FUTURE STUDY NEEDS

This report represents an EIR level TM review of the Proposed Project Amendment. As the project design

progresses, additional 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.

10.0 CLOSURE

10.1. 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.

10.2. Limitations

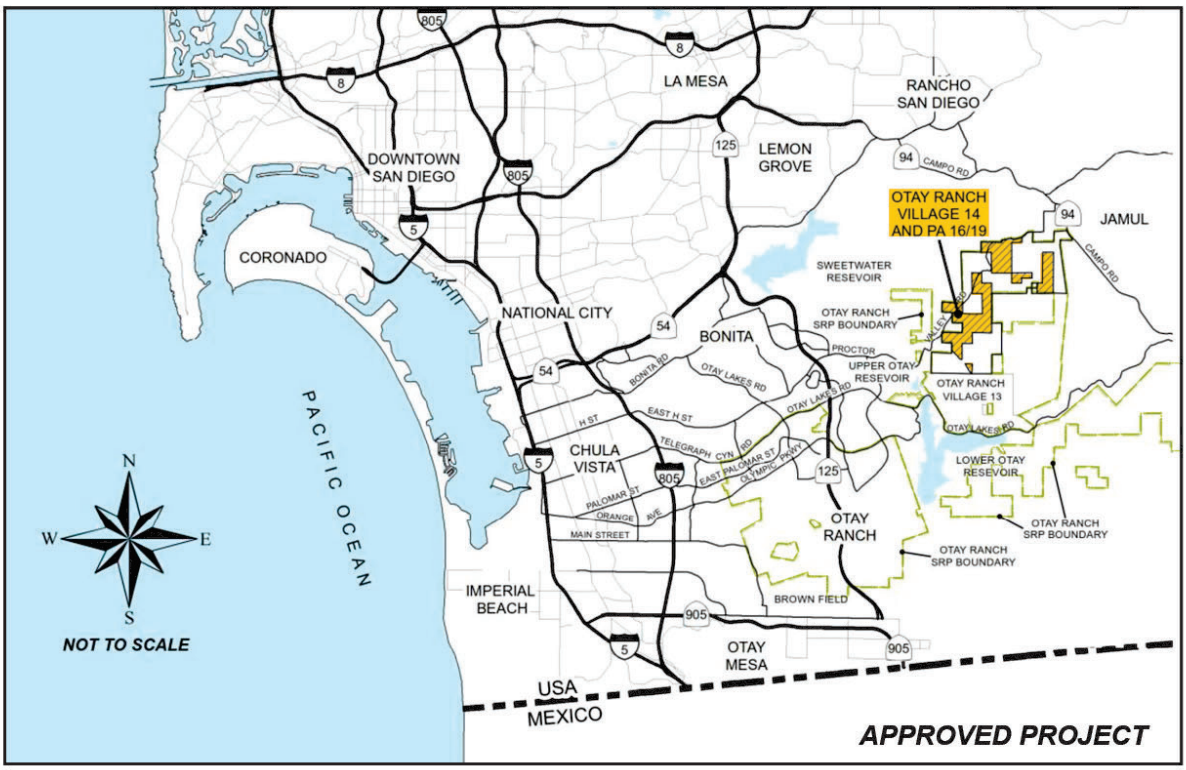
This report is based on the project as described and the information obtained from the test pits and the borings at the locations indicated on the plan. The findings are based on the review of the field and laboratory data combined with an interpolation and extrapolation of conditions between and beyond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

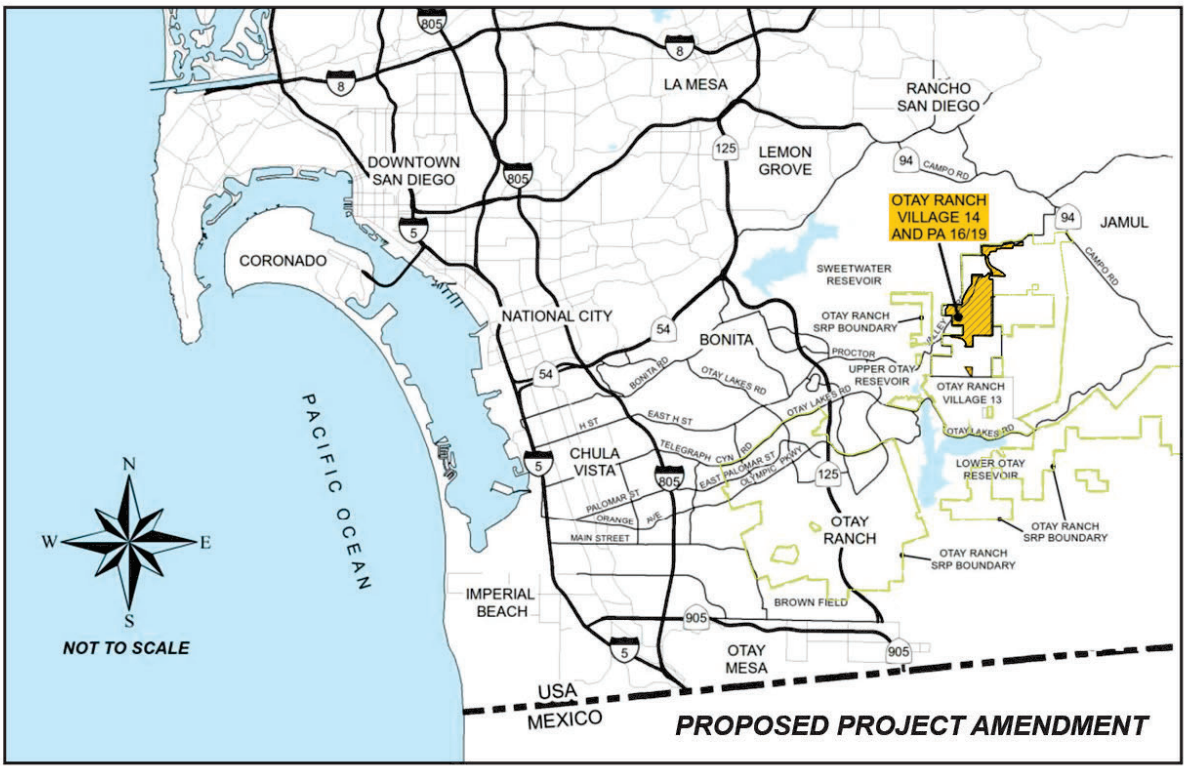
The recommendations presented in this report are based on the assumption that additional exploration will be performed and an appropriate level of field review will be provided by geotechnical engineers and engineering geologists who are familiar with the design and Project Area geologic conditions. 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 and future reports. 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.





P/W 1312-02

FIGURE 1

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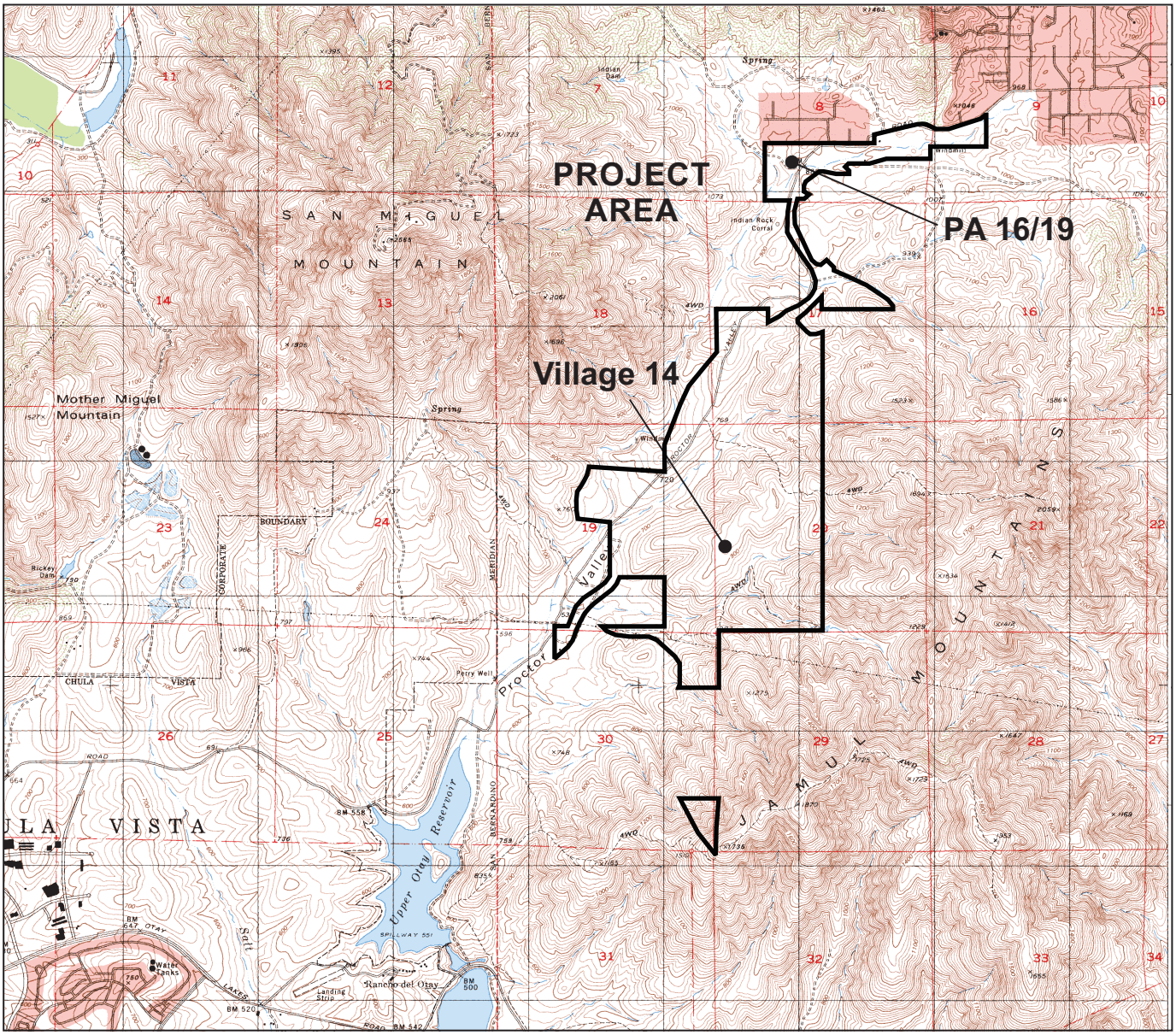

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1"=approx. 4000 ft.

PROJET AREA LOCATION MAP
PROPOSED PROJECT AMENDMENT
OTAY RANCH-VILLAGE 14 AND PA 16/19
SAN DIEGO COUNTY, CALIFORNIA

P/W 1312-02

FIGURE 2

SOURCE MAP - TOPOGRAPHIC MAP OF THE
JAMUL MOUNTAINS 7.5 MINUTE QUADRANGLE,
SAN DIEGO COUNTY, CALIFORNIA

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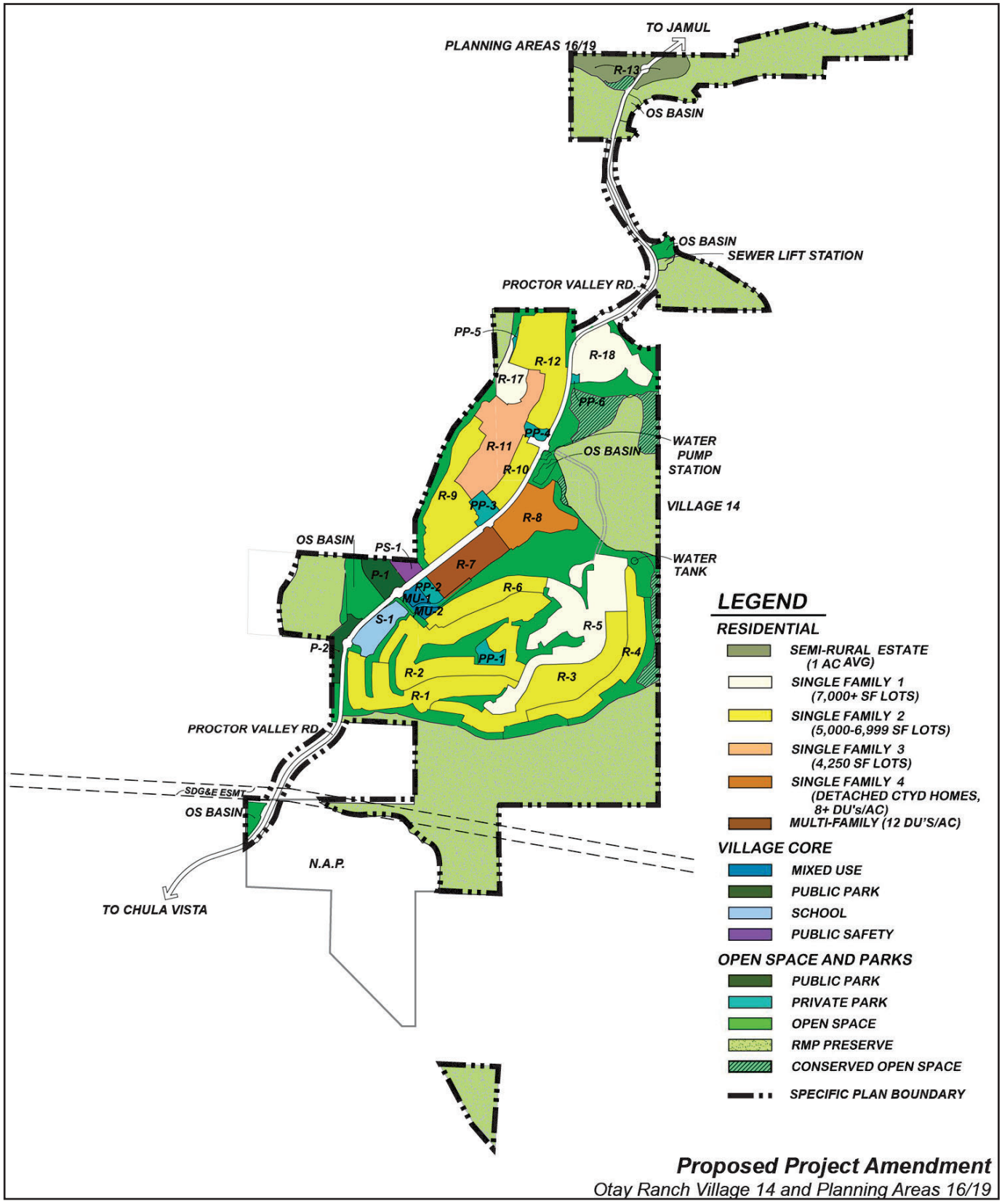

Proposed Project Amendment

Otay Ranch Village 14 and Planning Areas 16/19

P/W 1312-02

FIGURE 3

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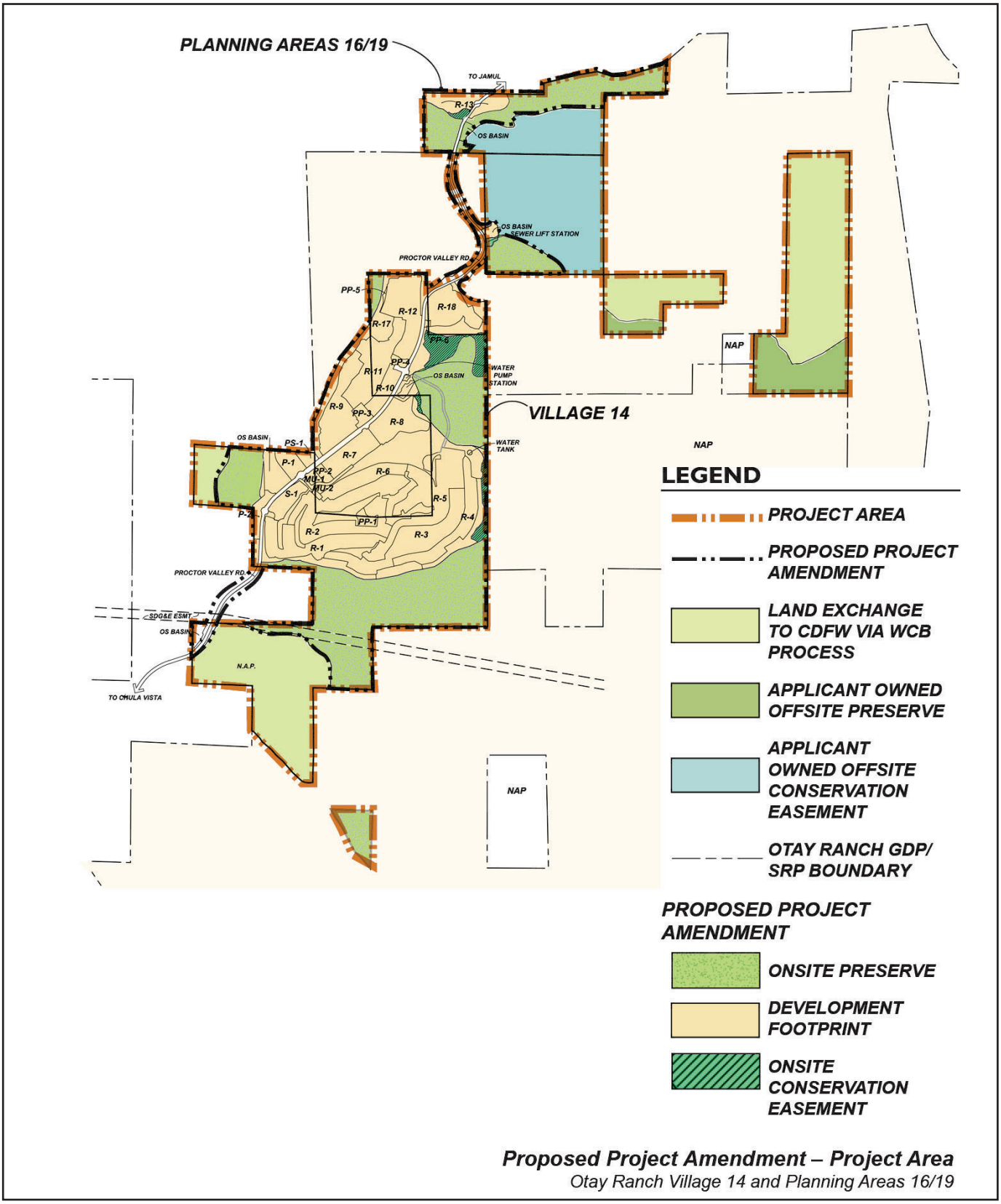

Proposed Project Amendment – Project Area

Otay Ranch Village 14 and Planning Areas 16/19

P/W 1312-02

FIGURE 4

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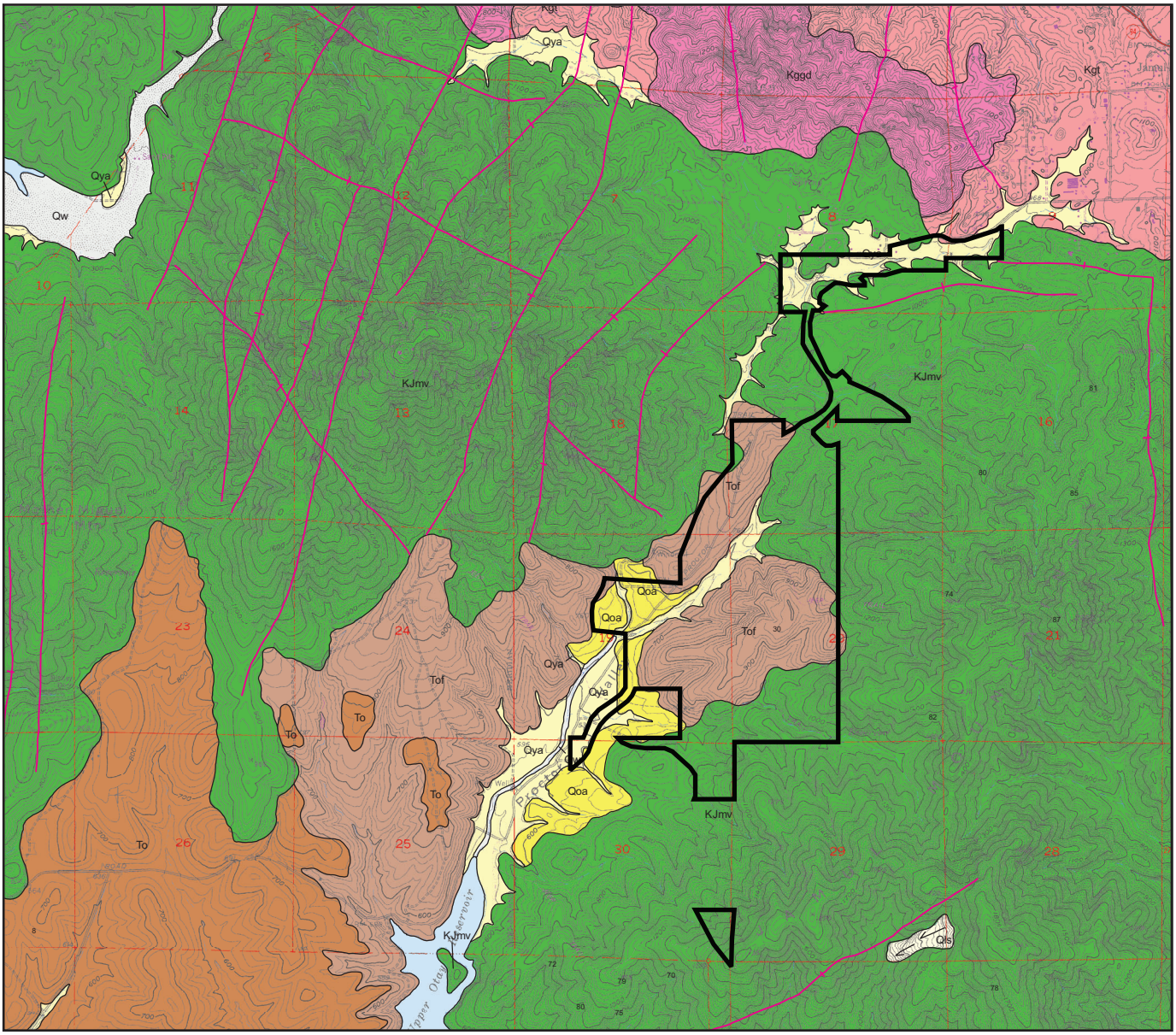

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1"=approx. 4000 ft.

REGIONAL GEOLOGY MAP
OTAY RANCH-VILLAGE 14 AND PA 16/19
SAN DIEGO COUNTY, CALIFORNIA

Qya	Holocene alluvial deposits unconsolidated to locally poorly consolidated silt, clay, sand and gravel. Included modern active sedmients along small drainage channels.
Qoa	Late Pleistocene alluvial deposits; moderately consolidated, poorly sorted flood plain deposits consisting of gravelly sandy silt and clay.
Tof	Otay Formation - fanglomerate facies (Oligocene to Miocene); poorly cemented bouldery conglomerate and coarse-grained sandstone. Interfingered with overlying To.
Kgt	Tonalite (Cretaceous); includes some granodiorite and quartz diorite; medium-grained; generally dark colored and severely weathered.
KJmv	Metavolcanic rocks (Jurassic and Cretaceous); mildly metamorphosed volcanic, volcaniclastic and sedimentary rocks. Volcanic rocks range from basalt to rhyolite, but are predominantly andesite and dacite.

P/W 1312-02

FIGURE 5

SOURCE MAP - GEOLOGIC MAP OF THE 7.5' JAMUL MOUNTAINS QUADRANGLE, TAN, S.S., 2002.

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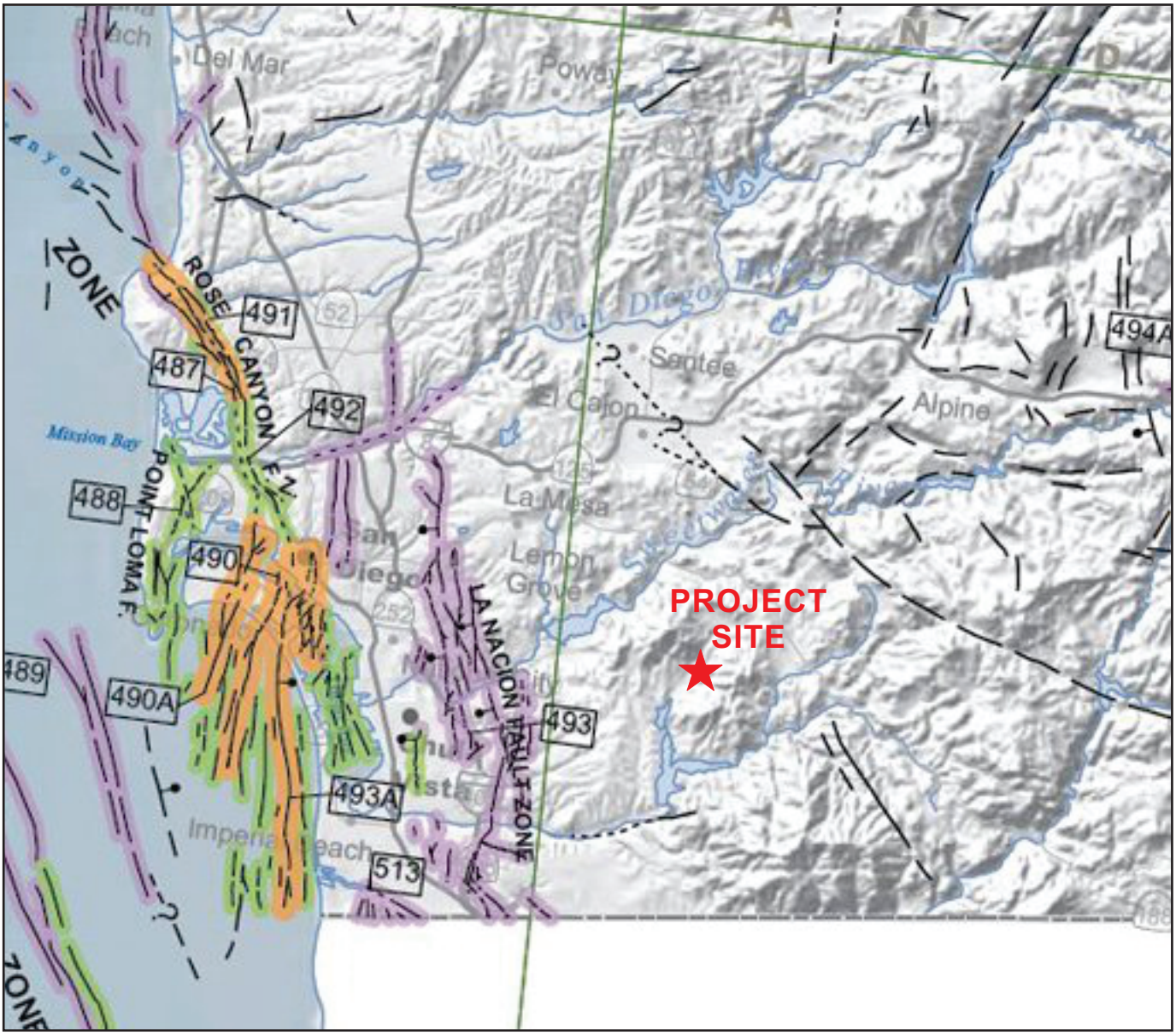

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FAULT ACTIVITY MAP OF CALIFORNIA

P/W 1312-02

FIGURE 6

SOURCE MAP - 2010, CALIFORNIA GEOLOGICAL SURVEY, 150TH ANNIVERSARY FAULT ACTIVITY MAP OF CALIFORNIA

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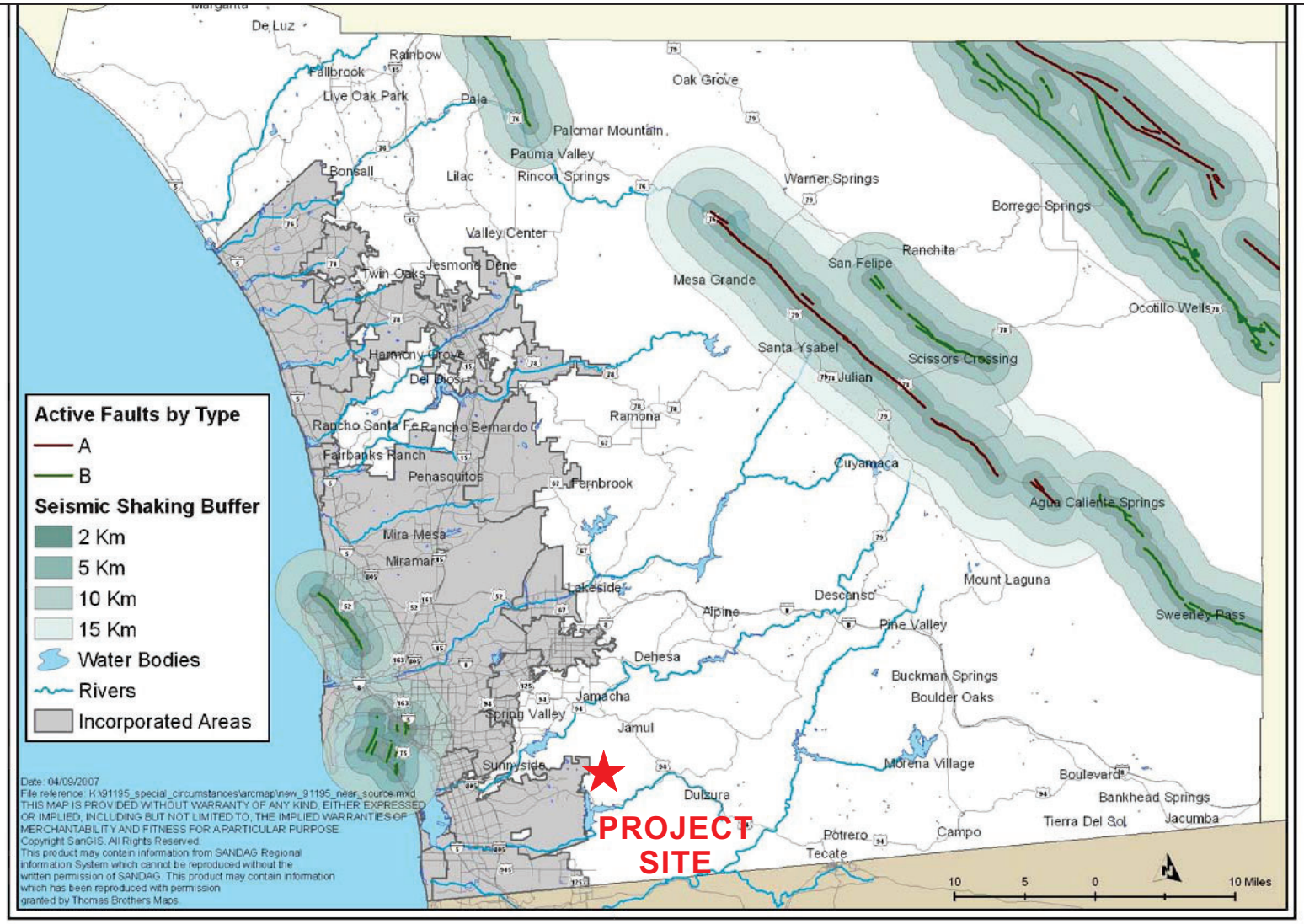

N

Guidelines for Determining Significance
Geologic Hazards

25

NEAR SOURCE SHAKING ZONES

P/W 1312-02

FIGURE 7

SOURCE MAP - COUNTY OF SAN DIEGO GUIDELINES FOR DETERMINING SIGNIFICANCE, GEOLOGIC HAZARDS, JULY 30, 2007

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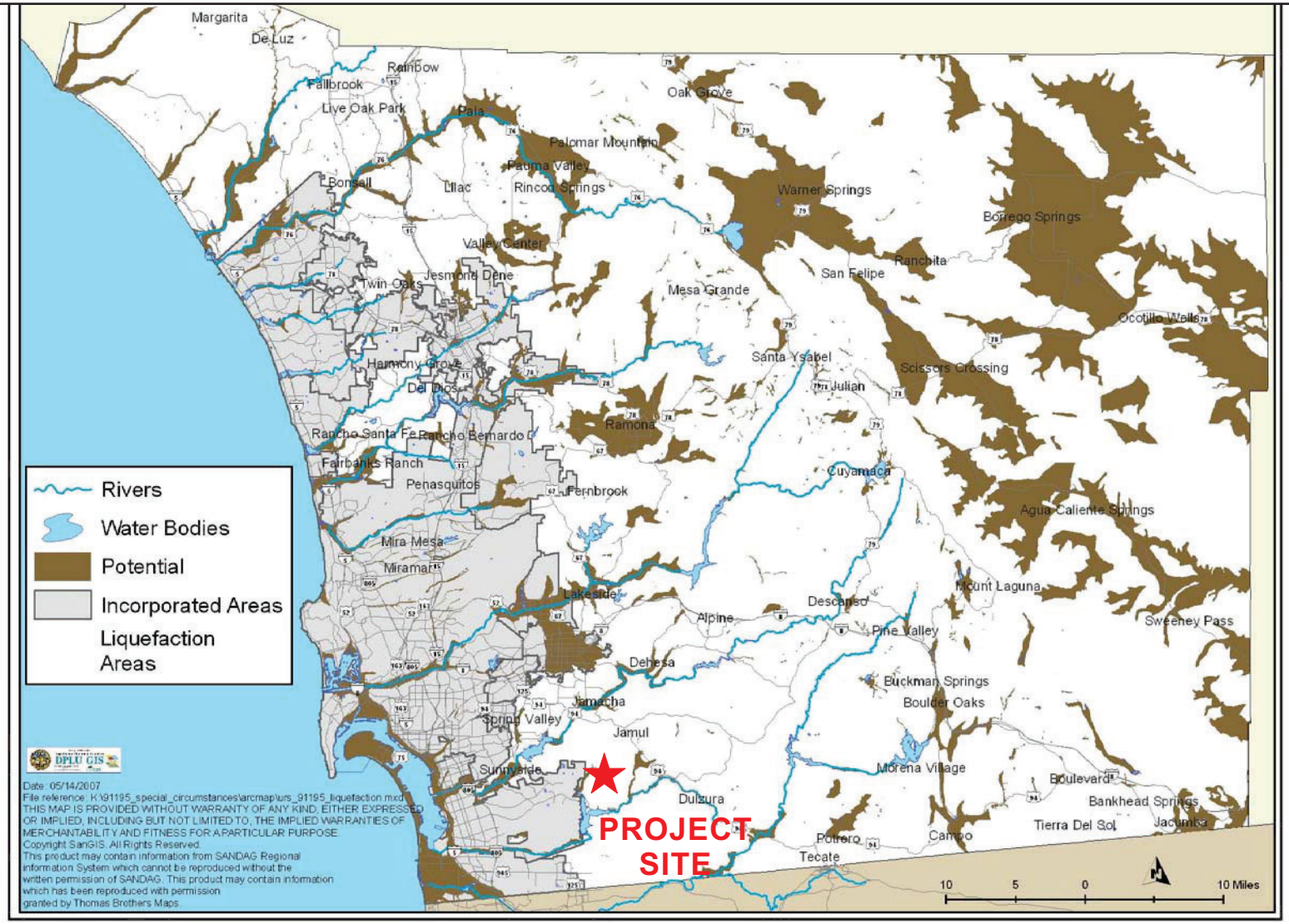

POTENTIAL LIQUEFACTION AREAS

P/W 1312-02

FIGURE 8

SOURCE MAP - COUNTY OF SAN DIEGO GUIDELINES FOR DETERMINING SIGNIFICANCE, GEOLOGIC HAZARDS, JULY 30, 2007

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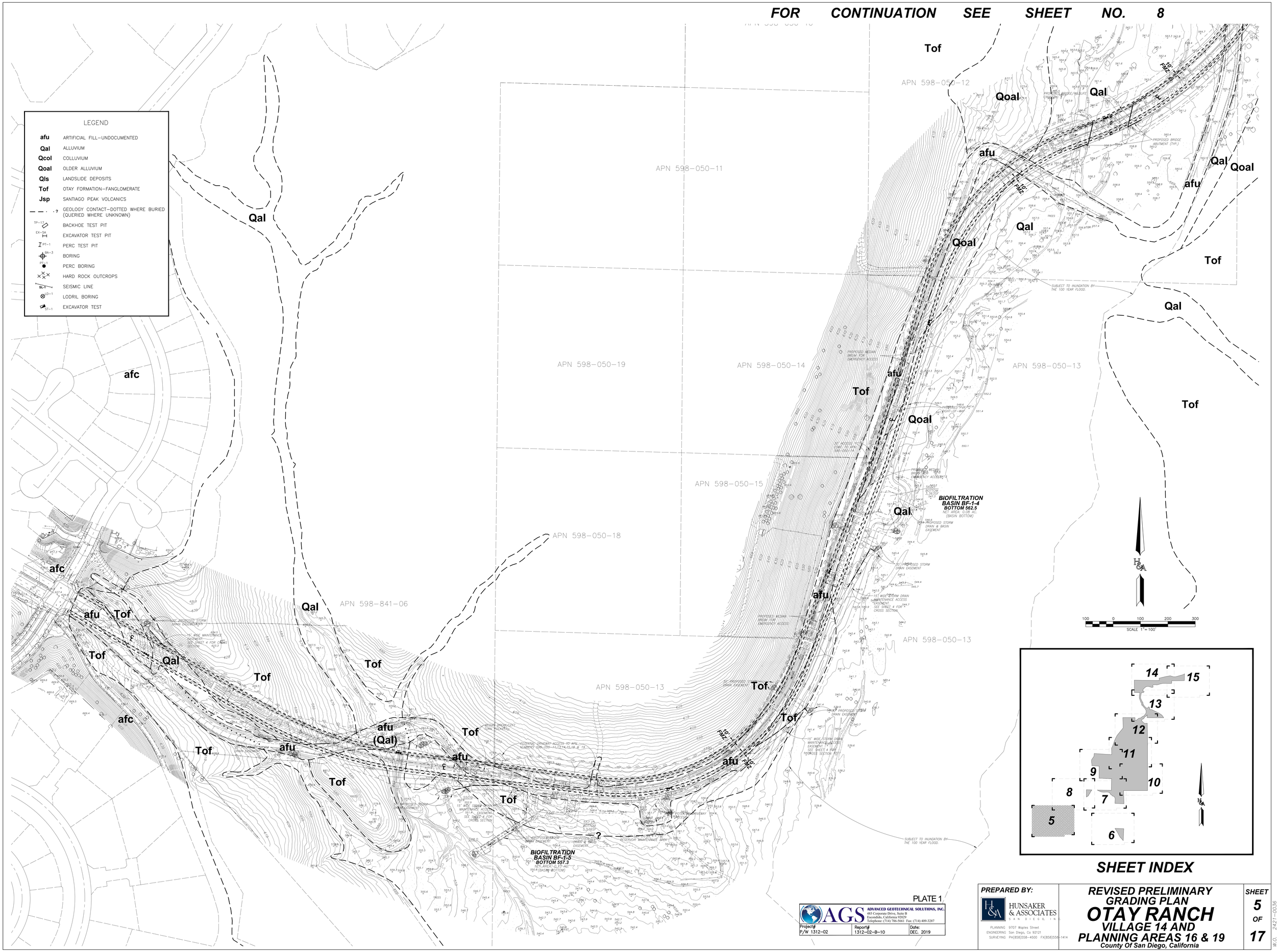

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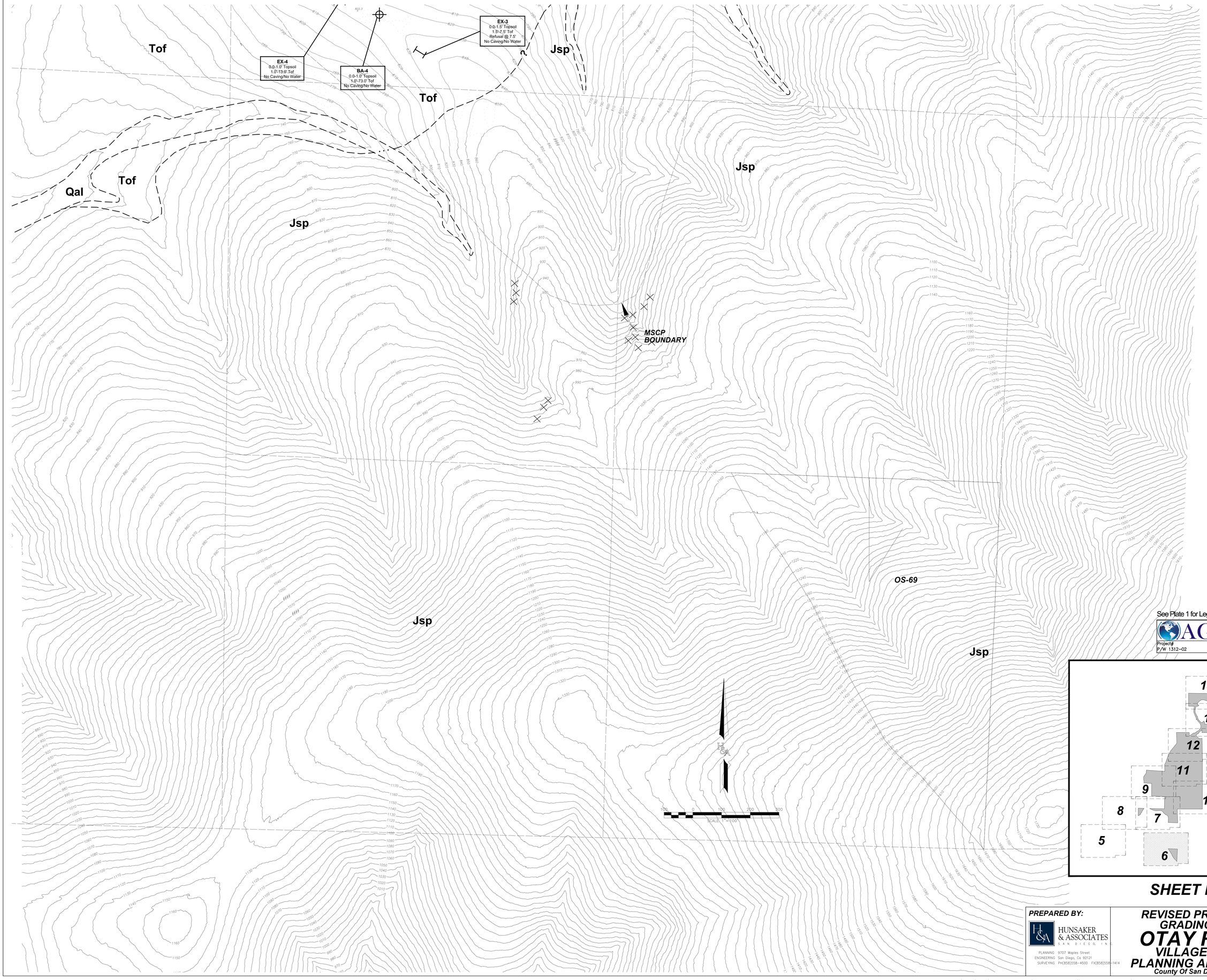

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PLATE 2

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Project#	Report#	Date:
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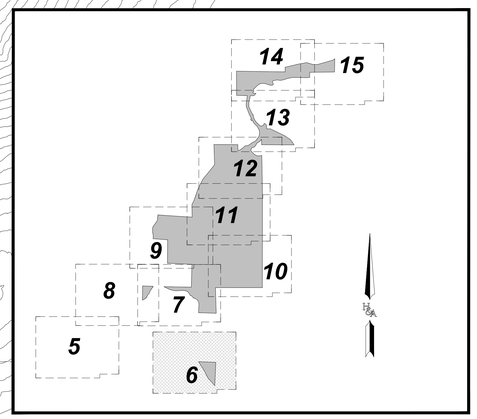

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REVISED PRELIMINARY GRADING PLAN
OTAY RANCH
VILLAGE 14 AND PLANNING AREAS 16 & 19
County Of San Diego, California

SHEET 6 OF 17

W.O. 2421-0036

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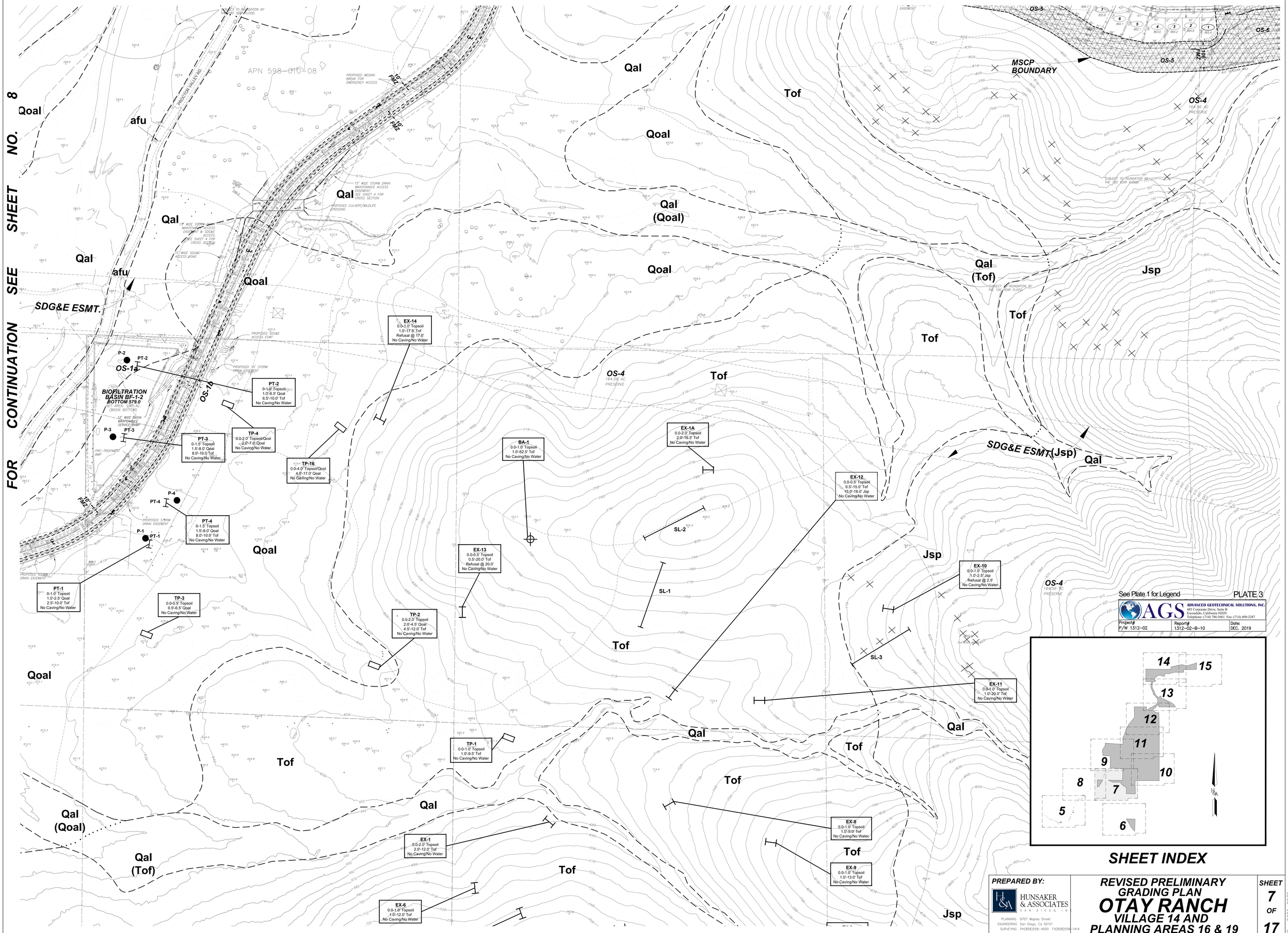

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REVISED PRELIMINARY GRADING PLAN
OTAY RANCH
VILLAGE 14 AND PLANNING AREAS 16 & 19
County Of San Diego, California

SHEET 7 OF 17

W.O. 2421-0036

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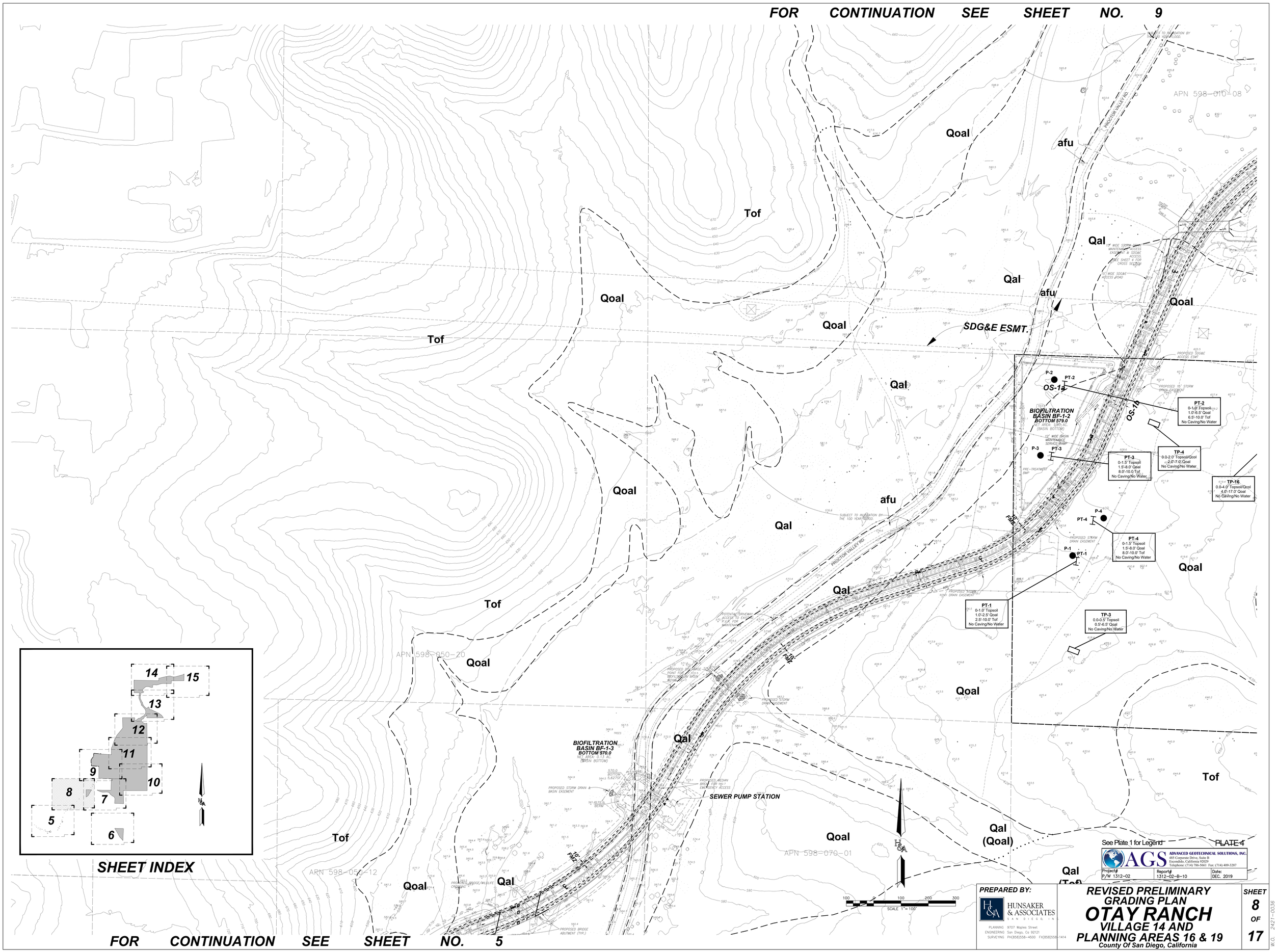

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PLATE 4

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OTAY RANCH
VILLAGE 14 AND PLANNING AREAS 16 & 19
County Of San Diego, California

SHEET 8 OF 17

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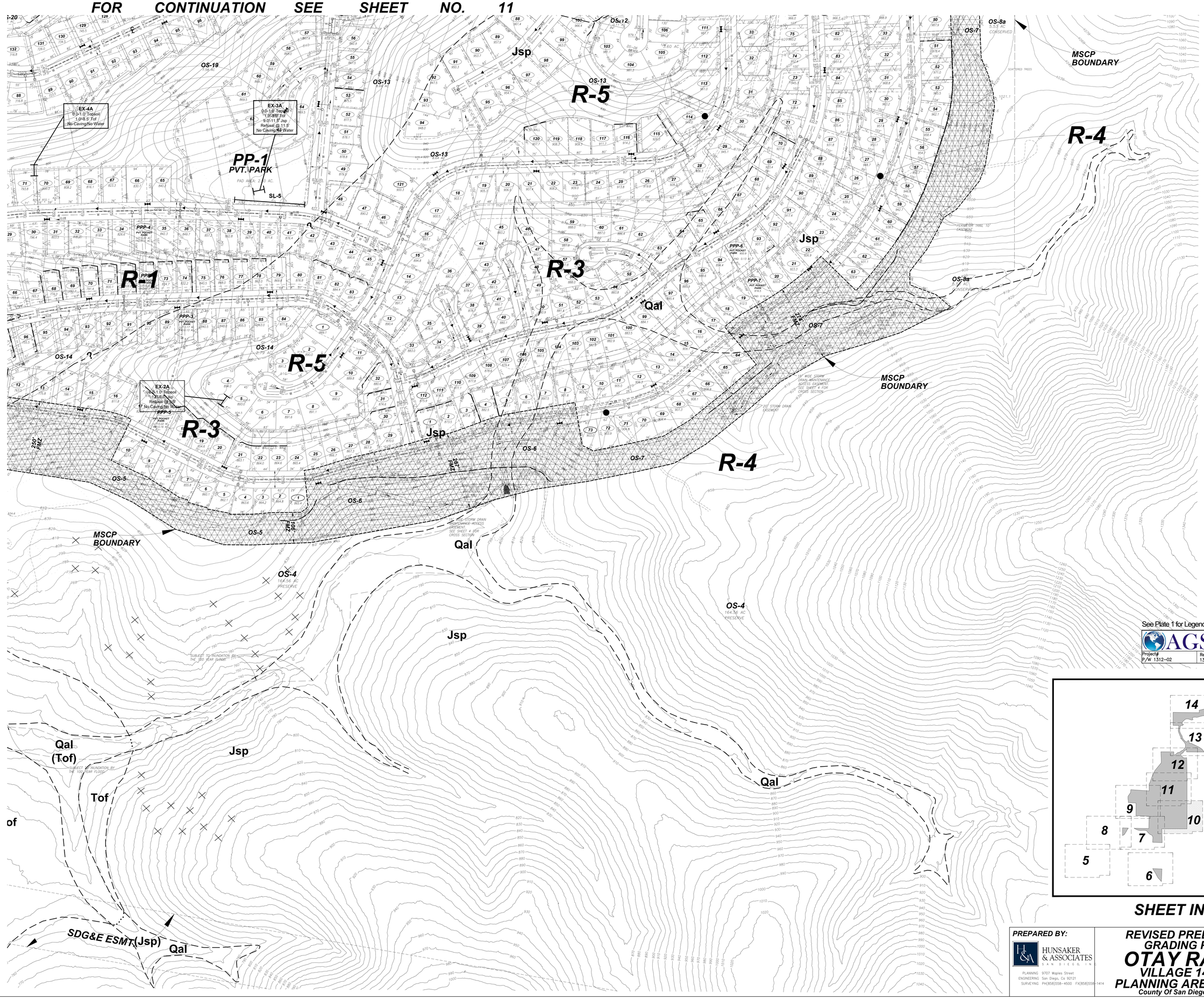

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PLATE 6

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Project#	Report#	Date:
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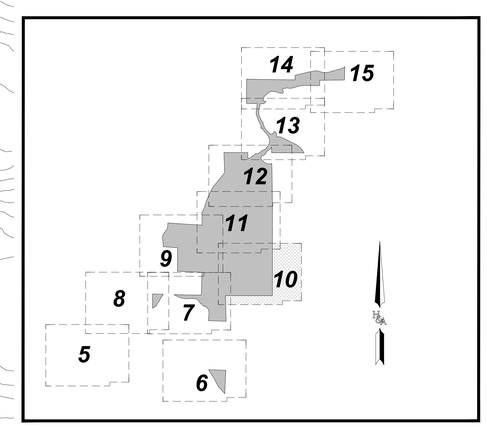

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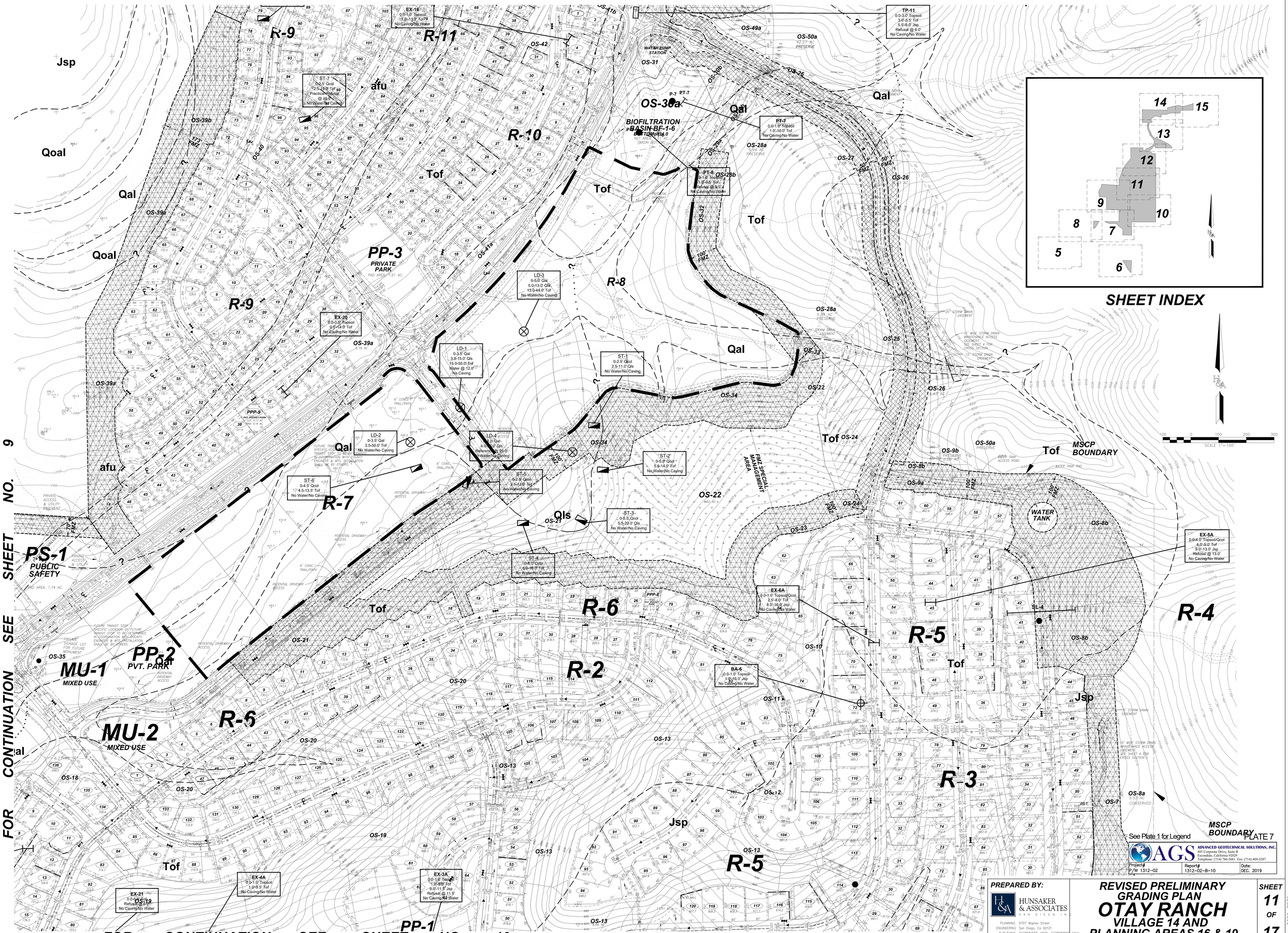

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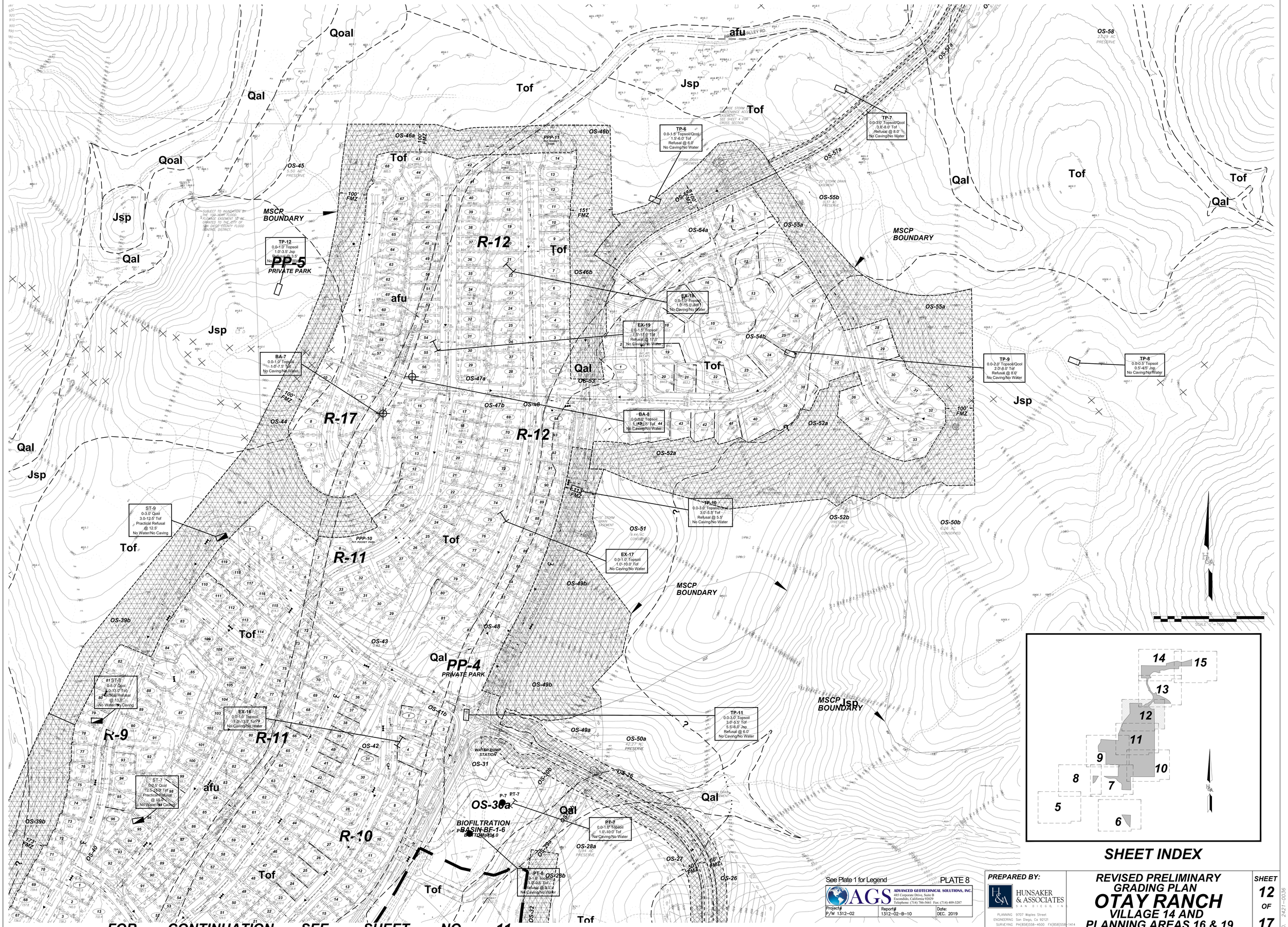

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