PRELIMINARY GEOTECHNICAL AND FAULT RUPTURE HAZARD INVESTIGATION

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

STRATAEQUITY GROUP, INC.
SAN DIEGO, CALIFORNIA

DECEMBER 12, 2012
REVISED MARCH 26, 2015
PROJECT NO. T2540-22-02

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2540-22-02
December 12, 2012
Revised March 26, 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
TRACT 34301, NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

Dear Mr. Flodine:

In accordance with your authorization of our proposal dated October 12, 2012 we have performed a preliminary geotechnical and fault rupture hazard investigation of approximately 35 acres of vacant land located immediately northwest of the intersection of Baxter Road and Interstate 15 (I-15) in the City of Wildomar, California. The accompanying report presents the findings of our study, and our preliminary conclusions and recommendations pertaining to the geotechnical aspects of construction. This report has been revised to included design parameters in accordance with 2013 California Building Code (CBC).

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.

PROFESSIONAL GEOLOGIST
LISA A. BATTIATO
No. 2316
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Lisa A. Battiato
CEG 2316

REGISTERED PROFESSIONAL ENGINEER
CHET E. ROBINSON
No. 2890
GEOTECHNICAL
STATE OF CALIFORNIA

Chet E Robinson
GE 2890

LAB:KEC:SK:JT:CER:

(1 pdf) Addressee

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1

2. SITE AND PROJECT DESCRIPTION 2
   2.1 Site and Project Description 2
   2.2 Site History 2

3. PREVIOUS GEOTECHNICAL INVESTIGATION 3

4. GEOLOGIC SETTING 4

5. SOIL AND GEOLOGIC CONDITIONS 4
   5.1 Younger Alluvium (Qal) 4
   5.2 Colluvium (Qcol) 4
   5.3 Pauba Sandstone (Qps) 5
   5.4 Granitic Bedrock (Kgdd) 5

6. GROUNDWATER 5

7. GEOLOGIC HAZARDS 5
   7.1 Surface Fault Rupture 5
   7.2 Seismicity 6
   7.3 Estimation of Peak Ground Accelerations 7
   7.4 Deterministic Analysis 9
   7.5 Probabilistic Analysis 9
   7.6 Seismic Design Criteria 9
   7.7 Liquefaction Potential 11
   7.8 Seismically-Induced Settlement 11
   7.9 Landslides 11
   7.10 Earthquake-Induced Flooding 11
   7.11 Tsunamis and Seiches 12
   7.12 Subsidence 12

8. CONCLUSIONS AND RECOMMENDATIONS 13
   8.1 General 13
   8.2 Soil and Excavation Characteristics 15
   8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate 15
   8.4 Grading 16
   8.5 Shrinkage 19
   8.6 Foundation Design 19
   8.7 Miscellaneous Foundations 21
   8.8 Lateral Design 21
   8.9 Concrete Slabs-on-Grade 22
   8.10 Preliminary Pavement Recommendations 23
   8.11 Swimming Pool/Spa 24
   8.12 Retaining Wall Design 25
   8.13 Dynamic (Seismic) Lateral Forces 26
   8.14 Retaining Wall Drainage 26
   8.15 Elevator Pit Design 27
   8.16 Elevator Piston 27
   8.17 Temporary Excavations 28
   8.18 Surface Drainage 28

8.19 Plan Review 29

APPENDIX D 37

FAULT RUPTURE HAZARD INVESTIGATION 37
GEOLOGIC REVIEW 37
FAULT ACTIVITY CRITERIA 37
LINEAMENT ANALYSIS 38
FIELD INVESTIGATION 38
SUMMARY OF FINDINGS 39
CONCLUSIONS 40

LIMITATIONS AND UNIFORMITY OF CONDITIONS

LIST OF REFERENCES

MAPS, TABLES, AND ILLUSTRATIONS
Figure 1, Vicinity Map
Figure 2, Geologic Map
Figure 3, Regional Fault Map
Figure 4, Regional Seismicity Map
Figure 5, Probability of Exceedance
Figure 6, Retaining Wall Drainage Details
Figure 7, Retaining Wall Footing Details
Table 1, Faults within 60 Miles of the Site – Deterministic Site Parameters

APPENDIX A
FIELD INVESTIGATION
Figures A-1 through A-8, Boring Logs

APPENDIX B
LABORATORY TESTING
Figure B1, Direct Shear Test Results
Figure B2, Laboratory Test Results
Figure B3, Grain Size Distribution
Figure B4, Corrosivity Test Results

APPENDIX C
Boring Logs from 2005 LandMark Consultants' Investigation

APPENDIX D
FAULT RUPTURE HAZARD INVESTIGATION
Plates D1 and D2, Trench Logs

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 the approximately 35 acre site located immediately northwest of the intersection of Baxter Road and Interstate 15 (I-15) in Wildomar, California, see *Vicinity Map*, Figure 1. The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the property, and based on conditions encountered to provide 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 previous geotechnical reports for the site prepared by LandMark Consultants (LandMark), review of published and unpublished geologic reports and maps, review of aerial photographs, 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, 2012 by excavating eight 8-inch diameter borings with a CME 75 drill rig. The borings were excavated to depths between 20 and 51.25 feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on Figure 2, *Geologic Map and Site Plan*. A detailed discussion of the geotechnical field investigation, including boring 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. Boring logs presented in a 2005 LandMark report for the site are presented in Appendix C and the locations of these borings are depicted on the Figure 2.

The eastern portion of the site is located within a Riverside County Fault Hazard Zone. A fault rupture hazard investigation is required with the county-designated fault hazard zone prior to site development. To evaluate the absence or presence of faults within the county-designated fault hazard zone at the site, we performed a fault rupture hazard investigation that included the excavation of two fault trenches, totaling 690 lineal feet of trench. The trenching was performed from October 23 through October 30, 2012. The details and results of our fault rupture hazard investigation are presented in Appendix D.

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

2.1 Site and Project Description

The property is bounded on the south by Baxter Road; the east by I-15; the north by rural residential housing; and the west by White Road (mapped but not present), a horse ranch, and rural residential housing. The site is currently vacant with the exception of a former residence and an agricultural observation tower in the southern portion of the site. Both structures have been raised for relocation. Large trees are present in the southeastern and south-central areas of the site, and within a main drainage which meanders in a south-southwesterly direction across the western portion of the site. Topographically the site consists of a dissected alluvial fan which descends gently to the southwest from granitic hills to the northeast. Site elevations range from approximately 1,365 feet above mean sea level (MSL) in the northeastern area to approximately 1,335 feet MSL at the southwest corner. A main drainage is present within the western portion of the site and consists of a gently sloping valley approximately ten feet below the adjacent alluvial plain with a smaller stream incised about two to three feet into the valley. Several smaller southwest trending drainages cut the alluvial plain in the eastern area of the site. A drainage channel is also present along the southern boundary of the site and extends from the south central site area deepening to the southeast corner where it is approximately five feet below the surrounding elevations. On-site sewage disposal systems and domestic water wells may be present in association with the previous land use.

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

Site development is planned to consist of mixed-use commercial, multi-family and single-family residential construction. Grading is anticipated to result in cuts and fills on the order of 10 feet or less, exclusive of remedial grading. The structures are anticipated to be lightly loaded wood frame structures three stories or less in height. It is estimated that column loads 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.

2.2 Site History

The site history was determined based on a review of aerial photographs for the years 1962, 1974, 1980, 1983, 1990, 1995, 2000, 2005, and 2010 obtained at the Riverside County Flood Control and Water

Conservation District (RCFC), and geotechnical research at the County and local level. Based on our aerial photograph review, an olive grove occupied the western half of the site between 1962 and 1974. A former residence was observed in the 1983 and later aerial photos. It is our understanding that the existing raised house and tower were transported to and are now stored on the site. The remainder of the site appears to have been unimproved. Partial plowing of the site and dirt trails were observed on the aerial photos since 1974.

3. PREVIOUS GEOTECHNICAL INVESTIGATION

LandMark performed a geotechnical investigation of the site in 2005 and subsequently updated their geotechnical report in 2007 with no additional information or analyses. LandMark excavated eight borings within the property from depths of 16.5 to 51.5 feet, mostly within the southern portion of the site. The locations of the borings are indicated on the *Geologic Map and Site Plan* (Figure 2). Based on a review of the boring logs, groundwater was encountered at depths of 17 and 24 feet below the existing grade in borings LB-6 and LB-1, respectively. LandMark reported encountering medium dense to very dense interbedded sands, clayey sandy silts, silty clays, and silty sands. They did not identify the formation name of the geologic units on their logs.

Based on geologic mapping (Kennedy, 1977) and our knowledge of the soil conditions in the vicinity of the site, it appears the geologic units encountered in the LandMark borings are primarily Pauba Sandstone. Younger alluvium to a depth of 14 feet below the existing grade was encountered within one boring drilled in the drainage area (LB-6).

Landmark reported low sulfate, low chloride, high resistivity, and generally neutral pH. Atterberg Limit tests were performed on two samples and the test results indicated a plasticity index of 21 and 26. They stated that no active faulting was present on the site, and no landslide, seiche, or tsunami hazards were present at the site. Results of their liquefaction analysis indicate a liquefiable layer is present at 47 to 50 feet below existing ground. They identified Bundy Canyon Creek as the closest 100 year flood plain to the site. LandMark did not perform direct shear testing, consolidation or collapse testing, and did not test the soil samples for in-place moisture and density. The liquefaction analysis was based on assumptions and not grain-size analyses or moisture/density data from the boring samples. Although they reportedly encountered clayey soils, no Expansion Index testing was performed to provide site specific foundation recommendations. The pavement recommendations provided by LandMark are based on an assumed R-value and not actual test data for the site. Accordingly, it appears as though the geotechnical recommendations provided by LandMark are very conservative, including the estimated settlement values and foundation design parameters. The boring logs from the 2005 LandMark report are presented in Appendix C.

4. GEOLOGIC SETTING

The site is located within the Peninsular Ranges Geomorphic Province, characterized by northwest trending alluviated valleys and geologic structures such as the nearby Elsinore Fault Zone and Santa Ana Mountains. The Peninsular Ranges are bounded on the north by the Santa Monica, Hollywood, Raymond, Cucamonga, and Sierra Madre Fault Zones, the east by the San Jacinto Fault, and the west by the Pacific Ocean. The Peninsular Ranges extend southward into Mexico.

Locally, the site is located on the eastern edge of the Elsinore Trough, a graben which formed as a result of a left step over from the Wildomar to the Willard faults on the eastern and western sides of Lake Elsinore, respectively. Ground fissures have been documented south of the site in the Elsinore Trough since the 1980s. The ground fissures have generally developed along pre-existing fault traces as a result of groundwater withdrawal (Kuperman, 1998).

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of younger alluvium, colluvium, Pauba Sandstone, and granitic bedrock. Geologic mapping by Kennedy (1977) identifies the geologic units at the site as primarily Pauba Sandstone with granitic bedrock occurring along the eastern site boundary. The granitic bedrock underlies the site at depth. In general the upper foot of existing site soils has been disturbed by periodic plowing. Detailed stratigraphic profiles are presented in the boring logs in Appendix A.

5.1 Younger Alluvium (Qal)

Younger alluvium of Holocene age was encountered within the drainage areas consisting of loose to medium dense interlayered silty sands, sands, and clays. The younger alluvium was generally moist and medium dense and very stiff. Younger alluvium within the main drainage ranged in thickness from 2 to 18 feet with the thickness increasing toward the south. The area of deepest younger alluvium was encountered in the south-central portion of the site (B-6) in a low lying area.

5.2 Colluvium (Qcol)

Colluvium of Pleistocene age is locally present at the ground surface and was observed to overlie the granitic bedrock where encountered in boring B-4. The colluvium and consists of red-brown clayey sand with abundant carbonate nodules and stringers. Where encountered in our borings and trenches, the colluvium ranges in thickness from 1 to 8 feet and is generally dense and dry to moist, and blocky.

5.3 Pauba Sandstone (Qps)

Early Pleistocene-age Pauba Sandstone was encountered within the borings drilled as part of this study with exception of boring B-4 in the eastern portion of the site where colluvium was observed to directly overlie the granitic bedrock. Where encountered in the borings, the Pauba Sandstone consists of brown to reddish-brown, massive, silty sand that is dry to wet and generally dense. The sandstone is locally exposed at the ground surface and was encountered to a maximum depth of 39 feet in boring B-1

5.4 Granitic Bedrock (Kgdd)

Granitic bedrock (granodiorite) of Cretaceous-age underlies the site at depth and is locally present within a few feet of the ground surface along the eastern portion of the site. The granitic bedrock was encountered at depths of 3 to 39 feet within the borings and trenches excavated for this study. The bedrock is highly weathered with some slightly weathered core stones to approximately three feet in diameter. Localized areas of completely weathered rock were observed in the upper approximately one foot within Fault Trench FT-1.

6. GROUNDWATER

Groundwater was encountered at depths of 29 feet and 13.5 feet below the existing ground surface in borings B-1 (in main drainage) and B-5 (in southeastern area), respectively. Groundwater was encountered in the previous borings by LandMark drilled at the site in 2005 at depths of 17 feet (LB-6 in main drainage) and 24 feet (LB-1 in south-central area of the site near Baxter Road) in 2005.

Groundwater is locally present at the site, particularly within the drainages and the southeastern portion of the site. Additionally, groundwater seepage is common at a soil/bedrock contact. Based on these considerations, 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 storm water 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).

7. GEOLOGIC HAZARDS

7.1 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) for the Alquist-Priolo Earthquake Fault Zone Program (Byrant 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, the eastern portion of the site is located within a Riverside County Fault Hazard Zone established for a possible fault shown to traverse the eastern portion of the site on Riverside County Fault Maps (see Figure 2). The fault location is based on Kennedy's mapping (1977) and may be associated with the Glen Ivy fault known to be active northwest of the site (Rockwell, McElwain, Millman and Lamar, 1986; Millman, and Rockwell, 1986; Lamar and Rockwell, 1986). The potential for faulting at the site was not addressed by LandMark in their geotechnical investigation report (2005) or updated report (2007).

Geocon performed a fault rupture hazard investigation. A detailed discussion and the results of our fault rupture hazard investigation are presented in Appendix D.

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 San Jacinto fault, the Julian branch of the Elsinore fault, and the Chino-Central Avenue fault located approximately 5 miles northwest, 21 miles east, 22 miles south, and 23 miles north of the site, respectively (EZ-FRISK V 7.62).

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*.

7.2 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 110 years. A partial list of these earthquakes is included in Table 7.2, below.

TABLE 7.2
LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Lake Elsinore area	May 15, 1910	6.0	10	W
San Jacinto-Hemet area	April 21, 1918	6.8	18	NE
Near Redlands	July 23, 1923	6.3	27	NE
Long Beach	March 10, 1933	6.4	40	W
North San Diego County	March 25, 1937	6.0	60	S
Desert Hot Springs	December 4, 1948	6.0	55	ENE
Pinto Mountain	May 2, 1949	5.8	66	E
Arroyo Salada	March 19, 1954	6.4	66	SE
Borrego Mountain	April 9, 1968	6.5	72	SE
Borrego Springs	April 28, 1969	5.8	56	SE
Palm Springs	April 23, 1992	6.1	60	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	NE

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.

7.3 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.

7.4 Deterministic Analysis

Table 1, after the report text, 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 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 0.84g. This value is provided as geologic background information. The code specified peak ground acceleration in Section 7.6 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.

7.5 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 the 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 1.05g and 0.75g, respectively. Graphical representation of the analysis is presented on Figure 5. These values are provided as geologic background information. The code specified peak ground acceleration in Section 7.6 is used to calculate seismic and liquefaction settlement, for evaluation of seismic lateral earth pressures, and for structural design.

7.6 Seismic Design Criteria

7.6.1 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 7.6.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.6.1
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	2.298g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.927g	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}	2.298g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.205g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.532g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.803g	Section 1613.3.4 (Eqn 16-40)

7.6.2 Table 7.6.2 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 7.6.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.914g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.914g	Section 11.8.3 (Eqn 11.8-1)

7.6.3 Conformance to the criteria in Tables 7.6.1 and 7.6.2 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.

7.7 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 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 soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. 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. The younger alluvium present in the drainages at the site may be subject to liquefaction during strong ground motion. However, the Pauba Sandstone and the granitic bedrock are well-consolidated and are not considered to be susceptible to liquefaction. Provided the recommendations for remedial grading presented herein are followed, it is our opinion that the potential for liquefaction of the site soils is not a design consideration. Further, no surface manifestations of liquefaction are expected at the site.

7.8 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 dense and well consolidated nature of the soils underlying the site, appreciable seismically-induced settlements are not anticipated.

7.9 Landslides

The gently sloping topography at the site precludes slope stability hazards. There are no known landslides near the site, nor is the site in the path of any known or potential landslides.

7.10 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 not a design consideration.

7.11 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, are not a design consideration 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. The site is located approximately 5 miles south of and at a higher elevation than Lake Elsinore. The potential for flooding from a seismically induced seiche is not a design consideration.

The site is in FEMA Zone X per Flood Insurance Rate Map Panel 06065C2682G dated August 28, 2008. Therefore, potential for flood hazards at the site is not a design consideration.

7.12 Subsidence

Subsidence and associated ground fissuring has been well documented in Riverside County. 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. Areas subject to subsidence and fissuring are primarily alluviated structural valleys such as the San Jacinto Valley and Elsinore Trough that are bound by active faults that offset unconsolidated Holocene age alluvium. The location of ground fissures are typically controlled by underlying geologic structure and typically coincide with pre-existing fault traces.

In southerly portion of the Elsinore Trough, ground subsidence and associated ground fissuring related to changes in groundwater levels has occurred from Murrieta on the north to the upper Wolf Valley on the south. The documented subsidence and fissuring has been confined to the area between fault traces where significant groundwater pumping has occurred.

The site is within an area that is considered susceptible to subsidence per Riverside County. The site conditions include Pauba Sandstone and alluvium over lying granitic bedrock which was a factor in subsidence in the Murrieta area to the south in the late 1980s and 1990s. After remedial grading at the site the subsurface conditions which make the site vulnerable to subsidence will no longer be present and the possibly of subsidence will not be a design consideration.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 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.

8.1.2 We did not encounter evidence of faulting during our subsurface geotechnical investigation. Therefore, no building setback zones due to surface fault rupture are recommended for the site.

8.1.3 We encountered younger alluvial soils and colluvium overlying Pauba sandstone and granitic bedrock within the site. It is our opinion that the younger alluvium and the upper three feet of colluvium are not suitable for direct support of proposed foundations or slabs. The 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 8.4).

8.1.4 Based on these considerations, it is recommended that existing alluvium as well as the upper three feet of colluvium, and completely weathered Pauba Sandstone and granitic bedrock, within proposed building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to completely remove existing artificial fill (if encountered), alluvium, colluvium, Pauba Sandstone, or granitic bedrock at the direction of the Geocon representative.

8.1.5 Where building foundations will be supported on compacted fill, the removal depths should be deepened where necessary to create a minimum fill depth of 18 inches below the bottom of the proposed footings.

8.1.6 Where excavation and compaction is to be performed, the excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area 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 building should also be included in the required lateral over-excavation. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 8.4).

8.1.7 The fault trench excavations were loosely backfilled with no moisture conditioning or compactive effort. During site grading operations, the fault trenches should be re-excavated and properly backfilled with engineered fill. The grading contractor should be prepared for excavations on the order of 9 feet in height to excavate the artificial fill associated with the fault trench excavations.

8.1.8 Laboratory tests indicate site soils are not corrosive and have a moderate sulfate exposure rating in accordance with the 2013 California Building Code. Grading operations and soil mixing will likely result in different values at finish grade. Additional testing should be performed on the finish grade soil. Corrosion protection for metal piping and structures as well as appropriate concrete mix design may be required for the design of improvements for the site.

8.1.9 Subsequent to the recommended grading, the structures may be supported on conventional foundation systems deriving support in the newly placed engineered fill or competent Pauba sandstone.

8.1.10 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.17).

8.1.11 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 structure, 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. The foundation excavation bottom must be observed and approved by a Geocon representative.

8.1.12 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.

8.1.13 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.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soil can be excavated with moderate to heavy effort using conventional excavation equipment. Some caving or sloughing should be anticipated if loose or granular soil is encountered. In addition, due to the presence of localized core stones within the granitic bedrock, the contractor should be prepared to handle some over-sized material (greater than 6 inches) in accordance with section 8.4.

8.2.2 It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

8.2.3 Onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.17).

8.2.4 The upper few feet of soil encountered during this investigation are considered to have a "very low" (EI = 18) expansive potential and is classified as "non-expansive" based on the 2013 California Building Code (CBC) Section 1803.5.3. The recommendations in this report assume that foundations and slabs will derive support in soil with an EI less than 20.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing were performed on a representative sample of the surficial soil to generally evaluate the corrosion potential to buried utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the site would not be classified as corrosive in accordance with Caltrans Corrosion Criteria (Caltrans, 2012). The results are presented in Appendix B (Figure B4) and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of the surficial soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B4) and indicate that the on-site soil possesses "moderate" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318-11 Sections 4.2 and 4.3. The table below presents a summary of concrete requirements set forth by 2013 CBC and ACI 318. Additional testing should be performed at the time of grading to verify

the need for specific concrete mix designs. 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 8.3.2 - REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Negligible	S0	0.00-0.10	--	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+Pozzolan or Slag	0.45	4,500

8.3.3 Geocon does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate the corrosion test results and recommend the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soil.

8.4 Grading

8.4.1 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.

8.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon. The existing geologic units encountered during exploration are suitable for re-use as an engineered fill, provided oversize material (greater than 6 inches) and deleterious debris is removed.

8.4.3 The fault trench excavations were loosely backfilled with no moisture conditioning or compactive effort. During site grading operations, the fault trenches should be re-excavated and properly backfilled with engineered fill. The grading contractor should be prepared for excavations on the order of 9 feet in height to excavate artificial fill associated with the fault trench excavations.

8.4.4 Grading should commence with the removal of 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). 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.

8.4.5 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.

8.4.6 As a minimum in building pad areas or areas to receive structural fill it is recommended that the existing alluvium, the upper 3 feet of colluvium, and completely weathered Pauba Sandstone and granitic bedrock, where exposed at the surface, be excavated and properly compacted for foundation and slab support. Anticipated depths of removals at the boring locations are indicated on the *Geologic Map and Site Plan*, Figure 2. Deeper excavations should be conducted as necessary to completely remove existing unsuitable soils at the direction of the Geocon representative.

8.4.7 Where excavation and compaction is to be conducted, the excavations should extend laterally a minimum distance of three feet beyond the building footprint area 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 building should also be included in the required lateral over-excavation.

8.4.8 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. The bottom of the undercut should be sloped at a minimum of 1 percent towards the adjacent street. 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/3, where H is the maximum fill thickness.

8.4.9 Over-excavation of cut lots exposing granitic bedrock should be performed to reduce the difficulty of excavating footing trenches within the bedrock. Cut lots which expose granitic

bedrock should be over-excavated three feet, or 18inches below the bottom of the proposed footings, whichever is deeper. The bedrock should be sloped 1 percent or more toward the street.

8.4.10 Excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. If unsuitable soils are exposed at the excavation bottom, additional excavation may be required at the direction of the Geotechnical Engineer (a representative of Geocon).

8.4.11 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) D 1557 (latest edition).

8.4.12 Where new paving is to be placed, it is recommended that 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 8.10).

8.4.13 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 foundations for small outlying structures cannot be supported in engineered fill, Geocon should be contacted to provide alternate recommendations once project plans are available for review. 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.

8.4.14 Utility trenches should be properly backfilled in accordance with the requirements of the Greenbook (latest edition). 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).

8.4.15 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.

8.4.16 Imported fill shall be observed, tested, and approved by Geocon 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 than those of the existing onsite soil (see Figure B4). Direct shear properties of import soils should be at or higher 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). If the engineering properties of the import soil are significantly different from those of the onsite soil presented herein, updated foundation, wall, and slope recommendations may be required.8.4.17 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.

8.5 Shrinkage

8.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 0 and 10 percent should be anticipated when excavating and compacting the existing alluvium; 0 to 5 percent should be anticipated for colluvium; Pauba Sandstone will likely result in 0 shrinkage and bulking; and granitic bedrock will likely bulk 5 percent when compacted to an average relative compaction of 92 percent.

8.6 Foundation Design

8.6.1 Subsequent to the recommended grading, the proposed structures may be supported on a conventional foundation system deriving support in either newly placed engineered fill or the competent Pauba Sandstone.

8.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,400 pounds per square foot, 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.

8.6.4 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 3,500 pounds per square foot should be utilized.

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

8.6.6 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.

8.6.7 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.

8.6.8 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.

8.6.9 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.

8.6.10 The maximum expected static settlement for structures supported on a conventional foundation system 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 distance of twenty feet. If construction details differ significantly from those presented herein, modifications to the foundation recommendations including anticipated settlement will be required.

8.6.11 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), 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.

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

8.7 Miscellaneous Foundations

8.7.1 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. If foundations for small outlying structures cannot be supported in engineered fill, Geocon should be contacted to provide alternate recommendations once project plans are available for review.

8.7.2 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.

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

8.8 Lateral Design

8.8.1 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 0.38 may be used with the dead load forces for concrete footings bearing in properly compacted engineered fill, and 0.4 may be used in formational units (Pauba or granitic bedrock).

8.8.2 Passive earth pressure for the sides of foundations and slabs poured against engineered fill or formational units may be computed as an equivalent fluid having a density of 350 pcf with a maximum earth pressure of 3,500 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.9 Concrete Slabs-on-Grade

8.9.1 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 8.10).

8.9.2 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.

8.9.3 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 E 1643 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.

8.9.4 For seismic design purposes, a coefficient of friction of 0.38 may be utilized between concrete slabs and subgrade soil without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.9.5 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 90 percent relative compaction, as determined by ASTM Test Method D 1557 (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.

8.9.6 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.

8.10 Preliminary Pavement Recommendations

8.10.1 Where new paving is to be placed, it is recommended that existing undocumented fill and soft or disturbed alluvium be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of 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 D 1557 (latest edition).

8.10.2 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. The actual Traffic Index for each area should be determined by the project civil engineer or the building official. 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.

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

8.10.3 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (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 *"Standard Specifications for Public Works Construction"* (Greenbook).

8.10.4 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 D 1557 (latest edition).

8.10.5 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.

8.11 Swimming Pool/Spa

8.11.1 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 undisturbed alluvium found at or below a depth of 2 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 alluvium found at or below a depth of two feet. It is the intent of the Geotechnical Engineer to allow swimming pool foundation systems to bear in the competent undisturbed alluvium or newly placed engineered fill or both as necessary.

8.11.2 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 8.6 and 8.12). 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.

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

8.11.4 If a proposed pool is in proximity to a 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.

8.12 Retaining Wall Design

8.12.1 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.

8.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 8.6).

8.12.3 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 31 pcf.

8.12.4 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 51 pcf.

8.12.5 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.

8.12.6 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 80 pcf. This value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.12.7 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.

8.13 Dynamic (Seismic) Lateral Forces

8.13.1 In accordance with the 2013 California Building Code, if the project possesses a seismic design category of D, E, or F, retaining walls should be designed with seismic lateral earth pressure. The structural engineer should determine the seismic design category for the project. The maximum dynamic (seismic) lateral pressure is equal to the sum of the initial static active pressure and the dynamic (seismic) pressure increment.

8.13.2 The seismic lateral earth pressure on unbraced retaining walls is applied to check the overall sliding resistance of the structure. Braced retaining walls should be designed for the greater of either the at-rest earth pressure or the seismic lateral earth pressure.

8.13.3 The application of seismic loading should be performed at the discretion of the project Structural Engineer and in accordance with the requirements of the Building Official. If seismic loading is to be applied, we recommend a seismic load of 26 pounds per cubic foot be used for design applied as a triangular distribution of pressure along the wall height. This dynamic (seismic) pressure increment is for horizontal backfill behind the wall and does not account for an inclined backfill surface. The seismic pressure is based on a site modified peak ground acceleration of 0.914g and by applying a pseudo-static coefficient of 0.33.

8.14 Retaining Wall Drainage

8.14.1 Retaining walls should be provided with a drainage system extended 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.

8.14.2 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 7). 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.

8.14.3 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 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.

8.15 Elevator Pit Design

8.15.1 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 8.12).

8.15.2 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.

8.15.3 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 8.14).

8.15.4 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.

8.16 Elevator Piston

8.16.1 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.

8.16.2 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) is required.

8.16.3 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.

8.17 Temporary Excavations

8.17.1 The excavations are expected to expose alluvium, dense native soil, and bedrock 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.

8.17.2 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.

8.17.3 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 may be sloped back at a uniform 1:1 slope gradient or flatter to a maximum height of 10 feet. A uniform slope does not have a vertical portion.

8.17.4 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 in accordance with OSHA requirements so that modifications of the slopes can be made if variations in the soil conditions occur. Excavations should be stabilized within 30 days of initial excavation.

8.18 Surface Drainage

8.18.1 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.

8.18.2 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 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.

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

8.18.4 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 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.

8.19 Plan Review

8.19.1 Grading and foundation should be reviewed by the Geotechnical Engineer (a representative of Geocon), 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.

LIST OF REFERENCES

Applied Technology Council, 1978, *Tentative Provisions for Development of Seismic Regulations for Buildings*, ATC Publication ATC 3-06, NBS Special Publication 510, NSF Publication 78-8.

Association of Engineering Geologists, 2009, *AEG Inland Empire Spring 2009 Field Trip, Hydrology of the Thermal Springs in the Palm Springs Area-Indian Canyons & Agua Caliente Hot Springs*.

Blake, T.F., 2000, EQFAULT, *A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults*, Version 2.20.

Blake, T.F., 2000, EQSEARCH, *A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs,* Version 2.20.

Blake, T.F., 2000, FRISKSP, *A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using 3-D Faults as Earthquake Sources.*

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, *Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes,* A *Summary of Recent Work*, Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.

Bryant, W. A. and Hart, E.W., 2007, "*Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps*, California Geological Survey Special Publication 42, Interim Revision 2007

California Department of Conservation, Division of Mines and Geology: *Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region, DMG,* CD 2000-004.

California Department of Transportation (Caltrans), Division of Engineering Services, Materials Engineering and Testing Services, *Corrosion Guidelines, Version 2.0*, dated November, 2012.

California Department of Water Resources, Water Data Library, www.water.ca.gov/waterlibrary.

California Geological Survey, *California Geomorphic Provinces*, CGS Note 36 Revised 12/2002.

California Geological Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California,* Special Publication 117.

Chang, S.W., et. A.L., 1994, *Ground Motions and Local Site Effects,* University of California at Berkeley Earthquake Engineering Research Center, Report No. UCB/EERC-94/08, p.28.

Ishihara, K., *Stability of Natural Deposits During Earthquakes,* Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, A. A. Balkema Publishers, Rotterdam, Netherlands, 1985, vol. 1, pp. 321-376.

Jennings, C.W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

Jennings, C. W., 1994, *Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions*, California Division of Mines and Geology Map No. 6.

Kennedy, Michael P., 1977, *Recency and Character of Faulting Along the Elsinore Fault Zone in Southern Riverside County, California*, CDMG Special Report 131.

Kuperman, S.A., 1998, *Regulating Land Use and Development in Areas Subject to Subsidence and Earth Fissures: An Update, Riverside County*, in Land Subsidence, Case Studies and Current Research, Proceedings of the Dr. Joseph F. Poland Symposium on Land Subsidence, edited by Borchers, J.W, AEG special Publication No. 8.

Lamar, D.L. and Rockwell, T.K., 1986, *An Overview of the Tectonics of the Elsinore Fault Zone* in Neotectonics and Faulting in Southern California, Geological Society of America 82nd Annual Meeting, March 25-28, 1986.

LandMark, 2005, *Geotechnical Investigation Proposed Baxter Crossing, 35-Acre Property, Wildomar, California*, LCI Report No. LP05282, dated October 18.

LandMark, 2007, *Geotechnical Investigation Report for Baxter Crossing, prepared by LandMark Consultants, dated October 18, 2005 (LP05282)*, dated December 3.

Legg, M. R., J. C. Borrero, and C. E. Synolakis, 2002, *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

Martin, G.R., and Lew, M., 1999, Co-chairs and Editors of the Implementation Committee, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Organized through the Southern California Earthquake Center, University of Southern California.

McGuire, R. K., 1978, *FRISK-A computer based program for seismic risk analysis.* U. S. Geological Survey Open-File Report 78-1007

Millman, D.E., and Rockwell, T.K. 1986, Neotectonics of the Elsinore Fault in Temescal Valley, California, in Neotectonics and Faulting in Southern California, Geological Society of America 82nd Annual Meeting, March 25-28, 1986.

Riverside County , 2003, *General Plan, Final Integrated Version (including all elements);* Adopted October 7, 2003.

Riverside County Land Information System, www.3.tlma.co.riverside.ca.us.

Riverside County Flood Control and Water Conservation District Aerial Photographs:

Date	Photograph Number	Scale
1/28/62	1-69/1-70	1” = 2000’
6/20/74	727/728	1” = 2000’
5/4/80	756/757	1” = 2000’
12/15/83	584/585	1” = 1600’
1/25/90	15-15/15-16	1” = 1600’
1/29/95	15-12/15-13	1” = 1600’

3/18/00	15-15/15-16	1” = 1600’
4/14/05	15-14/15-15	1” = 1600’
4/2/10	15-14/15-15	1” = 1600’

Rockwell, T.K., McElwain, R.S., Millman, D.E., and Lamar, D.L., 1986, *Recurrent Late Holocene Faulting on the Glen Ivy North Strand of the Elsinore Fault at Glen Ivy Marsh* in Neotectonics and Faulting in Southern California, Geological Society of America 82nd Annual Meeting, March 25-28, 1986.

Seed, H.B., Idriss, I.M., and Arango, I., 1983, *Evaluation of Liquefaction Potential Using Field Performance Data*, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

Shlemon, R.J. and Hakakian, M., undated, *Fissures Produced Both by Groundwater Rise and Groundwater Fall: A Geologic Paradox in the Temecula-Murrieta Area, Southwestern Riverside County, California.*

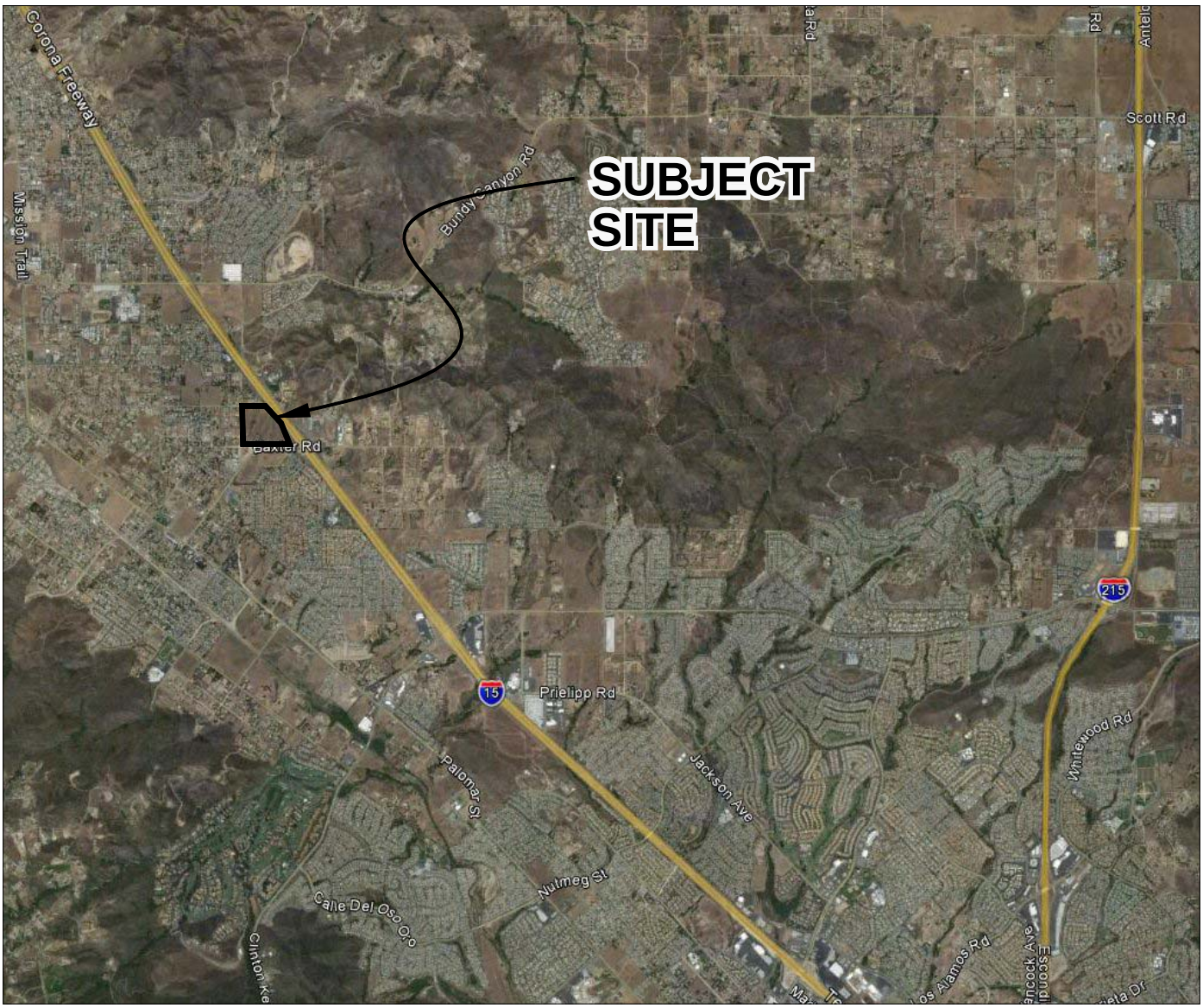
State of California Special Studies Zones, *Wildomar Quadrangle*, January 1, 1980.

Tinsley, J.C., Youd, T.L., Perkins, D.M., and Chen, A.T.F., 1985, *Evaluating Liquefaction Potential in Evaluating Earthquake Hazards in the Los Angeles Region-An Earth Science Perspective*, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.

Tokimatsu, K., and Yoshimi, Y., 1983, *Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.

Wesnousky, S.G., 1986, *Earthquakes, Quaternary Faults and Seismic Hazard in California*, Journal of Geophysical Research, Vol. 91, No. B12, pp. 12,587-12,631.

Ziony, J.I., and Jones, L.M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



REFERENCE: GOOGLE EARTH, 2012

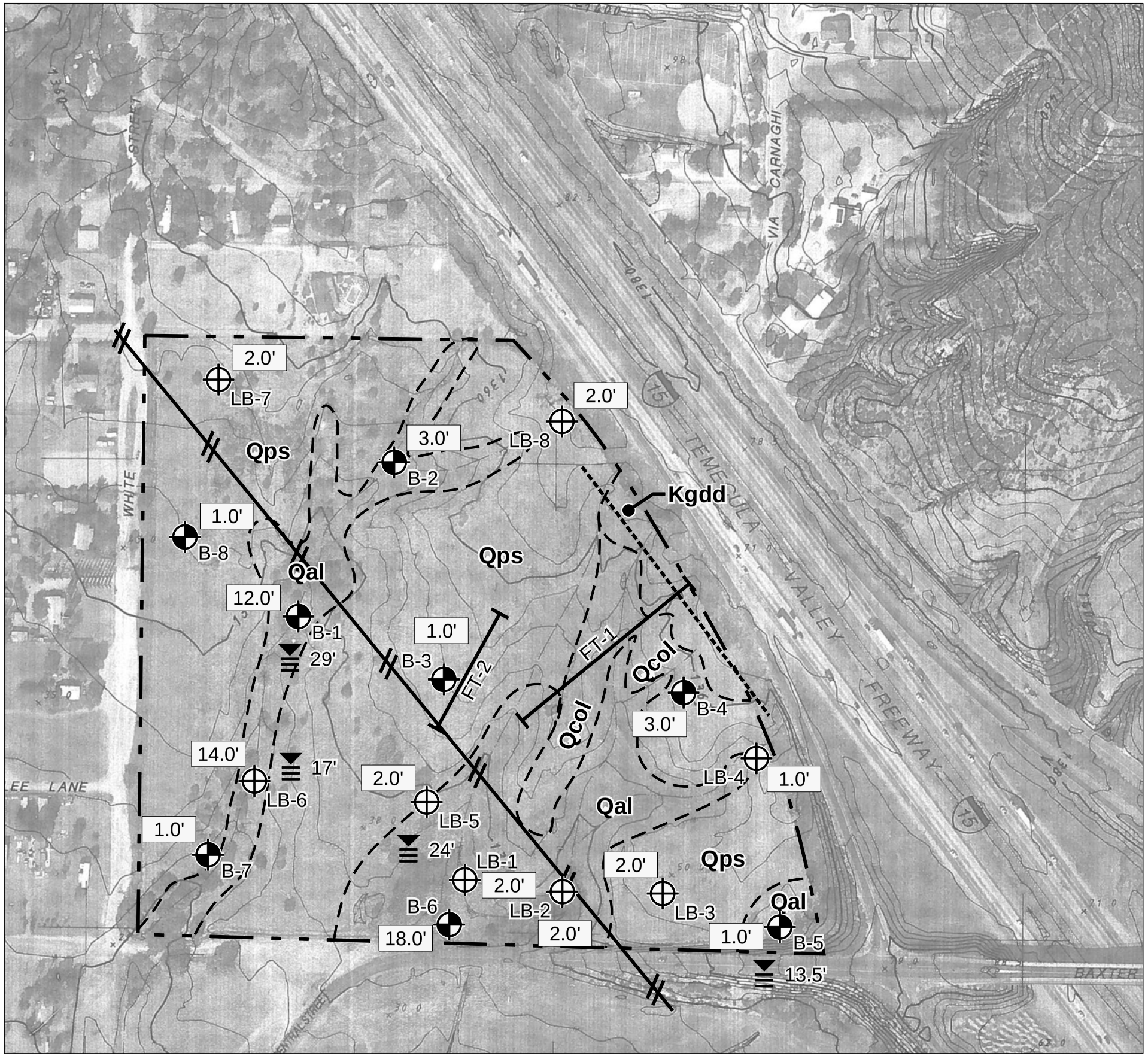
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

CHL / JL		2000

VICINITY MAP

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. 1



LEGEND

B-8	Geotechnical Boring, Geocon (this investigation)
LB-8	Geotechnical Boring, LandMark Consultants (2005)
13.5'	Groundwater Depth Encountered During Drilling
FT-2	Fault Trench Location
18.0'	Estimated Depth of Remedial Removal Recommendation
	Western Limit of Riverside County Fault Zone Limit
	Mapped Trace of Unnamed Riverside County Fault
Qal	Young Alluvium
Qcol	Colluvium
Qps	Pauba Sandstone
Kgdd	Granitic Bedrock
	Geologic Contact
	Site Boundary

Note: Geologic contacts based on Kennedy, (1977)

0 250' 500'

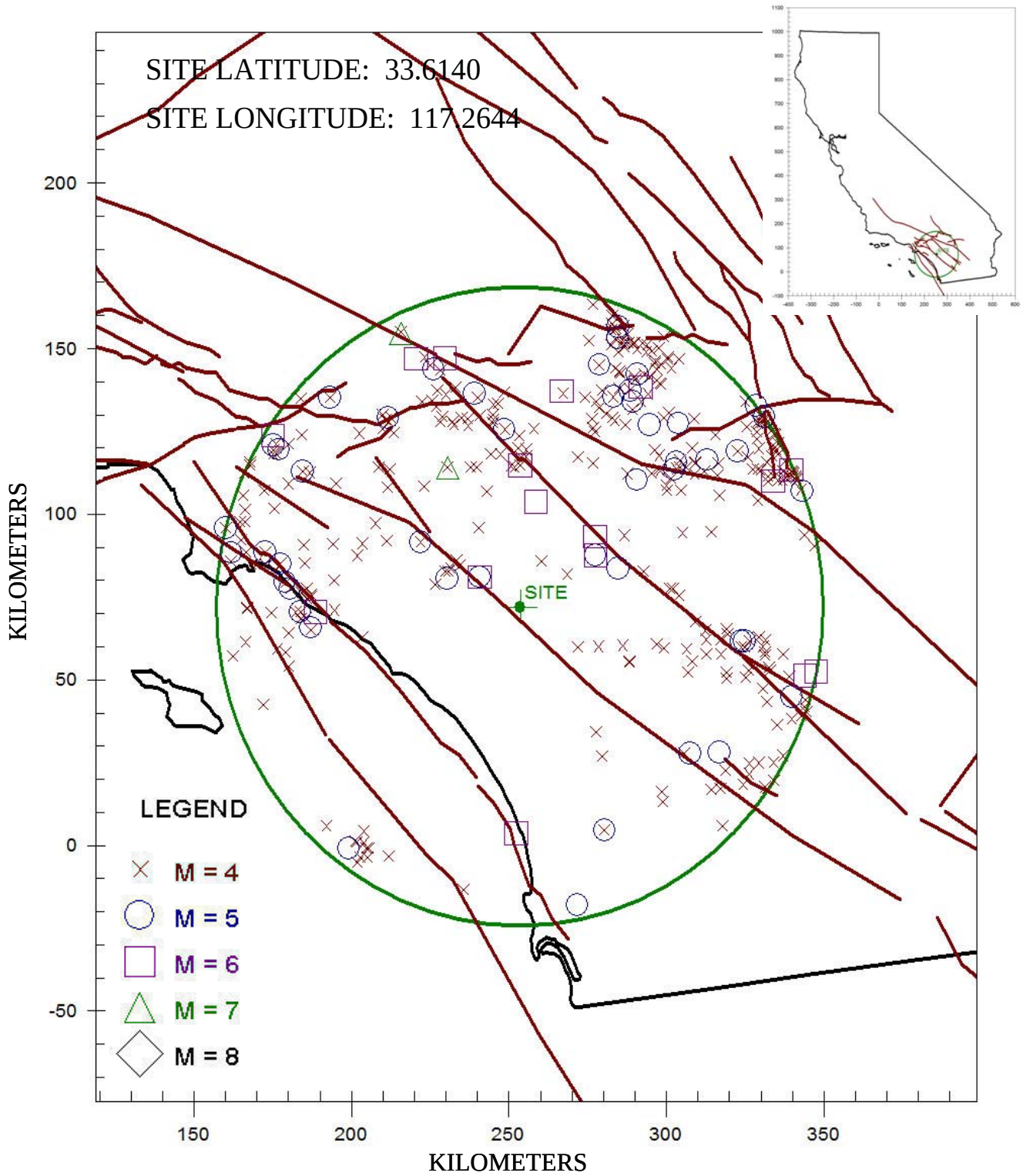
GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CALIFORNIA 92562
PHONE 951.304.2300 FAX 951.304.2392

JL		2000

GEOLOGIC MAP AND SITE PLAN

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. 2

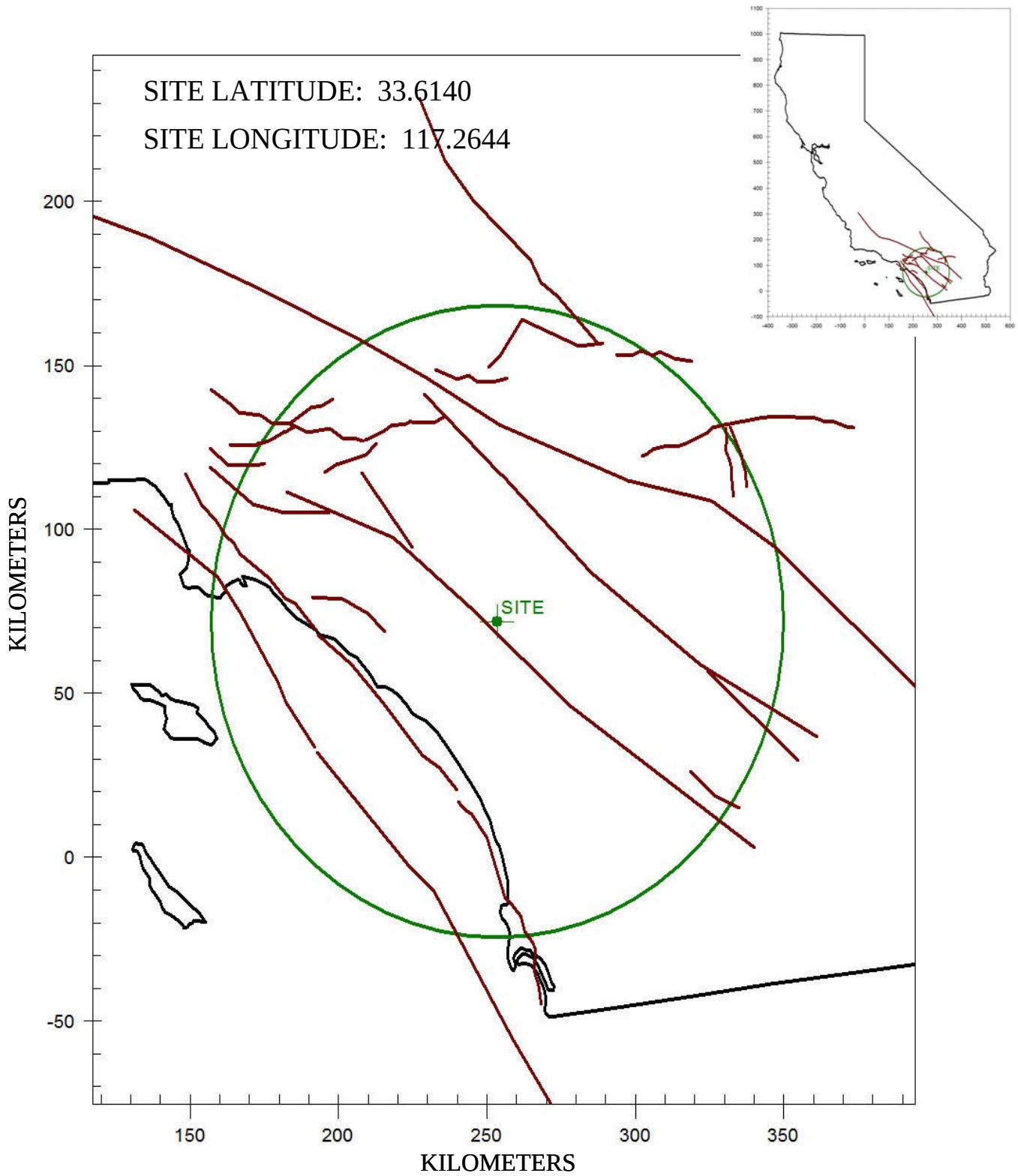


GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

REGIONAL SEISMICITY MAP

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

CHL / JL		2000	12 / 2012	PROJECT NO. T2540-22-02	FIG. 4



GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

CHL / JL		2000

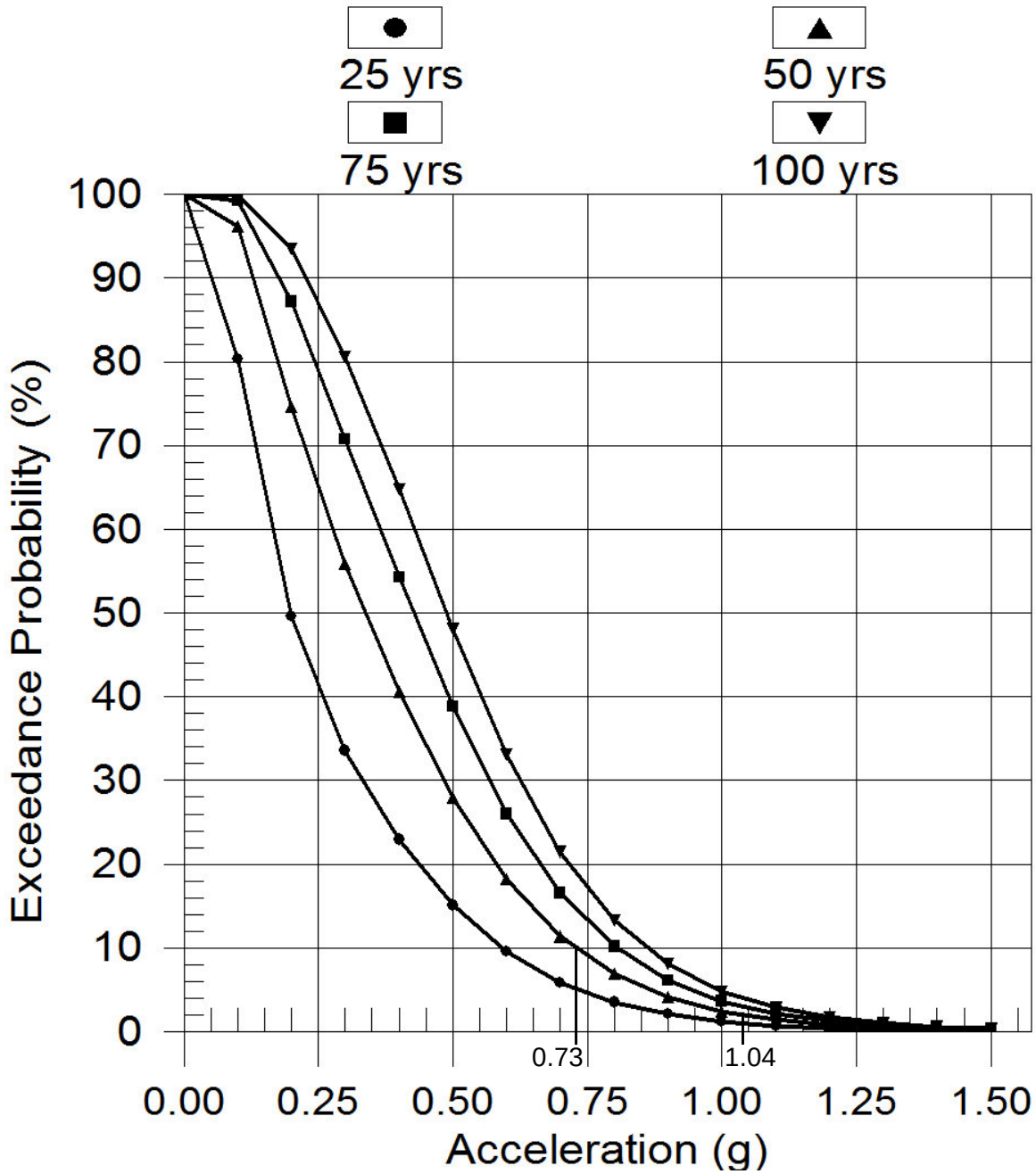
REGIONAL FAULT MAP

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. 3

PROBABILITY OF EXCEEDANCE

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



GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562

PHONE 951.304.2300 FAX 951.304.2392

CHL / JL		2000

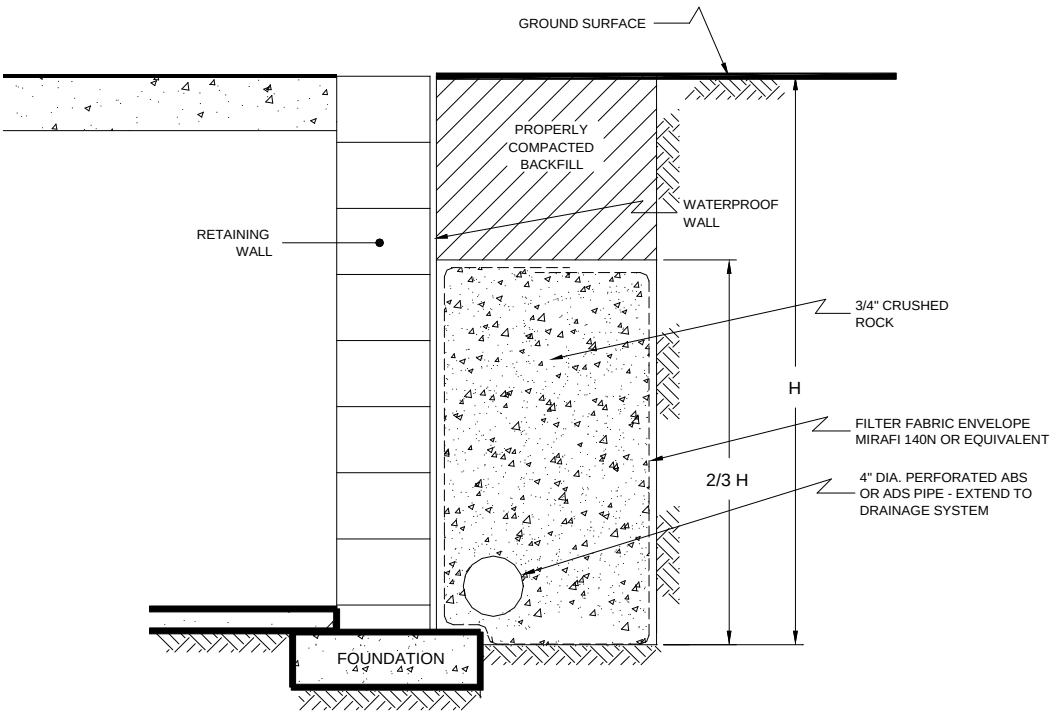
PROBABILITY OF EXCEEDANCE

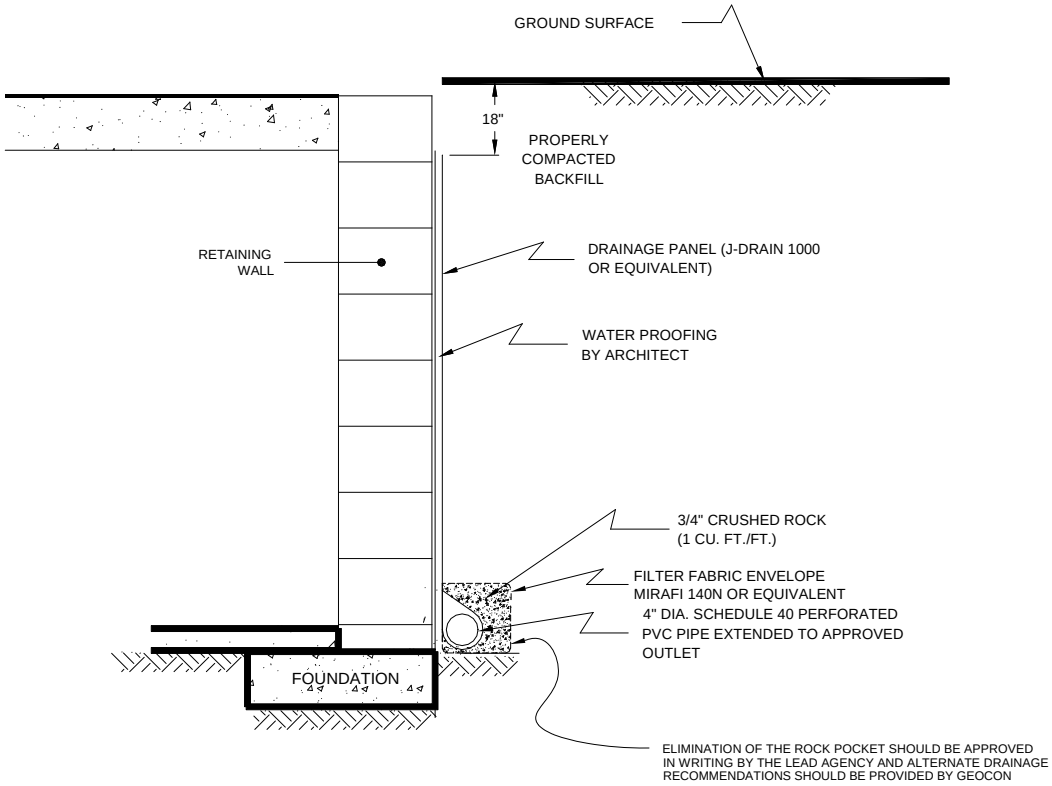
TRACT 34301

NWC BAXTER ROAD AND I-15

WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. 5





NO SCALE

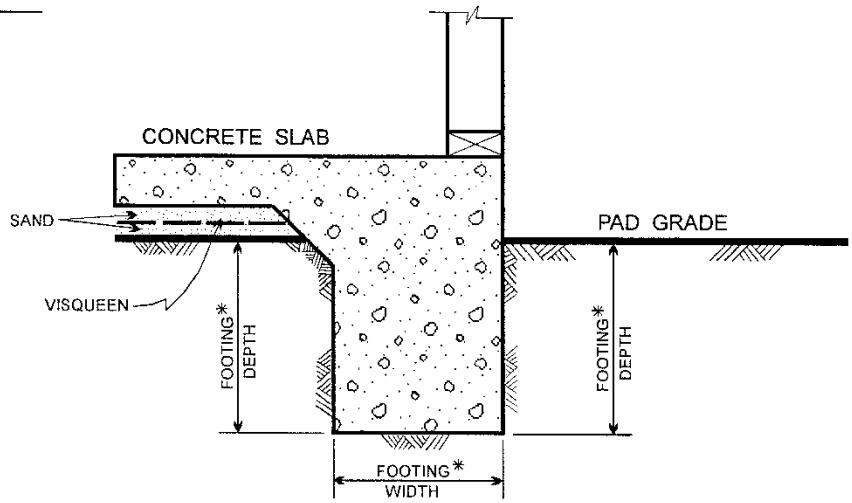
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

RETAINING WALL DRAIN DETAIL

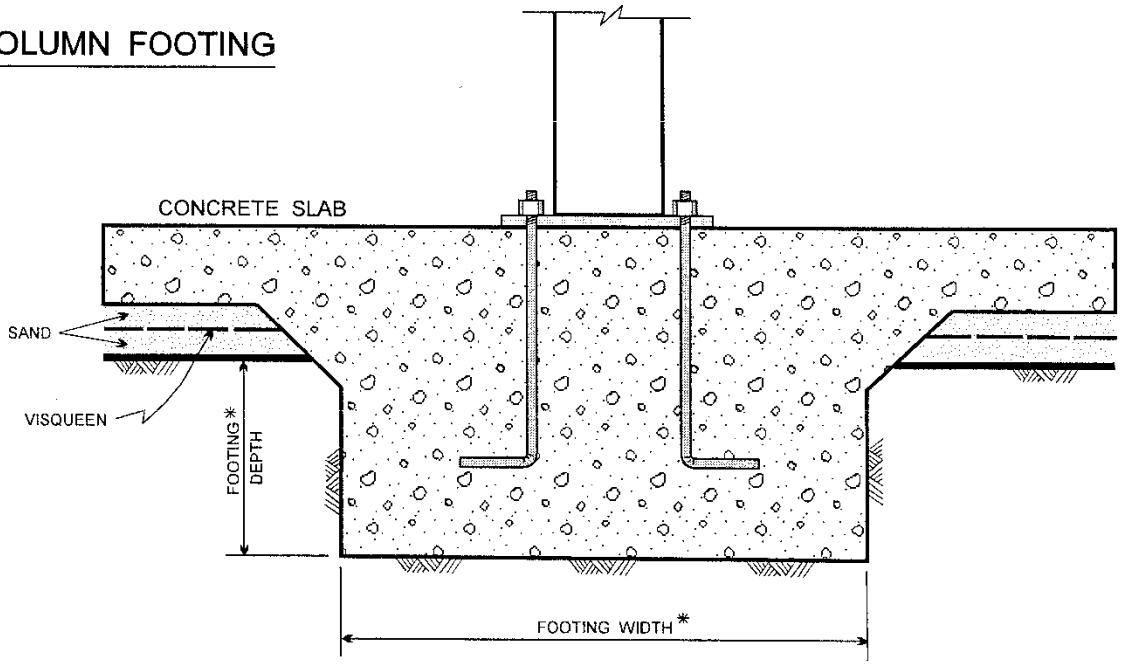
TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

CHL / JL		2000	12 / 2012	PROJECT NO. T2540-22-02	FIG. 6

WALL FOOTING



COLUMN FOOTING



*……SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE SUITE 101 MURRIETA, CA 92562-7065
PHONE 951.304.2300 FAX 951.304.2392

JL		2000

WALL / COLUMN FOOTING DETAIL

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	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)	2.6	(4.2)	6.8	0.844	XI
ELSINORE (GLEN IVY)	5.7	(9.1)	6.8	0.593	X
SAN JACINTO-SAN JACINTO VALLEY	20.6	(33.1)	6.9	0.183	VIII
ELSINORE (JULIAN)	21.8	(35.1)	7.1	0.196	VIII
SAN JACINTO-ANZA	21.9	(35.2)	7.2	0.209	VIII
CHINO-CENTRAL AVE. (Elsinore)	23.3	(37.5)	6.7	0.147	VIII
SAN JOAQUIN HILLS	24.9	(40.0)	6.6	0.127	VIII
WHITTIER	27.5	(44.2)	6.8	0.121	VII
SAN JACINTO-SAN BERNARDINO	28.1	(45.2)	6.7	0.109	VII
NEWPORT-INGLEWOOD (Offshore)	28.3	(45.5)	7.1	0.146	VIII
SAN ANDREAS - SB-Coach. M-1b-2	35.0	(56.3)	7.7	0.172	VIII
SAN ANDREAS - SB-Coach. M-2b	35.0	(56.3)	7.7	0.172	VIII
SAN ANDREAS - San Bernardino M-1	35.0	(56.3)	7.5	0.150	VIII
SAN ANDREAS - Whole M-1a	35.0	(56.3)	8.0	0.210	VIII
ROSE CANYON	35.1	(56.5)	7.2	0.121	VII
NEWPORT-INGLEWOOD (L.A.Basin)	38.2	(61.4)	7.1	0.099	VII
PUENTE HILLS BLIND THRUST	41.1	(66.2)	7.1	0.087	VII
CUCAMONGA	42.2	(67.9)	6.9	0.071	VI
SAN JOSE	42.8	(68.9)	6.4	0.047	VI
NORTH FRONTAL FAULT ZONE (West)	43.4	(69.8)	7.2	0.086	VII
PINTO MOUNTAIN	43.7	(70.4)	7.2	0.089	VII
CORONADO BANK	44.6	(71.8)	7.6	0.121	VII
SAN JACINTO-COYOTE CREEK	44.8	(72.1)	6.6	0.051	VI
PALOS VERDES	45.1	(72.6)	7.3	0.093	VII
CLEGHORN	45.8	(73.7)	6.5	0.045	VI
SIERRA MADRE	45.8	(73.7)	7.2	0.079	VII
SAN ANDREAS - Cho-Moj M-1b-1	48.7	(78.4)	7.8	0.125	VIII
SAN ANDREAS - Mojave M-1c-3	48.7	(78.4)	7.4	0.090	VII
SAN ANDREAS - 1857 Rupture M-2a	48.7	(78.4)	7.8	0.125	VIII
EARTHQUAKE VALLEY	49.3	(79.3)	6.5	0.040	V
NORTH FRONTAL FAULT ZONE (East)	50.1	(80.6)	6.7	0.047	VI
SAN ANDREAS - Coachella M-1c-5	50.5	(81.2)	7.2	0.072	VI
BURNT MTN.	54.9	(88.4)	6.5	0.034	V
CLAMSHELL-SAWPIT	56.7	(91.3)	6.5	0.033	V
RAYMOND	57.0	(91.7)	6.5	0.032	V
HELENDALE - S. LOCKHARDT	57.1	(91.9)	7.3	0.066	VI
UPPER ELYSIAN PARK BLIND THRUST	57.5	(92.6)	6.4	0.029	V
EUREKA PEAK	58.2	(93.7)	6.4	0.029	V

38 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE ELSINORE (TEMECULA) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.6 MILES (4.2 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8438 g

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on October 23 through 30 (fault trenching) and November 7, 2012 (geotechnical borings). Eight borings were excavated with a CME 75 truck mounted drill rig to depths between 20 and 50.25 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 $2^{3}/_{8}$-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. 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-8. 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 and Site Plan (see Figure 2).

PROJECT NO. T2540-22-02

BORING B-1

ELEV. (MSL.) 1336 DATE COMPLETED 11/7/2012

EQUIPMENT CME 75 HSA BY: LAB/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			SM	**YOUNG ALLUVIUM (Qal):** Silty SAND, loose, dry to moist, red brown, coarse - granitic detritus			
2								
4								
6	B1@5				-becomes dense (cemented), slightly porous	57	125.3	3.4
8					-becomes loose, easy drilling			
10	B1@10				-becomes brown, moist to wet, loose, coarse	13	118.4	16.3
12					**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, poorly graded, dense, moist, brown, hard drilling, weakly cemented			
14								
16	B1@15				Clayey SANDSTONE, dense, moist, red brown, mottled coloring, coarse grained sand, weathered granitic clasts, weakly cemented	87/11"	121.2	12.2
18								
20	B1@20					47	126.8	13.5
22								
24					Silty SANDSTONE to Clayey SANDSTONE, moist, yellow brown, coarse grained, manganese staining, mottled coloring, weakly cemented			
26	B1@25				SANDSTONE, poorly graded, medium dense, light yellow brown, coarse grained, non indurated, locally massive, granitic, weakly cemented @ 25.5'-26.0' blow sampler dropped - possible void	42		18.2
28			▼		-very soft easy drilling			

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

T2540-22-02 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. T2540-22-02

BORING B-1

ELEV. (MSL.) 1336 DATE COMPLETED 11/7/2012

EQUIPMENT CME 75 HSA BY: LAB/PDT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30				Silty SANDSTONE, very dense, moist, reddish brown, fine to coarse grained, slow advance, micaceous, trace clay, weakly cemented	70		
32								
34								
36	B1@35				-disturbed sample, water added to extract sample	50/3.5"		
38								
40	B1@40				**GRANITIC BEDROCK (Kgdd):** Black and white, fine to medium grained, granitic rock weathered, some clay	50/5"		
42					-Olive, very fine grained, moist, hard			
44								
46	B1@45				-Black and white fine grained granitic rock weathered	50/3"		
48								
50	B1@50					50/3.5"		

Total depth: 50.25'
Groundwater encountered at 29'
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 2 of 2

T2540-22-02 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. T2540-22-02

BORING B-2

ELEV. (MSL.) 1348 DATE COMPLETED 11/7/2012

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	B2@0-0			SM	**YOUNG ALLUVIUM (Qal):** Silty SAND, dense, dry, brown, fine to coarse, upper 12" disturbed (disked), annual grasses, some shrubs			
2								
	B2@2.5				**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, very dense, moist, reddish brown, fine to coarse grained, some porosity, weakly cemented	64	138.2	5.3
4								
	B2@5				Clayey SANDSTONE, dense, damp, brown, fine to coarse grained, some brownish red mottling, increase in fine sand, non porous, micaceous, weakly cemented	43	124.1	11.4
6								
8	B2@7.5					39	126.8	11.3
					-conglomerate layer			
10	B2@10				-becomes reddish brown, moist, some orange mottling	62	133.7	7.7
12								
	B2@13				-increase in sand, conglomerate in shoe	57		
14								
	B2@15				-becomes light brown with orange mottling, fine grained, trace coarse sand, increase in clay	45		
16								
18								
20	B2@20				**GRANITIC BEDROCK (Kgdd):** Highly weathered, clayey, fine to medium grained	41		
					Total depth: 21' 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

T2540-22-02 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. T2540-22-02

BORING B-3

ELEV. (MSL.) 1352 DATE COMPLETED 11/7/2012

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	B3@0-5				**PAUBA SANDSTONE (Qps):** Silty SANDSTONE/Sandy SILTSTONE, medium dense, dry, reddish brown, fine grained, trace medium to coarse grained sand, weakly cemented			
2	B3@2.5				-becomes very dense	53	131.0	4.0
4					Silty SANDSTONE, very dense, damp, reddish brown, fine to coarse grained, moderately cemented			
6	B3@5-10 B3@5				-conglomerate layer	50/4.5"	107.3	5.7
8	B3@7.5					50/5"		
10	B3@10				-becomes fine to medium grained, trace coarse grained sand, moist, well cemented	94		
12	B3@12.5				Clayey SANDSTONE, very dense, moist, fine to medium grained, trace coarse grained sand, well cemented, trace carbonate stringers	90		
14					-conglomerate layer			
16	B3@15				Silty SANDSTONE, very dense, moist, fine to medium grained, some clay, well cemented	92/10"		
18					**GRANITIC BEDROCK (Kgdd):** Weathered, moist, gray, white, orange, clayey, fine to medium grained, micaceous			
20	B3@20					50/5.5"		

Total depth: 20.5'
No groundwater encountered
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

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

T2540-22-02 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. T2540-22-02

BORING B-4

ELEV. (MSL.) 1356 DATE COMPLETED 11/7/2012

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	B4@0-5			SM	**COLLUVIUM (Qcol):** Silty SAND, dense, dry, reddish brown, fine to coarse, trace gravel, upper 1' disturbed			
2								
	B4@2.5				-becomes very dense, chattering, brownish red, well cemented, damp	50/3.5"	111.8	4.0
4								
	B4@5-10 B4@5				-becomes moist	50/3"	117.4	7.6
6								
8	B4@7.5				-becomes fine to medium grained, some coarse grained sand	50/3.5"		
10	B4@10				**GRANITIC BEDROCK (Kgdd):** Highly weathered, black, gray and white, fine to coarse grained, some clay, moist, micaceous -some olive very fine grained rock with clay	50/4"		
12								
	B4@12.5					50/4"		
14								
	B4@15					50/5"		
16								
18								
20	B4@20					50/3.5"		

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-4,
Log of Boring B-4, Page 1 of 1

T2540-22-02 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. T2540-22-02

BORING B-5

ELEV. (MSL.) 1346 DATE COMPLETED 11/7/2012

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	B5@0-5				**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, very dense, dry, light brown, fine to coarse, trace fine gravel, micaceous, weakly cemented			
2								
	B5@2.5					50/5"		
4								
	B5@5				-Conglomerate layer, moist	50/3.5"		
6					**GRANITIC BEDROCK (Kgdd):** Completely weathered granitic rock, wet, light brown, fine to coarse, micaceous			
8	B5@7.5					50/4"		
10	B5@10				-becomes saturated	50/3"		
12								
	B5@12.5		▼			50/3"		
14								
	B5@15					50/2"		
16								
18					Moderately weathered, white, gray and pink, medium to coarse grained, micaceous			
20	B5@20				-no recovery	50/0.5"		

Total depth: 20'
Groundwater encountered at 13/5'
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-5,
Log of Boring B-5, Page 1 of 1

T2540-22-02 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. T2540-22-02

BORING B-6

ELEV. (MSL.) 1332 DATE COMPLETED 11/7/2012

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	B6@0-5			SM	**YOUNG ALLUVIUM (Qal):** Silty SAND, medium dense, moist, black, fine to medium grained, some coarse grained sand, micaceous			
				CL	Sandy CLAY, very stiff, moist, fine to medium grained, micaceous			
2	B6@2.5					19	123.4	13.1
4								
	B6@5				-becomes hard	32	122.0	13.3
6								
8	B6@7.5				-becomes very stiff, dark grayish brown	20	121.3	15.0
10	B6@10			SW	SAND, loose, wet, gray, fine to coarse grained, micaceous, some silt	6	113.3	17.6
				CL	Sandy CLAY, soft, wet, dark brown, fine grained sand, some coarse sand			
12	B6@12.5					20	121.8	14.8
				SW	SAND, medium dense, wet, light brown, fine to coarse grained,			
14								
	B6@15					15	97.4	26.3
16					-cobbles and sandy gravel			
				CH	CLAY, stiff, moist, olive			
18					**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, loose, brown with orange mottling, micaceous, weakly cemented			
20	B6@20					73		

Total depth: 21'
No groundwater encountered
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-6,
Log of Boring B-6, Page 1 of 1

T2540-22-02 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. T2540-22-02

BORING B-7

ELEV. (MSL.) **1320** DATE COMPLETED **11/7/2012**

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	B7@0-5				**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, dense, damp, brown, fine to medium grained, trace coarse grained sand, weakly cemented, porous up to 1/8", rootlets			
2								
	B7@2.5					57	132.2	6.3
4								
	B7@5				-becomes olive brown, non porous	48	116.0	5.3
6								
8	B7@7.5				-becomes damp, light grayish brown, fine to coarse grained	69		
10	B7@10				-becomes reddish brown	50/5"		
12								
	B7@12.5					50/4"		
14					Clayey SANDSTONE, medium dense, moist, reddish brown, fine to coarse grained, weakly cemented			
	B7@15					45		
16					-becomes brown			
					-becomes reddish brown with orange mottling			
18								
20	B7@20					72		
					Total depth: 21' No groundwater encountered No caving Backfilled with cuttings and tamped Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer			

Figure A-7,
Log of Boring B-7, Page 1 of 1

T2540-22-02 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. T2540-22-02

BORING B-8

ELEV. (MSL.) **1344** DATE COMPLETED **11/7/2012**

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	B8@0-5				**PAUBA SANDSTONE (Qps):** Silty SANDSTONE, medium dense, dry, reddish brown, fine to medium grained, some coarse grained sand, micaceous, weakly cemented			
2	B8@2.5					55	123.7	3.0
4								
6	B8@5-10 B8@5				-becomes damp, trace fine gravel	64	128.2	4.3
8	B8@7.5				-becomes moist	63	122.0	5.9
10	B8@10				-becomes very dense	86		
12	B8@12.5				Clayey SANDSTONE, dense, moist, reddish brown, fine to coarse grained micaceous, weakly cemented	56		
14	B8@15				Sandy CLAYSTONE, hard, moist, reddish brown, fine to medium grained, weakly cemented	58		
16								
18					Clayey SANDSTONE, medium dense, moist, reddish brown, fine to medium grained, weakly cemented			
20	B8@20					47		

Total depth: 21'
No groundwater encountered
No caving
Backfilled with cuttings and tamped
Penetration resistance for 140-lb hammer falling 30 inches by auto-hammer

Figure A-8,
Log of Boring B-8, Page 1 of 1

T2540-22-02 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.

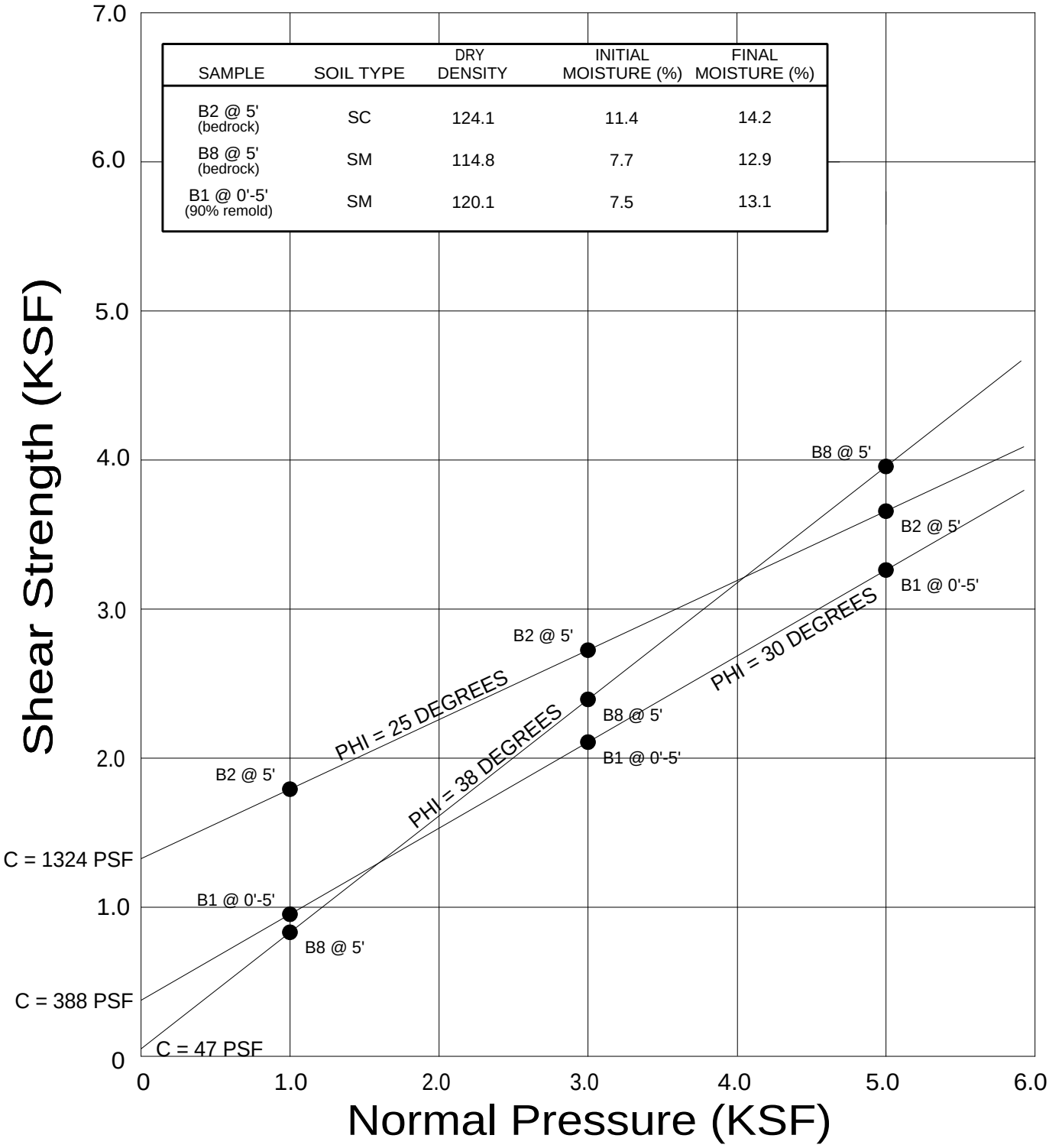
GEOCON

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 (%)
B2 @ 5' (bedrock)	SC	124.1	11.4	14.2
B8 @ 5' (bedrock)	SM	114.8	7.7	12.9
B1 @ 0'-5' (90% remold)	SM	120.1	7.5	13.1

● Direct Shear, Saturated

GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

JL | 2000

DIRECT SHEAR TEST RESULTS

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012 | PROJECT NO. T2540-22-02 | 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				
B6 @ 0'-5'	7.8	16.6	117.1	18	Very Low	Non-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

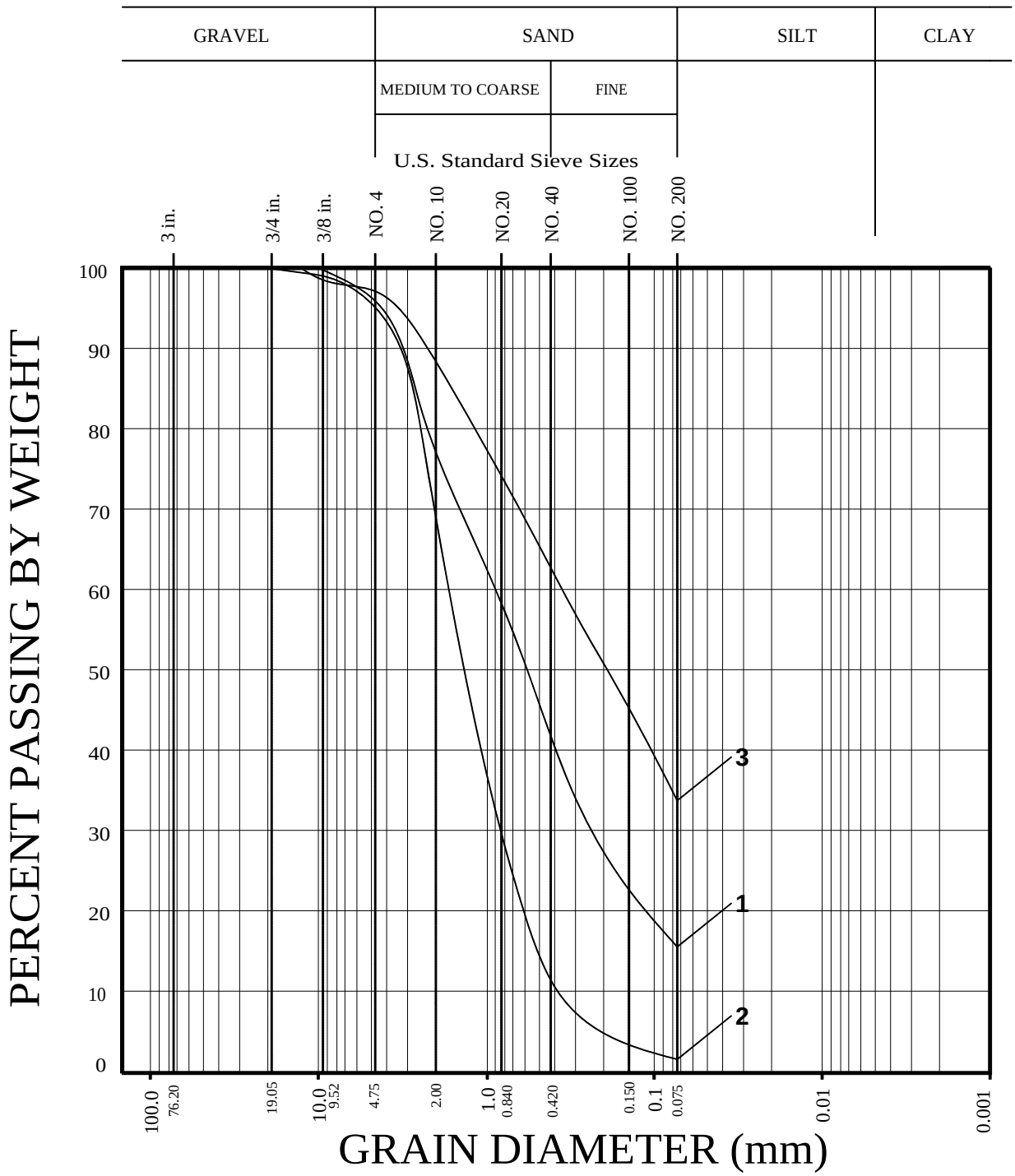
Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 @ 0'-5'	Reddish Brown Silty Sand	133.0	8.0
B8 @ 0'-5'	Reddish Brown Silty Sand	133.5	8.0

GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

LABORATORY TEST RESULTS

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

JL		2000	12 / 2012	PROJECT NO. T2540-22-02	FIG. B2



SAMPLE		UNIFIED SOIL CLASSIFICATION
1 -	B1 @ 20'	SC
2 -	B1 @ 25'	SP
3 -	B1 @ 30'	SM

GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 951.304.2300 FAX 951.304.2392

JL		2000

GRAIN SIZE DISTRIBUTION

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. B3

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

Sample No.	pH	Resistivity (ohm centimeters)
B5 @ 0'-5'	7.57	3600 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS AASHTO T291-94

Sample No.	Chloride Ion Content (%)
B5 @ 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*
B5 @ 0'-5'	0.507	Severe

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

GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562
PHONE 760.579.9926 FAX 951.304.2642

JL		2000

CORROSIVITY TEST RESULTS

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012	PROJECT NO. T2540-22-02	FIG. B4

APPENDIX C

APPENDIX C

BORING LOGS FROM 2005 LANDMARK CONSULTANT'S INVESTIGATION

FOR

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

PROJECT NO. T2540-22-02

CLIENT: The Bergman Companies
METHOD OF DRILLING: CME 55 w/rope and cathead
PROJECT: Proposed Baxter Crossing - Wildomar, CA
DATE OBSERVED: 10/07/05
LOCATION: See Site and Exploration Plan
LOGGED BY: TB

LOG OF BORING B-1

SHEET 1 OF 1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						32
5			60		very dense, with caliche	5.0	111.2				
10			66			5.5	119.8				
15			49		olive brown, dense, moist, trace clay	12.3	117.2				
20			60		CLAYEY SANDY SILT (ML): Olive brown, very dense, moist, with caliche.						
25			36		CLAYEY SILTY SAND (SM): Reddish brown, dense, wet, fine grained.						
30			56		very dense						
35			29	3.0	SILTY CLAY (CL): Reddish brown/olive brown, very stiff, wet.	23.0			39	26	
40			38		CLAYEY SANDY SILT/SILTY SAND (ML/SM): Olive brown, dense, wet, fine grained.						
45			24	2.0	SILTY CLAY (CL): Olive brown, stiff, wet.	27.6			34	21	78
50			13		SAND (SP): Yellowish brown, medium dense, wet.						5
55					End of Boring at 51.5 ft Groundwater Encountered at 24.0 ft ** Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.						

Project No: LP05282

LANDMARK
Geo-Engineers and Geologists

Plate B-1

CLIENT: The Bergman Companies
PROJECT: Proposed Baxter Crossing - Wildomar, CA
LOCATION: See Site and Exploration Plan

METHOD OF DRILLING: CME 55 w/rope and cathead
DATE OBSERVED: 10/07/05
LOGGED BY: TB

LOG OF BORING B-2

SHEET 1 OF 1

DEPTH	CLASSIFICATIO	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SURFACE ELEV. +/-						
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
			50 @ 5"		very dense, with caliche						
5											
			50 @ 4"								
10											
			38		olive brown, dense, trace clay						
15											
			41								
20											
25											
30											
35											
40					End of Boring at 19.0 ft No Groundwater Encountered						

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK
Geo-Engineers and Geologists

Plate B-2

CLIENT: The Bergman Companies
METHOD OF DRILLING: CME 55 w/rope and cathead
PROJECT: Proposed Baxter Crossing - Wildomar, CA
DATE OBSERVED: 10/07/05
LOCATION: See Site and Exploration Plan
LOGGED BY: TB

LOG OF BORING B-3

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
5			61		very dense, moist, trace clay	11.9	111.2				
10			41		olive brown, dense						
15			29		medium dense						
20			33								
25											
30											
35											
40					End of Boring at 21.5 ft No Groundwater Encountered						

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-3

CLIENT: The Bergman Companies
PROJECT: Proposed Baxter Crossing - Wildomar, CA
LOCATION: See Site and Exploration Plan

METHOD OF DRILLING: CME 55 w/rope and cathead
DATE OBSERVED: 10/07/05
LOGGED BY: TB

LOG OF BORING B-4

SHEET 1 OF 1

DEPTH	CLASSIFICATIO	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
5			50 @ 5"		very dense						
10			50 @ 4"								
15			50 @ 5"								
20											
25											
30											
35											
40					End of Boring at 16.5 ft No Groundwater Encountered **Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.						

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-4

CLIENT: The Bergman Companies
PROJECT: Proposed Baxter Crossing - Wildomar, CA
LOCATION: See Site and Exploration Plan

METHOD OF DRILLING: CME 55 w/rope and cathead
DATE OBSERVED: 10/07/05
LOGGED BY: TB

LOG OF BORING B-5

SHEET 1 OF 1

DEPTH	CLASSIFICATIO	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
			49		dense	4.5	111.4				
5											
			50 @ 5"		very dense	6.8	109.5				
10											
			55		moist	12.6	118.8				
15											
			32		dense, trace clay						
20											
25											
30											
35											
40											

End of Boring at 19.0 ft
No Groundwater Encountered

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-5

CLIENT: The Bergman Companies
PROJECT: Proposed Baxter Crossing - Wildomar, CA
LOCATION: See Site and Exploration Plan

METHOD OF DRILLING: CME 55 w/rope and cathead
DATE OBSERVED: 10/07/05
LOGGED BY: TB

LOG OF BORING B-6

SHEET 1 OF 1

SURFACE ELEV. +/-

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
5			14		medium dense	8.3	116.0				
10			22			21.3	97.5				
15			25		CLAYEY SILTY SAND (SM): Reddish brown, medium dense, wet, fine grained.						
20			33		dense						
25											
30											
35											
40					End of Boring at 21.5 ft Groundwater Encountered at 17.0 ft						

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-6

CLIENT: The Bergman Companies
PROJECT: Proposed Baxter Crossing - Wildomar, CA
LOCATION: See Site and Exploration Plan

METHOD OF DRILLING: CME 55 w/rope and cathead
DATE OBSERVED: 10/07/05
LOGGED BY: TB

LOG OF BORING B-7

SHEET 1 OF 1

DEPTH	CLASSIFICATIO	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						35
			40		dense, with caliche	7.0	112.6				
5											
			50 @ 5"		very dense	8.1	115.8				
10											
			28		medium dense						
15											
			37		dense						
20											
25											
30											
35											
40					End of Boring at 19.0 ft No Groundwater Encountered						

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-7

CLIENT: The Bergman Companies
METHOD OF DRILLING: CME 55 w/rope and cathead
PROJECT: Proposed Baxter Crossing - Wildomar, CA
DATE OBSERVED: 10/07/05
LOCATION: See Site and Exploration Plan
LOGGED BY: TB

LOG OF BORING B-8

SHEET 1 OF 1

DEPTH	CLASSIFICATION	SAMPLE TYPE	BLOWS/FOOT **	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/-	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING #200
					SILTY SAND (SM): Reddish brown, damp, fine grained.						
			40		dense, with caliche	3.0	113.8				
5											
			37			9.6	114.2				
10											
			20		medium dense						
15											
			20								
20											
25											
30											
35											
40					End of Boring at 19.0 ft No Groundwater Encountered						

**Blows not corrected for overburden pressure, sampler size or increase drive energy for hammers.

Project No: LP05282

LANDMARK Geo-Engineers and Geologists

Plate B-8

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger than No 200 sieve	Gravels More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands More than half of coarse fraction is smaller than No. 4 sieve	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays Liquid limit is less than 50%		ML	Inorganic silts, clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
			OL	Organic silts and organic clays of low plasticity
	Silts and clays Liquid limit is more than 50%		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 4 10 4 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples

Ring Sample — Standard Penetration Test — Shelby Tube — Bulk (Bag) Sample

Drilling Notes

1. Sampling and Blow Counts
   - Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   - Standard Penetration Test - Number of blows per foot.
   - Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery
4. GWT = Ground Water Table observed @ specified time.

LANDMARK Geo-Engineers and Geologists

Project No: LP05282

Key to Logs

Plate B-9

APPENDIX D

APPENDIX D

FAULT RUPTURE HAZARD INVESTIGATION

GEOLOGIC REVIEW

The eastern portion of the site is located within a Riverside County Fault Hazard Zone established on either side of a previously mapped fault shown to traverse the eastern portion of the site (see Figure 2). The inferred (dashed line) fault location is based on prior studies by Kennedy (1977). The location and orientation of the fault suggests it may be a possible southerly extension of the Glen Ivy fault. The Glen Ivy fault is known to be active northwest of the site (Rockwell, McElwain, Millman and Lamar, 1986; Millman, and Rockwell, 1986; Lamar and Rockwell, 1986).

FAULT ACTIVITY CRITERIA

The criteria used in our investigation to evaluate fault activity are the same criteria used by the California Geological Survey (CGS) that defines an active fault one that has had surface displacement within Holocene time (about the last 11,000 years). These criteria for defining an active fault are based on standards developed by the CGS (Bryant and Hart, 2007) for the Alquist-Priolo Earthquake Fault Zoning Program. Faults that have not moved in the last 11,000 years are not considered active.

In general, the activity rating of a fault is determined by establishing the age of the youngest materials displaced by the fault. If datable material is present, an absolute age can sometimes be established; if no datable material exists, then only a relative age can be assigned to movement on the fault. For faults that have evidence of movement in the last 11,000 years, to be included in an Alquist-Priolo fault zone, these faults must demonstrate evidence of being "sufficiently active and well-defined".

As indicted in CGS Special Publication 42:

- A fault is deemed "sufficiently active" if there is evidence of Holocene surface displacement along one or more of its segments or branches. Holocene surface displacement may be directly observable or inferred and does not need to be present everywhere along a fault to qualify a fault for zoning.

- A fault is considered "well-defined" if its trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The fault may be identified by direct observation or by indirect methods. The critical consideration is that the fault or

some part of it can be located in the field with sufficient precision and confidence to indicate that the required site-specific investigations would meet with some success.

LINEAMENT ANALYSIS

We performed an aerial photograph review to evaluate the location of mapped and unmapped fault traces that may be present at the site. Faults that cannot be observed in the field can often be identified by linear topographic expression or tonal lineaments observed on aerial photographs.

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, see References.

Lineaments observed on the aerial photographs 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.

The lineament associated with the mapped fault by Kennedy (1977) was observed on the aerial photographs. Kennedy's mapped lineament trends N33W and is dashed indicating the fault is inferred but no direct physical evidence for its existence has been observed in the field at the location. We observed the same location and trend of the lineament as Kennedy (1977). The lineament is weak and is the only linear feature identified on the aerial photos to be present on or projecting toward the site. The lineament location coincided with what appeared to be a discontinuous break in slope across some planar ridgelines.

FIELD INVESTIGATION

A fault rupture hazard investigation was performed to determine the presence, location, and relative age of faults that may be present within the county-designated fault hazard zone at the site. Our investigation was performed in general accordance with the Alquist-Priolo Act of 1972, with the California Geological Survey (CGS) *Guidelines for Evaluating the Hazard of Surface Fault Rupture* (Note 49) and with *Guidelines for Evaluating and Mitigating Seismic Hazards in California* (CGS Special Publication 117A, 2008).

Our field investigation was performed October 23 through 30, 2012 and consisted of excavation of two fault trenches totaling 690 lineal feet. The trenches extended from the eastern property line toward the west-southwest and encompassed the limits of the Riverside County Fault Hazard Zone at the site. The trenches were excavated approximately perpendicular to the mapped fault trace shown on the county fault maps. The depth of the trenches ranged from 4 to 9 feet deep. Where necessary, the trenches were benched at an effective slope ratio of 1:1 (horizontal:vertical) to provide safe working conditions. 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 unbroken formation below the feature, or which were overlaid my unbroken colluvial soils the features were classified as fractures/ground cracks. The trench walls were scraped clean of smeared soils and a level line was strung to accurately depict the trench geometry. Soil and rock conditions encountered in the trench excavations were visually observed, classified and logged at a scale of 1 inch equals 5 feet in general accordance with California Geologc Survey (CGS) criteria 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 in this Appendix as Plates D1 and D2. Locations of the trenches are shown on the Geologic map and Site Plan, Figure 2. Trenches were 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 from the eastern property line toward the southwest to intercept the mapped lineament (fault) and to encompass the majority of the Riverside County Fault Hazard Zone. FT-1 was 450 feet long and ranged in depth from 5 to 9 feet below the existing ground surface. The trench excavation exposed granitic bedrock (granodiorite) overlain by colluvium and younger alluvium. The contacts between these units was distinct and generally dipped gently to the west and southwest. Granodiorite was observed in the trench between Stations 0+00 and 2+35. The granodiorite was highly weathered and exhibited abundant joints which trended generally east-west to nearly east-west and dipped steeply to the north and south. Colluvium was observed in the trench between Stations 0+00 and 3+45 overlying the granitic bedrock. The colluvium consists of clayey sand which is dark red-brown (5YR 3/3) with abundant carbonate stringers and nodules. Soil structure was observed to be columnar prismatic with clay developed on parting surfaces and weathering rinds around granitic grains. The alluvium consisted of silty sand which was dark brown (7.5 YR 3/3) loose to medium dense, porous, weakly to moderately blocky with some clay development on ped facies, and had weak horizontal bedding. The alluvium was observed to be continuous for the total length of the trench.

The geologic units observed in FT-1 were laterally continuous and there is no evidence which indicates faulting occurred within the geologic units exposed in the trench excavation.

Fault Trench 2 (FT-2): FT-2 was excavated approximately 162 feet north of FT-1 from FT-1 Station 3+35, southeastward for 240 feet to cover the remaining Riverside County Fault Hazard Zone. FT-2 was necessary to provide coverage in older geologic units due to the deepening alluvium within the southwestern portion of FT-1. FT-2 was 4 to 6 feet deep and exposed Pauba sandstone overlain by 6- to 12-inches of alluvium/topsoil. The excavation exposed locally massive Pauba sandstone consisting of brown (7.5 YR 4/4) dense silty sand. The overlying alluvial soils consisted of silty sand which is dark brown (7.5 YR 3/3) loose to medium dense, porous, weakly blocky, and weak horizontal bedding. Both units extended the entire length of the trench. Locally, fractures were observed within the Pauba but did not extend to the top of the unit. Where they could be traced across the trench, they trended nearly east-west, similar to the jointing underlying granitic bedrock. Some fractures could not be traced across the trench. There was no evidence of movement (slickensides, clay development, offset units) that would indicate faulting has occurred within the geologic units exposed on the site.

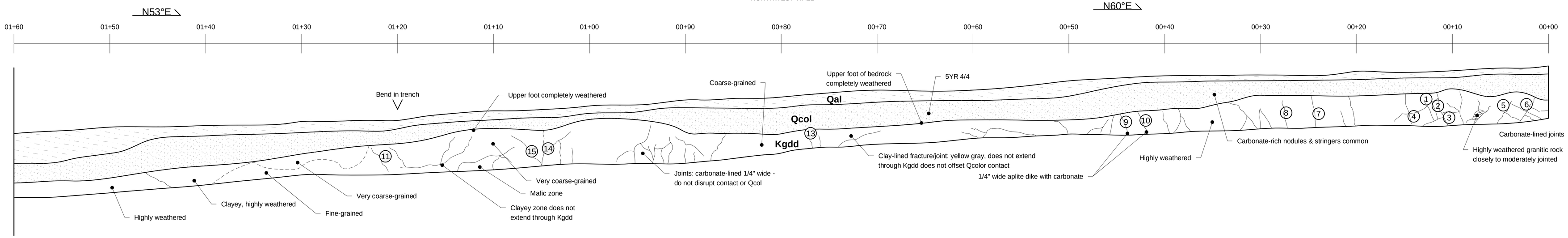
CONCLUSIONS

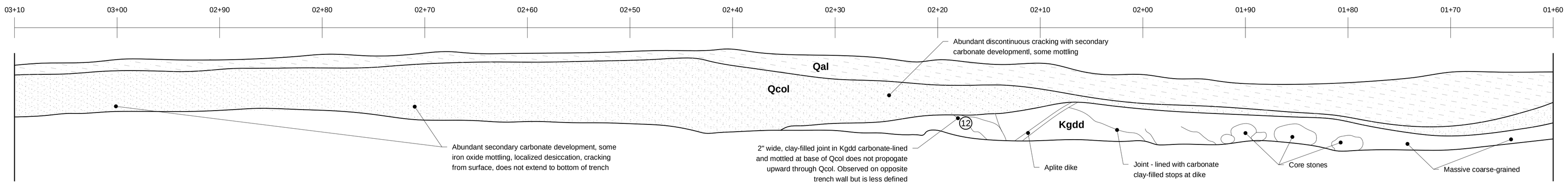
Based on the results of our investigation, we conclude that active faults are not present at the site. The contacts between geologic units are laterally continuous in the trench exposures and no evidence of offset or faulting was observed. If faults are present at depths below our explorations, these faults would not be considered active based on the minimum age of the sediments exposed at the base of the trench (Pleistocene age or older).

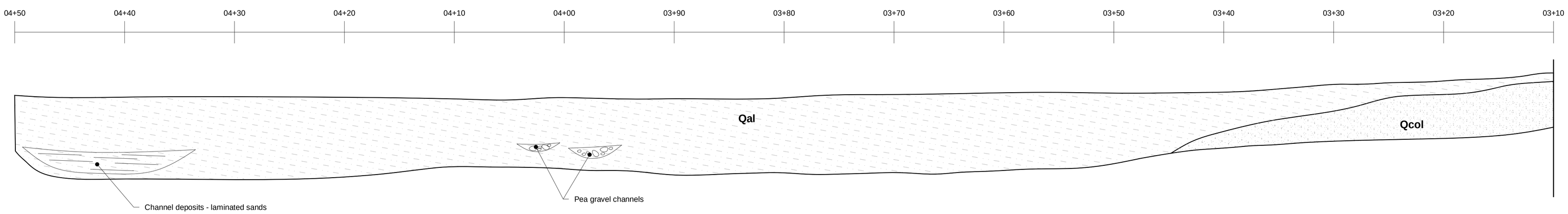
Based on our findings, no restrictions on future development of the site are necessary with respect to the hazard of surface fault rupture, beyond the standard seismic engineering requirements for all buildings in California.

FT-1

NORTHWEST WALL







1	Joint: EW, 84N
2	Joint: EW, 81N
3	Joint: N80E, 83N
4	Joint: N3W, 76SE
5	Joint: N70W, 60SW
6	Joint: N73W, 64SW
7	Joint: N80E, 83S, carbonate filled
8	Joint: EW, 86N
9	Joint: N80E, 81N
10	Joint: EW, V
11	Joint: N80E, V
12	Joint: N80E, 60N
13	Joint set: N79E, 82N. N73W 33SW
14	Joint: N85E, 87N
15	Joint: N80E, 81N

LEGEND

Qal	Alluvium - (topsoil in upper 6"-1') Silty Sand, loose to medium dense, dry, dark brown 7.5 YR 3/3 to dark yellow brown 10 YR 3/4, fine to coarse, some angular to subrounded gravel to 1/4" in diameter weakly developed horizontal bedding, porous, weak to moderate columnar structure with some clay development on ped facies
Qcol	Colluvium: Clayey Sand, dense, dry to moist, dark red brown 5 YR 3/3, abundant carbonate nodules and stringers, clay development on ped facies, well developed prismatic and columnar structure
Qps	Pauba Sandstone, dense, dry to moist, brown 7.5 YR 4/4, locally massive, coarse, slightly to moderately weathered, soft, unfractured
Kgdd	Granodiorite: Hard, moist, highly weathered complete weathering in upper foot is common, localized areas of fresh core stones, predominantly fine to medium grained with localized areas of coarse grained rock, occasionally cut by aplite dikes closely to moderately jointed
K	Krotovina (Animal Burrow)

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CALIFORNIA 92562
PHONE 951.304.2300 FAX 951.304.2392

JL		2000

TRENCH LOGS

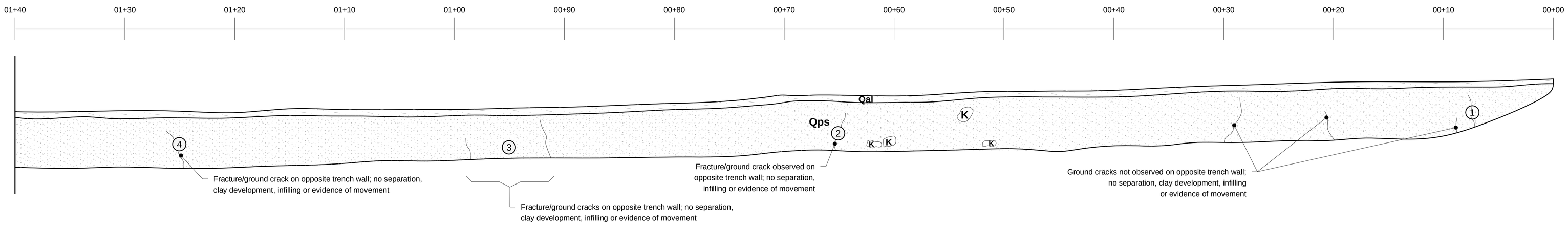
TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

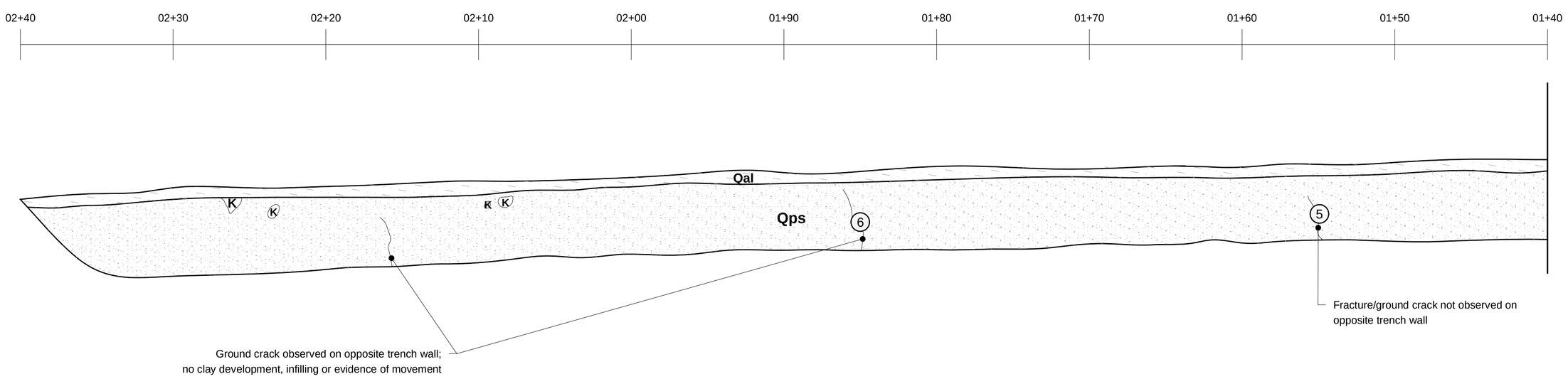
12 / 2012	PROJECT NO. T2540-22-02	PLATE D1

FT-2

N30°E

NORTHWEST WALL





1 fracture/ground crack: N80W, variable dip

2 fracture/ground crack: N80W, variable dip

3 fracture/ground cracks: N80W, variable dip

4 fracture/ground crack: N82W, variable dip

5 fracture/ground crack: N83W, variable dip

6 fracture/ground crack: N81W, variable dip

LEGEND

Qal	Alluvium - (topsoil in upper 6"-1') Silty Sand, loose to medium dense, dry, dark brown 7.5 YR 3/3 to dark yellow brown 10 YR 3/4, fine to coarse, some angular to subrounded gravel to 1/4" in diameter weakly developed horizontal bedding, porous, weak to moderate columnar structure with some clay development on ped facies
Qcol	Colluvium: Clayey Sand, dense, dry to moist, dark red brown 5 YR 3/3, abundant carbonate nodules and stringers, clay development on ped facies, well developed prismatic and columnar structure
Qps	Pauba Sandstone, dense, dry to moist, brown 7.5 YR 4/4, locally massive, coarse, slightly to moderately weathered, soft, unfractured
Kgdd	Granodiorite: Hard, moist, highly weathered complete weathering in upper foot is common, localized areas of fresh core stones, predominantly fine to medium grained with localized areas of coarse grained rock, occasionally cut by aplite dikes closely to moderately jointed
K	Krotovina (Animal Burrow)

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CALIFORNIA 92562
PHONE 951.304.2300 FAX 951.304.2392

JL | 2000

TRENCH LOGS

TRACT 34301
NWC BAXTER ROAD AND I-15
WILDOMAR, CALIFORNIA

12 / 2012 | PROJECT NO. T2540-22-02 | PLATE 02