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GEOTECHNICAL INVESTIGATION

OTAY RANCH RESORT VILLAGE
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR

MOLLER OTAY LAKES INVESTMENT, LLC
C/O OTAY LAKES, LP
CHULA VISTA, CALIFORNIA

SEPTEMBER 5, 2014
PROJECT NO. G1012-52-01D

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Project No. G1012-52-01D
September 5, 2014

Moller Otay Lakes Investment, LLC
c/o Otay Lakes, LP
1392 East Palomar Street, Suite 202
Chula Vista, California 91913

Attention: Mr. Sean Kilkenny

Subject: OTAY RANCH RESORT VILLAGE
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Kilkenny:

In accordance with your authorization, we herein submit the results of our geotechnical investigation for the subject site. The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. The study also includes an evaluation of the geologic units, geologic hazards, and rock rippability. Based on the results of this study, it is our opinion the site is considered suitable for development provided the recommendations of this report are followed.

Should you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Otay Ranch Resort Village Area A development located in south San Diego County, California (see Vicinity Map, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, to provide recommendations pertaining to the geotechnical aspects of developing the property. The overall proposed development of Area A and B will include the construction of single-family residential neighborhoods, a mixed use residential and commercial use neighborhood, a resort hotel complex with associated ancillary facilities, an elementary school, a site for public safety facilities, open space, Preserve land, park and recreational uses, and roadway and infrastructure improvements. Plans for development of Area A are presented on the Geologic Map, Figures 2 through 4 (map pocket). We understand that the Otay Ranch Resort Village will be developed in conjunction with the widening and realignment of Otay Lakes Road located south and west of the property.

The scope of our investigation included geologic mapping, subsurface exploration, laboratory testing, engineering analyses, and the preparation of this report. As a part of our investigation, we have reviewed stereoscopic aerial photographs, published geologic maps, published geologic reports, and previous geotechnical reports related to the property. A summary of the background information reviewed for this study is presented in the *List of References*.

The field investigation performed for the 1,869 acre Otay Ranch Resort Village, which Area A is a part, included geologic mapping and the excavation of 17 large-diameter borings, 48 excavator trenches, 71 trackhoe trenches, 22 air track borings, and 18 seismic refraction survey lines. A discussion of the field investigation and logs of the large-diameter borings, excavator trenches, and trackhoe trenches are presented in Appendix A. Logs of the air-track borings are presented in Appendix B. The results of the seismic refraction surveys prepared by Southwest Geophysics are presented in Appendix C. We also used geologic information from a previous report prepared by Neblett & Associates (2004) as a part of this investigation. The boring and trench logs and seismic survey information prepared by Neblett & Associates are presented in Appendix D. The approximate locations of the exploratory excavations and seismic surveys for Area A are presented on the Geologic Map (Figures 2 through 4). A rippability study on the metavolcanic rock materials based on the exploratory excavations is presented in Appendix E. We performed laboratory tests on soil and rock samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis and to evaluate aggregate suitability. The results of the laboratory testing are presented in Appendix F. We performed engineering analyses to evaluate the stability of the proposed slopes. The results of our slope stability analyses are discussed herein and also presented in Appendix G.

Hunsaker & Associates San Diego, Inc. provided the topographic information and proposed grading and development plans used during our field investigation and preparation of the Geologic Map. References to elevations presented in this report are based on the referenced topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

The Otay Ranch Resort Village Area A is located north of Lower Otay Lake in south San Diego County. The site is located within the Proctor Valley Parcel of the County's Otay Subregional Plan, Volume 2 (Otay SRP), and is approximately one-quarter mile east of the City of Chula Vista. Access is provided via Telegraph Canyon Road, which transitions into Otay Lakes Road and forms the southern boundary of the Project site.

The Otay Ranch Resort Village Area A planning area is approximately 867-acres consisting of a broad mesa sloping to the south, broken by several steep canyons draining southward. Portions of the gently sloping mesa extend north into the Jamul Mountains, becoming part of steeper slopes. Site elevations within the area of development range from approximately 525 feet above mean sea level (MSL) at the southeastern end of the planning area to approximately 800 feet MSL in the northwestern portions. Additionally, a water tank is proposed at an elevation of 950 feet MSL. The Project area lies within the watershed of the Otay River, a westerly flowing stream that drains an area of approximately 145 square miles. The site is upstream of Savage Dam, which creates Lower Otay Lake. Site vegetation consists of native coastal sage scrub and grassland habitats disturbed by grazing. Some riparian vegetation occurs in the drainage areas of the site.

The Otay Ranch Resort Village Area A is located at the interface of urban development and scenic open space. The Otay Valley Parcel of Otay Ranch, the Eastlake Vistas residential community, the Eastlake Woods residential community, and the U.S. Olympic Training Center compose the edge of urban development to the west. Area A of the Otay Ranch Resort Village is immediately north and west of the Area B planning area. Lower Otay Lake, a recreational reservoir and water supply owned by the City of San Diego, is located further to the south. Upper Otay Lake and the Birch Family Estate are located to the northwest of Area A. A temporary ultra-light gliding and parachuting airport is located at the eastern end of Lower Otay Lake. An inactive quarry operation is located further to the east.

The land uses proposed by Otay Ranch Resort Village Area A consist of single-family residential neighborhoods, a site for public safety facilities, park and recreational uses, open space, and Preserve land. The proposed plan includes approximately 305 acres designated for 1,007 single-family detached homes. Two single-family residential neighborhoods are planned with an average density of 3.3 dwelling units per acre. The proposed plan reserves a 2.1-acre public safety site. Five parks are planned on 10 acres, ranging from 0.9 to 2.9 acres. The Otay Ranch Resort Village Area A also includes about 64 acres of open space and approximately 475 acres of Preserve land. Open space generally consists of large manufactured slopes outside of neighborhoods or brush management areas. Preserve land is usually undisturbed lands or restored habitats set aside for dedication to the Otay Ranch Preserve Owner Manager in satisfaction of Otay Ranch RMP conveyance requirements. Internal roadway circulation comprises about 12.3 acres of Area A.

Grading of Area A will consist of maximum cuts and fills of approximately 110 and 70 feet, respectively with cut and fill slopes having a maximum height of 90 and 75 feet, respectively, with a maximum slope inclination of 2:1 (horizontal to vertical). Cut slopes located along the northern perimeter of the project and some interior cut slopes excavated within metavolcanic rock will have slope inclinations of 1.5:1 (horizontal to vertical) with a maximum height of about 105 feet. The proposed grading will require approximately 13.3 million cubic yards of excavation and approximately 2.8 million cubic yards of remedial grading. Several water quality basins will also be constructed. The geotechnical recommendations for the widening and realignment of Otay Lakes Road is provided in a separate report (see List of References).

The locations and descriptions provided herein are based on a site reconnaissance, and review of the referenced plans and project information provided by the client and Hunsaker & Associates, San Diego.

3. GEOLOGIC SETTING

The site is located in the coastal plain of the Peninsular Ranges province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. Crystalline basement rocks exist along the western side of the Peninsular Ranges and are dominated by pre-batholithic andesitic Metavolcanic Rock previously known as the Santiago Peak Volcanics with a late Jurassic and early Cretaceous age. The Metavolcanic Rock was intruded during the early to mid-Cretaceous by a variety of granitic to gabbroic plutons of the Southern California batholith. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. In places, the outliers of metavolcanic and granitic rock protrude through the Tertiary sedimentary sequence to form resistant isolated hills. Geomorphically, the coastal plain is characterized by a stair stepped series of marine terraces which young to the west and have been dissected by west flowing rivers that drain the Peninsular Ranges to the east. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault zone and the active Rose Canyon Fault Zone. The Peninsular Ranges is also dissected by the Elsinore Fault zone that is associated with and sub-parallel to the San Andreas Fault zone, which is the plate boundary between the Pacific and North American Plates.

The site is located on the eastern edge of the coastal plain and is in contact with the Metavolcanic Rock of the Jamul Mountains. The Metavolcanic Rock makes up the northern, western and eastern portions of the site. Marine sedimentary units unconformably overlie the Metavolcanic Rock, make up the southern portions of the site, and consist of the Tertiary age Otay Formation and late Tertiary to Pleistocene age Fanglomerate Deposits. The Otay Formation typically consists of three lithostratagraphic members composed of a basal conglomerate member, a middle gritstone member and an upper sandstone-claystone member with a maximum reported regional thickness of roughly 400 feet. The thickness of the Otay Formation varies at the site as it is underlain by the Metavolcanic Rock but generally increases to the south. The Fanglomerate Deposits consist of conglomeratic sandstone that provides a resistant cap on the more gently sloping areas of the site. The site has been dissected by a series of south trending canyons that have locally exposed the Otay Formation on the steeper sloping areas and along the sides of the canyon drainages.

4. GEOLOGIC MATERIALS

4.1 General

During our field investigation, we encountered six surficial deposits, consisting of undocumented fill, topsoil, lacustrine deposits, alluvium, colluvium and older alluvium. Three geologic formations exist at the site consisting of Late Tertiary- to Pleistocene-age Fanglomerate Deposits, Tertiary-age Otay Formation, and the Jurassic- to Cretaceous-age Metavolcanic Rock. The lateral extent of the materials encountered is shown on the Geologic Map, Figures 2 through 4 (map pocket). Figures 5 through 7 (map pocket) present Geologic Cross-Sections providing an interpretation of the subsurface geologic conditions. The descriptions of the soil and geologic conditions are shown on the boring and trench logs located in Appendix A and described herein in order of increasing age.

4.2 Undocumented Fill (Qudf)

Undocumented fill has been placed at several locations across the site within canyon drainages for the construction of water impoundments used in previous cattle grazing operations. In general, the fill consists of loose, slightly moist to moist, silt and sand with rock fragments and cobbles. In its present condition, the fill soil is not suitable for support of additional fill or structures and remedial grading will be necessary. However, the fill is generally suitable for reuse as compacted fill, provided it is substantially free of organics and debris.

4.3 Topsoil (unmapped)

Holocene-age topsoil is present as a relatively thin veneer locally overlying formational materials across the site. The topsoil has an average thickness of approximately 3 feet and can be characterized as soft to stiff and loose to medium dense, dry to damp, dark brown, sandy clay to clayey sand with gravel and cobble. The topsoil is typically expansive and compressible. Removal of the topsoil will be necessary in areas to support proposed fill or structures. Due to the relatively thin thickness and discontinuity of these deposits, topsoil is not shown on the Geologic Map.

4.4 Lacustrine Deposits (Ql)

The areas of the Upper and Lower Otay Lakes and the canyon drainage between the two lakes are underlain by Lacustrine Deposits. These sediments, derived from the surrounding landforms, were deposited at the bottom or adjacent to the existing lakes. This soil is typically saturated and difficult to excavate for reuse as fill soil. We do not expect Lacustrine Deposits to be encountered during grading of the site since it is generally located south of Otay lakes Road or within the connecting drainage with Upper Otay Lake, outside of the planned grading areas.

4.5 Alluvium (Qal)

Holocene-age alluvium is sheet-flow or stream deposited material found within the canyon drainages and generally vary in thickness dependent upon the size of the canyon and extent of the drainage area. The alluvium within the canyon drainages is loose to medium dense and can become saturated and difficult to excavate during the rainy season. The thickness of the alluvium encountered in our exploratory excavations and in previous trenches excavated by Neblitt & Associates (2004) ranged up to approximately 10 feet and may be thicker in the larger canyon drainages. Due to the relatively unconsolidated nature of these deposits, remedial grading will be necessary in areas to receive proposed fill or structures.

4.6 Colluvium (unmapped)

Holocene-age colluvium, derived from weathering of the underlying bedrock materials at higher elevations and deposited by gravity and sheet-flow, is locally present on the side slopes of canyon drainages. The colluvium can be characterized as sandy clay and clayey sand with varying amounts of gravel and cobble. The thickness of colluvium generally ranges from approximately 2 to 7 feet, but can be thicker along the lower portions of canyons and toes of natural slopes. Due to the relatively thin thickness and discontinuity of these deposits, the colluvium is not shown on the Geologic Map. Remedial grading of the colluvium will be required in areas that will support proposed fill or structures.

4.7 Older Alluvium (Qoal)

Pleistocene-age older alluvium is located south of the roadway along a portion of the eastern limits of the project. This unit typically consists of dark to reddish brown, dense, slightly cemented, clayey sand and sandy clay with some gravel and cobble. This unit will not be encountered during site grading and we do not expect that remedial grading of the unit would be required.

4.8 Fanglomerate Deposits (Tof)

Late Tertiary- to Pleistocene-age Fanglomerate Deposits are located throughout the site and provide a resistant cap on the more gently sloping areas with an estimated maximum thickness of up to 20 to 25 feet. This unit typically consists of dense to very dense, moderately cemented, clayey to silty sandstone and occasional sandy claystone with cobbles and boulders. Subrounded to subangular cobbles and boulders up to 40 percent and generally up to 2 feet in diameter are present within the formation. We expect excavations within this unit will be very difficult and require specialized heavy-duty grading equipment and oversized cemented boulders will likely be generated during grading operations. The Fanglomerate Deposits are generally suitable for the support of proposed fill and structural loads and they are generally stable when excavated to construct cut slopes. However, this unit will not be suitable for capping lots and streets without screening of the oversize cobble and boulders.

4.9 Otay Formation (To)

Tertiary-age Otay Formation is located along most of the southern portion of the site on the steeper sloping areas and along the side slopes of canyon drainages. This unit consists of dense to very dense and hard, slightly and moderately cemented, clayey sandstone and sandy claystone with interbeds of gravel, cobble, and boulders up to 30 percent and generally with a maximum dimension of 2½ feet. In addition, localized and discontinuous layers of sheared claystone bedding are present within this unit that can create slope instability. Excavations within the unit will generally be possible with heavy-duty grading equipment with moderate to heavy effort; however, moderately cemented zones will create very difficult ripping and generate oversize cemented boulders. The Otay Formation is suitable for the support of proposed fill and structural loads. However, the cobble and boulder layers will not be suitable for capping lots and streets without screening the oversize material. The sandstone portions of this unit are generally stable when excavated to construct cut slopes. The sheared claystone layers may require slope stabilization in cut slopes, if encountered during grading operations. Slope drains may be necessary to intercept potential seepage on cut slopes created by landscape irrigation.

4.10 Metavolcanic Rock (KJmv)

Metavolcanic Rock is present on the northern, northwestern and northeastern portions of the site and is generally moderately strong to strong, highly to slightly weathered, and jointed. Highly weathered portions of the Metavolcanic Rock consist of highly expansive clay and soft rock. The highly weathered portion is generally rippable to shallow depths with heavy-duty grading equipment. The majority of this unit is moderately to slightly weathered and will generally be unrippable. Blasting will be required to excavate the rock and will generate oversize material. The Metavolcanic rock is generally suitable for the support of proposed fill and structural loads; however, the intensely weathered clayey upper portions of this unit will require remedial grading. This unit is not suitable to cap streets and lots unless properly crushed. The Metavolcanic Rock is considered stable for construction of the proposed cut slopes if free of loose rock after blasting and excavation. Areas of rock fall mitigation of adjacent open space areas will be required as discussed in the *Rock Fall Hazard* section of the report.

5. GEOLOGIC STRUCTURE

The geologic structure within the sedimentary units at the site is characterized by a gentle southwesterly dip. The contact between the sedimentary units and the underlying metavolcanic rock generally slopes down to the west and south.

The geologic structure within the portions of the metavolcanic rock not subject to intense weathering is characterized as a hard rock mass displaying a relatively consistent, northwest-southeast trending foliation with dips generally ranging between 70 and 82 degrees to the southwest and 78 to 86 degrees to the northeast. The dominant structural feature within the rock mass is jointing. Joints are surfaces, fractures or partings within a rock mass that do not show evidence of displacement. Jointing within the rock mass was formed as a result of regional tectonic stresses and joints generally have dips between 70 and 88 degrees. The joints are generally tight or filled with precipitated clay minerals and are typically 1 to 3 feet apart. The orientations of geologic structure are presented on the Geologic Map, Figures 2 through 4.

Structural orientations within the Metavolcanic Rock suggest several dominant joint sets with attitudes conducive to block or toppling failures if exposed along cut slopes. Geologic structure within the hard rock units is highly variable and should be evaluated for each proposed cut slope individually during grading operations. We used structural data obtained during our field exploration, structural data presented by Neblett & Associates (2004), and structural orientations mapped by the California Geologic Survey (2002) for the kinematic slope stability analysis presented herein.

6. GROUNDWATER

We did not encounter a static groundwater table in the exploratory excavations performed for this study. However, we did encounter seepage conditions within localized layers of the formational units and surficial deposits especially during the rainy season. It is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered. During the rainy season, perched water conditions are likely to develop within the drainage areas that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

7. GEOLOGIC HAZARDS

7.1 Seismic Hazard Analysis

It is our opinion, based on a review of published geologic maps and reports, that the site is not located on any known active, potentially active, or inactive fault traces. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Special Study Zone.

According to the computer program *EZ-FRISK (Version 7.62)*, six known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. Based on this database, the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 14 miles west of the site, are the nearest known active faults and are the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood and Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.23g, respectively. Table 7.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relation to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 7.1.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport-Inglewood	14	7.5	0.23	0.18	0.22
Rose Canyon	14	6.9	0.20	0.19	0.21
Coronado Bank	22	7.4	0.18	0.12	0.14
Palos Verde Connected	22	7.7	0.20	0.13	0.17
Elsinore	36	7.85	0.15	0.10	0.13
Earthquake Valley	40	6.8	0.09	0.06	0.05

In the event of a major earthquake on the referenced faults or other significant faults in the southern California and northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to others in the general vicinity.

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 in the analysis. Table 7.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	**Peak Ground Acceleration**		
	Boore-Atkinson, 2008 (g)	**Campbell-Bozorgnia, 2008 (g)**	**Chiou-Youngs, 2007 (g)**
2% in a 50 Year Period	0.40	0.34	0.39
5% in a 50 Year Period	0.30	0.25	0.28
10% in a 50 Year Period	0.23	0.20	0.21

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 7.1.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 7.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.21	0.23	0.27

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be performed in accordance with the 2013 California Building Code (CBC) guidelines currently adopted by the County of San Diego.

7.2 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, static groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the dense nature of proposed fill and the very dense nature of the formational materials.

7.3 Expansive Soil

The majority of the geologic units will likely possess a “very low” to “medium” expansion potential (Expansion Index of 90 or less). However, some of the geologic units may contain a “high” expansive potential (Expansion Index of 91 to 130). These units can include topsoil, colluvium, alluvium, and the claystone beds within the Otay Formation, Fanglomerate Deposits, and the highly weathered Metavolcanic Rock. We expect the proposed grading will expose claystone within cut slopes and near finish grade within lots and public rights-of-ways. Consequently, undercutting of lots, and street, curb and gutter, and sidewalk subgrade will be required where highly expansive clay is exposed or located near grade. Stability fills may also be required if sheared claystone beds are exposed in cut slopes.

7.4 Landslides

Examination of stereoscopic aerial photographs in our files, our geologic reconnaissance, and review of available geotechnical and geologic reports for the site vicinity indicate that landslides are not present at the property or at a location that could impact the site. The California Geologic Survey geologic map for the Jamul Mountain Quadrangle indicates a landslide roughly 3,000 feet to the north of the proposed development. We do not consider landsliding to be a geologic hazard to the project.

7.5 Rock Fall Hazard

Due to the steep terrain and localized areas of large boulder outcrops in the northern and eastern portions of the property, the potential hazard for future rock fall is a consideration for development. The natural slopes were evaluated for their potential rock fall impact to proposed development by performing detailed field mapping of the rock slopes. The purpose of the mapping was to categorize the risk of rock fall by assigning a risk factor of low, medium or high to the existing slopes. A low risk is defined as having no potential impact to proposed development and mitigation will not be required. A medium risk is defined as having some potential impact to proposed development and mitigation may be required. A high risk is an area that rock fall is eminent and significant mitigation will be required. The site has been classified as having both a low and medium risk. We did not observe areas that would be classified as having a high risk. A Rock Fall Hazard Map has been provided on Figure 8. The map indicates areas of development that encroach into the medium risk rock fall areas. Based on our experience with rock fall potential, mitigation measures will be required along portions of the edge of grading when cut slopes or daylight cuts encroach within the medium risk zone. Mitigation measures will not be required when fill slopes are constructed at the edge of grading that encroach into the medium risk zone as the fill slope provides a manufactured mitigation barrier to the adjacent development. The areas of potential rock fall mitigation have been plotted on the Geologic Map, Figures 2 through 4. The areas are located on the northwestern and eastern portions of the site. The mitigation may consist of the construction of a rock fall debris fence or other acceptable catchment devise at the toe of proposed cut slopes or at the edge of daylight cut or fill areas. The hard rock slopes should be evaluated by an engineering geologist during site development and final locations of the debris fences or alternative method can be provided at that time. Based upon the available information including detailed field mapping of the rock slopes and the mitigation measures suggested and included on the Geologic Map (Figures 2 through 4), it is our professional opinion that the potential risk for rockfall hazards to impact the proposed development would be less than significant once mitigation measures are implemented. With mitigation measures incorporated per our recommendations, the proposed development would be considered safe for human occupancy.

7.6 Slope Stability

We evaluated the proposed slope configurations, as depicted on the Geologic Map, to evaluate both surficial and global stability based on the current geologic information. The portions of the site planned for development are generally underlain by Quaternary-age surficial soil, Tertiary-age Otay Formation and Fanglomerate Deposits, and Jurassic- to Cretaceous-age Metavolcanic Rock. The unit most likely to be subject to slope instability is the claystone portion of the Otay Formation encountered at several locations throughout the site. The stability of graded slopes composed of

Metavolcanic Rock is highly dependent on the degree of weathering and the geologic structure of the slope face. Slope stability analyses using the two-dimensional computer program *GeoStudio2007* created by Geo-Slope International Ltd. are presented in Appendix G. The proposed slopes should be stable from shallow sloughing conditions provided the recommendations for grading and drainage are incorporated into the design and construction of the proposed slopes.

In general, it is our opinion that permanent, graded fill slopes or cut slopes excavated within the sedimentary formational materials at the site with gradients of 2:1 (horizontal to vertical) or flatter would possess Factors of Safety of 1.5 or greater. However, stability fill construction may be required during grading operations if claystone beds are encountered on proposed cut slopes. The majority of rock cut slopes should be comprised of *good quality* (Hoek and Bray, 1981), moderately strong to very strong Metavolcanic Rock. Based on the results of our slope stability analyses, slopes composed of moderately to slightly weathered rock should possess Factors of Safety of 1.5 or greater against large-scale, deep-seated slope failures at their present and proposed slope inclinations. Graded slopes in metavolcanic rock should possess Factors of Safety of 1.5 or greater at an inclination of 1.5:1 (horizontal to vertical) or flatter.

We performed kinematic analyses of the proposed 1.5:1 (horizontal to vertical) rock cut slopes along a representative geologic cross-section using structural data obtained during our field exploration, structural data presented by Neblett & Associates (2004), and structural orientations mapped by the California Geologic Survey (2002). The purpose of a kinematic analysis is to evaluate the critical discontinuities within a rock mass that may result in failures of the rock slope based on geologic structure and slope geometry. We used the computer program Rockpack III (2003) to create a stereonet of the dip vectors (dips and dip directions) of the discontinuities within the rock mass. Based on the results of the stereonet analysis, we performed Markland’s Tests for kinematically possible failures on the data set. The resulting kinematic stereonet with the Markland’s Test results are presented in Figure 9. An angle of internal friction of 20 degrees was used for the Markland’s Tests based on parameters for gouge-filled shears (Afrouz, 1992). The Markland’s Test results indicate that localized minor hazards due to wedge and toppling failures may exist along portions of the proposed slopes where discontinuities intersect the slope face. The majority of cut slopes within moderately strong to very strong metavolcanic rock should not be subject to localized failures at the proposed slope inclinations. In areas where loose or potentially hazardous rock is encountered during grading, the loose material should be scaled off the slope face to mitigate the hazard.

Because of the potential presence of adverse geologic structures, the geologic structure of permanent cut slopes composed of Metavolcanic Rock should be analyzed in detail by an engineering geologist during the grading operations. Additional recommendations for slope stabilization may be necessary if adverse geologic structure is encountered. Grading of cut and fill slopes and intermediate terrace benching should be designed in accordance with the requirements of the local building codes or the

2013 California Building Code (CBC). Mitigation of unstable cut slopes can be achieved by the use of drained stability fills and rock slope stabilization measures such as rock bolting, or rockfall protection systems.

7.7 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The County of San Diego Hazard Mitigation Plan (2004) maps zones of high risk for tsunami run-up for coastal areas throughout the county. The site is not included within one of these high-risk hazard areas. The site is approximately 11 miles from the Pacific Coast and ranges between approximately 500 feet and 900 feet above MSL. Therefore, we consider the risk associated with tsunamis to be negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The lowest elevation at the site proposed for development is located up-gradient from the reservoirs and is at least 30 feet higher than the reservoir water level. It is our opinion that the proposed site elevations are sufficient to mitigate the risk. Therefore, the potential of seiches affecting the site is considered very low.

7.8 Dam Failure Inundation

According to the County of San Diego Hazard Mitigation Plan (2004), the site is located nearly 2 miles upstream from the nearest dam considered to have a "high" relative hazard rating because of the location of the Savage Dam at the Lower Otay Reservoir. A dam is considered to have a high hazard if it stores more than 1,000 acre-feet of water, is higher than 150 feet tall, has a potential for downstream property damage, and potential for downstream evacuation. We consider the risk to the site from dam failure inundation to be negligible.

8. ROCK RIPPABILITY

8.1 Seismic Refraction Surveys

Southwest Geophysics performed a seismic refraction survey to evaluate the rippability of the Metavolcanic Rock along 18 seismic lines. The locations of the seismic traverses are presented on the Geologic Map, Figures 2 through 4 and the report is presented in Appendix C. Neblett & Associates previously performed 17 seismic surveys as part of a prior investigation for the property. The results of their previous seismic surveys are presented in Appendix D.

Based on our experience, we have summarized the estimated rippability characteristics for various excavation methods related to seismic velocity in Table 8.1. Estimates for mass grading rippability are based on using a D-9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Estimates for trenching rippability are based on using a Caterpillar 345 excavator. It is often found to be more cost effective to blast marginally rippable rock.

TABLE 8.1
SUMMARY OF ESTIMATED RIPPABILITY FROM SEISMIC REFRACTION

Excavation Method	Seismic Velocity (ft/s)	Estimated Rippability
Mass Grading	Less than 4,000	Rippable
Mass Grading	4,000 to 5,500	Marginal Ripping (Possible Blasting)
Mass Grading	Greater than 5,500	Non-Rippable (Pre-Blasting Required)
Trenching	Less than 3,500	Rippable
Trenching	3,500 to 4,000	Marginal Ripping
Trenching	Greater than 4,000	Non-Rippable

The results of the seismic refraction surveys indicate that velocities less than approximately 3,000 ft/s are likely associated with surficial soil and highly weathered rock. Velocities between 3,000 and 5,000 ft/s are likely associated with sedimentary units and moderately weathered rock. Velocities between 5,000 and 7,000 ft/s are likely associated with slightly weathered rock, with higher velocities associated with unweathered rock. Rippability is highly dependent upon the degree of weathering, fracturing, and jointing within the rock mass and the rippability of the various soil and rock units is, correspondingly, variable. Appendix E presents a summary of the seismic refraction surveys with rippable and non-rippable depths estimated using approximately 5,500 ft/s as a cut-off between rippable (includes marginally rippable) and non-rippable for the Metavolcanic Rock during grading operations.

8.2 Air-Track Borings

The scope of our services included drilling hydraulic rotary percussion borings (generically referenced herein as air-track borings) in the areas where significant excavations within rock are proposed. In addition, air-track borings were used to assist in delineating the subsurface contact between the Metavolcanic Rock and the overlying Fanglomerate Deposits containing rock clasts. The borings were advanced using an ECM-370 drill rig equipped with a 4-inch drill bit. A total of 22 borings were drilled during this study and 10 air track borings were previously drilled during the exploration by Neblett & Associates (2004). The Air-Track Boring Logs performed by Geocon Incorporated and Neblett & Associates are presented in Appendices B and D, respectively. The approximate locations of the air-track borings are shown on the Geologic Map, Figures 2 through 4 (map pocket). Air-track drill penetration rates can be used to aid in evaluating rock rippability and to estimate the depth at which excavation difficulty will occur. Table 8.2 presents a summary of the estimated rippability of rock based on air-track drill penetration rates as well as developed from our experience with grading operations in rock materials. These general guidelines are typically based on drill rates using a hydraulic rotary percussion drill rig similar to an IR-360 with a 3.5-inch drill bit and a Caterpillar D-9 (or equivalent) bulldozer.

TABLE 8.2
SUMMARY OF ESTIMATED RIPPABILITY FROM AIR-TRACK BORINGS

Drill Penetration Rate (seconds/foot)	Estimated Rippability
0 to 20	Rippable
20 to 30	Marginal Ripping (Possible Blasting)
Greater than 30	Non-Rippable (Blasting Likely)

The penetration rates (recorded in seconds per foot) for each air track boring are presented in Appendix B, Figures B-1 through B-22. The geologic material(s) encountered and the estimated thickness of rippable material for each air-track boring in rock using 20 seconds per foot as the boundary between rippable and non-rippable rock is presented on Table E-I in Appendix E and on the Geologic Map, Figures 2 through 4. The estimate is derived from a literal interpretation of the penetration rate from each air-track boring. Prospective contractors formulating construction cost estimates should consider their experience with productive and non-productive ripping when evaluating the penetration rate boundary between rippable and non-rippable rock.

8.3 Rippability of Fanglomerate Deposits

We excavated exploratory trenches and air track borings, and performed seismic traverses in proposed cut areas to evaluate this units rippability characteristics. In general, the Fanglomerate Deposits are rippable to the proposed cut depths; however, the presence of large boulders and strong cementation may require the use of specialized rock-breaking equipment and will likely generate oversize material. We encountered refusal during our investigation using a Komatsu PC 400 excavator when excavations extended to depths ranging up to 18 feet into the Fanglomerate Deposits as shown on the excavator trench logs and in several large diameter borings and trackhoe trenches (see Appendix A).

8.4 Rippability of Metavolcanic Rock

We excavated exploratory trenches and air track borings, and performed seismic traverses in the Metavolcanic Rock unit (KJmv), located generally in proposed cut areas, to evaluate rock rippability characteristics. The rippability of this rock unit is variable, and is generally limited to the depth of the weathered mantle. Proposed excavations within the Metamorphic Rock will require very difficult ripping and blasting as excavations extend beyond the rippable weathered mantle. Based on an air-track penetration rate of 20 spf and seismic shear wave velocities of 5,500 ft/s, we estimate the thickness of the rippable rock mantle to vary between 1 foot to 26 feet. Estimates for the rippable depth at each of the exploratory excavation and seismic line locations, or an average if multiple data was obtained, within the Metavolcanic Rock are presented on Table E-I in Appendix E and on the Geologic Map, Figures 2 through 4.

Heavy ripping and/or blasting should also be expected in areas of concentrated surface rock outcroppings. Estimates of the expected volume of hard rock materials generated from proposed excavations should be evaluated based on the information from each air-track boring and drill penetration rate criteria acceptable to the contractor. Roadway/utility corridor and lot undercutting criteria should also be considered when calculating the volume of hard rock. Proposed cuts in hard rock areas can be expected to generate oversized fragments (rocks greater than 12 inches in dimension) which will necessitate typical rock handling and placement procedures during grading operations. The grading contractor may want to perform an additional investigation to observe the rippability characteristics for estimating purposes.

8.5 Capping Material

Capping material refers to select material placed within 3 feet from building pad grade, 8 feet from roadway grade, and to at least 2 feet below the deepest utility within roadways. The capping material should consist of “soil” fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent soil passing the ¾-inch sieve. In addition, the upper 3 feet of pad grade should have at least 20 percent of the soil passing the No. 4 screen. In general, capping material can be readily obtained from localized areas within the surficial units and from the sandstone layers of the Otay Formation. Screening will be necessary to remove cobbles and boulders larger than 6 inches in diameter within the cobbly portions of the surficial units, the Otay Formation, and within the Fanglomerate Deposits. High expansive material derived from the claystone portions of the Otay Formation and Fanglomerate Deposits and the highly weathered clay of the Metavolcanic Rock should not be used as capping material. The availability of capping material easily obtained from the Metavolcanic Rock is considerably less and screening and crushing operations will likely be necessary to generate capping material within this unit. Estimates for the thicknesses of capping material at each of the exploratory excavation and seismic line locations within the Metavolcanic Rock are presented in Appendix E and on the Geologic Map, Figures 2 through 4. It may be cost effective to over excavate the proposed cut and fill areas within the sandstone portions of the Otay Formation to generate capping material and to provide additional areas for disposal of oversize rock material.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 It is our opinion that no soil or geologic conditions were encountered during the investigation that would preclude the proposed development of the Otay Ranch Resort Village Area A project provided the recommendations presented herein are followed and implemented during construction.

9.1.2 Potential geologic hazards at the site include rock fall, seismic shaking, and expansive and compressible soil. Based on our investigation and available geologic information, active or potentially active faults are not present underlying or trending toward the site.

9.1.3 The existing onsite surficial soil units including undocumented fill, topsoil, colluvium, alluvium, and highly weathered formational materials are potentially compressible and unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of the surficial soil and highly weathered formational materials will be required and recommendations for remedial grading are provided herein. The rock units and dense portions of the Otay Formation and Fanglomerate Deposits are suitable for the support of proposed fill and structural loads.

9.1.4 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, seepage within formational materials and perched groundwater conditions within the canyon drainages may be encountered during the grading operations, especially during the rainy seasons.

9.1.5 The rippability of the surficial units is expected to range from easy to moderate. We expect the Otay Formation and Fanglomerate Deposits to be rippable with moderate to heavy effort to proposed finish grades. Cobbles, boulders, and cemented zones should be expected within portions of the Otay Formation and Fanglomerate Deposits. These will generate oversized material during grading, and screening and crushing should be expected if used as capping material. The rippability of the Metavolcanic Rock is variable and ranges between moderate to very difficult with refusal expected. Rock breaking and blasting should be expected during grading within the Metavolcanic Rock.

9.1.6 In general, cut slopes composed of Otay Formation and/or Fanglomerate Deposits should possess Factors of Safety at least 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter. Cut slopes composed of metavolcanic rock should possess Factors of Safety greater than 1.5 at inclinations of 1.5:1 (horizontal to vertical), or flatter. The geologic structure of cut slopes composed of hard rock should be evaluated during grading operations by the geotechnical consultant.

9.1.7 Proposed cut slopes that expose sheared claystone within the formational units may require slope stabilization. Recommendations for stabilization of potentially unstable slopes consisting of stability fills are discussed herein.

9.1.8 The proposed residential and commercial structures and site retaining walls may be supported on conventional foundations bearing in either competent formational materials or properly compacted fill. Geocon Incorporated should evaluate the foundation systems when the locations of these structures have been finalized. Structures proposed in the resort site may have large fill differentials that would require special foundation considerations. Transitioning foundations and slabs from bedrock to compacted fill should not occur. Where fill will be utilized for foundation support, the foundation system for the entire structure should be underlain by properly compacted fill. Bedrock over-excavations will be required where engineered fill is to be utilized for foundation support. General recommendations for the design of shallow foundations are provided herein.

9.1.9 Due to the existence of hard rock at or near the proposed grades at many locations throughout the site, the building pads, streets, and utility corridors underlain by Metavolcanic Rock should be over excavated to facilitate future excavation of footings, subgrade, and utility trenches. Recommendations for overexcavation operations are provided herein.

9.1.10 Due to the potential for rock fall to impact portions of the development, rock fall debris fences or other acceptable mitigation methods will be required at the toe of cut slopes or at the edge of daylight cuts and fills that encroach into the medium risk hazard zone.

9.1.11 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

9.1.12 We understand that improvements to Otay Lakes Road, south and west of the property are also planned. A separate Geotechnical Investigation report for the roadway widening and realignment has been prepared as listed in the List of References.

9.2 Soil Characteristics

9.2.1 The soil encountered in the field investigation is considered to be “expansive” (Expansion Index [EI] greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 9.2.1 presents soil classifications based on the expansion index.

TABLE 9.2.1
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

9.2.2 Based on laboratory tests of representative samples of the materials expected at proposed grades presented in Appendix F (Table F-III), the on-site material is expected to possess a “very low” to “high” expansion potential (Expansion Index of 130 or less). We expect the surficial soil and the claystone portions of the Otay Formation and Fanglomerate Deposits and highly weathered clay of the Metavolcanic Rock will likely possess a “medium” to “high” expansion potential (Expansion Index of 51 to 130). The sandstone portions of the Otay Formation and Fanglomerate Deposits will likely possess a “very low” to “low” expansion potential (Expansion Index of 50 or less). Additional testing for expansion potential should be performed once final grades are achieved.

9.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix F (Table F-V) and indicate that the on-site materials at the locations tested possess “not applicable” (S0) to “severe” (S2) sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. Table 9.2.2 presents a summary of concrete requirements set forth by 2013 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 9.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	SO_4<0.10	--	--	2,500
Moderate	S1	0.10≤SO_4<0.20	II	0.50	4,000
Severe	S2	0.20≤SO_4≤2.00	V	0.45	4,500
Very Severe	S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

9.2.4 We performed laboratory tests on a sample of the site materials encountered to check the corrosion potential to subsurface metal structures. We performed the laboratory tests in accordance with California Test Method No. 643. In addition, we performed a laboratory test to check the water-soluble chloride ion content in accordance with AASHTO Test No. T 291. The laboratory test results are presented in Appendix F (Tables VI and VII).

9.2.5 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

9.3 Seismic Design Criteria

9.3.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 9.3.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The structures should be designed using Site Class C where there is less than 20 feet of fill and Site Class D where the fill thickness is 20 feet or greater. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.7.1 are for the risk-targeted maximum considered earthquake (MCE_R). We will evaluate the structure site class for each building once the final grading has been completed.

TABLE 9.3.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2013 CBC Reference
Site Class	C	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.810g	0.810g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.315g	0.315g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.076	1.176	Table 1613.3.3(1)
Site Coefficient, F_V	1.485	1.771	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.872g	0.953g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.467g	0.557g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.581g	0.635g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.312g	0.371g	Section 1613.3.4 (Eqn 16-40)

9.3.2 Table 9.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 9.3.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value		ASCE 7-10 Reference
Site Class	C	D	Section 1613.3.2 (2013 CBC)
Mapped MCE_G Peak Ground Acceleration, PGA	0.309g	0.309g	Figure 22-7
Site Coefficient, F_{PGA}	1.091	1.191	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.337g	0.368g	Section 11.8.3 (Eqn 11.8-1)

9.3.3 Conformance to the criteria in Tables 9.3.1 and 9.3.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

9.4 Temporary Excavations

9.4.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

9.4.2 Temporary slopes should be made in conformance with OSHA requirements. The compacted fill and surficial soils should be considered a Type B soil (Type C if seepage is encountered), and the formational materials should be considered a Type A soil (Type B soil if seepage or groundwater is encountered) in accordance with OSHA requirements. In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

9.5 Slope Stability Analyses

9.5.1 We performed the slope stability analyses using the computer software program *GeoStudio 2007*, to calculate the factor of safety with respect to deep-seated stability. This program uses conventional slope stability equations and a two-dimensional, limit-equilibrium method. We performed the rotational-mode and block-mode analyses using Spencer's method. Output of the computer program including the calculated Factor of Safety and the failure surface is shown on Figures G-1 – G-4 in Appendix G.

9.5.2 Slope stability analyses utilizing average drained direct shear strength parameters based on laboratory tests and our experience with similar soil types in nearby areas indicates that the proposed cut and fill slopes, constructed of on-site materials, should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions if the recommendations of this report are followed. The shear strength parameters used in the slope stability analyses are presented on Table G-1 in Appendix G.

9.5.3 We selected Cross-Sections B1-B1′, C1-C1′, D-D′, and P-P′ (see Figures 5 through 7) to perform the slope stability analyses. The results and the computer output of the analyses are presented in Appendix G. Table G-II provides a description of the cross-sections, their corresponding factor of safety, and the condition of the slope stability analyses. A factor of safety of 1.5 for static conditions is currently required by the County of San Diego for permanent graded slopes.

9.5.4 If claystone layers are exposed on cut slopes within the Otay Formation or Fanglomerate Deposits, the construction of a stability fill may be required to stabilize the slope face. A typical stability fill detail has been provided on Figure 10.

9.5.5 We performed surficial slope stability calculations for a 2:1 (horizontal to vertical) slope in compacted fill. The calculated factor of safety is greater than the required minimum factor of safety of 1.5. Plants with variable root depth should be planted as soon as practical once the fill slopes have been constructed. Surficial slope stability calculations are presented on Figure G-5 in Appendix G.

9.5.6 Stability fill drains should be surveyed during construction and depicted on the As-Graded Geologic Map in the final report of grading.

9.5.7 Excavations including cut slopes, shear keys and stability fills should be observed during grading by an engineering geologist to evaluate whether soil and geologic conditions do not differ significantly from those expected.

9.6 Grading

9.6.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix H and the County of San Diego Grading Ordinance.

9.6.2 Prior to commencing grading, a preconstruction conference should be held at the site with the county inspector, owner or developer, grading contractor, civil engineer, environmental consultant, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

9.6.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

9.6.4 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be filled with properly compacted material as part of the remedial grading.

9.6.5 Topsoil, colluvium, alluvium, undocumented fill, and highly weathered or decomposed Otay Formation, Fanglomerate Deposits and Metavolcanic Rock within the limits of grading should be removed to expose firm formational materials. The actual depth of removal should be evaluated by the geotechnical engineering consultant during the grading operations. The bottom of the excavations should be scarified to a depth of at least 12 inches (where possible), moisture conditioned as necessary, and properly compacted. Excavated soil with an Expansion Index between 51 and 90 should be kept at least 3 feet below finish grade. Soil with an Expansion Index greater than 90 should be kept at least 5 feet below finish grade. Sheet-graded pads should be capped with at least 6 feet of fill soil that possesses a “very low” to “low” expansion potential (EI of 50 or less) to accommodate future pad regrading.

9.6.6 If perched groundwater is encountered during remedial grading within the surficial soil, top loading of wet material may be required. This condition may potentially occur within the canyon drainages, especially during the rainy season. The excavated materials should then be moisture conditioned as necessary to near optimum moisture content prior to placement as compacted fill.

9.6.7 The geotechnical engineering consultant should observe the removal bottoms to check the exposure of the formational materials. Deeper excavations may be required if highly weathered formational materials are present at the base of the removals.

9.6.8 The site should be brought to final finish grade elevations with structural fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. Fill placed in excess of 50 feet from finish grade should be compacted to a dry density of at least 92 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

9.6.9 To reduce the potential for differential settlement, the building pads with cut-fill transitions should be undercut at least 3 feet, sloped 1 percent to the adjacent street or deepest fill, and replaced with properly compacted fill with a “very low” to “low” expansion potential (EI of 50 or less). Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut for cut-fill transition lots, should be increased to one-fifth of the maximum fill thickness to a maximum depth of 10 feet. The large sheet-graded pads should be undercut at least 6 feet to allow for future re-grading of the pads.

9.6.10 Building pads underlain by hard rock units at grade should also be undercut to facilitate future trenching. Building pads that expose hard rock should be undercut a minimum of 3 feet and replaced with properly compacted fill and the base of the undercut should be sloped a minimum of 1 percent toward the adjacent street. The sheet graded pads that expose hard rock should be undercut at least 6 feet and replaced with compacted fill to facilitate future grading and utility excavation.

9.6.11 Roadways underlain by hard rock should be undercut a minimum of 8 feet for the areas inside of the public right-of-way (including joint utility structures and sidewalk areas). The undercut zone should include the areas at least 1 foot below the lowest utility or drain line. Figure 11 presents a typical detail for the overexcavation of streets.

9.6.12 Recommendations for the handling and disposal of oversized rock in fill areas are presented in Figure 12 and in Appendix H. In general, structural fill placed and compacted at the site should consist of material that can be classified into four zones:

Zone A: Material placed within 3 feet from building pad grade, 8 feet from roadway grade, and to at least 1 foot below the deepest utility within roadways should consist of "soil" fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent of the soil passing the ¾-inch sieve. In addition, the upper 3 feet of pad grade should have at least 20 percent of the soil passing the No. 4 sieve.

Zone B: Material placed below 8 feet from grade (below *Zone A* and *C*) may consist of "rock" fill or "soil/rock" fill (as defined in Appendix H). Blasted rock should generally consist of 2 foot minus rock material with occasional rock up to 4 foot in maximum dimension. Alternatively, "soil" fill may be placed in Zone B containing rock with a maximum dimension of 2 feet. Rocks up to 4 feet in maximum dimension can be individually placed in a properly compacted soil matrix with rocks separated at least 8 feet apart.

Zone C: Within 3 to 8 feet of pad grade and between 5 and 15 feet from face of slope, fill material should consist of "soil" fill with an approximate maximum particle dimension of 1 foot. Rocks up to 2 feet in maximum dimension may be placed, provided they are distributed in a matrix of compacted "soil" fill.

Zone D: Within the outer 5 feet of fill slopes, the fill should consist of rock up to 1 foot in maximum dimension in a matrix of compacted "soil" fill.

9.6.13 Import fill (if necessary) should consist of granular materials with a "very low" to "low" expansion potential (EI of 50 or less) generally free of deleterious material and rock fragments larger than 6 inches in maximum size if used for capping and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

9.6.14 Cut slopes located within weak and/or sheared claystone beds may require stability fills as evaluated by the engineering geologist. In addition, cut slopes exposing cohesionless surficial deposits or rock slopes with unfavorable geologic structure may require stability fills. In general, the Typical Stability Fill Detail presented on Figure 10 should be used for design and construction of stability fills, where required. The backcut for the stability fills should commence at least 10 feet from the top of the proposed finish-graded slope and should extend at least 3 feet into formational materials. For slopes that exceed 30 feet in height, the inclination of the backcut may be flattened as determined by the engineering geologist during grading operations.

9.6.15 Cut slope excavations including fill slope shear keys should be observed during grading operations to check that soil and geologic conditions do not differ significantly from those expected. Cut slopes excavated in Metavolcanic Rock will need to be scaled of loose rock.

9.6.16 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soil with an Expansion Index of 90 or less and at least 35 percent sand-size particles should be acceptable as "soil" fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. The use of cohesionless soil in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet to maintain the moisture content of the fill. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished slope.

9.6.17 Placement of rock fills should be planned in the deeper fill areas to facilitate rock disposal. Overexcavation of fill areas may be required to accommodate the necessary rock volumes generated during blasting. Capping material used for placement near finish grade within roadways, building pads, and slope zones should be stockpiled during excavation and remedial grading operations. Overexcavation of units that generate capping material may be necessary to achieve sufficient volumes to achieve finish grade.

9.6.18 Rock fill placement should be performed in accordance with the Recommended Grading Specifications provided in Appendix H. Blasting of rock material should be performed to maximize rock breakage to 2-foot minus material. Rock fill placement should generally be limited to 2-foot-thick horizontal layers and compacted using rock trucks and bulldozers. Significant volumes of water are typically required during rock fill placement. The downstream areas can generate large volumes of water that can be re-used during construction.

9.6.19 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, the slopes should be drained and properly maintained to reduce erosion.

9.7 Earthwork Grading Factors

9.7.1 Estimates of bulking and shrinkage factors are based on empirical judgments comparing the material in its natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Bulking of rock units is a function of rock density, structure, overburden pressure, and the physical behavior of blasted material. Based on our experience, the shrinkage and bulking factors presented in Table 9.6 can be used as a basis for estimating how much the on-site soil may shrink or swell (bulk) when excavated from their natural state and placed as compacted fill. Please note that these estimates are for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area that can also accommodate rock should be provided to accommodate these variations.

TABLE 9.6
SHRINKAGE AND BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Undocumented Fill (Qudf)	10-15% shrink
Topsoil (unmapped)	10-15% shrink
Alluvium (Qal)	10-15% shrink
Colluvium (unmapped)	10-15% shrink
Fanglomerate Deposits (Tof)	4-8% bulk
Otay Formation (To)	4-8% bulk
Metavolcanic Rock (KJmv) – rippable	10-15% bulk
Metavolcanic Rock (KJmv) – blasted	20-30% bulk
Metavolcanic Rock (KJmv) – crushed to Class 2 aggregate base	20-30% bulk

9.8 On-Site Construction Material Resources

9.8.1 We understand that the development plans may include processing the on-site rock and cobble materials to manufacture construction materials including aggregate road base and crushed aggregate rock. Major deposits of aggregate-quality Metavolcanic Rock and cobble within the Fanglomerate Deposits and to a lesser extent within the Otay Formation exist at the site.

9.8.2 We performed laboratory testing on samples of the on-site rock and cobble materials to evaluate the suitability for reuse as construction materials including aggregate base and crushed rock. We performed laboratory testing including Apparent Specific Gravity, Absorption, and Density (ASTM C128); Durability Index (California Test 229); and L.A. Abrasion (ASTM C131) on samples of Metavolcanic Rock (KJmv) and cobble with the Fanglomerate Deposits. The results of our laboratory tests are presented on Table IX in Appendix F and indicate that the tested rock materials generally meet the *Standard Specifications for Public Works Construction (Greenbook)* or *Standard Specifications for the State of California Department of Transportation (Caltrans)* criteria for aggregate quality. The spatial distribution of the geologic units is presented on the Geologic Map, Figures 2 through 4.

9.9 Subdrains

9.9.1 Conditions encountered prior to and during grading do not necessarily reveal the conditions that will be encountered once construction of the proposed homes is completed. Specifically, irrigation in both the subject lots and up gradient lots cannot be reasonably predicted. Therefore the design and implementation of additional drainage mechanisms may be necessitated. The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to groundwater seepage. The use of canyon subdrains will be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Figure 13 depicts a typical canyon subdrain detail. Subdrains with lengths in excess of 750 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes for that portion over 750 feet. Subdrains less than 750 feet in length should use 6-inch-diameter pipes. Drains within stability fill keyways should use 4-inch-diameter pipes (see Figure 10). The locations of proposed canyon subdrains are presented on the Geologic Map, Figures 2 through 4, and should be shown on the final 40-scale grading plans. The actual subdrain locations will be determined in the field subsequent to the remedial grading. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

9.9.2 Rock fill areas should be provided with subdrains along their down-slope perimeters to mitigate the potential buildup of water derived from construction or landscape irrigation. Subdrains within and/or at the base of rock fill areas as determined by the engineering geologist should use 6-inch-diameter pipes. Rock fill drains should be constructed using the same requirements as canyon subdrains as shown on Figure 13.

9.9.3 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/perforated interface, a seepage cutoff wall should be constructed on the downslope side of the junction in accordance with Figure 14. Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure in accordance with Figure 15.

9.9.4 Building pad areas adjacent to large ascending slopes may experience wet to saturated soil conditions due to water migration or seepage. To reduce the potential for this to occur, consideration should be given to placing a subdrain along the base of the slopes to collect potential seepage and convey it to a suitable outlet. The drain should be sufficiently deep to intercept the seepage (on the order of 3 feet below finish grade) and constructed in accordance with the recommendations in the subdrain section of this report. The necessity for the drains should be discussed prior to grading on a slope specific basis. In addition, the project civil engineer should be consulted to evaluate the appropriate drain locations and necessary easements, building restriction zones or disclosure requirements that may be necessary. The drains should be surveyed for location and shown on the project as-built drawings.

9.9.5 The final grading plans should show the location of the proposed subdrains. Upon completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map depicting the existing conditions. The final outlet and connection locations should be determined during grading. Subdrains that will be extended on adjacent projects shortly after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and to check that the pipe has not been crushed. The contractor is responsible for the performance of the drains.

9.10 Foundation and Concrete Slabs-On-Grade Recommendations

9.10.1 The foundation recommendations presented herein are for proposed one- to two-story residential structures. We separated the foundation recommendations into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 9.10.1. Final foundation categories for each lot will be provided once site grading has been completed.

TABLE 9.10.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	T<20	--	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90
III	T≥50	D≥20	90<EI≤130

9.10.2 Table 9.10.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 9.10.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions at slab mid-point
III	24	Four No. 5 bars, two top and two bottom	No. 3 bars at 18 inches on center, both directions at slab mid-point

9.10.3 The embedment depths presented in Table 9.10.2 should be measured from the lowest adjacent pad grade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively. Figure 16 presents a wall/column footing dimension detail depicting lowest adjacent pad grade.

9.10.4 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture.

The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

9.10.5 Placement of 3 inches and 4 inches of sand is common practice in Southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation engineer present concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.10.6 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition, as required by the 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 9.10.3 for the particular Foundation Category designated. The parameters presented in Table 9.10.3 are based on the guidelines presented in the PTI, Third Edition design manual. The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer.

TABLE 9.10.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1	4.9
Edge Lift, y_M (inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47	0.66

9.10.7 If the structural engineer proposes a post-tensioned foundation design method other than the 2013 CBC:

- The criteria presented in Table 9.10.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

9.10.8 Foundation systems for the buildings that possess a foundation Category I and a "very low" expansion potential (Expansion Index of 20 or less) can be designed using the method described in in Section 1808 of the 2013 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI Third Edition) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, including additional laboratory testing, if necessary.

9.10.9 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

9.10.10 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

9.10.11 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

9.10.12 Isolated footings, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular foundation category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III.

Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

9.10.13 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

9.10.14 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in such concrete placement.

9.10.15 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal to vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal to vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to help prevent distress in the structures associates with slope creep and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- Geocon Incorporated should be contacted to review the pool plans and the specific site conditions to provide additional recommendations, if necessary.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

9.10.16 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

9.10.17 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

9.11 Exterior Concrete Flatwork

9.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 90 or less. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

9.11.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

9.11.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

9.11.4 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

9.12 Conventional Retaining Wall Recommendations

9.12.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For those lots where backfill materials do not conform to the criteria herein, Geocon Incorporated should be consulted for additional recommendations.

9.12.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the active soil pressure.

9.12.3 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 20H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.368g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

9.12.4 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

9.12.5 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 17 presents a typical retaining wall drain detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

9.12.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected.

9.12.7 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls are planned, Geocon Incorporated should be consulted for additional recommendations.

9.13 Mechanically Stabilized Earth (MSE) Retaining Walls

9.13.1 MSE retaining walls are alternative walls that consist of modular block facing units with geogrid reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. The grid length and spacing will be determined by the wall designer.

9.13.2 The geotechnical parameters listed in Table 9.13 can be used for preliminary design of the MSE walls (including proposed plantable MSE walls). Once the location of the backfill soil has been determined, laboratory testing should be performed to check that the shear strength parameters used in the design of the MSE walls meet the required strength within the reinforced zone.

TABLE 9.13
GEOTECHNICAL PARAMETERS FOR MSE WALLS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	28 degrees	28 degrees	28 degrees
Cohesion	200 psf	200 psf	200 psf
Wet Unit Density	130 pcf	130 pcf	130 pcf

9.13.3 The soil parameters presented in Table 9.13 are based on our experience, direct shear-strength tests performed during the geotechnical investigation and represent some of the onsite materials. The wet unit density values presented in Table 6.13 can be used for design but actual in-place densities may range from approximately 90 to 135 pounds per cubic foot. Geocon has no way of knowing which materials will actually be used as backfill behind the wall during construction. It is up to the wall designers to use their judgment in selection of the design parameters. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to check that they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer reinforcement embedment lengths and/or steel reinforcement).

9.13.4 The foundation zone is the area where the footing is embedded, the reinforced zone is the area of the backfill that possesses the reinforcing fabric, and the retained zone is the area behind the reinforced zone.

9.13.5 Wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf. This soil pressure may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 3,500 psf.

9.13.6 Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment width of the reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the reinforcement grid within the uncompacted zone should not be relied upon for reinforcement, and overall embedment lengths will have to be increased to account for the difference.

9.13.7 Select backfill materials may be required to be in accordance with the MSE retaining wall system. Materials as outlined in the specifications of the retaining wall plans may be generated and stockpiled during grading, if encountered, or may require import. Geocon should perform laboratory tests during the backfill materials to check that soil properties are in accordance with the retaining wall plans and specifications.

9.13.8 The wall should be provided with a drainage system sufficient to prevent excessive seepage through the wall and the base of the wall, thus preventing hydrostatic pressures behind the wall.

9.13.9 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement.

9.13.10 The MSE wall contractor should provide the estimated deformation of wall and adjacent ground in associated with wall construction. The calculated horizontal and vertical deformations should be determined by the wall designer. The estimated movements should be provided to the project structural engineer to determine if the planned improvements can tolerate the expected movements.

9.13.11 The MSE wall designer/contractor should review this report, including the slope stability requirements, and incorporate our recommendations as presented herein. We should be provided the plans for the MSE walls to check if they are in conformance with our recommendations prior to issuance of a permit and construction.

9.14 Lateral Loads

9.14.1 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 350 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill or formational materials. The allowable passive pressure assumes a horizontal surface extending away from the base of the wall at least 5 feet or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance. An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

9.15 Preliminary Pavement Recommendations

9.15.1 The final pavement sections for the roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the County of San Diego specifications when final Traffic Indices and R-Value test results of subgrade soil are completed. Based on the results of our laboratory R-Value testing, we have assumed R-Values of 5 and 25 for the subgrade soil for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 9.15.1.

TABLE 9.15.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Cul-de-sac	5.0	5	3	10
		25	3	7
Local Residential	6.0	5	3.5	13
		25	3.5	9
Collector Residential	7.0	5	4	16
		25	4	11
Arterial Roadways	8.0	5	5	18
		25	5	12

9.15.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content beneath pavement sections.

9.15.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

9.15.4 A rigid Portland cement concrete (PCC) pavement section should be placed in all roadway cross-gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 9.15.2.

TABLE 9.15.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	C
Average daily truck traffic, ADTT	300

9.15.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 9.15.3.

TABLE 9.15.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Roadway Cross-gutters (TC=C)	7.5

9.15.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

9.15.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

9.15.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12.5 feet and 15 feet for the 5.5 and 7-inch-thick slabs, respectively (e.g., a 7-inch-thick slab would have a 15-foot spacing pattern), and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

9.15.9 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

9.15.10 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

9.16 Site Drainage and Moisture Protection

9.16.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

9.16.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

9.16.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

9.16.4 If detention basins, bioswales, retention basins, water infiltration, low impact development (LID), or storm water management devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design.

9.16.5 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. Based on our experience with similar soil conditions, infiltration areas are considered infeasible due to the poor percolation and lateral migration characteristics. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

9.16.6 Storm water management devices should be properly constructed to prevent water infiltration and lined with an impermeable liner (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC, liner). The devices should also be installed in accordance with the manufacturer’s recommendations.

9.16.7 The United States Department of Agriculture (USDA), Natural Resources Conservation Services possesses general information regarding the existing soil conditions for areas within the United States. Table 9.16.1 presents the soil name based on the USDA website.

TABLE 9.16.1
EXISTING SOIL CONDITIONS BASED ON USDA WEBSITE

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Friant rocky fine sandy loam, 30 to 70 percent slopes	FxG	33.3	D
Olivenhain cobbly loam, 9 to 30 percent slopes	OhE	48.2	D
San Miguel-Exchequer rocky silt loams, 9 to 70 percent slopes	SnG	18.3	D
Water	W	0.2	N/A

9.16.8 The USDA website also provides the Hydrologic Soil Group. Based on the USDA website, the soil is classified as a Soil Group D. Table 9.16.2 presents the description of Hydrologic Soil Group. If the soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in the natural condition are in group D are assigned to dual classes.

TABLE 9.16.2
SATURATED PERMEABILITY TEST RESULTS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

9.17 Grading and Foundation Plan Review

9.17.1 Geocon Incorporated should review the 40-scale grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analysis and/or recommendations are required.

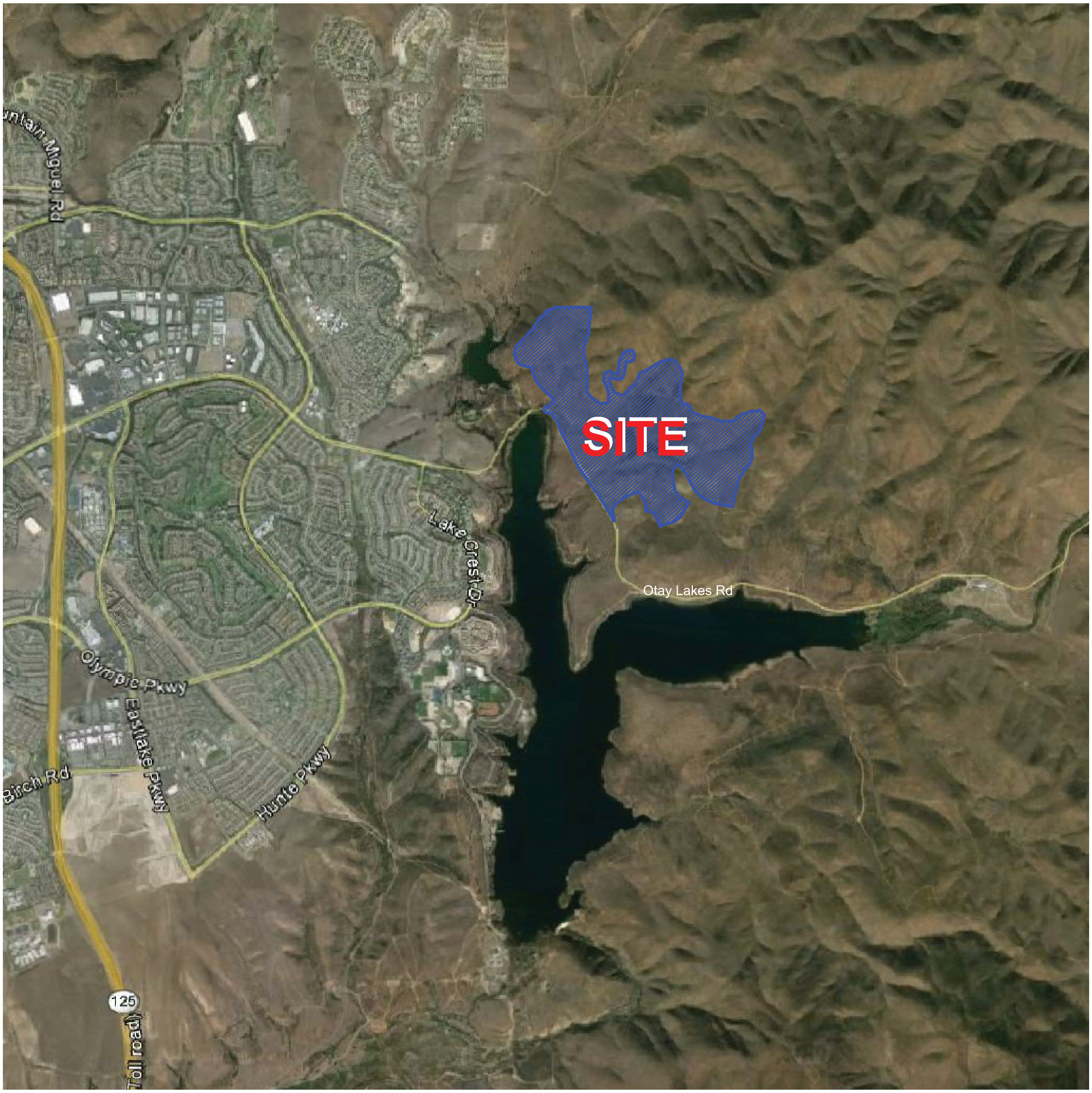
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.



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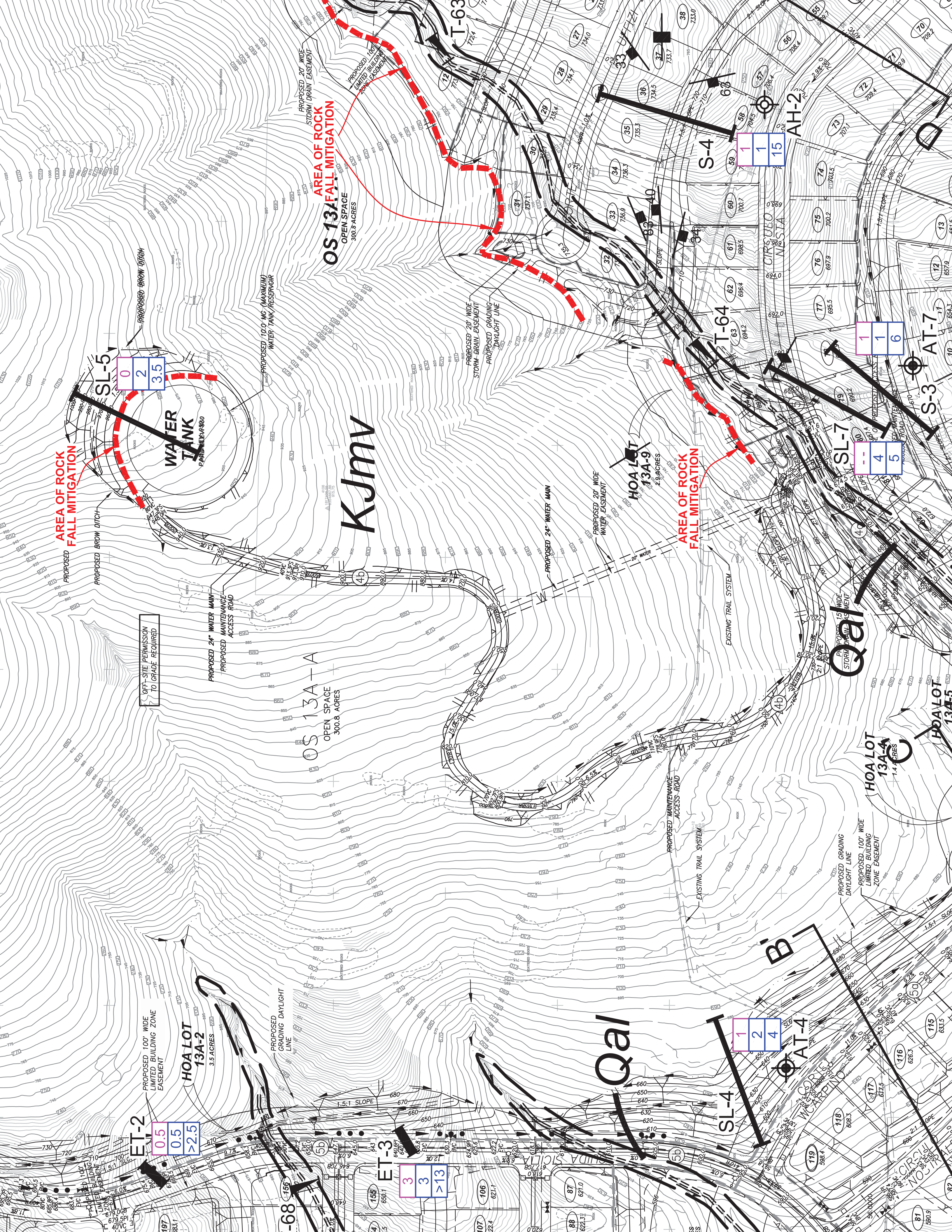
VICINITY MAP

OTAY RANCH RESORT VILLAGE
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

JH / AML		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 1

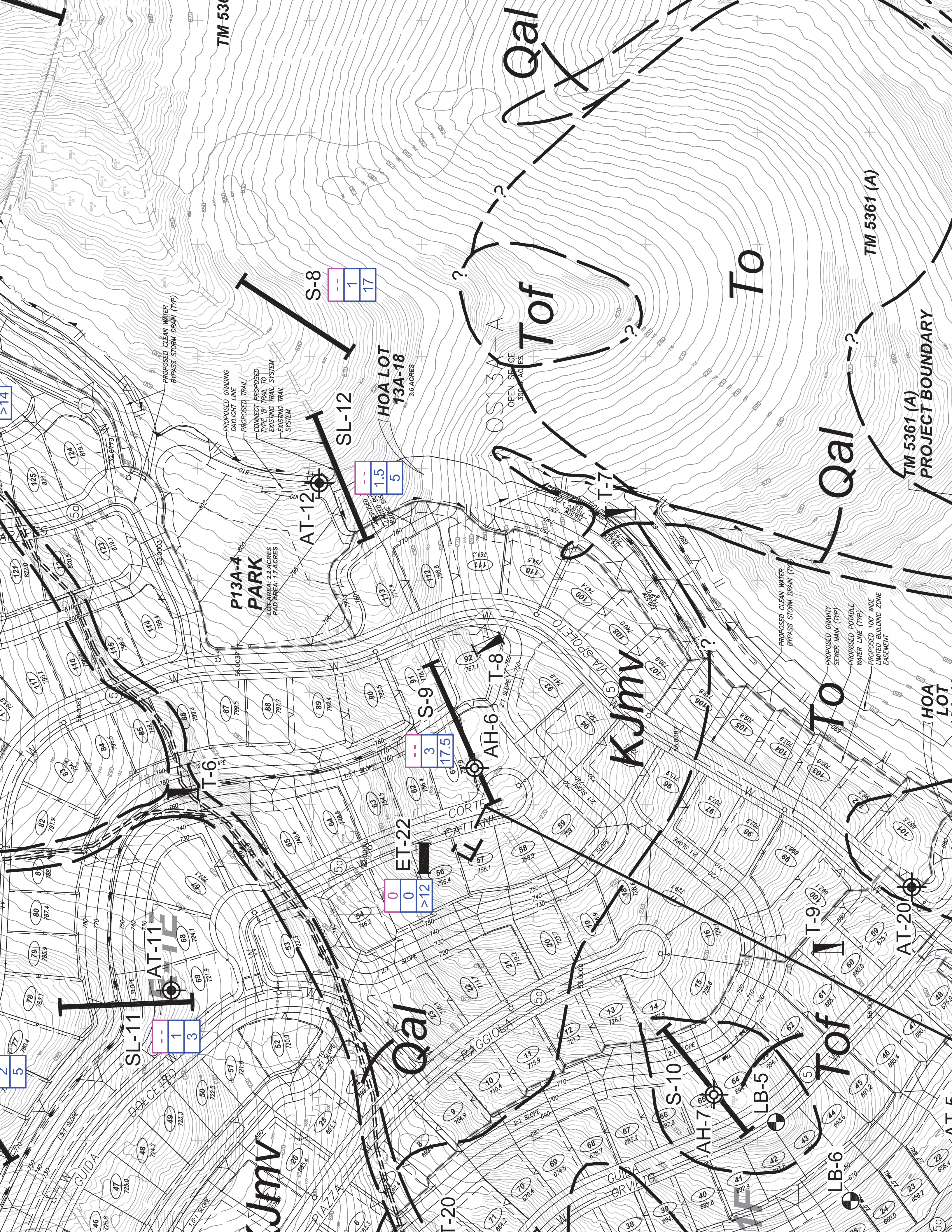
Ladrillono

Y:\PROJECTS\G1012-52-01D OTAY RANCH VILLAGE 13-AREA A (Upd 09-05-2014)\DETAILS\G1012-52-01D Vic Map.dwg



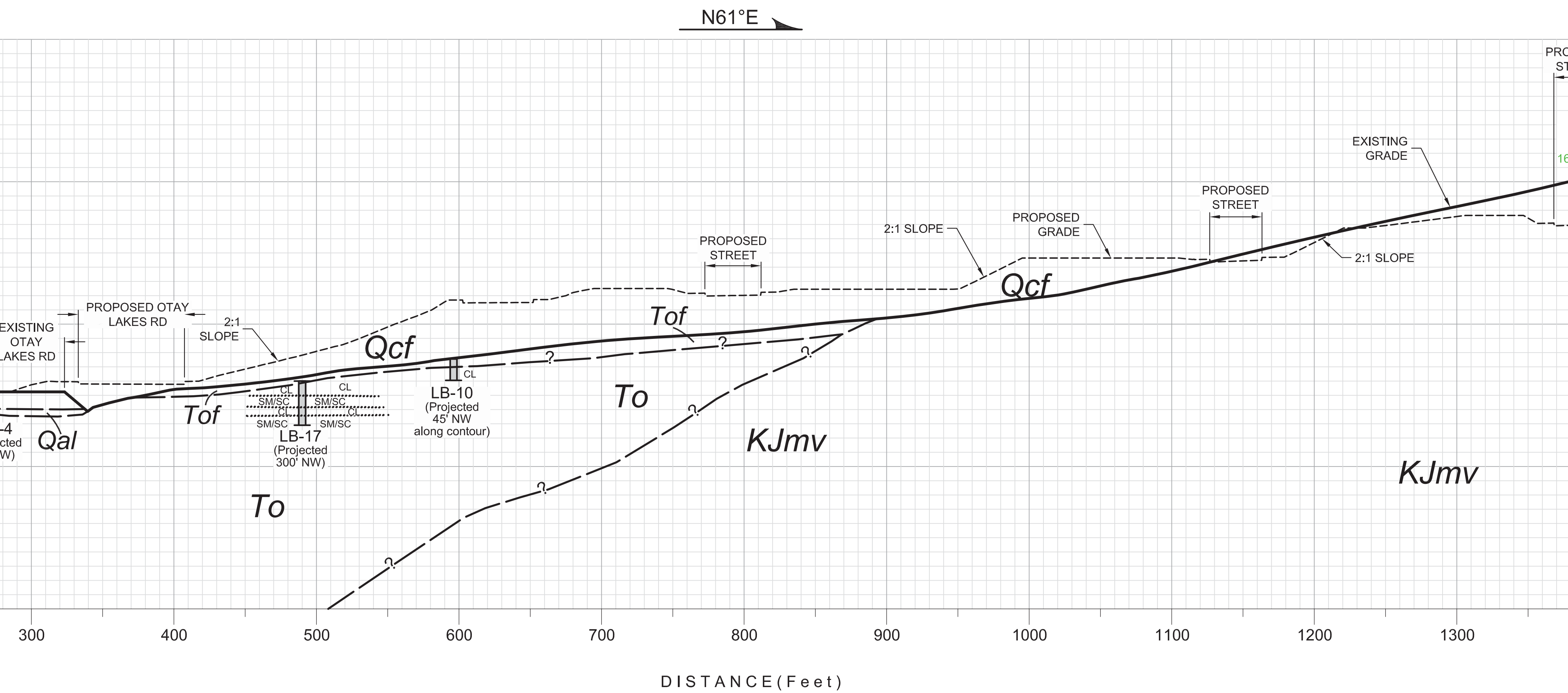
AREA OF ROCK FALL MITIGATION
AREA OF ROCK FALL MITIGATION
AREA OF ROCK FALL MITIGATION
OS 13A-A
OPEN SPACE
300.8 ACRES
WATER TANK
KJmv
Qal
Qal
HOA LOT 13A-9
2.9 ACRES
HOA LOT 13A-2
3.5 ACRES
HOA LOT 13A-4
1.4 ACRES
OFF-SITE PERMISSION TO GRADE REQUIRED
PROPOSED BROW DITCH
PROPOSED 10.0 MG (MAXIMUM) WATER TANK/RESERVOIR
PROPOSED 24" WATER MAIN
PROPOSED MAINTENANCE ACCESS ROAD
PROPOSED 20' WIDE WATER EASEMENT
PROPOSED 20' WIDE STORM DRAIN EASEMENT
PROPOSED GRADING DAYLIGHT LINE
PROPOSED 100' WIDE LIMITED BUILDING ZONE EASEMENT
EXISTING TRAIL SYSTEM
CIRCULO NESTA
SL-5
S-4
AH-2
T-63
T-64
AT-7
S-3
SL-7
AT-4
SL-4
ET-2
ET-3

MU-1
MIXED USE
TM 5361 (B)
EXISTING PROJECT BOUNDARY
PROPOSED PROJECT BOUNDARY
HOA LOT 13A-22
4.7 ACRES
HOA LOT 13A-25
HOA LOT 13A-24
OS 13A-E
OPEN SPACE
13.4 ACRES
RANCHO JAMUL MAP NO. 989
PIAZZA SAN LINO
PIAZZA SORANO
SUBDRAIN OUTLET
WATER QUALITY BASIN #3
DAYLIGHT LINE
OFF-SITE PERMISSION TO GRADE REQUIRED
EXISTING TRAIL SYSTEM
Qal
To
Tof
Qudf
ET-10
ET-11
ET-12
ET-13
T-51
T-52
T-53
T-54
T-55
T-57
NT-4
NT-5
NT-7
NB-1
SB-5



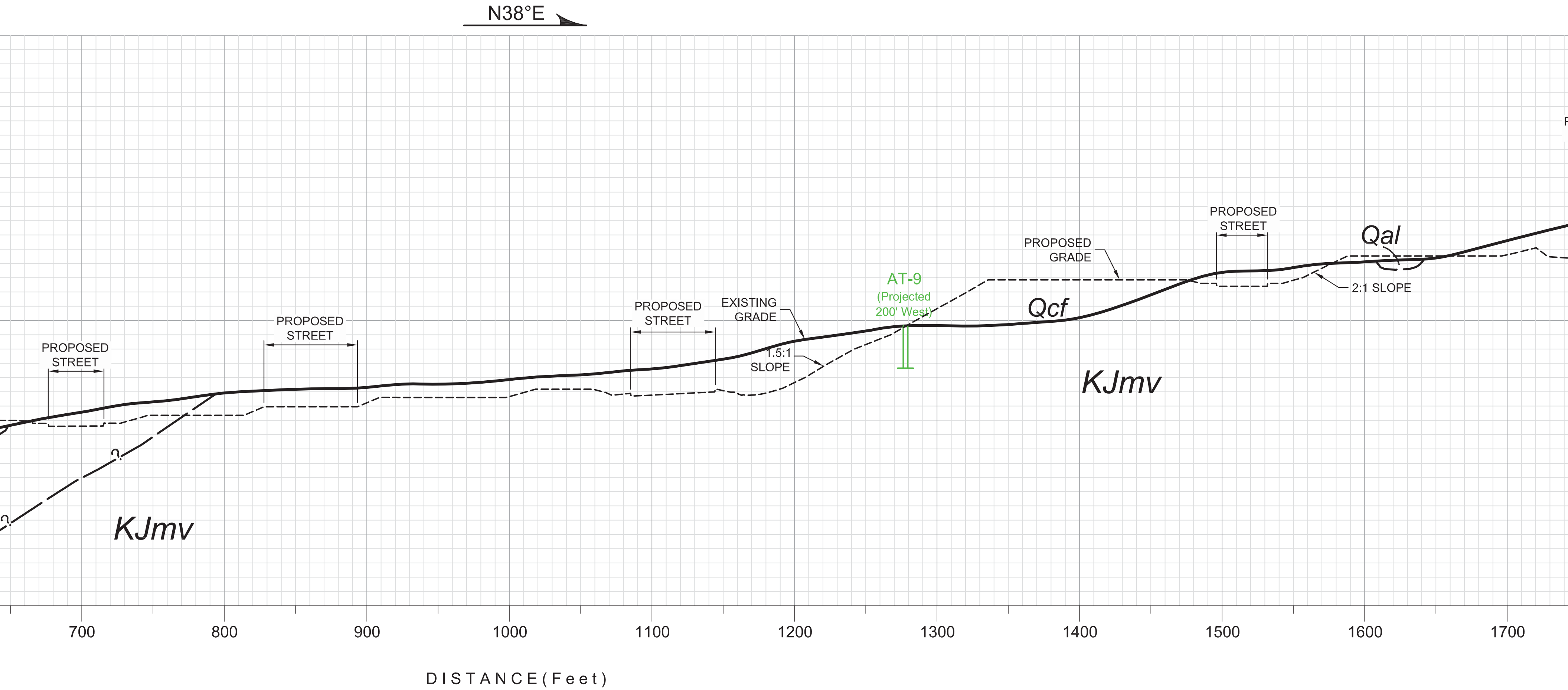
TM 5361 (A)
TM 5361 (A) PROJECT BOUNDARY
Qal
Tof
To
KJmv
S-8
SL-12
HOA LOT 13A-18
3.6 ACRES
OS13A-A
OPEN SPACE
AT-12
T-7
T-8
T-6
T-9
S-9
AH-6
ET-22
SL-11
EAT-11
S-10
AH-7
LB-5
LB-6
AT-20
T-20
P13A-4 PARK
LOT AREA: 2.2 ACRES
PAD AREA: 1.7 ACRES
PROPOSED CLEAN WATER BYPASS STORM DRAIN (TYP)
PROPOSED GRADING DAYLIGHT LINE
PROPOSED TRAIL
CONNECT PROPOSED TYPE 'B' TRAIL TO EXISTING TRAIL SYSTEM
EXISTING TRAIL SYSTEM
PROPOSED GRAVITY SEWER MAIN (TYP)
PROPOSED POTABLE WATER LINE (TYP)
PROPOSED 100' WIDE LIMITED BUILDING ZONE EASEMENT
VIA SPOLETO
CORTE CATTANI
RAGGIOLA
DOLCETTO
GUIDA ORVIETO
PIAZZA
HOA LOT

SCALE: 1" = 100' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B′

SCALE: 1" = 100' (Vert. = Horiz.)

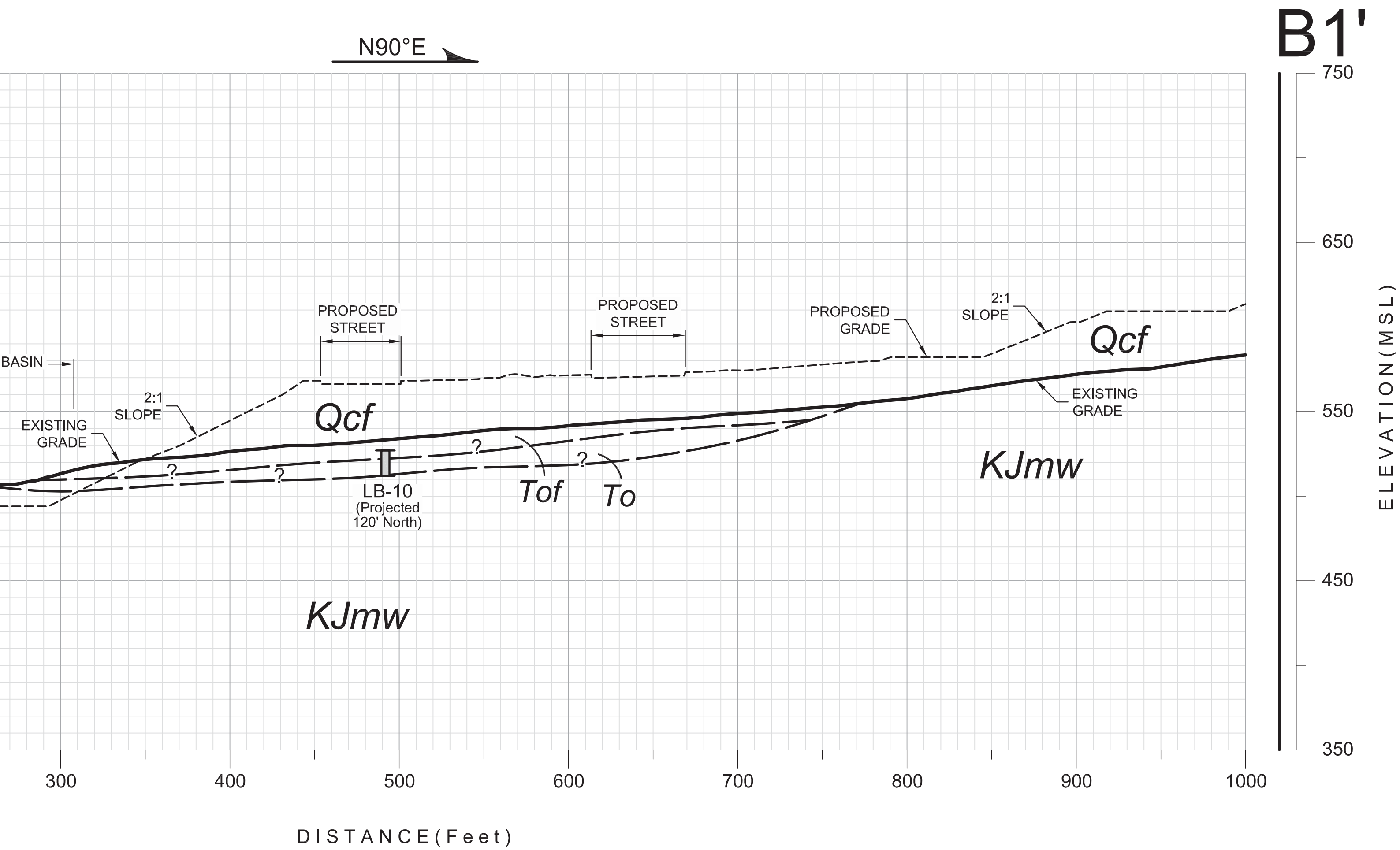


GEOLOGIC CROSS-SECTION E-E′

SCALE: 1" = 100' (Vert. = Horiz.)

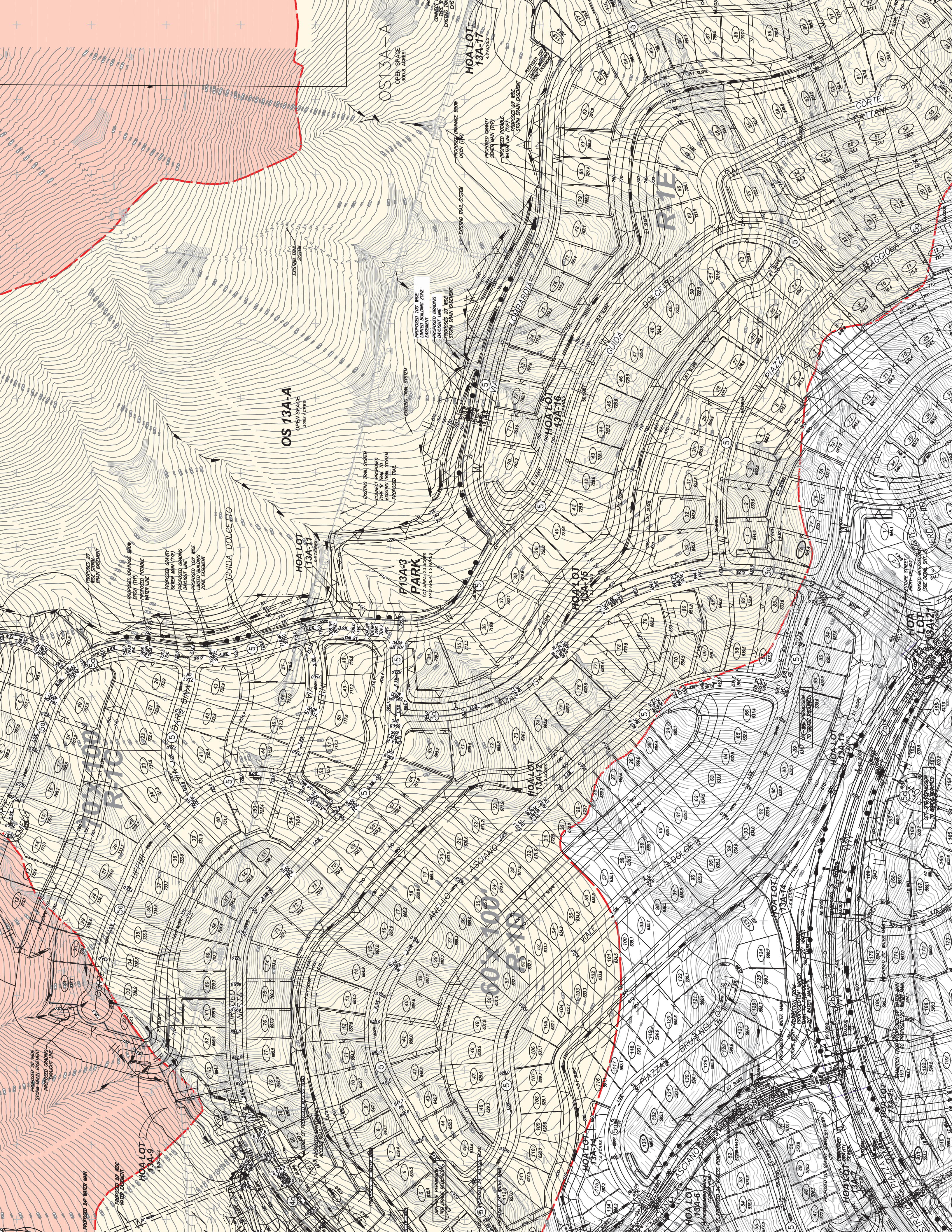
N27°E

SCALE: 1" = 100' (Vert. = Horiz.)

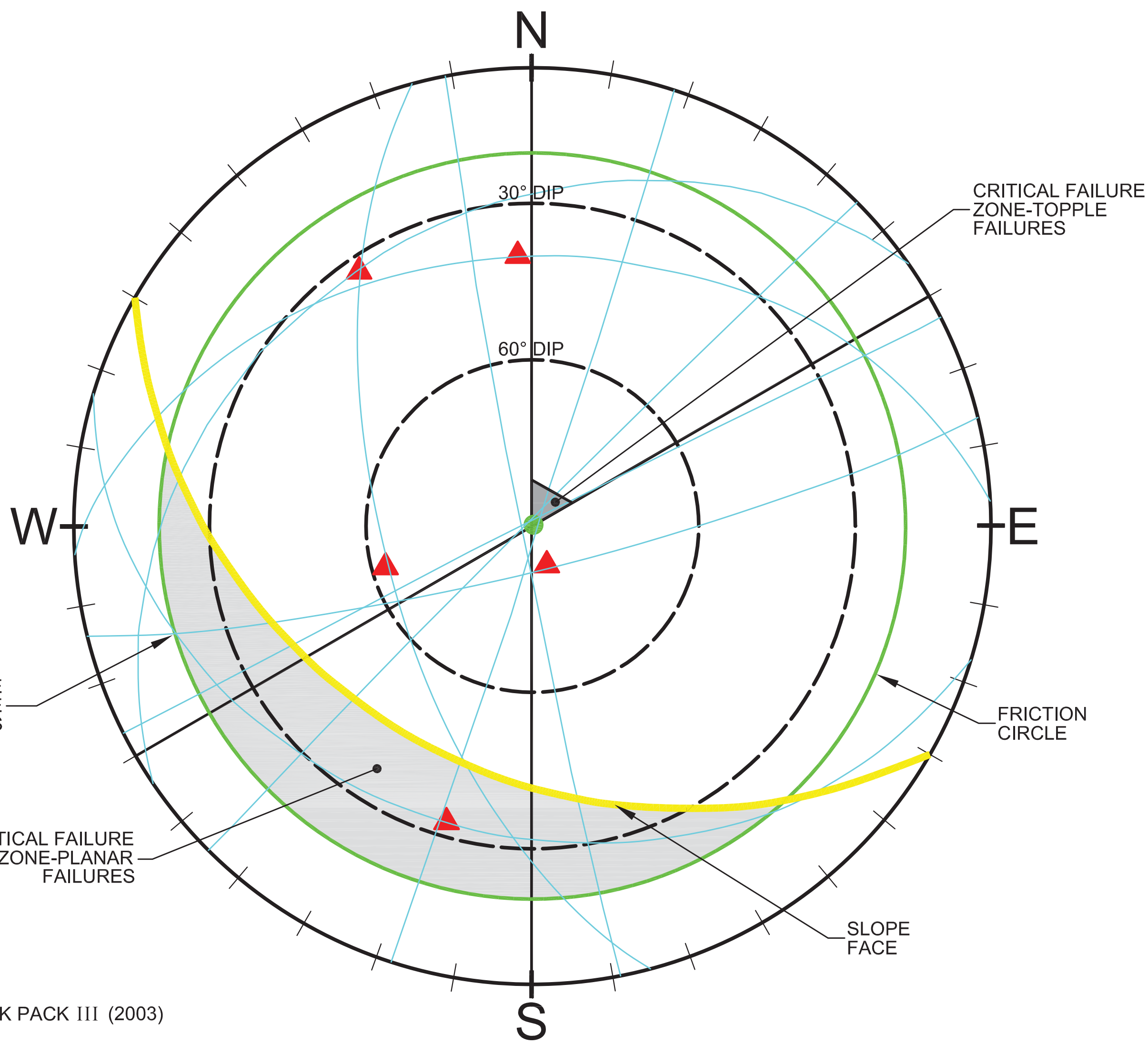


GEOLOGIC CROSS-SECTION B-B′

SCALE: 1" = 100' (Vert. = Horiz.)



OS13A-A
HOA LOT
13A-17
OS 13A-A
OPEN SPACE
HOA LOT
13A-16
HOA LOT
13A-11
P13A-3
PARK
HOA LOT
13A-15
HOA LOT
13A-12
HOA LOT
13A-14
HOA LOT
13A-13
HOA LOT
13A-9
HOA LOT
13A-6
CORTE
GUIDA DOLCETTO
LOMBARDIA
PIAZZA
ASCIANO
DOLCE
BRUNELLO



FROM: ROCK PACK III (2003)

GEOCON LEGEND

▲ ....JOINT DIP VECTOR

● ....FOLIATION DIP VECTOR

KINEMATIC STEREONET ANALYSIS

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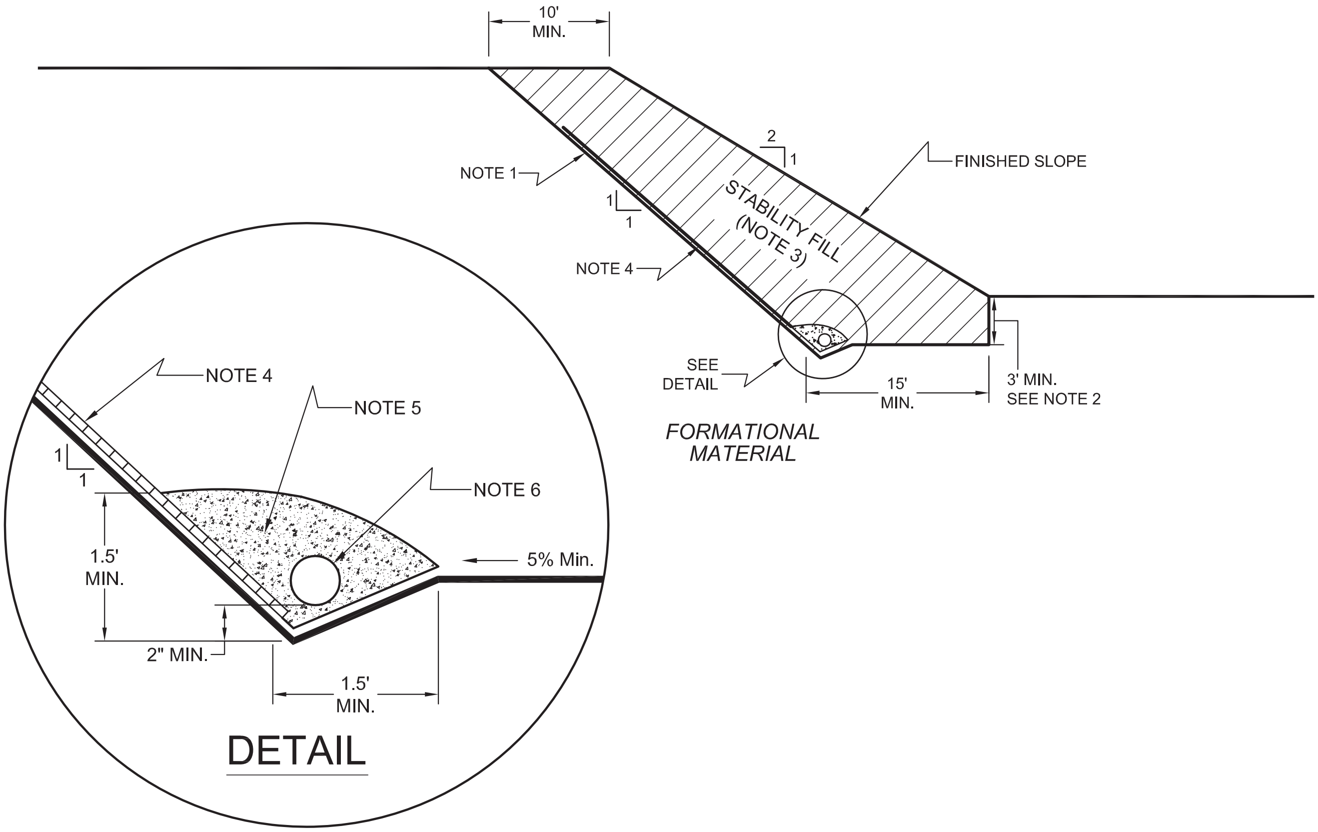
OTAY RANCH RESORT VILLAGE
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

JH / RA		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 9



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

TYPICAL STABILITY FILL DETAIL

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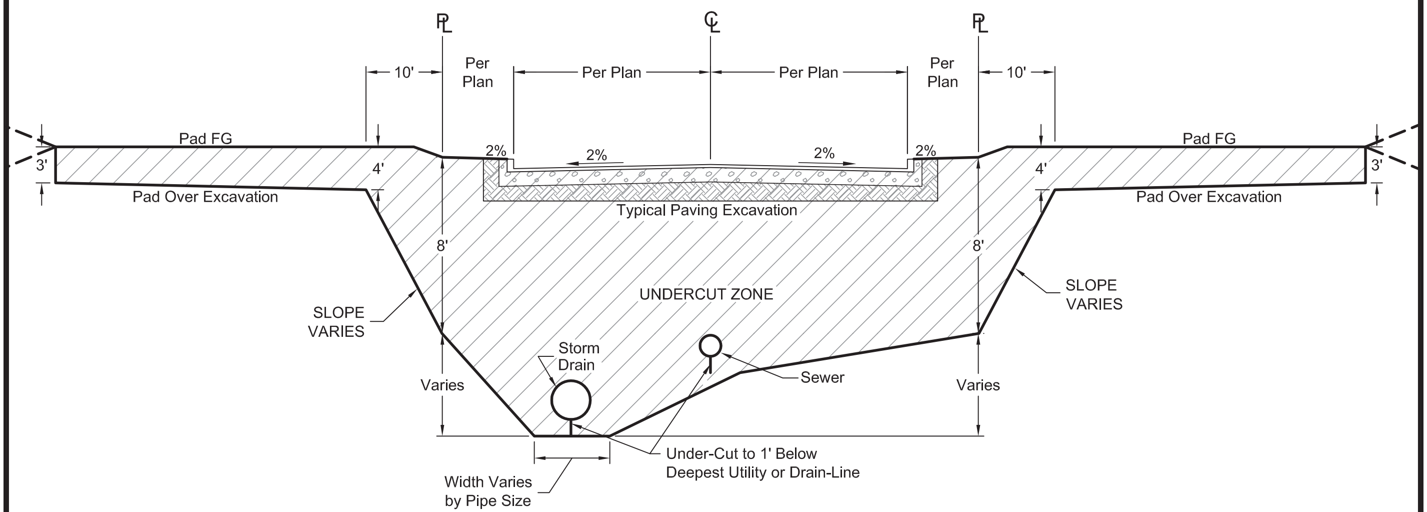
SAN DIEGO COUNTY, CALIFORNIA

JH / RA		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 10



NOT TO SCALE

NOTE:
UNDERCUT ZONE SHOULD CONTAIN COMPACTED SOIL FILL WITH MAXIMUM ROCK FRAGMENTS LESS THAN 6 INCHES IN DIMENSION AND A MINIMUM OF 40 PERCENT SOIL PASSING THE 3/4-INCH SIEVE

TYPICAL STREET OVEREXCAVATION DETAIL

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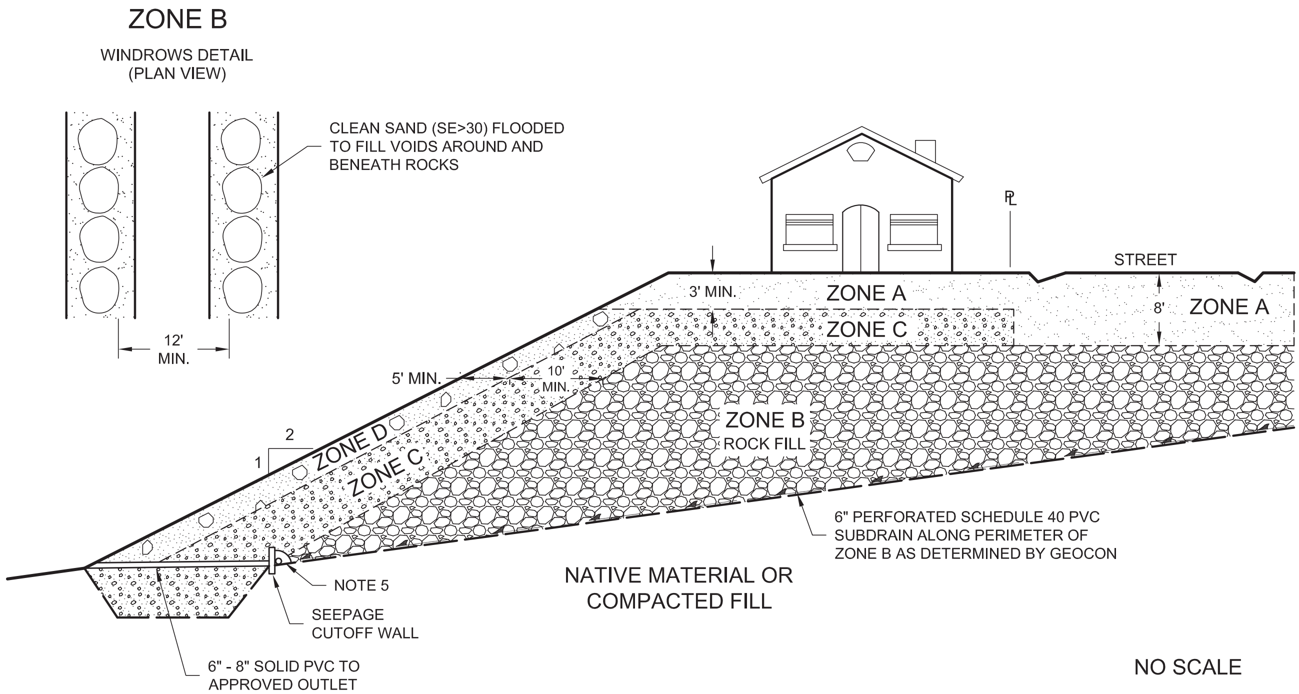
SAN DIEGO COUNTY, CALIFORNIA

JH / RA		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 11



LEGEND

ZONE A: COMPACTED SOIL FILL. NO ROCK FRAGMENTS OVER 6 INCHES IN DIMENSION.

ZONE B: BLASTED ROCK FILL GENERALLY CONSISTING OF 2 FOOT MINUS MATERIAL WITH OCCASIONAL INDIVIDUAL ROCK UP TO 4 FEET MAXIMUM DIMENSION
ALTERNATE: ROCKS 2 TO 4 FEET IN MAXIMUM DIMENSION CAN BE PLACED IN WINDROWS IN COMPACTED SOIL FILL POSSESSING A SAND EQUIVALENT OF AT LEAST 30.

ZONE C: ROCKS UP TO 2 FEET IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL WITHIN BUILDING PADS AND SLOPE AREAS ONLY.

ZONE D: ROCKS UP TO 1 FOOT IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL.

NOTES

1. COMPACTED SOIL FILL IN UPPER 8 FEET SHALL CONTAIN AT LEAST 40 PERCENT SOIL PASSING THE 3/4 - INCH SIEVE (BY WEIGHT) AND IN THE UPPER 3 FEET OF PAD GRADE AT LEAST 20% SOIL PASSING THE NO. 4 SIEVE (BY WEIGHT) AND COMPACTED IN ACCORDANCE WITH SPECIFICATIONS FOR STRUCTURAL FILL.
2. CONTINUOUS OBSERVATION REQUIRED BY GEOCON DURING ROCK PLACEMENT.
3. ROCK FILL (LESS THAN 40 PERCENT SOIL SIZES) MAY BE PERMITTED IN DESIGNATED AREAS UPON THE RECOMMENDATION OF THE GEOTECHNICAL ENGINEER.
4. DEPTH OF ZONE A SHOULD BE AT LEAST 8 FEET AND EXTENDED TO AT LEAST 2 FEET BELOW DEEPEST UTILITY WITHIN ROADWAYS.
5. 6" PERFORATED SCHEDULE 40 PVC SUBDRAIN ALONG THE TOE AND PORTIONS OF THE PERIMETER OF ZONE B.
6. BASE OF ZONE B SHOULD SLOPE A MINIMUM OF 3 PERCENT.

OVERSIZE ROCK DISPOSAL DETAIL

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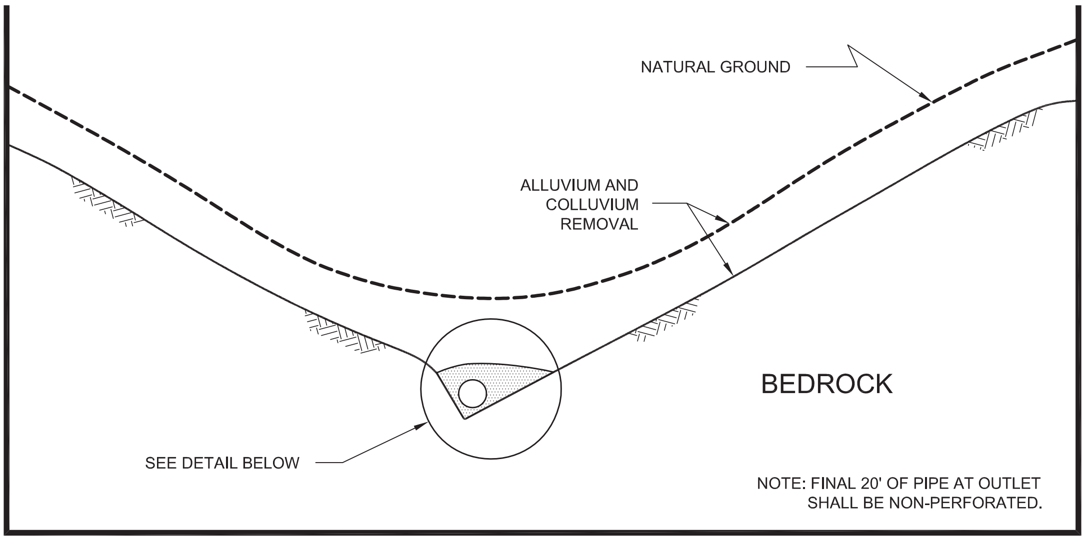
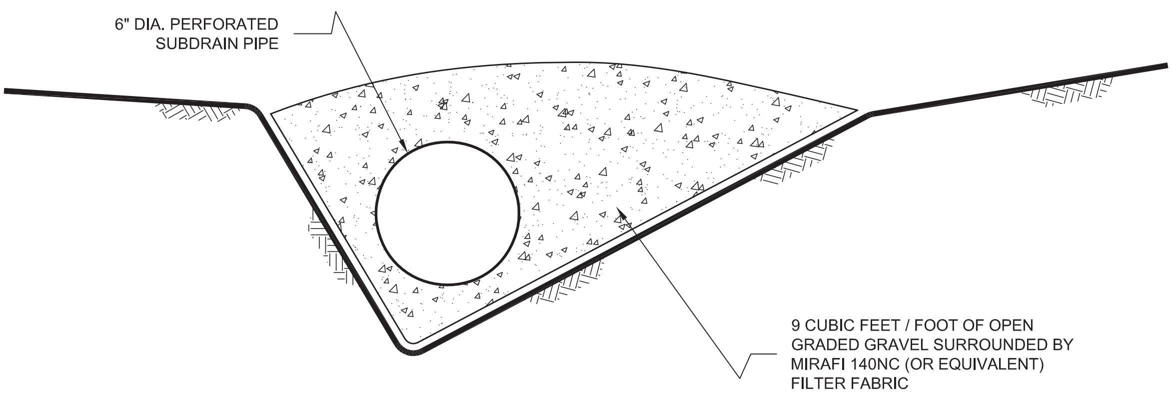
JH / RA		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 12





NOTES:

1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 750 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 750 FEET.

NO SCALE

TYPICAL CANYON SUBDRAIN DETAIL

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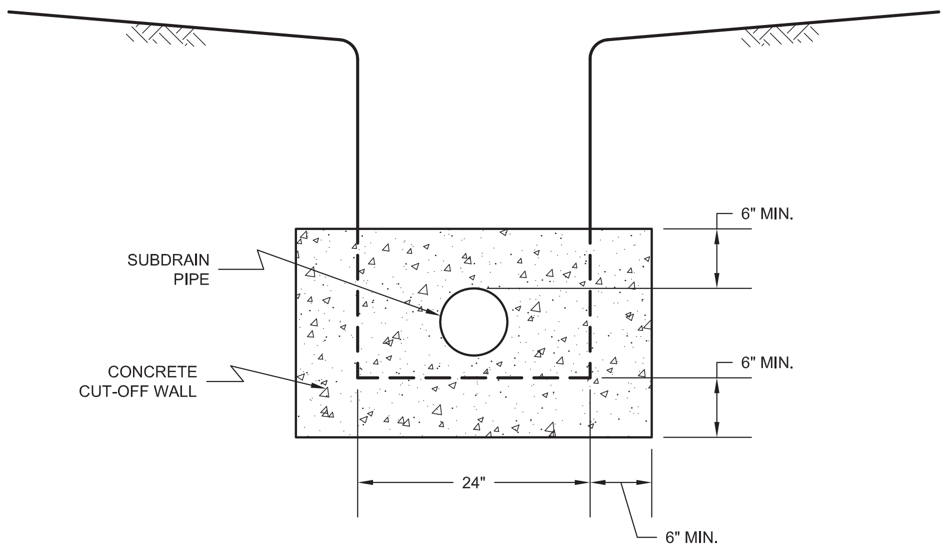
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

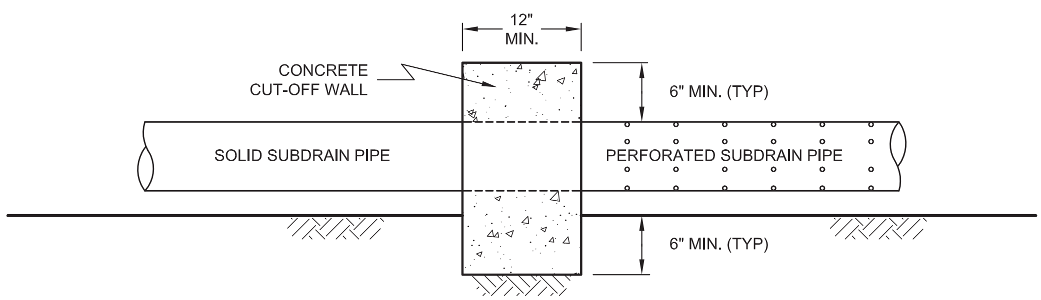
JH / RA		DSK/E0000	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 13

FRONT VIEW



NO SCALE

SIDE VIEW



NO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL DETAIL

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OTAY RANCH RESORT VILLAGE
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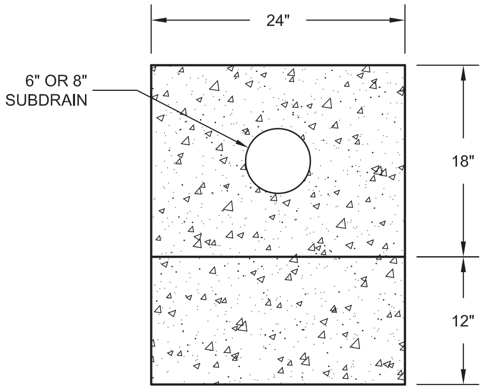
SAN DIEGO COUNTY, CALIFORNIA

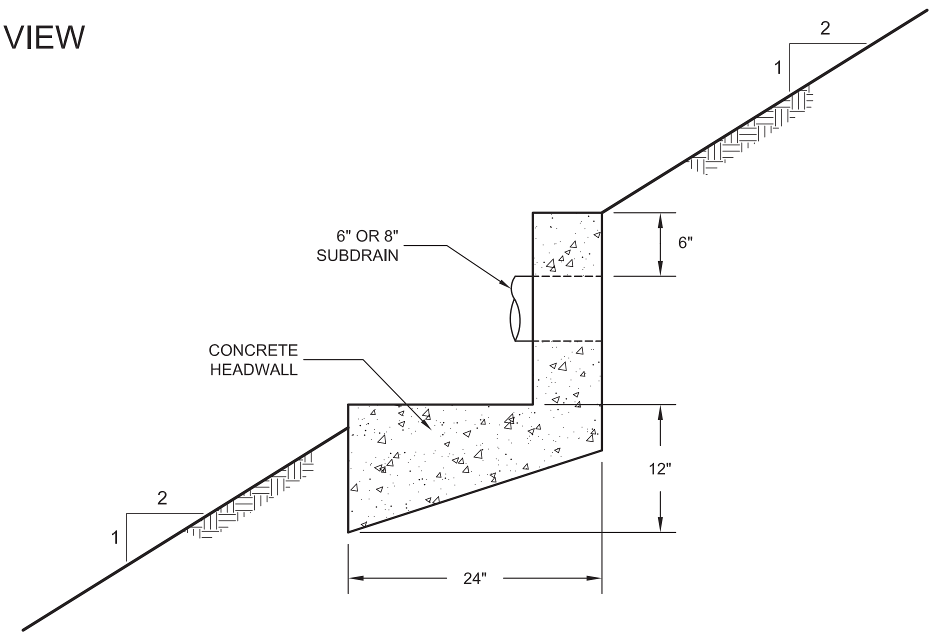
JH / RA		DSK/E0000	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 14

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

SUBDRAIN OUTLET HEADWALL DETAIL

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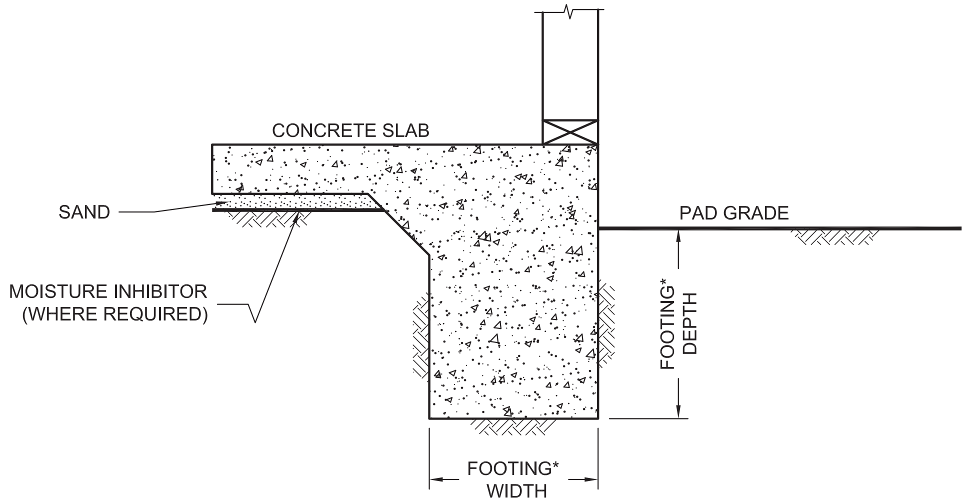
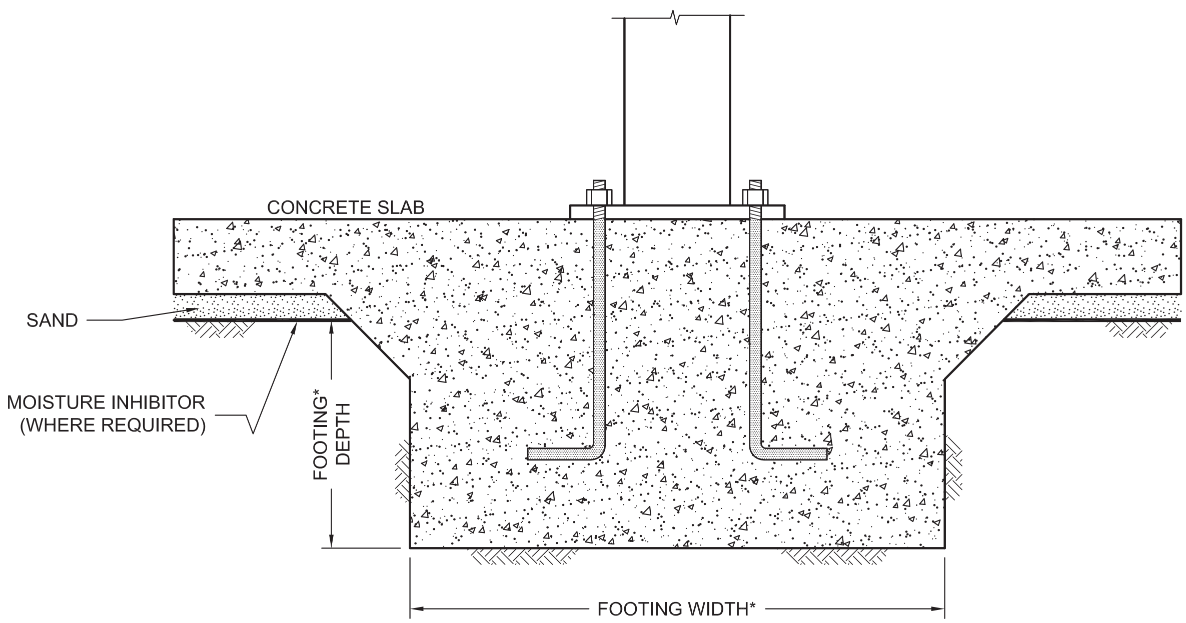
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

JH / RA		DSK/E0000	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 15





*....SEE REPORT FOR FOUNDATION WITDH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

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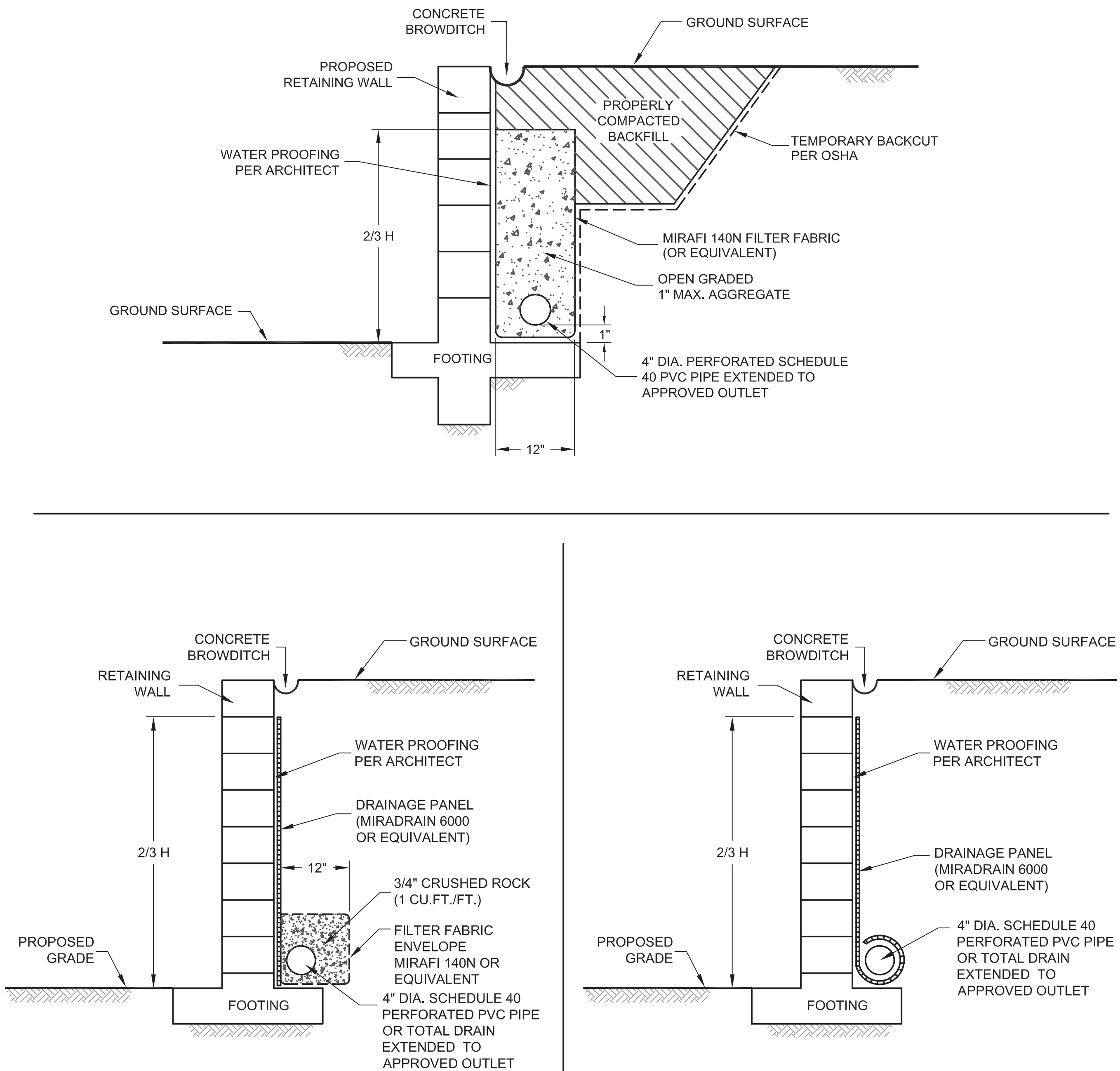

NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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OTAY RANCH RESORT VILLAGE
AREA A TENTATIVE MAP
SAN DIEGO COUNTY, CALIFORNIA

JH / AML		DSK/GTYPD	DATE 09 - 05 - 2014	PROJECT NO. G1012 - 52 - 01D	FIG. 17

RWDD7A

Y:\PROJECTS\G1012-52-01D OTAY RANCH VILLAGE 13-AREA A (Upd 09-05-2014)\DETAILS\RWDD7A.dwg

LIST OF REFERENCES

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