SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-95

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification
	Before	After			
B1 @ 0-3'	8.6	20.7	107.1	39	Low
B3 @ 9-12'	9.1	18.5	113.8	74	Medium
B3 @ 18-21'	13.2	24.1	101.2	89	Medium

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

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-02

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 @ 0-3'	Olive Brown Silty Sand	121.5	9.0
B3 @ 9-12'	Grey Clayey Sand	115.0	16.0
B3 @ 18-21'	Brown Sandy Silt	96.0	26.0

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

LABORATORY TEST RESULTS

VENTURE PROFESSIONAL CENTER
VENTURE CORPORATION
AGOURA HILLS, CALIFORNIA

APPENDIX C

APPENDIX C

PRIOR BORINGS AND LABORATORY TEST RESULTS BY AGS

GEOCON
INLAND EMPIRE, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL INVESTIGATION

PROPOSED COMMERCIAL DEVELOPMENT VENTURE PROFESSIONAL CENTER AGOURA HILLS 29508 ROADSIDE DRIVE AGOURA HILLS, CALIFORNIA

PREPARED FOR

VENTURE CORPORATION
LARKSPUR, CALIFORNIA

DECEMBER 19, 2006

PROJECT NO. A8487-06-01A

Project No. A8487-06-01A
December 19, 2006

VIA OVERNIGHT COURIER

Venture Corporation
123 East Sir Francis Drake Blvd., 3rd Floor
Larkspur, California 94939

Attention: Mr. Walter S. Hallanan, III

Subject: PROPOSED COMMERCIAL DEVELOPMENT
VENTURE PROFESSIONAL CENTER AGOURA HILLS
29508 ROADSIDE DRIVE
AGOURA HILLS, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Hallanan:

In accordance with your authorization of our proposal dated November 13, 2006, we have performed a geotechnical investigation for the proposed commercial development at 29508 Roadside Drive, in Agoura Hills, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of 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 INLAND EMPIRE, INC.

REGISTERED PROFESSIONAL ENGINEER
DANH T. TRAN
C 59705
EXP. 12/31/2007
CIVIL
STATE OF CALIFORNIA

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CEG 2251

REGISTERED GEOLOGIST
GERALD KASMAN
No. 2251
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STATE OF CALIFORNIA

DTT:NDB:GAK:am

(7) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a proposed commercial development located at 29508 Roadside Drive in Agoura Hills, California. The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the property and based on conditions encountered, provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of our investigation included a site reconnaissance, field explorations, laboratory testing, engineering analysis, review of existing available reports for the site, and the preparation of this report. Two reports by Advanced Geotechnical Services, Inc. (AGS), dated January 17, 2001 and September 18, 2001, as well as a topographic survey map prepared by Development Resource Consultants, were also reviewed. The field explorations were performed on November 27 and 28, 2006, and consisted of excavating four large diameter borings utilizing an eighteen inch diameter bucket auger type drilling machine and seven test pits utilizing a backhoe. The borings were conducted to depths between 16 and 24 feet below the existing ground surface and all borings encountered bedrock. The backhoe test pits were conducted to a depth of six feet below the existing ground surface. The approximate locations of the exploratory borings and test pits 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 soil properties. Appendix B presents a summary of the laboratory test results.

Relevant exploratory excavations and laboratory test results prepared by AGS are included in Appendix C.

The recommendations presented herein are based on analysis of the data obtained during this 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 bounded by Roadside Drive and the 101 Freeway on the north, a construction equipment rental facility on the east, the Los Angeles County Animal Shelter on the west, and by Agoura Road on the south. The subject property is located at 29508 Roadside Drive, in Agoura Hills, California. The site and surrounding topography is shown on Figure 1, Vicinity Map. The property is undeveloped, vacant land (see Site Plan, Figure 2).

The subject property is located in a historical stream drainage area. Several natural terraces are located throughout the property. Surface water drainage at the site appears to be by stream flow from the west, along existing channels to the center of the property, where an east-west trending concrete flood control structure has been constructed. Vegetation on the site consists of oak trees and shrubs located along Agoura Road and the interior of the site. The neighboring developments to the east and to the west consist primarily of on-grade commercial structures.

Information concerning the proposed project is conceptual at this time. It is our understanding that the project will consist of a new commercial center consisting of three, two-story buildings constructed at or near present grade. The buildings will range from 22,000 to 27,600 square feet in plan area.

Due to the preliminary nature of the design at this time, wall and column loads were not made available. It is estimated that column loads for the proposed structure will be up to 600 kips. Wall loads are estimated to be up to 3 kips per linear foot.

Once the design phase 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 Inland Empire, Inc. should be contacted to determine the necessity for review and possible revision of this report.

2.1 Previous Work

Advanced Geotechnical Services Inc. (AGS) performed a geotechnical investigation for a proposed Home Depot and restaurant development. The report is entitled Geotechnical Engineering Study, Proposed Home Depot and Restaurant Pad, Ladyface Village Phase I, Agoura Road West of Kanan, Agoura Hills, California, dated September 18, 2001. Within the area of the proposed development, 19 borings were drilled and 6 backhoe test pits were excavated to a maximum depth of 25½ feet beneath the existing ground surface. In addition, 6 Cone Penetrometer Tests (CPTs) were advanced to a maximum depth of 41 feet below ground surface. The prior AGS borings have been reviewed as part of this investigation and logs of the prior explorations are depicted on Figure 2 and included in Appendix C.

3. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of artificial fill, alluvium and Quaternary Age terrace deposits, overlying sedimentary bedrock units of the Tertiary Age Topanga Formation and Conejo Volcanics. Topanga Formation bedrock was encountered within 18 feet of the ground surface. Detailed stratigraphic profiles are provided on the Boring Logs in Appendix A (see Figures A-1 through A-11).

3.1 Artificial Fill

Artificial fill materials were encountered in numerous borings and test pits throughout the subject property. Artificial fill was observed in Test Pits 1, 3 and 4, ranging in depth from six to seven feet below ground surface. In addition, several of the prior explorations by AGS encountered fill materials to a maximum depth of 15 feet below the ground surface. The encountered fill material generally consists of yellowish brown clayey gravel with sand, and lesser amounts of clayey sand with gravel, some volcanic clasts, and fragments of concrete. Fill may have been placed as a part of a retention basin constructed between 1970 and 1976. The placement of fill has blocked surface flow of water onto the neighboring property to the east.

3.2 Native Topsoil and Colluvium (Qc)

Topsoil and colluvium were encountered during the site exploration performed by AGS. Topsoil was observed in Boring 27, 29 through 32, and Test Pits 10 and 11, to maximum depth of 7.5 feet below existing ground surface in Boring 32. Topsoil observed by AGS consisted of dark brown to dark grayish brown sandy clay or silt to silty or clayey sand. Fine grained materials were found to be stiff to hard, and coarse grained materials were loose to moderately dense.

3.3 Alluvium

Natural alluvial soils were observed in several explorations by both Geocon as well as AGS to a maximum depth of 19½ feet below the existing ground surface. The alluvium generally consists of sandy clay, silty sand, and sandy gravel, and lesser amounts of gravelly clay with sand. The alluvium was observed to be moist, medium dense to dense, and firm to stiff. Alluvial deposits were massive without internal structures or bedding and were associated with the stream channel area of the site.

3.4 Quaternary Terrace Deposits

Quaternary Age terrace deposits were encountered in Borings 1 through 4 to depths ranging from 13 feet to 18 feet below ground surface. The terrace deposits are typically up to 10 feet thick throughout the site and consist of gray to strong brown gravelly clay, with volcanic clasts that are firm and moist. Borings 1 through 4 are located at the highest elevations on the subject site and within an area where stockpiling activities have changed the original topography. Fill thickness is expected to increase to the west and southwest with decreasing thickness of terrace material. Approximate depths of terrace are indicated on Cross-Section A-A’ on the Site Plan (Figure 2).

3.5 Topanga Formation

The fill, alluvial soils and terrace are underlain by sedimentary units of the Tertiary Age Topanga Formation (Weber, 1984). As observed in the borings, the bedrock is yellowish brown to olive, thinly bedded siltstone and claystone with isolated thin interbeds of fine-grained sandstone that contain thin gypsum stringers and iron oxidation staining along bedding planes and joint surfaces. Topanga Formation was encountered in all four borings and the majority of the prior AGS explorations with minimum depths below ground surface ranging from 4 feet to 18 feet. Approximate depths to bedrock are indicated on Cross-Section A-A’ on the Site Plan (Figure 2).

3.6 Conejo Volcanics

Geocon did not encounter Tertiary Age Conejo Volcanics during the Site investigation; however, AGS presents information on this formation where it exists in the southern part of the subject property. According to AGS, the fill and alluvial deposits are underlain by Conejo Volcanics within the southern portion of the Site. Conejo Volcanics were encountered at 19.5 feet below ground surface on the subject property in AGS Boring B-4.

4. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation of the Thousand Oaks 7.5 Minute Quadrangle (California Division of Mines & Geology, 1998), the historically highest groundwater in the area is 10 feet beneath the ground surface and depicts the subject property as being located in an alluviated valley. Groundwater information presented in this document is generated from data collected in the early 1900's to present.

Groundwater seepage was not encountered during the field investigation performed by Geocon. Several borings excavated for the study performed by AGS encountered minor groundwater seepage. AGS indicated that these groundwater occurrences were highly variable and subject to local subsurface conditions. AGS encountered groundwater in Borings B-10 and B-13, at depths of 8 and 9 feet below the ground surface, respectively. It is our opinion that the groundwater encountered does not represent the static groundwater table, but rather exists in near surface discontinuous perched zones associated with sandy alluvium or structures built for flood control. The amount of seepage in these granular zones may fluctuate seasonally.

It is not uncommon for more shallow seepage conditions to develop where none previously existed, especially in heavily irrigated areas or after seasonal rainfall. Proper surface drainage of irrigation and precipitation should be maintained. Recommendations for drainage are provided in the *Surface Drainage* section of this report (see Section 6.13).

5. GEOLOGIC HAZARDS

5.1 Faulting

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, California Fault Map.

According to the "Maps of Known Active Fault Near Source Zones in California and Adjacent Portions of Nevada" (Feb. 1998), the nearest known active surface fault is the Malibu Coast Fault which is located approximately 5.4 miles, 8.7 kilometers, from the site.

The closest surface trace of an active fault to the site is the Malibu Coast Fault located approximately 7.4 miles south of the site (California Division of Mines & Geology, 1984). Other nearby active faults are the Simi fault, and the Northridge Hills fault located 9.3 miles north-northwest, and 13.8 miles north-northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault zone is located approximately 41 miles northeast of the site.

The closest potentially active fault to the site is the Burro Flats fault located approximately 5.9 miles to the north. Other nearby potentially active faults are the Boney Mountain North fault, and the Chatsworth Reservoir fault located 6.0 miles west, and 8.9 miles northeast of the site, respectively.

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.

5.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. 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, California Seismicity Map.

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.

5.3 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 Campbell and Bozorgnia (1997 revised), modeling the soil underlying the site as a Building Code Soil Profile Type S_c. The resulting calculated peak horizontal accelerations at the site are shown on Table 1. These values are one standard deviation above the mean.

Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 7.3 event on the Anacapa-Dume Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.63g.

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.

5.4 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 of 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 fault's slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone. Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Campbell and Bozorgnia (1997 revised) were utilized in the analysis.

The Upper-Bound Earthquake Ground Motion (UBE) is the level of ground motion that has a 10 percent chance of exceedance in 100 years, with a statistical return period of 949 years. The UBE is typically 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 UBE and DBE are expected to generate motions at the site of approximately 0.47g and 0.39g, respectively. Graphical representations of the analyses are presented on Figures 5 and 6.

5.5 Seismic Design Criteria

The following table summarizes site-specific seismic design criteria obtained from the 1997 Uniform Building Code (UBC). The values listed in the following table are for the Malibu Coast Fault, identified as a Type B Fault.

SEISMIC DESIGN PARAMETERS

Parameter	Value	UBC Reference
Seismic Zone Factor, Z	0.40	Table 16-I
Soil Profile Type	S_C	Table 16-J
Seismic Coefficient, C_a	0.40	Table 16-Q
Seismic Coefficient, C_v	0.59	Table 16-R
Near-Source Factor, N_a	1.0	Table 16-S
Near-Source Factor, N_v	1.1	Table 16-T
Control Period, Ts	0.59	---
Control Period, To	0.12	---
Seismic Source	B	Table 16-U

5.6 Liquefaction Potential

Liquefaction involves a sudden loss in strength of saturated, cohesionless soils that are subject to ground vibration and results in temporary transformation of the soil to a fluid mass. If the liquefying layer is near the surface, the effects are much like that of quicksand for any structure located on it. If the layer is deeper in the subsurface, it may provide a sliding surface for the material above it.

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

According to the County of Los Angeles Seismic Safety Element (Leighton, 1990) and the State of California Seismic Hazard Zone, Thousand Oaks Quadrangle Map (2000), the site is not within an area identified as having a potential for liquefaction. The site is underlain by bedrock of the Tertiary Age Topanga Formation and Tertiary Age Conejo Volcanics; and bedrock by its nature is not subject to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is considered to be nil. Further, no surface manifestations of liquefaction are expected at the subject site.

5.7 Landslides

According to the Los Angeles County Seismic Safety Element (Leighton, 1990), the site is not within an area identified as having a potential for slope instability. Additionally, according to the California Geological Survey (2000), the site is not located within an area identified as having a potential for seismic slope instability. The site and surrounding vicinity is gently sloping to the south. 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.

5.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Los Angeles County Seismic Safety Element (Leighton, 1990), the site is not located within an inundation boundary. The probability of earthquake-induced inundation is considered very low.

5.9 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.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 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 construction.

6.1.2 Up to fifteen feet of artificial fill materials were encountered during exploration at the site. The fill is likely the result of past grading activities at the site and in its present condition is not suitable for support of proposed foundations, floor slabs, or additional fill. The results of laboratory testing indicate that the existing fill materials and some alluvial and colluvial soils, in their present condition, are not suitable for support of proposed foundations, floor slabs or additional fill. The existing fill and unsuitable soils are considered suitable for re-use as an engineered fill provided the procedures outlined in the *Grading* section of this report are followed (see Section 6.4). The actual limits of removal will have to be determined by the Geotechnical Engineer (a representative of Geocon) during excavation and grading activities. Soft soils should be overexcavated as necessary.

6.1.3 Based on these considerations, it is recommended that the proposed structures and improvements be supported on a conventional foundation system bearing on a blanket of newly placed engineered fill. As a minimum, it is recommended that the upper five feet of existing site soils in the building footprint area be removed and properly recompacted for foundation and slab support. The removal should extend a minimum of ten feet beyond the proposed building footprint area, or a distance equal to the depth of fill below the foundations, whichever is greater. A minimum three-foot thick blanket of newly placed, properly compacted engineered fill should underlie all foundations and building slabs. The site soils should be well blended prior to placement as engineered fill. Any encountered fill or soft soils in the building footprint area must be completely removed at the direction of the Geotechnical Engineer (a representative of Geocon).

6.1.4 Foundations for small outlying structures such as property line walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing in newly placed engineered fill or native soils at or below a depth of two feet. Existing uncertified fill is not recommended for foundation support. It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundations supported therein.

6.1.5 Where new flatwork or paving is to be placed, it is recommended that all existing fill soils and weak native soils be removed and properly recompacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and recompacted. The client should be aware that removal and recompaction of all existing fill and weak native soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or weak native soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (Section 6.10).

6.1.6 Based on the depth of existing fill and required grading sloping measures will be required during grading activities. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 6.12).

6.1.7 The suitability of the existing buried east-west trending concrete flood control structure should be evaluated by the project civil engineer. If a new drainage structure is required, geotechnical recommendations for the new drainage structure will be provided under separate cover.

6.2 Soil and Excavation Characteristics

6.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the generally cohesive nature of the site soils, excessive caving is not anticipated during shallow vertical excavations.

6.2.2 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.

6.2.3 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 shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 6.12).

6.2.4 The soils encountered during this investigation have a “low” to “medium” expansion potential as defined by the Uniform Building Code (UBC) Table No.18-I-B. Recommendations presented herein assume that the building foundations will derive support in these materials.

6.3 Minimum Resistivity, pH and Water-Soluble Sulfate

6.3.1 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 and deep subterranean 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 B5) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is suggested that ABS pipes be considered, in lieu of cast-iron, for any retaining wall drains or subdrains required beneath the structures.

6.3.2 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 B5) and indicate that the soils at the proposed foundation level possess “negligible” sulfate exposure to concrete structures as defined by UBC Table 19-A-4.

6.3.3 Geocon Inland Empire, Inc. does not practice in the field of corrosion engineering. 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 on buried metal pipes and concrete structures in direct contact with the soils.

6.4 Grading

6.4.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon Inland Empire, Inc.

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

6.4.3 Grading should commence with the removal of all existing improvements from the area to be graded. The areas to receive compacted fill shall be stripped of all vegetation, existing fill, and soft, weak or disturbed soils. As a minimum, it is recommended that the upper five feet of existing site soils in the building area be removed and properly recompacted for foundation support. The removal should extend a minimum of ten feet beyond the proposed building footprint area, or a distance equal to the depth of fill below the foundations, whichever is greater. A minimum three-foot thick blanket of newly placed, properly compacted engineered fill should underlie all foundations.

6.4.4 Deleterious debris such as wood and tree roots should be excavated and removed from the site. Deleterious debris must not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. Rocks larger than six inches in diameter shall not be used in the fill. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

6.4.5 All imported fill shall be observed, tested and approved by Geocon Inland Empire, Inc. prior to use in the building pad area. Imported soils used in the building pad areas should have an expansion index less than 30. If imported soils are to be placed in the building area they must be placed uniformly and evenly at the direction of the Geotechnical Engineer (a representative of Geocon Inland Empire, Inc.).

6.4.6 All excavated site soils should be thoroughly blended and moisture conditioned prior to placement and compaction. All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to 2 percent above optimum moisture content, and compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557-02.

6.4.7 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. 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.

6.5 Shrinkage

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 earth materials on the site to an average relative compaction of 92 percent. Import soils may be required to attain finish grade elevations and maintain proper site drainage.

6.6 Foundation Design

6.6.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed structures and improvements.

6.6.2 Continuous footings supported in engineered fill may be designed for an allowable bearing capacity of 2,300 pounds per square foot, and should be a minimum of 12 inches in width and 24 inches in depth below the lowest adjacent grade.

6.6.3 Isolated spread foundations supported in engineered fill may be designed for an allowable bearing capacity of 2,700 pounds per square foot, and should be a minimum of 24 inches square and 24 inches in depth below the lowest adjacent grade.

6.6.4 The soil bearing pressure above may be increased by 100 psf and 300 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

6.6.5 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. Further, additional grading may be necessary in order to maintain the required three-foot-thick engineered fill blanket beneath proposed foundations.

6.6.6 Small outlying structures, such as property line walls, planter walls and trash enclosures, which will not be rigidly connected to the proposed structure may be supported on conventional foundations bearing in properly compacted fill and/or undisturbed native soils at or below a depth of two feet. Existing uncertified fill is not suitable for foundation support. Miscellaneous foundations bearing in native soils may be designed for a bearing value of 1,000 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials. Excavations should be deepened as necessary to extend into satisfactory soils. Due to the weak nature of the upper native soils, it is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundations supported therein.

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

6.6.8 Unless specifically design by the project structural engineer, 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.

6.6.9 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.

6.6.10 Provided the building moisture content in the engineered building pad is maintained, 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.

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

6.6.12 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.

6.7 Conventional Foundation Settlement

6.7.1 The maximum expected settlement for a structure supported on a conventional foundation system in engineered fill is estimated to be less than ¾ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of twenty feet.

6.8 Lateral Design

6.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the engineered fill and undisturbed native soils.

6.8.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted fill and undisturbed native soils may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

6.9 Concrete Slabs-on-Grade

6.9.1 Conventional concrete slabs-on-grade may be utilized subsequent to the recommended grading.

6.9.2 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 6.10). Building slabs-on-grade, not subject to vehicle loading, should be a minimum of 4 inches thick and should be reinforced with a minimum of No. 4 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. Wire mesh is not recommended.

6.9.3 Where moisture sensitive floor coverings are planned, the concrete slab-on-grade should be underlain by at least 4 inches of clean, dry sand (Sand Equivalent greater than 30), and a moisture barrier should be placed at the midpoint of the sand cushion. The moisture barrier may consist of a polyethylene sheet (visqueen) having a minimum thickness of 15 mils.

6.9.4 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

6.9.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Wire mesh is not recommended. Prior to construction of slabs, the upper 12 inches of the subgrade should be moisture conditioned to at least 2 percent above optimum moisture content and properly compacted. 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.

6.9.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement 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.

6.10 Preliminary Pavement Recommendations

6.10.1 Where new flatwork or paving is to be placed, it is recommended that all existing fill soils and weak native soils be removed and properly recompacted for paving support. As a minimum, the upper twelve inches of soil should be scarified and recompacted. The client should be aware that removal and recompaction of all existing fill and weak native soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or weak native soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs

6.10.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an actual R-Value should be obtained by laboratory testing prior to placing pavement. Pavement thickness was determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking	3.5	3	4½
Driveways	5	3	6
Trash Truck & Fire Lanes	7	4	12

6.10.3 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).

6.10.4 Where concrete paving will be utilized for support of heavy vehicles, it is recommended that the concrete be a minimum of 6 inches in thickness 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.

6.10.5 Prior to placing base material, the subgrade should be scarified; moisture conditioned and recompacted to a minimum of 95 percent relative compaction. The depth of compaction should be at least 12 inches. All base material should also be compacted to a minimum of 95 percent relative compaction.

6.10.6 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.

6.11 Retaining Walls

6.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls supporting a level backcut of fill and/or native soil, having a maximum height of seven feet. In the event that walls higher than seven feet or other types of walls are planned, Geocon Inland Empire should be contacted for additional recommendations.

6.11.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report (Section 6.6).

6.11.3 Retaining walls not restrained at the top and having a level backfill surface should be designed utilizing a triangular distribution of pressure as indicated in the table below:

HEIGHT OF WALL (Feet)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 7	35

6.11.4 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.

6.11.5 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 or weepholes should be provided to prevent the buildup of hydrostatic pressure. If a subdrain is utilized it should be 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 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer, (a representative of Geocon Inland Empire, Inc.), prior to placement of gravel or compacting backfill.

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

6.11.7 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, it is recommended that the entire below grade portion of the retaining wall be designed for full hydrostatic pressure based on a water level at the ground surface. The equivalent fluid pressure to be used in design of the walls if groundwater is at the ground surface would be 80 pounds per cubic foot. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

6.12 Temporary Excavations

6.12.1 Excavations on the order of 5 to 20 feet in vertical height will be required for the proposed grading of the site. The excavations are expected to expose fill and medium dense to firm soils, which are suitable for vertical excavations up to five feet in height where not surcharged by adjacent traffic or structures.

6.12.2 Excavations greater than five feet in height or those that are surcharged by adjacent traffic or structures will require sloping measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion. Should excavations be required adjacent to an existing structure, the bottom of any unshored excavation should be restricted so as not to extend below a plane drawn at 1:1 downward from the foundation of the existing structure.

6.12.3 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. Our personnel should inspect the soils exposed in the cut slopes during excavation 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.

6.13 Surface Drainage

6.13.1 Proper surface drainage is critical to the future performance of the project. 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.

6.13.2 All site drainage should be collected and transferred to the street 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. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers is not recommended onto unprotected soils within five feet of the building perimeter. It is recommended that planters, which are located adjacent to foundations, 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.

6.13.3 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.

6.13.4 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. A subdrain, which collects excess irrigation water and transmits it to drainage structures, should also be considered.

6.14 Plan Review

6.14.1 Grading, foundation and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon Inland Empire, 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 Inland Empire, 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 Inland Empire, 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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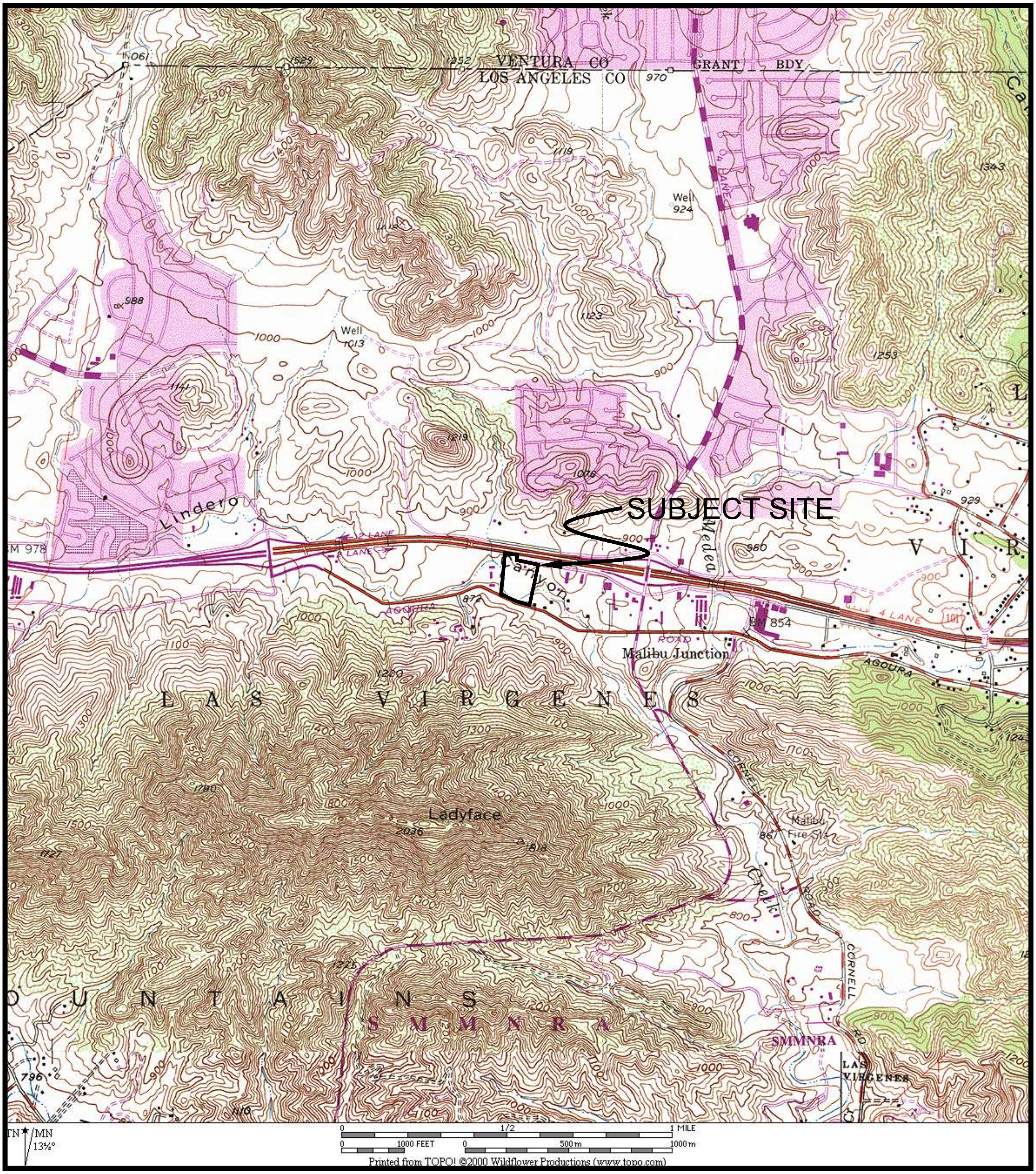

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VICINITY MAP

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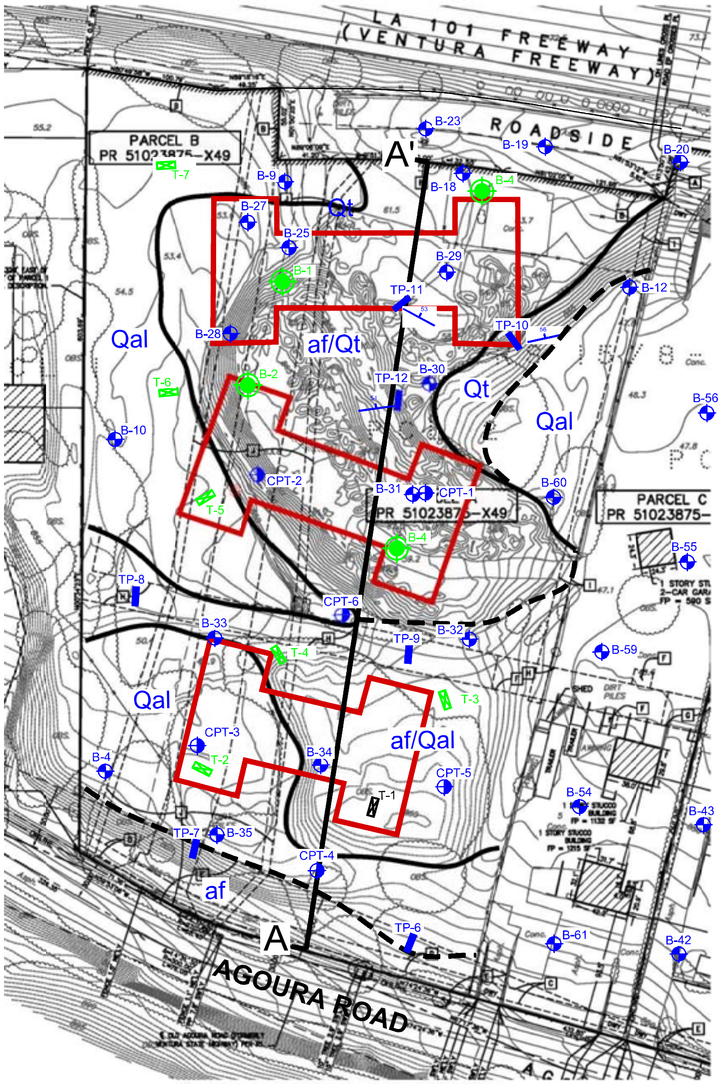
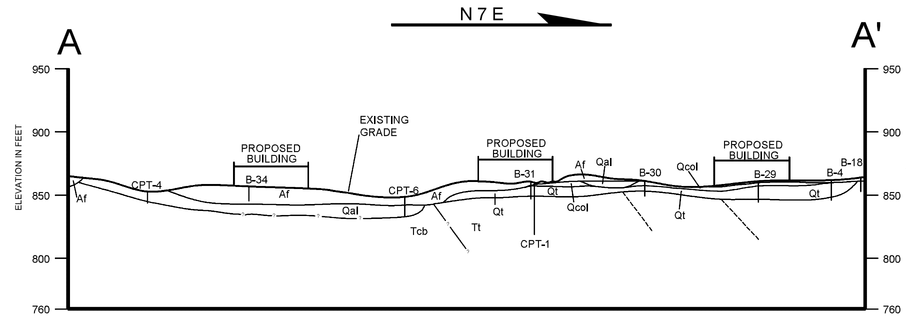
DEC. 19, 2006	PROJECT NO. A8487-06-01A	FIG. 1



0 100' 200'



LEGEND

B-4	Approximate Boring Location (Geocon)
T-7	Approximate Test Pit Location (Geocon)
B-60	Approximate Boring Location (AGS)
CPT-6	Approximate CPT Location (AGS)
T-13	Approximate Test Pit Location AGS)
54	Strike & Dip of Bedding
	Approximate Location of Proposed Structures
	Geologic Contact (Dashed where Approx.)

af	Artificial Fill
Qcol	Topsoil/Colluvium
Qal	Alluvium (Holocene)
Qt	Terrace Deposits (Quaternary)
Tt	Topanga Formation (Tertiary)
Tcb	Conejo Volcanics (Tertiary)
	Cross Section Location

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SITE PLAN

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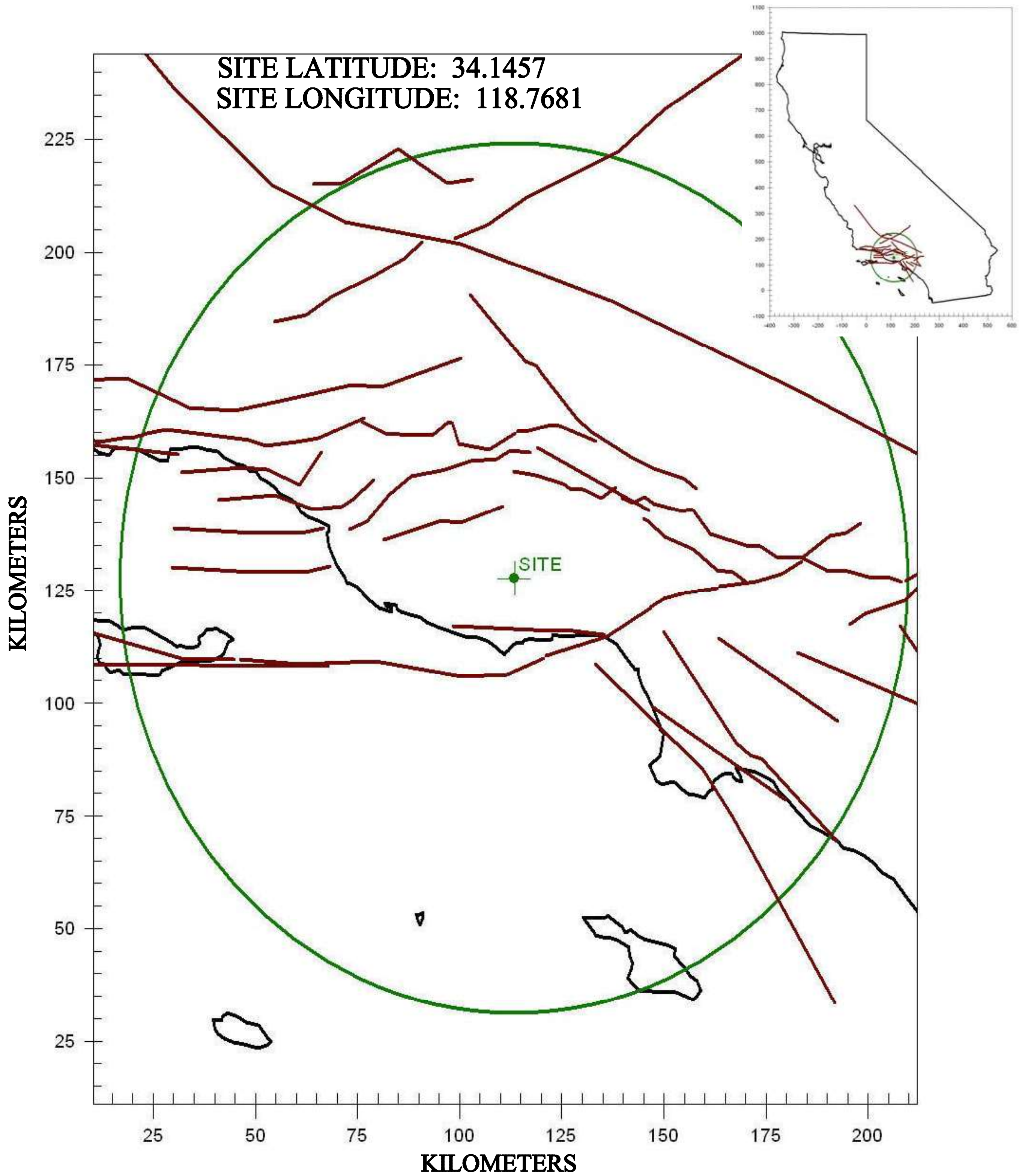
DEC. 19, 2006 | PROJECT NO. A8487-06-01A | FIG. 2



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CALIFORNIA FAULT MAP

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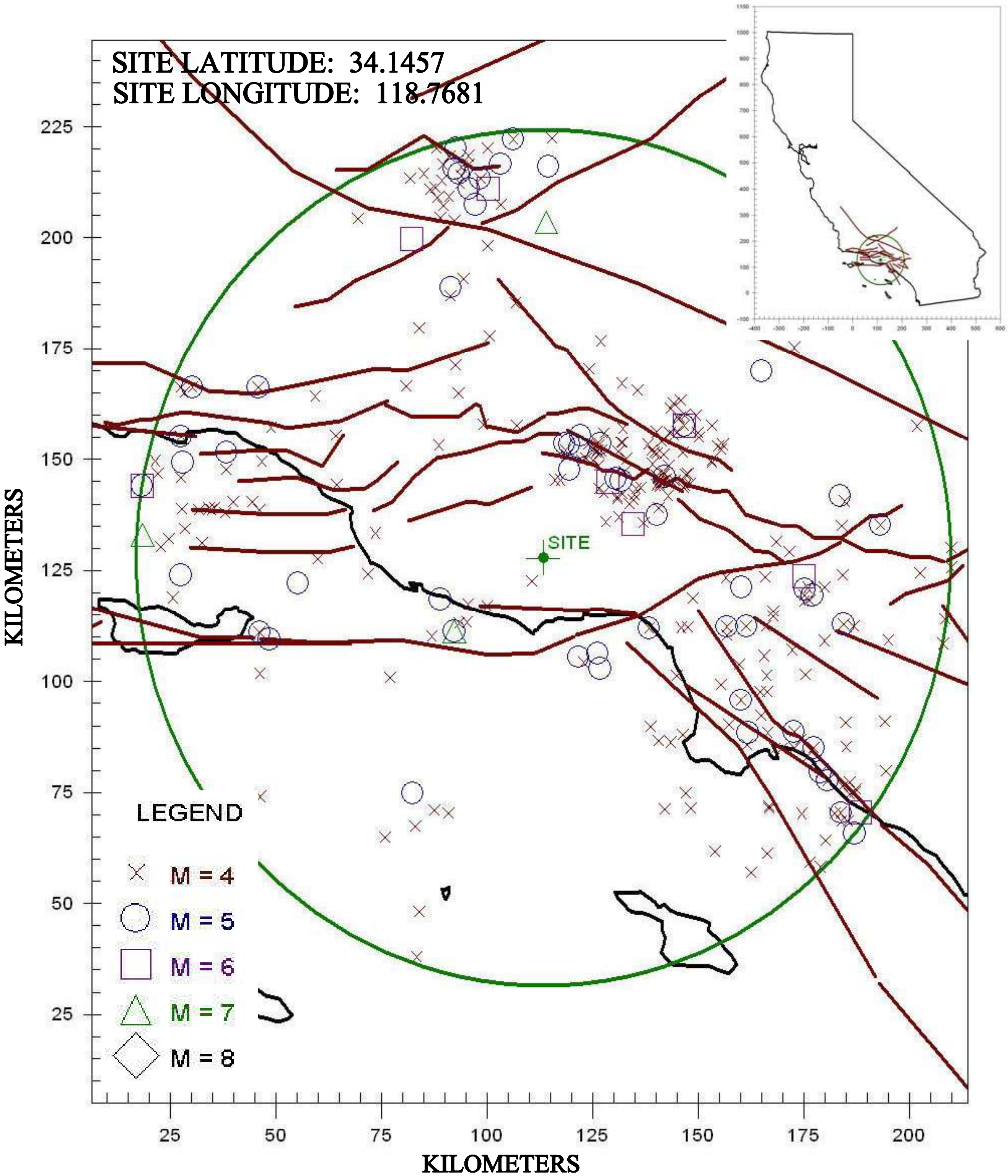
DEC. 19, 2006	PROJECT NO. A8487-06-01A	FIG. 3



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CALIFORNIA SEISMICITY MAP

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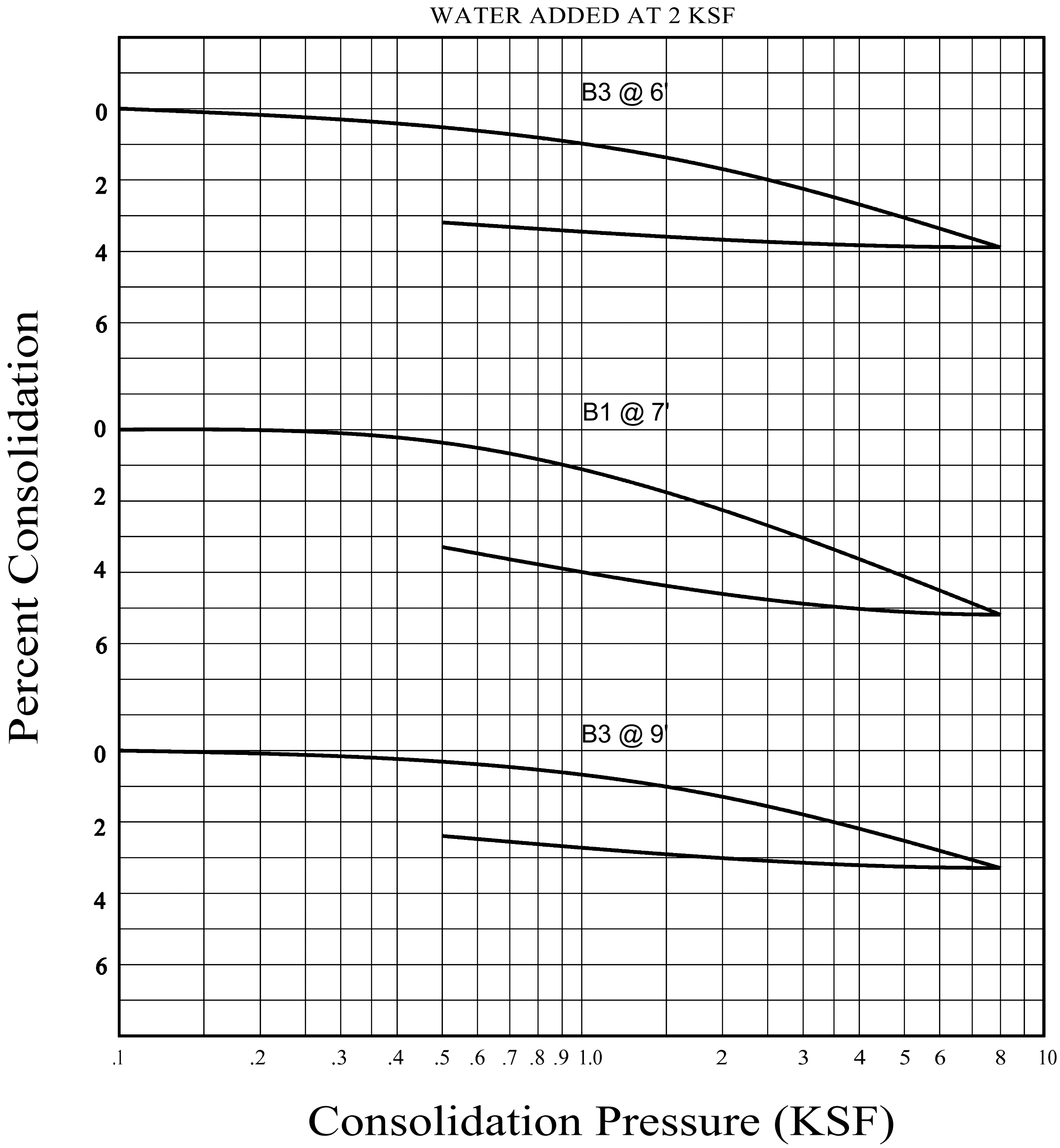
DEC. 19, 2006 | PROJECT NO. A8487-06-01A | FIG. 4



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CONSOLIDATION TEST RESULTS

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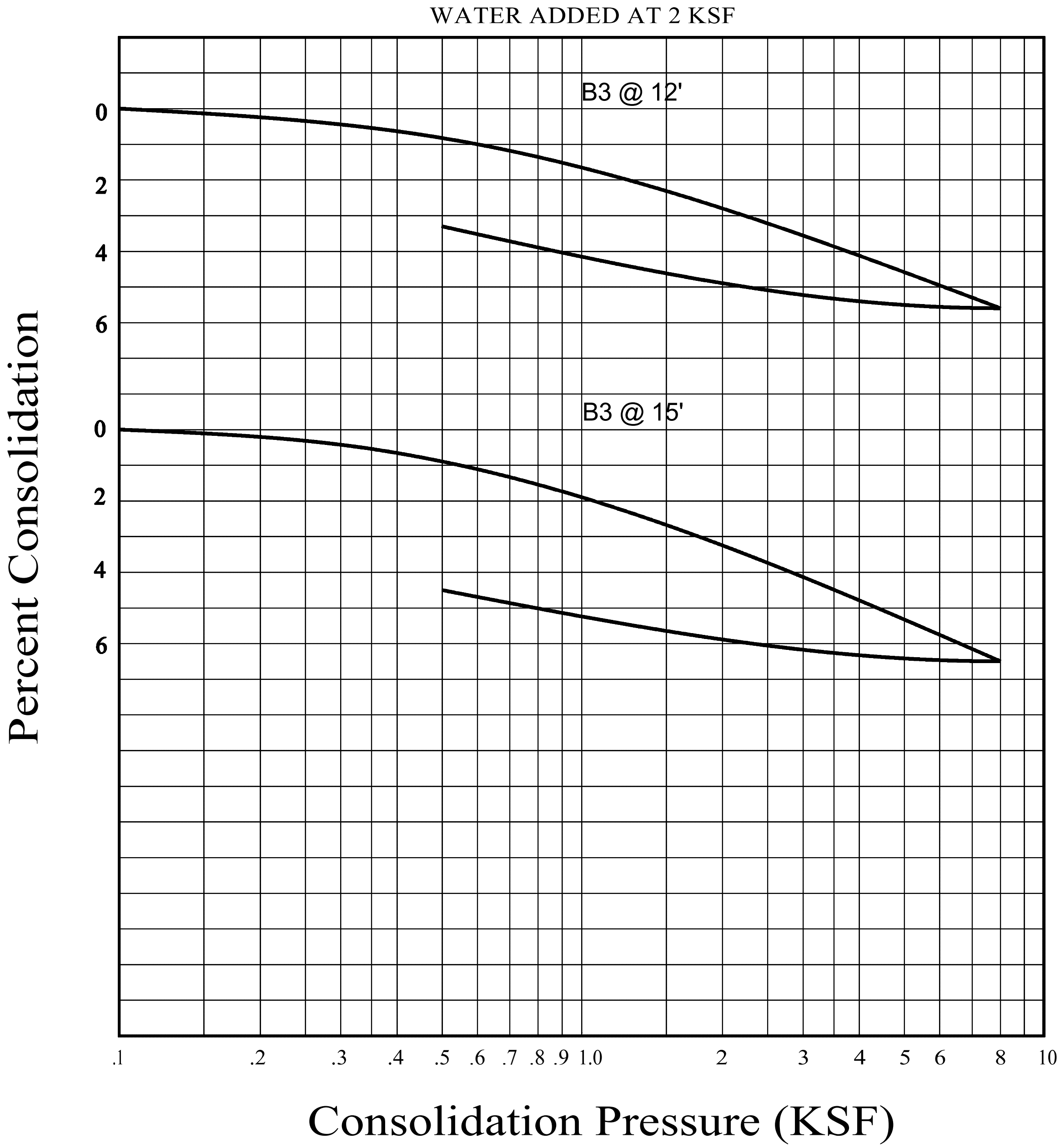
BRG		8000	DEC. 19, 2006	PROJECT NO. A8487-06-01A	FIG. B3



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CONSOLIDATION TEST RESULTS

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DEC. 19, 2006	PROJECT NO. A8487-06-01A	FIG. B4

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0-3'	7.4	2400 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS CALIFORNIA TEST NO. 422

Sample No.	Chloride Ion Content (%)
B1 @ 0-3'	0.003

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 0-3'	0.005	Negligible

* Reference: 1997 Uniform Building Code, Table 19-A-4.

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CORROSIVITY TEST RESULTS

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BRG		8000	DEC. 19, 2006	PROJECT NO. A8487-06-01A	FIG. B5