PRELIMINARY GEOTECHNICAL AND FAULT RUPTURE HAZARD INVESTIGATION

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

STRATA EQUITY GROUP, INC.
SAN DIEGO, CALIFORNIA

REVISED FEBRUARY 24, 2015
MARCH 24, 2014
PROJECT NO. T2539-22-02A

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2539-22-02A
March 24, 2014
REVISED February 24, 2015

Strata Equity Group, Inc.
4370 La Jolla Village Drive, Suite 960
San Diego, California 92122

Attention: Mr. Eric Flodine

Subject: PRELIMINARY GEOTECHNICAL AND
FAULT RUPTURE HAZARD INVESTIGATION
GROVE PARK, APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

Dear Mr. Flodine:

In accordance with your request, we have performed a preliminary geotechnical investigation and fault rupture hazard investigation of a 20-acre parcel located south of Clinton Keith Road in Wildomar, California. The accompanying report presents the findings of our study, and our preliminary conclusions and recommendations pertaining to the geotechnical aspects of future design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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PRELIMINARY GEOTECHNICAL AND FAULT RUPTURE HAZARD INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical and fault rupture hazard investigation for an approximately 20-acre parcel in the City of Wildomar known as Grove Park. The site is APN 380-250-003 and is located immediately southwest of Clinton Keith Road and Yamas Drive (proposed), as depicted on the Vicinity Map, Figure 1. The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the parcel, and based on conditions encountered to provide preliminary conclusions and recommendations pertaining to the geotechnical and geologic aspects of future design and construction.

The scope of our investigation included a site reconnaissance, review of available geologic literature, geotechnical field exploration, laboratory testing, engineering analysis, fault trench excavations, geologic logging, and the preparation of this report. Geotechnical drilling was performed on November 7th and 13th, 2012 by excavating three 8-inch diameter borings with a CME 75 drill rig. The borings were excavated to depths between 10.5 and 50.5 feet below the existing ground surface. Three test pits were excavated in areas which were inaccessible to the drill rig. The test pits were excavated utilizing a four-wheel drive backhoe equipped with a 30-inch bucket to depths between 5 and 8 feet. The approximate locations of the exploratory borings, test pits, and fault trenches are depicted on the Geologic Map, Figure 2. A detailed discussion of the geotechnical field investigation, including boring and test pit logs, is presented in Appendix A. Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

Fault trenching was performed to determine if active faulting was present on the site. The trenching was performed on October 25 through November 13, 2012 and entailed the excavation of 194 lineal feet of fault trench (FT-1 and FT-2) on the subject site APN 380-250-003 and 225 lineal feet of fault trench (FT-3) on the APN 380-250-023. Riverside County has depicted an unclassified fault crossing both of the sites based on the Land Information System data base. The County has not established a fault hazard zone around this fault. It appears that this fault was taken from regional geologic mapping performed by Kennedy and Morton from their *Preliminary Geologic Map of the Murrieta 7.5' Quadrangle* (Version 1.0). Kennedy did not include the site faults in his previous 1977 study. A detailed discussion of the fault hazard investigation, including logs of the trench excavations are provided in Appendix C of this report.

At the time of our investigation, the project site area included APN 380-250-003 and APN 380-250-023 as one project. Therefore our laboratory testing and fault hazard investigation included data for both sites. At your request, this report is limited to APN 380-250-003 only, except where the Fault Trench FT-3 is required to support the conclusions of the fault investigation.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section. If project details vary significantly from those described above, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

Parcel 380-250-003 is bounded on the north by Clinton Keith Road, the west by a vacant 20 acre parcel and Inland Valley Drive, the south by multi-family residential units and the east by a 10 acre residential parcel and undeveloped land. The site is currently vacant. Topographically the site consists of an alluvial plain which descends gently to the southwest from Clinton Keith Road. The plain is incised by two drainage areas which flow south to southwest across the property. Vegetation consists of dry grasses, occasional shrubs along the hillsides, and large oak trees and weeds within the drainage areas. A storm drain culvert under Clinton Keith Road discharges into the western drainage from the parcel to the north. A corrugated metal pipe discharges surface storm water drainage from Clinton Keith Road onto the site above the culvert. Dense vegetation was present within this channel however, no surface water flow was observed at the time of our reconnaissance or site work in October and November, 2012. A larger channel crosses southwesterly from the east central portion of the site to the southwestern area. A large borrow site with estimated removals on the order of 8 to 10 feet has been graded along the channel in the southwestern quadrant of the property. The grading was evident in the 1962 aerial photographs reviewed for this site. Several earthen dams were observed across channels in the vicinity of the property. Dirt berms are present along the project boundaries with heights on the order of 3 feet and appeared to be derived from local soil with minor amounts of construction debris (asphalt and concrete). Overhead utilities supported by wood poles were observed along the eastern property line extending into the site approximately 100 feet from Clinton Keith Road. It is our understanding that proposed site development will include commercial, office, assisted living, and multifamily living. All structures are assumed to be typically wood or steel frame construction on slab-on-grade foundations and are anticipated to be three stories or less in height.

The locations and descriptions herein are based on a site reconnaissance, review of the referenced Parcel Maps, aerial photographs, previous geotechnical reports, and project information provided by the client, as well as our knowledge and experience of the surrounding areas.

Based on topography, cuts and fills on the order of 10 feet (exclusive of remedial grading) will likely be accomplished during development. Structural loads estimated for the proposed structures may be up to 10 kips. Wall loads are for the proposed structures may be up to 1.5 kips per linear foot.

Once the design phase and foundation loading configuration are developed, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

Parcel 380-250-003 is located southeast of the Elsinore trough within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on the north by the Transverse Ranges and the Cucamonga/Sierra Madre faults, the east by the San Jacinto fault, the west by the Elsinore fault and the Santa Ana Mountains. The Peninsular Ranges extend southward into Mexico. The Peninsular Ranges are characterized by granitic highlands of low to moderate relief surrounded by alluvial plains and valleys. Locally, the Elsinore trough is the dominant geomorphic feature of the area and was created by a graben that formed as a result of a left step over from the Wildomar to the Willard faults which are mapped on the eastern and western sides of the lake, respectively. Geologic mapping by Kennedy (1977) identifies the geologic units at the site as Unnamed Sandstone and Pauba Sandstone with granitic rock mapped northeast of the site. Geologic units within the southern parcel are mapped as Pauba Sandstone (Kennedy, 1977).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of topsoil and colluvium overlying Pauba and Unnamed Sandstone. Unnamed Sandstone is present directly beneath the topsoil and colluvium within the central portion of the site as exposed in Fault Trench 1 (FT-1). The western portion of the site is capped with Pauba Sandstone overlying Unnamed Sandstone as exposed in Fault Trench 2 (FT-2). Alluvium is present within the drainages and undocumented fill forms the berms and earthen dams on and adjacent to the site. Detailed stratigraphic profiles are provided in the boring and test pit logs in Appendix A and fault trench logs in Appendix C.

4.1 Undocumented Fill (Qudf)

Undocumented artificial fill was observed in the southern areas of the site and west of the site. The fill formed earthen dams across the southern drainage. It appeared to be locally derived silty sand which was dry and medium dense. The undocumented fill is unsuitable to support structures or additional fill loads and should be removed during grading. The fill materials may be reused as fill providing all deleterious materials are removed.

4.2 Younger Alluvium/Topsoil (Qal)

Topsoil overlies the hillsides of the site to depths of 6 to 18 inches. It consists of dry, loose (recently plowed), slightly blocky silty sand. Younger alluvium is present within the drainage areas to depths of 3 to 10 feet. The alluvium generally consists of moist, loose to medium dense, inter-layered silty sand, sand, and cobbles. It is unsuitable for the support of structures or additional fill and will require removal during grading. The alluvium can be reused as fill providing all deleterious materials are removed.

4.3 Colluvium (Qcol)

Colluvium overlies the Unnamed and Pauba Sandstones. It is generally 12 to 18 inches deep. The colluvium consists of red-brown clayey sand. The unit is dense, dry to moist, and blocky with clay development on ped facies and weathering rinds on gravel and cobbles. The colluvium is not suitable to provide a base for structures or fill loads and should be removed during grading. It may be used as fill for the site providing all deleterious materials are removed.

4.4 Pauba Sandstone (Qps)

Early Pleistocene-age Pauba Sandstone was encountered within the western ridge of the site. The Pauba Sandstone consists of locally massive yellow-red silty sandstone which was poorly bedded, moderately to highly weathered, moderately fractured, hard and dry to moist. Unweathered Pauba is suitable for the support of structural and fill loads.

4.5 Unnamed Sandstone (Qus)

Earliest Pleistocene-age Unnamed Sandstone was present within the central portion of the site and consists of light yellow brown, coarse, poorly bedded, highly weathered silty sandstone which is dry to moist and hard. Occasional cobbles and siltstone beds were observed within the Fault Trench FT-1 excavation. Localized areas of completely weathered rock were observed in the upper approximately one foot within Fault Trench FT-1. The upper approximately one foot of the Unnamed Sandstone will require remedial grading prior to the placement of structural or fill loads.

5. GROUNDWATER

Groundwater was encountered within the main drainage of the site at a depth of 15 feet below existing grade within Boring 3 (B-3). The California Water Library data indicates groundwater wells in the vicinity of the site (07S03W06A001S elevation 1380 ft mean sea level (MSL) and 07S03W06B001S elevation 1355 ft MSL) reported groundwater depths ranging of 10 and 18 feet below ground surface in 1968.

Groundwater may be encountered during grading and drainage measures such as sub-drains and back-drains may be recommended to mitigate subsurface water. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.16).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS), formerly known as California Division of Mines and Geology (CDMG), for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. However, Riverside County depicts a mapped fault trending northwest across APN 380-250-003. The mapped fault is based on Kennedy and Morton’s mapping. The fault is mapped trending approximately N32W from near the center of the eastern site boundary to near the northwest corner, see Figure 2. A fault rupture hazard investigation was performed by LGC on an adjacent property located east of the parcel; however, the report was inconclusive with respect to the location and activity of the faulting. As a result, Geocon performed a fault hazard investigation by excavating trenches approximately perpendicular to the mapped fault trace within APN 380-250-003 (FT-1 and FT-2) and APN 380-250-023 (FT-3). We did encounter faulting within the Unnamed Sandstone (FT-1) however, we did not observe evidence of faulting within the Pauba Sandstone (FT-2 and FT-3). Therefore, the faulting likely occurred prior to the deposition of the Pauba Sandstone and the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. A detailed discussion of the subsurface fault hazard investigation is provided in Appendix C.

The site is located in the seismically active southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Temecula branch of the Elsinore fault located approximately 2 miles west of the site. Other nearby active faults are the Glen Ivy branch of the Elsinore fault, the Julian branch of the Elsinore fault, San Jacinto fault, the Anza branch of the Elsinore fault, and the Chino-Central Avenue fault located approximately 7.5 miles northwest, 20 miles southeast, 20 miles northeast, 21 miles east, and 25 miles north of the site, respectively (EZ-FRISK V 7.62).

6.2 Seismicity

As with all of southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 4.0 within a radius of 60 miles of the site are depicted on Figure 4, Regional Seismicity Map. A number of earthquakes of moderate to major magnitude have occurred in the southern California area within the last 100 years. A partial list of these earthquakes is included in Table 6.2, below.

TABLE 6.2
LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	17	NE
Near Redlands	July 23, 1923	6.3	28	NE
Long Beach	March 10, 1933	6.4	42	NW
North San Diego County	March 25, 1937	6.0	57	S
Desert Hot Springs	December 4, 1948	6.0	54	SE
Pinto Mountain	May 2, 1949	5.8	93	E
Arroyo Salada	March 19, 1954	6.4	64	SE
Borrego Mountain	April 9, 1968	6.5	69	SE
Borrego Springs	April 28, 1969	5.8	54	SE
Palm Springs	April 23, 1992	6.1	58	E
Landers	June 28, 1992	7.3	62	NE
Big Bear	June 28, 1992	6.4	48	NE
Hector Mine	October 16, 1999	7.1	88	E

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Estimation of Peak Ground Accelerations

The seismic exposure of the site may be investigated in two ways. The deterministic approach recognizes the Maximum Earthquake, which is the theoretical maximum event that could occur along a fault. The deterministic method assigns a maximum earthquake to a fault derived from formulas that correlate the length and other characteristics of the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion and is calculated by consideration of risk contributions from regional faults.

6.3.1 Deterministic Analysis

Table 1, attached at the end of this report, shows known faults within a 60 mile radius of the site. The maximum earthquake magnitude is indicated for each fault. In order to measure the distance of known faults to the site, the computer program *EQFAULT*, (Blake, 2000), was utilized. Principal references used within *EQFAULT* in selecting faults to be included are Jennings (1994), Anderson (1984) and Wesnousky (1986). For this investigation, the ground motion generated by maximum earthquakes on each of the faults is assumed to attenuate to the site per the attenuation relation by Campbell and Bozorgnia (1997 Revised). The resulting calculated peak horizontal accelerations at the site are shown on the attached Table 1. These values are one standard deviation above the mean.

Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 6.8 event on the Elsinore fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.82g. This value is provided as geologic background information. The code specified peak ground acceleration in Section 6.4 is used to calculate seismic and liquefaction settlement, for evaluation of seismic lateral earth pressures, and for structural design.

While listing of 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.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on any of the faults referenced above or other faults in southern California. With respect to seismic shaking, the site is considered comparable to the surrounding developed area.

6.3.2 Probabilistic Analysis

The computer program *FRISKSP* (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance for given horizontal accelerations for each line source. Geologic parameters not included 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 closest distance from the site to the rupture zone.

Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault.

After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Campbell and Bozorgnia (1997 Revised) were utilized in the analysis.

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,500 years. According to 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the design of critical structures such as schools and hospitals. The Design-Basis Earthquake Ground Motion (DBE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years. The DBE is typically used for the design of non-critical structures.

Based on the computer program *FRISKSP* (Blake, 2000), the MCE and DBE is expected to generate ground motions at the site of approximately 1.02g and 0.72g, respectively. Graphical representation of the analysis is presented on Figure 5.

6.4 Seismic Design Criteria

6.4.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 6.4.1 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 building structure and improvements should be designed using a Site Class C. 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 6.4.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.4.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.220g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.895g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.3	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.220g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.163g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.480g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.775g	Section 1613.3.4 (Eqn 16-40)

6.4.2 Table 6.4.2 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 6.4.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.879g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.879g	Section 11.8.3 (Eqn 11.8-1)

6.4.3 Conformance to the criteria in Tables 6.4.1 and 6.4.2 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.

6.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of Division of Mines and Geology (DMG) Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soil below the water table is composed primarily of poorly consolidated, fine to medium-grained sand. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

According to the Riverside County Land Information System, 2003, the site is located within an area of moderate liquefaction potential based on the underlying soil deposits. However, as stated previously, the Unnamed Sandstone and Pauba Sandstone underlying the site are composed of dense, cemented sandstone and siltstone. Provided the recommendations for remedial grading operations presented herein are followed, and based on the shallow depth to the dense, competent bedrock, it is our opinion that the potential for liquefaction of the site soils is very low. Liquefaction is not a design consideration for the project.

6.6 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. Based on the shallow depth to bedrock at the site and the recommended remedial grading, appreciable seismically-induced settlements are not anticipated.

6.7 Landslides

The site has relatively low-lying hills with intervening drainages. We did not observe any evidence of large scale slope stability issues on the site. We did observe localized surficial failures along the existing streams due to undercutting of minor stream channels. We did not observe evidence of slope failures on the aerial photographs reviewed for this study. Therefore, it is our opinion that the potential for slope instability at the site is considered low. Localized surficial slope failures along the drainages are likely to occur until the site is graded and developed.

6.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. There are no water-retaining structures up gradient from the site. Therefore, the probability of earthquake-induced flooding is considered very low.

6.9 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. And the site is located nearly 5 miles away from and at a higher elevation than Lake Elsinore. The potential for flooding from a seismically induced seiche is considered is low.

The potential for flood hazards at the site is considered low. The site is in Federal Emergency Management Agency (FEMA) Zone X per Flood Insurance Rate Map Panel 06065C2705G dated August 28, 2008.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is located within an area that is considered susceptible to subsidence per Riverside County. The site is near The Colony which experienced significant subsidence in the late 1980's and early 1990's where alluvium over granitic bedrock became saturated and settled after residential and golf course irrigation began. The subject site conditions and recommended remedial grading measures (removal of alluvium) will not result in the same conditions as at The Colony. Therefore, the potential for ground subsidence at the site is considered low once the recommendations in this report have been implemented.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude development of the site provided the recommendations presented herein are followed and implemented during design and construction. This report should be considered "preliminary" and a more detailed, design level geotechnical study will be required in order to verify the suitability of the preliminary geotechnical design parameters presented herein once development plans become available.

7.1.2 The faulting encountered does not appear to be active based on the subsurface investigation. Therefore, no building setback zones due to surface fault rupture are recommended for the site at this time.

7.1.3 We encountered younger alluvium and colluvium overlying Pauba Sandstone and Unnamed Sandstone within the site. Isolated areas of undocumented fill were also observed comprising the earthen dam. In addition, the fault trenches excavated for this report were loosely backfilled without testing and observation and are classified as undocumented fill. It is our opinion that the undocumented fill, younger alluvium, and colluvium are not suitable for direct support of proposed foundations or slabs. The fill, alluvium, and colluvium are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.3).

7.1.4 Remedial excavations on the order of 2 to 10 feet in depth are anticipated to be required. Deeper excavations should be conducted as necessary to completely remove all existing undocumented fill, alluvium, and colluvium, and other unsuitable soil encountered during grading operations at the direction of the Geocon representative. Geocon should review site development plans once they become available to determine if the recommendations in this report are applicable and to provide any additional recommendation which may be necessary. General recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.3).

7.1.5 Subsequent to the recommended grading, the structures may be supported on conventional foundation systems deriving support in the newly placed engineered fill or bedrock units (Pauba and/or Unnamed Sandstones).

7.1.6 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.7 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the undisturbed bedrock units found at or below a depth of 2 feet below the existing grade, and should be deepened as necessary to maintain a minimum 12 inch embedment into the undisturbed bedrock units. If the soil exposed in the excavation bottom is soft or loose, compaction of the soil will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.1.8 Once the design and foundation loading configuration for the proposed development proceeds, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.9 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Soil and Excavation Characteristics

7.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in dimension) may be generated with the granitic rock materials that can be incorporated into landscape use or deep compacted fill areas, if available.

7.2.2 The soil encountered in the field investigation is considered to be “expansive” (expansion index [EI] of greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 7.2.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a “very low” to “low” expansion potential (expansion index of 50 or less).

TABLE 7.2.2
EXPANSION 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	

7.2.3 We performed laboratory tests on samples of the site soil to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site soil at the locations tested possess “Not Applicable” and “S0” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. Table 7.2.3 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 7.2.3
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

7.2.4 We tested samples for potential of hydrogen (pH) and resistivity laboratory tests to aid in evaluating the corrosion potential to subsurface metal structures. The soil is classified as highly corrosive to metallic components. The laboratory test results are presented in Appendix B.

7.2.5 Geocon does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer should be performed if improvements susceptible to corrosion are planned.

7.3 Grading

7.3.1 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

7.3.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill, alluvium, and colluvium encountered during exploration is suitable for re-use as engineered fill, provided oversize material (rocks greater than 6 inches in diameter) are not placed in areas of future utilities or in areas within three feet of finish grades and all deleterious debris is removed.

7.3.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.). Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soil. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer. Any existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

7.3.4 Due to the preliminary nature of the project at this time, the grading recommendations should also be considered preliminary. Once information regarding existing and proposed site elevations becomes available, the recommendations presented herein should be reviewed and revised if necessary.

7.3.5 As a minimum, all existing artificial fill, alluvium, and colluvium should be excavated and properly compacted for foundation and slab support. Where Pauba and Unnamed Sandstones are present at the ground surface, excavation of weathered bedrock on the order of one foot is anticipated. Where undocumented fill, alluvium, and colluvium are present, removals of up to approximately 10 feet should be anticipated. We anticipate that the deeper excavations will be required along the drainage channels. In addition, the fault trenches excavated as a part of the site investigation were loosely backfilled without testing and observation and will require re-excavation and compaction. See the Geologic Map for locations of the fault trenches and the trench logs in Appendix C for trench depths. Deeper excavations should be conducted as necessary to completely remove all existing undocumented fill and unsuitable alluvium and colluvium at the direction of the Geocon representative. The anticipated depths of remedial grading are indicated adjacent to our trenches, borings and test pits on the Geologic Map, Figure 2.

7.3.6 Where excavation and compaction is to be conducted, the excavation should extend laterally a minimum distance of five feet beyond building footprint areas or for a distance equal to the depth of fill below the foundation, whichever is greater. Appurtenances, such as patio or canopy footings and other improvements that are adjacent to or structurally connected to the buildings should also be included in the required lateral over-excavation.

7.3.7 Building pads graded with a cut/fill transition will require undercutting to reduce the potential for differential settlement. The cut portion of the cut/fill transition should be undercut to a depth of at least 3 feet and replaced with properly compacted low expansive fill. In areas where a steep transition exists, additional removal will be required such that the maximum fill differential across any one building pad will be less than H/4, where H is the maximum fill thickness.

7.3.8 All fill and backfill soil should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted. Fill shall be compacted to a minimum 90 percent of the maximum dry density per ASTM International (ASTM) D1557 (latest edition).

7.3.9 Where new paving is to be placed, it is recommended that all existing unsuitable soil be excavated and properly compacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.9).

7.3.10 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will not be structurally tied into the proposed building, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. If the soil exposed in the excavation bottom is soft or loose, compaction of the soil will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.3.11 Utility trenches should be properly backfilled in accordance with the requirements of the latest edition of the Standard Specifications for Public Works Construction (Greenbook). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.3.12 Jetting of backfill should only be performed where trench sidewalls have an SE of 15 or greater to allow the water to dissipate and prevent future settlement. Geotechnical laboratory testing of the sidewall soil should be performed in areas where jetting is considered to verify acceptable sand equivalent values are present within the trench excavation.

7.3.13 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soil used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soil (see Figure B4), and shear strength properties equal to or greater than site soils. Import soil placed in the building area should be placed uniformly or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.3.14 All excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel or concrete.

7.4 Shrinkage

7.4.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of approximately 5 and 12 percent should be anticipated when excavating and compacting the existing alluvium, approximately 0 to 5 percent shrinkage for the existing colluvium, and approximately 0 percent shrinkage for the sandstones when considering an average relative compaction of 92 percent. 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. Please note that these estimates are for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate these variations.

7.5 Foundation Design

7.5.1 Subsequent to the recommended grading, the proposed structures may be supported on a conventional foundation system deriving support in newly placed engineered fill or the competent Pauba or Unnamed Sandstone at or below a depth of 2 feet.

7.5.2 Continuous footings may be designed for an allowable bearing capacity of 2,500 pounds per square foot (psf), and should be a minimum of 18 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.4 The soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively. In order to minimize static settlement of the proposed foundations, a maximum allowable soil bearing value of 4,000 psf should be utilized.

7.5.5 The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.5.6 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.5.7 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.5.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.5.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

7.5.10 Excavations within the cobble layers may result in irregular surfaces. Where rocks are removed from foundation excavations, such as for swimming pools, if present, and voids are generated, the void space should be filled with concrete during the foundation pour. Backfilling of void spaces with soil is not permitted.

7.5.11 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.5.12 The maximum expected static settlement for structures supported on a conventional foundation system deriving support in engineered fill is estimated to be less than ½ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a horizontal distance of twenty feet.

7.5.13 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.6 Miscellaneous Foundations

7.6.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 2 feet. Remedial grading for miscellaneous foundations should be individually reviewed by a representative of Geocon and additional recommendations provided as needed.

7.6.2 If the soil exposed in the excavation bottom is soft, compaction of the soft soil will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.6.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.7 Lateral Design

7.7.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces in properly compacted engineered fill, undisturbed alluvial soil, or Pauba sandstone.

7.7.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted fill, undisturbed alluvial soil, or Pauba sandstone may be computed as an equivalent fluid having a density of 300 pounds per cubic foot (pcf) with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.8 Concrete Slabs-on-Grade

7.8.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.9).

7.8.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4-inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.8.3 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for*

Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643-98 and the manufacturer's recommendations. If California Green Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of ½-inch clean aggregate and the vapor retarder should be in direct contact with the concrete slab. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel.

7.8.4 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soil without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.8.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.8.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

7.9 Preliminary Pavement Recommendations

7.9.1 Where new paving is to be placed, it is recommended that all existing artificial fill and soft or disturbed alluvium and colluvium be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all soft or unsuitable soil in the area of new paving is not required, however, paving constructed over existing unsuitable soil may experience increased settlement or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of soil should be scarified and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

7.9.2 The following pavement sections are based on an assumed R-Value of 30. Once site grading activities are complete, it is recommended that laboratory testing confirm the properties of the soils serving as paving subgrade prior to placing pavement. The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

TABLE 7.9.2
PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking & Driveways	Up to 5	3.0	5.5
Trash Truck & Fire Lanes	7	4.0	9.5

7.9.3 Asphalt concrete should conform to Section 203-6 of the Greenbook. Class 2 aggregate base should conform to Section 26-1.02A of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans). Crushed Miscellaneous Base should conform to Section 200-2.4 of the Greenbook.

7.9.4 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles at the ground surface, it is recommended that the concrete be a minimum of 5 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to at least 95 percent relative compaction as determined by ASTM Test Method D1557 (latest edition).

7.9.5 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.10 Swimming Pool/Spa

7.10.1 If swimming pools or spas are planned, the proposed swimming pool shell bottom should be designed as a free-standing structure and may derive support in newly placed engineered fill or the competent native sandstone found at or below a depths of between 2 and 10 feet. It is recommended that uniformity be maintained beneath the proposed swimming pools where possible. However, swimming pool foundations may derive support in both engineered fill and undisturbed native sandstone. It is the intent of the Geotechnical Engineer to allow swimming pool foundation systems to derive support in both the competent undisturbed sandstone and newly placed engineered fill as necessary.

7.10.2 Swimming pool foundations and walls may be designed in accordance with the *Conventional Foundation Design* and *Retaining Wall Design* sections of this report (See Sections 7.5 and 7.11). A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

7.10.3 If a spa is proposed it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

7.10.4 If the proposed pool is in proximity to the proposed structure, consideration should be given to construction sequence. If the proposed pool is constructed after building foundation construction, the excavation required for pool construction could remove a component of lateral support from the foundations and would therefore require shoring. Once information regarding the pool location and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 7 feet. In the event that walls significantly higher than 7 feet are planned, Geocon should be contacted for additional recommendations.

7.11.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.5).

7.11.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 40 pcf.

7.11.4 Restrained walls are those that are not 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, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 66 pcf.

7.11.5 These pressures assume non-expansive granular soil is placed as the wall backfill. If expansive, or fine grained soils are used, Geocon should be contacted to provide additional recommendations.

7.11.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 83 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.11.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. In addition, seismic lateral forces presented below should be incorporated into the design as necessary.

7.11.8 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 35H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.88g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.12 Retaining Wall Drainage

7.12.1 Retaining walls should be provided with a drainage system extending at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 6). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.12.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 6). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a one-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.12.3 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.13 Elevator Pit Design

7.13.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Retaining Wall Design* section of this report (see Section 7.11).

7.13.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project, if necessary.

7.13.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.12).

7.13.4 It is suggested that the elevator pit walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.14 Elevator Piston

7.14.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction.

7.14.2 Casing may be required if caving is experienced in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.14.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.15 Temporary Excavations

7.15.1 The excavations are expected to expose fill, alluvium, colluvium and dense native soil which are suitable for vertical excavations up to five feet where loose soil or caving sand is not present, or where not surcharged by adjacent traffic or structures.

7.15.2 Vertical excavations greater than five feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation.

7.15.3 It is anticipated that sufficient space is available to complete the required earthwork for this project using sloping measures. Where sufficient space is available, temporary unsurcharged embankments up to 10 feet in height may be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion.

7.15.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's competent person should inspect the soil exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur in accordance with Occupational Safety and Health Administration (OSHA) requirements. All excavations should be stabilized within 30 days of initial excavation.

7.16 Surface Drainage

7.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the supporting soil can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. 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, drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers is not recommended onto unprotected soil within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the engineered fill providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

7.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. Building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.16.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement’s subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.17 Plan Review

7.17.1 Grading and foundation should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

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 West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

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.

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Date	Photograph Number	Scale
1/28/62	1-68/1-69	1” = 2000’
5/4/80	829 not in stereo	1” = 2000’
12/15/83	557/558	1” = 2000’
11/25/90	15-17/15-18	1” = 1600’
1/29/95	15-13/15-14/15-15	1” = 1600’
3/18/00	15-16/15-17	1” = 1600’
4/14/05	15-15/15-16	1” = 1600’
4/2/10	15-16/15-17	1” = 1600’

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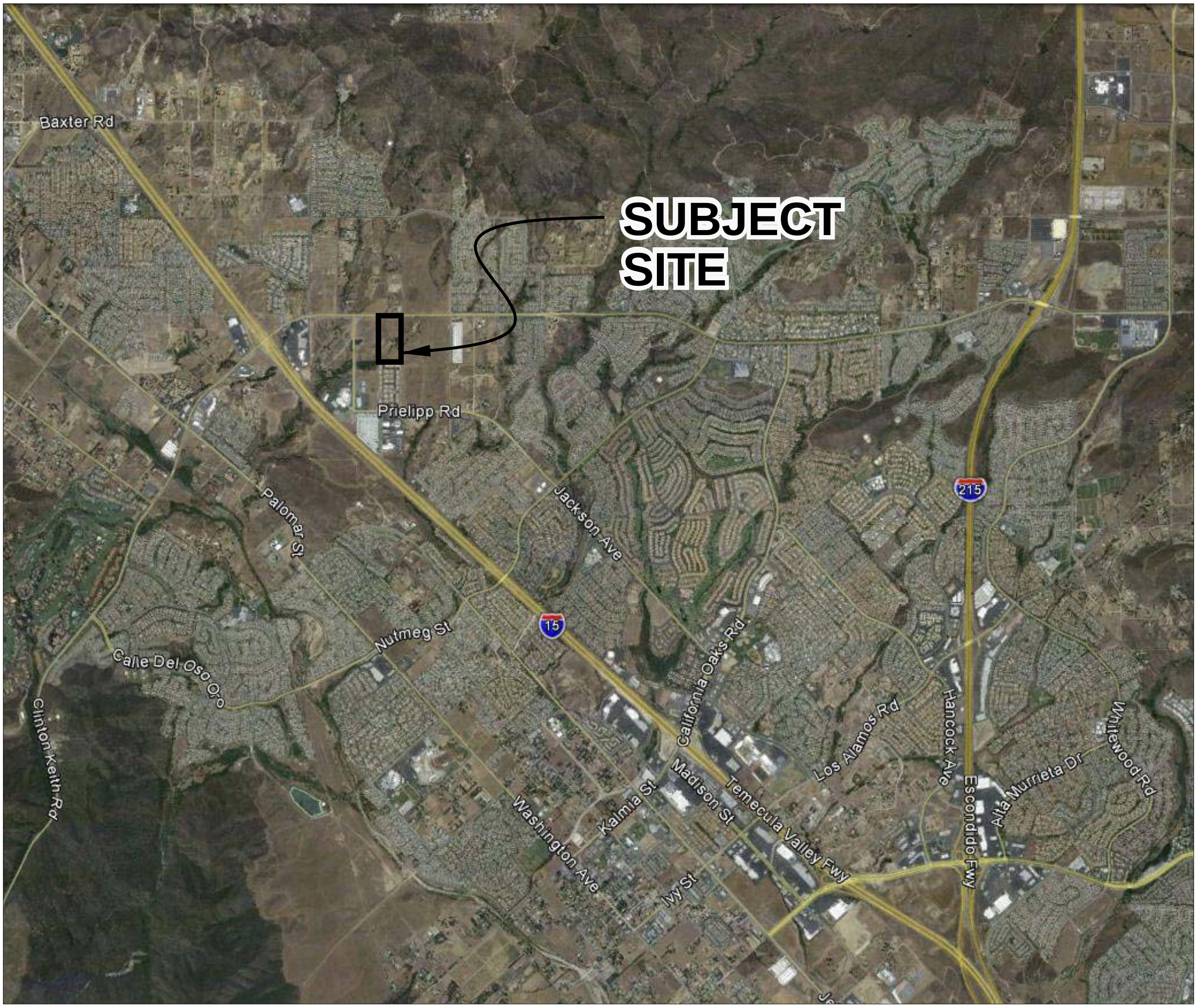

REFERENCE: GOOGLE EARTH, 2012

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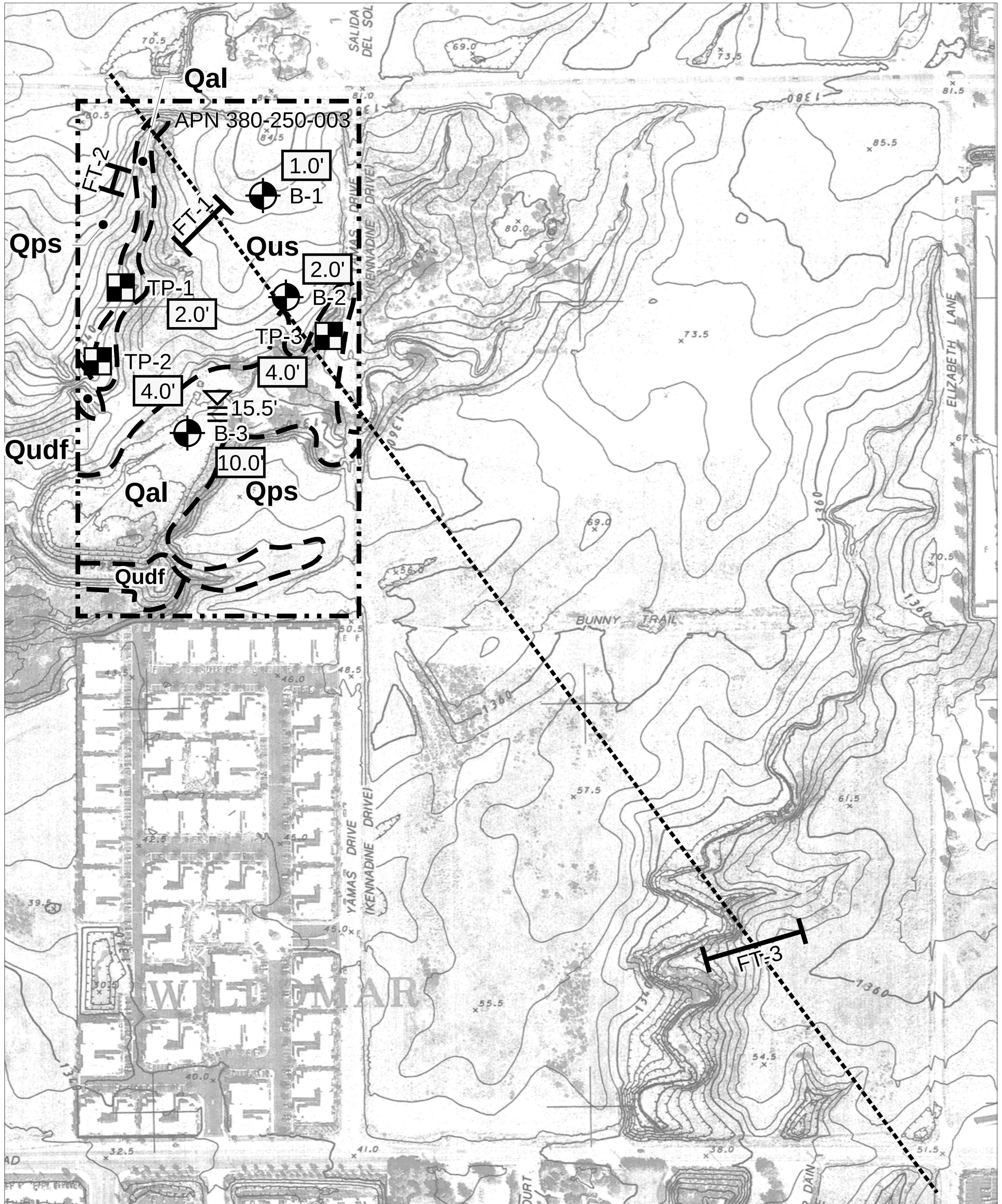
JL		2000

VICINITY MAP

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 1



LEGEND

B-3	Approximate Location of Geotechnical Boring
TP-3	Approximate Location of Geotechnical Test Pit
FT-3	Approximate Location of Geologic Fault Trench
	Mapped Trace of Unnamed Riverside County Fault
15.5'	Depth to Groundwater (this investigation)
10.0'	Depth of Recommended Removals
Qudf	Undocumented Fill
Qal	Alluvium
Qps	Pauba Sandstone
Qus	Unnamed Sandstone
	Geologic Contact
	Site Boundary

0 300' 600'

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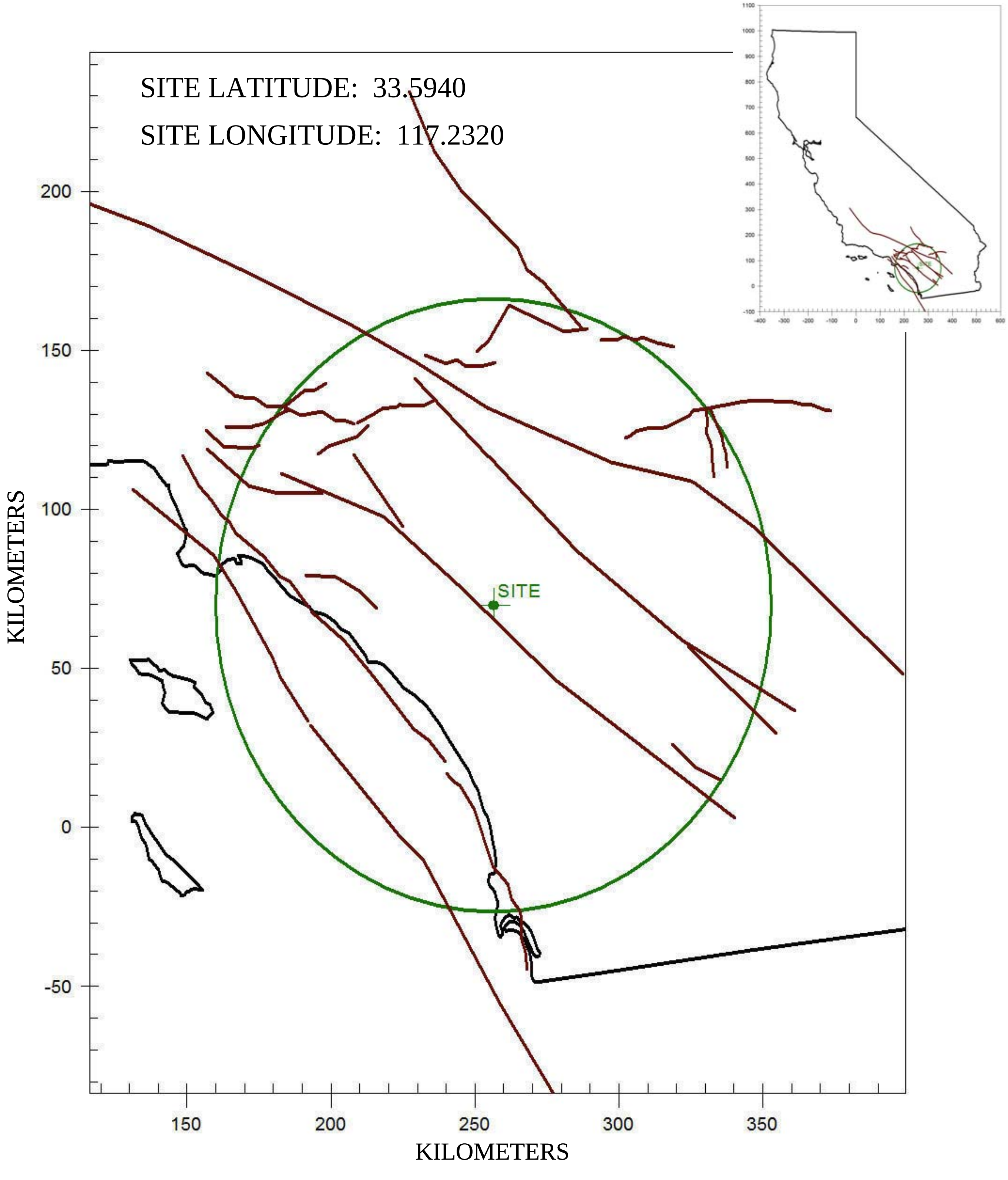
JL		2000

GEOLOGIC MAP

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 2



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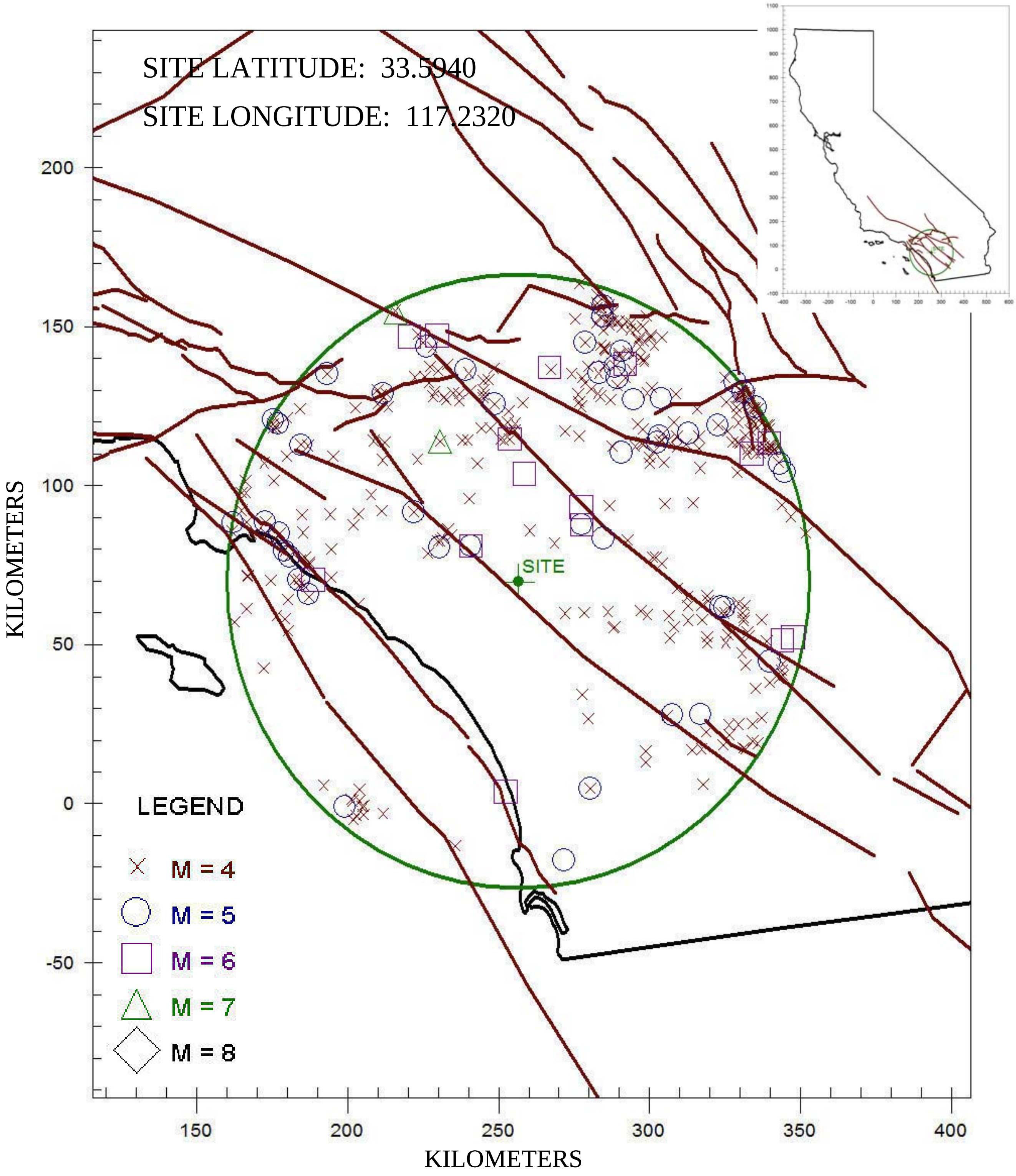
REGIONAL FAULT MAP

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

CHL / JL		2000	FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 3



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REGIONAL SEISMICITY MAP

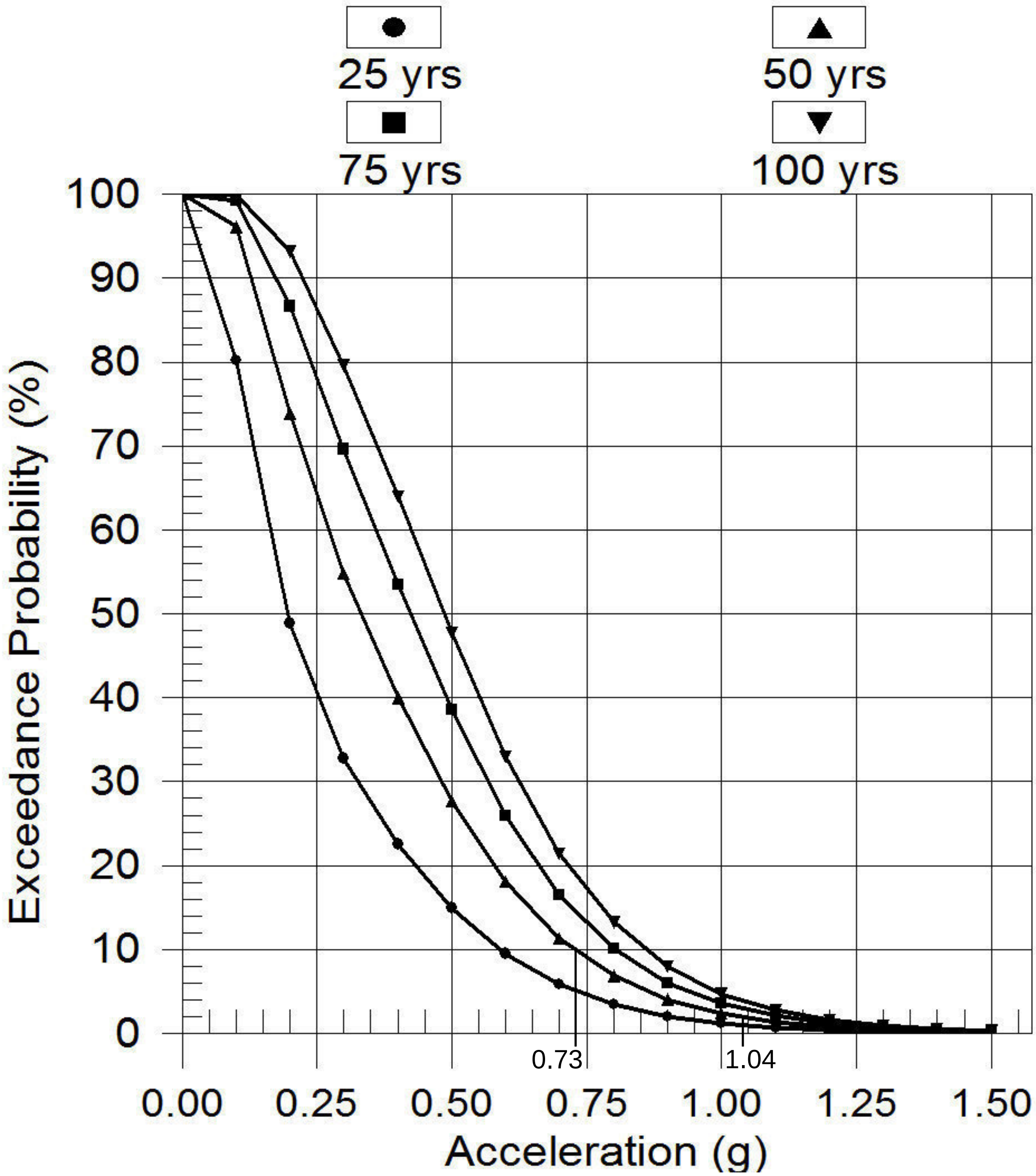
GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

CHL / JL		2000

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 4

PROBABILITY OF EXCEEDANCE

CAMP. & BOZ. (1997 Rev.) SR 1



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PROBABILITY OF EXCEEDANCE

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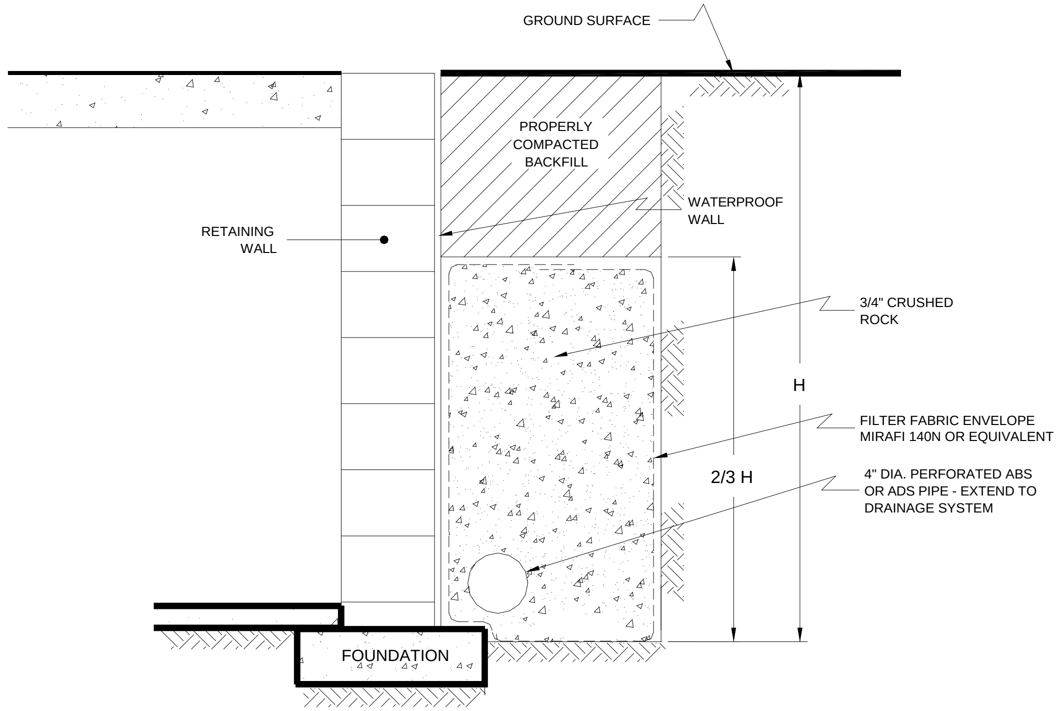
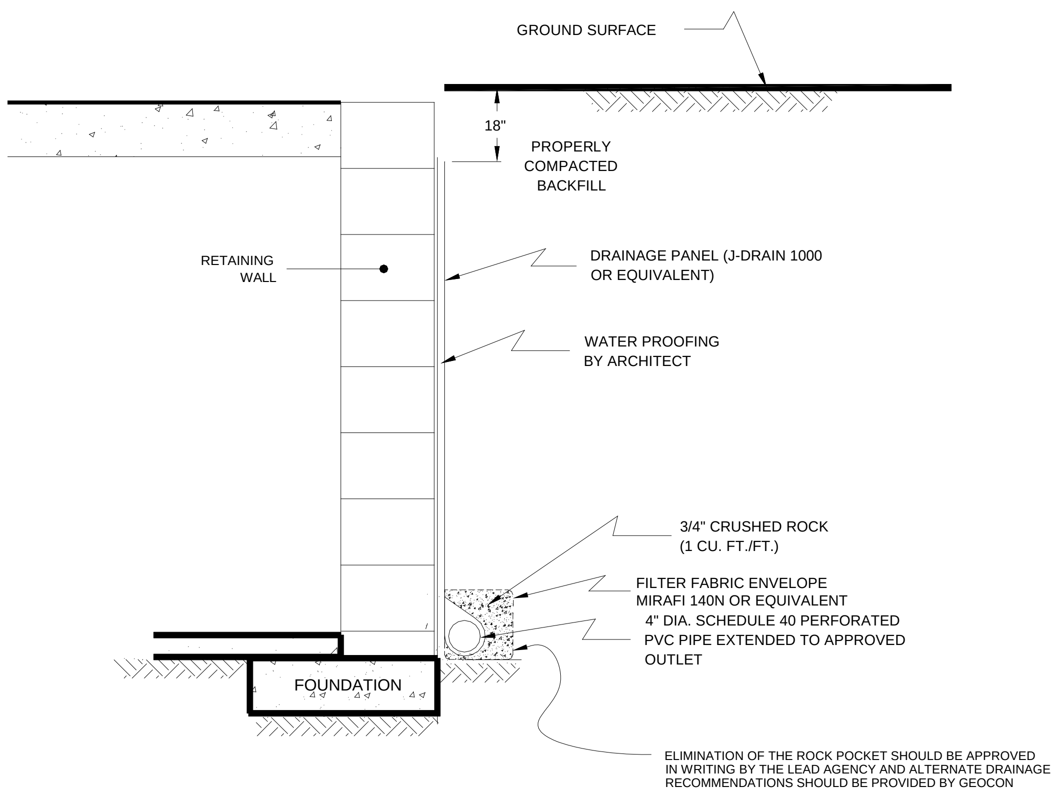
FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 5





NO SCALE

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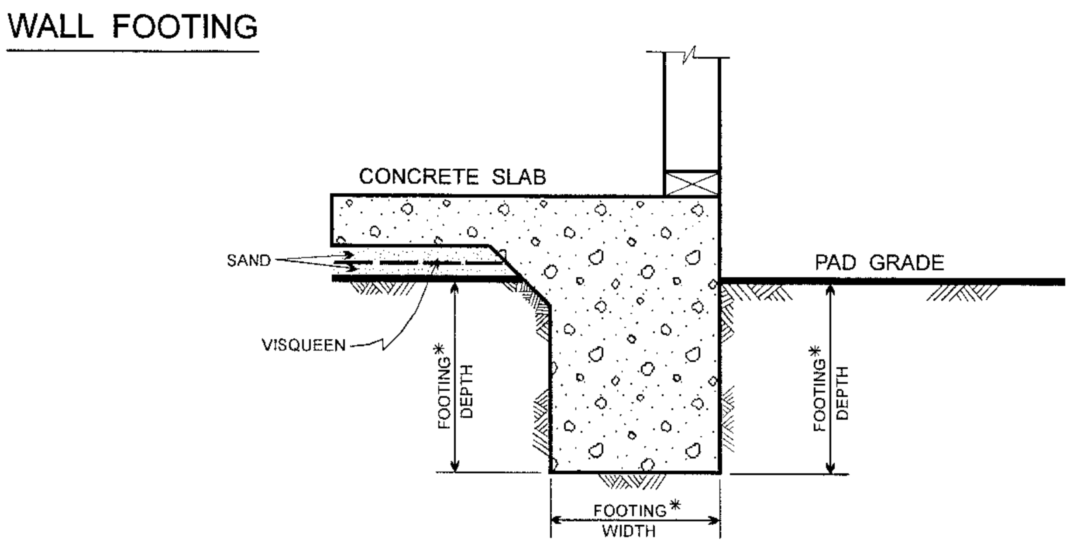
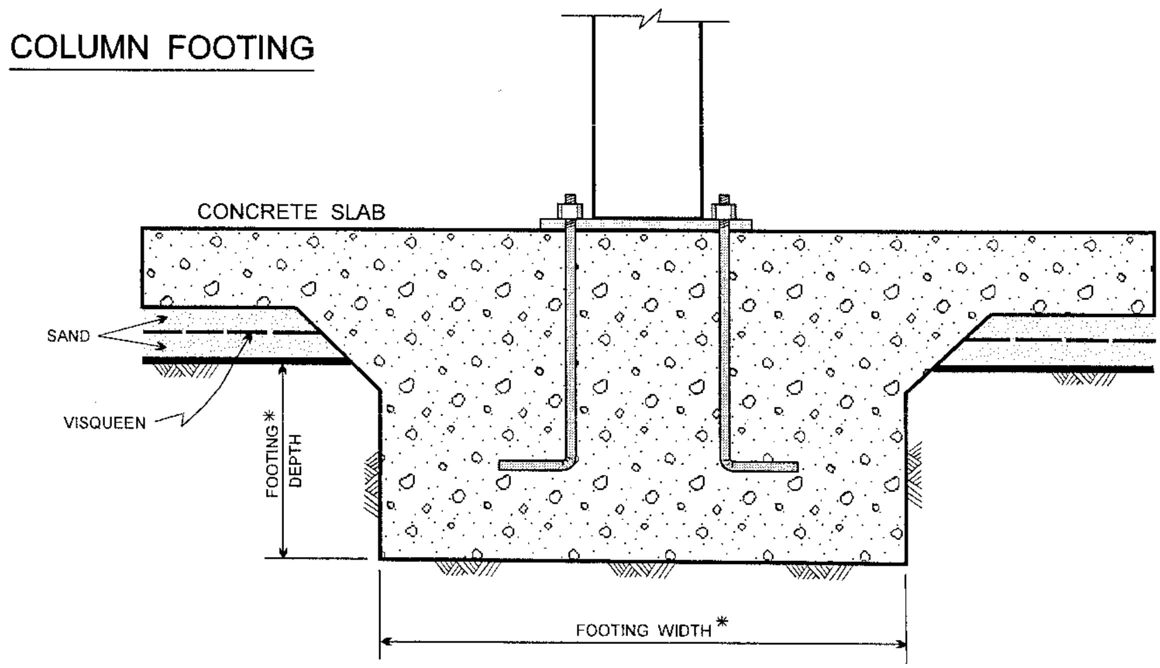
RETAINING WALL DRAIN DETAIL

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

JL		2000	FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 6





*……SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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KC / KC

WALL/COLUMN FOOTING DIMENSION DETAIL

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. 7

GEOCON

TABLE 1
FAULTS WITHIN 60 MILES OF THE SITE
DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELSINORE (TEMECULA)	2.9(4.6)	6.8	0.819	XI
ELSINORE (GLEN IVY)	7.6(12.2)	6.8	0.477	X
ELSINORE (JULIAN)	19.8(31.9)	7.1	0.218	VIII
SAN JACINTO-SAN JACINTO VALLEY	20.1(32.4)	6.9	0.188	VIII
SAN JACINTO-ANZA	20.8(33.5)	7.2	0.221	IX
CHINO-CENTRAL AVE. (Elsinore)	25.4(40.8)	6.7	0.132	VIII
SAN JOAQUIN HILLS	26.5(42.6)	6.6	0.117	VII
NEWPORT-INGLEWOOD (Offshore)	29.0(46.6)	7.1	0.141	VIII
SAN JACINTO-SAN BERNARDINO	29.1(46.9)	6.7	0.103	VII
WHITTIER	29.5(47.4)	6.8	0.111	VII
ROSE CANYON	34.5(55.6)	7.2	0.123	VII
SAN ANDREAS - SB-Coach. M-1b-2	35.4(56.9)	7.7	0.170	VIII
SAN ANDREAS - SB-Coach. M-2b	35.4(56.9)	7.7	0.170	VIII
SAN ANDREAS - San Bernardino M-1	35.4(56.9)	7.5	0.148	VIII
SAN ANDREAS - Whole M-1a	35.4(56.9)	8.0	0.208	VIII
NEWPORT-INGLEWOOD (L.A.Basin)	39.8(64.1)	7.1	0.093	VII
SAN JACINTO-COYOTE CREEK	42.9(69.0)	6.6	0.054	VI
PUENTE HILLS BLIND THRUST	43.2(69.5)	7.1	0.080	VII
PINTO MOUNTAIN	43.4(69.8)	7.2	0.090	VII
CUCAMONGA	43.8(70.5)	6.9	0.067	VI
NORTH FRONTAL FAULT ZONE (West)	44.4(71.4)	7.2	0.083	VII
SAN JOSE	44.8(72.1)	6.4	0.044	VI
CORONADO BANK	45.2(72.7)	7.6	0.119	VII
PALOS VERDES	46.1(74.2)	7.3	0.090	VII
CLEGHORN	46.9(75.5)	6.5	0.043	VI
EARTHQUAKE VALLEY	47.2(76.0)	6.5	0.043	VI
SIERRA MADRE	47.7(76.8)	7.2	0.074	VII
SAN ANDREAS - Coachella M-1c-5	49.5(79.6)	7.2	0.074	VII
NORTH FRONTAL FAULT ZONE (East)	50.2(80.8)	6.7	0.046	VI
SAN ANDREAS - Mojave M-1c-3	50.3(81.0)	7.4	0.086	VII
SAN ANDREAS - Cho-Moj M-1b-1	50.3(81.0)	7.8	0.121	VII
SAN ANDREAS - 1857 Rupture M-2a	50.3(81.0)	7.8	0.121	VII
BURNT MTN.	53.9(86.7)	6.5	0.035	V
EUREKA PEAK	57.2(92.0)	6.4	0.030	V
HELENDALE - S. LOCKHARDT	57.5(92.5)	7.3	0.065	VI
CLAMSHELL-SAWPIT	58.7(94.4)	6.5	0.031	V
RAYMOND	59.0(95.0)	6.5	0.031	V
UPPER ELYSIAN PARK BLIND THRUST	59.6(95.9)	6.4	0.028	V

38 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE ELSINORE (TEMECULA) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.9 MILES (4.6 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8192 g

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on October 25 through November 16, 2012. We excavated 184 lineal feet of fault trench within APN 380-250-003 (FT-1 & FT-2) and 225 lineal feet of fault trench within APN 380-250-023 (FT-3). FT-1 was 4 to 6.5 feet deep, FT-2 was generally 5 feet deep, and FT-3 was 4 to 15 feet deep. The trenches were excavated utilizing a rubber-tire backhoe. Where the depth exceeded 5 feet they were benched to provide a general slope of 1:1 (horizontal:vertical) in accordance with Cal OSHA requirements. The trenches were geologically logged by a Certified Engineering Geologist from our firm and were loosely backfilled with soil cuttings. Trench Logs are presented in Appendix C and trench locations are depicted on the Geologic Map, Figure 2. We contacted the City of Wildomar to give them the opportunity to review the excavations prior to backfill. They indicated it was not necessary and would rely on our report.

The borings were excavated with a CME 75 truck mounted drill rig. Borings B-1 through B-3 were excavated to depths between 10.5 and 50.5 feet. Representative and relatively undisturbed samples were obtained by driving a 3 inch O. D., California Modified Sampler into the “undisturbed” soil mass with blows from an above-ground auto-hammer. The sampler was equipped with 1-inch by 2⅜-inch brass sampler rings to facilitate removal and testing. Bulk samples were also obtained. Standard Penetrometer (SPT) samples were alternated with California ring samplers in areas where ground water was encountered (B-3). SPT soil samples were bagged, sealed, and transported to our laboratory for testing. The soil conditions encountered in the excavations were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-3. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are indicated on the Geologic Map (see Figure 2).

Geotechnical test pits were excavated in areas that were inaccessible with the drill rig, typically within the drainage areas. Test pits TP-1 through TP-3 were excavated to depths of 2.5 to 8 feet deep and the encountered soil conditions were logged by a Geologist or Engineer from our firm. The excavations were loosely backfilled immediately after logging. The Test pit locations are indicated on the Geologic Map, Figure 2 and the logs are presented in Appendix A, Figures A-4 through A-6.

BORING B-1

ELEV. (MSL.) **1376** DATE COMPLETED **11/7/12**

EQUIPMENT **CME 75 HSA** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1@0-5				**UNNAMED SANDSTONE (Qus):** Clayey SANDSTONE, reddish yellow with white (sand grains), fine to coarse grained, massive, hard,			
2								
	B1@2.5					53/6"	134	6.4
4					Silty SANDSTONE, poorly graded, very dense, slightly moist, reddish brown, fine to medium grained, some coarse grained sand, trace gravel, well cemented			
	B1@5					50/5"	91.5	17.55
6					-slow advance -refusal, moved hole 2'			
8	B1@7.5				Sandy SILTSTONE, hard, slightly moist, olive, trace black stains, fine grained sand, well cemented	50/6"	114.7	18.1
10	B1@10				Silty SANDSTONE, poorly graded, very dense, slighlty moist, olive, fine grained, well cemented	50/5"	101.9	12.7
12					-gravel layer approximately 1' thick			
	B1@12.5				-becomes well graded, fine to coarse grained, trace gravel	50/4"		
14					-gravel layer approximately 1' thick			
	B1@15				-slow advance	50/3"		
16								
18								
20	B1@20				-no recovery	50/1"		

Total depth: 20.25'
No groundwater encountered
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-1,
Log of Boring B-1, Page 1 of 1

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B-2

ELEV. (MSL.) **1360** DATE COMPLETED **11/13/12**

EQUIPMENT **CME 75 HSA** BY: **JL**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B2@0-5			SM	**ALLUVIUM (Qal):** Silty SAND, poorly graded, medium dense, dry, light reddish brown, medium to coarse grained, debris at upper 1'			
2	B2@2.5				**UNNAMED SANDSTONE (Qus):** SANDSTONE, very dense, slightly moist, yellowish light brown and white with orange staining, coarse grained, weakly cemented, some gravel, some fine to medium grained sand	50/5"	122	5
4								
6	B2@5				-becomes moist, increase in fine grained sand, micaceous	50/3"	123.7	4.7
8	B2@7.5				-becomes fine to medium grained, increase in mica	50/3"	109.2	6.9
10	B2@10				-becomes medium to coarse grained	50/4"		

Total depth: 10.5'
No groundwater encountered
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-2,
Log of Boring B-2, Page 1 of 1

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2539-22-02A

BORING B-3

ELEV. (MSL.) **1332** DATE COMPLETED **11/13/12**

EQUIPMENT **CME 75 HSA** BY: **JL**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3@0-5			SP	**ALLUVIUM (Qal):** SAND with Gravel, poorly graded, loose, dry, yellowish light brown, coarse graind sand, fine gravel			
2	B3@2.5				-becomes light brown, medium to coarse grained sand, some fine grained sand	26	112	5.8
4				ML	Sandy SILT, firm, slightly moist, reddish brown, fine grained sand, micaceous			
	B3@5				-becomes wet, dark brown	11	112.3	13.1
6				SM	Silty SAND, poorly graded, loose, moist, grayish dark brown, fine grained, trace coarse grained sand			
8	B3@7.5				-becomes medium dense, mixed reddish brown, black and white, trace gravel	35	119.7	6.6
10	B3@10				**PAUBA SANDSTONE (Qps):** Sandy SILTSTONE, stiff, moist, yellowish light brown, some black veinlets, fine grained, trace medium grained sand, micaceous, weakly cemented	67	111.3	18.4
12					SANDSTONE with some Silt, well graded, dense, moist, yellowish light brown, fine to coarse grained, micaceous, trace fine gravel, weakly cemented			
14								
	B3@15		▼		-becomes very dense, wet, decrease in silt	50/6"	116.5	10.7
16								
18								
20	B3@20				-becomes poorly graded, medium to coarse grained, some fine grained sand, trace gravel	50/3"	121.6	15.6
22								
24								
	B3@25				-becomes mixed yellowish light brown, white and medium brown, some orange staining, water added to extract sampler	50/5"		
26								
28								
					Silty SANDSTONE, poorly graded, very dense, wet, mixed yellowish light brown, white and light reddish brown, fine to medium grained, some			

Figure A-3,
Log of Boring B-3, Page 1 of 2

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

BORING B-3

ELEV. (MSL.) 1332 DATE COMPLETED 11/13/12

EQUIPMENT CME 75 HSA BY: JL

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30				coarse grained sand, trace gravel, micaceous, weakly cemented	86/9"		
32								
34								
	B3@35				Sandy SILTSTONE, hard, wet, yellowish brown with black blebs, fine grained, micaceous, visible structure, weakly cemented	61		
36					Silty SANDSTONE, poorly graded, very dense, wet, mixed yellowish brown, black and white, fine to medium grained, some coarse grained sand, micaceous, weakly cemented			
38								
40	B3@40				SANDSTONE, poorly graded, very dense, wet, yellowish light brown, fine to medium grained, trace coarse grained, micaceous, weakly cemented	96/10"		
42					Silty SANDSTONE, poorly graded, very dense, wet, mixed yellowish light brown and light brown, fine to medium grained, trace fine gravel, micaceous, weakly cemented			
44								
	B3@45				-becomes light grayish brown, white and black	50/3"		
46								
48								
50	B3@50				-becomes light gray, black and white with some orange staining, fine grained, 1" lense of yellowish brown silt	50/4"		

Total depth: 50.5'
Groundwater encountered at 15.5'
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-3,
Log of Boring B-3, Page 2 of 2

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TEST PIT TP-1 ELEV. (MSL.) 1345 DATE COMPLETED 11/7/12 EQUIPMENT Backhoe BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qal):** Silty SAND, poorly graded, loose, dry to moist, brown, porous			
2					**UNNAMED SANDSTONE (Qus):** Silty SANDSTONE, dense, moist, yellow brown, coarse grained, locally massive, well cemented			
4								
					Total depth: 5' No groundwater encountered Backfilled with cuttings and tamped			

Figure A-4,
Log of Test Pit TP-1, Page 1 of 1

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TEST PIT TP-2 ELEV. (MSL.) 1344 DATE COMPLETED 11/7/12 EQUIPMENT Backhoe BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qal):** Silty SAND, poorly graded, loose, dry to moist, brown, porous			
2								
4					**UNNAMED SANDSTONE (Qus):** Silty SANDSTONE, dense, moist, yellow brown, coarse grained, locally massive, well cemented			
6								
8					Total depth: 8' No groundwater encountered Backfilled with cuttings and tamped			

Figure A-5,
Log of Test Pit TP-2, Page 1 of 1

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

TEST PIT TP-3

ELEV. (MSL.) **1349** DATE COMPLETED **11/7/12**

EQUIPMENT **Backhoe** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 4				SM	**ALLUVIUM (Qal):** Silty SAND, poorly graded, loose, dry to moist, brown, porous			
4 – 6					**UNNAMED SANDSTONE (Qus):** Silty SANDSTONE, dense, moist, yellow brown, coarse grained, locally massive, well cemented			

Total depth: 6'
No groundwater encountered
Backfilled with cuttings and tamped

Figure A-6,
Log of Test Pit TP-3, Page 1 of 1

T2539-22-02A BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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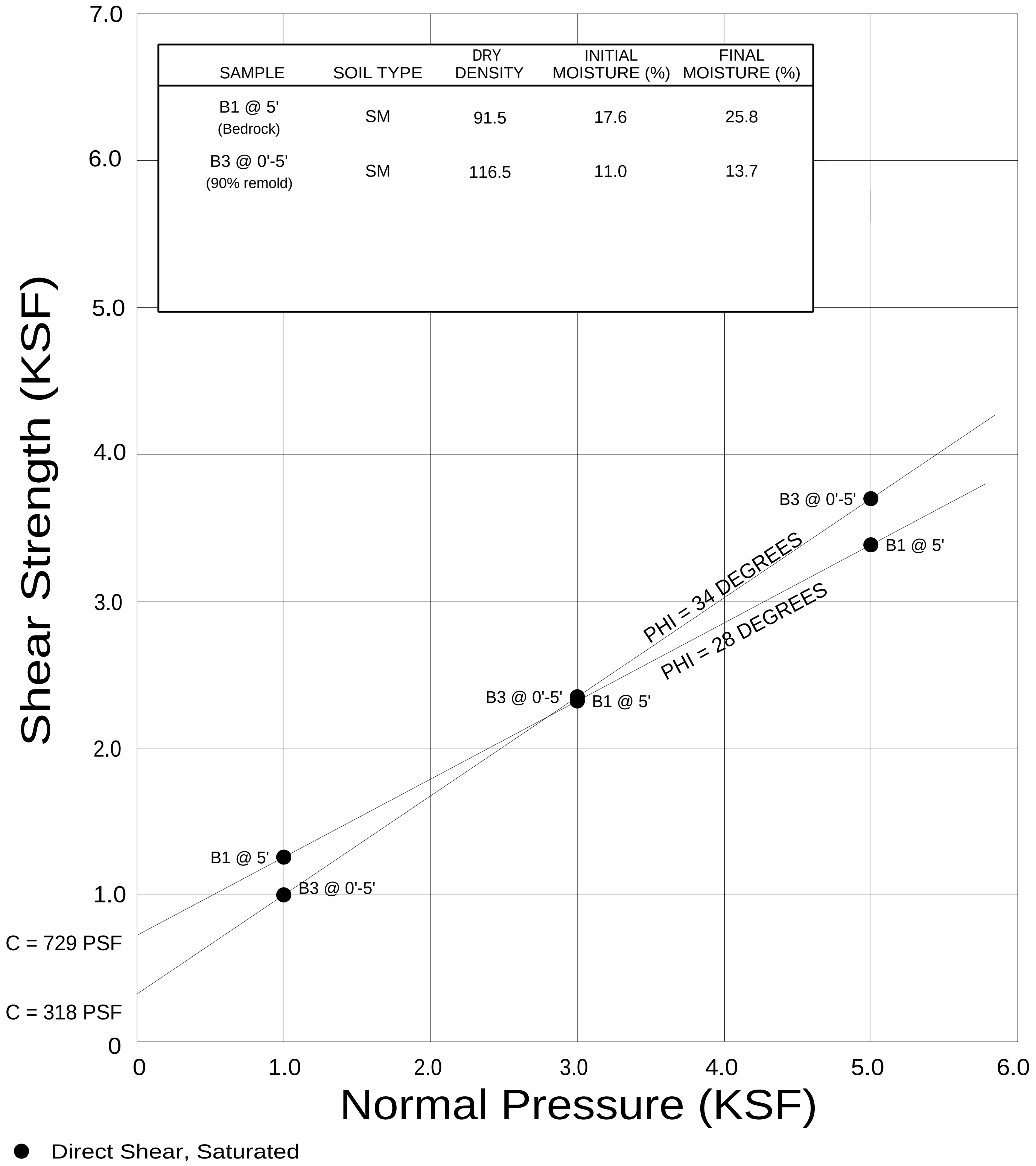
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of ASTM International (ASTM), or other suggested procedures. Selected samples were tested for direct shear strength, compaction characteristics, expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B4. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5' (Bedrock)	SM	91.5	17.6	25.8
B3 @ 0'-5' (90% remold)	SM	116.5	11.0	13.7

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JL		2000

DIRECT SHEAR TEST RESULTS

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. B1

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-08A

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 0'-5'	6.0	20.4	113.0	21	Low	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2010 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND
AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 0-5'	Yellowish Brown Silty Sand	123.5	10.5
B3 @ 0-5'	Dark Brown Sand with Silt	133.0	8.0

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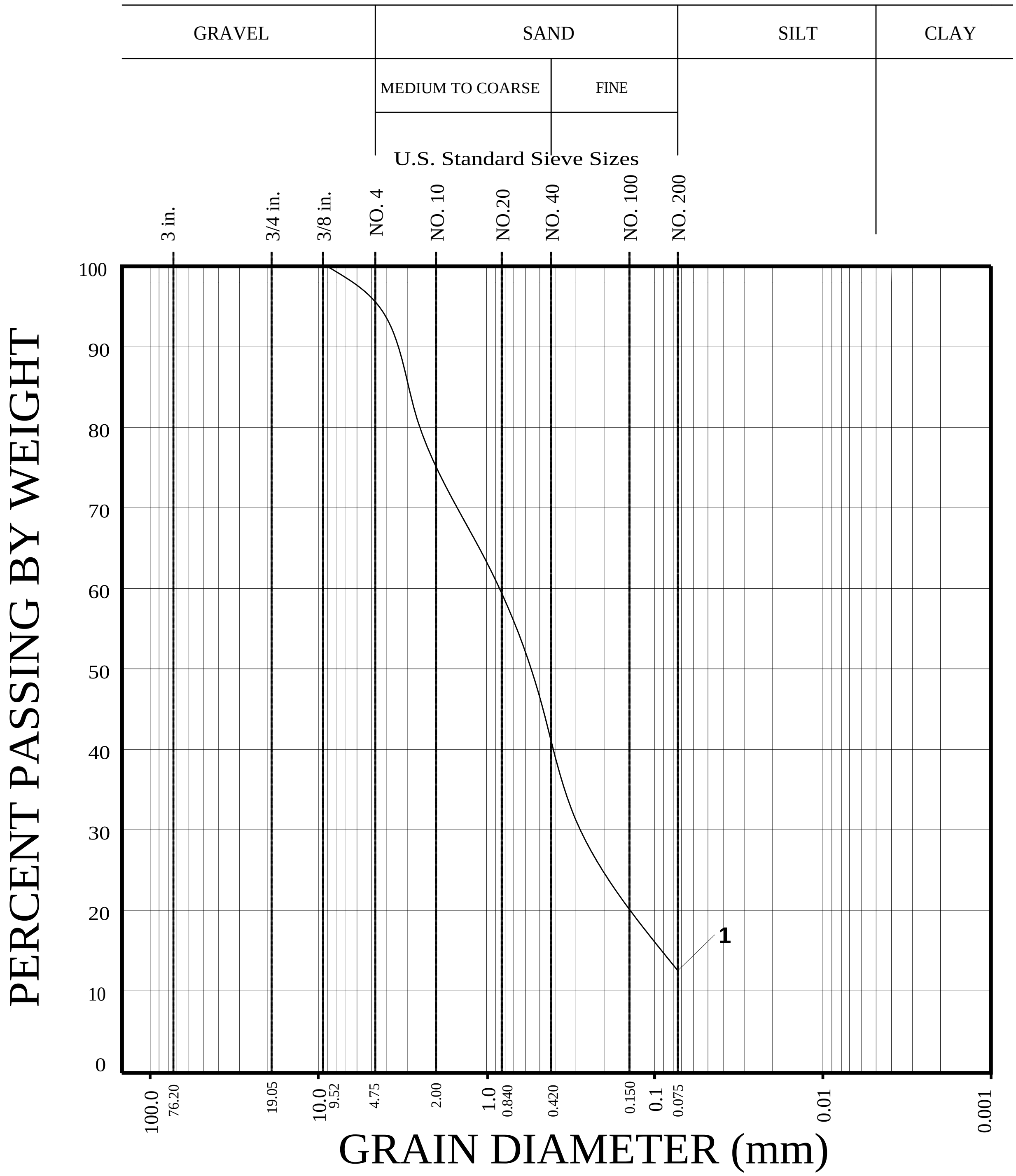
LABORATORY TEST RESULTS

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

JL		2000	FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. B2



SAMPLE	UNIFIED SOIL CLASSIFICATION
1 - B3 @ 25'	SP

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JL		2000

GRAIN SIZE DISTRIBUTION

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015	PROJECT NO. T2539-22-02A	FIG. B3

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0'-5'	8.04	1900 (Highly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS AASHTO T291-94

Sample No.	Chloride Ion Content (%)
B1 @ 0'-5'	0.009

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 0'-5'	0.042	Negligible

* Reference: 2010 California Building Code, Section 1904.3 and ACI 381 Section 4.3.

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JL | | 2000

CORROSIVITY TEST RESULTS

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015 | PROJECT NO. T2539-22-02 | FIG. B4

APPENDIX C

APPENDIX C

FAULT RUPTURE HAZARD INVESTIGATION

Geologic Review

Riverside County has depicted an unclassified fault crossing both APN 380-250-003 and 380-250-023 based on the Land Information System data base. The County has not established a fault hazard zone around this fault. It appears that this fault was taken from regional geologic mapping performed by Kennedy and Morton from their *Preliminary Geologic Map of the Murrieta 7.5' Quadrangle* (Version 1.0), see Figure 2, *Riverside County Fault Hazard Zones*. Kennedy did not include the site faults in his previous 1977 study.

In 2005, LGC performed a fault rupture hazard investigation as part of a preliminary geotechnical investigation on a neighboring site which is located, between APNs 380-250-003 and 380-250-023 (Riverside County GEO Report 001986). They reported the excavation of two fault trenches. Ten open caliche filled fractures within sandstone were observed in one of their trenches at the location of the mapped fault. However, the soil overlying the fractures was described as alluvium and no age-dating was provided to determine if the fractures were older than 11,000 years (thereby making the fault not active). The second fault trench was excavated northeast of the mapped fault and did not intercept the trace. The County Geologist issued a review letter to LGC on April 23, 2008 with several geologic and geotechnical questions regarding both their 2005 and 2006 reports. However, a response was never provided and GEO001986 remains an open file at the County.

The fault mapped on both parcels is in alignment with more prominent faulting projecting from the southeast within the City of Murrieta. We reviewed geologic reports prepared for the residential tracts located along Jackson Avenue to determine the age of faulting encountered to the southeast. Geologic studies to the southeast of the sites indicated mapped faults by Kennedy (1977) were present within the Unnamed Sandstone unit and were capped by unbroken Pauba Sandstone, therefore, they were deemed inactive (RMA, 1991). The fault located to the southeast of the site, mapped by Kennedy (1977), investigated by Pacific Soils (1987), and discussed in the RMA report (1991) was noticeable in aerial photographs as well as in ground surface expression, however, it was found to be inactive.

Furthermore, similar faulting mapped west of the site on the Oak Springs Ranch property, located approximately ½ mile west of parcel 003, was determined to be older and inactive (Hunt, 2005).

Lineament Analysis

In order to identify possible unmapped faults and to evaluate topographic expressions of published fault traces, Geocon performed a lineament analysis of the site. Aerial photographs obtained from Riverside

County Flood Control and Water Conservation District and Continental Aerial Photo were reviewed. The photographs covered the years 1962 through 2010 and were at scales ranging from 1 inch equals 1,600 feet to 1 inch equals 2,000 feet.

Lineaments were classified according to their development as strong, moderate or weak. A strong lineament is a well-defined feature, which can be continuously traced several hundred feet to a few thousand feet. A moderate lineament is less well defined, somewhat discontinuous and can be traced for only a few hundred feet. A weak lineament is discontinuous, poorly defined, and can be traced for a few hundred feet or less. We did observe a moderate lineament crossing both sites and extending to the northwest and southeast. The lineament associated with the mapped fault by Kennedy and Morton was the only lineament noted on or projecting toward the site. The mapped fault trends N32W and is mapped as solid line within the Pauba on APN 380-250-003 and dashed within the Pauba on APN 380-250-023. The fault is not mapped within the Unnamed Sandstone on APN 380-250-003.

Field Investigation

The Geocon fault investigation was performed October 25 through November 16, 2012 and consisted of excavation of two fault trenches totaling 184 feet (FT-1 and FT-2) within APN 380-250-003 to depths of 4 to 6.5 feet. The fault trench within APN 380-250-023 (FT-3) was 225 lineal feet and 4 to 15 feet deep. Trenches deeper than 5 feet were benched at an effective slope ratio of 1:1 (horizontal:vertical) to provide safe working conditions. The trench walls were scraped clean of smeared soils and a level line was strung to accurately depict the trench geometry. Soil conditions encountered in the trench excavations were visually observed, classified and logged in general accordance with ASTM International (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D2488). The fault trenches were geologically logged at a scale of 1 inch equals 5 feet by a Certified Engineering Geologist from our firm. The soil color was classified in accordance with the 2000 Munsel Soil Color Chart. Logs of the trenches are presented on Figure C-1. Locations of the trenches are shown on the Geologic Map, Figure 2. We were looking for evidence of fault rupture which extended through the bedrock units and the overlying younger soils. Features such as through going fractures/ground cracks, faults, soft or disturbed zones, or abrupt changes in geologic units were examined and traced out to determine if they extended into overlying soils or extended into the bottom of the trench and were also present on the opposite trench wall. Where features were not present on the opposite trench wall, were underlain by continuous bedding below the feature, or which were overlaid my unbroken colluvial soils the features were classified as fractures or older faulting (older than the 11,000+ year old colluvium encountered). During logging we invited City of Wildomar Building Official, Van Wilfinger to review the trenches or send a representative to view the trenches. The invitation was declined and it was stated that the City of Wildomar would rely on our report for the project. Trenches were loosely backfilled with little compactive effort and should be re-excavated during grading and replaced with compacted fill.

Summary of Findings

Fault Trench 1 (FT-1) – FT-1 was excavated within APN 380-250-003 roughly perpendicular to the trace of the mapped fault. The trench was 150 feet long and was 4 to 6.5 feet deep. It trended N37E and the southwest wall was geologically logged and depicted. The trench excavation exposed Unnamed Sandstone overlain by colluvium and topped with alluvium/topsoil. The colluvium was red (10R 4/8) dense, had columnar blocky structure and clay development on the ped facies indicating significant age (much older than 11,000 years before present which governs fault activity classification). The bedrock was moderately weathered in the upper few feet and stained red-brown from infiltration from the overlying colluvium. This unit also closely resembles the massive Pauba Sandstone observed in FT-2 and could be a gradation transition from the Unnamed Sandstone to the overlying Pauba Sandstone. The Unnamed Sandstone was locally massive to locally bedded with occasional siltstone and sand beds. Several fractures were observed within the Unnamed Sandstone at the northeastern end of the trench between Station 0+10 and 0+30. These fractures extended downward from the top of the unit or from a silt bed within the unit and did not extend to the bottom of the trench. A fault was encountered from Stations 46+00 to 52+00 trending an average of N63W dipping 33 degrees to the southwest. The fault offset beds within the Unnamed Sandstone. There appeared to be detritus of Unnamed Sandstone incorporated into the base of the colluvial unit. However, this fault could not be traced into the colluvial unit and did not offset the contact of the Unnamed Sandstone and colluvium. Older faults were observed within the trench to the southwest where the colluvium was present in association with the faults. The faults south of Station 52+00 appear to be healed (cemented) and colluvium was observed overlying the faults on one or both of the trench walls. The faults that were observed within the Unnamed Sandstone do not appear to be active due to the presence of undisturbed older colluvial soils overlying the faults.

Fault Trench 2 (FT-2) – FT-2 was excavated on APN 380-250-003 across the drainage from FT-1 to intercept the projection of the faults observed in FT-1 between Stations 46+00 to 52+00 which trended an average of N63W and were in line with the active Glen Ivy fault to the northwest. The trench was 35 feet long and generally 5 feet deep. The trench trended N20E and the northwest trench wall was logged and is depicted herein. The excavation exposed locally massive Pauba Sandstone overlain by approximately 18 inches of topsoil. The Pauba was intact with no evidence of ground cracking/fractures or faults. Therefore, the faulting encountered within FT-1 at Stations 46+00 to 52+00 was ruled out as an extension of the Glen Ivy fault.

Fault Trench 3 (FT-3) – FT-3 was excavated on APN 380-250-023 across the mapped lineament. The trench was 225 feet long and 4 to 15 feet deep. It trended N72E and the southeast wall was logged and is depicted herein. The trench excavation exposed Pauba Sandstone overlaid by colluvium, alluvium and topsoil. The colluvium was red brown (5YR 4/4), had very blocky prismatic structure with clay development on ped facies and weathering rinds on cobbles and gravel clasts indicating substantial age to the unit in excess of 11,000 years before present. The northeastern portion of the trench exposed locally massive to subtly bedded coarse Pauba Sandstone. Bedding definition increased to the southwest where thin sand and silt beds within the Pauba were traced above or below suspected fault features. Several large krotovina (animal burrows) were observed within the trench excavation. These features can be indicative of faulting, therefore, excavations were deepened in areas of krotovina revealing bedded, unbroken Pauba Sandstone beds beneath the krotovina. Some ground cracks (Station 0+05 to 0+35) were also observed within the excavation. The cracks were generally linear with slightly irregular surfaces and were simply slightly weaker zones within the rock with no carbonate or clay deposits. These ground cracks either projected down from the ground surface and did not extend to the bottom of the trench or projected up from the bottom and did not project into the overlying colluvium. We deepened the trench in this area which revealed unbroken Pauba Sandstone beds beneath the ground cracks/fractures. The Pauba Sandstone deepened in the southwestern portion of the trench where alluvium thickened along an erosional/depositional contact. This area of the trench was deepened to 15 feet to provide a continuous exposure of Pauba Sandstone throughout the trench excavation where bedding could be traced to verify no faulting was present. There were no features within this trench which were classified as faults.

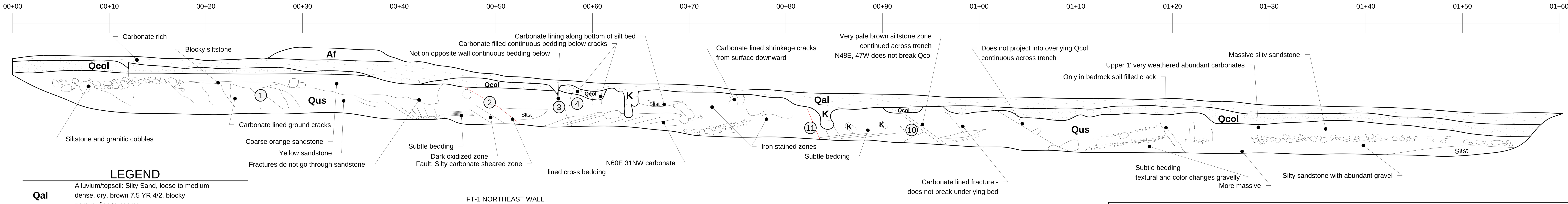
Conclusions

Based on our exploration as described above we did not observe active faulting within either site. Older faulting was observed within the Unnamed Sandstone (very early Pleistocene approximately 1.6 million years old) on the northern site. However, no faulting was observed within the Pauba Sandstone (early Pleistocene approximately 1 million years old) on either site. We have included the fault location as dotted (buried) on the Geologic Map. We are not recommending building setback zones on either site based on this evidence.

FT-1

N37°E

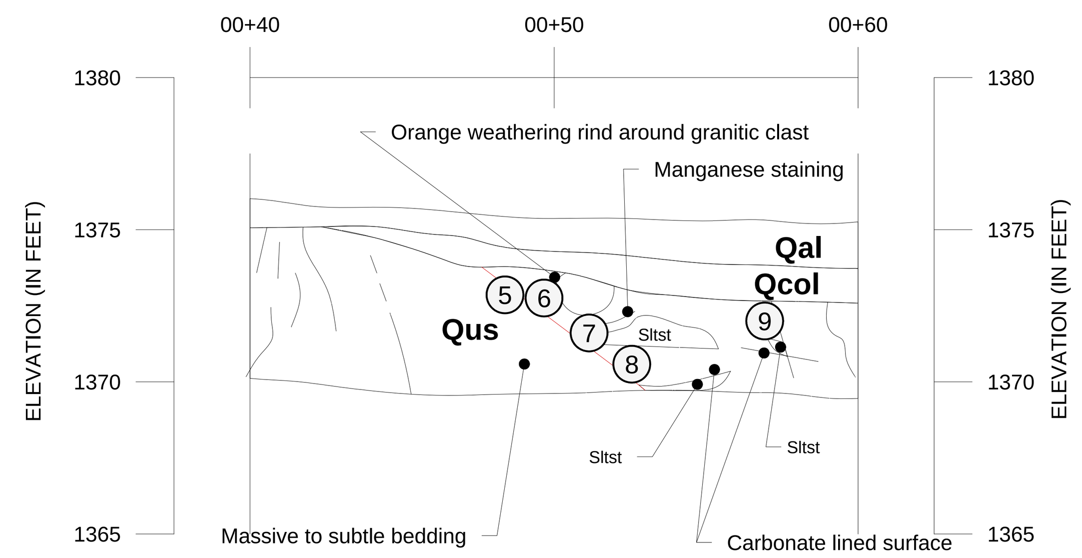
SOUTHWEST WALL



LEGEND

Qal	Alluvium/topsoil: Silty Sand, loose to medium dense, dry, brown 7.5 YR 4/2, blocky porous, fine to coarse
Qcol	Colluvium: Silty to Clayey Sand, medium dense to dense, dry, red 10 R 4/8 to 5 YR 4/3, coarse with trace cobbles, rock common along base, blocky columnar structure, clay development on ped facies
Qus	Unnamed Sandstone: Silty Sandstone, hard, dry to moist, light yellow brown 2.5 Y 6/4 to light olive brown 2.5 Y 5/3, coarse with gravel, occasional large siltstone clasts, locally conglomeratic with lenses of siltstone and granitic cobbles, cemented, upper portion is orange (possibly grading into Qps), locally poorly bedded, moderately to highly weathered, fractured
Qps	Pauba Sandstone: Silty Sandstone, hard, dry to moist, coarse, yellow red 2.5 YR 4/6, massive, moderately weathered, unfractured
Af	Artificial fill
K	Krotovina (Animal Burrow)
Sltst	Siltstone Bed or Clast

FT-1 NORTHEAST WALL

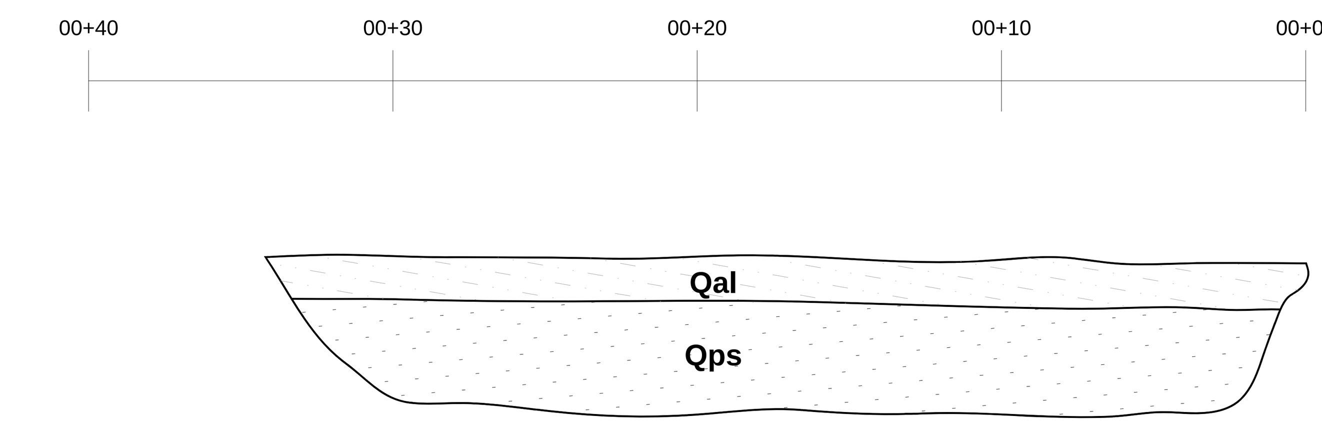


1. fracture: N30W, SW - continues on opposite wall discoloration along crack
2. Fault: N75W, 28S, continuous on surface N65W across trench
3. fracture: N20E, 50S
4. fracture: N20W, 80S carbonate filled
5. through 8 Fault Zone: Average N63W, 33SW
9. fracture: N68E, 80S, fracture/ground crack
10. fracture: N20E, 26W ground crack
11. Fault: EW, S continues across trench does not disrupt underlying bedding

FT-2

N20°E

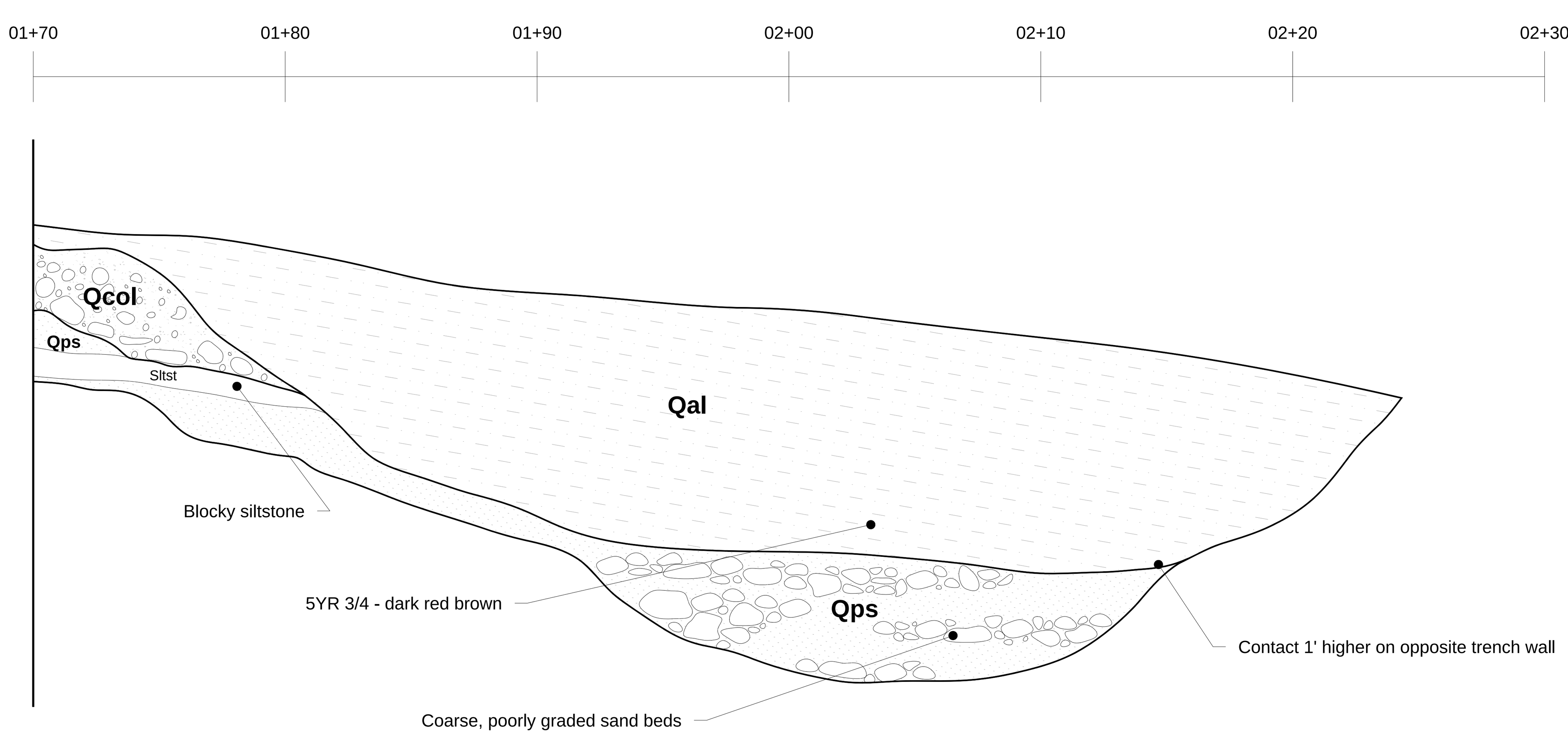
NORTHWEST WALL



FT-3

N27°E

SOUTH WALL



LEGEND

Qal	Alluvium/topsoil: Silty Sand, loose to medium dense, dry, porous, fine to coarse, slighly blockey, dark brown 7.5 YR 3/4 to dark red brown 5 YR 3/4
Qcol	Colluvium: Silty to Clayey Sand (SM/SC), medium dense to dense, dry, red brown 5 YR 4/4, fine to coarse, very blocky, prismatic structure, clay lining on ped facies, weathering rinds on cobbles and gravel - cobbles are semi rounded and highly weathered
Qps	Pauba Sandstone: Interbedded poorly graded sand, silty sand, silt with gravel and cobble channels, more massive at NE of trench, becoming thinly bedded to laminated to the SW, soft to moderately hard, moist, light red 2.5 YR 6/8 to 5 YR 4/6, silt beds are very pale brown 10 YR 7/4, resembles terrace deposit near SW end of trench at depth across trench, poorly bedded to locally laminated - lateral facies changes common, moderately weathered, moderately fractured
K	Krotovina (Animal Burrow)
Sltst	Siltstone Bed or Clast

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TRENCH LOGS

GROVE PARK
APN 380-250-003
SW CLINTON KEITH RD & YAMAS DR
WILDOMAR, CALIFORNIA

FEBRUARY, 2015 | PROJECT NO. T2539-22-02A | PLATE 1