Attachment H:
Geology and Soils

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT LAS TERRAZAS AT COLTON 275 AND 291 CYPRESS AVENUE UNINCORPORATED SAN BERNARDINO COUNTY, CALIFORNIA APN: 0274-182-34, -43, & -46

PREPARED FOR
LAS TERRAZAS FUND, L.P.
AGOURA HILLS, CALIFORNIA

PROJECT NO. A8898-06-01

SEPTEMBER 5, 2014

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A8898-06-01
September 5, 2014

VIA OVERNIGHT DELIVERY

Las Terrazas Fund, L.P.
30141 Agoura Road, Suite 100
Agoura Hills, CA 91301

Attention: Mr. Jay Ross

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
LAS TERRAZAS AT COLTON
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CALIFORNIA
APN: 0274-182-34, -43, &-46

Dear Mr. Ross:

In accordance with your authorization of our proposal, we have performed an updated geotechnical investigation for the proposed Las Terrazas at Colton residential development located at 275 and 291 Cypress Avenue in the Unincorporated San Bernardino County, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations in this report are followed and implemented during design and construction.

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

Very truly yours,
GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
JELISA M. THOMAS
74946
CIVIL
STATE OF CALIFORNIA

Jelisa M. Thomas
PE 74946

REGISTERED PROFESSIONAL ENGINEER
NEAL D. BERLINER
2576
GEOTECHNICAL
STATE OF CALIFORNIA

Neal D. Berliner
GE 2576

(Email) Addressee

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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE

This report presents the results of an updated geotechnical investigation for the proposed Las Terrazas at Colton residential development located at 275 and 291 Cypress Avenue in the Unincorporated San Bernardino County, California (see Figure 1, Vicinity Map). 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 aspects of proposed design and construction.

The scope of our investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was initially explored on December 19, 2011 by excavating nine 7-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 5½ and 20½ feet below the existing ground surface. Percolation testing for the design of a stormwater infiltration system was performed two of the borings. A supplemental site exploration was performed on January 28, 2013 by excavating four 4-inch diameter borings using manual hand auger equipment. The borings were advanced to depths between 4½ and 10½ feet below the ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan, Figure 2. A detailed discussion of the 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.

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.

2. SITE AND PROJECT DESCRIPTION

The subject property is located at 275 and 291 Cypress Avenue in Unincorporated San Bernardino County, California. The property is a 6.14-acre, irregularly shaped parcel. The majority of the parcel is currently vacant, with a vacant single-family residential structure located within the eastern portion of the site. The property is bounded by existing single-family residential structures to the north and northeast, by Cypress Avenue to the southeast, by West Valley Boulevard to the south, and by an existing public storage facility to the west. The site slopes gently to the south and southwest with approximately 10 feet of vertical relief across the site. Water drainage at the site appears to be by sheet flow along the existing ground contours towards the city streets. Vegetation on site consists of grass and shrubs located throughout the site.

Information concerning the proposed development was furnished by the client and is preliminary in nature. It is our understanding that the proposed development will consist of one 2-story and four 3-story multi-family residential structures, a 2,000 square-foot single-story community building, a 2,800 square-foot child care center, a community garden, a swimming pool, and paved parking lot areas to be constructed at or near existing site grade (see Site Plan, Figure 2).

Due to the preliminary nature of the design at this time, wall and column loads were not made available. It is anticipated that Type V wood-frame construction will be utilized, and it is estimated that wall loads for the proposed structures could be up to 3 kips per linear foot, and column loads could be up to 300 kips.

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

3. GEOLOGIC SETTING

The site is located along the eastern edge of the Chino Basin in San Bernardino County. The Chino Basin encompasses a broad area of coalescing alluvial fans that extend southward from the San Gabriel Mountains. The Chino Basin overlies a down-dropped structural block, the Perris Block which is bounded by the Chino and Elsinore Faults to the southeast, the Puente hills to the west, the San Gabriel Mountains to the north, by the San Jacinto fault to the northeast, and the La Sierra Hills and Juniper Mountains to the south east. The alluvial deposits within the Chino Basin have been reworked by wind during the Holocene (last 11,000 years) and Pleistocene (11,000 to 2 million years) epochs. As a result, a thin veneer of eolian sand covers extensive areas of the Chino Basin.

Regionally, the Chino Basin is located within the Peninsular Ranges geomorphic province. This province comprises the northwesterly-trending mountains and valleys extending from the southern Baja Peninsula to the Transverse Ranges in Southern California.

4. GEOLOGIC MATERIALS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of artificial fill underlain by Pleistocene Age older alluvial deposits (Morton, 1978). The soil and geologic units encountered at the site are discussed below. Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Various amounts of artificial fill were encountered throughout the area of the proposed development. The fill was observed in our field explorations to a maximum depth of 4½ feet below existing ground surface. The artificial fill generally consists of brown to yellowish brown silty sand and sandy silt. The artificial fill is characterized as dry and medium dense or soft. The fill is likely the result of past grading and demolition activities at the site. Deeper fill may occur between borings and on other parts of the site that were not directly explored.

4.2 Older Alluvium

The artificial fill is underlain by Pleistocene Age older alluvial deposits generally consisting of brown to yellowish brown poorly graded sand, silty sand, and sandy silt with varying amounts of gravel. The soils are primarily dry to slightly moist and medium dense to very dense, and become denser with increased depth.

5. GROUNDWATER

A review of data provided by the California Department of Water Resources (CDWR, 2011) indicates that several wells have historically been drilled in the site vicinity. The closest wells to the site are Well No. 01S04W19E001S and Well No. 01S05W24H002S, located approximately 0.29 miles west and 0.36 miles northwest of the site. The State well numbering system is based on the township, range, section, and tract in which the well is located.

Review of the monitoring data between 1964 and 1997 for Well No. 01S04W19E001S indicates that the depth to groundwater has fluctuated between 148.4 and 193.9 feet beneath the ground surface. The most recent groundwater level measurement for Well No. 01S04W19E001S was measured in October 1997 at a depth of 162.8 feet below the existing ground surface (CDWR, 2011).

Review of the monitoring data between 1997 and 2008 for Well No. 01S05W24H002S indicates that the depth to groundwater has fluctuated between 172.4 and 190.7 feet beneath the ground surface. The most recent groundwater level measurement for Well No. 01S05W24H002S was measured in April 2008 at a depth of 189.1 feet below the existing ground surface (CDWR, 2011).

Site exploration drilled to a maximum depth of 20½ feet below the ground surface, did not encounter groundwater. Based on these considerations, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to excessive irrigation or precipitation. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the *Surface Drainage* section of this report (see Section 7.23).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (formerly known as California Division of Mines and Geology (CDMG)) for the Alquist-Priolo Earthquake Fault Zone Program (Hart, 1999). 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. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The site, however, is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Rialto Colton Fault located approximately 0.4 miles northeast of the site (Ziony and Jones, 1989). Other nearby active faults are the San Jacinto Fault Zone, the San Andreas Fault Zone, the Mill Creek Fault, and the Crafton Hills Fault Zone located 2.0 miles northeast (CDMG, 1977), 8.0 miles northeast, 8.3 miles northeast and 8.8 miles east-southeast of the site, respectively (Ziony and Jones, 1989).

The closest potentially active fault to the site is the Little Creek Fault located approximately 3.5 mile north of the site (Ziony and Jones, 1989). Other nearby potentially active faults are the Grass Valley Fault and the Tunnel Ridge Fault located approximately 15 miles north and 15 miles north-northeast of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows Earthquake, and the January 17, 1994 M_w 6.7 Northridge Earthquake were a result of movement on the buried thrust faults. These thrust faults are not exposed at the surface and do not present a potential surface fault rupture hazard; however, these active features are capable of generating future earthquakes.

6.2 Seismicity

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

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	26	S
San Jacinto-Hemet area	April 21, 1918	6.8	30	SE
Near Redlands	July 23, 1923	6.3	7	SE
Long Beach	March 10, 1933	6.4	48	SW
Tehachapi	July 21, 1952	7.5	115	NW
San Fernando	February 9, 1971	6.6	65	NW
Whittier Narrows	October 1, 1987	5.9	42	W
Sierra Madre	June 28, 1991	5.8	40	NW
Landers	June 28, 1992	7.3	53	ENE
Big Bear	June 28, 1992	6.4	31	NE
Northridge	January 17, 1994	6.7	69	WNW
Hector Mine	October 16, 1999	7.1	71	NNE

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

6.3 Estimation of Peak Ground Accelerations

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

6.3.1 Deterministic Analysis

Table 1 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 Sadigh et al. (1997). The resulting calculated peak horizontal accelerations at the site are indicated 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 6.7 event on the San Jacinto – San Bernardino Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.731.

While listing of peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site.

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

6.3.2 Probabilistic Analysis

The computer program *FRISKSP* (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance for given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary Fault is proportional to the faults' slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone.

Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh et al. (1997) 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 (DE) 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 DE is typically used for the design of non-critical structures.

Based on the computer program *FRISKSP* (Blake, 2000), the MCE and DE are expected to generate ground motions at the site of approximately 1.28g and 0.90g, respectively. Graphical representation of the analysis is presented on Figure 5.

6.4 Seismic Design Criteria

The following table summarizes 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 data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.122g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.959g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.122g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.439g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.415g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.959g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.821g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.821	Section 11.8.3 (Eqn 11.8-1)

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

6.5 Liquefaction Potential

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

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California" requires liquefaction analysis to a depth of fifty 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 County of San Bernardino General Plan (2005) this site is not located in an area designated as "liquefiable". As stated previously, the depth to groundwater at the site is greater than 50 feet beneath the existing ground surface. Based on these considerations, it is our opinion that the potential for liquefaction of the site soils is very low. Further, no surface manifestations of liquefaction are expected at the subject site.

6.6 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. Based on the relatively dense nature of the older alluvium, appreciable seismically-induced settlements are not anticipated.

6.7 Landslides

According to the County of San Bernardino General Plan (2005) the site is not located within an area identified as having a potential for seismic slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.

6.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. A review of the County of San Bernardino General Plan (2005) indicates that the site is not located within the inundation boundaries of upgradient dams or reservoirs. The probability of earthquake-induced flooding is considered very low.

6.9 Tsunamis and Seiches

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

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is in an area which flood hazards are undetermined, but possible (Zone D) as defined by the Federal Emergency Management Agency (FEMA).

6.10 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-7, the site is not located within the boundaries of an oil field. No oil wells are located in the immediate vicinity of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map. Other wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a known oil field; therefore, the potential for the presence of a methane zone is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.11 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 The depth of artificial fill encountered during field exploration was observed to be variable, with a maximum depth of 4½ feet. The existing fill encountered is believed to be the result of past grading and/or demolition activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations, slabs, or additional fill.

7.1.3 The results of our laboratory testing indicate that the existing upper alluvial soils are subject to excessive hydro-consolidation upon saturation (see Figures B4 through B14). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils or foundations supported therein.

7.1.4 It is our opinion that the existing artificial fill and upper alluvial soils, in their present condition, are not suitable for direct support of proposed foundations, slabs, or additional fill. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (See Section 7.4).

7.1.5 Based on these considerations, as a minimum it is recommended that the upper six feet of existing site soils be excavated and properly compacted for foundation and slab support. Deeper excavation should be conducted as necessary at the direction of the Geocon representative to completely remove all existing artificial fill and soft alluvial soil. The excavation should extend laterally a minimum distance of five feet beyond the building footprint areas or for a distance equal to the depth of fill below the foundations, whichever is greater. Prior to placing any fill, the excavation bottom must be proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon). If determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. Recommendations for earthwork and bottom stabilization are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 Subsequent to the recommended grading, the proposed structure may be supported on conventional foundations deriving support on the newly placed engineered fill. As a minimum, all proposed building foundations deriving support in engineered fill should be underlain by at least three feet of newly placed engineered fill, and grading should be conducted as necessary to maintain the recommended three-foot-thick engineered fill blanket beneath all foundations.

7.1.7 As an alternative to conventional foundations, a post-tensioned concrete slab and foundation system may be utilized for the support of the proposed on-grade structures. A post-tensioned foundation system can be utilized to reduce the potential for foundation distress resulting from differential settlement of the underlying soils. As a minimum, post-tensioned foundations deriving support in engineered fill should be underlain by at least two feet of newly placed engineered fill, and grading should be conducted as necessary to maintain the recommended 2-foot-thick engineered fill blanket beneath all foundations.

7.1.8 It is anticipated that stable excavations can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.19).

7.1.9 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 4 feet. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils 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.

7.1.10 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium may experience increased settlement and/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 properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.11 Percolation testing of the site soils indicates that the soils are capable of infiltration. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.22).

7.1.12 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.1.13 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all 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.

7.2.3 All 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 shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.19).

7.2.4 The upper few feet of soils encountered during this investigation are considered to have a "very low" expansive potential (EI=9); and are 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 these materials.

7.3 Minimum Resistivity, pH, Chloride and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that a potential for corrosion of buried ferrous metals exists on site. The results are presented in Appendix B (Figure B20) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B20) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318-08 Sections 4.2 and 4.3.

7.3.3 Geocon West, Inc. 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 corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

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

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

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

7.4.4 As a minimum, it is recommended that the upper 6 feet of existing site soils be excavated and properly compacted within the proposed building footprint areas. Any encountered deeper fill or soft soils should be completely over-excavated or stabilized as necessary at the direction of the Geotechnical Engineer. Deeper excavations should be conducted as necessary to maintain the recommended 3-foot-thick engineered fill blanket beneath proposed conventional foundations, and 2-foot-thick engineered fill blanket beneath proposed post-tensioned foundations. Where excavation and compaction is to be conducted, the excavation 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. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. Prior to placing any fill, the upper twelve inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 If subgrade stabilization is required at the excavation bottom, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved by introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils.

7.4.7 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.4.8 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/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, moisture conditioned to optimum moisture content and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.9 Prior to construction of exterior slabs and paving, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.4.10 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.19).

7.4.11 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will not be tied-in to the proposed building, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 4 feet below the ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.12 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.4.13 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (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 observed 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 minimum2-sack slurry is also acceptable. Prior to placing any bedding materials or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.14 Import soils may be required to maintain site elevations during grading. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B20). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 20 percent should be anticipated when excavating and compacting the existing fill and alluvium on site to an average relative compaction of 92 percent.

7.5.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed structures provided foundations derive support exclusively in newly placed engineered fill. Conventional spread foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.6.2 As an alternative to conventional foundations, a post-tensioned concrete slab and foundation system may be utilized for the support of the proposed on-grade structures. Recommendations for post-tensioned foundations are provided in Section 7.10.

7.6.3 Continuous foundations may be designed for an allowable bearing capacity of 2,500 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.

7.6.4 Isolated spread foundations for the proposed building may be designed for an allowable bearing capacity of 2,800 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 18 inches into the recommended bearing material.

7.6.5 The soil bearing pressure above may be increased by 200 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

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

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

7.6.8 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. Additional grading should be performed as necessary in order to maintain the required three-foot-thick engineered fill blanket beneath conventional spread foundations.

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

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

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

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

7.7 Miscellaneous Foundations

7.7.1 Foundations for small outlying structures, such as block walls up to6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 4 feet. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.7.2 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.7.3 If the soils exposed in the excavation bottom are soft, compaction of the soft soils 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, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.8 Conventional Foundation Settlement

7.8.1 The maximum expected settlement for the structure supported on a conventional foundation system with a maximum allowable soil bearing pressure of 4,000 psf is estimated to be approximately 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential static settlement is not expected to exceed ¾ inch over a distance of twenty feet.

7.8.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Lateral Design

7.9.1 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 0.34 may be used with the dead load forces in the properly compacted engineered fill and the undisturbed alluvium found at or below a depth of 4 feet.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or the undisturbed alluvium found at or below a depth of 4 feet may be computed as an equivalent fluid having a density of 200 pcf with a maximum earth pressure of 2,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Foundation Design – Post-Tensioned Foundation System

7.10.1 If utilized, post-tensioned concrete slab and foundation systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) Third Edition as required by the 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The parameters presented in the following table are based on the guidelines presented in the PTI, Third Edition design manual, as well as the consideration of the granular, non plastic nature of the upper site soils.

POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Value
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3
Edge Lift, y_M (inches)	0.61
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	0.30

7.10.2 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

7.10.3 Consideration should be given to using interior stiffening beams and connecting isolated footings as well as patio slabs which exceed 5 feet in width to the building foundation to reduce the potential for future separation to occur.

7.10.4 If the structural engineer proposes a post-tensioned foundation design method other than the PTI, Third Edition design manual:

- The post-tensioned foundation system design parameters above are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 18 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.10.5 Foundations may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf) and should derive support exclusively in engineered fill. This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. Based on an anticipated allowable bearing pressure of 2,500 psf, it is recommended that the proposed structures be designed for a differential settlement of ½-inch over a distance of 20 feet.

7.10.6 The upper five feet of existing site soils encountered during this investigation are considered to have a "very low" expansive potential (EI=9). Post-tensioned foundation systems deriving support in soil possessing a "very low" expansion potential (expansion index of 20 or less) may be designed using the method described in Section 1808 of the 2013 CBC; or an alternative, commonly accepted design method (other than PTI Third Edition) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of ¾ inch. Geocon West, Inc. should be contacted to review the plans and provide additional information, if necessary.

7.10.7 Provided the moisture content in the soil is maintained subsequent to completion of grading, special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be maintained at two percent above optimum moisture content prior to and at the time of concrete placement as would be expected in any such concrete placement.

7.10.8 During the construction of the post-tension foundation system, the concrete should be placed monolithically and must be observed and approved by a Geocon inspector. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.12).

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

7.11.3 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 1643-98 and the manufacturer's recommendations. A minimum thickness of 15 mils and a permeance of less than 0.01 perms is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. 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. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.4 For seismic design purposes, a coefficient of friction of 0.34 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.11.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of the subgrade should be moisture conditioned to near optimum moisture content and compacted to at least 95 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. Construction joints should be designed by the project structural engineer.

7.11.6 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 expansive soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all soft or unsuitable alluvial soils in the area of new paving is not required, however, paving constructed over existing unsuitable soils may experience increased settlement and/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 recompacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 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.

7.12.3 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. 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)
Driveways	5	3	6
Trash Truck & Fire Lanes	7	4	10

7.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.12.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 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).

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

7.13 Swimming Pool/Spa

7.13.1 The proposed swimming pool shell bottom should derive support exclusively in newly placed engineered fill and should be underlain by at least 3 feet of engineered fill. Swimming pool foundations and walls may be designed in accordance with the *Conventional Foundation Design* and *Retaining Wall Design* sections of this report (See Sections 7.6 and 7.14). A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

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

7.14 Retaining Wall Design

7.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls significantly higher than 10 feet are planned, Geocon should be contacted for additional recommendations.

7.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* sections of this report (see Section 7.8).

7.14.3 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 38 pcf.

7.14.4 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 56 pcf.7.14.4 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 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

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

7.15 Dynamic (Seismic) Lateral Forces

7.15.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2013 CBC).

7.15.2 A seismic load of 28 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. This pressure is based on the Design Earthquake peak ground acceleration (⅔PGA_M) calculated from ASCE 7-10 Section 11.8.3.

7.16 Retaining Wall Drainage

7.16.1 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.

7.16.2 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.

7.16.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

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

7.17 Elevator Pit Design

7.17.1 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. The elevator slab and retaining wall footings should derive support in newly placed engineered fill and excavations should be conducted as necessary during mass grading to maintain at least two feet of engineered fill beneath blanket beneath the elevator pit slab and retaining wall foundations. Elevator pit walls may be designed in accordance with the recommendations in the *Conventional Foundation Design and Retaining Wall Design* section of this report (see Sections 7.8 and 7.14).

7.17.2 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.

7.17.3 Retaining wall drainage should be designed in accordance with Section 7.16 of this report. The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.17.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

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

7.18 Elevator Piston

7.18.1 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.

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

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

7.19 Temporary Excavations

7.19.1 Excavations on the order of 6 feet in vertical height may be required for the proposed grading of the site. The excavations are expected to expose fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose fill or sands are not present and where not surcharged by adjacent traffic or structures.

7.19.2 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. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.19.3 Continuous vertical excavation adjacent to and which extend below the existing footings could remove vertical and lateral support from the existing footings and are not recommended. Slot cutting or shoring will be required where the proposed excavations will be deeper than an existing adjacent foundation. Recommendations for both excavation methods are provided in the following sections.

7.19.4 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 soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.20 Slot Cutting

7.20.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation is made at a slope of 1:1. Alternate "A" slots of 3.9 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should each be 3.9 feet in width. The wall, foundation, or backfill should be completed in the "A" slots before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. If preferable to the contractor A-B slot-cutting may be utilized. Slot-cutting is not recommended for vertical excavations greater than 5 feet in height or where surcharged by more than 1,000 pounds per linear foot. The surcharge load from the existing offsite structure to the west should be evaluated by a qualified structural engineer, and the slot-cut calculation revised as necessary. A slot-cut calculation is provided below.

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	115.0	pcf
Friction Angle of Soils	(ϕ)	28.0	degrees
Cohesion of Soils	(c)	95.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.53	

Surcharge Pressure:

Line Load	(q_L)	1000.0	psf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

$b = H/(\tan\alpha)$

$A = 0.5 * H * b$

$W = 0.5 * H * b * \gamma$ *(per lineal foot of slot width)*

$F_1 = d * W * (\sin\alpha) * (\cos\alpha)$

$F_2 = d * L$

$R_1 = d * [W * (\cos^2\alpha) * (\tan\phi) + (c * b)]$

$R_2 = 2 * \Delta F$

$\Delta F = A * [1/3 * \gamma * H * K_o * (\tan\phi) + c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	Driving Force Wedge + Surcharge per lineal foot of Slot Width	Resisting Force Failure Wedge per lineal foot of Slot Width	Resisting Force Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
65	2 3	6	670 3	639 8	380 1	868 9	4 1
66	2.2	6	640 0	609 4	355 7	829 6	4 1
67	2 1	5	610 2	579 1	332 3	790 9	4 0
68	2 0	5	580 8	549 1	309 9	752 8	4 0
69	19	5	5518	519 2	288 3	715 3	4 0
70	18	5	523.2	489 5	267 6	678 2	3 9
71	17	4	495 0	460 2	247 8	6416	3 9
72	16	4	467 1	4312	228 8	605 4	3 9
73	15	4	439 5	402 5	210 6	569 7	3 9
74	14	4	412 2	374 2	193 3	534 3	3 9
75	13	3	385 2	346 3	176 6	499 3	3 9
76	12	3	358 4	318 9	160 7	464 6	3 9
77	12	3	3319	2919	145 5	430 2	3 9
78	11	3	305 6	265 5	1310	396 1	3 9
79	10	2	279 4	239 6	117 1	362 2	4 0
80	0 9	2	253 5	214 4	103 9	328 6	4 0
81	0 8	2	227 7	189 7	912	295.1	4 0
82	0 7	2	202 0	165 7	79 1	2619	4.1
83	0 6	2	176 5	142 3	67 6	228 8	4 1
84	0 5	1	1511	119 7	56 6	195 8	4 2
85	0 4	1	125 8	97 7	46 1	163 0	4 3
86	0 3	1	100 5	76 6	36 1	130.3	4 4
87	0 3	1	75 3	56 2	26 5	97.7	4.5
88	0 2	0	50 2	36 6	17.3	65 1	4 6
89	0 1	0	25 1	17 9	8 5	32 5	4 7
90	0 0	0	0 0	0 0	0 0	0 0	4 8

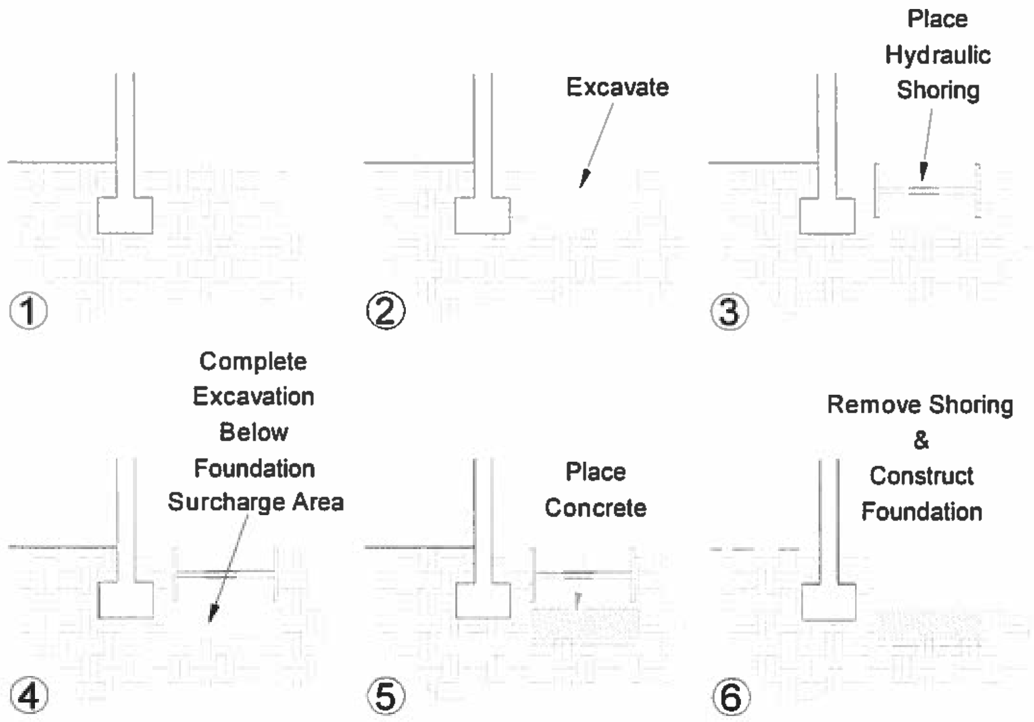
*Width of Slots to achieve a minimum of 1.5 Factor of Safety, with a Maximum Allowable Slot Width of 8-feet

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

d_{allow} = 3.9 feet

7.21 Shoring

7.21.1 As an alternative to slot cutting, hydraulic trench shoring may be implemented where excavations would remove a component of lateral support from adjacent foundations. The excavation may be conducted adjacent to the foundation but continuous excavation should not extend below the surcharge area of the existing foundation until the shoring is installed. The surcharge area may be defined by a 1:1 project down and away from the bottom of an existing foundation. Once shoring is installed, the excavation can be completed and the foundation can be constructed. Once the concrete backfill is placed to an elevation that is slightly above the bottom of the existing adjacent foundation, the shoring may be removed and the new foundation constructed. See illustration below.



7.21.2 It is recommended that an equivalent fluid pressure based on the table below, be utilized for design of hydraulic shoring.

HEIGHT OF SHORED EXCAVATION (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 5	30	50

7.21.3 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, the at-rest pressure should be considered for design purposes.

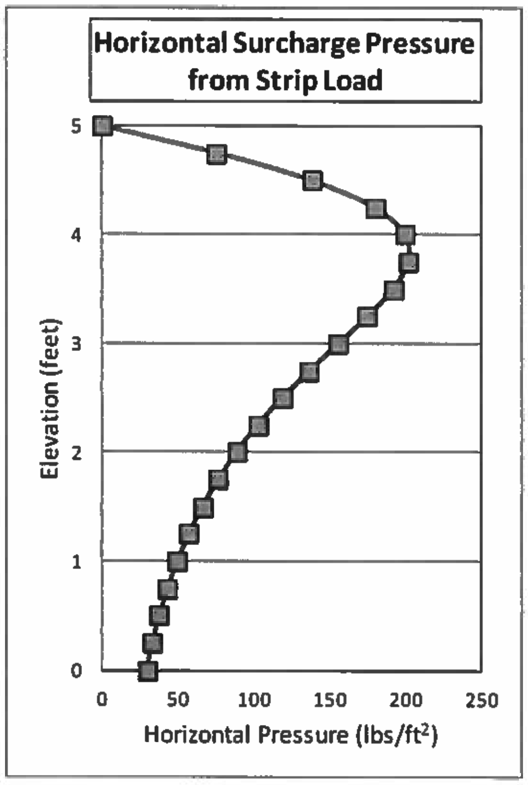
7.21.4 Additional active pressure should be added for a surcharge condition due to the adjacent structure as indicated in the calculation and diagram below. This calculation is based on several assumptions and should be verified once actual footing loads are available.

Description: Surcharge on Shoring From Existing Foundation

Horizontal Surcharge Pressure from Strip Load

Stip Load Ql=	1000 lbs/lf
Height of Cut H=	5 ft
Distance Away X1=	0 ft
m =	0

Elevation (feet)	n-value	Horizontal Pressure (lbs/ft^2)
5	0	0.00
4.75	0.05	75.74
4.5	0.1	138.41
4.25	0.15	180.15
4	0.2	200.00
3.75	0.25	201.99
3.5	0.3	192.00
3.25	0.35	175.42
3	0.4	156.25
2.75	0.45	136.98
2.5	0.5	118.98
2.25	0.55	102.85
2	0.6	88.76
1.75	0.65	76.63
1.5	0.7	66.27
1.25	0.75	57.47
1	0.8	50.00
0.75	0.85	43.66
0.5	0.9	38.26
0.25	0.95	33.66
0	1	29.73



Maximum Pressure = 201.99 lbs/ft^2
Total Load per Lineal Foot of Wall = 537.08 lbs/ft

7.22 Stormwater Infiltration

7.22.1 During the December 19, 2011 site exploration, borings B4 and B8 were utilized to perform percolation testing. The borings were advanced to the depths listed in the table below. Slotted casing was placed in each boring, and the annular space between the casing and excavation was filled with filter pack. The borings were then filled with water to pre-saturate the soils. On December 20, 2011, the casing was refilled with water, maintained at a depth of at least 1 foot above the excavation bottom for at least 30 minutes, and then percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the average infiltration rate (adjusted percolation rate) per boring for the earth materials encountered is listed in the following table.

Boring	Infiltration Depth (ft.)	Predominate USCS Soil Classification	Average Infiltration Rate (in / hour)
B4	10-15	Sand (SP)	2.9
B8	10-15	Silty Sand (SM) / Sand (SP)	1.2

7.22.2 Based on the results of the subsequent laboratory testing, the upper alluvial soils are subject to excessive settlement when saturated. Therefore, it is recommended that infiltration of storm water occur below a depth of 15 feet to minimize saturation of the soils supporting the proposed structures.

7.22.3 Provided infiltration occurs below a depth of 15 feet, resulting settlements from stormwater infiltration are anticipated to be less than ¼ inch at the ground surface, if any, and are not expected to affect existing or proposed structures or improvements. In addition, it is our opinion that the introduction of stormwater at these depths will not create a perched groundwater condition, and will not increase the potential for liquefaction.

7.22.4 Stormwater infiltration should be kept a minimum of 10 feet horizontally from adjacent foundations. In addition, where adjacent to any subterranean retaining walls, such as the proposed swimming pool, the discharge of stormwater should occur at a depth such that the retained soils do not become saturated. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.22.5 If the stormwater infiltration systems will be located in close proximity to a building pad, it is recommended that the stormwater infiltration system be installed during the mass grading of the site and prior to construction of any nearby building foundations. If installed after building foundation construction, the excavation required for installation of the stormwater infiltration system could remove a component of lateral support from the foundations and therefore would require shoring.

7.22.6 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation side walls and the infiltration system with two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.22.7 The design drawings and installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.23 Surface Drainage

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

7.23.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the engineered fill providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

7.23.3 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.

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

7.24 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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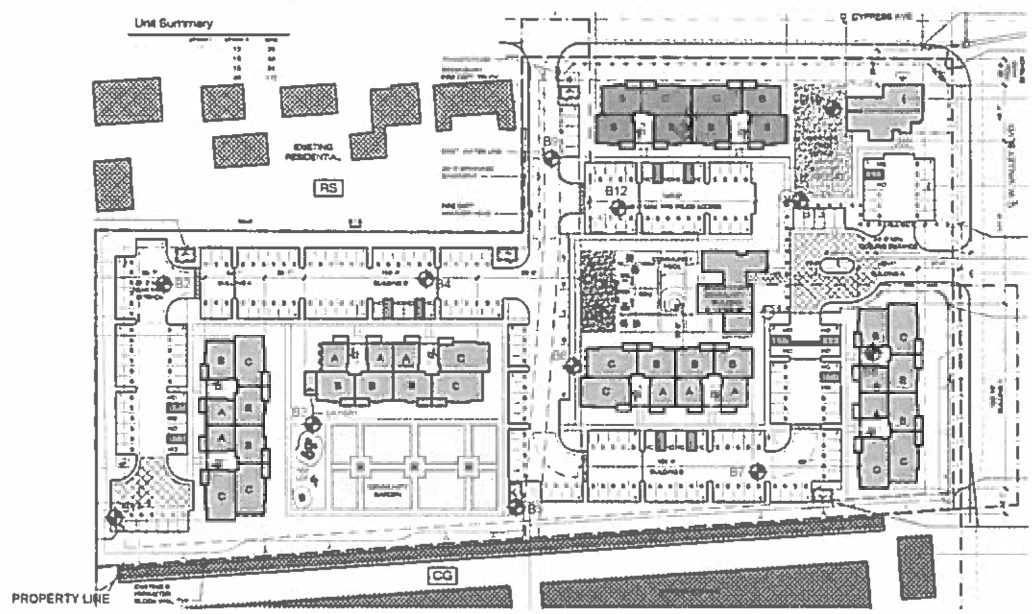
CHL		8000

VICINITY MAP LAS TERRAZAS AT COLTON AMCAL LAS TERRAZAS FUND, L.P. 275 & 291 CYPRESS AVENUE UNINCORPORATED SAN BERNARDINO COUNTY, CA		
SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. 1

PLAN BY: Withee Malcolm Architects, LLP



LEGEND

B13 Approximate Location of Boring

Approximate Location of Proposed 2-Story Structure

Approximate Location of Proposed 3-Story Structure

Approximate Location of Offsite Structures

0 80' 160'

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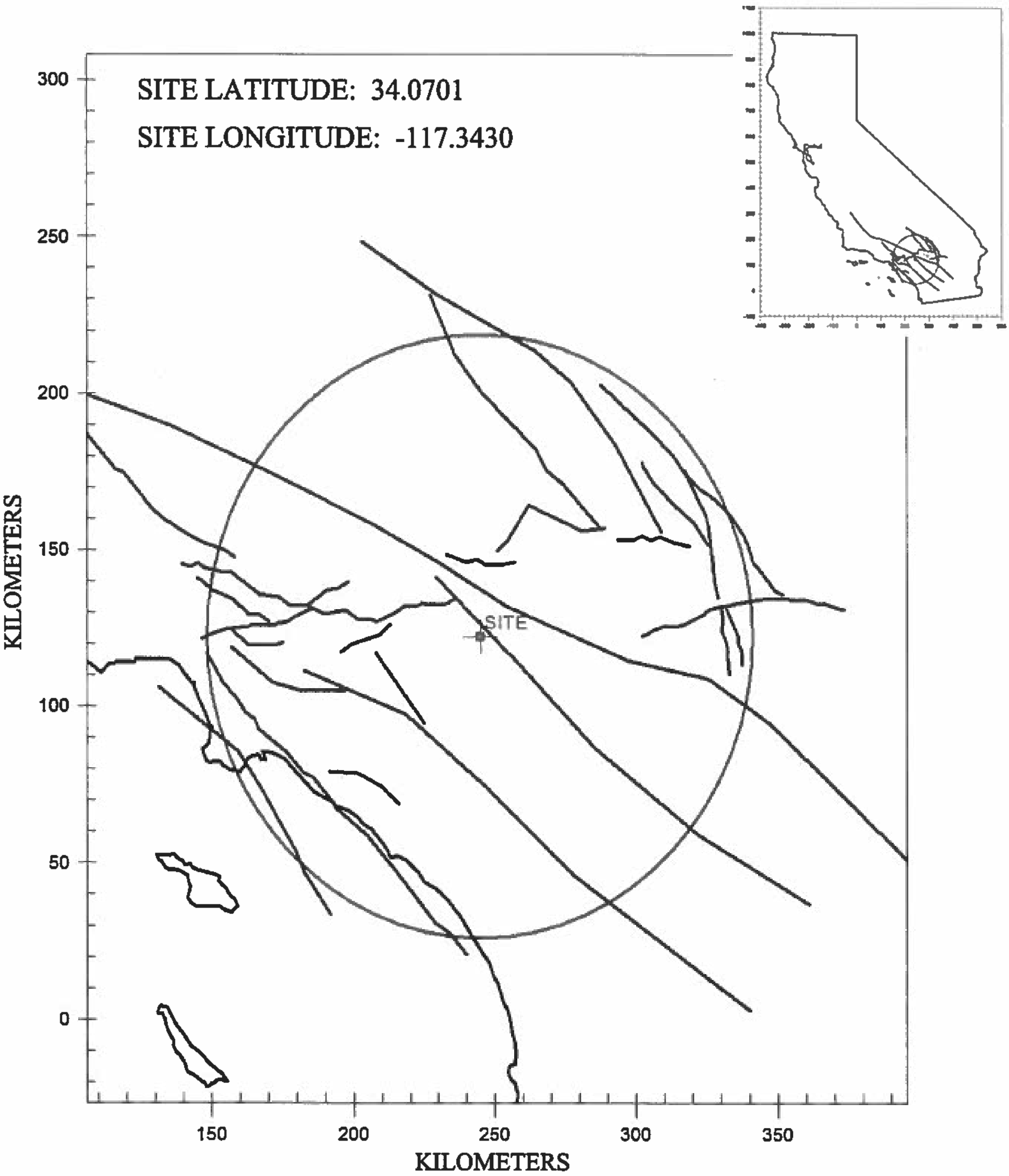
RP		8000

SITE PLAN
LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8698-06-01	FIG 2



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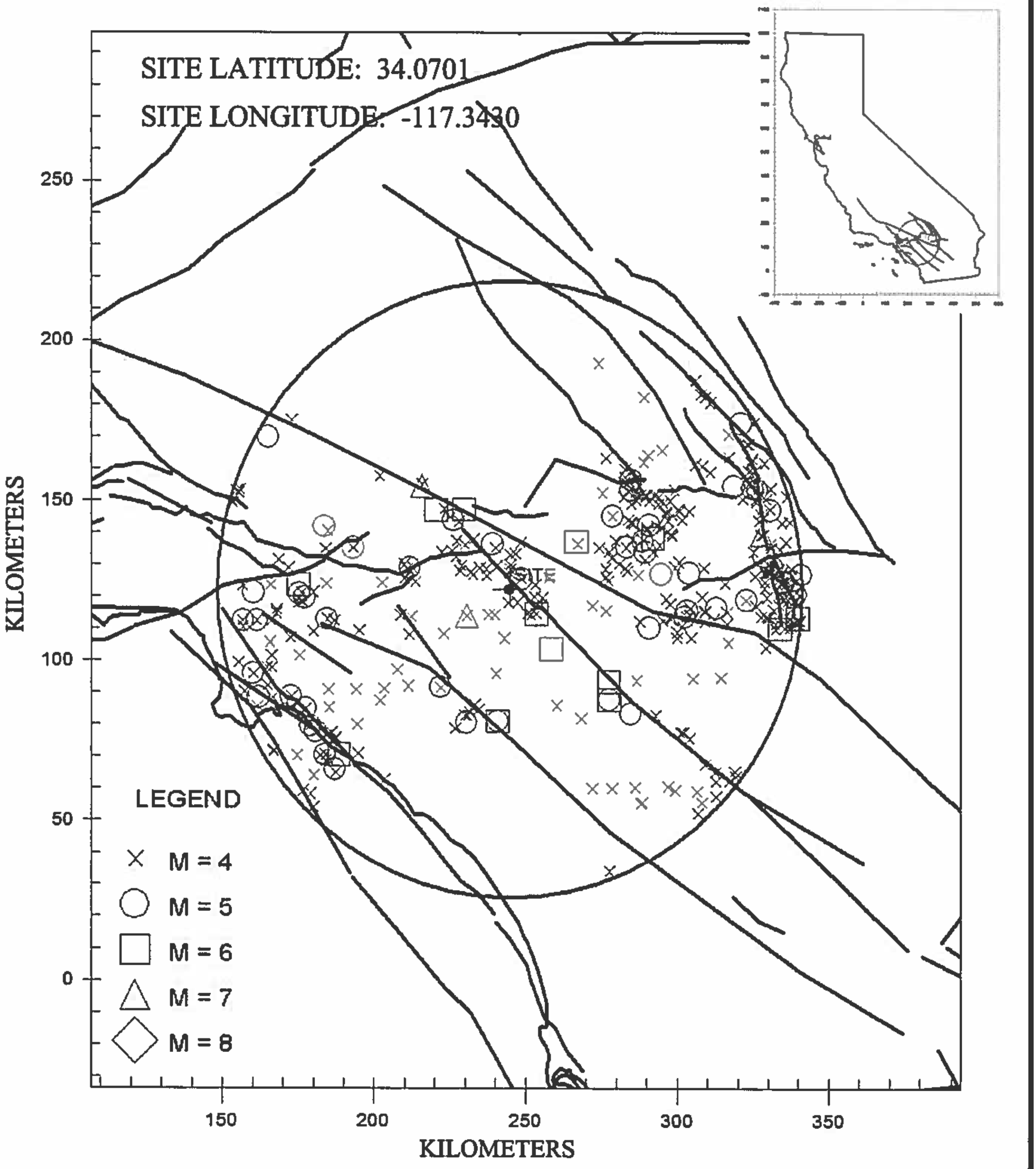
REGIONAL FAULT MAP

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. 3



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REGIONAL SEISMICITY MAP

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
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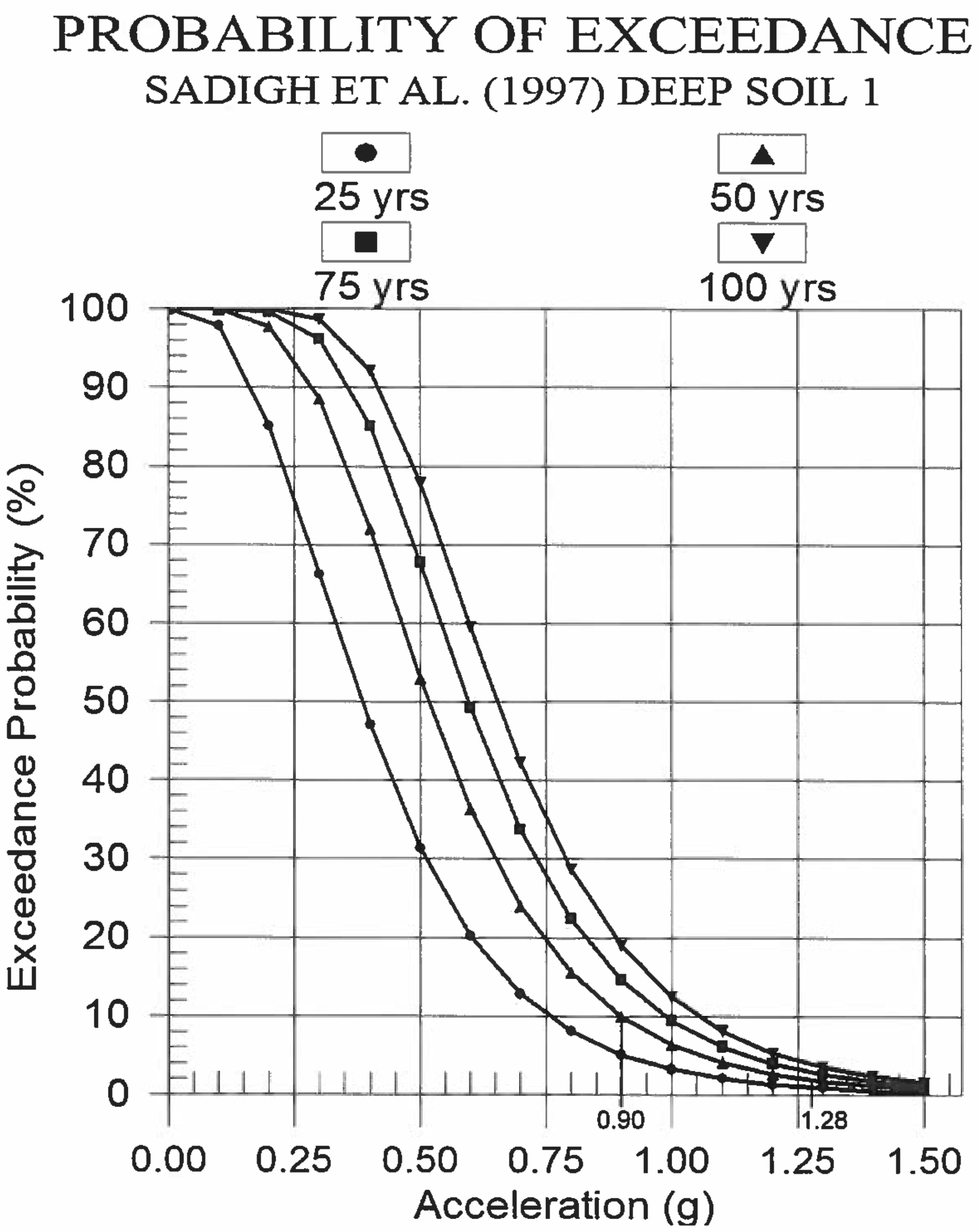
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. 4



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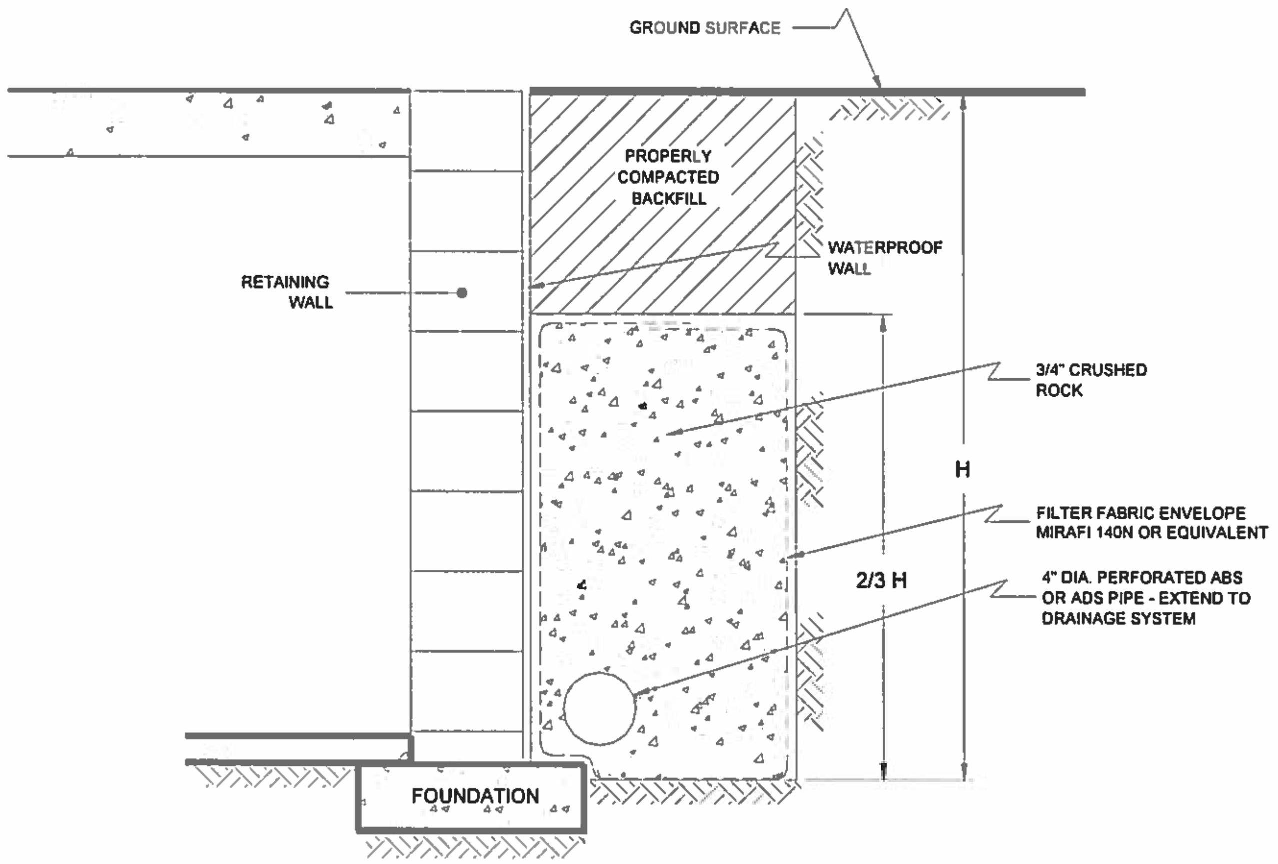
PROBABILITY OF EXCEEDANCE		
LAS TERRAZAS AT COLTON AMCAL LAS TERRAZAS FUND, L.P. 275 & 291 CYPRESS AVENUE UNINCORPORATED SAN BERNARDINO COUNTY, CA		
SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. 5



NO SCALE

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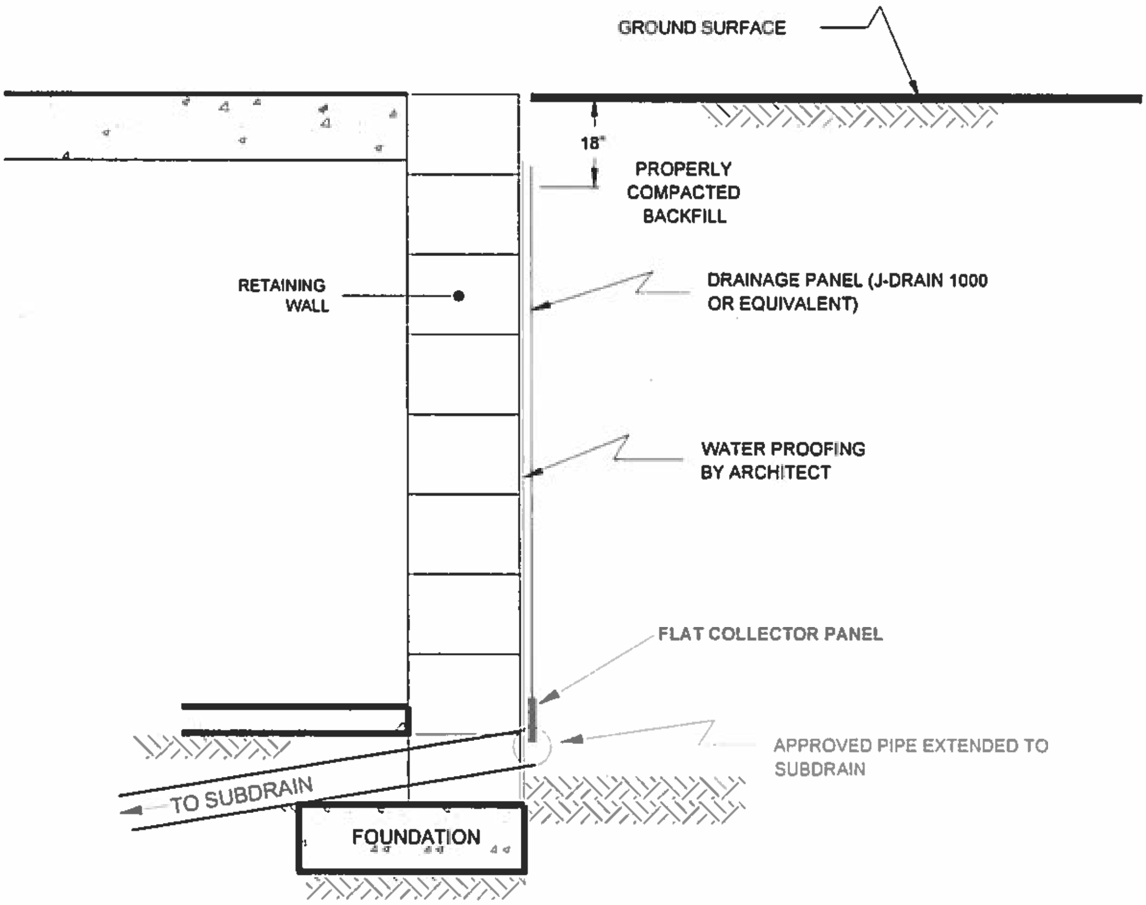
RETAINING WALL DRAIN DETAIL

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. 6



NO SCALE

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NDB		8000	SEPTEMBER 2014	PROJECT NO. A8898-06-01	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.
SAN JACINTO-SAN BERNARDINO	1.7	(2.8)	6.7	0.731	XI
SAN JACINTO-SAN JACINTO VALLEY	7.3	(11.7)	6.9	0.431	X
SAN ANDREAS - San Bernardino M-1	8.1	(13.1)	7.5	0.484	X
SAN ANDREAS - SB-Coach. M-2b	8.1	(13.1)	7.7	0.512	X
SAN ANDREAS - SB-Coach. M-1b-2	8.1	(13.1)	7.7	0.512	X
SAN ANDREAS - Whole M-1a	8.1	(13.1)	8.0	0.552	X
CUCAMONGA	9.5	(15.3)	6.9	0.466	X
CLEGHORN	14.4	(23.2)	6.5	0.225	IX
NORTH FRONTAL FAULT ZONE (West)	15.9	(25.6)	7.2	0.354	IX
SAN ANDREAS - 1857 Rupture M-2a	17.6	(28.3)	7.8	0.336	IX
SAN ANDREAS - Cho-Moj M-1b-1	17.6	(28.3)	7.8	0.336	IX
SAN ANDREAS - Mojave M-1c-3	17.6	(28.3)	7.4	0.280	IX
SAN JOSE	20.1	(32.3)	6.4	0.192	VIII
CHINO-CENTRAL AVE. (Elsinore)	20.3	(32.6)	6.7	0.223	IX
WHITTIER	21.9	(35.2)	6.8	0.168	VIII
ELSINORE (GLEN IVY)	22.3	(35.9)	6.8	0.165	VIII
SIERRA MADRE	22.8	(36.7)	7.2	0.255	IX
ELSINORE (TEMECULA)	29.5	(47.5)	6.8	0.120	VII
PUENTE HILLS BLIND THRUST	30.6	(49.2)	7.1	0.176	VIII
CLAMSHELL-SAWPIT	31.1	(50.0)	6.5	0.122	VII
NORTH FRONTAL FAULT ZONE (East)	33.2	(53.4)	6.7	0.126	VIII
SAN JACINTO-ANZA	33.4	(53.7)	7.2	0.132	VIII
HELENDALE - S. LOCKHARDT	34.1	(54.8)	7.3	0.138	VIII
PINTO MOUNTAIN	35.7	(57.4)	7.2	0.123	VII
SAN JOAQUIN HILLS	36.9	(59.4)	6.6	0.104	VII
RAYMOND	37.6	(60.5)	6.5	0.096	VII
UPPER ELYSIAN PARK BLIND THRUST	43.4	(69.8)	6.4	0.073	VII
VERDUGO	44.1	(70.9)	6.9	0.099	VII
LENWOOD-LOCKHART-OLD WOMAN SPRGS	45.0	(72.4)	7.5	0.115	VII
NEWPORT-INGLEWOOD (L.A.Basin)	46.0	(74.1)	7.1	0.084	VII
NEWPORT-INGLEWOOD (Offshore)	46.4	(74.7)	7.1	0.083	VII
JOHNSON VALLEY (Northern)	49.0	(78.9)	6.7	0.059	VI
HOLLYWOOD	50.9	(81.9)	6.4	0.059	VI
SAN ANDREAS - Coachella M-1c-5	51.0	(82.1)	7.2	0.079	VII
ELSINORE (JULIAN)	51.4	(82.8)	7.1	0.072	VII
LANDERS	52.6	(84.7)	7.3	0.082	VII
BURNT MTN.	53.3	(85.7)	6.5	0.046	VI
EUREKA PEAK	54.4	(87.5)	6.4	0.042	VI
EMERSON So. - COPPER MTN.	55.9	(89.9)	7.0	0.060	VI
SAN GABRIEL	56.3	(90.6)	7.2	0.070	VI
SIERRA MADRE (San Fernando)	56.3	(90.6)	6.7	0.063	VI
PALOS VERDES	56.7	(91.2)	7.3	0.074	VII

42 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE SAN JACINTO-SAN BERNARDINO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.7 MILES (2.8 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7306 g

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was initially explored on December 19, 2011 by excavating nine 7-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 5½ and 20½ feet below the existing ground surface. Percolation testing for the design of a stormwater infiltration system was performed in two of the borings. 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 a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch brass sampler rings to facilitate removal and testing. Bulk samples were also obtained.

A supplemental site exploration was performed on January 28, 2013 by excavating four 4-inch diameter borings using manual hand auger equipment. The borings were advanced to depths between 4½ and 10½ feet below the ground surface. 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 a slide hammer. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch brass sampler rings to facilitate removal and testing.

The soil conditions encountered in the borings 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-13. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are depicted on the Site Plan, Figure 2.

PROJECT NO. A8898-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, light brown, fine-grained with trace medium-grained			
2					**OLDER ALLUVIUM**			
	B1@2.5'				Silty Sand, medium dense, slightly moist, light yellowish brown, fine-grained with trace medium-grained	45	114.1	3.1
4								
	B1@5'					44	106.2	2.4
6								
8	B1@7.5'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, light yellowish brown, fine-grained with trace medium- to coarse-grained	23	113.9	1.5
10	B1@10'			SP	Sand, poorly graded, dense, dry, yellowish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	55	113.3	1.9
12					-Very dense, pale brown to light yellowish brown			
	B1@12.5'					50 (5")	121.3	1.8
14								
	B1@15'				-Dense, pale brown	85	138.0	2.5
16								
18								
20	B1@20'				-Very dense, fine-grained with some medium- to coarse-grained, trace fine-gravel	50 (3")	108.4	2.0

End at 20.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A1,
Log of Boring 1, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... 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. A8898-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL**			
	B2@1'				Silty Sand, medium dense, dry, yellowish brown, fine-grained with trace medium-grained	21	98.6	4.1
2					**OLDER ALLUVIUM**			
	B2@3'				Silty Sand, medium dense, dry, light yellowish brown, fine-grained	35	95.6	3.4
4								
	B2@5'			SM		30	103.8	3.6
6								
	B2@7'				-Fine-grained with trace coarse-grained	31	120.6	3.3
8								
	B2@9'				Sand, poorly graded, medium dense, dry, yellowish brown, fine- to medium-grained with some coarse-grained, trace fine-gravel	38	105.9	1.9
10								
12				SP				
	B2@13'				-Dense, trace fine- to coarse-gravel	81	112.9	1.4
14								
	B2@15'					62	131.0	1.5

End at 15.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

A8898-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

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. A8898-06-01

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, dark brown, fine-grained with trace medium-grained			
2	B3@2'					26	97.0	4.2
4	B3@4'			SM	**OLDER ALLUVIUM** Silty Sand, medium dense, dry, light yellowish brown, fine-grained	25	89.6	2.0
6	B3@6'				-Fine-grained with trace medium-grained	31	101.7	3.4
8	B3@8'			SP	Sand, poorly graded, medium dense, dry, brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel	24	116.1	2.6
10	B3@10'					21	112.4	2.9
12	B3@12'				-Dense, dark yellowish brown, fine-grained with trace medium- to coarse-grained	68	125.6	1.7
14								
16								
18	B3@17'				-Yellowish brown to pale brown, trace fine- to coarse-gravel	54	126.0	1.2
20	B3@20'			SM	Silty Sand, dense, dry, yellowish brown to pale brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel	74	129.9	2.6

End at 20.5 feet.
Artificial fill to 3.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A3,
Log of Boring 3, Page 1 of 1

A8898-06-01 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. A8898-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) -- DATE COMPLETED 12/19/11 EQUIPMENT HOLLOW STEM AUGER BY: CA MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 4.5	B4@2.5'			SM	**OLDER ALLUVIUM** Silty Sand, loose, slightly moist, light yellowish brown, fine-grained	11	93.1	4.7
4.5 – 7	B4@5'			ML	Silt with Sand, firm, slightly moist, dark yellowish brown, fine-grained, low plasticity	13	118.6	8.2
7 – 9.5	B4@7.5'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, reddish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	36	133.4	2.0
9.5 – 15	B4@10'			SP	Sand, poorly graded, medium dense, dry, yellowish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	35	120.7	3.0

End at 15 feet.
No artificial fill encountered.
No groundwater encountered.
Percolation testing conducted on 12/20/11.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A4,
Log of Boring 4, Page 1 of 1

A8898-06-01 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. A8898-06-01

BORING 5

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Silt, very soft, wet, brown, fine-grained			
2	B5@2.5'					3	107.0	17.2
4								
	B5@5'				**OLDER ALLUVIUM** Sandy Silt, very soft, wet, brown, fine-grained	5	107.0	15.0
6								
8	B5@7.5'			ML		5	113.7	13.5
10	B5@10'			SP-SM	Sand with Silt, poorly graded, medium dense, wet, brown, fine-grained with trace medium-grained	20	107.8	7.4

End at 10.5 feet.
Artificial fill to 4.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A5,
Log of Boring 5, Page 1 of 1

A8898-06-01 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. A8898-06-01

BORING 6

ELEV. (MSL.) - -   DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER   BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**OLDER ALLUVIUM**			
	B6@1'			ML	Sandy Silt, stiff, dry, light yellowish brown, fine-grained with trace medium-grained	22	99.1	3.0
	B6@3'				-Firm	17	115.0	3.3
	B6@5'					16	107.2	4.4
	B6@7'				-Stiff, reddish brown	22	91.1	6.6
	B6@9'			SP-SM	Sand with Silt, poorly graded, loose, dry, reddish brown, fine-grained with trace medium-grained	17	111.6	3.1
12	B6@12'			SP	Sand, poorly graded, dense, slightly moist, yellowish brown, fine- to coarse-grained, trace fine- to coarse-gravel	68	138.0	2.3
	B6@15'				-Some fine- to coarse-gravel	80	129.4	2.4

End at 15.5 feet.
No artificial fill encountered.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A6,
Log of Boring 6, Page 1 of 1

A8898-06-01 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. A8898-06-01

BORING 7

ELEV. (MSL.) -- DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL**			
	B7@1'				Silty Sand, dense, dry, pale brown, fine- to medium-grained	59	94.9	3.5
2								
					OLDER ALLUVIUM			
	B7@3'				Silt with Sand, stiff, dry, yellowish brown, fine-grained	33	95.3	2.8
4								
	B7@5'				-Light brown	35	100.1	2.7
6								
	B7@7'				-Increase in sand content, yellowish brown	28	100.1	3.2
8				ML				
10	B7@10'					21	102.3	4.1
12								
14								
	B7@15'				Sand, poorly graded, medium dense, dry, olive brown, fine- to medium-grained with trace coarse-grained	48	125.6	1.9
16								
18				SP				
20	B7@20'				-Dense, fine- to coarse-grained, trace fine-gravel	84	127.7	2.6

End at 20.5 feet.
Artificial fill to 2.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A7,
Log of Boring 7, Page 1 of 1

A8898-06-01 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. A8898-06-01

BORING 8

ELEV. (MSL.) -- DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, dense, dry, light yellowish brown, fine- to medium-grained with trace coarse-grained			
2	B8@2'					56	109.3	2.0
4	B8@4'				**OLDER ALLUVIUM** Sandy Silt, stiff, dry, yellowish brown, fine-grained with trace medium-grained	24	100.2	2.9
6	B8@6'					33	98.9	2.9
8	B8@8'			ML		34	109.7	2.8
10	B8@10'					44	108.0	3.2
12	B8@12'			SM	Silty Sand, medium dense, dry, yellowish brown, fine-grained with trace medium-grained	22	116.4	2.4
14	B8@14'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, dark yellowish brown, fine-grained with trace fine-gravel	20	105.9	1.8

End at 15 feet.
Artificial fill to 4 feet.
No groundwater encountered.
Percolation testing conducted on 12/20/11.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A8,
Log of Boring 8, Page 1 of 1

A8898-06-01 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. A8898-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 9 ELEV. (MSL.) -- DATE COMPLETED 12/19/11 EQUIPMENT HOLLOW STEM AUGER BY: CA	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4	BULK 0-2'				**ARTIFICIAL FILL** Silty Sand, medium dense, dry, light brown, fine-grained with trace medium-grained			
	B9@5'			ML	**OLDER ALLUVIUM** Silt with Sand, hard, dry, light brown to yellowish brown, fine-grained with trace medium-grained	56	107.1	4.1
					End at 5.5 feet. Artificial fill to 4 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. *Penetration resistance for 140 pound hammer falling 30 inches.			

Figure A9,
Log of Boring 9, Page 1 of 1

A8898-06-01 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. A8898-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 10 ELEV. (MSL.) -- DATE COMPLETED 1/28/13 EQUIPMENT HAND AUGER BY: RG	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
2	B10@2'							
4					-Increase in silt content			
	B10@5'			SM				
6					-Increase in silt content			
					-Some coarse-grained sand, some fine-gravel			
8	B10@8'				-Decrease in silt content			
10	B10@10'							
					End at 10.5 feet. No artificial fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A10,
Log of Boring 10, Page 1 of 1

A8898-06-01 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. A8898-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 11 ELEV. (MSL.) -- DATE COMPLETED 1/28/13 EQUIPMENT HAND AUGER BY: RG	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
2	B11@2'							
					-Increase in silt content			
4	B11@4'							
6				SM				
	B11@7'							
8					-Decrease in silt content, some fine- to coarse-gravel			
10	B11@10'							
					End at 10.5 feet. No artificial fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A11,
Log of Boring 11, Page 1 of 1

A8898-06-01 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. A8898-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 12 ELEV. (MSL.) -- DATE COMPLETED 1/28/13 EQUIPMENT HAND AUGER BY: RG	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark brown, fine- to coarse-grained, trace fine-gravel			
2	B12@2'				**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
4					-Increase in silt content			
	B12@5'							
6				SM				
					-Decrease in silt content, some coarse-gravel, trace fine-gravel			
8	B12@8'							
10	B12@10'							
					End at 10.5 feet. Artificial fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A12,
Log of Boring 12, Page 1 of 1

A8898-06-01 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. A8898-06-01

BORING 13

ELEV. (MSL.) -- DATE COMPLETED 1/28/13

EQUIPMENT HAND AUGER BY: RG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel			
2	B13@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
4	B13@4'							
					End at 4.5 feet. Artificial fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A13,
Log of Boring 13, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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

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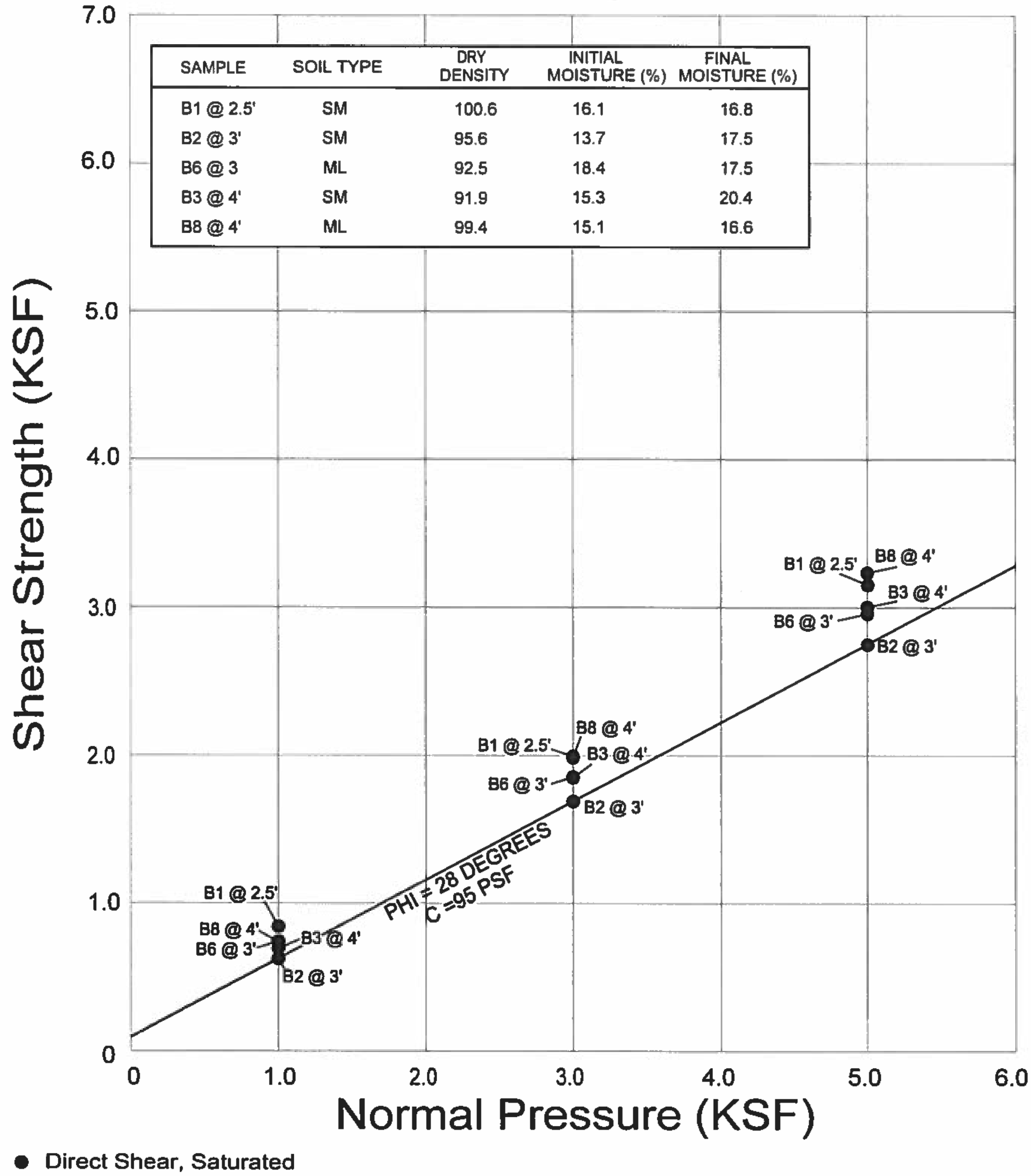
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, compaction characteristics, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B20. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



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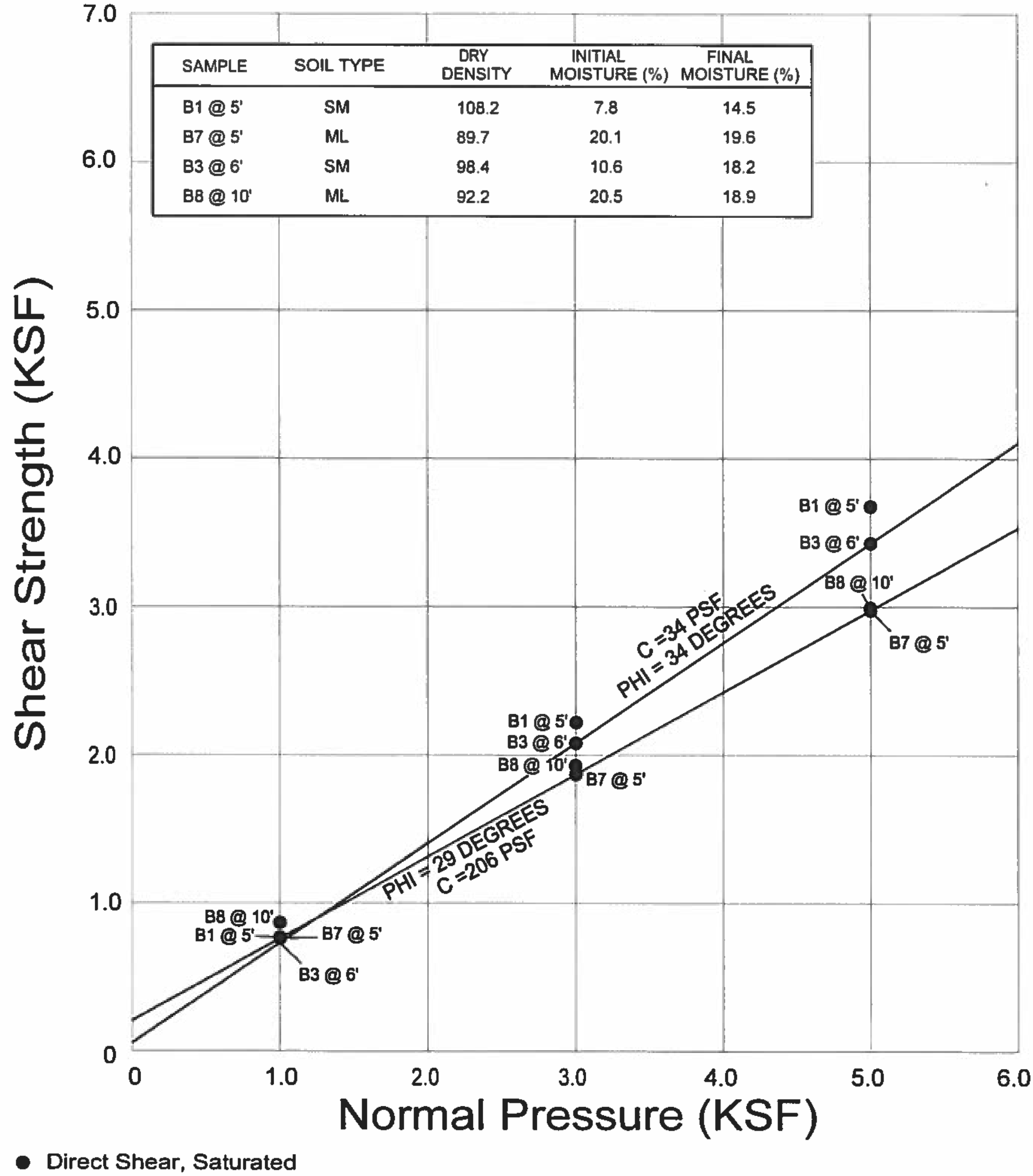
PHONE (818) 841-8388 - FAX (818) 841-1704

TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	SM	108.2	7.8	14.5
B7 @ 5'	ML	89.7	20.1	19.6
B3 @ 6'	SM	98.4	10.6	18.2
B8 @ 10'	ML	92.2	20.5	18.9

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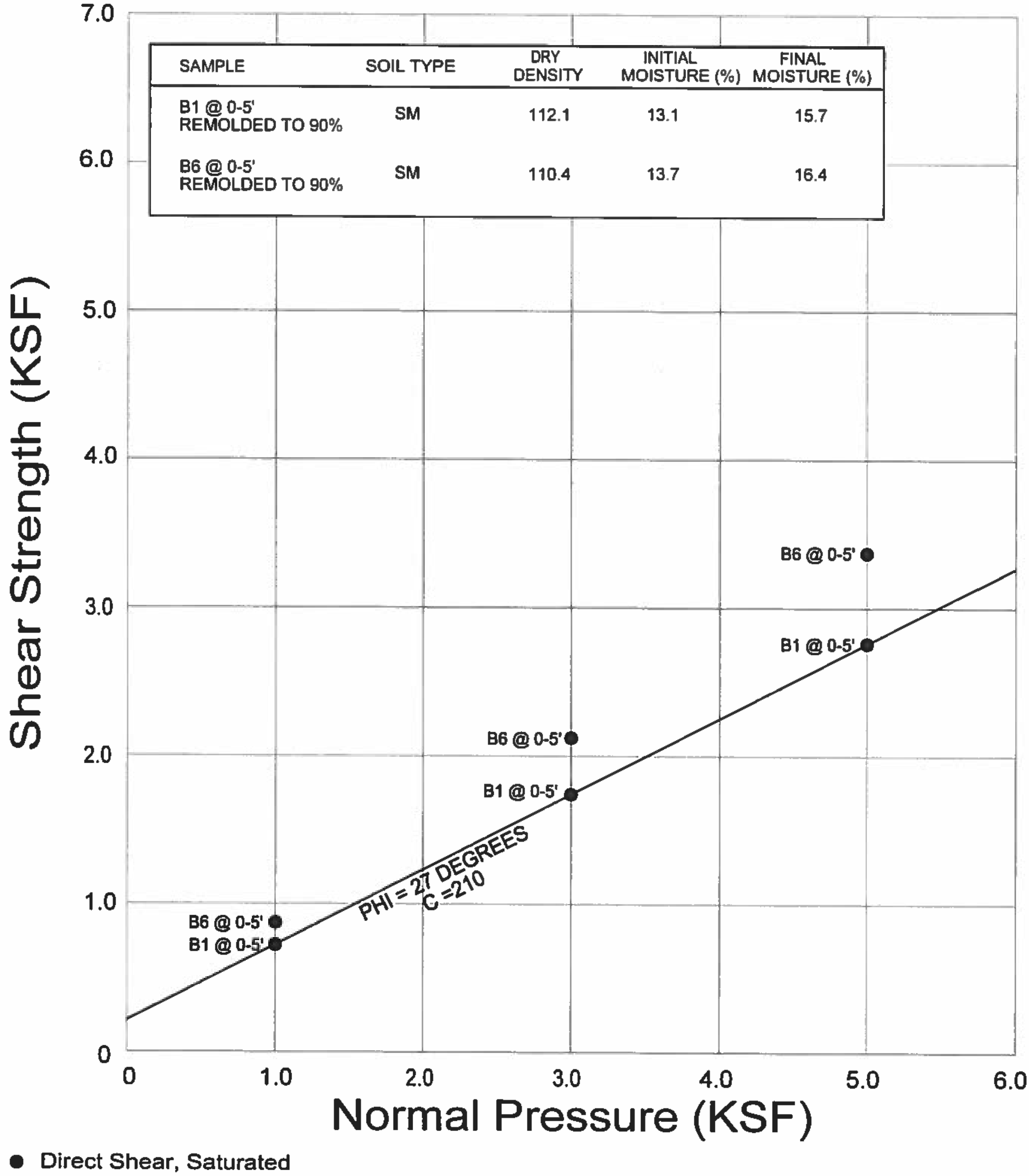
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 0-5' REMOLDED TO 90%	SM	112.1	13.1	15.7
B6 @ 0-5' REMOLDED TO 90%	SM	110.4	13.7	16.4

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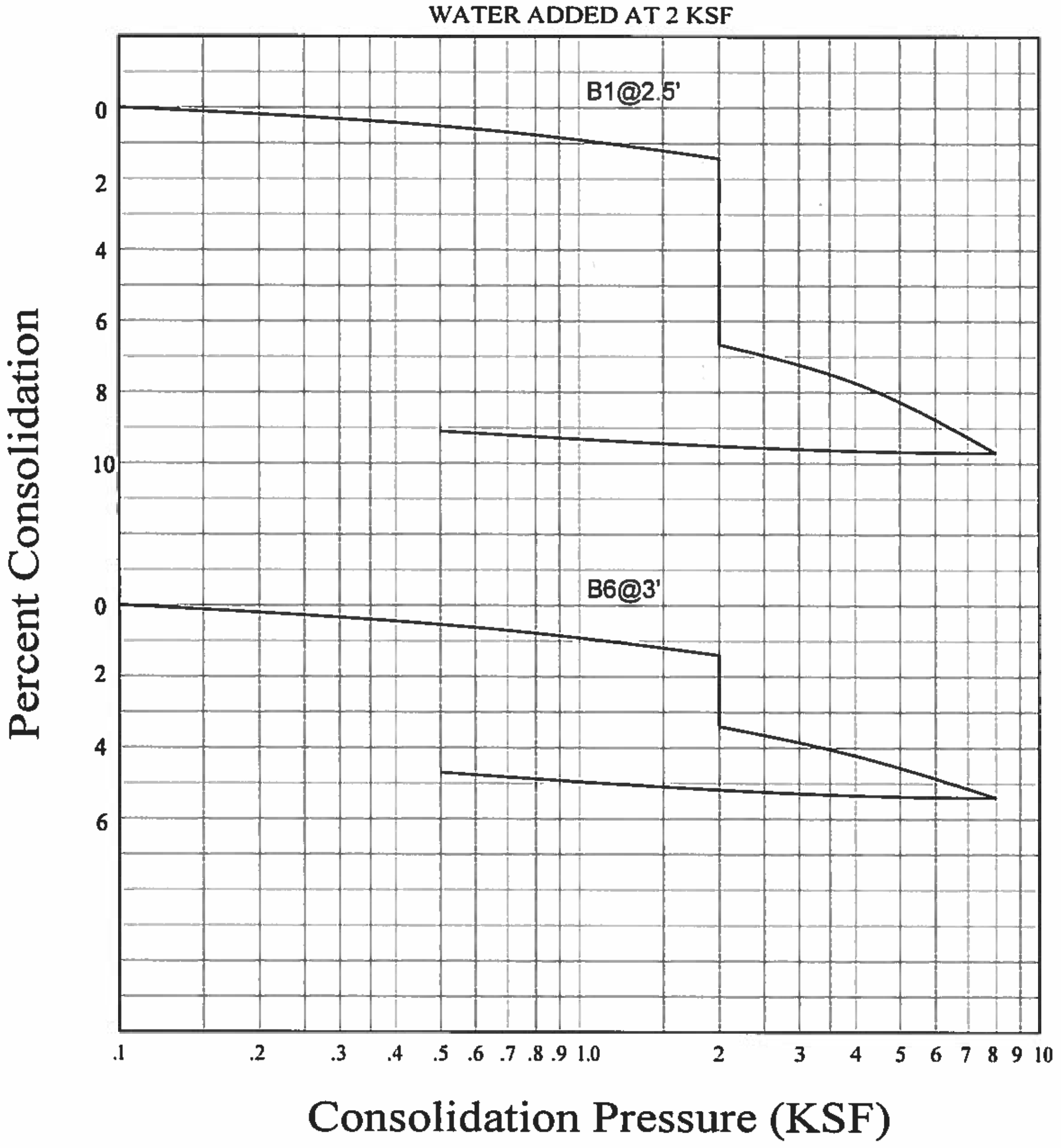
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B3



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PHONE (818) 841-8388 - FAX (818) 841-1704

TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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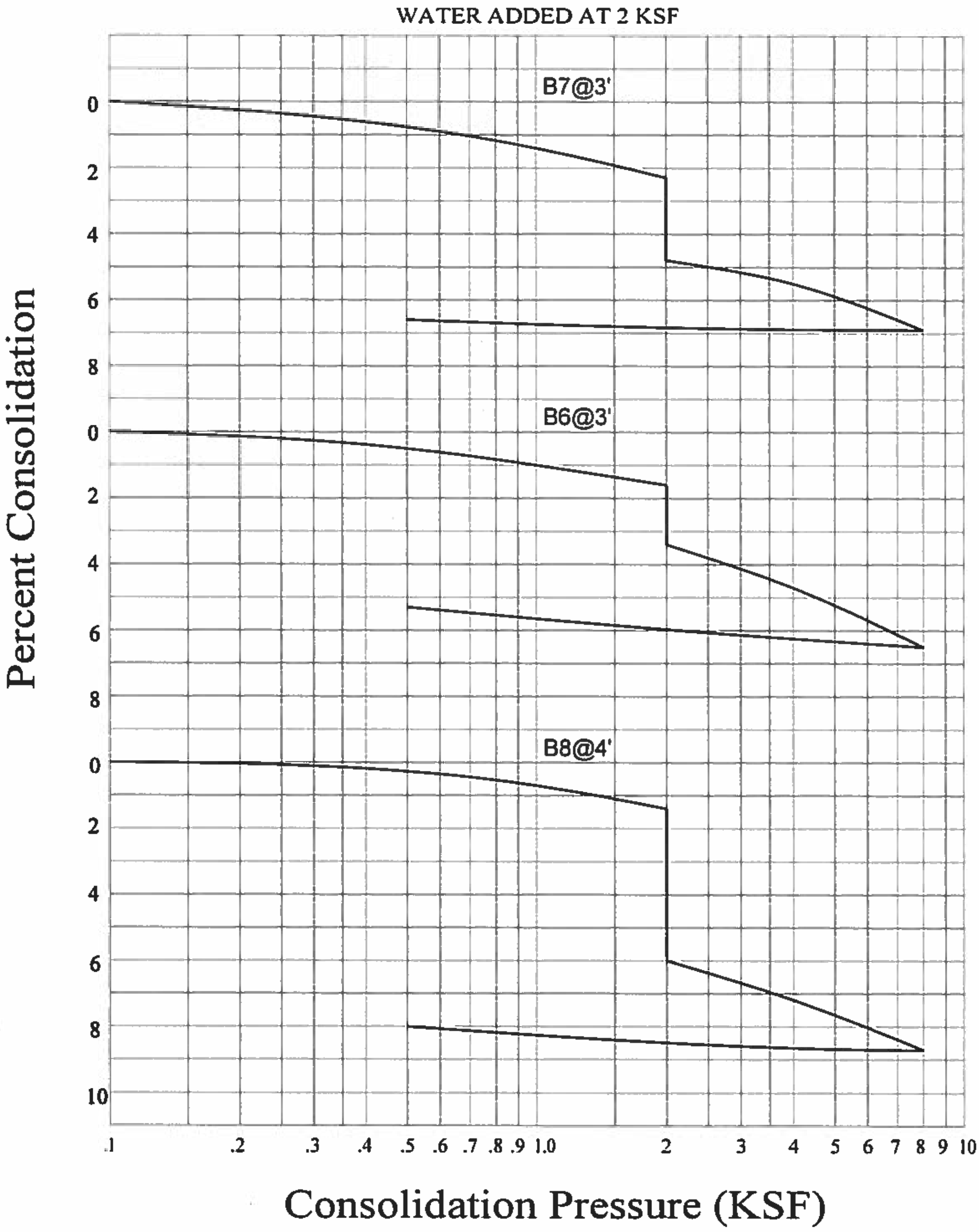
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B4



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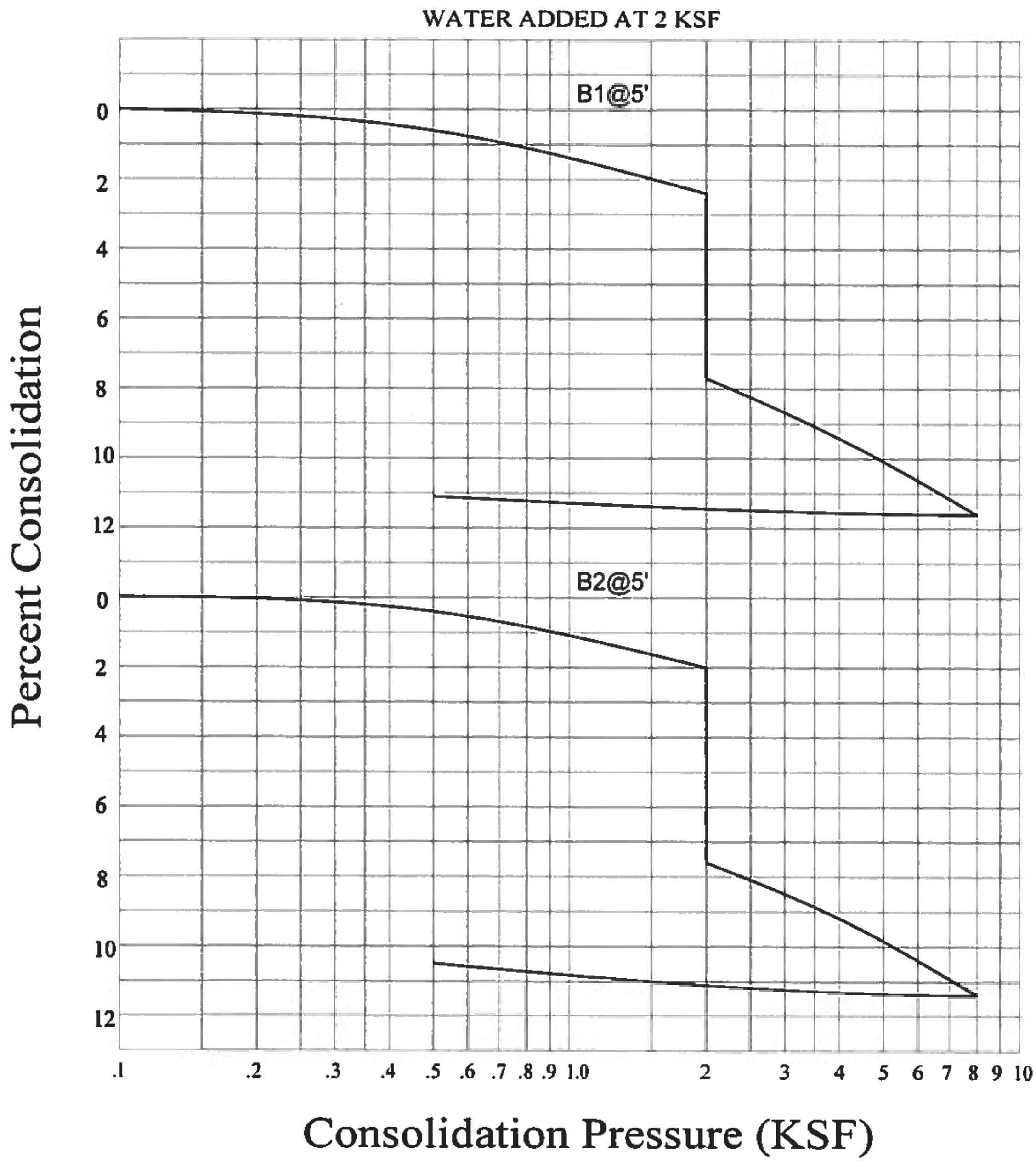
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B5



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TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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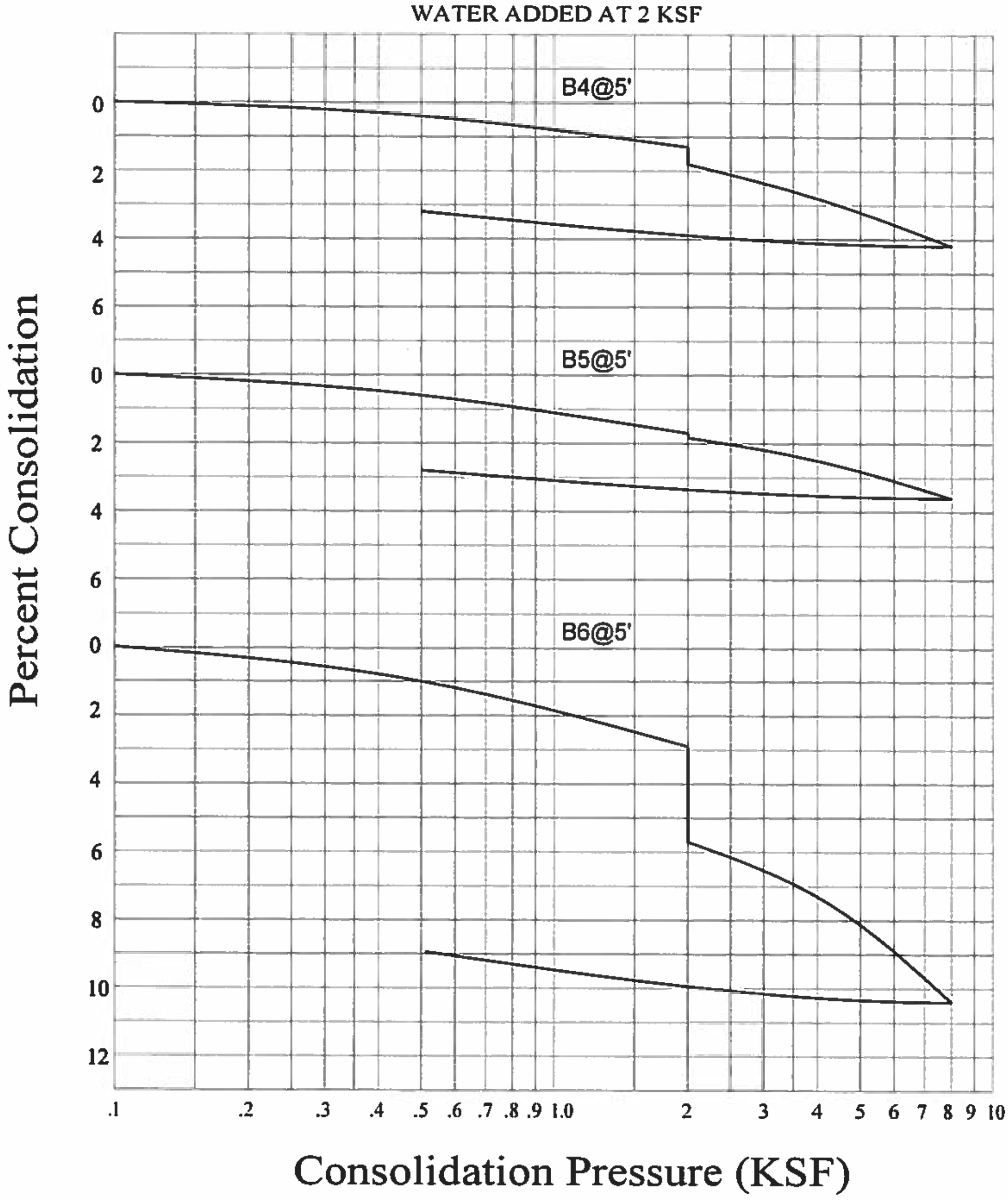
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B6



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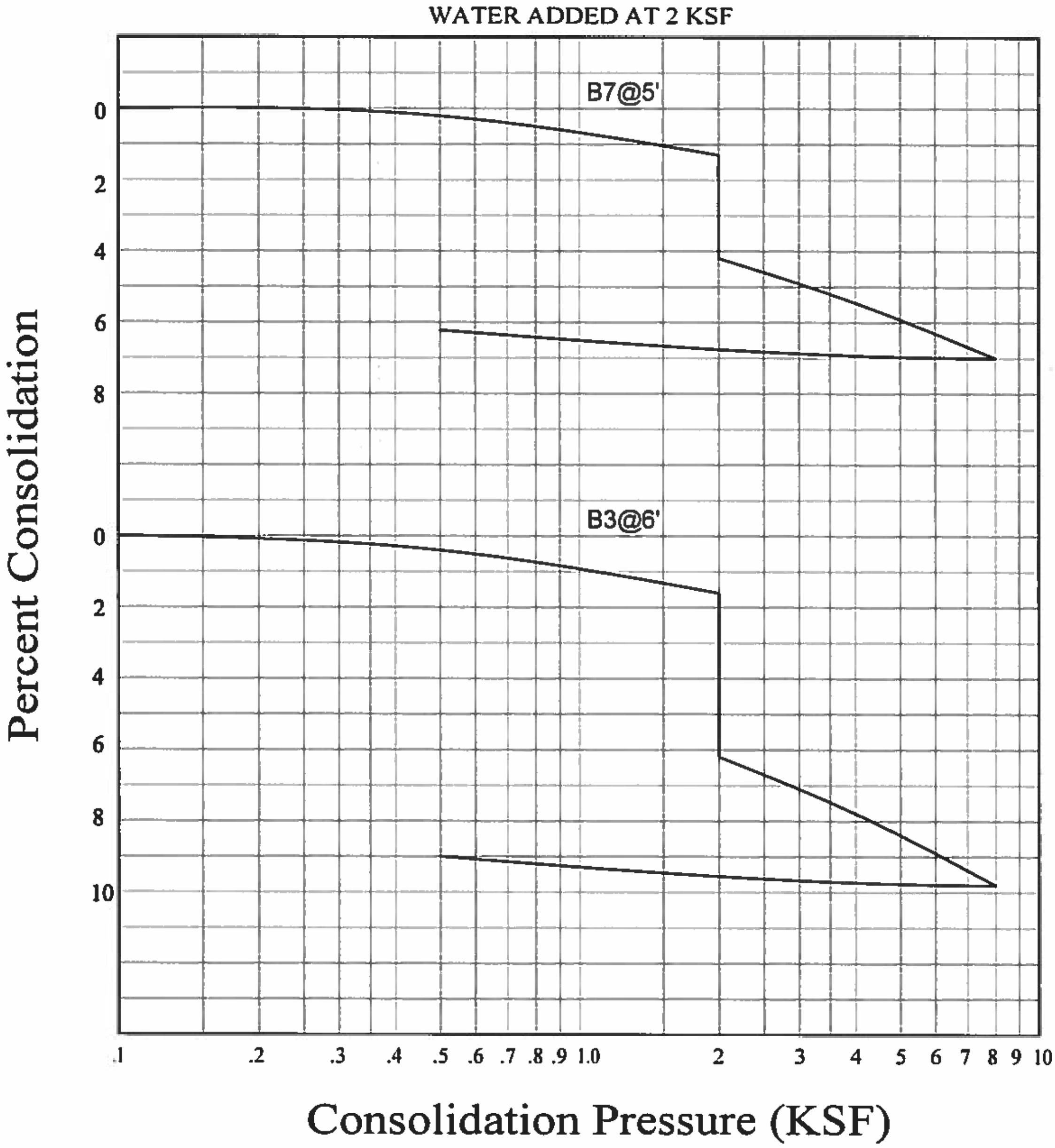
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B7



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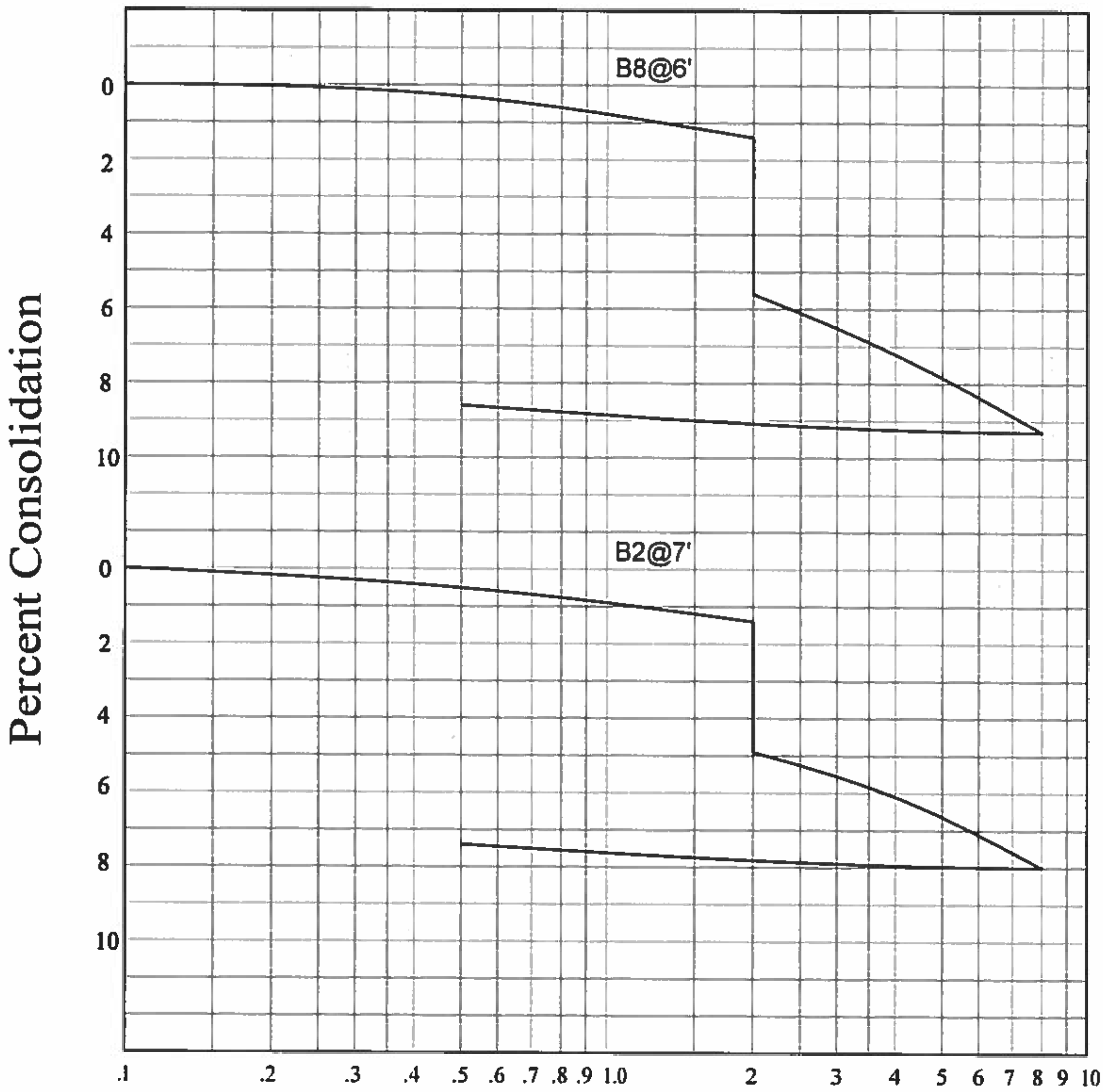
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B8

WATER ADDED AT 2 KSF



Consolidation Pressure (KSF)

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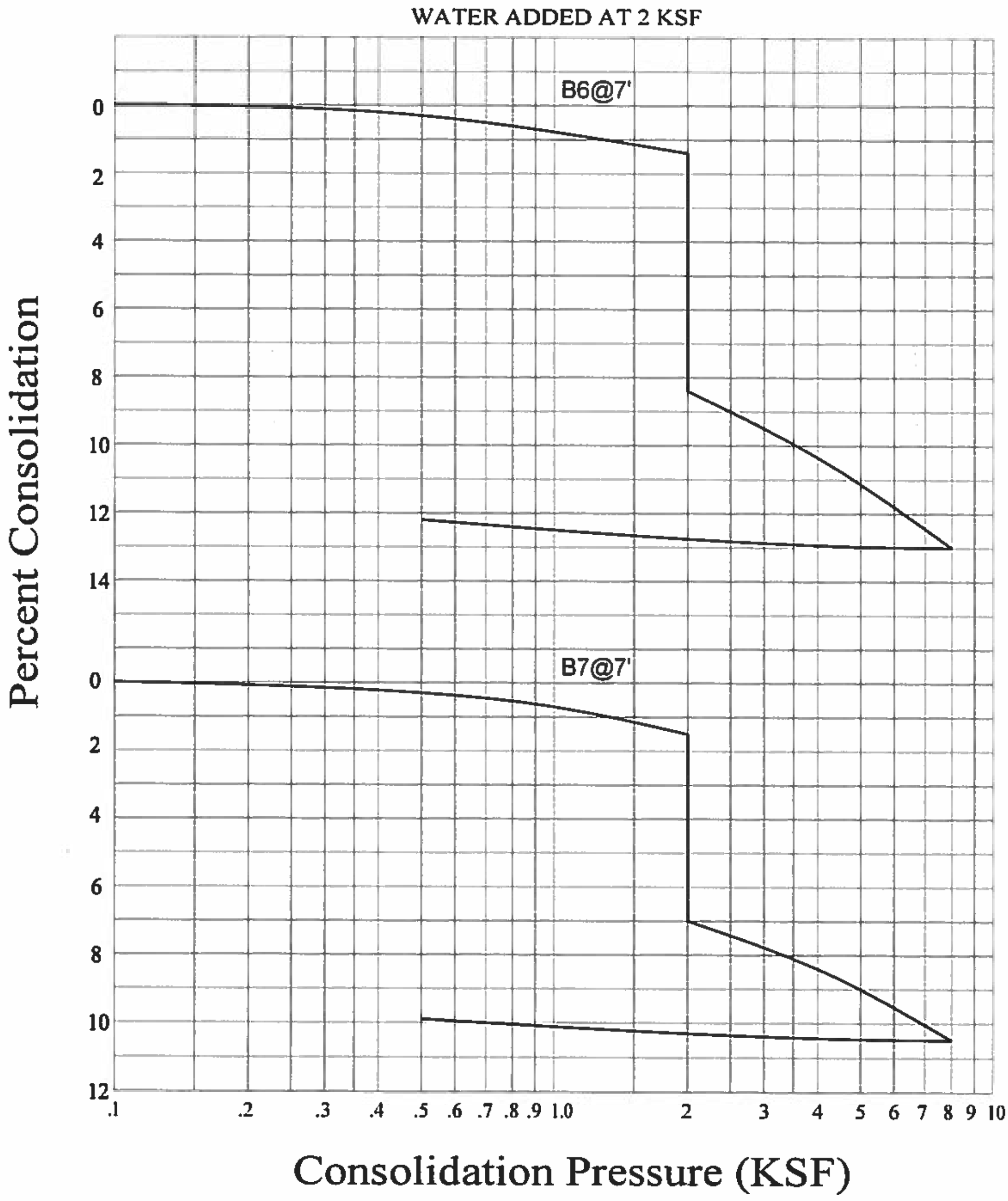
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B9



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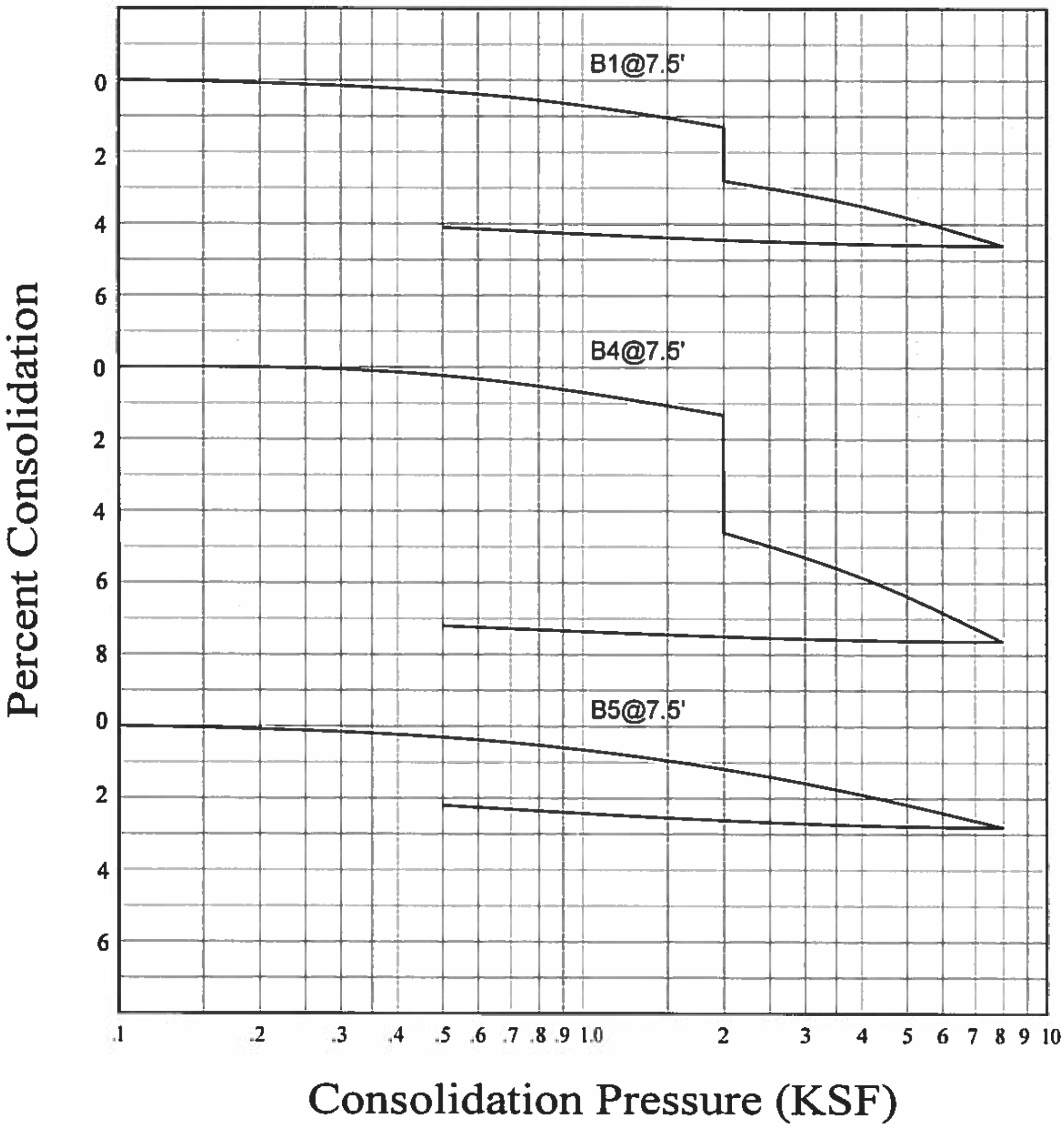
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B10

WATER ADDED AT 2 KSF



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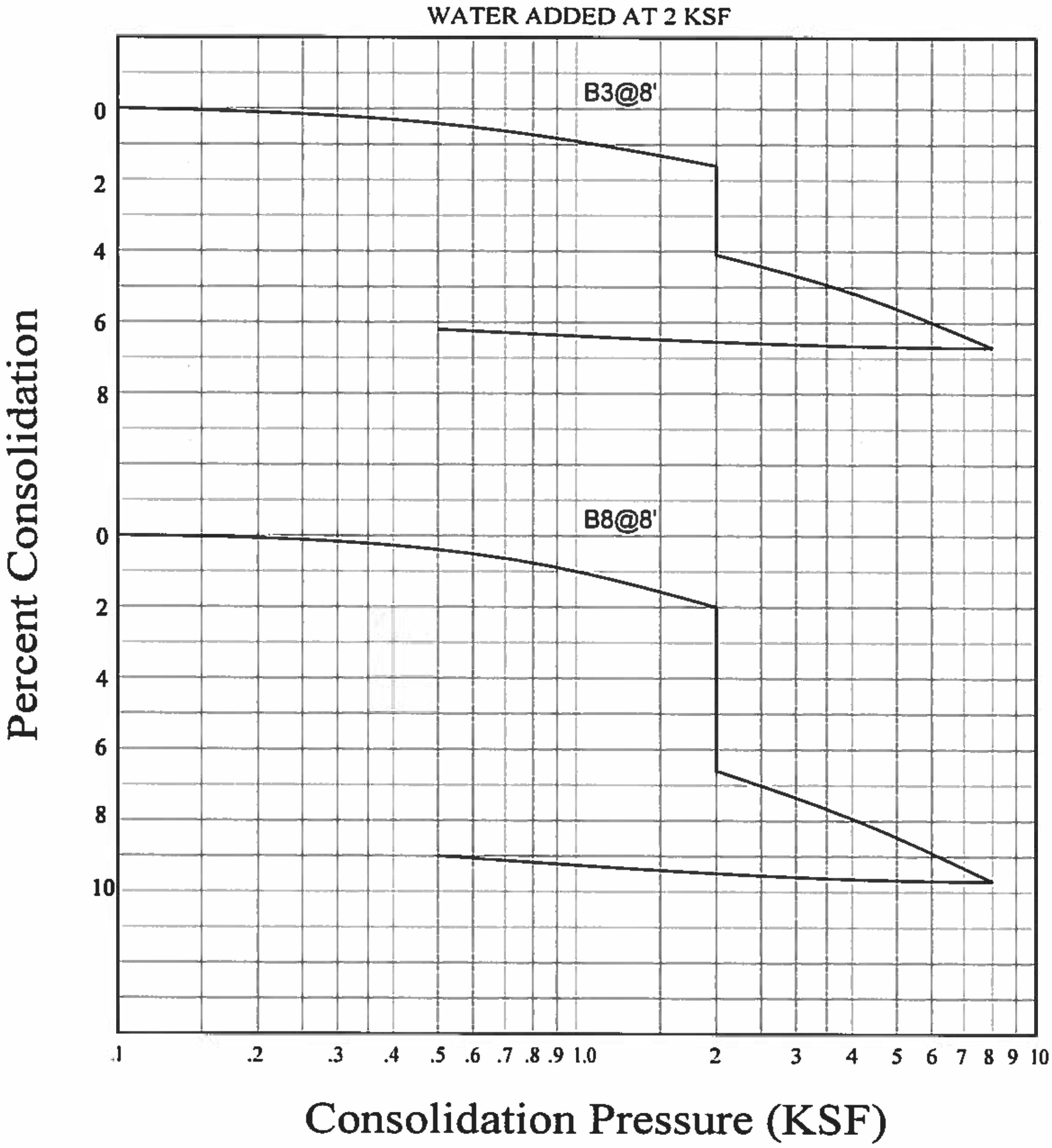
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B11



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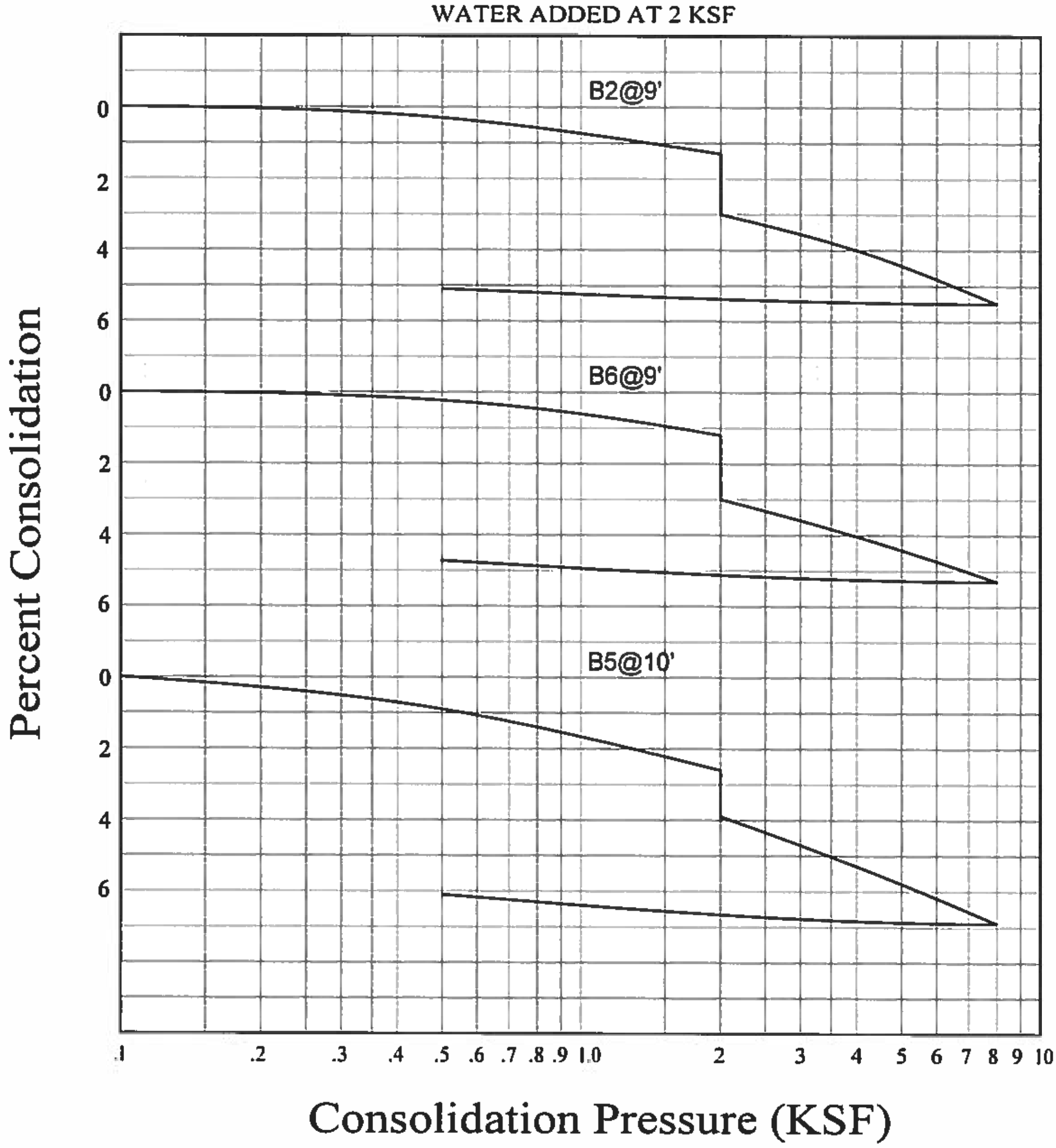
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B12



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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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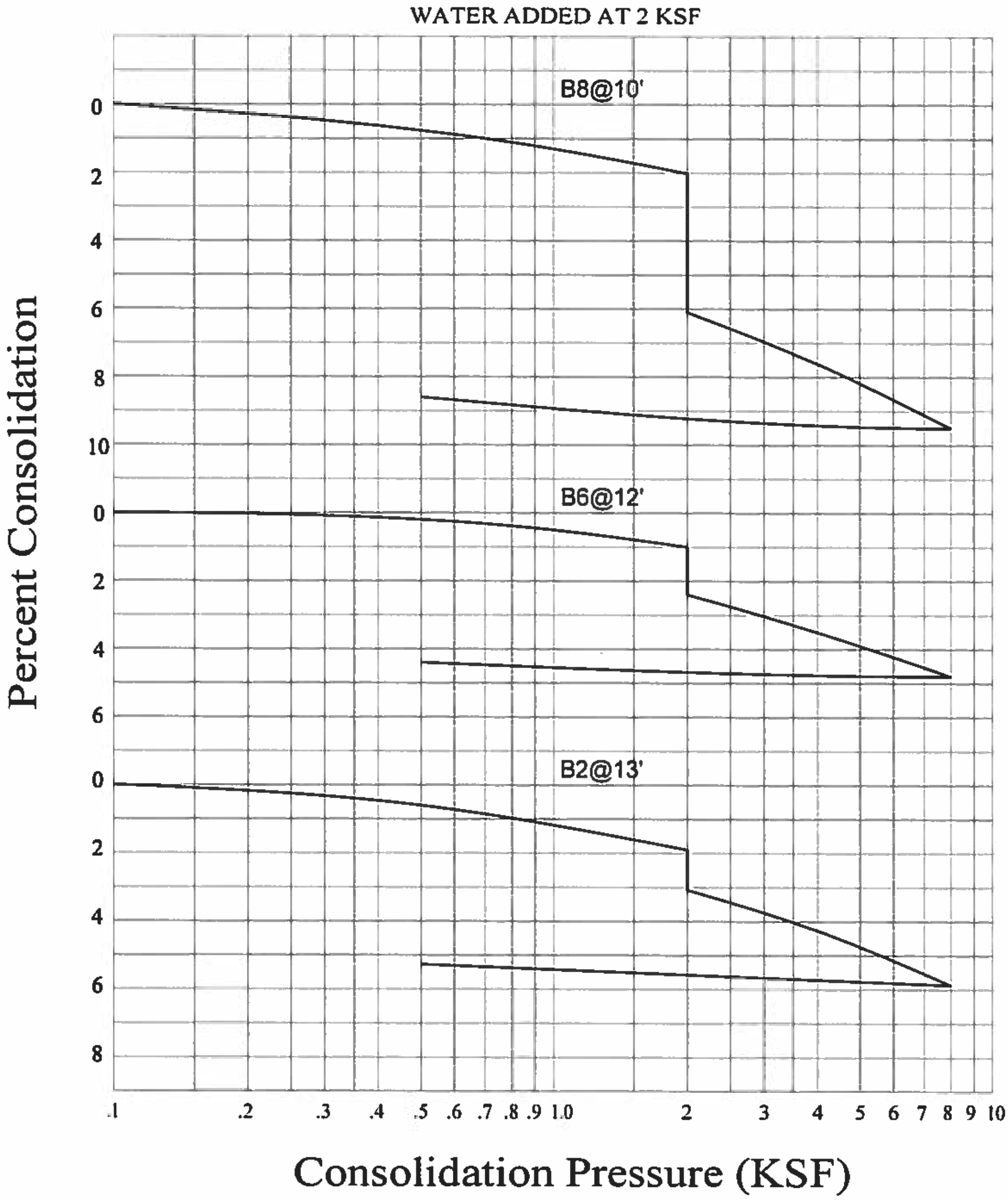
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B13



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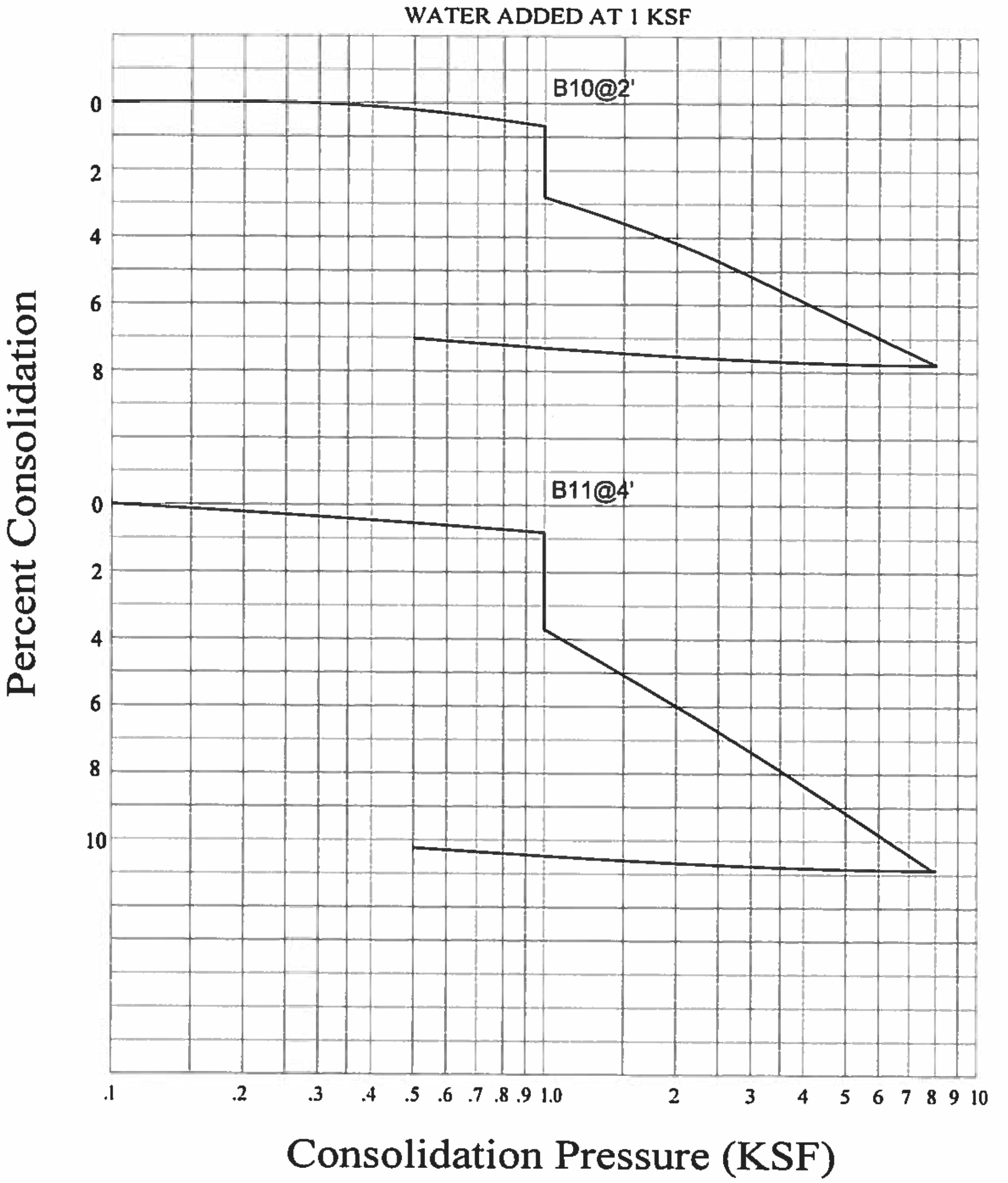
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B14



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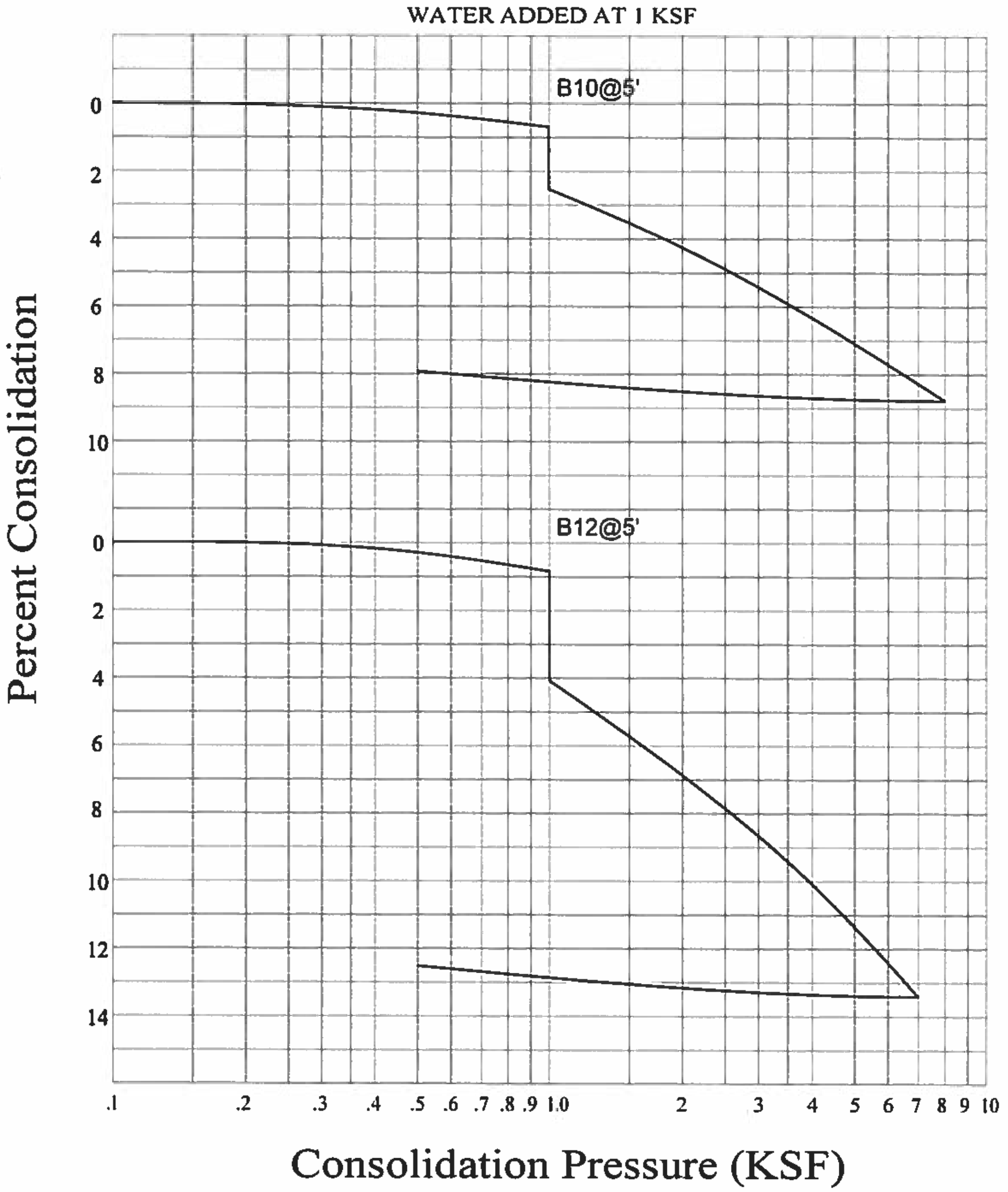
JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B15



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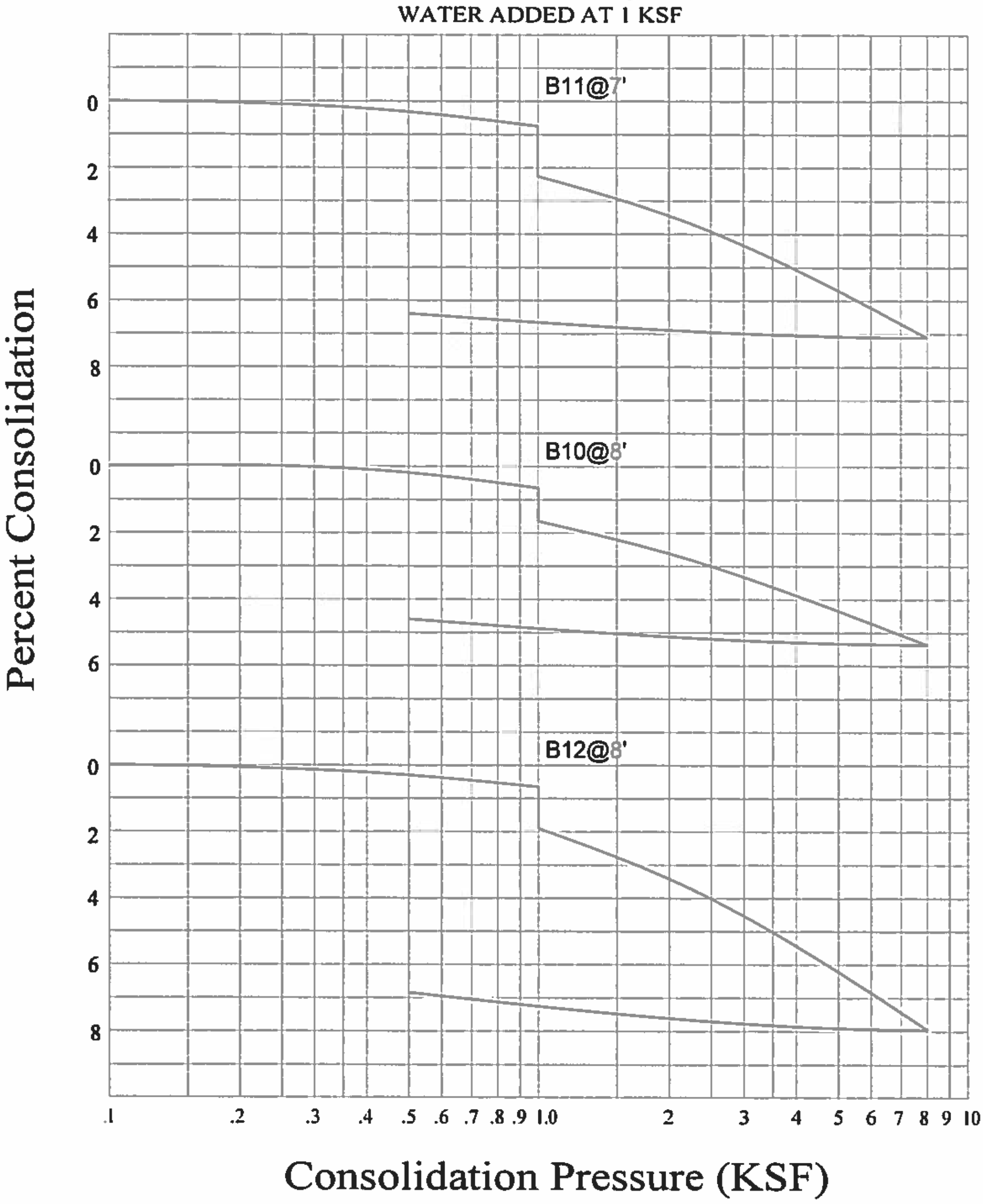
JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B16



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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
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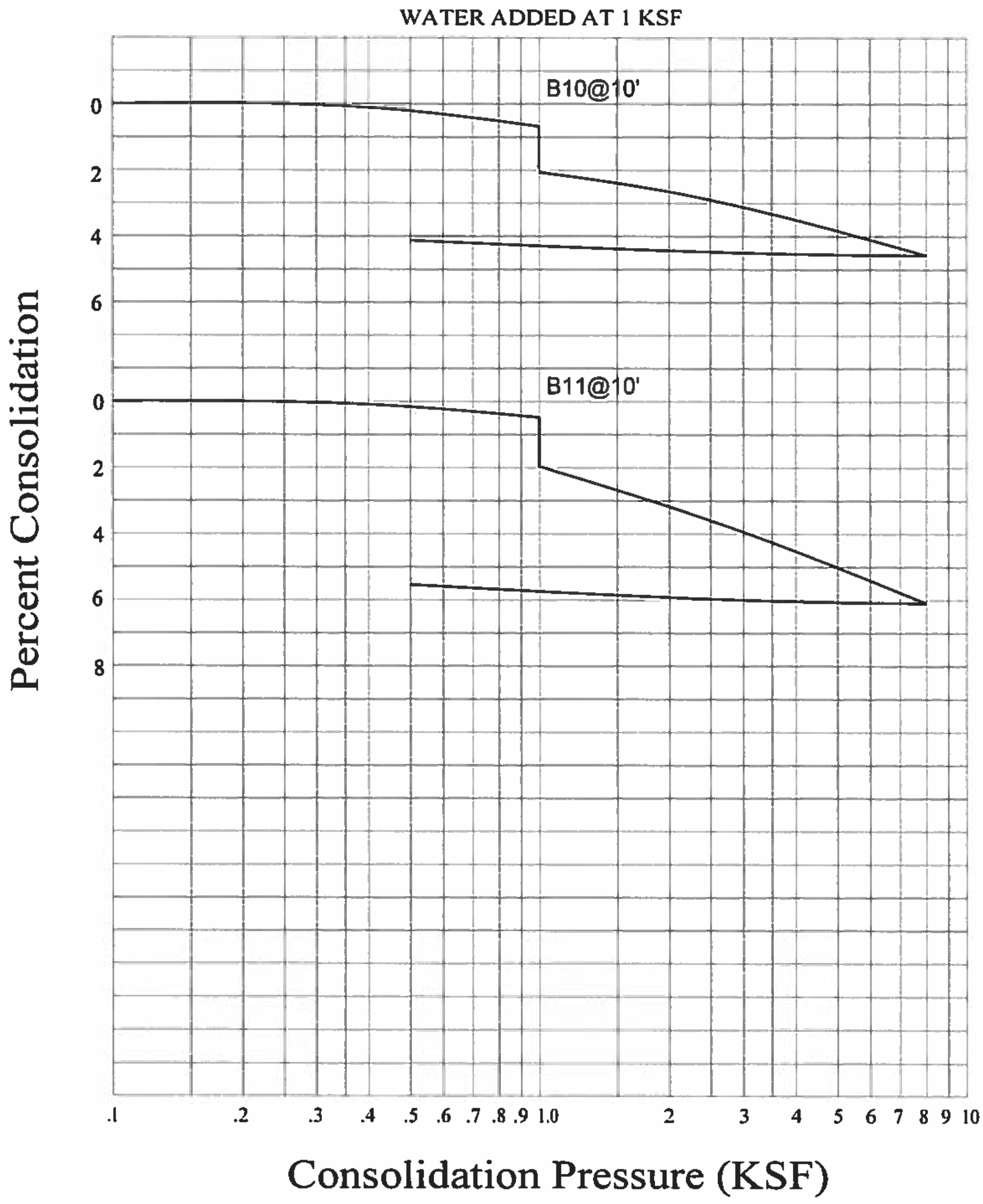
JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B17



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JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B18

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 0-5'	11.2	14.6	116.4	9	Very Low	Non-Expansive

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

** Reference: 2013 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'	Light Yellowish Brown Silty Sand	128.0	8.5
B6 @ 0-5'	Light Yellowish Brown Silty Sand	128.0	9.0

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TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B19

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0-5'	7.81	6300 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 0-5'	0.001

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 0-5'	0.010	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 381-11 Section 4.3.

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CORROSIVITY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

SEPTEMBER 2014	PROJECT NO. A8898-06-01	FIG. B20

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT
LAS TERRAZAS AT COLTON
275 AND 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CALIFORNIA
APN: 0274-182-34, -43, & -46

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LAS TERRAZAS FUND, L.P.
AGOURA HILLS, CALIFORNIA

PROJECT NO. A8898-06-01

FEBRUARY 20, 2013

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A8898-06-01
February 20, 2013

VIA OVERNIGHT DELIVERY

Las Terrazas Fund, L.P.
30141 Agoura Road, Suite 100
Agoura Hills, CA 91301

Attention: Mr. Jay Ross

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
LAS TERRAZAS AT COLTON
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CALIFORNIA
APN: 0274-182-34, -43, &-46

Dear Mr. Ross:

In accordance with your authorization of our proposal, we have performed an updated geotechnical investigation for the proposed Las Terrazas at Colton residential development located at 275 and 291 Cypress Avenue in the Unincorporated San Bernardino County, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations in this report are followed and implemented during design and construction.

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

Very truly yours,
GEOCON WEST, INC.

César H. Larios	Jelisa M. Thomas	Neal D. Berliner
CEG 2578	PE 74946	GE 2576

(4+CD) Addressee

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE

This report presents the results of an updated geotechnical investigation for the proposed Las Terrazas at Colton residential development located at 275 and 291 Cypress Avenue in the Unincorporated San Bernardino County, California (see Figure 1, Vicinity Map). 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 aspects of proposed design and construction.

The scope of our investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was initially explored on December 19, 2011 by excavating nine 7-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 5½ and 20½ feet below the existing ground surface. Percolation testing for the design of a stormwater infiltration system was performed two of the borings. A supplemental site exploration was performed on January 28, 2013 by excavating four 4-inch diameter borings using manual hand auger equipment. The borings were advanced to depths between 4½ and 10½ feet below the ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan, Figure 2. A detailed discussion of the 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.

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.

2. SITE AND PROJECT DESCRIPTION

The subject property is located at 275 and 291 Cypress Avenue in the Unincorporated San Bernardino County, California. The property is a 6.14-acre, irregularly shaped parcel. The majority of the parcel is currently vacant, with a vacant single-family residential structure located within the eastern portion of the site. The property is bounded by existing single family residential structures to the north and northeast, by Cypress Avenue to the southeast, by West Valley Boulevard to the south, and by an existing public storage facility to the west. The site slopes gently to the south and southwest with approximately 10 feet of vertical relief across the site. Water drainage at the site appears to be by sheet flow along the existing ground contours towards the city streets. Vegetation on site consists of grass and shrubs located throughout the site.

Information concerning the proposed development was furnished by the client and is preliminary in nature. It is our understanding that the proposed development will consist of two 1- and 2-story multi-family residential structures, a 3,000 square-foot single-story community building, a 3,500 square-foot child care / learning center, a 1,000 square-foot neighborhood service building, a swimming pool and paved parking lot areas to be constructed at or near existing site grade (see Site Plan, Figure 2).

Due to the preliminary nature of the design at this time, wall and column loads were not made available. It is anticipated that Type V wood-frame construction will be utilized, and it is estimated that wall loads for the proposed structures could be up to 3 kips per linear foot, and column loads could be up to 300 kips.

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

3. GEOLOGIC SETTING

The site is located along the eastern edge of the Chino Basin in San Bernardino County. The Chino Basin encompasses a broad area of coalescing alluvial fans that extend southward from the San Gabriel Mountains. The Chino Basin overlies a down-dropped structural block, the Perris Block which is bounded by the Chino and Elsinore Faults to the southeast, the Puente hills to the west, the San Gabriel Mountains to the north, by the San Jacinto fault to the northeast, and the La Sierra Hills and Juniper Mountains to the south east. The alluvial deposits within the Chino Basin have been reworked by wind during the Holocene (last 11,000 years) and Pleistocene (11,000 to 2 million years) epochs. As a result, a thin veneer of eolian sand covers extensive areas of the Chino Basin.

Regionally, the Chino Basin is located within the Peninsular Ranges geomorphic province. This province comprises the northwesterly-trending mountains and valleys extending from the southern Baja Peninsula to the Transverse Ranges in Southern California.

4. GEOLOGIC MATERIALS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of artificial fill underlain by Pleistocene Age older alluvial deposits (Morton, 1978). The soil and geologic units encountered at the site are discussed below. Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Various amounts of artificial fill were encountered throughout the area of the proposed development. The fill was observed in our field explorations to a maximum depth of 4½ feet below existing ground surface. The artificial fill generally consists of brown to yellowish brown silty sand and sandy silt. The artificial fill is characterized as dry and medium dense or soft. The fill is likely the result of past grading and demolition activities at the site. Deeper fill may occur between borings and on other parts of the site that were not directly explored.

4.2 Older Alluvium

The artificial fill is underlain by Pleistocene Age older alluvial deposits generally consisting of brown to yellowish brown poorly graded sand, silty sand, and sandy silt with varying amounts of gravel. The soils are primarily dry to slightly moist and medium dense to very dense, and become denser with increased depth.

5. GROUNDWATER

A review of data provided by the California Department of Water Resources (CDWR, 2011) indicates that several wells have historically been drilled in the site vicinity. The closest wells to the site are Well No. 01S04W19E001S and Well No. 01S05W24H002S, located approximately 0.29 miles west and 0.36 miles northwest of the site. The State well numbering system is based on the township, range, section, and tract in which the well is located.

Review of the monitoring data between 1964 and 1997 for Well No. 01S04W19E001S indicates that the depth to groundwater has fluctuated between 148.4 and 193.9 feet beneath the ground surface. The most recent groundwater level measurement for Well No. 01S04W19E001S was measured in October 1997 at a depth of 162.8 feet below the existing ground surface (CDWR, 2011).

Review of the monitoring data between 1997 and 2008 for Well No. 01S05W24H002S indicates that the depth to groundwater has fluctuated between 172.4 and 190.7 feet beneath the ground surface. The most recent groundwater level measurement for Well No. 01S05W24H002S was measured in April 2008 at a depth of 189.1 feet below the existing ground surface (CDWR, 2011).

Based on a review of the Chino Basin Watermaster, Depth to Groundwater Contours map (Chino Basin Watermaster, 2006), groundwater levels in the area are approximately 150 feet beneath the ground surface, which is relatively consistent with water level measurements observed in CDWR Well No. 01S04W19E001S and Well No. 01S05W24H002S.

Site exploration drilled to a maximum depth of 20½ feet below the ground surface, did not encounter groundwater. Based on these considerations, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to excessive irrigation or precipitation. 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 7.23).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (formerly known as California Division of Mines and Geology (CDMG)) for the Alquist-Priolo Earthquake Fault Zone Program (Hart, 1999). 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. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The site, however, is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Rialto Colton Fault located approximately 0.4 miles northeast of the site (Ziony and Jones, 1989). Other nearby active faults are the San Jacinto Fault Zone, the San Andreas Fault Zone, the Mill Creek Fault, and the Crafton Hills Fault Zone located 2.0 miles northeast (CDMG, 1977), 8.0 miles northeast, 8.3 miles northeast and 8.8 miles east-southeast of the site, respectively (Ziony and Jones, 1989).

The closest potentially active fault to the site is the Little Creek Fault located approximately 3.5 mile north of the site (Ziony and Jones, 1989). Other nearby potentially active faults are the Grass Valley Fault and the Tunnel Ridge Fault located approximately 15 miles north and 15 miles north-northeast of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows Earthquake, and the January 17, 1994 M_w 6.7 Northridge Earthquake were a result of movement on the buried thrust faults. These thrust faults are not exposed at the surface and do not present a potential surface fault rupture hazard; however, these active features are capable of generating future earthquakes.

6.2 Seismicity

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

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	26	S
San Jacinto-Hemet area	April 21, 1918	6.8	30	SE
Near Redlands	July 23, 1923	6.3	7	SE
Long Beach	March 10, 1933	6.4	48	SW
Tehachapi	July 21, 1952	7.5	115	NW
San Fernando	February 9, 1971	6.6	65	NW
Whittier Narrows	October 1, 1987	5.9	42	W
Sierra Madre	June 28, 1991	5.8	40	NW
Landers	June 28, 1992	7.3	53	ENE
Big Bear	June 28, 1992	6.4	31	NE
Northridge	January 17, 1994	6.7	69	WNW
Hector Mine	October 16, 1999	7.1	71	NNE

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

6.3 Estimation of Peak Ground Accelerations

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

6.3.1 Deterministic Analysis

Table 1 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 Sadigh et al. (1997). The resulting calculated peak horizontal accelerations at the site are indicated 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 6.7 event on the San Jacinto – San Bernardino Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.731.

While listing of peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site.

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

6.3.2 Probabilistic Analysis

The computer program *FRISKSP* (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance for given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary Fault is proportional to the faults' slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone.

Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh et al. (1997) 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 2010 California Building Code and ASCE 7-05, 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 motions at the site of approximately 1.28g and 0.90g, respectively. Graphical representation of the analysis is presented on Figure 5.

6.4 Seismic Design Criteria

The following table summarizes site-specific design criteria obtained from the 2010 California Building Code (CBC; Based on the 2009 International Building Code [IBC]), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The values were derived using the computer program Seismic Hazard Curves and Uniform Hazard Response Spectra, provided by the USGS. The short spectral response uses a period of 0.2 second.

CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2010 CBC Reference
Site Class	D	Table 1613.5.2
Spectral Response – Class B (short), S_S	1.786g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.624g	Figure 1613.5(4)
Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Site Coefficient, F_v	1.5	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.786g	Section 1613.5.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.935g	Section 1613.5.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.191g	Section 1613.5.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.624g	Section 1613.5.4 (Eqn 16-39)

Conformance to the criteria in the above table 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 intent of the code is “Life Safety,” not to completely prevent damage to the structure, since such design may be economically prohibitive.

6.5 Liquefaction Potential

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

The current standard of practice, as outlined in the “Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California” requires liquefaction analysis to a depth of fifty 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 County of San Bernardino General Plan (2005) this site is not located in an area designated as “liquefiable”. As stated previously, the depth to groundwater at the site is greater than 50 feet beneath the existing ground surface. Based on these considerations, it is our opinion that the potential for liquefaction of the site soils is very low. Further, no surface manifestations of liquefaction are expected at the subject site.

6.6 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. Based on the relatively dense nature of the older alluvium, appreciable seismically-induced settlements are not anticipated.

6.7 Landslides

According to the County of San Bernardino General Plan (2005) the site is not located within an area identified as having a potential for seismic slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.

6.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. A review of the County of San Bernardino General Plan (2005) indicates that the site is not located within the inundation boundaries of upgradient dams or reservoirs. The probability of earthquake-induced flooding is considered very low.

6.9 Tsunamis and Seiches

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

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is in an area which flood hazards are undetermined, but possible (Zone D) as defined by the Federal Emergency Management Agency (FEMA).

6.10 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-7, the site is not located within the boundaries of an oil field. No oil wells are located in the immediate vicinity of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map. Other wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a known oil field; therefore, the potential for the presence of a methane zone is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.11 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 The depth of artificial fill encountered during field exploration was observed to be variable, with a maximum depth of 4½ feet. The existing fill encountered is believed to be the result of past grading and/or demolition activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations, slabs, or additional fill.

7.1.3 The results of our laboratory testing indicate that the existing upper alluvial soils are subject to excessive hydro-consolidation upon saturation (see Figures B4 through B14). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils or foundations supported therein.

7.1.4 It is our opinion that the existing artificial fill and upper alluvial soils, in their present condition, are not suitable for direct support of proposed foundations, slabs, or additional fill. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (See Section 7.4).

7.1.5 Based on these considerations, as a minimum it is recommended that the upper six feet of existing site soils be excavated and properly compacted for foundation and slab support. Deeper excavation should be conducted as necessary at the direction of the Geocon representative to completely remove all existing artificial fill and soft alluvial soil. The excavation should extend laterally a minimum distance of five feet beyond the building footprint areas or for a distance equal to the depth of fill below the foundations, whichever is greater. Prior to placing any fill, the excavation bottom must be proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon). If determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. Recommendations for earthwork and bottom stabilization are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 Subsequent to the recommended grading, the proposed structure may be supported on conventional foundations deriving support on the newly placed engineered fill. As a minimum, all proposed building foundations deriving support in engineered fill should be underlain by at least three feet of newly placed engineered fill, and grading should be conducted as necessary to maintain the recommended three-foot-thick engineered fill blanket beneath all foundations.

7.1.7 As an alternative to conventional foundations, a post-tensioned concrete slab and foundation system may be utilized for the support of the proposed on-grade structures. A post-tensioned foundation system can be utilized to reduce the potential for foundation distress resulting from differential settlement of the underlying soils. As a minimum, post-tensioned foundations deriving support in engineered fill should be underlain by at least two feet of newly placed engineered fill, and grading should be conducted as necessary to maintain the recommended 2-foot-thick engineered fill blanket beneath all foundations.

7.1.8 It is anticipated that stable excavations can be achieved with sloping measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.19).

7.1.9 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 4 feet. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils 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.

7.1.10 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium may experience increased settlement and/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 properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.11 Percolation testing of the site soils indicates that the soils are capable of infiltration. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.22).

7.1.12 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.1.13 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all 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.

7.2.3 All 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 shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.19).

7.2.4 The upper few feet of soils encountered during this investigation are considered to have a “very low” expansive potential (EI=9); and are classified as “non-expansive” based on the 2010 California Building Code (CBC) Section 1803.5.3. The recommendations in this report assume that foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, Chloride and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that a potential for corrosion of buried ferrous metals exists on site. The results are presented in Appendix B (Figure B20) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B20) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2010 CBC Section 1904.3 and ACI 318-08 Sections 4.2 and 4.3.

7.3.3 Geocon West, Inc. 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 corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

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

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

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

7.4.4 As a minimum, it is recommended that the upper 6 feet of existing site soils be excavated and properly compacted within the proposed building footprint areas. Any encountered deeper fill or soft soils should be completely over-excavated or stabilized as necessary at the direction of the Geotechnical Engineer. Deeper excavations should be conducted as necessary to maintain the recommended 3-foot-thick engineered fill blanket beneath proposed conventional foundations, and 2-foot-thick engineered fill blanket beneath proposed post-tensioned foundations. Where excavation and compaction is to be conducted, the excavation should extend laterally a minimum distance of five feet beyond the building footprint area or for a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

7.4.5 Prior to placing any fill, the excavation bottom must be proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon) and approved in writing. If determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate.

7.4.6 If subgrade stabilization is required at the excavation bottom, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved by introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils.

7.4.7 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.4.8 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.9 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will not be tied-in to the proposed building, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 4 feet below the ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.10 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.4.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (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 observed 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 2-sack slurry is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.12 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B20). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 20 percent should be anticipated when excavating and compacting the existing fill and alluvium on site to an average relative compaction of 92 percent.

7.5.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed structures provided foundations derive support exclusively in newly placed engineered fill. Conventional spread foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.6.2 As an alternative to conventional foundations, a post-tensioned concrete slab and foundation system may be utilized for the support of the proposed on-grade structures. Recommendations for post-tensioned foundations are provided in Section 7.10.

7.6.3 Continuous foundations may be designed for an allowable bearing capacity of 2,500 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.

7.6.4 Isolated spread foundations for the proposed building may be designed for an allowable bearing capacity of 2,800 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 18 inches into the recommended bearing material.

7.6.5 The soil bearing pressure above may be increased by 200 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

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

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

7.6.8 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. Additional grading should be performed as necessary in order to maintain the required three-foot-thick engineered fill blanket beneath conventional spread foundations.

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

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

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

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

7.7 Miscellaneous Foundations

7.7.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 4 feet. The contractor should be aware that special excavation measures may be required to construct continuous foundations adjacent to property lines or existing offsite improvements. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.7.2 It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundation, slabs, paving or improvements supported therein. The site soils are highly sensitive to excessive moisture and proper drainage should be maintained at all times.

7.7.3 If the soils exposed in the excavation bottom are soft, compaction of the soft soils 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, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.8 Conventional Foundation Settlement

7.8.1 The maximum expected settlement for the structure supported on a conventional foundation system with a maximum allowable soil bearing pressure of 4,000 psf is estimated to be approximately 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential static settlement is not expected to exceed ¾ inch over a distance of twenty feet.

7.8.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Lateral Design

7.9.1 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 0.34 may be used with the dead load forces in the properly compacted engineered fill and the undisturbed alluvium found at or below a depth of 4 feet.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or the undisturbed alluvium found at or below a depth of 4 feet may be computed as an equivalent fluid having a density of 200 pcf with a maximum earth pressure of 2,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Foundation Design – Post-Tensioned Foundation System

7.10.1 If utilized, post-tensioned concrete slab and foundation systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) Third Edition as required by the 2010 California Building Code (CBC Section 1806.8). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The parameters presented in the following table are based on the guidelines presented in the PTI, Third Edition design manual, as well as the consideration of the granular, non plastic nature of the upper site soils.

POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Value
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3
Edge Lift, y_M (inches)	0.61
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	0.30

7.10.2 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

7.10.3 Consideration should be given to using interior stiffening beams and connecting isolated footings as well as patio slabs which exceed 5 feet in width to the building foundation to reduce the potential for future separation to occur.

7.10.4 If the structural engineer proposes a post-tensioned foundation design method other than the PTI, Third Edition design manual:

- The post-tensioned foundation system design parameters above are still applicable.

- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 18 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.10.5 Foundations may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf) and should derive support exclusively in engineered fill. This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. Based on an anticipated allowable bearing pressure of 2,500 psf, it is recommended that the proposed structures be designed for a differential settlement of ½-inch over a distance of 20 feet.

7.10.6 The upper five feet of existing site soils encountered during this investigation are considered to have a "very low" expansive potential (EI=9). Post-tensioned foundation systems deriving support in soil possessing a "very low" expansion potential (expansion index of 20 or less) may be designed using the method described in Section 1806 of the 2010 CBC; or an alternative, commonly accepted design method (other than PTI Third Edition) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of ¾ inch. Geocon West, Inc. should be contacted to review the plans and provide additional information, if necessary.

7.10.7 Provided the moisture content in the soil is maintained subsequent to completion of grading, special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be maintained at two percent above optimum moisture content prior to and at the time of concrete placement as would be expected in any such concrete placement.

7.10.8 During the construction of the post-tension foundation system, the concrete should be placed monolithically and must be observed and approved by a Geocon inspector. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.12).

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

7.11.3 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 1643-98 and the manufacturer's recommendations. If California Green Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of ½-inch clean aggregate and the vapor retarder should be in direct contact with the concrete slab. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel.

7.11.4 For seismic design purposes, a coefficient of friction of 0.34 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.11.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of the subgrade should be moisture conditioned to near optimum moisture content and compacted to at least 95 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. Construction joints should be designed by the project structural engineer.

7.11.6 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 expansive soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all soft or unsuitable alluvial soils in the area of new paving is not required, however, paving constructed over existing unsuitable soils may experience increased settlement and/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 recompacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 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. 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	3.5	3	4
Driveways	5	3	6
Trash Truck & Fire Lanes	7	4	10

7.12.3 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials 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*" (Green Book).

7.12.4 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 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).

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

7.13 Swimming Pool/Spa

7.13.1 The proposed swimming pool shell bottom should derive support exclusively in newly placed engineered fill and should be underlain by at least 3 feet of engineered fill. Swimming pool foundations and walls may be designed in accordance with the *Conventional Foundation Design* and *Retaining Wall Design* sections of this report (See Sections 7.6 and 7.14). A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

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

7.14 Retaining Wall Design

7.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls significantly higher than 10 feet are planned, Geocon should be contacted for additional recommendations.

7.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* sections of this report (see Section 7.8).

7.14.3 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 38 pcf.

7.14.4 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 56 pcf.

7.14.5 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 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

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

7.15 Dynamic (Seismic) Lateral Forces

7.15.1 In accordance with the 2010 California Building Code, if the project possesses a seismic design category of D, E, or F, the proposed 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.

7.15.2 Braced retaining walls should be designed for the greater of either the at-rest earth pressure or the dynamic (seismic) lateral earth pressure (sum of the static active pressure and the dynamic (seismic) pressure increment).

7.15.3 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 27 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 peak ground acceleration of 0.42g (S_{DS}/2.5) and by applying a pseudo-static coefficient of 0.5.

7.16 Retaining Wall Drainage

7.16.1 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.

7.16.2 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.

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

7.17 Elevator Pit Design

7.17.1 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. The elevator slab and retaining wall footings should derive support in newly placed engineered fill and excavations should be conducted as necessary during mass grading to maintain at least two feet of engineered fill beneath blanket beneath the elevator pit slab and retaining wall foundations. Elevator pit walls may be designed in accordance with the recommendations in the *Conventional Foundation Design and Retaining Wall Design* section of this report (see Sections 7.8 and 7.14).

7.17.2 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.

7.17.3 Retaining wall drainage should be designed in accordance with Section 7.16 of this report. The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.17.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

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

7.18 Elevator Piston

7.18.1 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.

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

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

7.19 Temporary Excavations

7.19.1 Excavations on the order of 6 feet in vertical height may be required for the proposed grading of the site. The excavations are expected to expose fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose fill or sands are not present and where not surcharged by adjacent traffic or structures.

7.19.2 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. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.19.3 Continuous vertical excavation adjacent to and which extend below the existing footings could remove vertical and lateral support from the existing footings and are not recommended. Slot cutting or shoring will be required where the proposed excavations will be deeper than an existing adjacent foundation. Recommendations for both excavation methods are provided in the following sections.

7.19.4 The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.20 Slot Cutting

7.20.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation is made at a slope of 1:1. Alternate "A" slots of 3.9 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should each be 3.9 feet in width. The wall, foundation, or backfill should be completed in the "A" slots before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. If preferable to the contractor A-B slot-cutting may be utilized. Slot-cutting is not recommended for vertical excavations greater than 5 feet in height or where surcharged by more than 1,000 pounds per linear foot. The surcharge load from the existing offsite structure to the west should be evaluated by a qualified structural engineer, and the slot-cut calculation revised as necessary. A slot-cut calculation is provided below.

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	115.0	pcf
Friction Angle of Soils	(ϕ)	28.0	degrees
Cohesion of Soils	(c)	95.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.53	

Surcharge Pressure:

Line Load	(q_L)	1000.0	psf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

$b = H/(\tan\alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin\alpha)*(\cos\alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2\alpha)*(\tan\phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan\phi)+c]$

FS = Resistance Force/Driving Force

FS = $(R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	Driving Force Wedge + Surcharge per lineal foot of Slot Wdith	Resisting Force Failure Wedge per lineal foot of Slot Width	Resisting Force Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
65	2.3	6	670.3	639.8	380.1	868.9	4.1
66	2.2	6	640.0	609.4	355.7	829.6	4.1
67	2.1	5	610.2	579.1	332.3	790.9	4.0
68	2.0	5	580.8	549.1	309.9	752.8	4.0
69	1.9	5	551.8	519.2	288.3	715.3	4.0
70	1.8	5	523.2	489.5	267.6	678.2	3.9
71	1.7	4	495.0	460.2	247.8	641.6	3.9
72	1.6	4	467.1	431.2	228.8	605.4	3.9
73	1.5	4	439.5	402.5	210.6	569.7	3.9
74	1.4	4	412.2	374.2	193.3	534.3	3.9
75	1.3	3	385.2	346.3	176.6	499.3	3.9
76	1.2	3	358.4	318.9	160.7	464.6	3.9
77	1.2	3	331.9	291.9	145.5	430.2	3.9
78	1.1	3	305.6	265.5	131.0	396.1	3.9
79	1.0	2	279.4	239.6	117.1	362.2	4.0
80	0.9	2	253.5	214.4	103.9	328.6	4.0
81	0.8	2	227.7	189.7	91.2	295.1	4.0
82	0.7	2	202.0	165.7	79.1	261.9	4.1
83	0.6	2	176.5	142.3	67.6	228.8	4.1
84	0.5	1	151.1	119.7	56.6	195.8	4.2
85	0.4	1	125.8	97.7	46.1	163.0	4.3
86	0.3	1	100.5	76.6	36.1	130.3	4.4
87	0.3	1	75.3	56.2	26.5	97.7	4.5
88	0.2	0	50.2	36.6	17.3	65.1	4.6
89	0.1	0	25.1	17.9	8.5	32.5	4.7
90	0.0	0	0.0	0.0	0.0	0.0	4.8

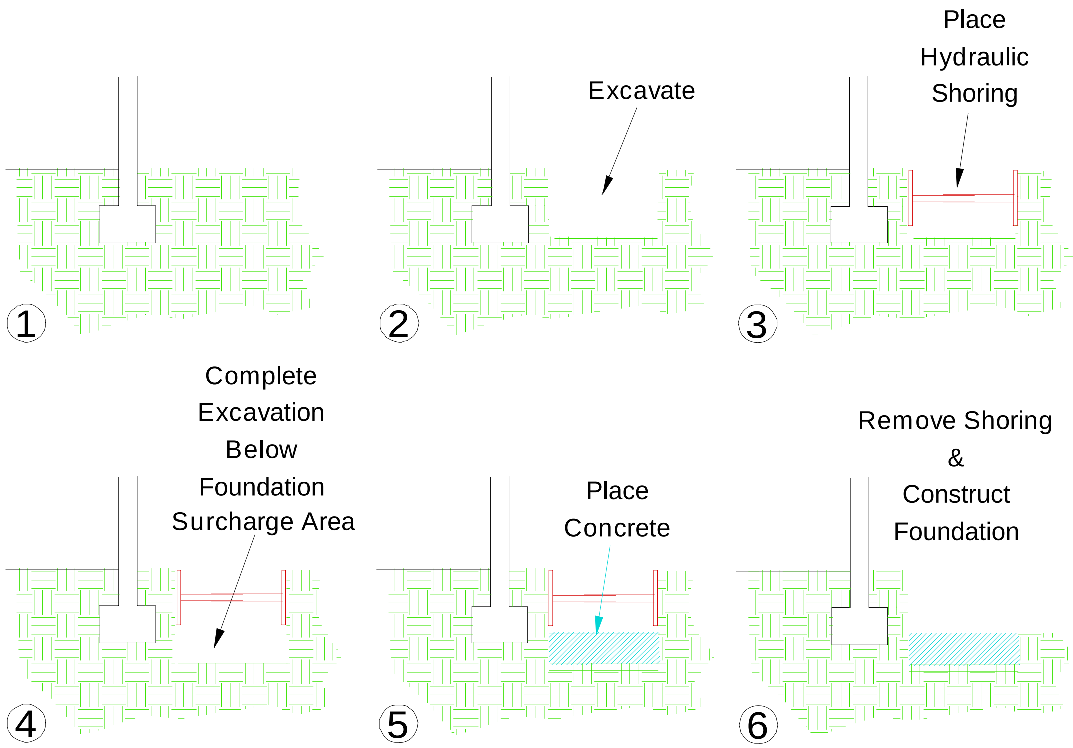
*Width of Slots to achieve a minimum of 1.5 Factor of Safety, with a Maximum Allowable Slot Width of 8-feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

d_{allow} = 3.9 feet

7.21 Shoring

7.21.1 As an alternative to slot cutting; hydraulic trench shoring may be implemented where excavations would remove a component of lateral support from adjacent foundations. The excavation may be conducted adjacent to the foundation but continuous excavation should not extend below the surcharge area of the existing foundation until the shoring is installed. The surcharge area may be defined by a 1:1 project down and away from the bottom of an existing foundation. Once shoring is installed, the excavation can be completed and the foundation can be constructed. Once the concrete backfill is placed to an elevation that is slightly above the bottom of the existing adjacent foundation, the shoring may be removed and the new foundation constructed. See illustration below.



7.21.2 It is recommended that an equivalent fluid pressure based on the table below, be utilized for design of hydraulic shoring.

HEIGHT OF SHORED EXCAVATION (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 5	30	50

7.21.3 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, the at-rest pressure should be considered for design purposes.

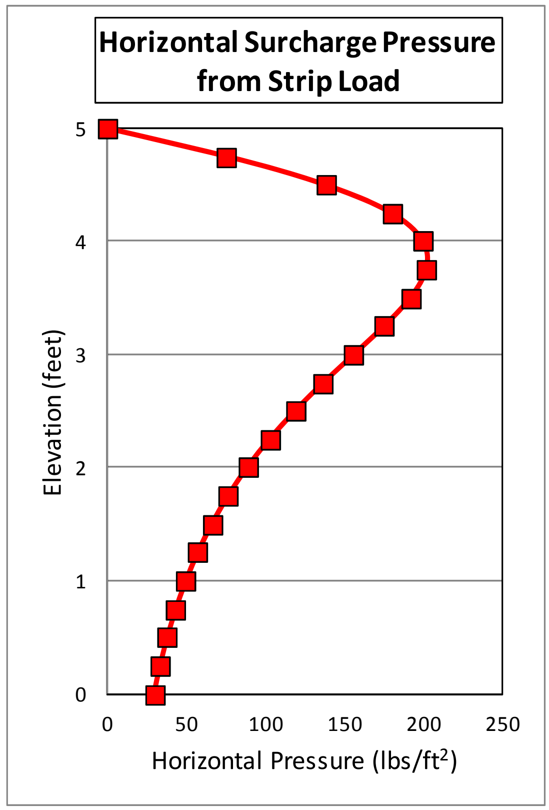
7.21.4 Additional active pressure should be added for a surcharge condition due to the adjacent structure as indicated in the calculation and diagram below. This calculation is based on several assumptions and should be verified once actual footing loads are available.

Description: Surcharge on Shoring From Existing Foundation

Horizontal Surcharge Pressure from Strip Load

Stip Load	Ql=	1000 lbs/lf
Height of Cut	H=	5 ft
Distance Away	X1=	0 ft
	m =	0

Elevation (feet)	n-value	Horizontal Pressure (lbs/ft^2)
5	0	0.00
4.75	0.05	75.74
4.5	0.1	138.41
4.25	0.15	180.15
4	0.2	200.00
3.75	0.25	201.99
3.5	0.3	192.00
3.25	0.35	175.42
3	0.4	156.25
2.75	0.45	136.98
2.5	0.5	118.98
2.25	0.55	102.85
2	0.6	88.76
1.75	0.65	76.63
1.5	0.7	66.27
1.25	0.75	57.47
1	0.8	50.00
0.75	0.85	43.66
0.5	0.9	38.26
0.25	0.95	33.66
0	1	29.73



Maximum Pressure =	201.99 lbs/ft^2
Total Load per Lineal Foot of Wall =	537.08 lbs/ft

7.22 Stormwater Infiltration

7.22.1 During the December 19, 2011 site exploration, borings B4 and B8 were utilized to perform percolation testing. The borings were advanced to the depths listed in the table below. Slotted casing was placed in each boring, and the annular space between the casing and excavation was filled with filter pack. The borings were then filled with water to pre-saturate the soils. On December 20, 2011, the casing was refilled with water, maintained at a depth of at least 1 foot above the excavation bottom for at least 30 minutes, and then percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the average infiltration rate (adjusted percolation rate) per boring for the earth materials encountered is listed in the following table.

Boring	Infiltration Depth (ft.)	Predominate USCS Soil Classification	Average Infiltration Rate (in / hour)
B4	10-15	Sand (SP)	2.9
B8	10-15	Silty Sand (SM) / Sand (SP)	1.2

7.22.2 Based on the results of the subsequent laboratory testing, the upper alluvial soils are subject to excessive settlement when saturated. Therefore, it is recommended that infiltration of storm water occur below a depth of 15 feet to minimize saturation of the soils supporting the proposed structures.

7.22.3 Provided infiltration occurs below a depth of 15 feet, resulting settlements from stormwater infiltration are anticipated to be less than ¼ inch at the ground surface, if any, and are not expected to affect existing or proposed structures or improvements. In addition, it is our opinion that the introduction of stormwater at these depths will not create a perched groundwater condition, and will not increase the potential for liquefaction.

7.22.4 Stormwater infiltration should be kept a minimum of 10 feet horizontally from adjacent foundations. In addition, where adjacent to any subterranean retaining walls, such as the proposed swimming pool, the discharge of stormwater should occur at a depth such that the retained soils do not become saturated. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.22.5 If the stormwater infiltration systems will be located in close proximity to a building pad, it is recommended that the stormwater infiltration system be installed during the mass grading of the site and prior to construction of any nearby building foundations. If installed after building foundation construction, the excavation required for installation of the stormwater infiltration system could remove a component of lateral support from the foundations and therefore would require shoring.

7.22.6 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation side walls and the infiltration system with two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.22.7 The design drawings and installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.23 Surface Drainage

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

7.23.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 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 soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the engineered fill providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

7.23.3 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.

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

7.24 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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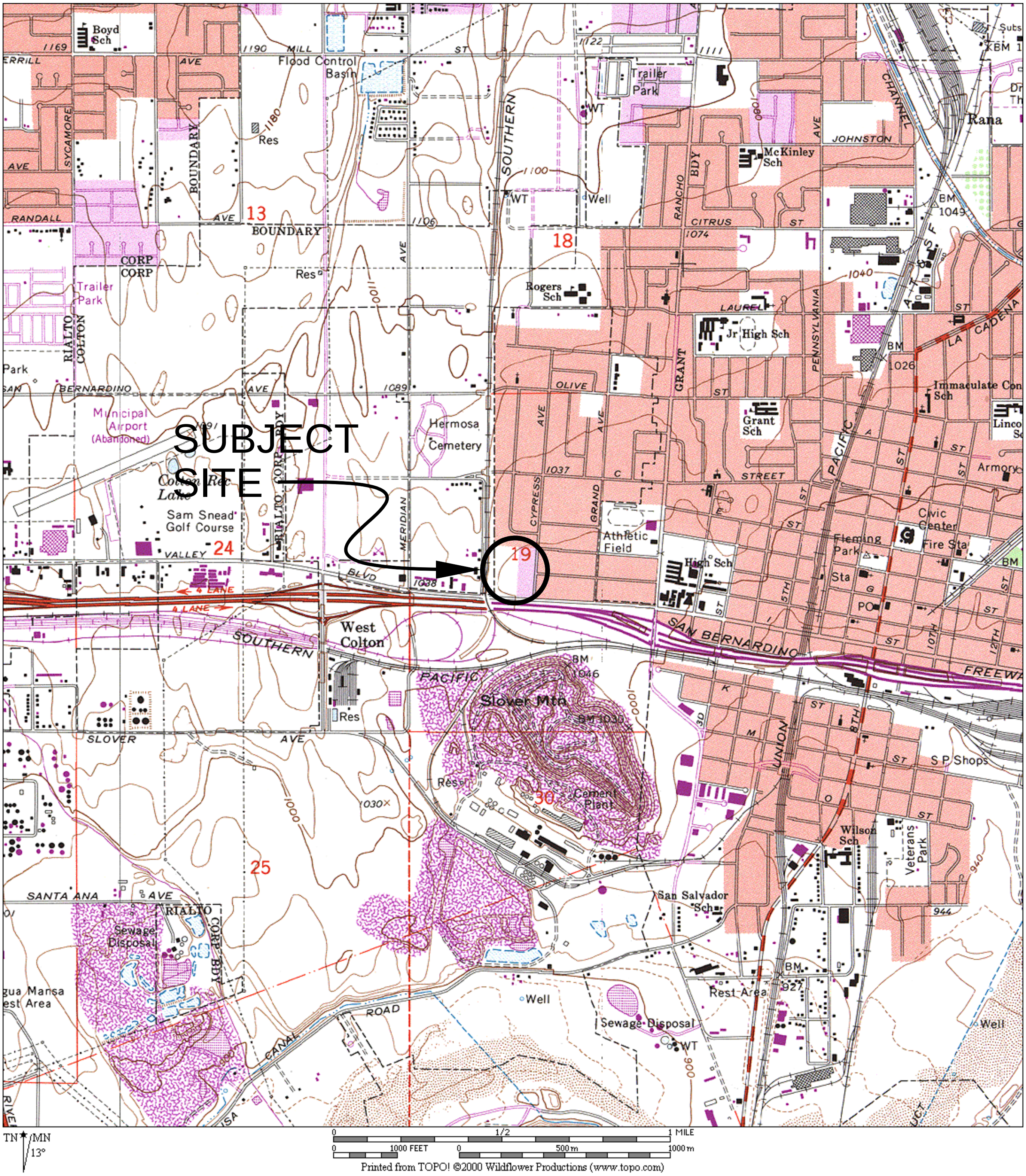

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3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

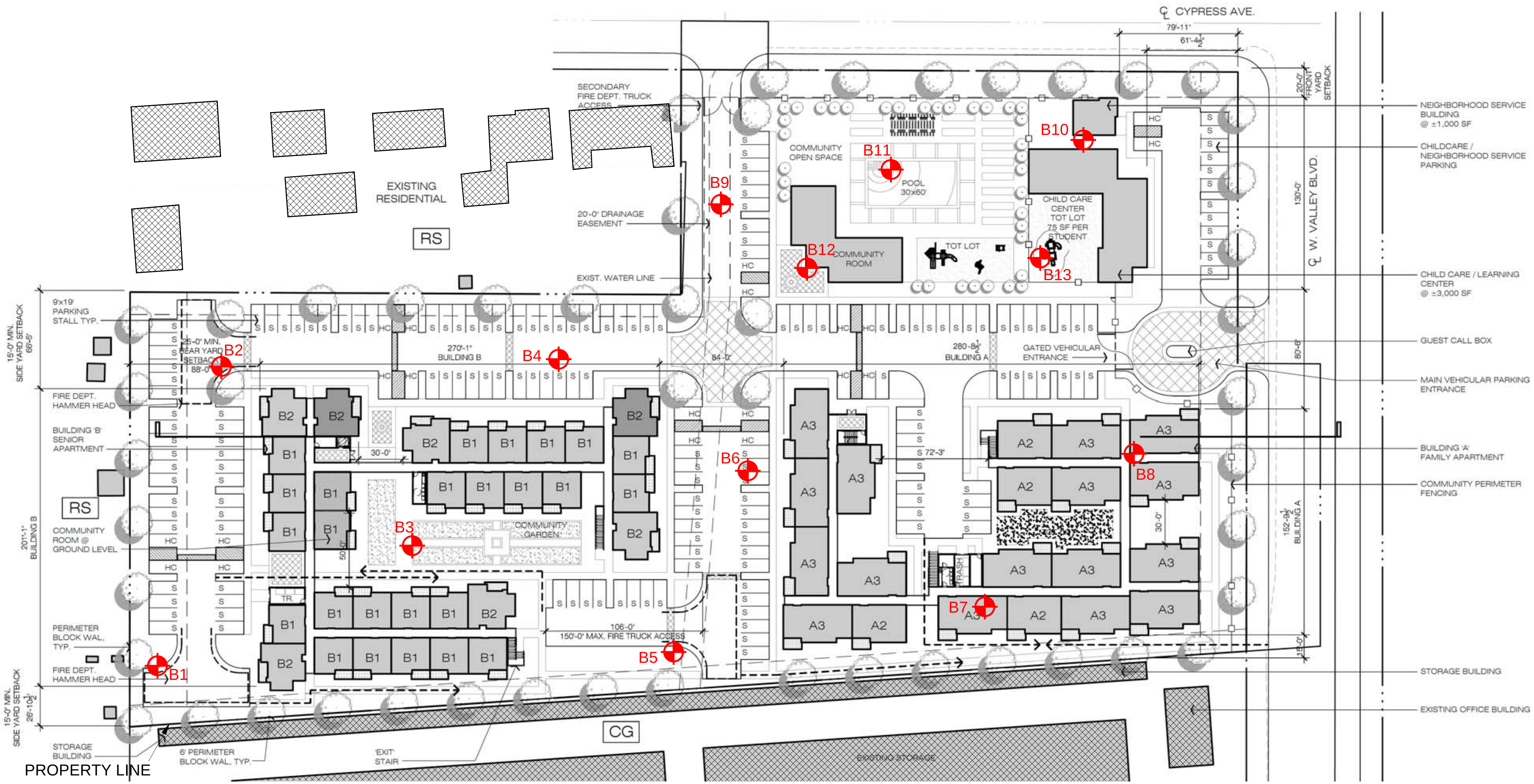
CHL		8000

VICINITY MAP

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 1

PLAN BY: Withee Malcolm Architects, LLP



LEGEND

B13 Approximate Location of Boring

Approximate Location of Proposed Structure

Approximate Location of Offsite Structures

0 80' 160'

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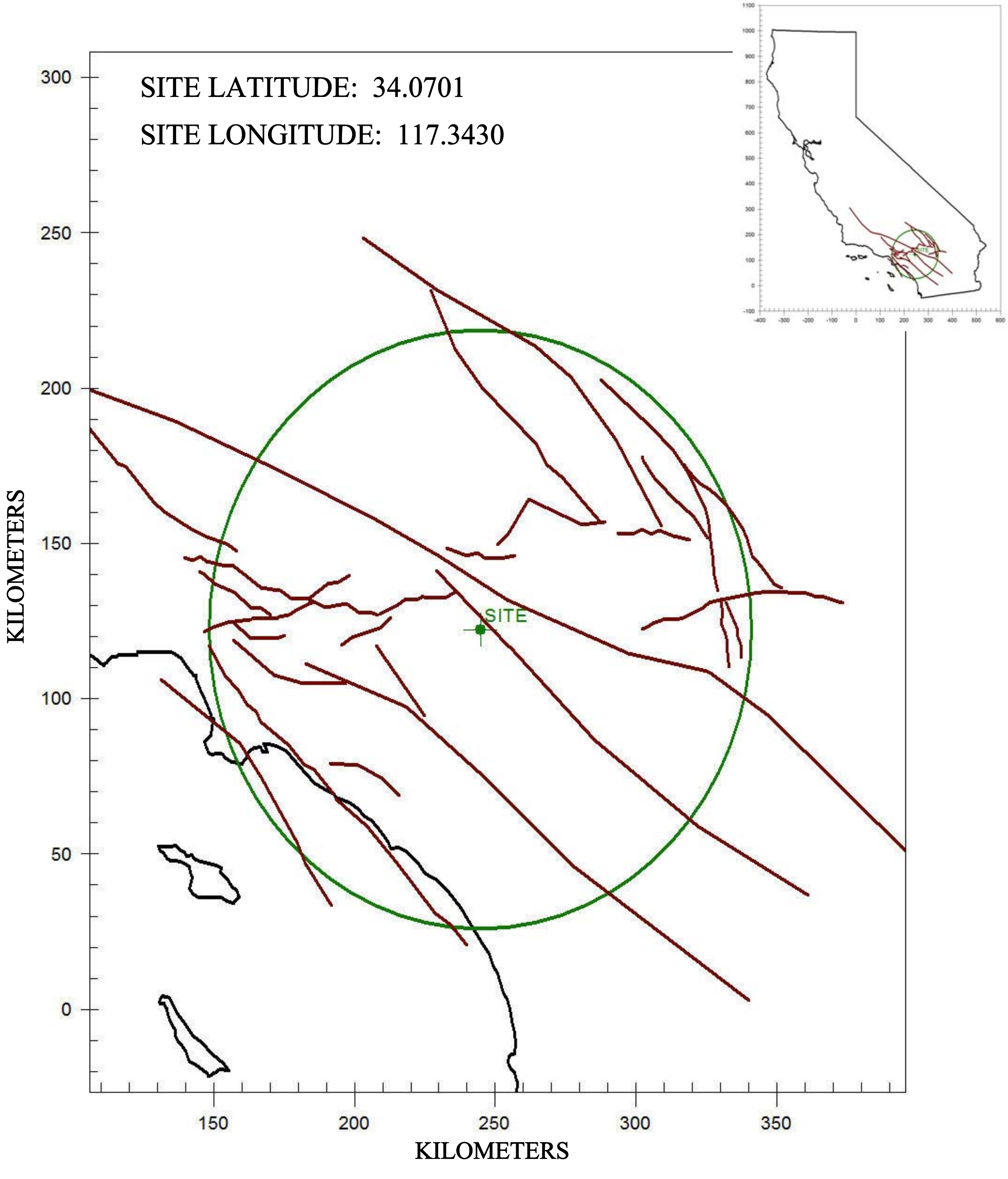
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

CHL		8000

SITE PLAN

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 2



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REGIONAL FAULT MAP

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
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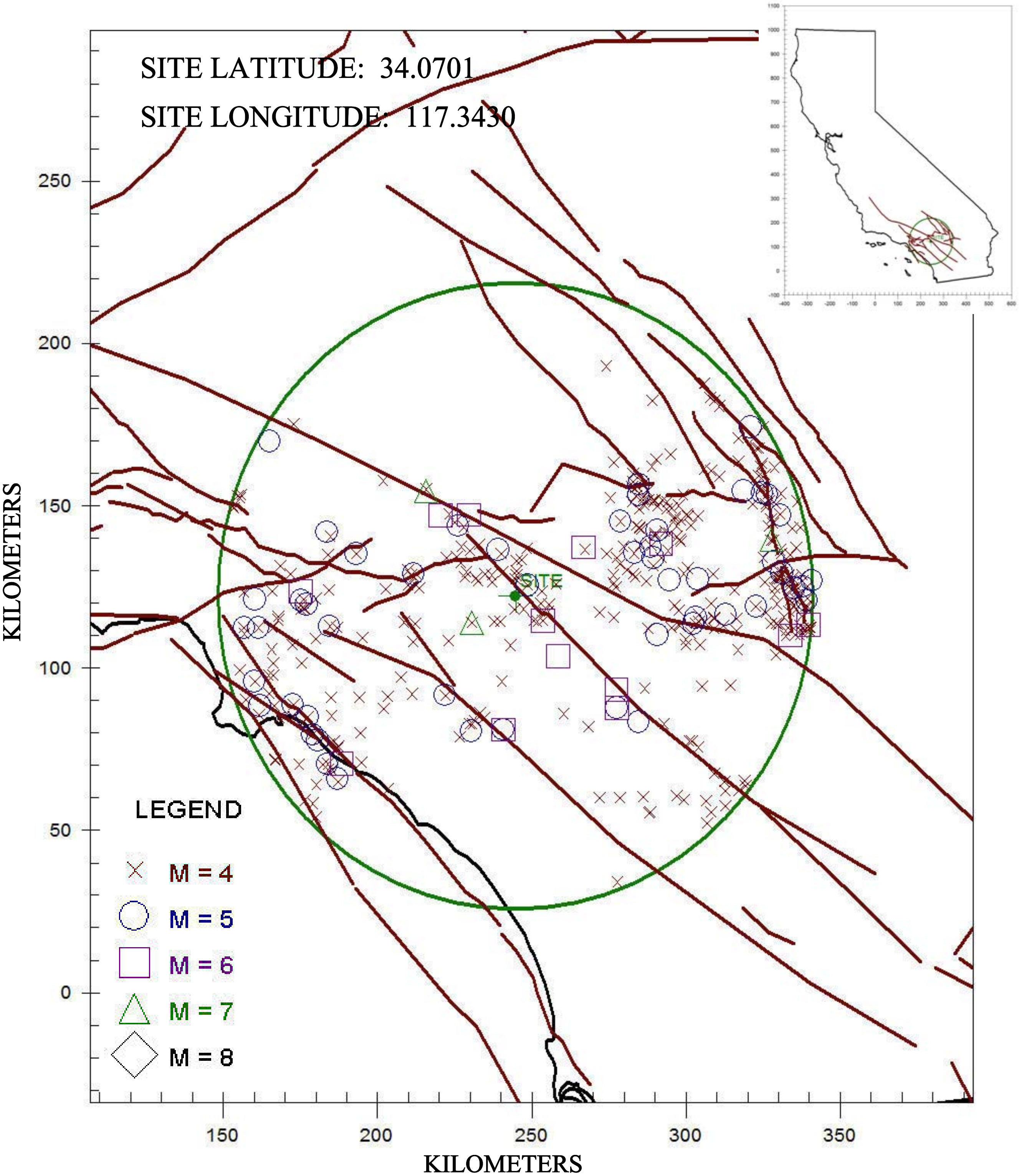
UNINCORPORATED SAN BERNARDINO COUNTY, CA

CHL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 3



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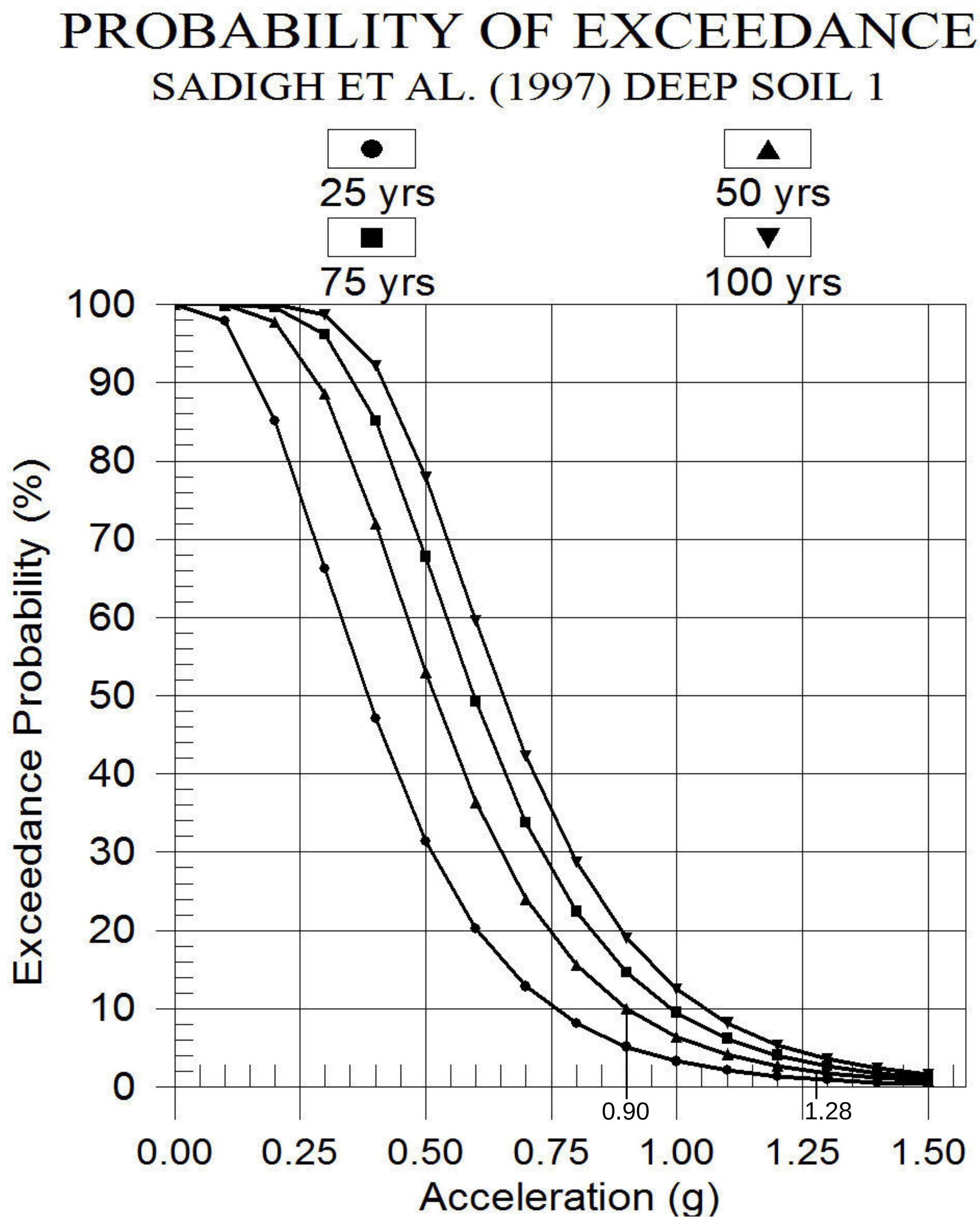
REGIONAL SEISMICITY MAP

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

CHL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 4



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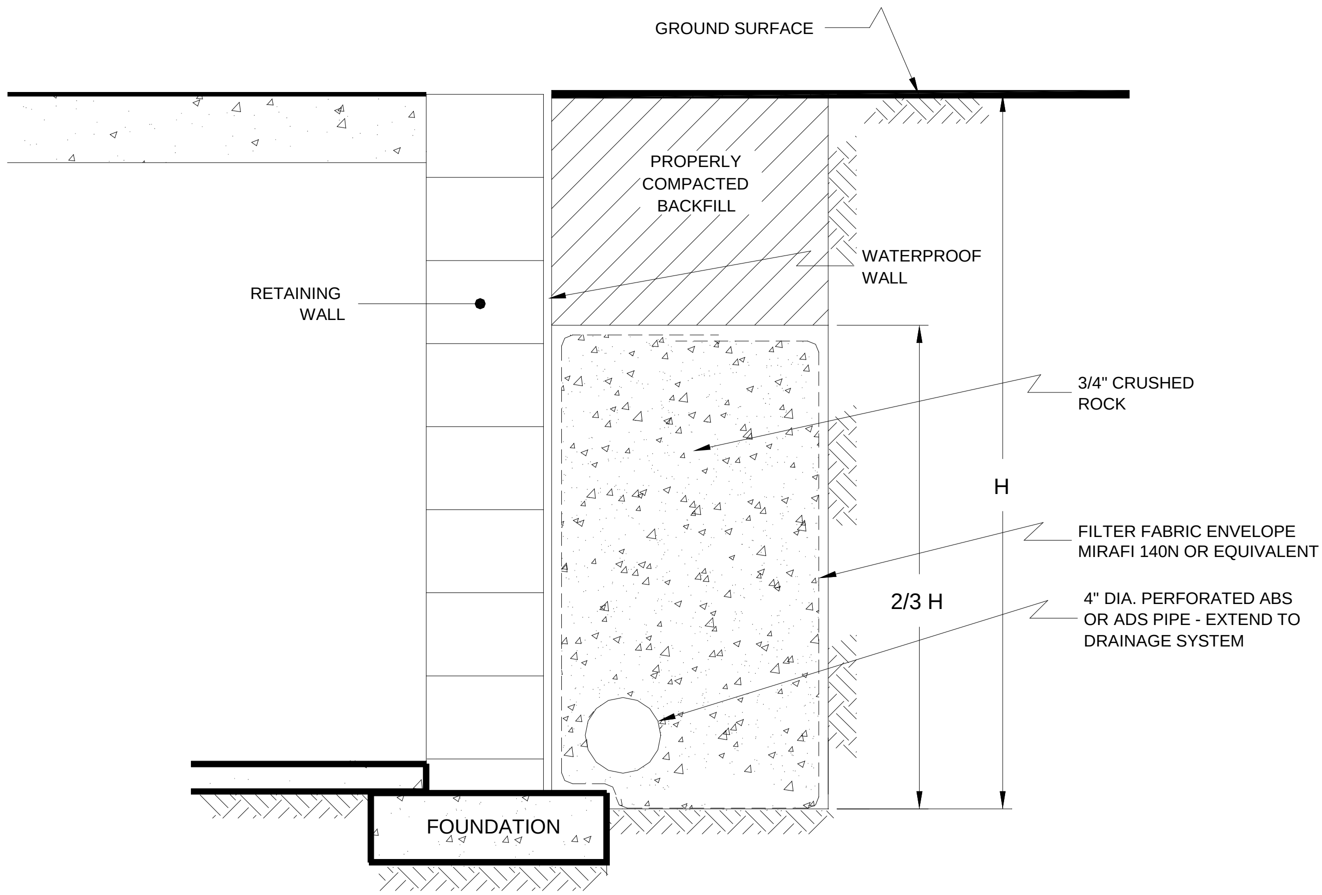
PROBABILITY OF EXCEEDANCE

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

CHL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 5



NO SCALE

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
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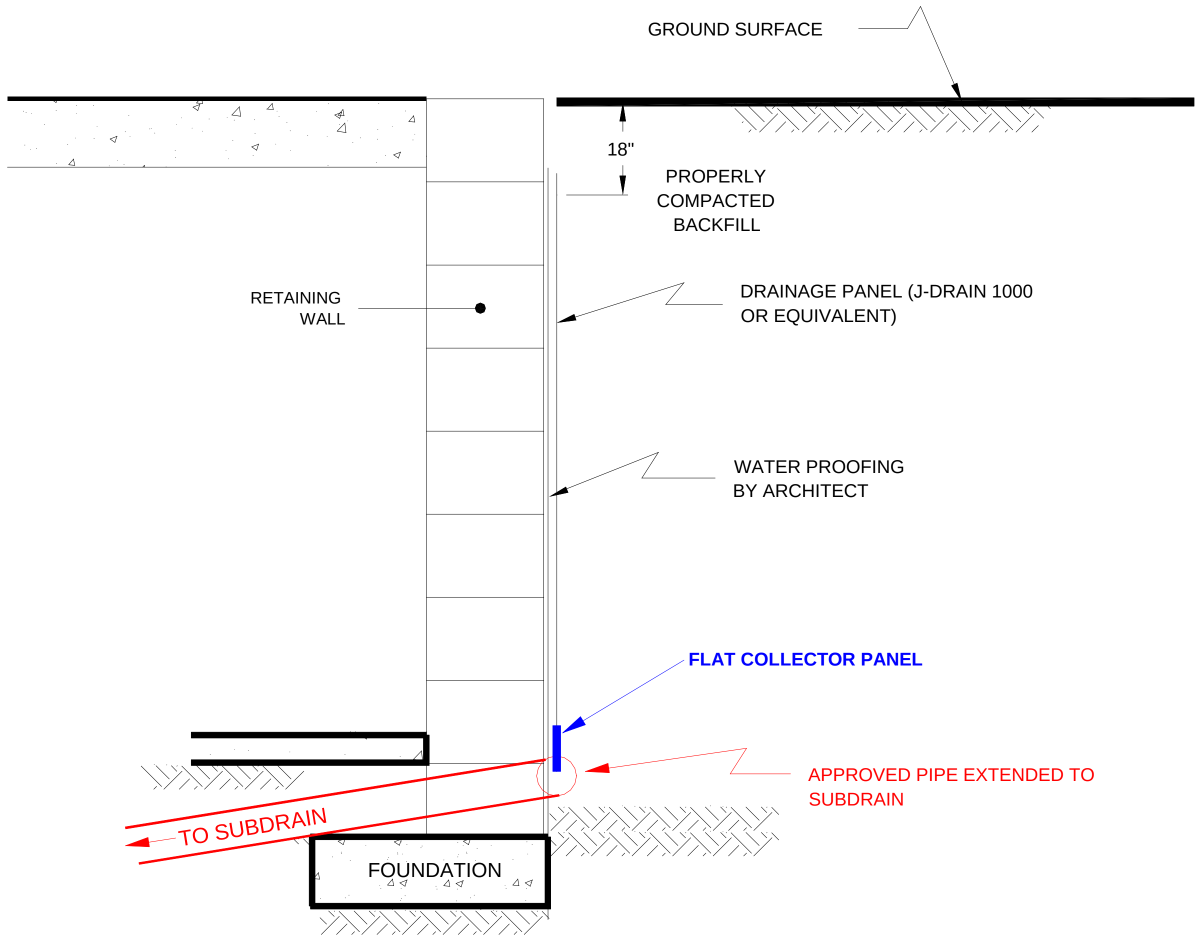
PHONE (818) 841-8388 - FAX (818) 841-1704

RK		8000

RETAINING WALL DRAIN DETAIL

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. 6



NO SCALE

GEOCON
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ENVIRONMENTAL GEOTECHNICAL MATERIALS
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PHONE (818) 841-8388 - FAX (818) 841-1704

NDB		8000

RETAINING WALL DRAIN DETAIL

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	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.
SAN JACINTO-SAN BERNARDINO	1.7	(2.8)	6.7	0.731	XI
SAN JACINTO-SAN JACINTO VALLEY	7.3	(11.7)	6.9	0.431	X
SAN ANDREAS - San Bernardino M-1	8.1	(13.1)	7.5	0.484	X
SAN ANDREAS - SB-Coach. M-2b	8.1	(13.1)	7.7	0.512	X
SAN ANDREAS - SB-Coach. M-1b-2	8.1	(13.1)	7.7	0.512	X
SAN ANDREAS - Whole M-1a	8.1	(13.1)	8.0	0.552	X
CUCAMONGA	9.5	(15.3)	6.9	0.466	X
CLEGHORN	14.4	(23.2)	6.5	0.225	IX
NORTH FRONTAL FAULT ZONE (West)	15.9	(25.6)	7.2	0.354	IX
SAN ANDREAS - 1857 Rupture M-2a	17.6	(28.3)	7.8	0.336	IX
SAN ANDREAS - Cho-Moj M-1b-1	17.6	(28.3)	7.8	0.336	IX
SAN ANDREAS - Mojave M-1c-3	17.6	(28.3)	7.4	0.280	IX
SAN JOSE	20.1	(32.3)	6.4	0.192	VIII
CHINO-CENTRAL AVE. (Elsinore)	20.3	(32.6)	6.7	0.223	IX
WHITTIER	21.9	(35.2)	6.8	0.168	VIII
ELSINORE (GLEN IVY)	22.3	(35.9)	6.8	0.165	VIII
SIERRA MADRE	22.8	(36.7)	7.2	0.255	IX
ELSINORE (TEMECULA)	29.5	(47.5)	6.8	0.120	VII
PUENTE HILLS BLIND THRUST	30.6	(49.2)	7.1	0.176	VIII
CLAMSHELL-SAWPIT	31.1	(50.0)	6.5	0.122	VII
NORTH FRONTAL FAULT ZONE (East)	33.2	(53.4)	6.7	0.126	VIII
SAN JACINTO-ANZA	33.4	(53.7)	7.2	0.132	VIII
HELENDALE - S. LOCKHARDT	34.1	(54.8)	7.3	0.138	VIII
PINTO MOUNTAIN	35.7	(57.4)	7.2	0.123	VII
SAN JOAQUIN HILLS	36.9	(59.4)	6.6	0.104	VII
RAYMOND	37.6	(60.5)	6.5	0.096	VII
UPPER ELYSIAN PARK BLIND THRUST	43.4	(69.8)	6.4	0.073	VII
VERDUGO	44.1	(70.9)	6.9	0.099	VII
LENWOOD-LOCKHART-OLD WOMAN SPRGS	45.0	(72.4)	7.5	0.115	VII
NEWPORT-INGLEWOOD (L.A.Basin)	46.0	(74.1)	7.1	0.084	VII
NEWPORT-INGLEWOOD (Offshore)	46.4	(74.7)	7.1	0.083	VII
JOHNSON VALLEY (Northern)	49.0	(78.9)	6.7	0.059	VI
HOLLYWOOD	50.9	(81.9)	6.4	0.059	VI
SAN ANDREAS - Coachella M-1c-5	51.0	(82.1)	7.2	0.079	VII
ELSINORE (JULIAN)	51.4	(82.8)	7.1	0.072	VII
LANDERS	52.6	(84.7)	7.3	0.082	VII
BURNT MTN.	53.3	(85.7)	6.5	0.046	VI
EUREKA PEAK	54.4	(87.5)	6.4	0.042	VI
EMERSON So. - COPPER MTN.	55.9	(89.9)	7.0	0.060	VI
SAN GABRIEL	56.3	(90.6)	7.2	0.070	VI
SIERRA MADRE (San Fernando)	56.3	(90.6)	6.7	0.063	VI
PALOS VERDES	56.7	(91.2)	7.3	0.074	VII

42 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE SAN JACINTO-SAN BERNARDINO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.7 MILES (2.8 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7306 g

APPENDIX A

FIELD INVESTIGATION

The site was initially explored on December 19, 2011 by excavating nine 7-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 5½ and 20½ feet below the existing ground surface. Percolation testing for the design of a stormwater infiltration system was performed in two of the borings. 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 a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by $2^3/_8$-inch brass sampler rings to facilitate removal and testing. Bulk samples were also obtained.

A supplemental site exploration was performed on January 28, 2013 by excavating four 4-inch diameter borings using manual hand auger equipment. The borings were advanced to depths between 4½ and 10½ feet below the ground surface. 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 a slide hammer. The California Modified Sampler was equipped with 1-inch by $2^3/_8$-inch brass sampler rings to facilitate removal and testing.

The soil conditions encountered in the borings 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-13. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are depicted on the Site Plan, Figure 2.

PROJECT NO. A8898-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, light brown, fine-grained with trace medium-grained			
2	B1@2.5'				**OLDER ALLUVIUM** Silty Sand, medium dense, slightly moist, light yellowish brown, fine-grained with trace medium-grained	45	114.1	3.1
	B1@5'					44	106.2	2.4
8	B1@7.5'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, light yellowish brown, fine-grained with trace medium- to coarse-grained	23	113.9	1.5
10	B1@10'			SP	Sand, poorly graded, dense, dry, yellowish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	55	113.3	1.9
12	B1@12.5'				-Very dense, pale brown to light yellowish brown	50 (5")	121.3	1.8
	B1@15'				-Dense, pale brown	85	138.0	2.5
20	B1@20'				-Very dense, fine-grained with some medium- to coarse-grained, trace fine-gravel	50 (3")	108.4	2.0

End at 20.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

A8898-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 1

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

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

GEOCON

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL**			
	B2@1'				Silty Sand, medium dense, dry, yellowish brown, fine-grained with trace medium-grained	21	98.6	4.1
2					**OLDER ALLUVIUM**			
	B2@3'				Silty Sand, medium dense, dry, light yellowish brown, fine-grained	35	95.6	3.4
4								
	B2@5'			SM		30	103.8	3.6
6								
	B2@7'				-Fine-grained with trace coarse-grained	31	120.6	3.3
8								
	B2@9'				Sand, poorly graded, medium dense, dry, yellowish brown, fine- to medium-grained with some coarse-grained, trace fine-gravel	38	105.9	1.9
10								
12				SP				
	B2@13'				-Dense, trace fine- to coarse-gravel	81	112.9	1.4
14								
	B2@15'					62	131.0	1.5

End at 15.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A2,
Log of Boring 2, Page 1 of 1

A8898-06-01 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.

PROJECT NO. A8898-06-01

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, dark brown, fine-grained with trace medium-grained			
2	B3@2'					26	97.0	4.2
4	B3@4'			SM	**OLDER ALLUVIUM** Silty Sand, medium dense, dry, light yellowish brown, fine-grained	25	89.6	2.0
6	B3@6'				-Fine-grained with trace medium-grained	31	101.7	3.4
8	B3@8'			SP	Sand, poorly graded, medium dense, dry, brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel	24	116.1	2.6
10	B3@10'					21	112.4	2.9
12	B3@12'				-Dense, dark yellowish brown, fine-grained with trace medium- to coarse-grained	68	125.6	1.7
14								
16								
	B3@17'				-Yellowish brown to pale brown, trace fine- to coarse-gravel	54	126.0	1.2
18								
20	B3@20'			SM	Silty Sand, dense, dry, yellowish brown to pale brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel	74	129.9	2.6

End at 20.5 feet.
Artificial fill to 3.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

A8898-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

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. A8898-06-01

BORING 4

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 4.5	B4@2.5'			SM	**OLDER ALLUVIUM** Silty Sand, loose, slightly moist, light yellowish brown, fine-grained	11	93.1	4.7
4.5 – 7	B4@5'			ML	Silt with Sand, firm, slightly moist, dark yellowish brown, fine-grained, low plasticity	13	118.6	8.2
7 – 9.5	B4@7.5'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, reddish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	36	133.4	2.0
9.5 – 15	B4@10'			SP	Sand, poorly graded, medium dense, dry, yellowish brown, fine- to medium-grained with trace coarse-grained, trace fine- to coarse-gravel	35	120.7	3.0

End at 15 feet.
No artificial fill encountered.
No groundwater encountered.
Percolation testing conducted on 12/20/11.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

A8898-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

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

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GEOCON

BORING 5

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Silt, very soft, wet, brown, fine-grained			
2	B5@2.5'					3	107.0	17.2
4								
	B5@5'				**OLDER ALLUVIUM** Sandy Silt, very soft, wet, brown, fine-grained	5	107.0	15.0
6								
8	B5@7.5'			ML		5	113.7	13.5
10	B5@10'			SP-SM	Sand with Silt, poorly graded, medium dense, wet, brown, fine-grained with trace medium-grained	20	107.8	7.4

End at 10.5 feet.
Artificial fill to 4.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A5,
Log of Boring 5, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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BORING 6

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'			ML	**OLDER ALLUVIUM** Sandy Silt, stiff, dry, light yellowish brown, fine-grained with trace medium-grained			
	B6@1'					22	99.1	3.0
2								
	B6@3'				-Firm	17	115.0	3.3
4								
	B6@5'					16	107.2	4.4
6								
	B6@7'				-Stiff, reddish brown	22	91.1	6.6
8								
	B6@9'			SP-SM	Sand with Silt, poorly graded, loose, dry, reddish brown, fine-grained with trace medium-grained	17	111.6	3.1
10								
12	B6@12'			SP	Sand, poorly graded, dense, slightly moist, yellowish brown, fine- to coarse-grained, trace fine- to coarse-gravel	68	138.0	2.3
14								
	B6@15'				-Some fine- to coarse-gravel	80	129.4	2.4

End at 15.5 feet.
No artificial fill encountered.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

A8898-06-01 BORING LOGS.GPJ

Figure A6,
Log of Boring 6, Page 1 of 1

SAMPLE SYMBOLS

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

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PROJECT NO. A8898-06-01

BORING 7

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL**			
	B7@1'				Silty Sand, dense, dry, pale brown, fine- to medium-grained	59	94.9	3.5
2								
					OLDER ALLUVIUM			
	B7@3'				Silt with Sand, stiff, dry, yellowish brown, fine-grained	33	95.3	2.8
4								
	B7@5'				-Light brown	35	100.1	2.7
6								
	B7@7'				-Increase in sand content, yellowish brown	28	100.1	3.2
8				ML				
10	B7@10'					21	102.3	4.1
12								
14								
	B7@15'				Sand, poorly graded, medium dense, dry, olive brown, fine- to medium-grained with trace coarse-grained	48	125.6	1.9
16								
18				SP				
20	B7@20'				-Dense, fine- to coarse-grained, trace fine-gravel	84	127.7	2.6

End at 20.5 feet.
Artificial fill to 2.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A7,
Log of Boring 7, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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BORING 8

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, dense, dry, light yellowish brown, fine- to medium-grained with trace coarse-grained			
2	B8@2'					56	109.3	2.0
4	B8@4'			ML	**OLDER ALLUVIUM** Sandy Silt, stiff, dry, yellowish brown, fine-grained with trace medium-grained	24	100.2	2.9
6	B8@6'					33	98.9	2.9
8	B8@8'					34	109.7	2.8
10	B8@10'					44	108.0	3.2
12	B8@12'			SM	Silty Sand, medium dense, dry, yellowish brown, fine-grained with trace medium-grained	22	116.4	2.4
14	B8@14'			SP-SM	Sand with Silt, poorly graded, medium dense, dry, dark yellowish brown, fine-grained with trace fine-gravel	20	105.9	1.8

End at 15 feet.
Artificial fill to 4 feet.
No groundwater encountered.
Percolation testing conducted on 12/20/11.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A8,
Log of Boring 8, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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BORING 9

ELEV. (MSL.) - - DATE COMPLETED 12/19/11

EQUIPMENT HOLLOW STEM AUGER BY: CA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-2'				**ARTIFICIAL FILL** Silty Sand, medium dense, dry, light brown, fine-grained with trace medium-grained			
2								
4				ML	**OLDER ALLUVIUM** Silt with Sand, hard, dry, light brown to yellowish brown, fine-grained with trace medium-grained			
	B9@5'					56	107.1	4.1

End at 5.5 feet.
Artificial fill to 4 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A9,
Log of Boring 9, Page 1 of 1

A8898-06-01 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.

BORING 10

ELEV. (MSL.) - - DATE COMPLETED 1/28/13

EQUIPMENT HAND AUGER BY: RG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
2	B10@2'							
4					-Increase in silt content			
	B10@5'			SM				
6					-Increase in silt content			
					-Some coarse-grained sand, some fine-gravel			
8	B10@8'				-Decrease in silt content			
10	B10@10'							
					End at 10.5 feet. No artificial fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A10,
Log of Boring 10, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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PROJECT NO. A8898-06-01

BORING 11

ELEV. (MSL.) - - DATE COMPLETED 1/28/13

EQUIPMENT HAND AUGER BY: RG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
2	B11@2'							
					-Increase in silt content			
4	B11@4'							
6				SM				
	B11@7'							
8					-Decrease in silt content, some fine- to coarse-gravel			
10	B11@10'							
					End at 10.5 feet. No artificial fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A11,
Log of Boring 11, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 12 ELEV. (MSL.) - - DATE COMPLETED 1/28/13 EQUIPMENT HAND AUGER BY: RG	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark brown, fine- to coarse-grained, trace fine-gravel			
2	B12@2'				**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
4					-Increase in silt content			
6	B12@5'			SM				
8	B12@8'				-Decrease in silt content, some coarse-gravel, trace fine-gravel			
10	B12@10'							
					End at 10.5 feet. Artificial fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

A8898-06-01 BORING LOGS.GPJ

Figure A12,
Log of Boring 12, Page 1 of 1

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

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GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 13 ELEV. (MSL.) - - DATE COMPLETED 1/28/13 EQUIPMENT HAND AUGER BY: RG	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained with trace coarse-grained, trace fine-gravel			
2	B13@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained			
4	B13@4'							
					End at 4.5 feet. Artificial fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A13,
Log of Boring 13, Page 1 of 1

A8898-06-01 BORING LOGS.GPJ

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

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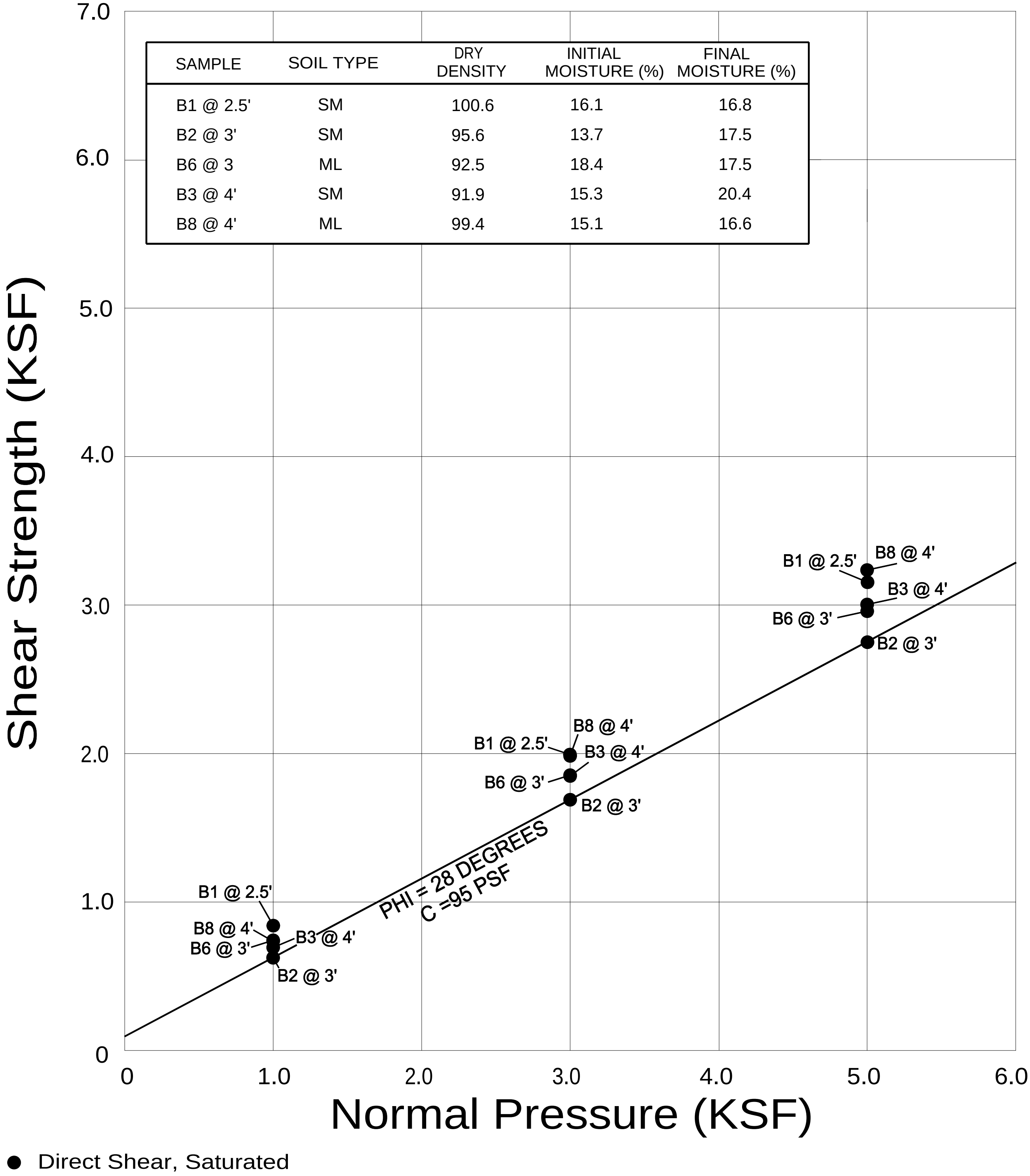
APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, compaction characteristics, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B20. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 2.5'	SM	100.6	16.1	16.8
B2 @ 3'	SM	95.6	13.7	17.5
B6 @ 3	ML	92.5	18.4	17.5
B3 @ 4'	SM	91.9	15.3	20.4
B8 @ 4'	ML	99.4	15.1	16.6

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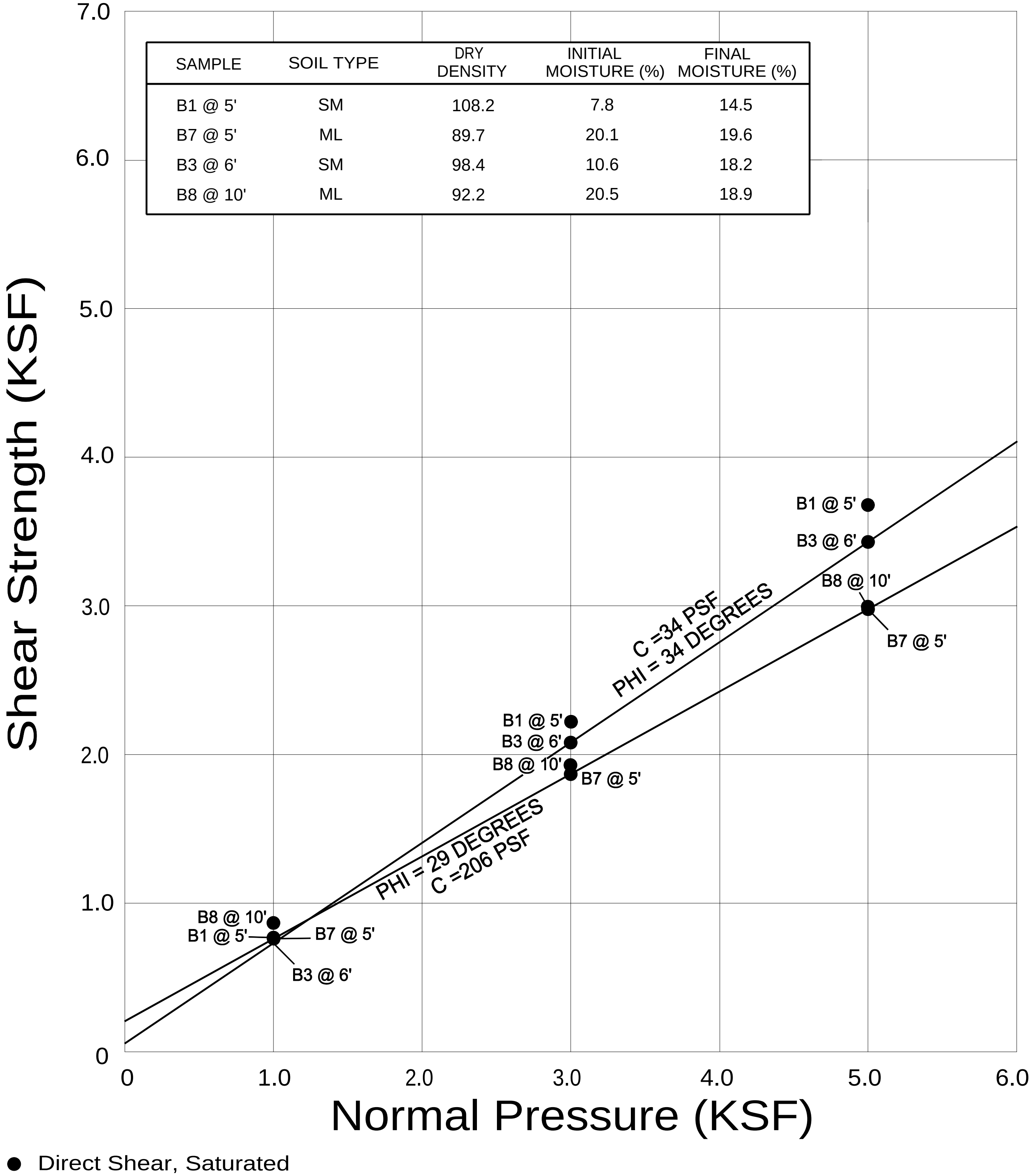
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	SM	108.2	7.8	14.5
B7 @ 5'	ML	89.7	20.1	19.6
B3 @ 6'	SM	98.4	10.6	18.2
B8 @ 10'	ML	92.2	20.5	18.9

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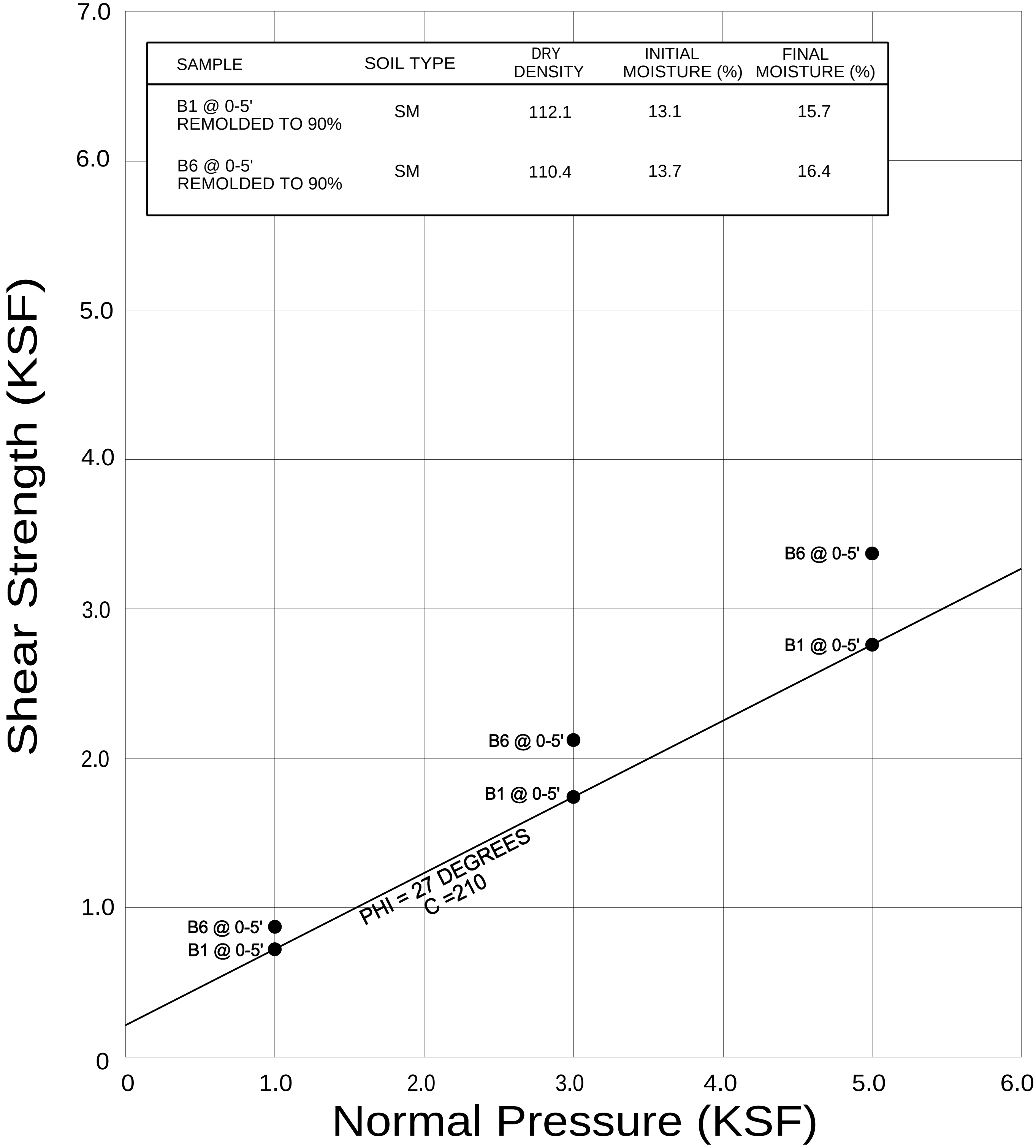
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B2

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 0-5' REMOLDED TO 90%	SM	112.1	13.1	15.7
B6 @ 0-5' REMOLDED TO 90%	SM	110.4	13.7	16.4



● Direct Shear, Saturated

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LABORATORY TEST RESULTS

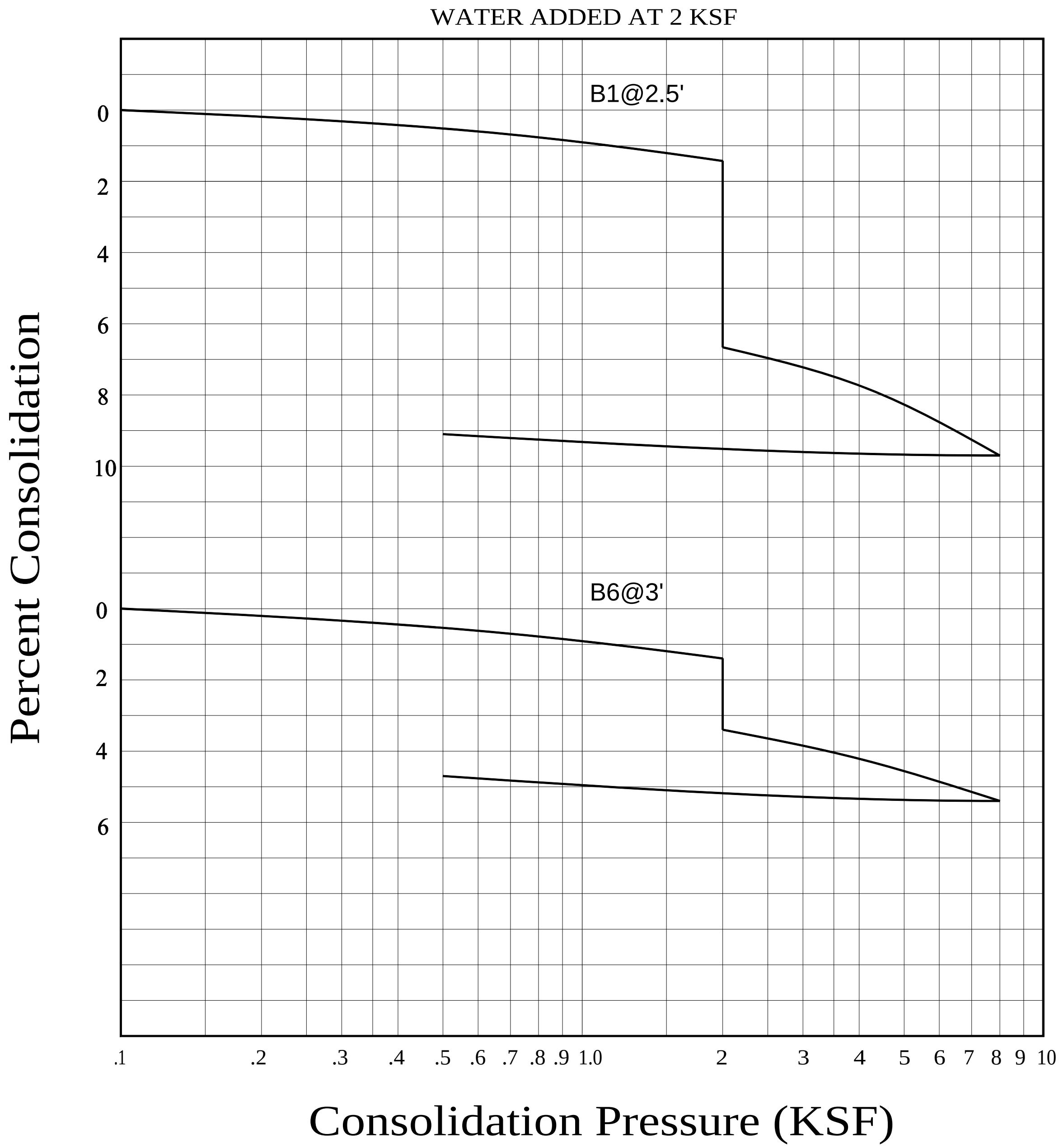
LAS TERRAZAS AT COLTON

AMCAL LAS TERRAZAS FUND, L.P.

275 & 291 CYPRESS AVENUE

UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B3



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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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275 & 291 CYPRESS AVENUE
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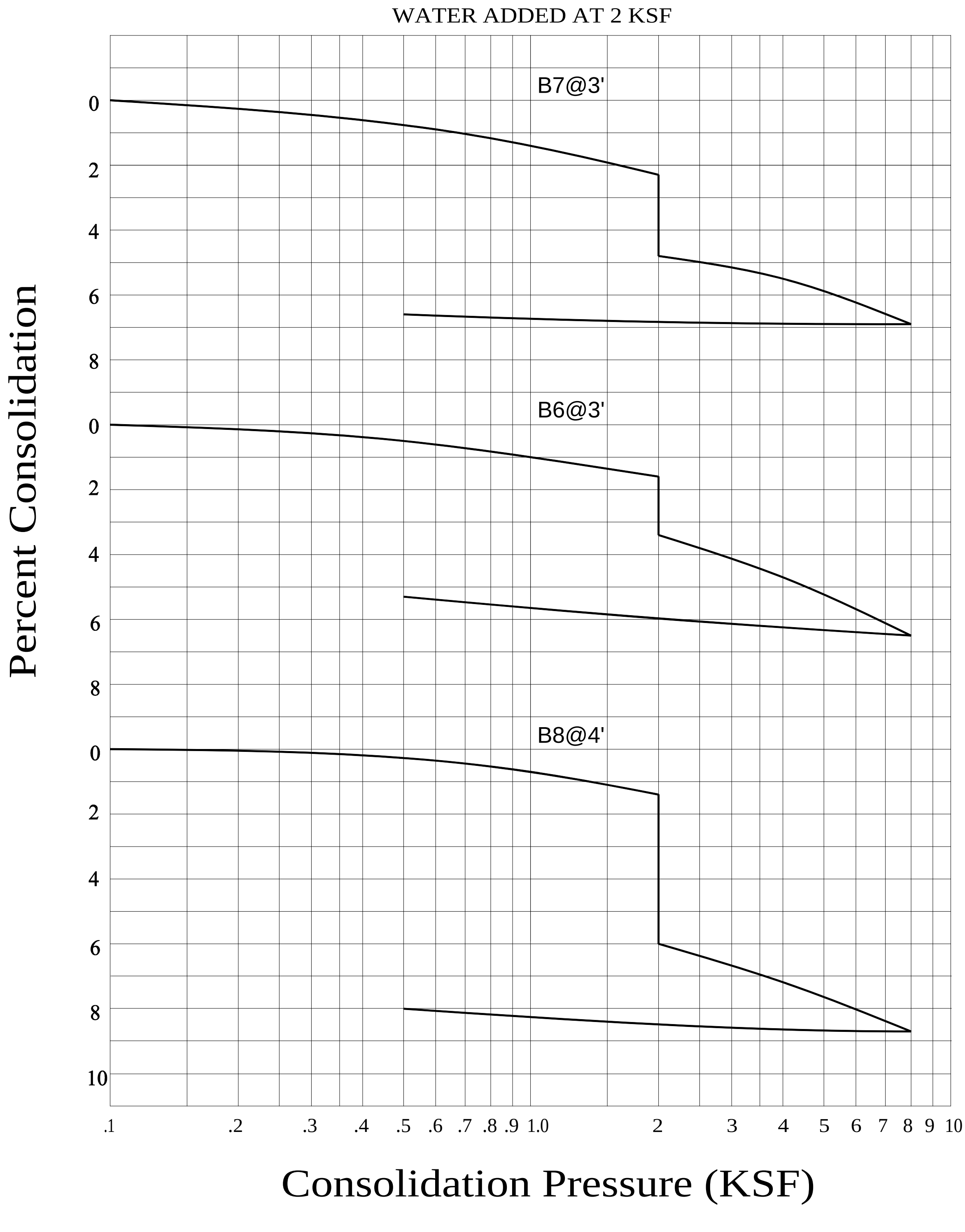
TL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B4



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TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
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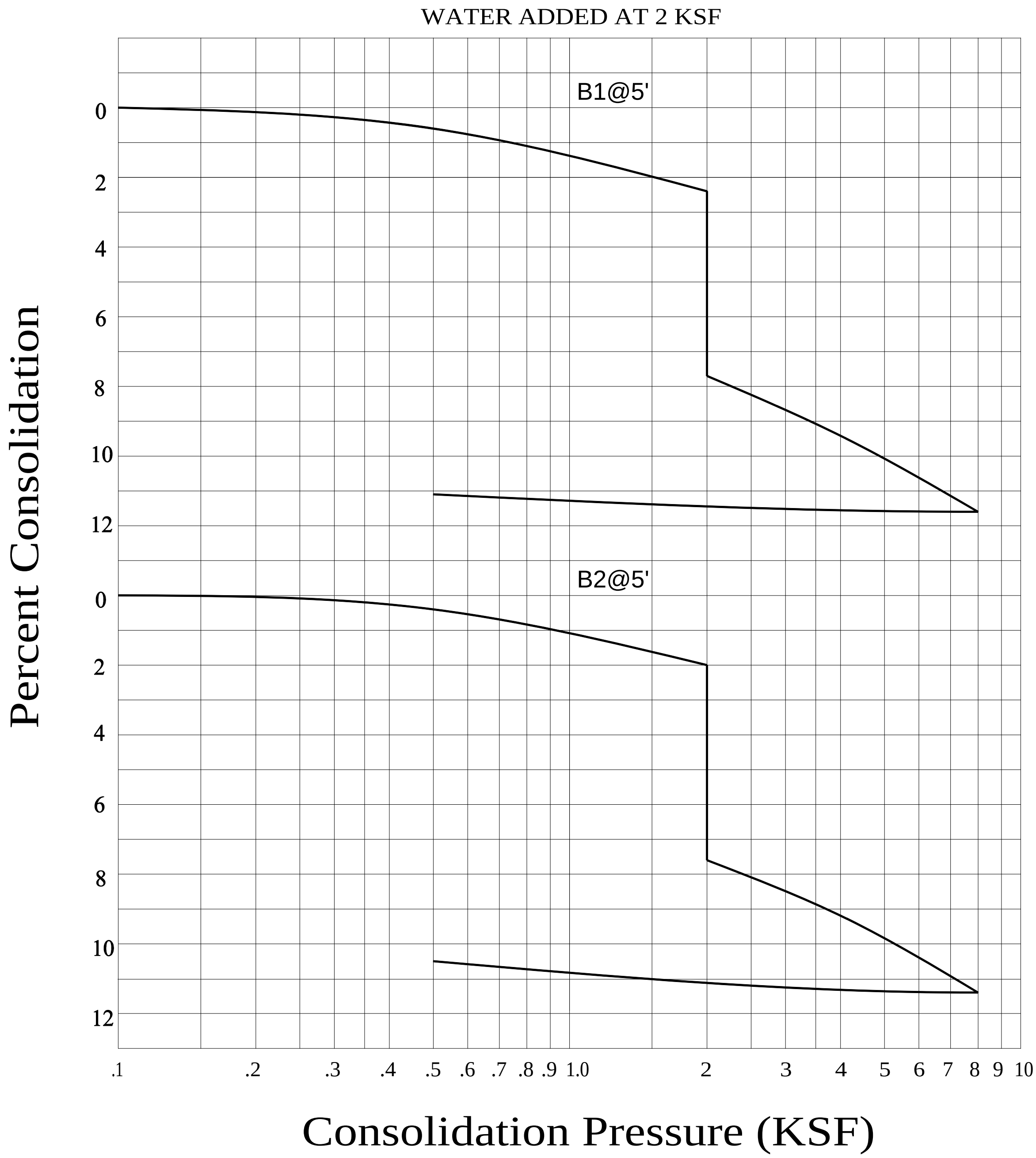
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B5



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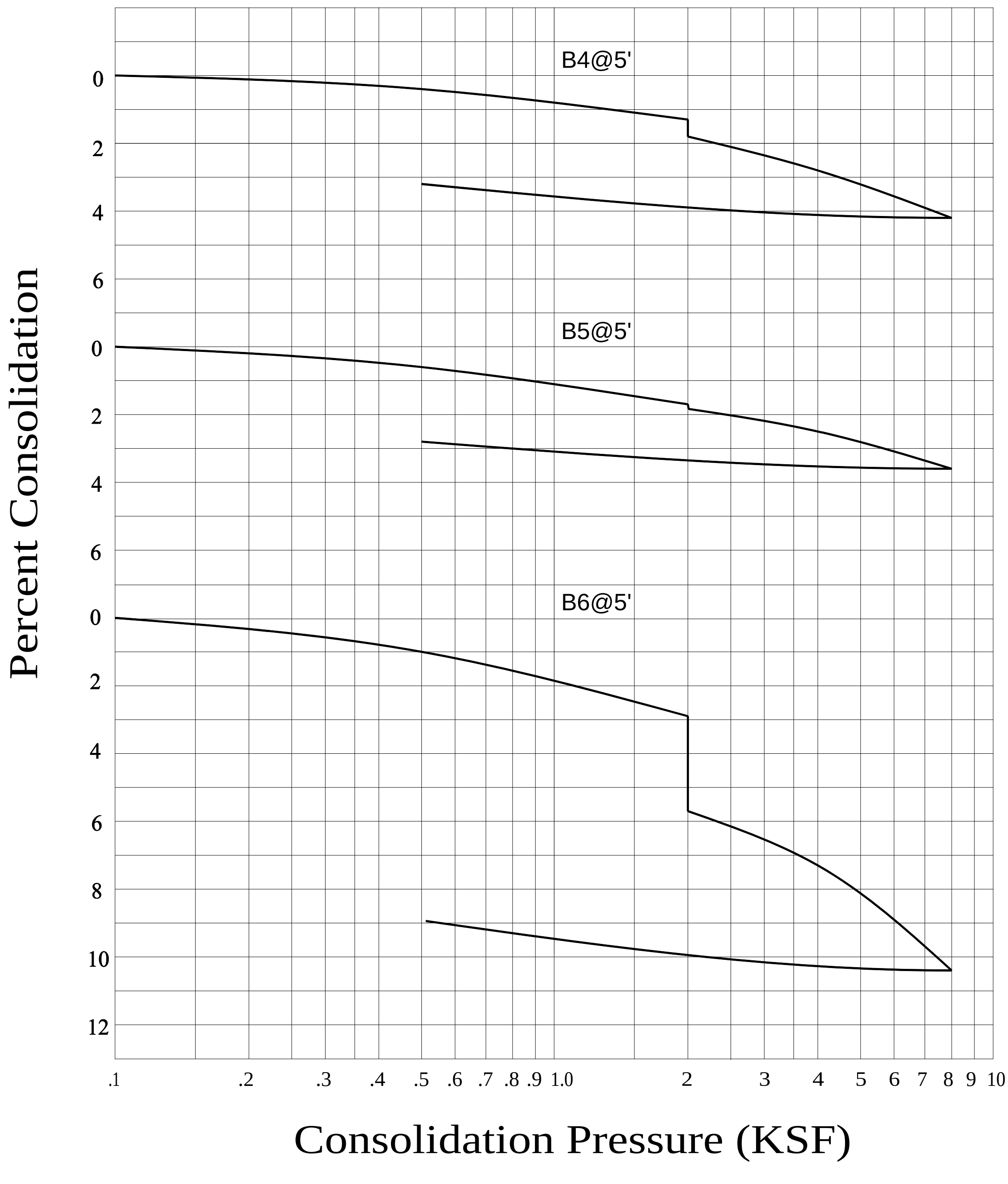
TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B6

WATER ADDED AT 2 KSF



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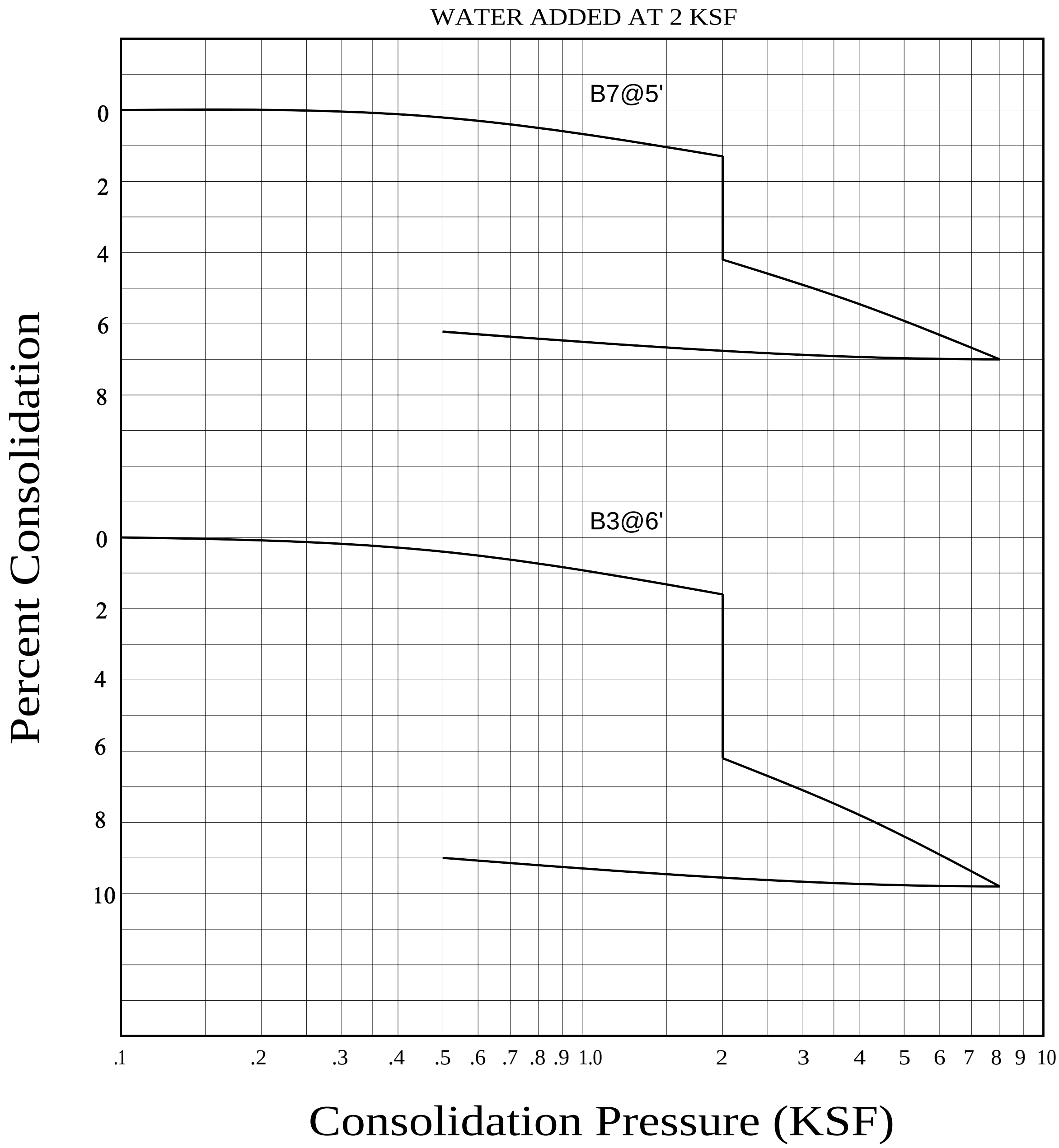
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B7



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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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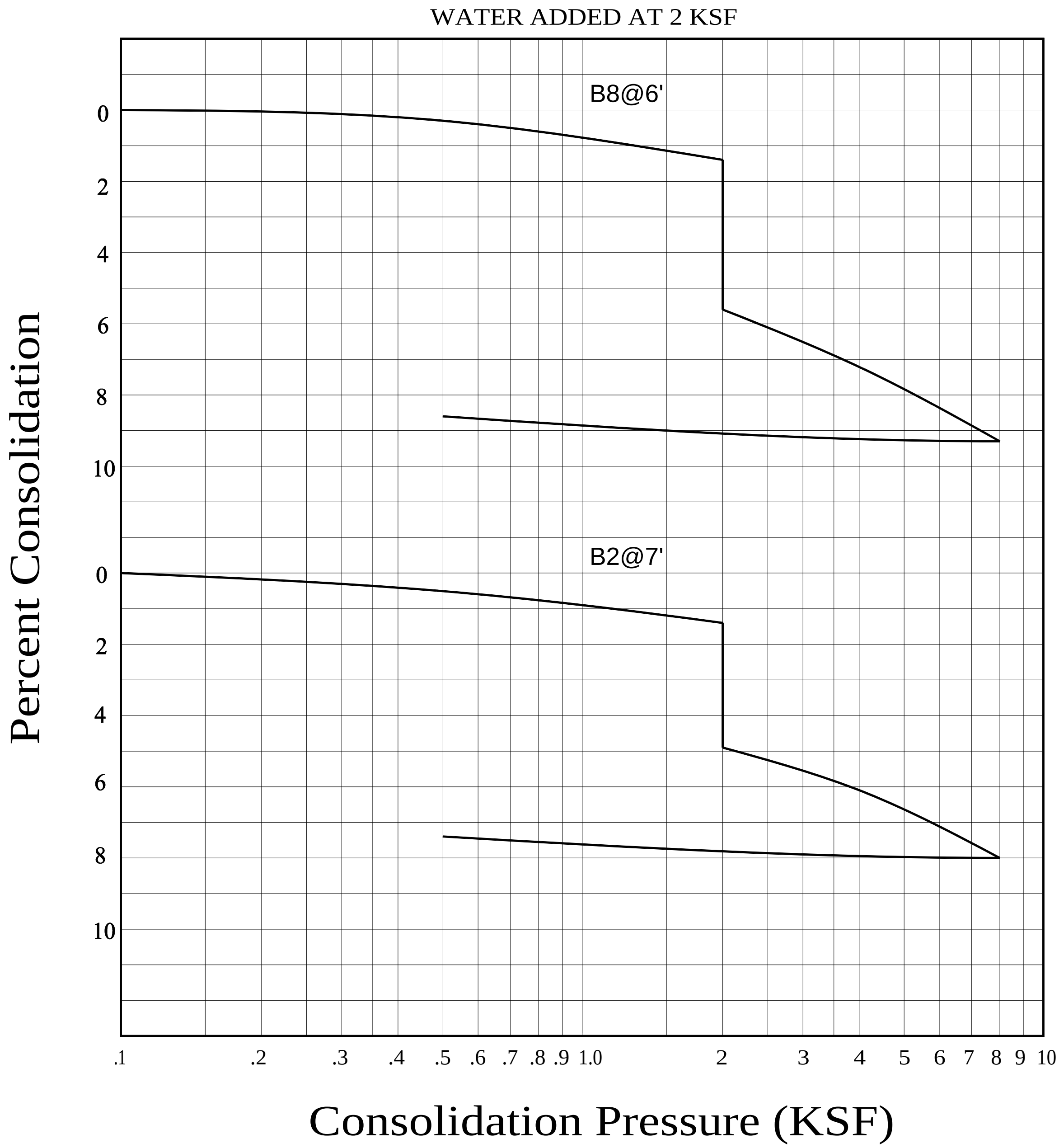
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B8



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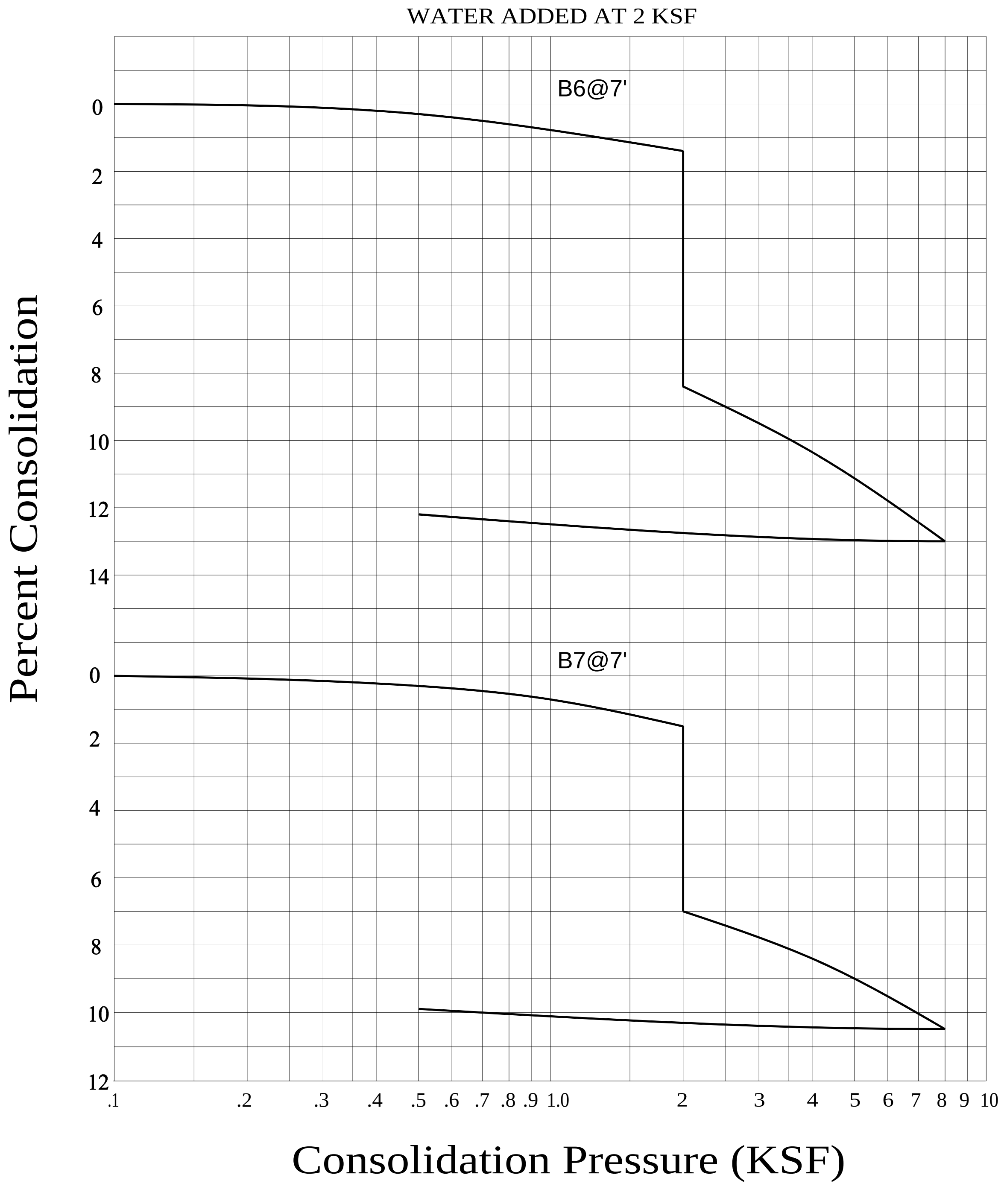
LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

TL		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B9



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TL		8000

LABORATORY TEST RESULTS

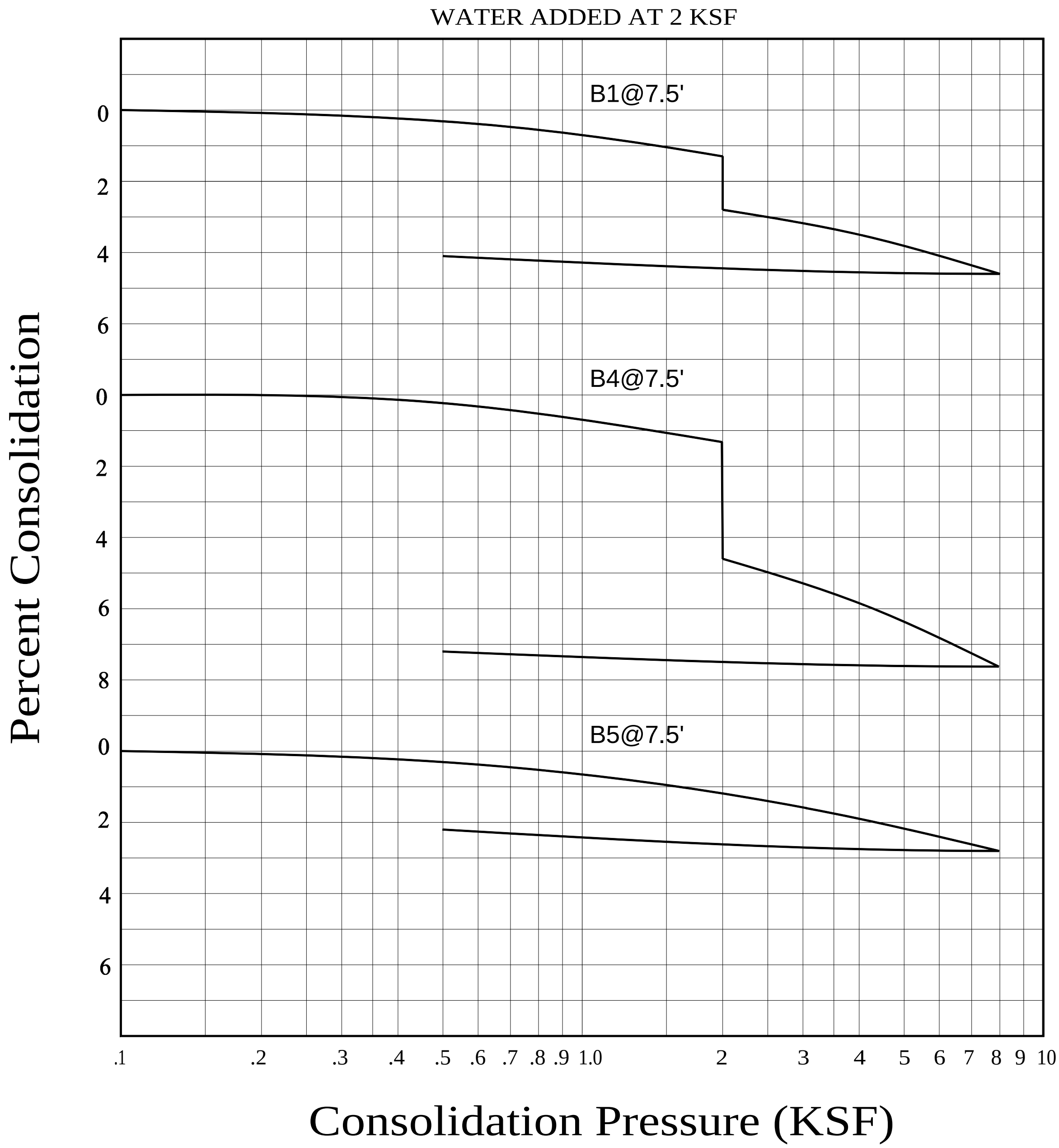
LAS TERRAZAS AT COLTON

AMCAL LAS TERRAZAS FUND, L.P.

275 & 291 CYPRESS AVENUE

UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B10



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TL		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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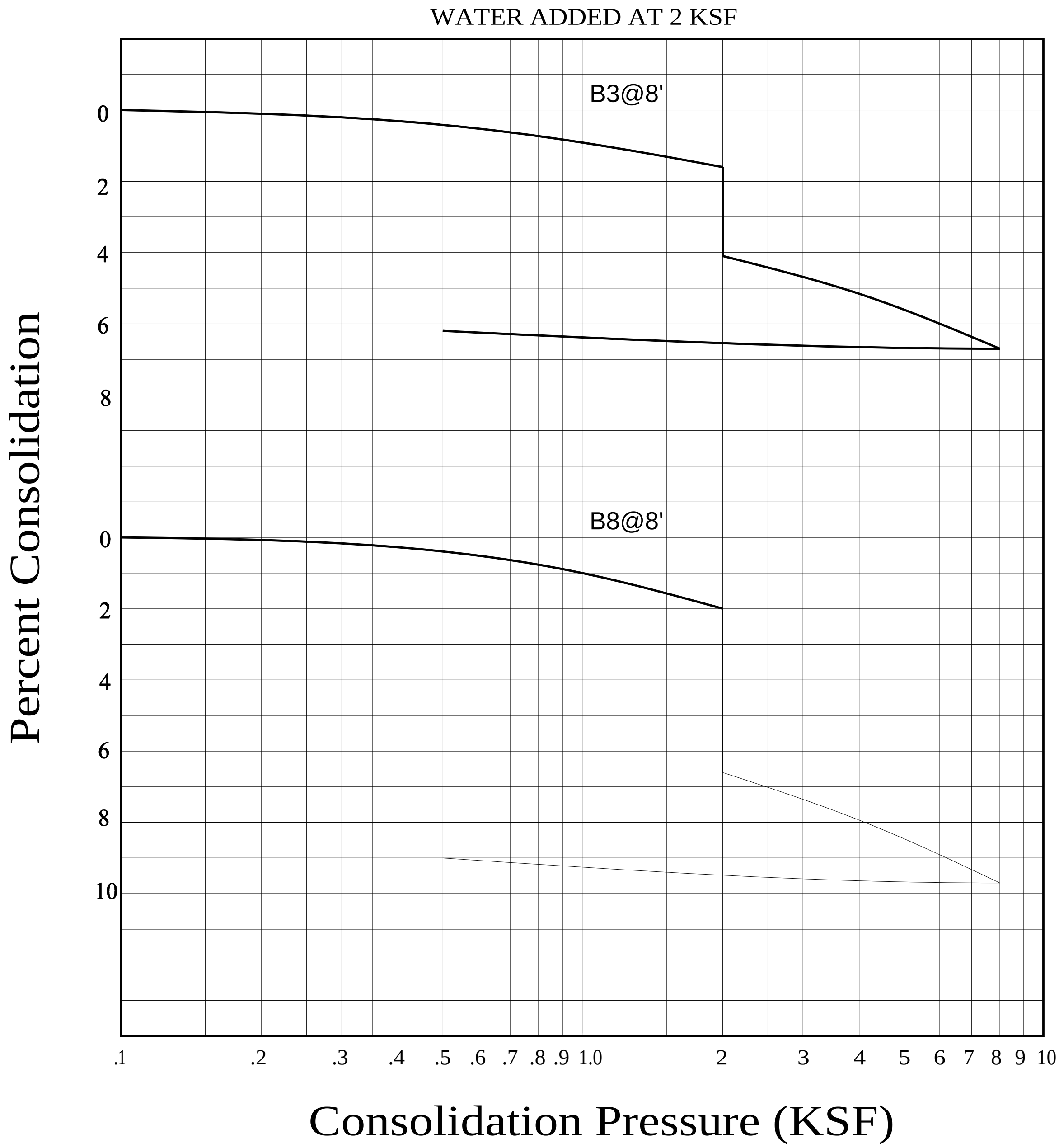
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B11



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TL		8000

LABORATORY TEST RESULTS

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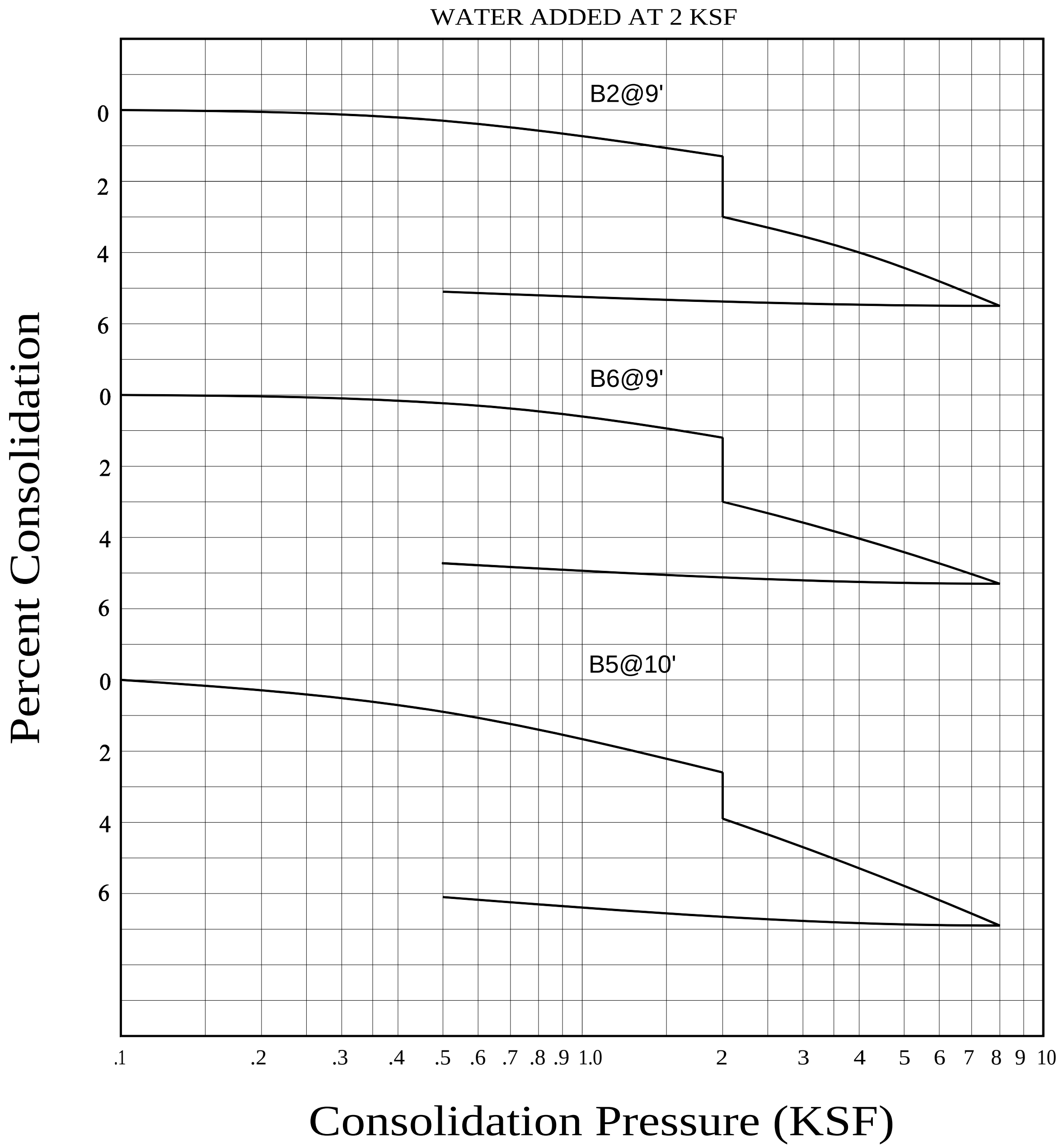
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B12



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LABORATORY TEST RESULTS

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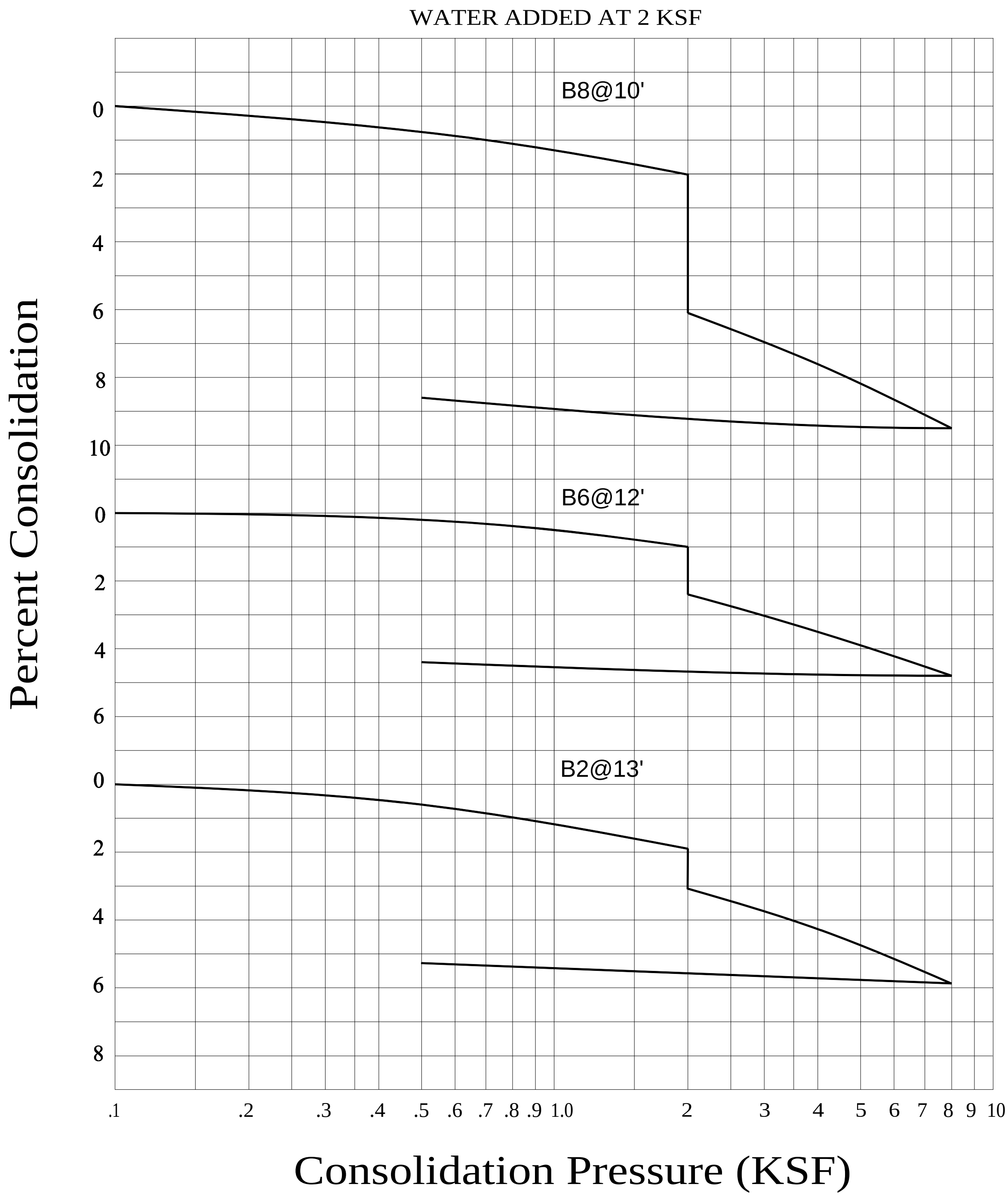
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B13



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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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275 & 291 CYPRESS AVENUE
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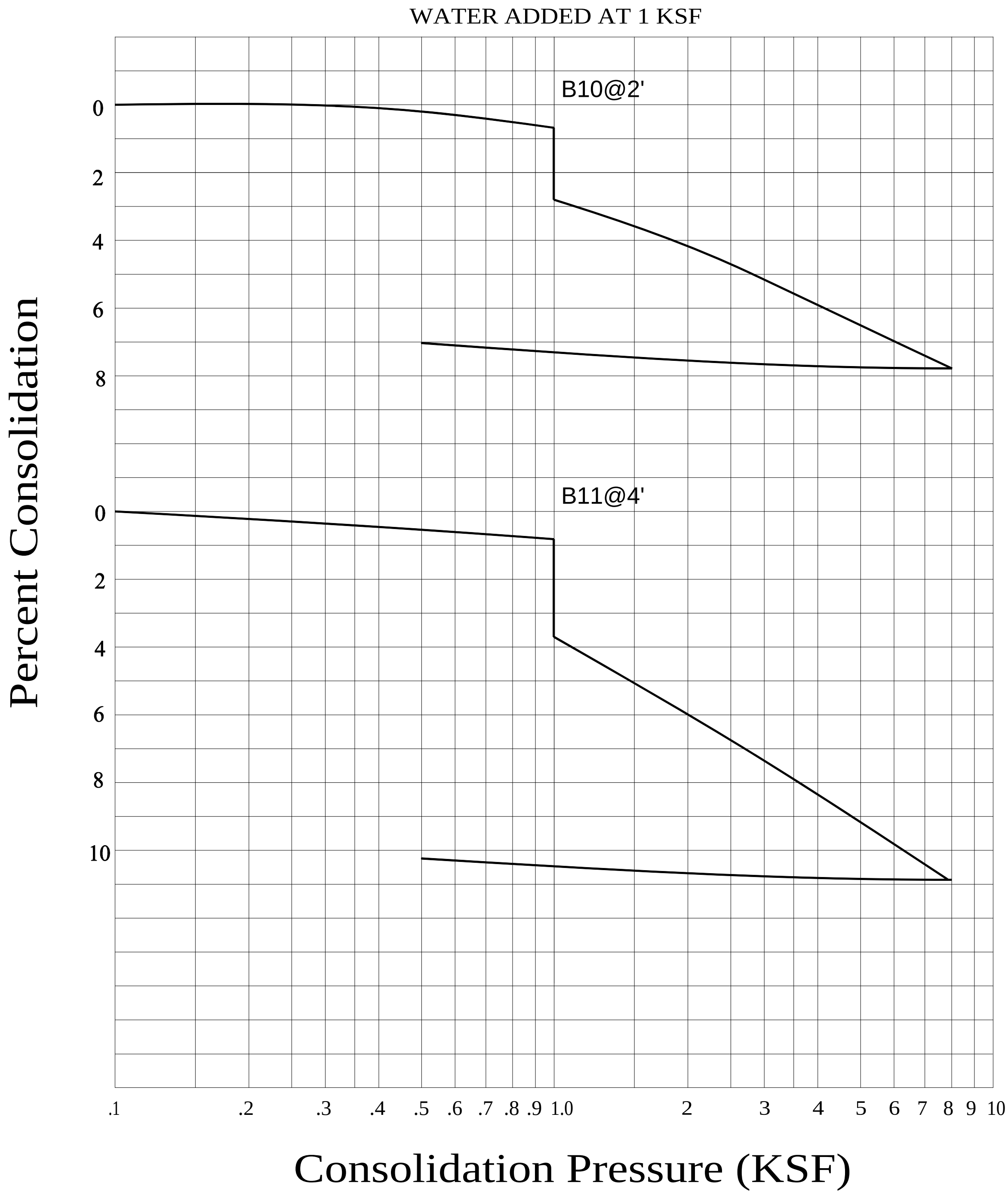
TL		8000

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B14



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JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON

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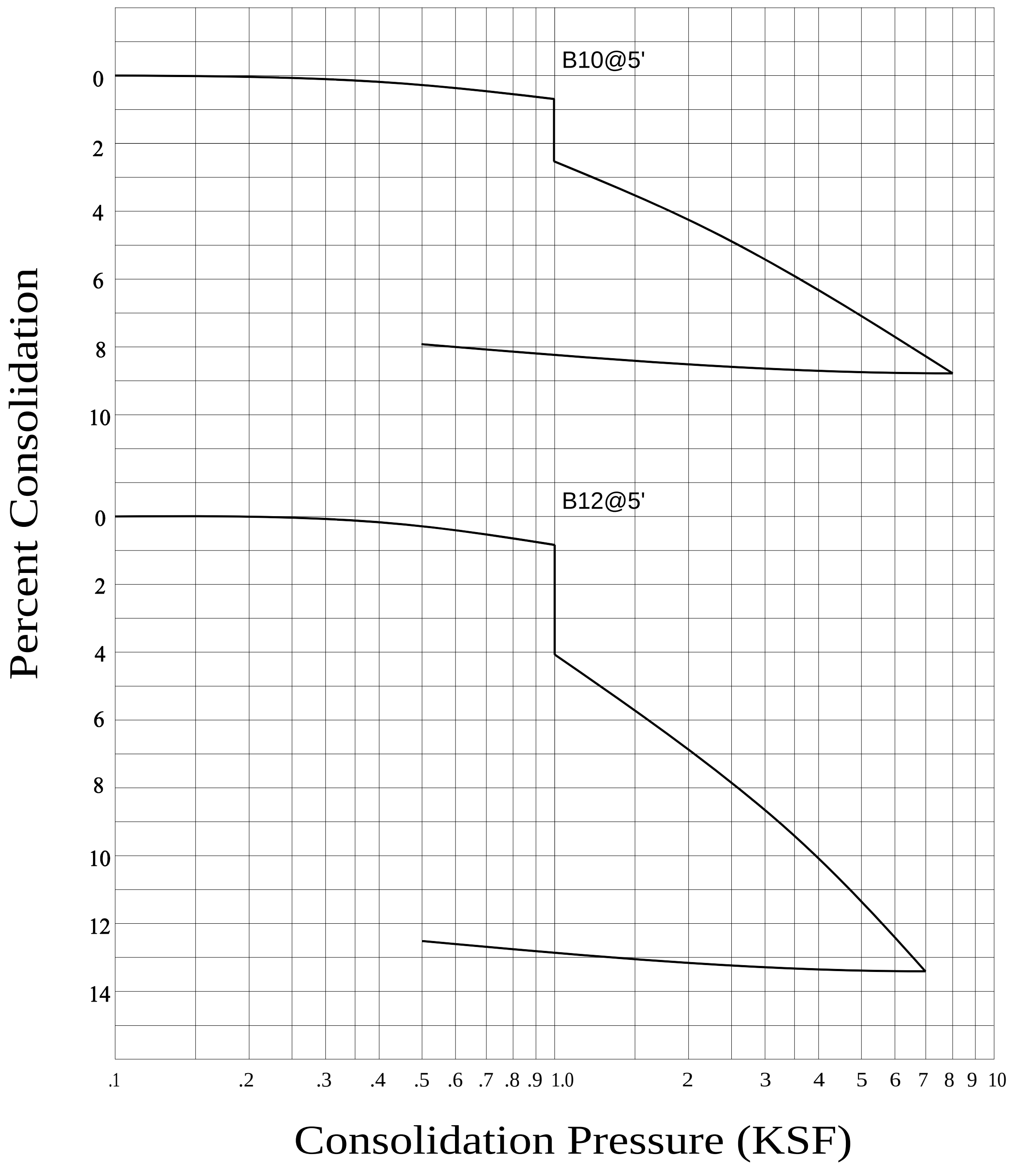
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B15

WATER ADDED AT 1 KSF



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JMT		8000

LABORATORY TEST RESULTS

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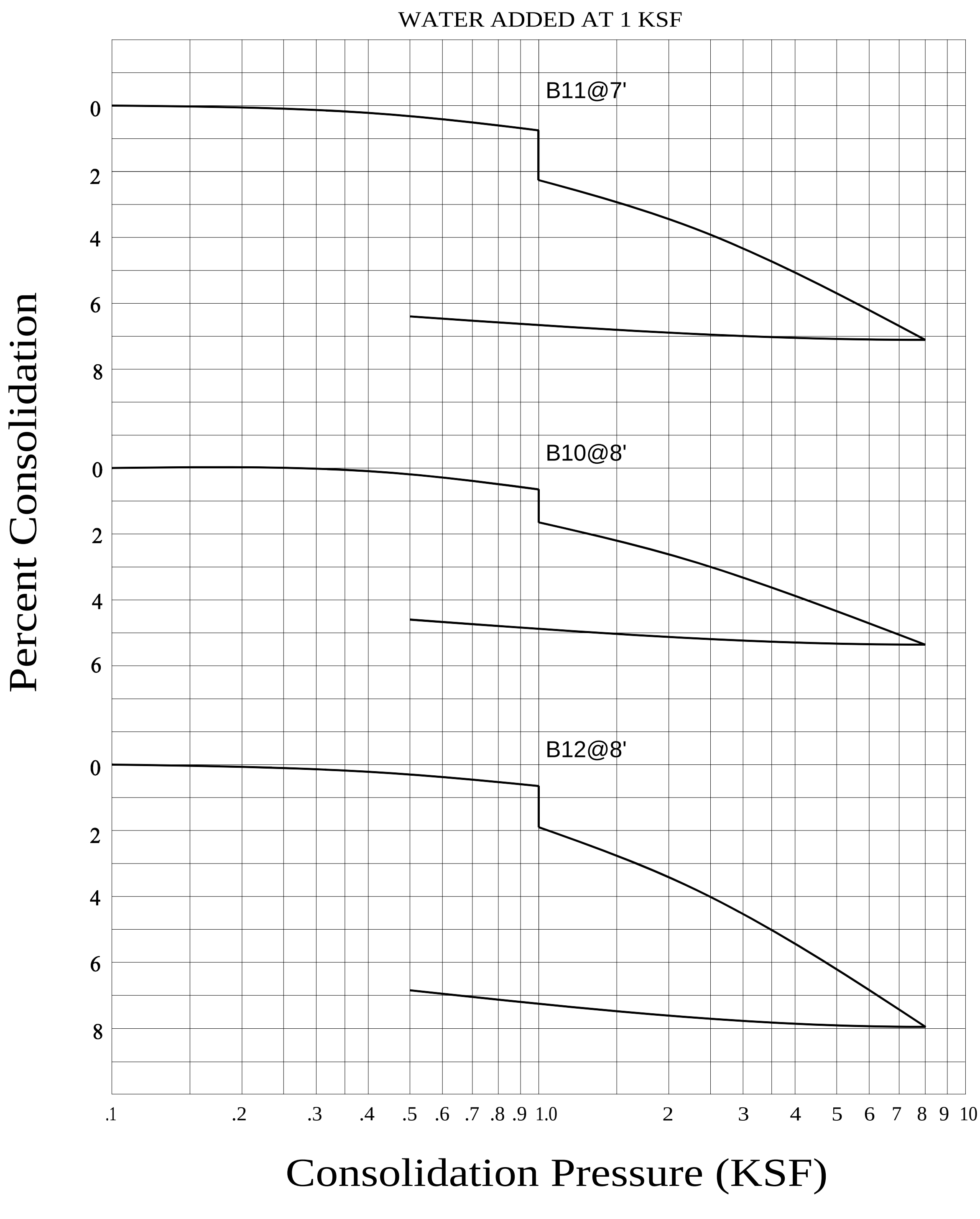
FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B16



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JMT		8000

LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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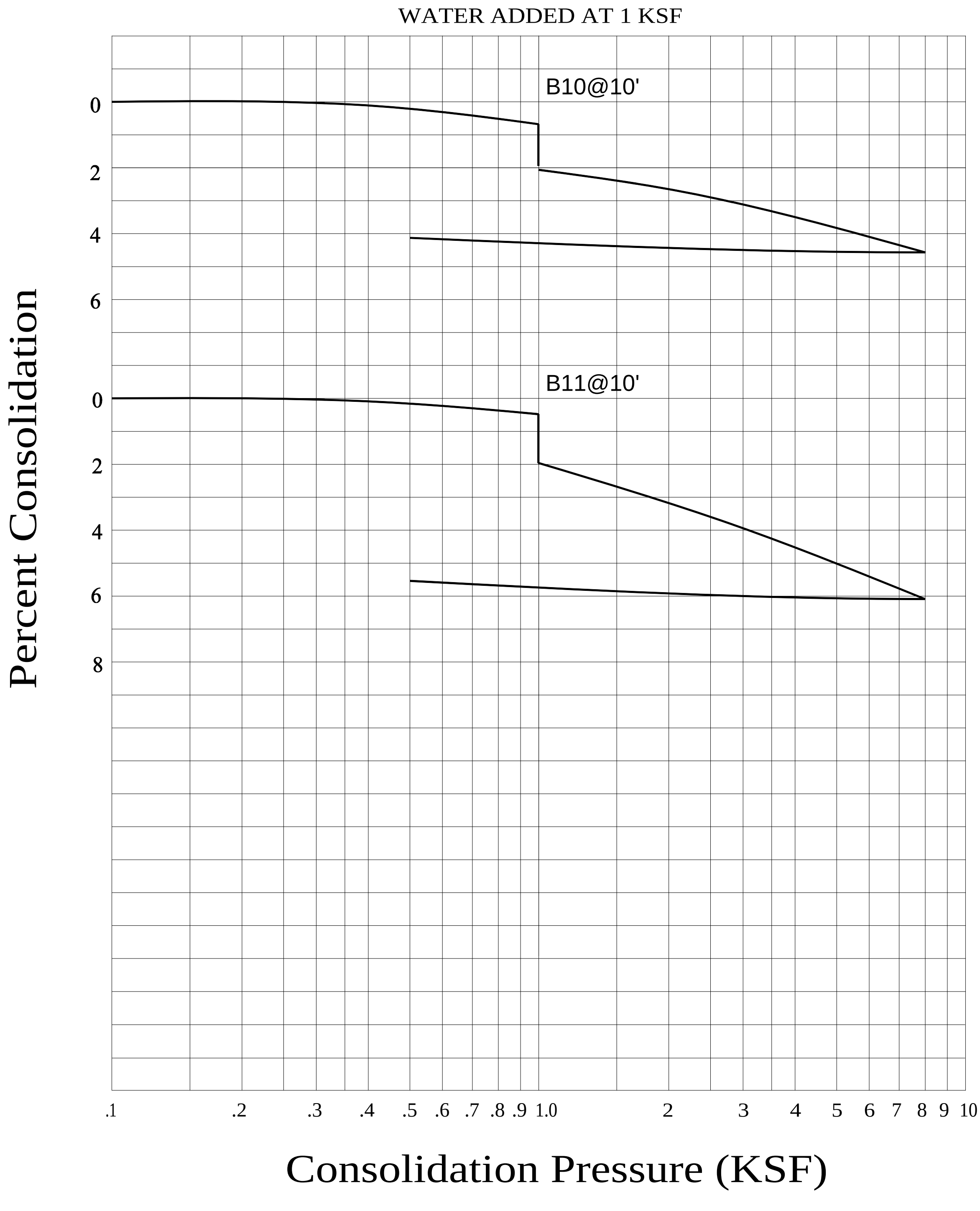
FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B17



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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON

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275 & 291 CYPRESS AVENUE

UNINCORPORATED SAN BERNARDINO COUNTY, CA

JMT		8000	FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B18

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 0-5'	11.2	14.6	116.4	9	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 1557-07

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 @ 0-5'	Light Yellowish Brown Silty Sand	128.0	8.5
B6 @ 0-5'	Light Yellowish Brown Silty Sand	128.0	9.0

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LABORATORY TEST RESULTS

LAS TERRAZAS AT COLTON
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FEB 20, 2013	PROJECT NO. A8898-06-01	FIG. B19

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0-5'	7.81	6300 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 0-5'	0.001

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 0-5'	0.010	Negligible

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

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CORROSIVITY TEST RESULTS

LAS TERRAZAS AT COLTON
AMCAL LAS TERRAZAS FUND, L.P.
275 & 291 CYPRESS AVENUE
UNINCORPORATED SAN BERNARDINO COUNTY, CA

FEB 20, 2013 | PROJECT NO. A8898-06-01 | FIG. B20

From: Gerry Kasman <kasman@geoconinc.com>
Sent: Friday, October 21, 2011 3:48 PM
To: Jay Ross
Subject: RE: San Bernardino Co. project: Valley/Cypress, uninc Colton

Jay,

The site is located in Unincorporated San Bernardino County. It is not in the State Alquist Priolo Earthquake Fault Zone for surface rupture. At its closest point to the site, the mapped trace of the active San Jacinto Fault is located ~1,170 feet to the northeast.

Gerry Kasman | *Senior Geologist / Associate*
Geocon West, Inc.
3303 N. San Fernando Blvd. Suite 100, Burbank, CA 91504
Tel 818.841.8388 Fax 818.841.1704 Cell 805.338-8600
www.geoconinc.com

From: Jay Ross [mailto:Jay@AmcalHousing.com]
Sent: Friday, October 21, 2011 10:23 AM
To: 'kasman@geoconinc.com'
Subject: San Bernardino Co. project: Valley/Cypress, uninc Colton

Gerry,

Here's a project in uninc Colton that we're looking at. It's the corner of Valley and Cypress Rd.

I can't tell if the project is in a County earthquake zone, because the map is poor marked and the demarcation lines for cities/counties are the same as for "county earthquake zones".

Can you figure this out?

One attachment is the County Geo Map, the other shows where our site is on the corner of Cypress/Valley, which is west of the 10/215 fwy interchange.

Thank you,

Jay Ross
Asst. Project Manager

AMCAL Multi-Housing, Inc.
30141 Agoura Rd., Ste. #100
Agoura Hills, CA 91301-4332
P: (818) 706-0694 x 128
F: (818) 706-3752
C: (818) 974-2843 (only call if I instruct you, it's usually turned off)
www.AmcalHousing.com

Thank you to AMCAL's partners for their generous donations to LifeSTEPS and a great a celebration and community fair on Sept. 10 to help our tenants with scholarships, rental assistance and other special programs.

Donations topped $110,000 with an AMCAL match of $25,000.