Appendix E Geological Data

MEMORANDUM

To: Ruta K. Thomas, Project Manager

From: George J. Burwasser, Senior Scientist II, Professional Geologist 7151

CC:

Date: April 15, 2008

Re: Geotechnical Technical Memo – Ripcurl Mixed Use Development EIR – Project Number 0D2138700

I. Project Description

The proposed project is a mixed-use residential and commercial development that would consist of four levels of housing over three levels of parking (one level of parking below grade and two levels of parking above grade). The retail component would be located on the ground level adjacent to the two levels of above grade parking. A mezzanine level would also be located on the roof. Overall, the project would be six stories in height and consist of approximately 440 residential units and up to 10,000 square feet (sf) of retail uses. The total project floor area, excluding parking and basement area, would be approximately 382,700 sf. Outdoor amenities would include a pool and spa area, fire pit and movie projection area. Indoor amenities would include a fitness center, business center, conference room, and clubhouse.

The depth of the subterranean parking level is anticipated to be between 10 and 22 feet below the existing ground surface, including footing depths. Therefore, it is anticipated that the proposed site development will include excavations of 10 to 22 feet below the existing ground surface.

The residential component would include approximately 301,098 sf of residential area and approximately 7,000 sf of leasing office, lobby and recreation space. Of the approximately 440 residential units that are proposed, it is estimated that 151 would be studio apartments, 190 would be one-bedroom units, 88 would be two-bedroom units, and 11 would be live-work loft units. Units would range in size from 465 sf (studio) to 1,037 sf (two-bedroom). Based on the existing average household size of 2.41 persons per renter-occupied unit for the City of Huntington Beach,[1] the residential component of the project would most likely generate approximately 1,060 residents. However, based on the applicant's experience with similar projects, the residential component of the project would most likely generate approximately 611 residents,[2] which is based on an average household size of 1.1 persons per studio and loft units, 1.4 persons per one-bedroom unit, and 2.0 persons per two-bedroom unit. The residential component would also likely employ approximately 11 full-time positions.[3] Amenities provided by the residential component would include a pool, spa, fitness center, business center, conference room, and clubhouse.

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1 United States Census Bureau, 2006 American Community Survey, <http://factfinder.census.gov>; (10 January 2007).

2 Red Oak Investments, LLC. November 2007.

3 Red Oak Investments, LLC. November 2007.

The commercial component of the proposed project would include up to 10,000 sf of ground floor retail that would be located on the corner of Gothard Street and Center Avenue. The commercial component would offer neighborhood-serving retail that would target students attending Golden West Community College and nearby residents. Potential retailers would include uses such as a convenience store, café, sandwich shop, cleaners, juice shop, and mailbox store. If commercial demand rises in the future, the live-work units could be converted to retail uses in the future. The commercial component would likely employ approximately 36 full-time positions.[4]

II. Geotechnical Report Peer Review

Introduction

Reports of geotechnical investigations are prepared routinely for proposed new developments throughout California to ensure that earthwork and structural designs address the soil and seismic conditions affecting the site of the proposed development. Such site-specific reports are required by most jurisdictions as part of a project application approval process. Generally the reports are titled Preliminary, in that the project design may remain mutable, and are intended to be supplemented with other detailed investigations at the time of foundation construction. The level of detail in the reports may vary considerably, depending on the reports' intended use: a report submitted at the plan check stage to demonstrate project feasibility would not be expected to contain the same detail as the report used to engineer the final design of a structure's foundation. The use of geotechnical reports in the preparation of California Environmental Quality Act (CEQA) documents provides part of the technical basis for assessing geologic impacts of, and on, proposed development. Rarely are they the sole documentation on which the geologic impact analysis rests and are used in context with other published and unpublished material, personal interviews, and site inspections to provide sufficient information to allow informed decisions regarding the probable geologic consequences of approving a project.

The City of Huntington Beach is preparing an Environmental Impact Report (EIR) for the Ripcurl Project. The City has requested review of the geotechnical report provided by the project applicant to determine the report's acceptability as an information source for the EIR. Comments in the Discussion section, below, focus on that concept, rather than on detailed construction specifications. The report's original purpose was to provide the applicant with information pertaining to the geotechnical aspects of project design and construction.

In general, the geotechnical report is well organized and is understandable to the technical reader. The literature search, onsite data collection, and laboratory testing follow approved methods: the results are presented in text form, as well as tabular/graphically (highly recommended). Some text interpretation/updating and graphic recreation will be necessary by the EIR consultant.

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[4] Ibid.

Major Sources of Information

The document under review is

Geotechnical Investigation
Proposed College Country Mixed-use Development
7302-7400 Center Avenue
Huntington Beach, California

prepared for

Mr. Alex Wong
23622 Calabasas Road, Suite 100
2101 Business Center Drive, #230
Irvine, California 92612

prepared by

Neal D. Berliner, GE 2576
Gerald A. Kasman, CEG 2251
Geocon Inland Empire, Inc.
3303 North San Fernando Boulevard, Suite 100
Burbank, California 91504

Project No. A8481-06-01
December 12, 2006

Subject Property

The nearly level project site at the intersection of Center Avenue and Gothard Street is approximately 3.8 acres in size, generally rectangular in shape, and occupied by one- and two-story commercial buildings and paved surface parking. The proposed development would be a mixed-use residential and commercial structure containing four levels of housing over three levels of parking. One level of parking would be below grade and two levels would be above grade. Street-level retail would occupy part of the at-grade parking level. Total depth of excavation is expected to be between 10 and 22 feet below ground level.

Discussion

Section 3, Soils and Geologic Conditions. The descriptions of the soil/geologic units are a bit brief for CEQA purposes, but adequate for the original purposes of the report. The descriptions can be expanded in the EIR from other available sources.

Section 4, Groundwater. This is a clear statement of historical conditions revealed through literature search, of the conditions at the time of field investigation, and of the considerations necessary for the proposed construction. The information can be adapted readily for use in the EIR.

Section 5, Geologic Hazards. The Faulting presentation is excellent, although the accompanying Figure 3 would be much improved by the addition of labels. The same is true of Figure 4 in the Seismicity presentation. The explanations of the Deterministic and Probabilistic Analyses for estimating peak horizontal ground accelerations are quite clear to the technical reader, but would need to be presented differently in the EIR for the general reader. The summary of Seismic Design Criteria form the 1997 Uniform Building Code has been rendered irrelevant by the implementation of the 2007 California Building Code (effective January 1, 2008). The individual explanations of secondary seismic hazards (liquefaction, settlement, slope instability, flooding) and geologic hazard (subsidence) are concise and correctly based on site observations/laboratory tests. The information can be adapted readily for use in the EIR.

Section 6, Conclusions and Recommendations. This section appears to contain standard construction recommendations that the applicant's engineers would need to refine to design the foundations and below-grade portions of the project. The technical reader will note that site-specific information has been incorporated throughout the recommendations although the general reader might not notice or be able to interpret it. The information can be adapted readily for use in the EIR.

Appendices. Complete logs of borings and tables/graphs of laboratory test results are included as appendices to the report. The information can be adapted readily for use in the EIR.

Recommendations

If regional or local geologic figures are to be included in the EIR, new ones will need to be created to convey the information in a meaningful context. This can be done readily by the EIR consultant's graphics or GIS personnel.

The seismic design parameters based on the pre-2007 Building Code requirements will need to be updated. This can be done readily by the EIR consultant's geologist.

Although not strongly stated in the geotechnical report, the EIR will need to make clear that the report's findings support the existing regulatory requirements that new structures at the project site withstand anticipated seismic vibration from a major earthquake on the Newport-Inglewood fault, remain stable during possible liquefaction-induced settlement, and provide adequate drainage to accommodate recognized high groundwater conditions.

III. Geologic Site Conditions

The basic geology and soils conditions are addressed in the geotechnical investigation report we have for the project site: the City of Huntington Beach is on a coastal plain underlain by relatively recent sediments ranging in age from Quaternary deposits of the Pleistocene epoch (11,000 to 1, 600,000 years) through the Holocene epoch (less than 11,000 years). The older sediments typically are shallow marine terrace deposits that have been uplifted by ongoing seismic movement and eroded to form the Bolsa Chica and Huntington Beach mesas. The mesas are bordered by younger (unconsolidated) alluvial soils that fill the gaps near Seal Beach, Bolsa Chica, and the Santa Ana River. Older alluvial and/or terrace deposits are present at this site. These sediments are estimated to be in excess of 50 feet thick. The project site is several miles inland from the coastal bluffs and the surface geology varies from the majority of Huntington Beach. Within the specific project area the soils consist of younger alluvial materials. The most significant fault to the City is the Newport-Inglewood fault 3.1 miles southwest of the project site. The fault zone is visible on the surface as a series of northwest-trending elongated hills, including Signal Hill and the Dominguez Hills, extending from Newport Beach to Beverly Hills.

Soil and Groundwater Conditions

Based on the Geocon Inland Empire, Inc (Geocon) .investigation, the soils underlying the Ripcurl site consist of artificial fill over alluvial deposits. Artificial fill was encountered at depths ranging from 1.5 to 7.5 feet below the existing ground surface (bgs). The fill consists primarily of dark brown, very fine-grained sandy silt with traces of gravel and scattered construction debris. The fill may be the result of past grading and construction activities. It is possible that deeper fill could be encountered during the construction period because not all subsurface parts of the site were explored by Geocon. Beneath the fill Holocene age alluvial soils were identified, consisting of relatively flat-lying layers of silt, sandy silt, silty sand and clay. The soils are primarily fine-grained and soft to firm with some loose to medium dense silty sand layers. Small amounts of peat were observed in the soils. The soils are flood plain deposits and are anticipated to extend to a depth of approximately 90 feet bgs.

As part of the proposed project, one level of subterranean parking would be included on the project site. The probable depth of the subterranean parking would be between 10 to 22 feet bgs, including footings. Materials exposed during excavations would consist of horizontally stratified to massive alluvium.

Geocon's literature search found the historic high groundwater level in the vicinity of the project site to be approximately 5 feet bgs and the depth to groundwater at the site between 5 and 10 feet bgs. Groundwater is present throughout the site and was encountered at a depth of 8 feet bgs. The soils at the site are very fine grained and not conducive to high permeability or allowing free flow of water through the alluvial materials. The majority of groundwater seepage encountered during excavation would emanate from the sand beds in the alluvium.

Regional and Local Faults

The nearest known active fault is the Newport-Inglewood Fault Zone, which is approximately 3.1 miles southwest of the project site. The closest surface projection of the Newport-Inglewood Fault Zone is the Seal Beach segment, 3.5 miles southwest of the site. Other nearby active faults include the Palos Verdes fault, 12.5 miles southwest; the Whittier Fault Zone, 15.5 miles north-northeast; and the Elsinore Fault Zone , 21 miles northeast of the site.

There are several potentially active faults in the vicinity of the project site, including Los Alamoitos fault, approximately 5.6 miles to the northwest; Pelican Hill fault, 8.5 miles southeast; El Moderno fault, 8.9 miles southeast; and the Norwalk fault, 10 miles north of the site. Summary of information about known faults in the project area is in Table EH-1 of the City's Environmental Hazards Element.

Historic and Future Seismicity

According to the City's Environmental Hazards Element, the estimated maximum earthquake assigned to the Newport-Inglewood fault zone is Richter magnitude (M) 7.0. The expected (average) amount of surface fault rupture on any given fault trace would range from zero to about one foot for events with magnitudes under M6.0, and from one foot to about ten feet for events with magnitudes between M6.0-7.5. Large earthquakes occurred in the area of the City in 1769 (fault unknown), 1812 (possible the Newport-Inglewood fault), 1855 (Newport-Inglewood fault or an unnamed concealed fault), and in 1920, 1933, and 1941 (all Newport-Inglewood fault). Events greater than M7.0 may occur on the Newport-Inglewood fault once in 200 to 2000 years. According to the Uniform California Earthquake Rupture Forecast, the probability of a M7.0 earthquake occurring in the Los Angeles area (although probably on the San Andreas fault rather than on the Newport-Inglewood fault) during the next 30 years is 82 percent.[5]

Geologic Hazards

The potential seismic hazards at the sites include groundshaking, liquefaction, settlement, and earthquake-induced flooding. Potential soil hazards include subsidence and expansion.

Groundshaking. The California Geological Survey (CGS) Probabilistic Seismic Hazards Assessment Program estimates peak ground accelerations in the alluvium at the site would be 0.389g (g = the force of gravity). The 2007 California Building Code (CBC) incorporates attenuation relationships developed by the CGS's Probabilistic Seismic Hazard Program, which consider vibration contributions from multiple seismic sources, including those generated by the nearby Newport-Inglewood fault and those of the more distant, but potentially more damaging, San Andreas fault. The resultant map (Figure 1613.5(3) of the 2007 CBC) of short term (0.2 second) ground response indicates the site would be subjected to average peak ground accelerations as high as 1.5g for the largest earthquakes in the Los Angeles area. The 2007 CBC requires the design earthquake (i.e., the maximum considered earthquake acceleration response for a given site) to be calculated using 2/3 of the mapped acceleration value – in this case, 1.0g, which accords reasonably well with the CGS calculated probabilistic short term ground response of 0.935g for alluvium at the site.

Liquefaction and Seismically Induced Settlement. The project site is in a high to very high liquefaction potential area identified by the City's Environmental Hazard Element, as well as being in a Liquefaction Hazard Investigation Zone on the State of California Seismic Hazard Zone Map for the Newport Beach Quadrangle (CDMG, 1997). Geocon concluded the historically highest groundwater in the area was approximately 5 feet bgs and conducted an analysis of the soils underlying the Ripcurl site to determine the liquefaction potential during a M7.5 earthquake with a peak horizontal acceleration of 0.40g. The analysis indicated the site soils would be prone to 1.0 inch of settlement as a result of liquefaction during such ground motion.

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[5] 2007 Working Group on California Earthquake Probabilities, Southern California Earthquake Center, Uniform California Earthquake Rupture Forecast 2, December 31, 2007, http://www.scec.org/ucerf/, last modified April 13, 2008; accessed by PBS&J Geologist April 14, 2008.

Subsidence. Given the presence of organic material in the soil at the site, the area is known to be subject to subsidence associated with hydrocompaction (the settling and hardening of land caused by application of large amounts of water), and/or peat oxidation (the decomposition of organic materials in the soil), although the amounts of subsidence appear insignificant.

Landslides. The project site and the surrounding area are relatively flat with no pronounced slopes. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Consequently, there is no potential for a landslide to be a hazard to the project site. The Ripcurl project could include excavations of 10 to 22 feet bgs. The majority of the materials exposed in the excavation would consist of horizontally stratified to massive alluvium which lacks well-defined planar features or discontinuities (such as bedding or joints) that could act as planes of weakness and allow sliding. These conditions are favorable for general excavation wall stability. The excavation would be shored and dewatered for stability, as required by the City's Municipal Code.

Expansive and Collapsible Soils. The City's Environmental Hazard Element identifies the project site as being in an area of "very high" potential for expansive soils. Soil testing to identify expansive characteristics and appropriate remediation are required by the City's Grading and Building Codes. As determined by the geotechnical investigation, soils at the site exhibit expansion characteristics. There do not appear to be collapsible soils at the site.

California Building Code and City of Huntington Beach Municipal Code

The state regulations protecting the public from geo-seismic hazards, other than surface faulting, are contained in 2007 California Code of Regulations, Title 24, Part 2 (the California Building Code [CBC]) and California Public Resources Code, Division 2, Chapter 7.8 (the Seismic Hazards Mapping Act). Both of these regulations apply to public buildings (and a large percentage of private buildings) intended for human occupancy.

Until January 1, 2008, the CBC was based on the then-current Uniform Building Code and contained Additions, Amendments and Repeals specific to building conditions and structural requirements in the State of California. The 2007 CBC, effective January 1, 2008, is based on the current (2006) International Building Code and contains prominent enhancement of the sections dealing with fire safety, equal access for disabled persons, and environmentally friendly construction.[6] Cities and counties are required to enforce the regulations of the 2007 CBC beginning January 1, 2008. Subsequently, each jurisdiction may adopt its own building code based on the 2007 CBC. Local codes are permitted to be more stringent than Title 24, but must, at a minimum, meet all state standards. The City of Huntington Beach has adopted the 2007 CBC as the basis for the City Building Code (Municipal Code Title 17.04). The City's enforcement of its Building Code ensures the project would be consistent with the CBC.

Chapters 16 and 16A of the 2007 CBC deal with Structural Design requirements governing seismically resistant construction, including (but not limited to) factors and coefficients used to

[6] California Building Standards Commission, *2007 California Building Code*, California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, effective January 1, 2008.

establish seismic site class and seismic occupancy category for the soil/rock at the building location and the proposed building design. Chapters 18 and 18A of the 2007 CBC include (but are not limited to) the requirements for foundation and soil investigations (§1802 & 1802A); excavation, grading, and fill (§1803 & 1803A); allowable load-bearing values of soils (§1804 & 1804A); and the design of footings, foundations, and slope clearances (§1805 & 1805A), retaining walls (§1806 & 1806A), and pier, pile, driven, and cast-in-place foundation support systems (§1808, 1808A, 1809, 1809A, 1810 & 1810A). Chapter 33 of the 2007 CBC includes (but is not limited to) requirements for safeguards at work sites to ensure stable excavations and cut or fill slopes (§3304).

Additionally, the a National Pollution Discharge Elimination System (NPDES) Phase I Permit, and the City's Grading and Excavation Code (Municipal Code Title 17.05) regulate erosion and sediment control, as well as excavation stability during the construction period.

Thresholds of Significance

The City's thresholds of significance are based on Appendix G to the 2006 CEQA Guidelines. With the exception of needing to update the references to the Building Code, they are generally applicable as written, although they could be strengthened with the addition of actual criteria against which to measure the potential effects of proposed projects. Nonetheless, adherence to the CBC, as adopted by the City, and the City's Grading and Excavation Code would ensure the maximum practicable protection available for people and structures on the project site and would render all potential geotechnical impacts less-than-significant. Compliance with the requirements of the City's Municipal Code is not optional and no discretionary action is necessary on the part of the City to enforce it. Compliance is part of the regulatory environment in which the project is proposed. No mitigation measures are necessary.

Exposure to Seismic-related Hazards

Project design is required to include the application of CBC seismic standards as the minimum seismic-resistant design. The applicable CBC requirements include seismic-resistant earthwork and construction design criteria, based on the site-specific recommendations of a project's California-registered geotechnical and structural engineers; engineering analyses that demonstrate satisfactory performance of any unsupported cut or fill slopes, and of alluvium and/or fill where they form part or all of the support for structures, foundations and underground utilities; and an analysis of soil expansion potential and appropriate remediation (compaction, removal-and-replacement, etc.) prior to using any expansive soils for foundation support.

Compliance with the CBC includes procedures to ensure protection of structures and occupants from geo-seismic hazards:

- During site preparation, a registered geotechnical professional must be on the site to supervise implementation of the recommended criteria.

- A California Certified Engineering Geologist, or California-licensed Civil Engineer (Geotechnical) for the applicant must prepare an "as built" map/report to be filed with the City showing details of the site geology, the location and type of seismic-restraint facilities, and documenting the following requirements, as appropriate.
    - Engineering analyses demonstrating satisfactory performance of compacted fill or natural unconsolidated sediments where either forms part or all of the support for any structures, especially where the possible occurrence of liquefiable, compressible, or expansive soils exists.
    - Engineering analyses demonstrating accommodation of settlement or compaction estimates by the site-specific Geotechnical Report for access roads, foundations, and underground utilities in fill or alluvium.

Exposure to Other Geotechnical Hazards

Potentially unstable soils discovered during excavation are required by provisions of the City's Municipal Code to be removed and replaced with engineered fill, or otherwise treated to provide appropriate foundation support and to protect them from failures such as liquefaction.

Slope stability issues related to the sides of excavations are regulated by the City's Municipal Code.

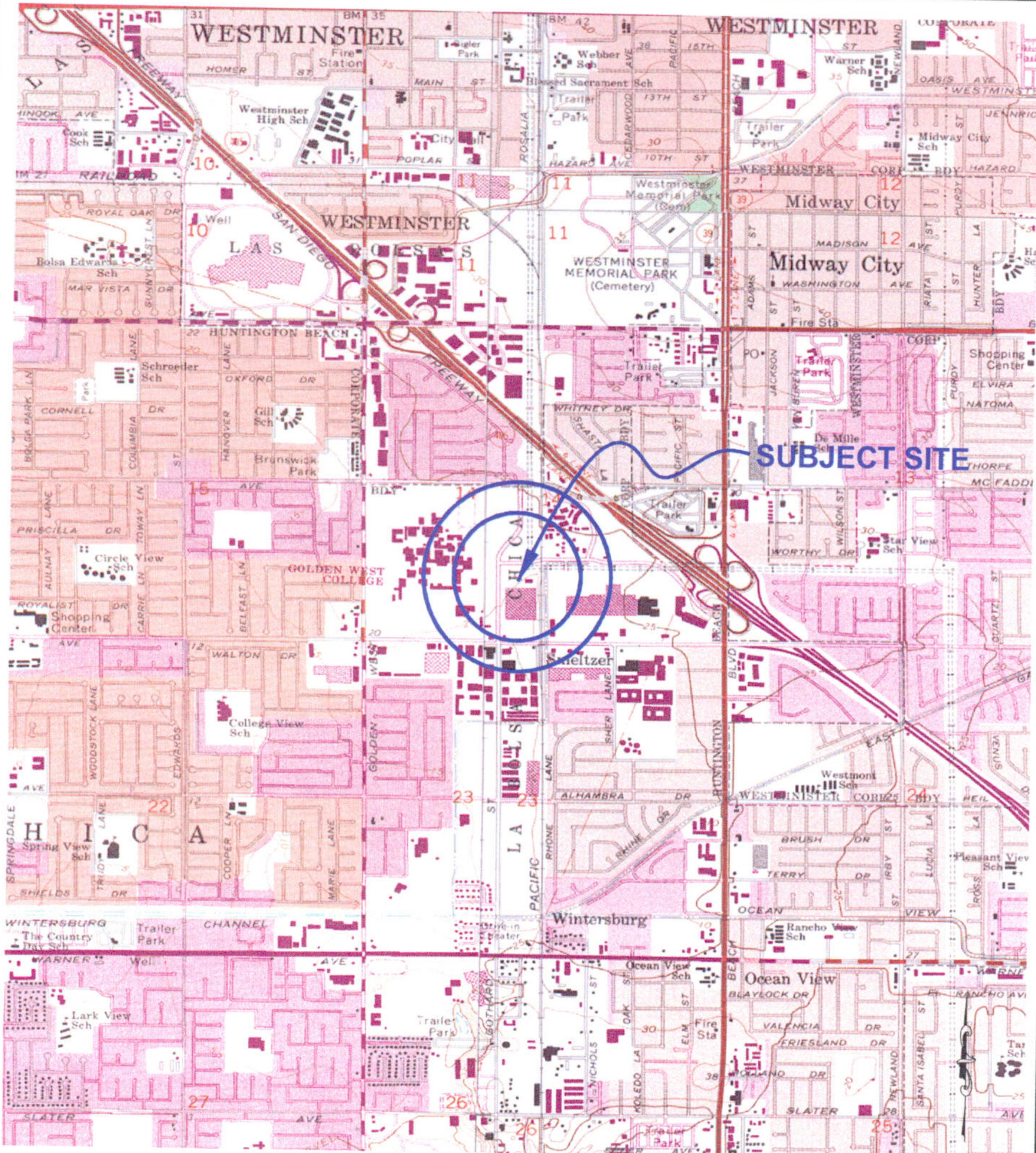
The existence of expansive subsoils makes it necessary to ensure the materials used for foundation support are sound to avoid future problems of foundation settlement and utility line disruption. An acceptable degree of soil stability can be achieved by treatment programs to eliminate expansion of soils which could include, but would not be limited to, lime grouting, wet recompaction, and excavation for replacement with non-expansive material, as described previously, to address the specific soil conditions at the construction sites. Where applicable, such treatment is required by the City's Municipal Code.

The exposure of previously covered soils during construction activities could lead to increased on-site erosion and off-site sediment transport because disturbed soils are susceptible to higher rates of erosion from wind, rain, and runoff of dewatering discharge or dust control water than undisturbed soils. The State Water Resources Control Board and the City's Municipal Code require erosion and sediment controls for construction projects with more than one acre of land disturbance. These requirements include preparation and implementation of a Storm Water Pollution Prevention Plan, with both construction-period and permanent erosion and sediment controls; preparation and implementation of an erosion and sediment control plan, describing both construction and permanent erosion and sediment controls; and construction site inspection by the City.

Conclusion

Because the City has the most recent Building Code and its Grading and Excavation Code established as part of the Municipal Code, there is no necessity for mitigation measures addressing seismic, geologic, or geotechnical issues. These issues, and the regulations governing them, will be explained in detail in the EIR so the reader will understand that application of the existing regulations renders each of them less than significant.

PROFESSIONAL GEOLOGIST
GEORGE J. [illegible]
No. 7151
EXP. 30 APRIL 2010
STATE OF CALIFORNIA



REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, NEWPORT BEACH, CA QUADRANGLE

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

BRG		8000

VICINITY MAP

AMSTAR/RED OAK HUNTINGTON BEACH, LLC
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 1

LEGEND

B6 Approximate Location of Boring

GEOCON
INLAND EMPIRE, INC.

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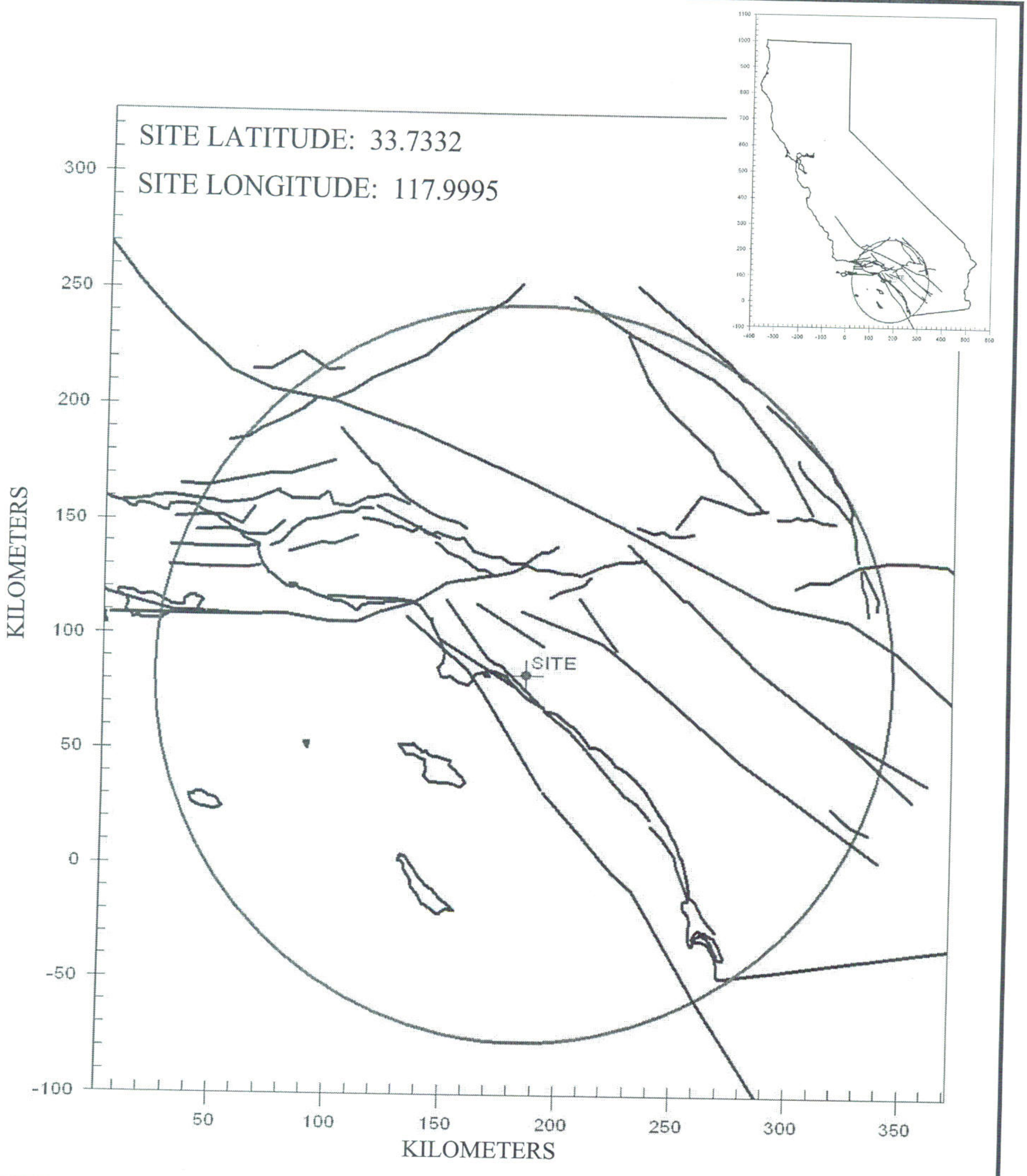
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

SITE PLAN

AMSTAR/RED OAK HUNTINGTON BEACH, LLC
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

GAK		8000

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 2



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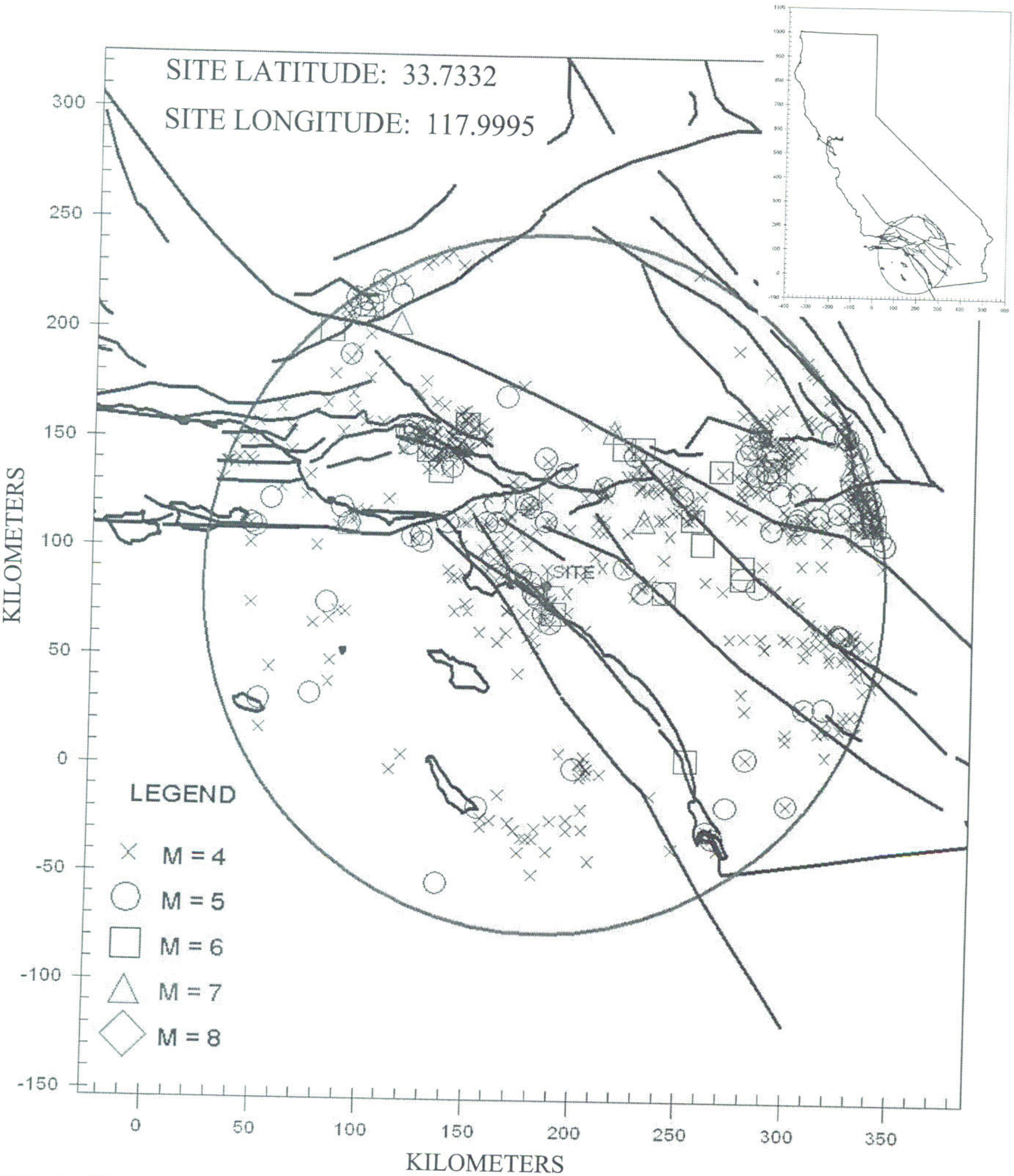
BRG		8000

CALIFORNIA FAULT MAP

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 3



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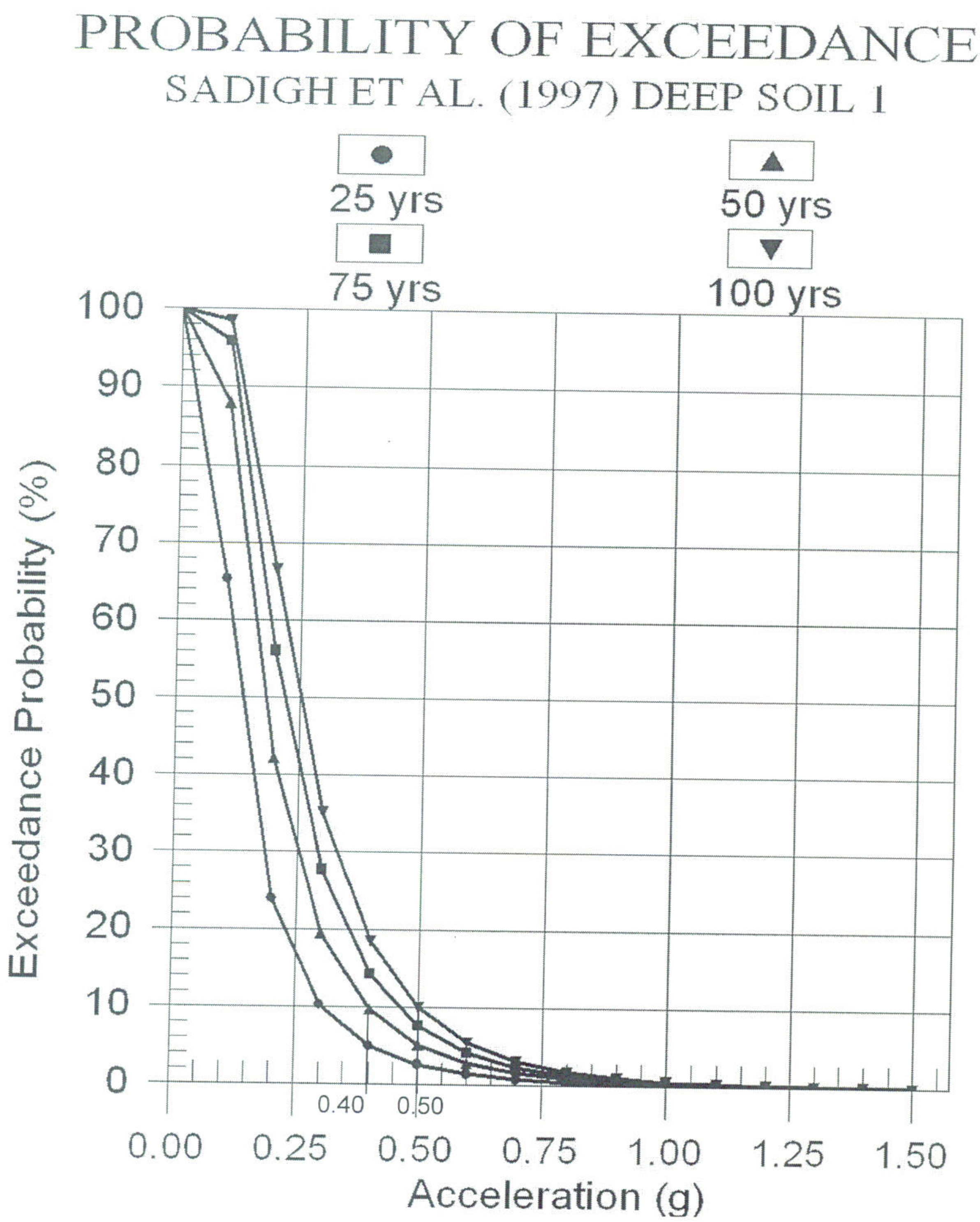
BRG		8000

CALIFORNIA SEISMICITY MAP

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 4



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

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

BRG		8000

PROBABILITY OF EXCEEDANCE

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.

COLLEGE COUNTRY CENTER

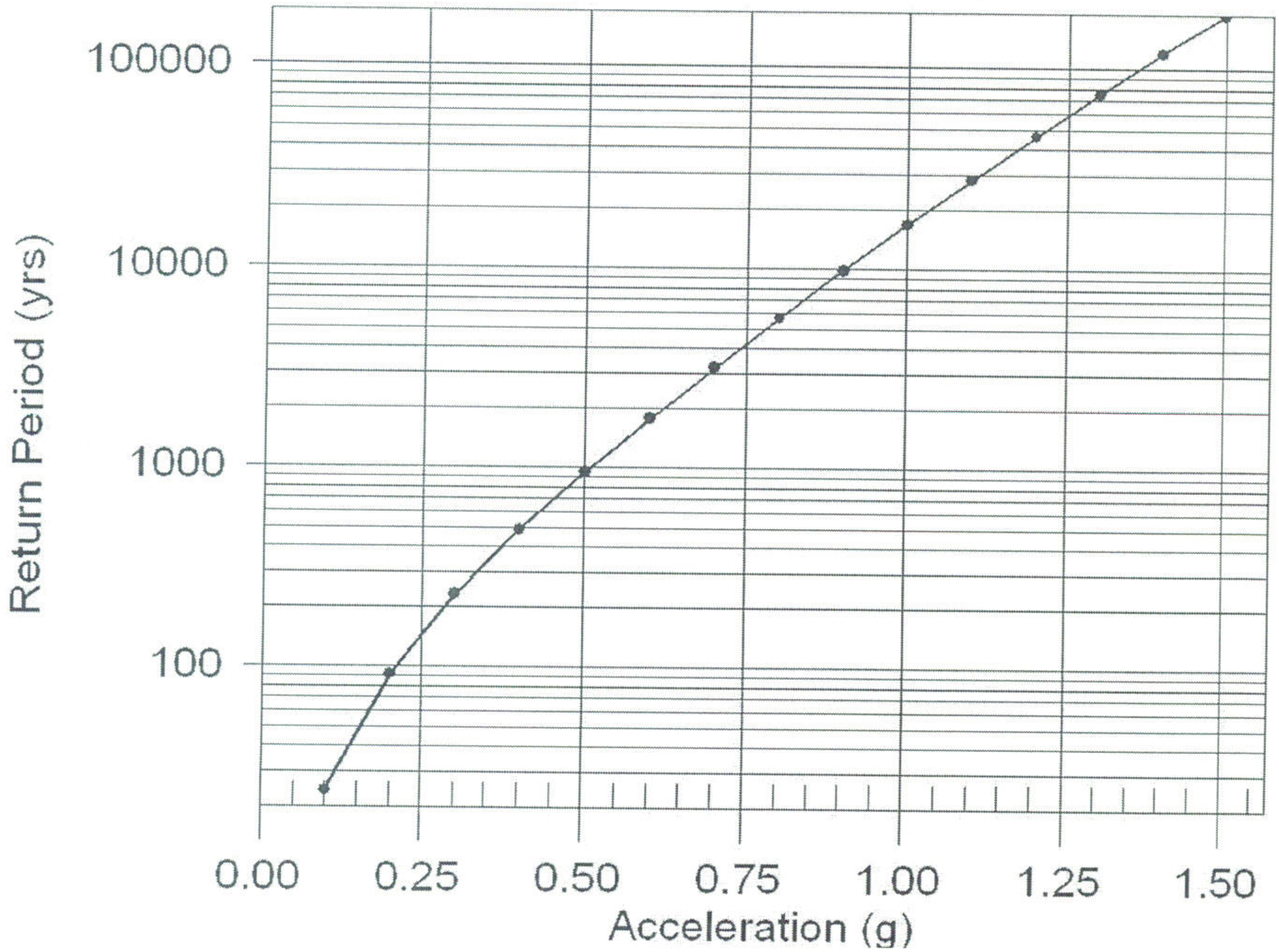
7302-7400 CENTER AVENUE

HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 5

RETURN PERIOD vs. ACCELERATION

SADIGH ET AL. (1997) DEEP SOIL 1



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BRG		8000

RETURN PERIOD vs ACCELERATION

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.

COLLEGE COUNTRY CENTER

7302-7400 CENTER AVENUE

HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 6

Client: AMSTAR/RED OAK HUNTINGTON BEACH
File No. A8481-06-01
Boring 5

GEOCON

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.5
Peak Horiz. Acceleration (g):	0.40
Calculated Mag.Wtg.Factor:	1.005
Historic High Groundwater:	5.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQ2_30.WQ1

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	136.2	0	3.0	5.0	1		37	2.000	6.8	136.2	0.079	0.998	0.261	--
2.0	136.2	0	3.0	5.0	1		37	2.000	6.8	136.2	0.079	0.993	0.259	--
3.0	136.2	0	3.0	5.0	1		37	2.000	6.8	136.2	0.079	0.989	0.258	--
4.0	136.2	0	3.0	5.0	1		37	2.000	6.8	136.2	0.079	0.984	0.257	--
5.0	136.2	1	3.0	5.0	0	86		1.895	13.4	73.8	~	0.979	0.270	~
6.0	136.2	1	3.0	5.0	0	86		1.785	13.0	73.8	~	0.975	0.291	~
7.0	120.2	1	14.0	7.5	0	10		1.701	28.0	57.8	~	0.970	0.308	~
8.0	120.2	1	14.0	7.5	0	10		1.637	27.0	57.8	~	0.966	0.323	~
9.0	120.2	1	1.0	10.0	0	10		1.580	2.9	57.8	~	0.961	0.335	~
10.0	120.2	1	1.0	10.0	0	89		1.528	8.7	57.8	~	0.957	0.346	~
11.0	120.2	1	1.0	10.0	0	89		1.481	8.7	57.8	~	0.952	0.355	~
12.0	123.1	1	5.0	12.5	0	89		1.437	15.1	60.7	~	0.947	0.362	~
13.0	123.1	1	5.0	12.5	0	89		1.395	14.8	60.7	~	0.943	0.368	~
14.0	123.1	1	5.0	12.5	0	89		1.357	14.6	60.7	~	0.938	0.373	~
15.0	123.1	1	5.0	15.0	0	98		1.322	15.0	60.7	~	0.934	0.378	~
16.0	123.1	1	5.0	15.0	0	98		1.290	14.8	60.7	~	0.929	0.382	~
17.0	124.3	1	7.0	17.5	1	48	50	1.260	18.3	61.9	0.199	0.925	0.385	0.52
18.0	124.3	1	7.0	17.5	1	48	50	1.231	18.1	61.9	0.196	0.920	0.387	0.51
19.0	124.3	1	3.0	20.0	0	80		1.204	11.9	61.9	~	0.915	0.389	~
20.0	124.3	1	3.0	20.0	0	80		1.179	11.7	61.9	~	0.911	0.391	~
21.0	124.3	1	3.0	20.0	0	80		1.156	11.7	61.9	~	0.906	0.393	~
22.0	120.4	1	5.0	22.5	0	80		1.134	14.9	58.0	~	0.902	0.394	~
23.0	120.4	1	5.0	22.5	0	80		1.114	14.8	58.0	~	0.897	0.395	~
24.0	120.4	1	5.0	22.5	0	80		1.096	14.6	58.0	~	0.893	0.396	~
25.0	120.4	1	6.0	25.0	0	86		1.078	16.3	58.0	~	0.888	0.397	~
26.0	120.4	1	6.0	25.0	0	86		1.061	16.1	58.0	~	0.883	0.398	~
27.0	120.4	1	6.0	25.0	0	86		1.045	16.0	58.0	~	0.879	0.398	~
28.0	123.5	1	15.0	27.5	0	47		1.029	29.7	61.1	~	0.874	0.398	~
29.0	123.5	1	5.0	30.0	0	89		1.013	14.6	61.1	~	0.870	0.398	~
30.0	123.5	1	5.0	30.0	0	89		0.998	14.5	61.1	~	0.865	0.398	~
31.0	123.5	1	5.0	30.0	0	89		0.984	14.4	61.1	~	0.861	0.397	~
32.0	110.2	1	5.0	32.5	0	89		0.972	14.3	47.8	~	0.856	0.397	~
33.0	110.2	1	5.0	32.5	0	89		0.962	14.2	47.8	~	0.851	0.398	~
34.0	110.2	1	5.0	32.5	0	89		0.951	14.1	47.8	~	0.847	0.398	~
35.0	110.2	1	4.0	35.0	0	99		0.942	12.7	47.8	~	0.842	0.398	~
36.0	110.2	1	4.0	35.0	0	99		0.932	12.6	47.8	~	0.838	0.398	~
37.0	121.0	1	17.0	37.5	0	77		0.922	30.5	58.6	~	0.833	0.397	~
38.0	121.0	1	17.0	37.5	0	77		0.911	30.2	58.6	~	0.829	0.396	~
39.0	121.0	1	11.0	40.0	1	20	54	0.901	18.4	58.6	0.194	0.824	0.395	0.49
40.0	121.0	1	11.0	40.0	1	20	54	0.891	18.2	58.6	0.192	0.819	0.394	0.49
41.0	121.0	1	11.0	40.0	1	20	54	0.881	18.0	58.6	0.190	0.815	0.393	0.48
42.0	116.5	1	9.0	42.5	0	62		0.872	18.8	54.1	~	0.810	0.392	~
43.0	116.5	1	9.0	42.5	0	62		0.864	18.7	54.1	~	0.806	0.391	~
44.0	116.5	1	9.0	42.5	0	62		0.855	18.5	54.1	~	0.801	0.390	~
45.0	116.5	1	10.0	45.0	0	62		0.847	19.7	54.1	~	0.797	0.389	~
46.0	116.5	1	10.0	45.0	0	62		0.840	19.6	54.1	~	0.792	0.388	~
47.0	116.5	1	10.0	45.0	0	62		0.832	19.5	54.1	~	0.787	0.387	~
48.0	116.5	1	10.0	45.0	0	62		0.825	19.4	54.1	~	0.783	0.385	~
49.0	128.4	1	17.0	50.0	0	43		0.817	27.8	66.0	~	0.778	0.384	~
50.0	128.4	1	17.0	50.0	0	43		0.808	27.6	66.0	~	0.774	0.382	~

Client: AMSTAR/RED OAK HUNTINC
File No. A8481-06-01
Boring 5

LIQUEFACTION SETTLEMENT ANALYSIS
AMERICAN SOCIETY OF CIVIL ENGINEERS

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.5
Peak Horiz. Acceleration (g)	0.40
Calculated Mag.Wtg.Factor:	1.005
Historic High Groundwater:	5.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	[illegible]	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [illegible]	EQ. SETTLE. Pe (in.)	
1	3	136.2	0.034	0.034	37	7	0.260	--	0.00	0.00	basement
2	3	136.2	0.102	0.102	37	7	0.260	--	0.00	0.00	basement
3	3	136.2	0.170	0.170	37	7	0.260	--	0.00	0.00	basement
4	3	136.2	0.238	0.238	37	7	0.260	--	0.00	0.00	basement
5	3	136.2	0.306	0.291		13	0.274	~	0.00	0.00	basement
6	3	136.2	0.375	0.328		13	0.297	~	0.00	0.00	basement
7	14	120.2	0.439	0.361		28	0.316	~	0.00	0.00	basement
8	14	120.2	0.499	0.390		27	0.333	~	0.00	0.00	basement
9	1	120.2	0.559	0.418		3	0.347	~	0.00	0.00	basement
10	1	120.2	0.619	0.447		9	0.360	~	0.00	0.00	basement
11	1	120.2	0.679	0.476		9	0.371	~	0.00	0.00	
12	5	123.1	0.740	0.506		15	0.380	~	0.00	0.00	
13	5	123.1	0.801	0.536		15	0.389	~	0.00	0.00	
14	5	123.1	0.863	0.567		15	0.396	~	0.00	0.00	
15	5	123.1	0.925	0.597		15	0.403	~	0.00	0.00	
16	5	123.1	0.986	0.627		15	0.409	~	0.00	0.00	
17	7	124.3	1.048	0.658	50	18	0.414	0.52	1.70	0.20	
18	7	124.3	1.110	0.689	50	18	0.419	0.51	1.70	0.20	
19	3	124.3	1.172	0.720		12	0.423	~	0.00	0.00	
20	3	124.3	1.234	0.751		12	0.427	~	0.00	0.00	
21	3	124.3	1.297	0.782		12	0.431	~	0.00	0.00	
22	5	120.4	1.358	0.812		15	0.435	~	0.00	0.00	
23	5	120.4	1.418	0.841		15	0.439	~	0.00	0.00	
24	5	120.4	1.478	0.870		15	0.442	~	0.00	0.00	
25	6	120.4	1.538	0.899		16	0.445	~	0.00	0.00	
26	6	120.4	1.599	0.928		16	0.448	~	0.00	0.00	
27	6	120.4	1.659	0.957		16	0.451	~	0.00	0.00	
28	15	123.5	1.720	0.986		30	0.453	~	0.00	0.00	
29	5	123.5	1.781	1.017		15	0.455	~	0.00	0.00	
30	5	123.5	1.843	1.048		14	0.457	~	0.00	0.00	
31	5	123.5	1.905	1.078		14	0.459	~	0.00	0.00	
32	5	110.2	1.963	1.105		14	0.462	~	0.00	0.00	
33	5	110.2	2.018	1.129		14	0.465	~	0.00	0.00	
34	5	110.2	2.074	1.153		14	0.468	~	0.00	0.00	
35	4	110.2	2.129	1.177		13	0.470	~	0.00	0.00	
36	4	110.2	2.184	1.201		13	0.473	~	0.00	0.00	
37	17	121	2.242	1.228		31	0.475	~	0.00	0.00	
38	17	121	2.302	1.257		30	0.476	~	0.00	0.00	
39	11	121	2.363	1.286	54	18	0.478	0.49	1.70	0.20	
40	11	121	2.423	1.315	54	18	0.479	0.49	1.70	0.20	
41	11	121	2.484	1.345	54	18	0.480	0.48	1.70	0.20	
42	9	116.5	2.543	1.373		19	0.482	~	0.00	0.00	
43	9	116.5	2.601	1.400		19	0.483	~	0.00	0.00	
44	9	116.5	2.659	1.427		19	0.485	~	0.00	0.00	
45	10	116.5	2.718	1.454		20	0.486	~	0.00	0.00	
46	10	116.5	2.776	1.481		20	0.487	~	0.00	0.00	
47	10	116.5	2.834	1.508		19	0.489	~	0.00	0.00	
48	10	116.5	2.892	1.535		19	0.490	~	0.00	0.00	
49	17	128.4	2.954	1.565		28	0.491	~	0.00	0.00	
50	17	128.4	3.018	1.598		28	0.491	~	0.00	0.00	

TOTAL SETTLEMENT = 1.0 INCHES

Figure 8

GEOCON

CLIENT: AMSTAR/RED OAK HUNTINGTON BEACH, LLC
FILE NO. A8481-06-01 Boring 5

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

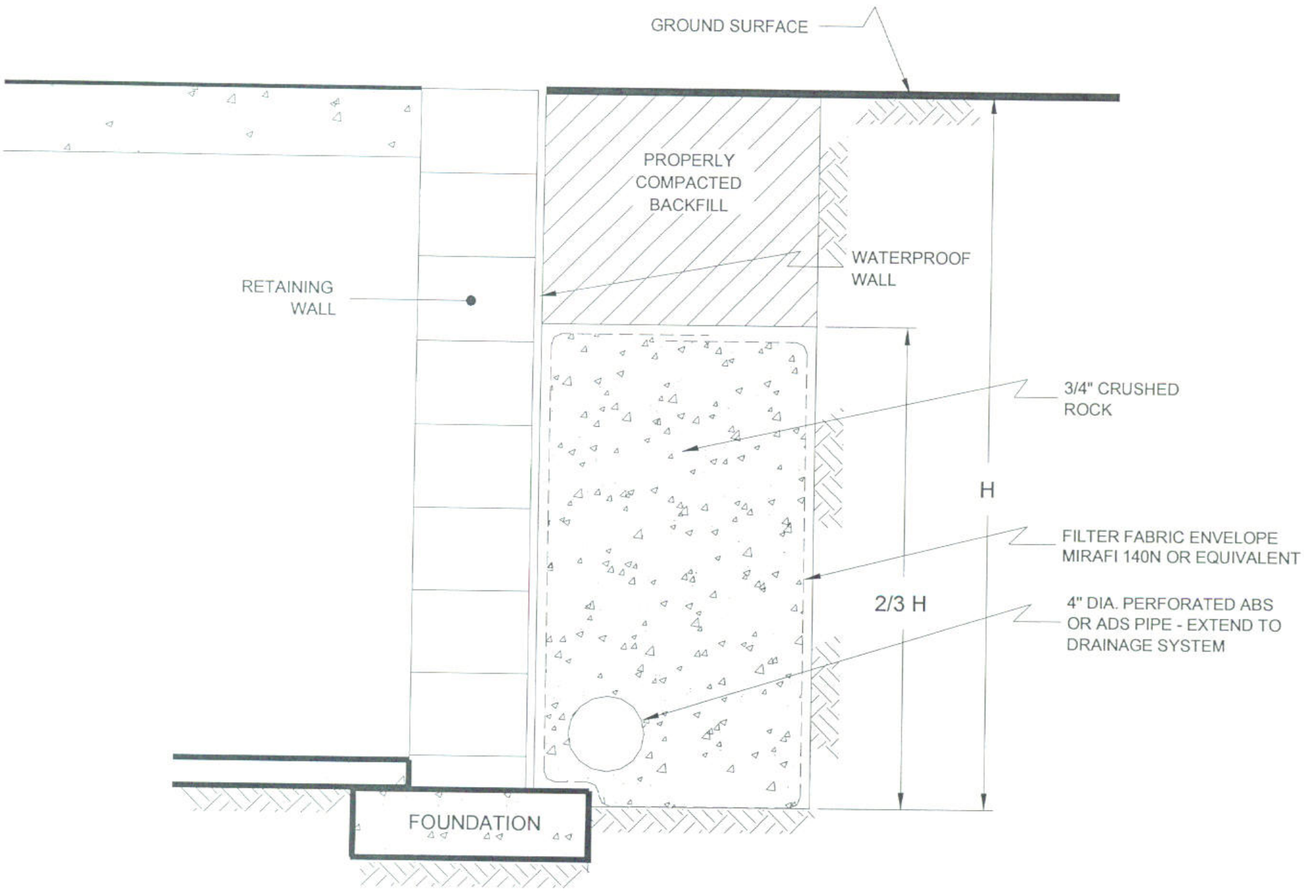
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.5
Peak Horiz. Acceleration (g):	0.40

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer] (Fig 4.1)	Relative Density [Dr] (%)	Correction Factor [Cn] (Fig 4.2)	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	yeff Shear Strain (Fig 4.4)	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	136.2	0.03	0.02	0.009	3	1.25	37.5	2.0	6.8	1.0	127.596	6.87E-05	1.00E-04	0.010	3.68E-02	15.0000	3.68E-02	0.00
2.0	1.0	1.5	136.2	0.10	0.07	0.027	3	1.25	37.5	2.0	6.8	1.0	221.003	1.17E-04	2.30E-04	0.023	8.47E-02	15.0000	8.47E-02	0.00
3.0	1.0	2.5	136.2	0.17	0.11	0.044	3	1.25	37.5	2.0	6.8	1.0	285.314	1.48E-04	1.70E-04	0.017	6.26E-02	15.0000	6.26E-02	0.00
4.0	1.0	3.5	136.2	0.24	0.16	0.062	3	1.25	37.5	2.0	6.8	1.0	337.588	1.71E-04	1.70E-04	0.017	6.26E-02	15.0000	6.26E-02	0.00
5.0	1.0	4.5	136.2	0.31	0.21	0.080	3	1.25	37.5	1.9	13.4	1.0	481.021	1.52E-04	1.50E-04	0.015	2.43E-02	15.0000	2.43E-02	0.00
6.0	1.0	5.5	136.2	0.37	0.25	0.097	3	1.25	37.5	1.8	13.0	1.0	526.837	1.66E-04	1.50E-04	0.015	2.51E-02	15.0000	2.51E-02	0.00
7.0	1.0	6.5	120.2	0.44	0.29	0.114	14	1.25	78.0	1.7	28.0	1.0	735.534	1.37E-04	1.50E-04	0.015	1.00E-02	15.0000	1.00E-02	0.00
8.0	1.0	7.5	120.2	0.50	0.33	0.129	14	1.25	78.0	1.6	27.0	1.0	774.716	1.45E-04	1.50E-04	0.015	1.05E-02	15.0000	1.05E-02	0.00
9.0	1.0	8.5	120.2	0.56	0.37	0.144	1	1.25	20.4	1.6	2.9	1.0	392.001	3.15E-04	1.00E-03	0.100	9.97E-01	15.0000	9.97E-01	0.00
10.0	1.0	9.5	120.2	0.62	0.41	0.159	1	1.25	20.4	1.5	8.7	1.0	592.455	2.27E-04	4.50E-04	0.045	1.22E-01	15.0000	1.22E-01	0.00
11.0	1.0	10.5	120.2	0.68	0.45	0.175	1	1.25	20.4	1.5	8.7	1.0	619.293	2.33E-04	4.50E-04	0.045	1.23E-01	15.0000	1.23E-01	0.03
12.0	1.0	11.5	123.1	0.74	0.50	0.190	5	1.25	44.3	1.4	15.1	0.9	777.556	1.99E-04	1.50E-04	0.015	2.10E-02	15.0000	2.10E-02	0.01
13.0	1.0	12.5	123.1	0.80	0.54	0.205	5	1.25	44.3	1.4	14.8	0.9	805.081	2.04E-04	3.70E-04	0.037	5.29E-02	15.0000	5.29E-02	0.01
14.0	1.0	13.5	123.1	0.86	0.58	0.220	5	1.25	44.3	1.4	14.6	0.9	831.408	2.09E-04	3.70E-04	0.037	5.38E-02	15.0000	5.38E-02	0.01
15.0	1.0	14.5	123.1	0.92	0.62	0.235	5	1.25	43.5	1.3	15.0	0.9	867.625	2.11E-04	3.70E-04	0.037	5.23E-02	15.0000	5.23E-02	0.01
16.0	1.0	15.5	123.1	0.99	0.66	0.250	5	1.25	43.5	1.3	14.8	0.9	892.122	2.15E-04	3.70E-04	0.037	5.31E-02	15.0000	5.31E-02	0.01
17.0	1.0	16.5	124.3	1.05	0.70	0.265	7	1.25	50.0	1.3	18.3	0.9	987.299	2.03E-04	3.70E-04	0.037	4.11E-02	15.0000	4.11E-02	0.01
18.0	1.0	17.5	124.3	1.11	0.74	0.280	7	1.25	50.0	1.2	18.1	0.9	1011.378	2.07E-04	3.70E-04	0.037	4.18E-02	15.0000	4.18E-02	0.01
19.0	1.0	18.5	124.3	1.17	0.79	0.295	3	1.25	32.2	1.2	11.9	0.9	903.147	2.40E-04	3.70E-04	0.037	6.93E-02	15.0000	6.93E-02	0.02
20.0	1.0	19.5	124.3	1.23	0.83	0.309	3	1.25	32.2	1.2	11.7	0.9	924.138	2.43E-04	3.70E-04	0.037	7.01E-02	15.0000	7.01E-02	0.02
21.0	1.0	20.5	124.3	1.30	0.87	0.324	3	1.25	32.2	1.2	11.7	0.9	944.559	2.46E-04	3.70E-04	0.037	7.07E-02	15.0000	7.07E-02	0.02
22.0	1.0	21.5	120.4	1.36	0.91	0.337	5	1.25	40.5	1.1	14.9	0.9	1048.856	2.28E-04	3.70E-04	0.037	5.27E-02	15.0000	5.27E-02	0.01
23.0	1.0	22.5	120.4	1.42	0.95	0.351	5	1.25	40.5	1.1	14.8	0.9	1068.553	2.30E-04	3.70E-04	0.037	5.33E-02	15.0000	5.33E-02	0.01
24.0	1.0	23.5	120.4	1.48	0.99	0.364	5	1.25	40.5	1.1	14.6	0.9	1087.778	2.32E-04	3.70E-04	0.037	5.39E-02	15.0000	5.39E-02	0.01
25.0	1.0	24.5	120.4	1.54	1.03	0.377	6	1.25	43.7	1.1	16.3	0.9	1149.828	2.24E-04	3.00E-04	0.030	3.84E-02	15.0000	3.84E-02	0.01
26.0	1.0	25.5	120.4	1.60	1.07	0.390	6	1.25	43.7	1.1	16.1	0.9	1168.594	2.26E-04	3.00E-04	0.030	3.89E-02	15.0000	3.89E-02	0.01
27.0	1.0	26.5	120.4	1.66	1.11	0.403	6	1.25	43.7	1.0	16.0	0.9	1186.957	2.27E-04	3.00E-04	0.030	3.93E-02	15.0000	3.93E-02	0.01
28.0	1.0	27.5	123.5	1.72	1.15	0.416	15	1.25	67.5	1.0	29.7	0.9	1485.275	1.85E-04	1.30E-04	0.013	8.10E-03	15.0000	8.10E-03	0.00
29.0	1.0	28.5	123.5	1.78	1.19	0.428	5	1.25	38.4	1.0	14.6	0.9	1193.533	2.35E-04	3.00E-04	0.030	4.38E-02	15.0000	4.38E-02	0.01
30.0	1.0	29.5	123.5	1.84	1.23	0.441	5	1.25	38.4	1.0	14.5	0.9	1210.941	2.36E-04	3.00E-04	0.030	4.42E-02	15.0000	4.42E-02	0.01
31.0	1.0	30.5	123.5	1.90	1.28	0.453	5	1.25	38.4	1.0	14.4	0.9	1228.024	2.37E-04	3.00E-04	0.030	4.46E-02	15.0000	4.46E-02	0.01
32.0	1.0	31.5	110.2	1.96	1.32	0.464	5	1.25	37.7	1.0	14.3	0.9	1244.066	2.37E-04	3.00E-04	0.030	4.49E-02	15.0000	4.49E-02	0.01
33.0	1.0	32.5	110.2	2.02	1.35	0.475	5	1.25	37.7	1.0	14.2	0.9	1259.116	2.37E-04	3.00E-04	0.030	4.52E-02	15.0000	4.52E-02	0.01
34.0	1.0	33.5	110.2	2.07	1.39	0.485	5	1.25	37.7	1.0	14.1	0.8	1273.933	2.37E-04	3.00E-04	0.030	4.55E-02	15.0000	4.55E-02	0.01
35.0	1.0	34.5	110.2	2.13	1.43	0.495	4	1.25	33.3	0.9	12.7	0.8	1243.852	2.46E-04	3.00E-04	0.030	5.20E-02	15.0000	5.20E-02	0.01
36.0	1.0	35.5	110.2	2.18	1.46	0.504	4	1.25	33.3	0.9	12.6	0.8	1257.969	2.46E-04	3.00E-04	0.030	5.23E-02	15.0000	5.23E-02	0.01
37.0	1.0	36.5	121.0	2.24	1.50	0.514	17	1.25	67.5	0.9	30.5	0.8	1711.842	1.83E-04	1.30E-04	0.013	7.83E-03	15.0000	7.83E-03	0.00
38.0	1.0	37.5	121.0	2.30	1.54	0.525	17	1.25	67.5	0.9	30.2	0.8	1729.549	1.83E-04	1.30E-04	0.013	7.92E-03	15.0000	7.92E-03	0.00
39.0	1.0	38.5	121.0	2.36	1.58	0.535	11	1.25	53.6	0.9	18.4	0.8	1483.789	2.16E-04	3.00E-04	0.030	3.32E-02	15.0000	3.32E-02	0.01
40.0	1.0	39.5	121.0	2.42	1.62	0.545	11	1.25	53.6	0.9	18.2	0.8	1498.113	2.16E-04	3.00E-04	0.030	3.36E-02	15.0000	3.36E-02	0.01
41.0	1.0	40.5	121.0	2.48	1.66	0.555	11	1.25	53.6	0.9	18.0	0.8	1512.214	2.16E-04	3.00E-04	0.030	3.40E-02	15.0000	3.40E-02	0.01
42.0	1.0	41.5	116.5	2.54	1.70	0.564	9	1.25	47.6	0.9	18.8	0.8	1550.663	2.13E-04	3.00E-04	0.030	3.24E-02	15.0000	3.24E-02	0.01
43.0	1.0	42.5	116.5	2.60	1.74	0.573	9	1.25	47.6	0.9	18.7	0.8	1565.133	2.13E-04	3.00E-04	0.030	3.26E-02	15.0000	3.26E-02	0.01
44.0	1.0	43.5	116.5	2.66	1.78	0.581	9	1.25	47.6	0.9	18.5	0.8	1579.416	2.13E-04	3.00E-04	0.030	3.28E-02	15.0000	3.28E-02	0.01
45.0	1.0	44.5	116.5	2.72	1.82	0.590	10	1.25	49.6	0.8	19.7	0.8	1629.320	2.08E-04	3.00E-04	0.030	3.05E-02	15.0000	3.05E-02	0.01
46.0	1.0	45.5	116.5	2.78	1.86	0.598	10	1.25	49.6	0.8	19.6	0.8	1643.435	2.07E-04	3.00E-04	0.030	3.07E-02	15.0000	3.07E-02	0.01
47.0	1.0	46.5	116.5	2.83	1.90	0.606	10	1.25	49.6	0.8	19.5	0.8	1657.378	2.07E-04	3.00E-04	0.030	3.10E-02	15.0000	3.10E-02	0.01
48.0	1.0	47.5	116.5	2.89	1.94	0.614	10	1.25	49.6	0.8	19.4	0.8	1671.153	2.07E-04	3.00E-04	0.030	3.12E-02	15.0000	3.12E-02	0.01
49.0	1.0	48.5	128.4	2.95	1.98	0.622	17	1.25	62.9	0.8	27.8	0.8	1905.488	1.83E-04	1.30E-04	0.013	8.75E-03	15.0000	8.75E-03	0.00
50.0	1.0	49.5	128.4	3.02	2.02	0.630	17	1.25	62.9	0.8	27.6	0.8	1921.086	1.83E-04	1.00E-02	1.000	6.79E-01	15.0000	6.79E-01	0.16

TOTAL SETTLEMENT = **0.56**

Figure 9



NO SCALE

GEOCON
INLAND EMPIRE, INC.

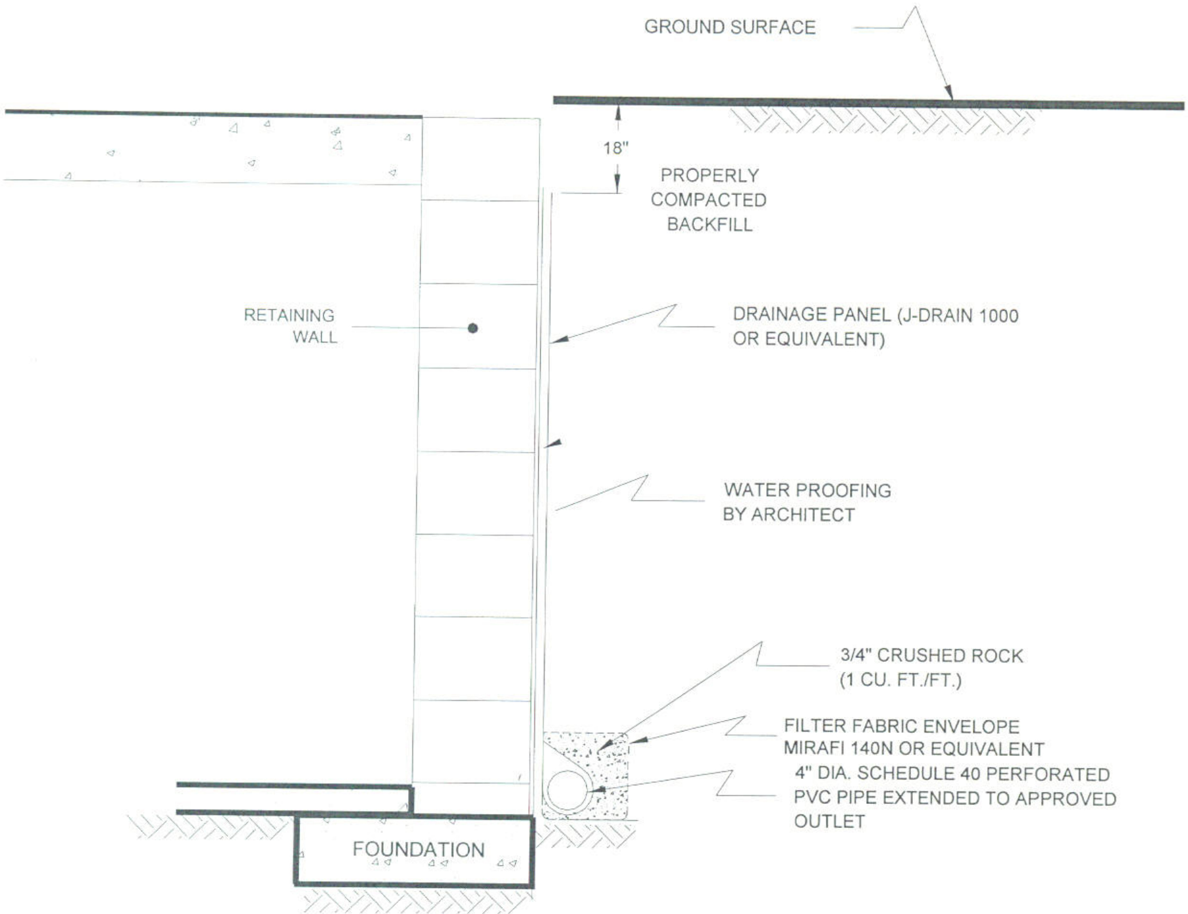
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

NDB		8000

RETAINING WALL DRAIN DETAIL

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 10



NO SCALE

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

NDB		8000

RETAINING WALL DRAIN DETAIL

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. 11

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.
NEWPORT-INGLEWOOD (L.A.Basin)	3.0	(4.9)	6.9	0.631	X
COMPTON THRUST	4.6	(7.4)	6.8	0.691	XI
NEWPORT-INGLEWOOD (Offshore)	11.0	(17.7)	6.9	0.327	IX
ELYSIAN PARK THRUST	11.1	(17.8)	6.7	0.393	X
PALOS VERDES	12.5	(20.1)	7.1	0.319	IX
WHITTIER	15.7	(25.3)	6.8	0.234	IX
CHINO-CENTRAL AVE. (Elsinore)	21.7	(34.9)	6.7	0.208	VIII
SAN JOSE	22.2	(35.8)	6.5	0.183	VIII
ELSINORE-GLEN IVY	22.4	(36.1)	6.8	0.164	VIII
RAYMOND	28.3	(45.6)	6.5	0.137	VIII
VERDUGO	28.9	(46.5)	6.7	0.150	VIII
SIERRA MADRE	29.0	(46.7)	7.0	0.174	VIII
HOLLYWOOD	29.8	(47.9)	6.4	0.120	VII
CLAMSHELL-SAWPIT	30.7	(49.4)	6.5	0.124	VII
CUCAMONGA	31.3	(50.3)	7.0	0.160	VIII
CORONADO BANK	32.4	(52.1)	7.4	0.156	VIII
SANTA MONICA	33.6	(54.1)	6.6	0.117	VII
MALIBU COAST	36.9	(59.4)	6.7	0.111	VII
ELSINORE-TEMECULA	38.0	(61.1)	6.8	0.088	VII
SIERRA MADRE (San Fernando)	41.3	(66.4)	6.7	0.096	VII
NORTHRIDGE (E. Oak Ridge)	42.0	(67.6)	6.9	0.106	VII
SAN GABRIEL	43.5	(70.0)	7.0	0.083	VII
ANACAPA-DUME	43.6	(70.1)	7.3	0.133	VIII
SAN JACINTO-SAN BERNARDINO	44.8	(72.1)	6.7	0.067	VI
SAN ANDREAS - Mojave	47.8	(76.9)	7.1	0.080	VII
SAN ANDREAS - 1857 Rupture	47.8	(76.9)	7.8	0.131	VIII
SAN ANDREAS - Southern	48.0	(77.2)	7.4	0.099	VII
SAN ANDREAS - San Bernardino	48.0	(77.2)	7.3	0.092	VII
SAN JACINTO-SAN JACINTO VALLEY	48.0	(77.3)	6.9	0.069	VI
SANTA SUSANA	49.8	(80.1)	6.6	0.070	VI
CLEGHORN	50.4	(81.1)	6.5	0.050	VI
ROSE CANYON	53.2	(85.6)	6.9	0.060	VI
HOLSER	55.4	(89.1)	6.5	0.056	VI
NORTH FRONTAL FAULT ZONE (West)	57.2	(92.1)	7.0	0.075	VII
SIMI-SANTA ROSA	59.9	(96.4)	6.7	0.058	VI

35 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE NEWPORT-INGLEWOOD (L.A. Basin) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.0 MILES (4.9 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6910 g

APPENDIX A

FIELD INVESTIGATION

The scope of the field investigation, performed on October 27, 2006 consisted of excavating six, 7-inch diameter borings utilizing a hollow stem-auger drilling machine. The borings were conducted to depths between 20½ and 50½ 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 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. Standard Penetration Tests were performed in the 50-foot boring and bulk samples were also obtained.

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-6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are shown on the Site Plan, (Figure 2).

GEOCON
INLAND EMPIRE, INC

BORING 1

Project No.: A8481-06-01
Client: AMSTAR/RED OAK HUNT. BEACH, LLC
Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: **October 27, 2006**
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
Bulk	0-5				0		**Asphalt: 4 inches** **Base: 5 inches** **Fill:** Sandy Silt, firm, moist, greenish brown, very fine- to fine-grained, some concrete fragments
					1		
Ring	2	29	25.9	99.4	2		
					3	ML	**Alluvium:** Sandy Silt, firm, moist, dark greenish grey, very fine- to fine-grained
Ring	4	24	23.9	99.5	4		
					5		Silt, firm, moist, dark olive green, slight plasticity
Ring	6	37	23.7	100.4	6		
					7		
Ring	8	26	22.5	98.6	8		trace very fine- to fine-grained Sand
					9		Sandy Silt, soft, moist, olive brown and yellowish brown, very fine- to fine-grained, some thin interbeds of Silty Clay, soft, moist, olive brown and yellowish brown
					10		
Ring	11	11	26.7	91.7	11		
					12		
					13		
					14		increased Sand content, very moist, greenish brown, fine-grained
Ring	15	12	29.1	93.4	15		
					16		
					17		
					18		
					19	CL	Clay, soft, very moist, greenish grey, trace carbon deposits
Ring	20	7	42.2	77.9	20		End boring at 20.5 feet. Fill to 2.5 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.

- Ring Sample from California Modified Sampler

Figure A-1

GEOCON

INLAND EMPIRE, INC

BORING 2

Project No.: A8481-06-01

Client: AMSTAR/RED OAK HUNT. BEACH, LLC

Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: October 27, 2006

Excavation Method: Hollow Stem Auger

Boring Diameter: 7 inches

Sampling Method: Cal-Mod

Hammer Drop: 30 inches

Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	**Surface Condition: Asphalt Paving** Description
					0		**Asphalt: 4 inches** **Base: 5 inches** **Fill:** Sandy Silt, firm, moist, dark olive brown, fine-grained, trace wood fragments
	1	29	19.9	107.4	1		
					2	**ML**	**Alluvium:** Sandy Silt, firm, moist, dark olive brown, fine-grained
	3	30	21.2	107.4	3		
					4		
	5	35	23.2	104.4	5		
					6		decreased Sand content, slight plasticity
					7		
	8	24	24.5	99.7	8	**SM/ML**	Silty Sand to Sandy Silt, medium dense, very moist, light olive brown and yellowish brown, fine-grained
					9		
					10		
					11	**ML**	Silt, soft, very moist, olive brown and yellowish brown, moderately porous, trace peat deposits, moderate plasticity
	12	7	50.9	69.2	12		
					13		
					14		
					15		
					16		
	17	6	29.5	90.5	17		high plasticity, olive brown, dark olive brown and yellowish brown, slightly porous
					18		
					19	**CL**	Clay, soft, very moist, olive brown and yellowish brown, trace peat
	20	10	42.9	78.1	20		End boring at 20.5 feet. Fill to 1.5 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.

- Ring Sample from California Modified Sampler

Figure A-2

GEOCON
INLAND EMPIRE, INC

BORING 3

Project No.:	**A8481-06-01**
Client:	AMSTAR/RED OAK HUNT. BEACH, LLC
Location:	7302 - 7400 Center Avenue Huntington Beach, California

Excavation Date:	**October 27, 2006**
Excavation Method:	Hollow Stem Auger
Boring Diameter:	7 inches
Sampling Method:	Cal-Mod
Hammer Drop:	30 inches
Hammer Weight:	140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	**Surface Condition: Asphalt Paving** **Description**
Bulk	0-5				0		**Asphalt: 5 inches** **Base: 6 inches** **Fill:** Sandy Silt, firm, moist, dark olive brown, fine-grained, trace gravel, trace brick fragments, some wire fragments, possible irrigation
					1		
Ring	2	25	23.4	102.6	2		
					3		
Ring	4	25	22.2	100.6	4		
					5		
					6		trace Silty Sand lenses
Ring	7	26	19.9	109.8	7		
					8	SM	**Alluvium:** Silty Sand, medium dense, moist, yellowish brown, fine-grained
					9		
Ring	10	12	20.7	108.4	10	ML	Sandy Silt, soft, very moist, olive brown, yellowish brown, dark olive brown and light greenish brown, fine-grained, some interbedded Silty Sand, loose, very moist, olive green, olive brown and dark yellowish brown, fine-grained
					11		
					12		
					13		
					14		
Ring	15	18	30.3	93.7	15		
					16		
					17		
					18		
					19	CL	Clay, soft, very moist, dark olive brown and greyish green
Ring	20	8	48.1	72.5	20		End boring at 20.5 feet. Fill to 7.5 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.

(Ring) - Ring Sample from California Modified Sampler

Figure A-3

GEOCON
INLAND EMPIRE, INC

BORING 4

Project No.: A8481-06-01
Client: AMSTAR/RED OAK HUNT. BEACH, LLC
Location: 7302 - 7400 Center Avenue, Huntington Beach, California

Excavation Date: October 27, 2006
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod
Hammer Drop: 30 inches
Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
	--				0		**Asphalt: 4 inches** **Base: 6 inches** **Fill:** Sandy Silt, firm, moist, dark brown, dark olive brown and brown, some gravel, trace wood fragments
Ring	1	21	25.3	97.2	1		
					2		
Ring	3	27	26.1	95.7	3	ML	**Alluvium:** Sandy Silt, firm, moist, olive brown and greenish grey, fine-grained, slight plasticity
					4		
					5		
Ring	6	28	20.5	104.0	6		
					7		yellowish brown and olive brown
					8	SM	Silty Sand, loose, very moist, light olive brown and yellowish brown, fine-grained, trace peat deposits
Ring	9	11	32.6	89.0	9		
					10		
					11		
					12		
					13	SM/ML	Interbedded Sandy Silt and Silty Sand, soft, very moist, dark olive brown, yellowish brown and dark yellow, fine-grained
Ring	14	16	29.6	93.4	14		
					15		
					16		
					17		
					18		
					19	ML	Silt, soft, very moist, dark olive brown and yellowish brown, moderately porous, some carbon deposits, high plasticity
Ring	20	6	35.5	85.5	20		End boring at 20.5 feet. Fill to 3 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.

Ring - Ring Sample from California Modified Sampler

Figure A-4

GEOCON

INLAND EMPIRE, INC

BORING 5

Project No.: A8481-06-01

Client: AMSTAR/RED OAK HUNT. BEACH, LLC

Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: October 27, 2006

Excavation Method: Hollow Stem Auger

Boring Diameter: 7 inches

Sampling Method: Cal-Mod / SPT

Hammer Drop: 30 inches

Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Surface Condition: Asphalt Paving Description
					0		**Asphalt:** **Base:**
					1		**Fill:** Sandy Silt, firm, moist, dark olive brown and dark brown, fine-grained with some medium- and coarse-grained, trace gravel, slight plasticity
	2.5	17	15.8	117.6	2–3		
					4		very moist
	5	3	46.9	-	5		
					6	ML	**Alluvium:** Silt, soft, very moist, dark olive brown and yellowish brown, some carbon deposits, slight plasticity
	7.5	21	24.0	96.9	7–8	SP-SM	Sand with Silt, loose, very moist, light yellowish brown and light brown, fine-grained
					9		groundwater encountered
	10	Push	81.2	-	10–11	ML	Silt, soft, very moist to wet, greyish green, slightly porous, some carbon deposits, some peat deposits
	12.5	8	28.3	95.9	12–13		Sandy Silt, soft, very moist to wet, greyish green, fine-grained, trace peat deposits
	15	5	40.9	-	14–16		Silt, soft, very moist to wet, greyish green, slightly porous, trace peat deposits
	17.5	11	26.8	98.0	17–18	SM	Silty Sand, loose, very moist to wet, greenish grey, fine-grained
	20	Push	40.3	-	19–20	CL	Clay, soft, very moist to wet, greenish grey, trace carbon deposits, trace caliche

- Ring Sample from California Modified Sampler

- Standard Penitrometer Test (SPT) Sampler

Figure A-5a

GEOCON

INLAND EMPIRE, INC

BORING 5 (continued)

Project No.: A8481-06-01

Client: AMSTAR/RED OAK HUNT. BEACH, LLC

Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: October 27, 2006

Excavation Method: Hollow Stem Auger

Boring Diameter: 7 inches

Sampling Method: Cal-Mod / SPT

Hammer Drop: 30 inches

Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					20		
					21	ML	Sandy Silt, soft, wet, greenish grey, trace carbon deposits
	22.5	8	36.2	88.4	22		
					23		Silt, soft, wet, greenish grey, trace carbon deposits, slight plasticity
					24		Sandy Silt, soft, wet, greenish grey, trace carbon deposits
	25	Push	34.2	-	25		
					26		
	27.5	23	28.6	96.0	27		
					28	SM	Silty Sand, medium dense, wet, greenish grey, fine-grained
					29	ML	Silt with Sand, soft, wet, greenish grey, fine-grained, trace peat deposits
	30	Push	36.8	-	30		
					31		
	32.5	7	49.1	74.0	32		Silt, soft, wet, greenish grey, high plasticity
					33		
					34		moderate plasticity
	35	Push	38.3	-	35		
					36		
	37.5	26	29.7	93.3	37	ML	Sandy Silt, firm to stiff, wet, greenish grey, fine-grained
					38		
					39		
	40	11	33.3	-	40	SM	Silty Sand, loose, wet, greenish grey, fine-grained, moderate plasticity, trace carbon deposits

- Ring Sample from California Modified Sampler

- Standard Penitrometer Test (SPT) Sampler

Figure A-5b

GEOCON

INLAND EMPIRE, INC

BORING 5 (continued)

Project No.: A8481-06-01
Client: AMSTAR/RED OAK HUNT. BEACH, LLC
Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: October 27, 2006
Excavation Method: Hollow Stem Auger
Boring Diameter: 7 inches
Sampling Method: Cal-Mod / SPT
Hammer Drop: 30 inches
Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	Description
					40		
					41		
Ring	42.5	14	36.8	85.1	42		some carbon deposits
					43		
					44		
SPT	45	10	23.1	-	45	ML	Sandy Silt, soft, wet, greyish brown, fine-grained
					46		
					47		
					48		
					49		
Ring	50	26	23.6	103.9	50	SM	Silty Sand, medium dense, wet, greenish grey, fine-grained
					51		End boring at 50.5 feet. Fill to 5.5 feet. Groundwater at 8.5 feet. Backfilled and tamped with soil cuttings. Capped with asphalt patch.
					52		
					53		
					54		
					55		
					56		
					57		
					58		
					59		
					60		

- Ring Sample from California Modified Sampler
- Standard Penitrometer Test (SPT) Sampler

Figure A-5c

GEOCON

INLAND EMPIRE, INC

BORING 6

Project No.: A8481-06-01

Client: AMSTAR/RED OAK HUNT. BEACH, LLC

Location: 7302 - 7400 Center Avenue
Huntington Beach, California

Excavation Date: **October 27, 2006**

Excavation Method: Hollow Stem Auger

Boring Diameter: 7 inches

Sampling Method: Cal-Mod

Hammer Drop: 30 inches

Hammer Weight: 140 pounds

Sample Type	Depth (feet)	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	Depth (feet)	USCS Class.	**Surface Condition: Asphalt Paving** Description
Bulk	0-5				0		**Asphalt: 1.5 inches** **Base: 6 inches**
							Fill: Sandy Silt, firm, moist, olive brown and dark brown, fine-grained, some gravel
					1		
Ring	2	16	19.1	108.7	2		
					3	ML	**Alluvium:** Sandy Silt, firm, moist, dark olive brown, fine-grained, slight plasticity
Ring	4	23	24.0	99.7	4		
					5		
					6		
						SM	Silty Sand, loose, moist, light olive brown and yellowish brown, fine-grained
Ring	7	11	33.4	86.8	7		
						ML	Silt, soft, very moist, olive brown and yellowish brown, fine-grained, slight plasticity
					8		
					9		
Ring	10	5	41.1	78.0	10		slightly porous, trace peat deposits
					11		some peat deposits
					12		
					13		
					14		greyish green, trace peat deposits
Ring	15	6	32.3	91.5	15		
					16		
					17		
					18		
					19		
Ring	20	8	45.1	75.5	20		End boring at 20.5 feet. Fill to 2.5 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.

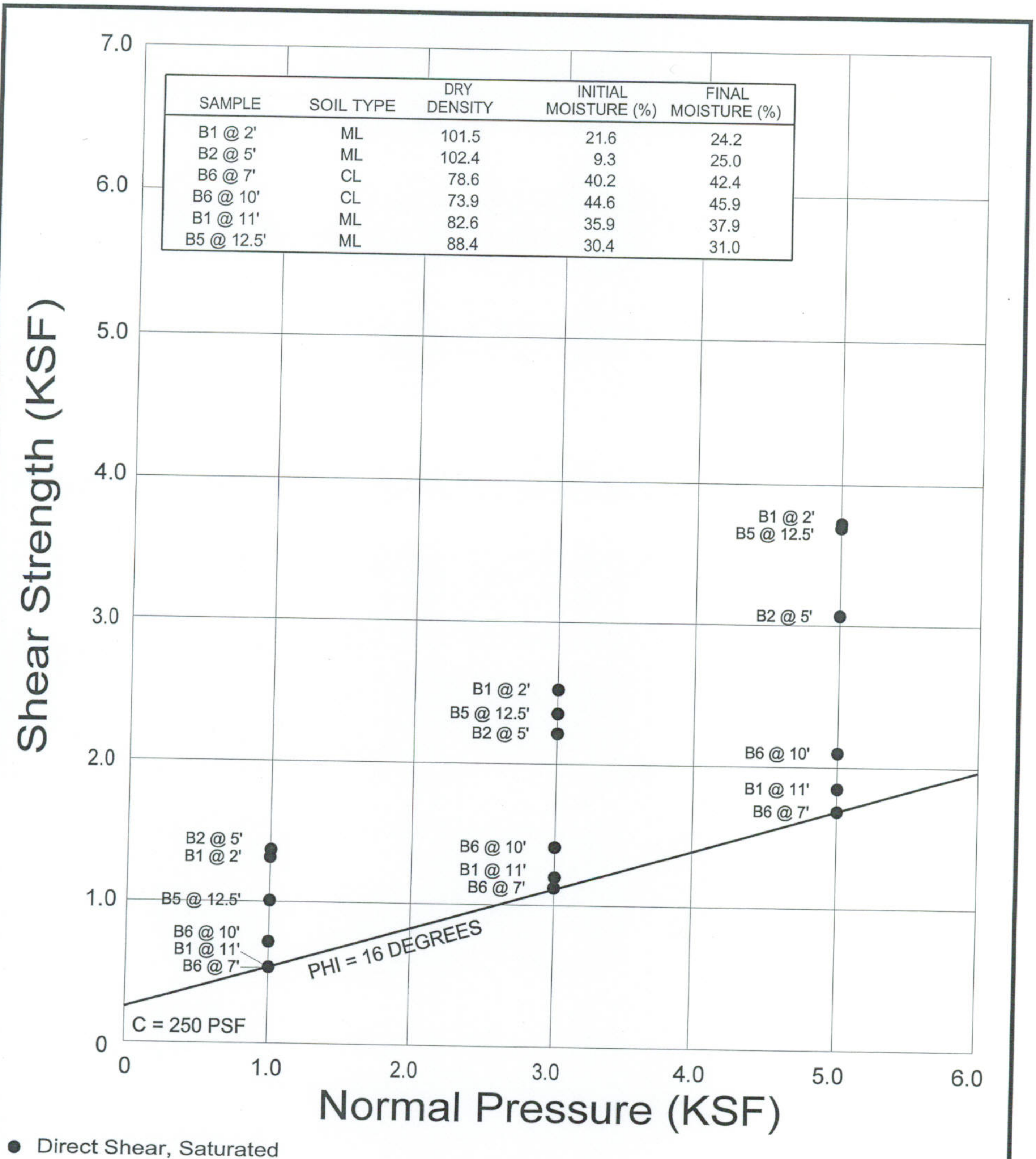
Ring symbol - Ring Sample from California Modified Sampler

Figure A-6

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, grain size, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B10. 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'	ML	101.5	21.6	24.2
B2 @ 5'	ML	102.4	9.3	25.0
B6 @ 7'	CL	78.6	40.2	42.4
B6 @ 10'	CL	73.9	44.6	45.9
B1 @ 11'	ML	82.6	35.9	37.9
B5 @ 12.5'	ML	88.4	30.4	31.0

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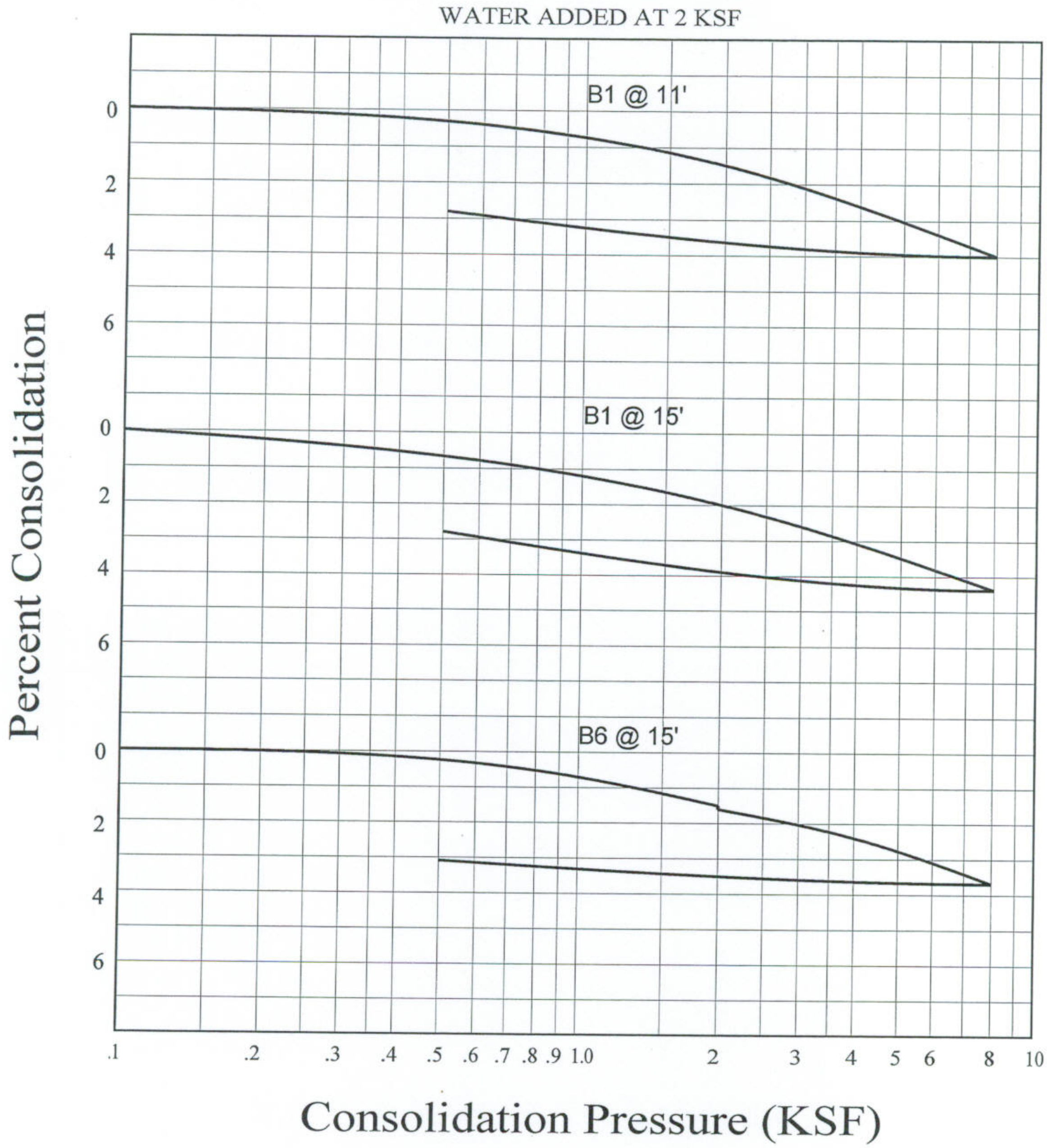
BRG | | 8000

DIRECT SHEAR TEST RESULTS

AMSTAR/RED OAK HUTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006 | PROJECT NO. A8481-06-01 | FIG. B1



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BRG		8000

CONSOLIDATION TEST RESULTS

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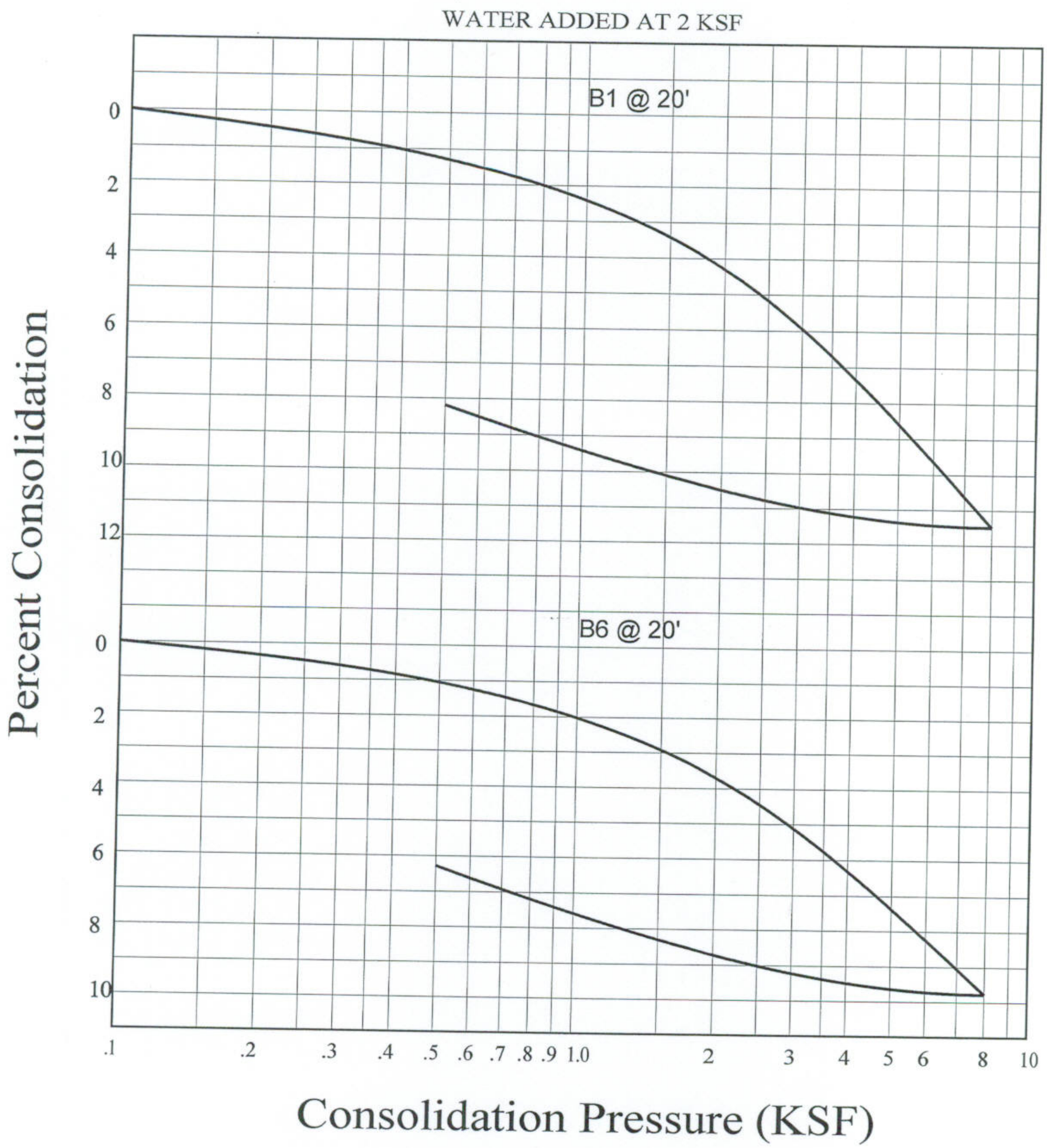
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B2



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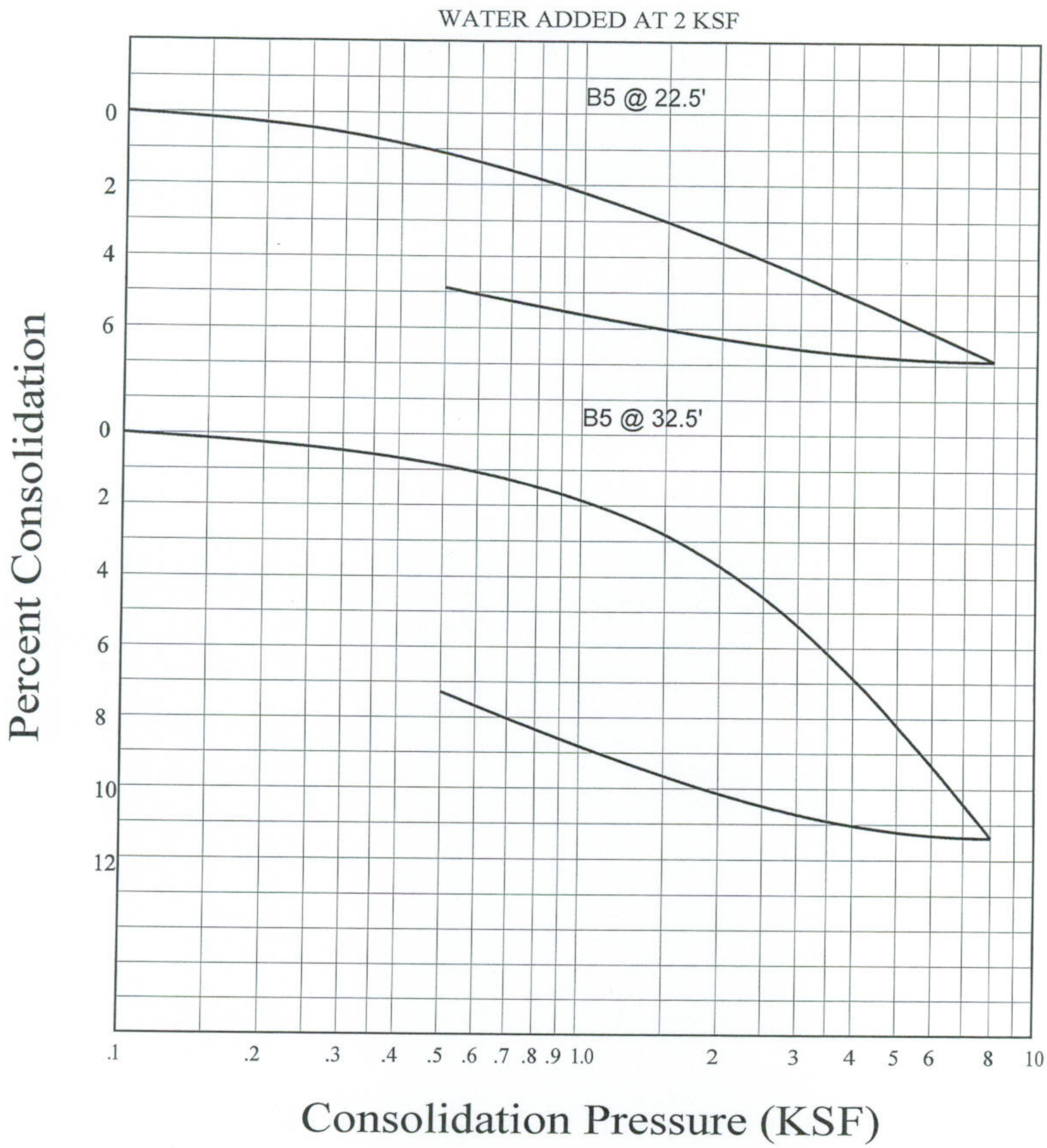
BRG		8000

CONSOLIDATION TEST RESULTS

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B3



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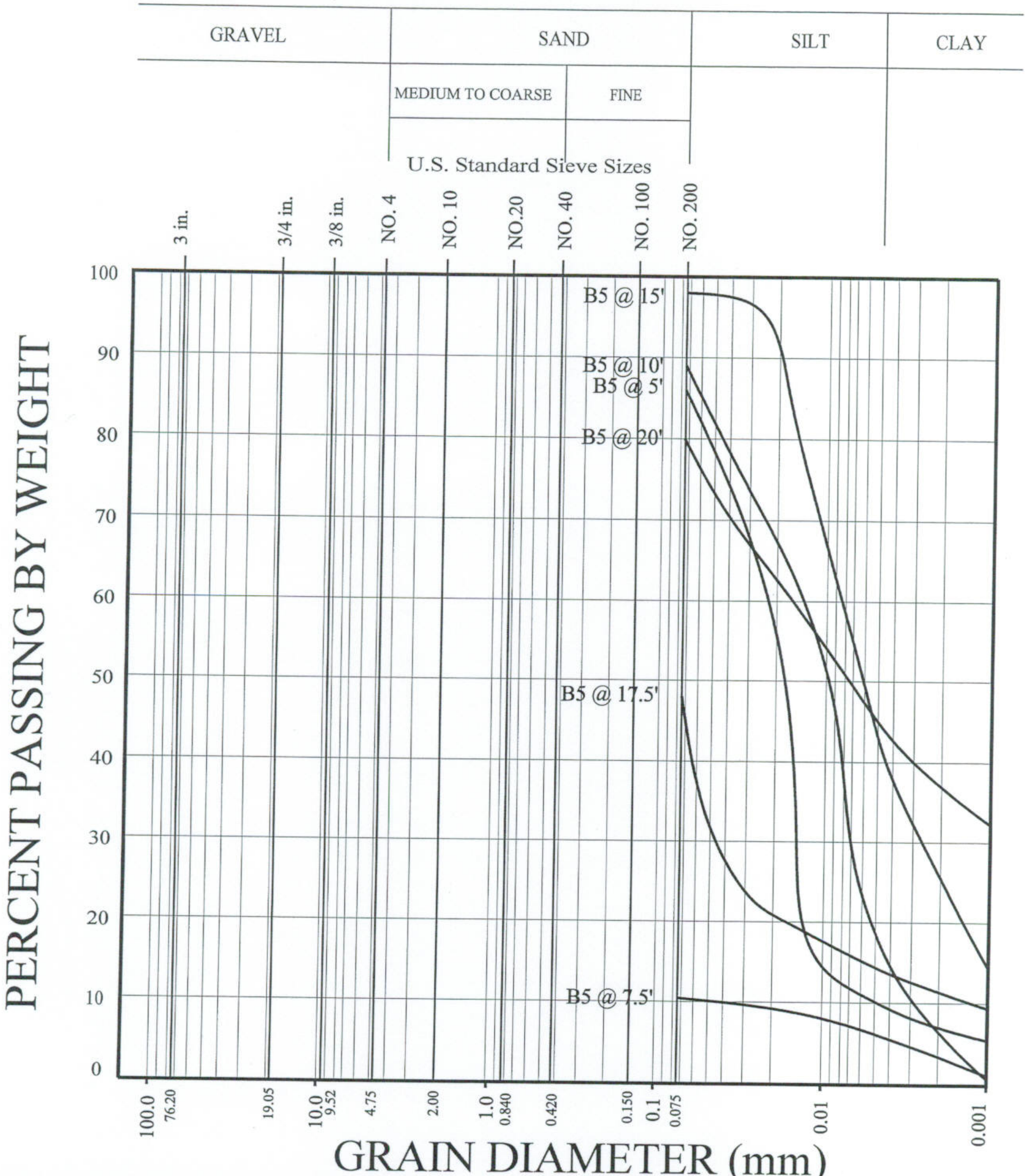
PHONE (818) 841-8388 - FAX (818) 841-1704

BRG		8000

CONSOLIDATION TEST RESULTS

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B4



SAMPLE	UNIFIED SOIL CLASSIFICATION
1- B5 @ 5'	ML
2- B5 @ 7.5'	SM
3- B5 @ 10'	ML

SAMPLE	UNIFIED SOIL CLASSIFICATION
4- B5 @ 15'	ML
5- B5 @ 17.5'	SM
6- B5 @ 20'	ML

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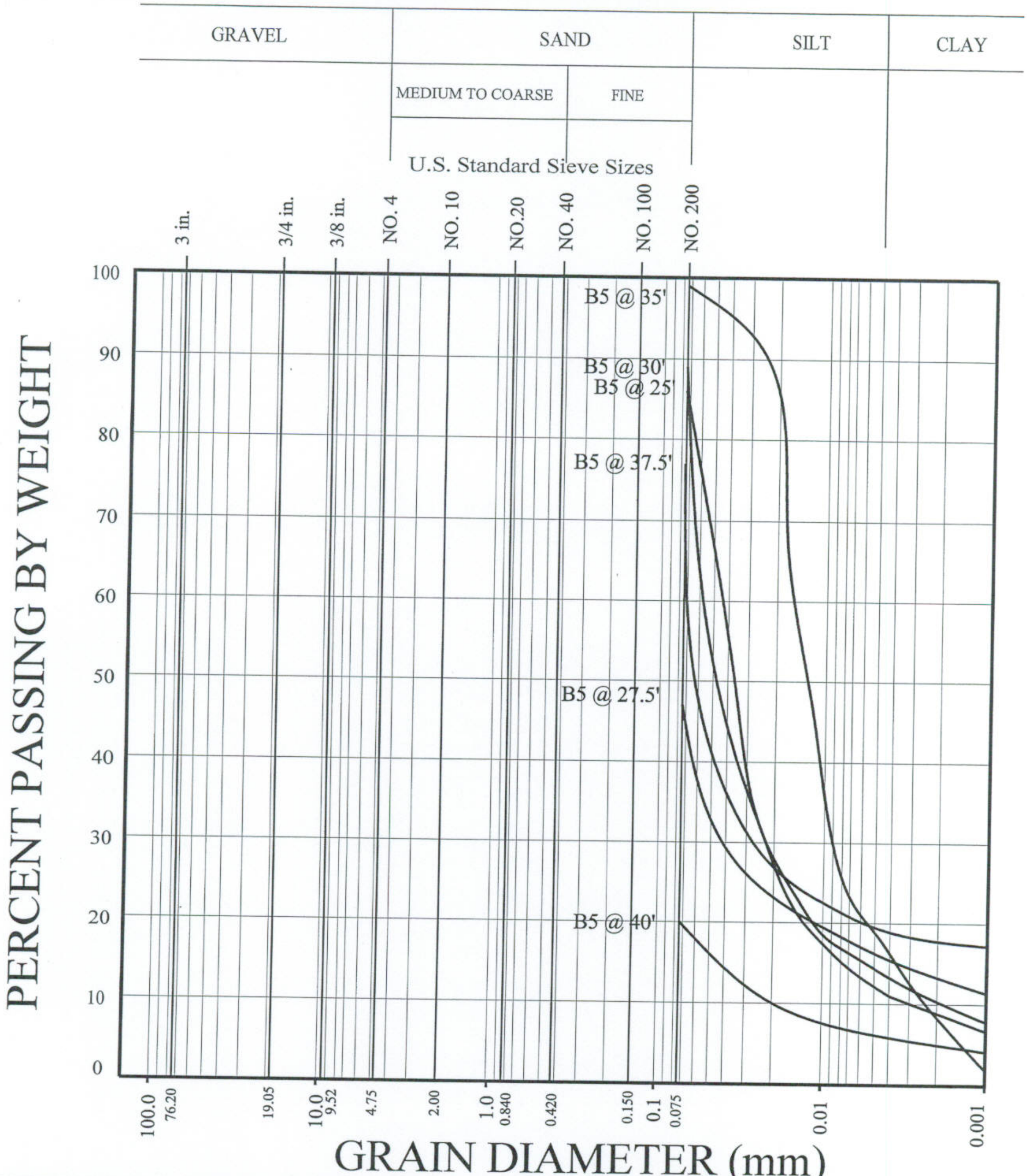
GRAIN SIZE DISTRIBUTION

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006 | PROJECT NO. A8481-06-01 | FIG. B5



SAMPLE	UNIFIED SOIL CLASSIFICATION
1- B5 @ 25'	ML
2- B5 @ 27.5'	SM
3- B5 @ 30'	ML

SAMPLE	UNIFIED SOIL CLASSIFICATION
4- B5 @ 35'	ML
5- B5 @ 37.5'	ML
6- B5 @ 40'	SM

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GRAIN SIZE DISTRIBUTION

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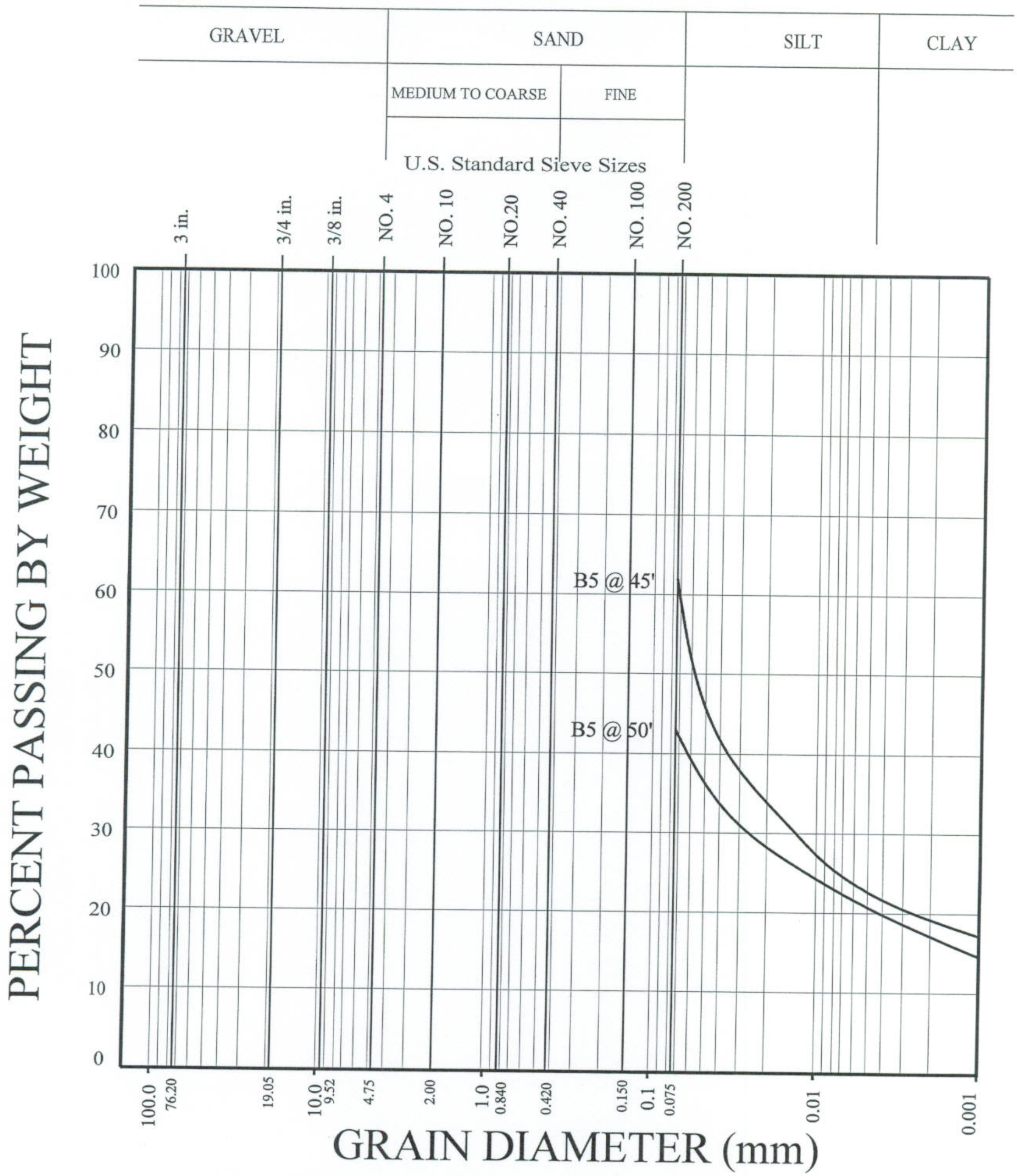
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006 | PROJECT NO. A8481-06-01 | FIG. B6



SAMPLE	UNIFIED SOIL CLASSIFICATION
1- B5 @ 45'	ML
2- B5 @ 50'	SM

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GRAIN SIZE DISTRIBUTION

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COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B7

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

Sample No.	pH	Resistivity (ohm centimeters)
B4 @ 3'	7.6	1200 (Corrosive)

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

Sample No.	Chloride Ion Content (%)
B4 @ 3'	0.007

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B3 @ 10'	0.032	Negligible

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

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CORROSIVITY TEST RESULTS

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B8

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification
	Before	After			
B2 @ 12'	14.3	39.6	94.2	138	Very High

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

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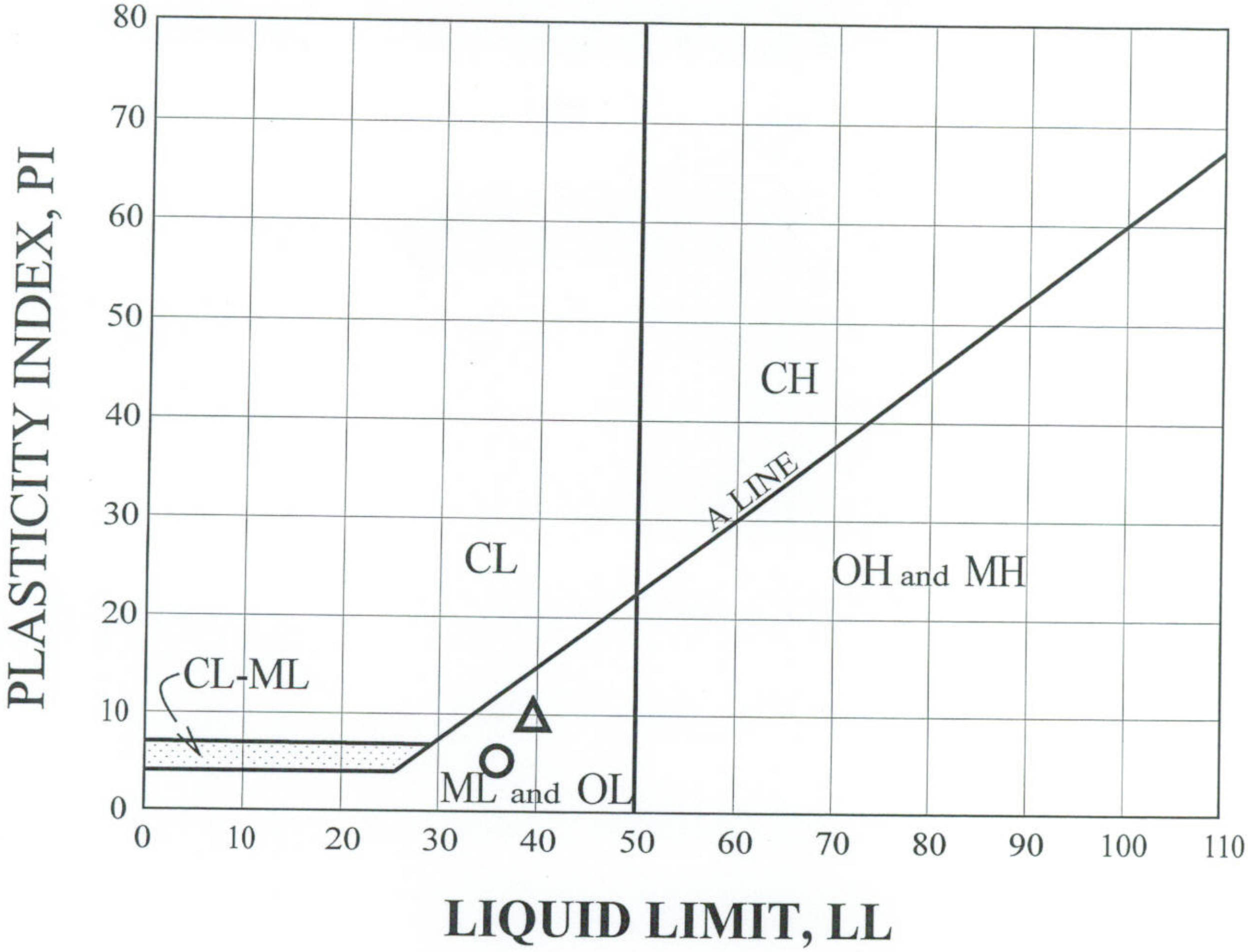
LABORATORY TEST RESULTS

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B5	**25**	○	**36.0**	**30.8**	**5.2**	**ML**
B5	**30**	△	**39.3**	**29.8**	**9.5**	**ML**

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BRG		8000

ATTERBERG LIMITS

AMSTAR/RED OAK HUNTINGTON BEACH, LLC.
COLLEGE COUNTRY CENTER
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

DEC. 12, 2006	PROJECT NO. A8481-06-01	FIG. B10

GEOTECHNICAL INVESTIGATION

PROPOSED COLLEGE COUNTRY
MIXED-USE DEVELOPMENT
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

PREPARED FOR

RED OAK INVESTMENTS, LLC
NEWPORT BEACH, CALIFORNIA

DECEMBER 12, 2006

PROJECT NO. A8481-06-01

Project No. A8481-06-01
December 12, 2006

VIA OVERNIGHT COURIER

Red Oak Investments, LLC
2101 Business Center Drive, #230
Irvine, CA 92612

Attention: Mr. Alex Wong

Subject: PROPOSED COLLEGE COUNTRY MIXED-USE DEVELOPMENT
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Wong:

In accordance with your authorization of our proposal dated September 18, 2006, we have performed a geotechnical investigation for the proposed mixed-use development located at 7302-7400 Center Avenue in the City of Huntington Beach, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of 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 of this report are followed and implemented during 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.

Neal D. Berliner
GE 2576

Gerald A. Kasman
CEG 2251

NDB:GAK:am

(7) Addressee

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a proposed mixed-use development located at 7302-7400 Center Avenue in the City of Huntington Beach, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the property and based on conditions encountered, provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of our investigation included a site reconnaissance, a field investigation, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on October 27, 2006 by drilling six 7-inch diameter borings utilizing a hollow-stem auger drilling machine. The borings were advanced to depths between 20½ and 50½ 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 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 consists of an approximate 3.8-acre, irregular-shaped parcel that is currently occupied by one-story and two-story commercial structures, with asphalt and concrete paved parking lots. The property is bounded by Center Avenue to the north, by electrical transmission towers and a railroad right-of-way on the east, by a single-story commercial building to the south, and by Gothard Street to the west. The subject property is roughly level with no pronounced highs or lows. Surface water drainage at the site appears to be by sheetflow along the ground surface to area drains and city streets.

The proposed development will consist of a multi-story mixed-use development over one to two levels of subterranean parking (see Site Plan, Figure 2). The probable depth of the subterranean parking level is anticipated to be between 10 and 22 feet below the existing ground surface, including footing depths.

Based on the preliminary nature of the design at this time, structural loads were not available. It is anticipated that column loads for the proposed residential structure will be between 50 and 600 kips, and wall loads are estimated to be between 1 and 6 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.

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 over alluvial deposits. The soil and geologic units encountered at the site are discussed below. General profiles are provided on the Boring Logs in Appendix A.

3.1 Artificial Fill

Artificial fill was encountered in all of our site explorations ranging from 1½ to 7½ feet in depth below the existing ground surface. The fill consists primarily of dark brown, very fine-grained sandy silt with traces of gravel and scattered construction debris. The fill is believed to be the result of past grading, construction, and or demolition activities at the site and deeper fill may occur between borings and on other parts of the site that were not directly explored.

3.2 Alluvium

The fill is underlain by Holocene Age alluvial soils consisting of relatively flat-lying layers of silt, sandy silt, silty sand, and clay. The alluvial soils are primarily fine-grained and soft to firm with some loose to medium dense silty sand layers. Minor amounts of peat were also observed in the soils. The soils consist of flood plain deposits and are anticipated to extend to a depth of approximately 90 feet below the existing ground surface (Sprotte, et al., 1980).

4. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Anaheim and Newport Beach Quadrangles (California Division of Mines and Geology, 2001), the historic high groundwater level in the vicinity of the site is approximately 5 feet below the existing ground surface. Groundwater information presented in this report is generated from data collected in the early 1900's to present. Additionally, according to Sprotte et al. (1980), the depth to groundwater at the site is between 5 and 10 feet below grade. Although five borings were drilled on the site, groundwater was only encountered in Boring 5 at a depth of 8 feet beneath the ground surface. It is anticipated that groundwater is present throughout the site; however, the soils are very fine grained and not conducive to high permeability or allowing free flow of water through the alluvial mass. It is anticipated that the majority of groundwater seepage encountered during excavation will emanate from the sand beds within the alluvial mass.

It is not uncommon for groundwater levels to vary seasonally or for groundwater conditions to develop where none previously existed, especially in permeable fine-grained soils which are heavily irrigated or after seasonal rainfall. 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 6.20).

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 subject 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 fault is the Newport-Inglewood Fault Zone which is located approximately 3.1 miles (5.0 kilometers) from the site.

The closest surface projection to an active fault is the Seal Beach segment of the Newport-Inglewood Fault Zone located approximately 3.5 miles southwest of the site. Other nearby active faults are the Palos Verdes Fault, the Whittier Fault Zone, and the Elsinore Fault Zone located 12½ miles southwest, 15½ miles north-northeast, and 21 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 48 miles northeast of the site.

The closest potentially active fault to the site is the Los Alamitos Fault located approximately 5.6 miles to the northwest. Other nearby potentially active faults are the Pelican Hill Fault, the El Modeno Fault, and the Norwalk Fault located 8.5 miles southeast, 8.9 miles southeast, and 10 miles north 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.

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 (1975), 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), modeling the soil underlying the site as a Building Code Soil Profile Type S_E. The resulting calculated peak horizontal accelerations at the site are shown on Table 1. These values are one standard deviation above the mean.

Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 6.8 event on the Compton Thrust Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 0.69g.

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 the 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 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 0.50g and 0.40g, respectively. Graphical representations of the analyses are presented on Figures 5 and 6; however, these accelerations do not take into account potential Blind Thrust Faults.

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 table below are for the Newport-Inglewood 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_E	Table 16-J
Seismic Coefficient, C_a	0.36	Table 16-Q
Seismic Coefficient, C_v	1.15	Table 16-R
Near-Source Factor, N_a	1.0	Table 16-S
Near-Source Factor, N_v	1.2	Table 16-T
Control Period, Ts	1.28	---
Control Period, To	0.26	---
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 Orange County Safety Element (2004), the Environmental Hazard Element of the City of Huntington Beach (1996), and the State of California Seismic Hazard Zone, Newport Beach Quadrangle Map (CDMG, 1997), the site is located within an area identified as having a potential for liquefaction. Based on a review of the Seismic Hazard Zone Report for the Newport Beach 7.5 Minute Quadrangle (CDMG 1997), the historically highest groundwater in the area is approximately 5 feet beneath the ground surface.

Liquefaction analysis of the soils underlying the site was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. The liquefaction potential evaluation was performed by assuming a groundwater table of 5 feet below the surface, a magnitude 7.5 earthquake and a peak horizontal acceleration of 0.40g. The peak horizontal acceleration of 0.40g corresponds to the DBE ground acceleration. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

The enclosed liquefaction analysis for Boring 5 indicates that the site soils would be prone to 1.0 inch of settlement as a result of liquefaction during UBE ground motion (see enclosed calculation sheets, Figures 7 and 8). Recommendations presented in this report are intended to minimize the effects of seismically-induced settlement on the proposed structures.

5.7 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. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9. The calculation is provided herein, and indicates that approximately 0.56 inch of total settlement could be expected as a result of the DBE ground motion (see enclosed calculation sheet, Figure 9). Recommendations presented in this report are intended to minimize the effects of seismically-induced settlement on the proposed structures.

5.8 Landslides

The site and surrounding vicinity is relatively flat with no pronounced slopes. 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.

It is anticipated that the proposed site development will include excavations of 10 to 22 feet below the existing ground surface. The majority of the materials exposed in the excavations will consist of horizontally stratified to massive alluvium. These materials lack any well-defined planar features or discontinuities (such as bedding or joints) that could act as planes of weakness. This condition is considered favorable for gross stability.

5.9 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 Orange County General Plan (2004), the site is located within a potential inundation area for an earthquake-induced dam failure from the Prado Dam. However, this dam, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum credible earthquake (MCE) for the site.

Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low. The site is in an area of minimal flooding potential (Zone C) as defined by the Federal Insurance Administration.

5.10 Subsidence

According to the Environmental Hazard Element of the City of Huntington Beach General Plan (1996), the site is located within an area of known peat deposits. Minor amounts of peat deposits were encountered in several of our site explorations; particularly in Boring 5 below a depth of 10 feet. Peat and organic soils are considered highly susceptible to long term consolidation settlements.

5.11 Tsunamis, Seiches & Flooding

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.

According to the Environmental Hazard Element of the City of Huntington Beach General Plan (1996), the site located within a 100 and 500 year flood zone.

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 mixed-use development provided the recommendations presented herein are followed and implemented during construction.

6.1.2 Minor amounts of artificial fill were encountered during exploration. In its present condition the fill is not suitable for support of proposed foundations, floor slabs or additional fill. Excavation of the proposed subterranean level is expected to penetrate and remove all existing fill on the property, exposing relatively soft, fine-grained alluvial soils throughout the excavation bottom. Operation of rubber tire equipment on the sub grade soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 6.5).

6.1.3 The enclosed seismically induced settlement calculation indicates that approximately 1.0 inch of settlement could occur as a result of the DBE ground motion. Based on the nature of the earth materials that will be exposed at the excavation bottom and the potential for compressibility and differential settlement, it is recommended that a structural foundation system, consisting of a reinforced concrete mat foundation, be utilized to support the proposed structure and mitigate minor to moderate differential soil movements. The mat foundation should derive support in the undisturbed alluvial soils or stabilized subgrade soils as detailed herein.

6.1.4 It should be noted that implementation of the following recommendations in the design and construction of the proposed structure is not meant to completely prevent damage to the structure during the occurrence of strong seismic event. Should excessive settlement occur as a result of a strong seismic event at the site, damage to the structure and foundation may occur. It is intended that the project be designed in such a way that the amount of damage incurred as a result of a seismic event be minimized.

6.1.5 Relatively shallow groundwater is expected to be present at the subject site. Proposed excavations and building design will require temporary and permanent dewatering considerations. If the subterranean portion of the structure is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Recommendations for temporary and permanent dewatering are discussed in Section 6.4 of this report.

6.1.6 Waterproofing of subterranean walls and slabs is recommended for this project. 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.

6.1.7 The utilization of a hydrostatic design (fully waterproofed bathtub) will generate uplift forces as a result of the high water table which can actually lift or float the structure unless the building is sufficiently heavy. Temporary dewatering must be maintained during construction of the subterranean level until the building is heavy enough to resist the buoyant forces. If the buoyant forces are greater than the weight of the structure, permanent anchoring of the mat will be required to prevent the building from lifting. This can be accomplished by structurally joining the mat to other structural elements, such as perimeter piles used for shoring, or vertical grouted anchors. The buoyant forces and weight of the structure should be analyzed by the project structural engineer.

6.1.8 Due to the depth of the excavation and the proximity to the property lines, city streets, and adjacent offsite structures, excavation of the proposed subterranean level will require shoring measures in order to provide a stable excavation. A soldier pile system is recommended for the shoring design. Where excavation depths exceed ten feet or surcharges are imposed on the shoring system, raker braces or tie-back anchors will likely be required in conjunction with the soldier piles as indicated in the *Shoring* section of this report (Section 6.15). The need for lateral bracing should be determined by a qualified shoring engineer.

6.2 Soil and Excavation Characteristics

6.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Moderate slumping and caving should be anticipated in unshored excavations and water may seep from granular soil zones.

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 6.14).

6.2.4 The soils encountered during the investigation at the subterranean level have a “very high” 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 B8) 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 retaining wall drains and subdrains beneath the structure.

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 Dewatering

6.4.1 Temporary Dewatering

6.4.1.1 Temporary dewatering will be necessary for this project since the proposed subterranean level is anticipated to penetrate below the encountered groundwater level. It is important to understand that groundwater levels can fluctuate seasonally throughout the life of the structure and that levels could rise above present levels. Consideration should be given to designs that account for fluctuations in the water table.

6.4.1.2 Of the five borings drilled for this project, groundwater was only encountered in Boring 5 at a depth of 8 feet beneath the ground surface. It is anticipated that groundwater is present throughout the site; however, the soils are very fine grained and not conducive to high permeability or allowing free flow of water through the alluvial mass. It is anticipated that during open excavations the majority of groundwater seepage will emanate from the sand beds within the alluvial mass.

6.4.1.3 It is recommended that a qualified dewatering consultant be retained to assess flow rates during the design phase of the project. Temporary dewatering consisting of perimeter wells with interior well points may not be completely effective due to the presence of fine grained soils and inability of a well to produce groundwater draw-down in its vicinity. It is our opinion that if wells are ineffective, the water may be collected and controlled within the excavation through the use of gravel filled trenches (French drains). The number and locations of the French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

6.4.1.4 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary to place an adjacent French drain system, or sub-slab drainage system. It is not anticipated that a perimeter French drain will be more than 24 inches in depth below the proposed excavation bottom where situated adjacent to a shoring pile. If a French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

6.4.2 Permanent Dewatering

6.4.2.1 It is recommended that the design for the subterranean slab and retaining wall assume the groundwater table at a depth of five feet below the ground surface. If the subterranean portion of the structure is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and direct groundwater to a sump and pumping unit.

6.4.2.2 A typical permanent sub-slab drainage system would consist of an eight to twelve-inch thick layer of compacted, ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent). Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least one foot below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately 40 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed over the waterproofing system for protection during placement of rebar and mat slab construction.

6.4.2.3 Recommendations for design flow rates for the permanent dewatering system should be determined by a qualified contractor or dewatering consultant.

6.5 Grading

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

6.5.2 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.

6.5.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. 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 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.

6.5.4 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 compacted to at least 90 percent relative compaction as determined by ASTM Test Method D 1557-02.

6.5.5 Due to the presence of high-moisture content soils at the bottom of the excavation, pumping of the soils may occur during operation of heavy equipment. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. If a permanent dewatering system is to be installed for this project, subgrade stabilization may be accomplished by placing a one-foot thick layer of washed, angular ¾-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures, as well as a stable material upon which heavy equipment may operate.

6.5.6 Where permanent dewatering is not required and subgrade stabilization will be necessary, alternative methods of subgrade stabilization are provided in the *Soil Stabilization* section of this report (see Section 6.7).

6.5.7 A reinforced concrete mat foundation may be utilized for support of the proposed structure and may derive support in the undisturbed native soils and/or, properly compacted engineered fill, and/or stabilized subgrade, and/or the compacted gravel blanket for permanent dewatering system. Any encountered peat deposits should be removed as necessary and any soils unintentionally disturbed should be properly recompacted or replaced with suitable stabilization materials.

6.5.8 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 and compacted as necessary until the required compaction is obtained.

6.5.9 All imported fill shall be observed, tested and approved by Geocon Inland Empire, Inc. prior to use in the building pad area. Rocks larger than six inches in diameter shall not be used in the fill. Imported soils used in the building pad area should have an expansion index less than 50.

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

6.6 Shrinkage

6.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 20 and 40 percent should be anticipated when excavating and compacting the existing earth materials on the site to an average relative compaction of 92 percent.

6.7 Soil Stabilization

6.7.1 In areas where the subgrade is saturated, over-optimum or soft, or may be subject to the introduction of additional moisture (seepage or rainy season), proper compaction will likely not be possible or achieved in a timely manner without introducing stabilization measures. Based on the typical construction schedule and necessity to avoid delays, the implementation of stabilization measures may be warranted.

6.7.2 Bottom stabilization 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. In order to prevent excessive disturbance to a soft subgrade, it is recommended that track mounted equipment be utilized to conduct the spreading operations. Once the excavation bottom has been stabilized, the foundation system or structural fill may be placed.

6.7.3 As an alternative, dry cement may be introduced into the upper 12 to 18 inches of the exposed bottom. The cement will mitigate the over-optimum soils and provide improved strength for the engineered fill. The cement should only be spread in an area where mixing and compaction can be completed in the same working day. The cement content for the required stabilization should be approximately five percent by dry weight of the combined cement/soil mixture. Once the stabilized soil has been processed, compacted, and allowed to cure for a minimum of two days, it is recommended that heavy construction equipment, such as a scraper, not be operated directly on the stabilized subgrade, until a minimum of one additional foot of engineered fill has been placed. The stabilization procedure will create a crust that will bridge across the underlying soft wet soils. The utilization of very heavy construction equipment could damage the existing bridging capacity of the soil crust resulting in soil pumping. Once the crust has been damaged there will no longer be a stable working surface for the placement and compaction of the paving section and obtaining the required compaction could become very difficult and time consuming. If necessary, the entire fill blanket of the building pad can be blended with cement to mitigate the over-optimum nature of the soils.

6.8 Mat Foundation Design

6.8.1 Subsequent to the recommended grading or bottom stabilization, a reinforced concrete mat foundation may be utilized for support of the proposed structure. The mat foundation may derive support in the undisturbed native soils and/or, properly compacted engineered fill, and/or stabilized subgrade, and/or the compacted gravel blanket for permanent dewatering system.

6.8.2 It is anticipated that the proposed mat foundation will impart an average pressure of approximately 1,500 psf, with locally higher pressures up to 3,000 psf. The maximum allowable bearing value is 3,000 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

6.8.3 It is recommended that a modulus of subgrade reaction of 25 pounds per cubic inch (pci) be utilized for the design of the mat foundation on the soft native soils exposed at the subterranean level. If the subgrade is stabilized or a gravel blanket is placed a higher subgrade modulus may be warranted.

6.8.4 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer. If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed at five feet below the existing ground surface.

6.8.5 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 exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.9 Foundation Settlement

6.9.1 It is anticipated that the proposed mat foundation will impart an average pressure of approximately 1,500 psf, with locally higher pressures up to 3,000 psf. The mat foundation is expected to undergo static settlements of less than 1 inch upon completion of the proposed construction. This maximum settlement is anticipated at the corners, the heaviest loaded portions, of the mat foundation. Differential static settlement across the mat is not expected to exceed ½ inch over a distance of 20 feet; however, these settlements should be further evaluated and verified once the structural design and pressure distribution for the foundation system becomes more finalized.

6.9.2 The enclosed seismically induced settlement calculation indicates that settlements on the order of 1.0 inch occur as a result of the DBE ground motion, with resulting differential settlements anticipated to be approximately ½ inch over a distance of 20 feet. The anticipated seismically induced settlement is in addition to the static settlements indicated above.

6.9.3 For design purposes, it is recommended that the mat foundation be designed for a combined static and seismic differential settlement of 1 inch over a distance of 20 feet.

6.10 Lateral Design

6.10.1 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 0.20 may be used with the dead load forces in the undisturbed alluvial soils.

6.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils may be computed as an equivalent fluid having a density of 150 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. A one-third increase in the passive value may be used for wind or seismic loads.

6.11 Concrete Slabs-on-Grade

6.11.1 Unless a thinner slab section is designed by the project structural engineer, the slab-on-grade and ramp for the subterranean parking garage should be a minimum of 6 inches of 4,000 psi portland cement concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade should be underlain by undisturbed native soils, properly compacted fill, stabilized subgrade, or compacted ¾-inch gravel where a *Permanent Dewatering* (Section 6.4.2) system is utilized. A flexible connection or construction joint should be considered where the slab or ramp joins the structural mat foundation.

6.11.2 For seismic design purposes, a coefficient of friction of 0.20 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

6.11.3 Waterproofing of subterranean walls and slabs is recommended for this project. 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

6.11.4 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 subgrade should be moisture conditioned to 2 percent over 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. The project structural engineer should design construction joints as necessary.

6.11.5 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.

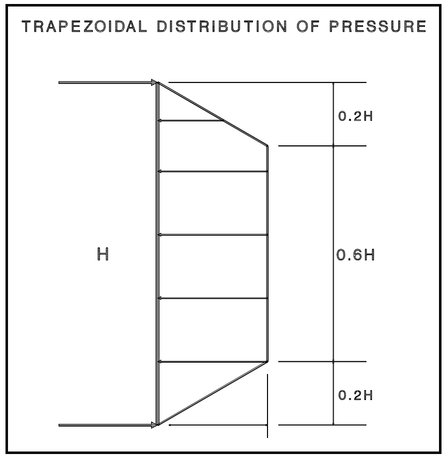
6.12 Retaining Wall Design

6.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls. 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 10	40
11 to 22	45

6.12.2 This soil pressure assumes that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall will have an EI of less than 50.

6.12.3 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 to resist a trapezoidal pressure distribution of lateral earth pressure as indicated in the diagram below.



6.12.4 Design walls for the following:

HEIGHT OF WALL "H" (Feet)	DESIGN WALL FOR (Where H is the height of the wall)
Up to 10	27H
11 to 22	33H

6.12.5 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, it is recommended that the entire below grade portion of the undrained 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 undrained cantilever walls and restrained walls would be would be 90 pcf and 55H (where H is the height of the wall in feet), respectively. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

6.12.6 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. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project, if necessary.

6.12.7 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the subterranean walls, the traffic surcharge may be neglected.

6.12.8 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 10). 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.

6.12.9 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 11). These vertical columns of drainage material would then be connected at the bottom of the wall to a continuous 4-foot high strip of similar drainage composite, a 4 inch perforated subdrain pipe covered with a minimum of 12 inches of gravel per lineal foot, or a 4 inch pipe that penetrates through the wall footing and connetcs to a subdrain pipe beneath the building.

6.12.10 Subdrainage pipes or rock pockets at the base of the retaining wall drainage system should outlet to an acceptable location or may be connected through pipes placed in sleeves through the bottom of the wall or footing, where they would then connect to a sump located below the floor slab.

6.12.11 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 inside the building. Due to the depth of the subterranean level and presence of groundwater, waterproofing is recommended for the subterranean level. 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.

6.13 Dynamic (Seismic) Lateral Forces

6.13.1 The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. The seismic increment in lateral earth pressure on the retaining side of the structure is applied to check the overall sliding resistance of the structure. The pressure is typically applicable where there is a differential of more than 6 feet in the height of the retained earth against opposite sides of the subterranean building level.

6.13.2 For seismic loading, we recommend a seismic active soil pressure equivalent to the pressure exerted by a fluid density of 18 pcf. This equivalent fluid pressure is in addition to static soil pressures and assumes low expansive soil will be used as backfill. The seismic active pressure is also for horizontal backfill behind the wall and does not account for an inclined backfill surface. The resultant seismic earth pressure acts at approximately 0.6H from the bottom of the wall (H is height of wall). The seismic loading is based on a horizontal ground acceleration of 0.40g, which corresponds to the DBE ground motion.

6.14 Temporary Excavations

6.14.1 Excavations on the order of 22 feet in vertical height may be required for the subterranean parking level. The excavations are expected to expose fill and alluvial soils, which are not suitable for vertical excavations in excess of 5 feet.

6.14.2 Vertical excavations will require shoring measures in order to provide a stable excavation. *Shoring* data is provided in Section 6.15 of this report.

6.14.3 Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion.

6.14.4 Should excavations be required adjacent to an existing structure or street, 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 or vehicle load in the street.

6.14.5 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.

6.15 Shoring – Soldier Pile Design and Installation

6.15.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

6.15.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Where maximum excavation heights are less than 10 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 10 feet, soldier piles will likely require lateral bracing utilizing drilled tie-back anchors or raker braces. The need for lateral bracing and the acceptable shoring deflection should be determined by the project shoring engineer.

6.15.3 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 150 pounds per square foot per foot. The allowable passive value may be doubled for isolated piles, spaced a minimum of twice the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

6.15.4 Groundwater is anticipated in excavations for the proposed soldier piles. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 4 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

6.15.5 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

6.15.6 Casing will likely be required since squeezing and caving of drilled excavations is anticipated in the saturated and/or granular soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon Inland Empire, Inc.) is required.

6.15.7 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.40 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 350 pounds per square foot.

6.15.8 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon Inland Empire, Inc.) to verify the cohesive nature of the soils and the areas where lagging may be omitted. As a minimum, it is required that the upper five feet of lagging be backfilled with slurry.

6.15.9 In areas where water will seep from the excavation face, the excavations for lagging and lagging placement should be carefully coordinated. The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but limited to a maximum of 400 psf.

6.15.10 For design of cantilevered or restrained shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design.

HEIGHT OF CANTILEVERED SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)**
Up to 12	35	50
16 to 25	40	60

6.15.11 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 or utility, the at-rest pressure should be considered for design purposes.

6.15.12 A trapezoidal distribution of lateral earth pressure would also be appropriate where shoring is to be restrained at the top by bracing or tie-backs. The lateral earth pressure to be used for design of temporary shoring is provided in the table below:

HEIGHT OF SHORING "H" (Feet)	**LATERAL EARTH PRESSURE (Where H is the height of the shoring in feet, plus the surcharge loading occurring due to traffic in the streets or any surcharge loading imposed by any adjacent structures.)**
Up to 12	24H
16 to 25	28H

6.15.13 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharge pressures should be incorporated into the shoring design as necessary.

6.15.14 In addition to the recommended earth pressure, the upper ten feet of the shoring adjacent to the street should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least ten feet from the shoring, the traffic surcharge may be neglected. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project.

6.15.15 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. The need for lateral bracing and the acceptable shoring deflection should be determined by the project shoring engineer.

6.15.16 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

6.16 Tie-Back Anchors

6.16.1 Tie-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

6.16.2 The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. Based on the height of the proposed excavation, two rows of anchors may be required. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows:

- Up to 5 feet below the top of the excavation – 200 pounds psf.
- Up to 12 feet below the top of the excavation – 350 pounds psf.
  (The above pressures take buoyant forces into account.)

6.16.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a friction capacity in excess of 1.5 kips per linear foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

6.17 Anchor Installation

6.17.1 Tied-back anchors are typically installed at an angle between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Caving of the anchor shafts, particularly within sand and gravel deposits should be anticipated during installation and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger drilling equipment be used to install the anchors.

The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

6.18 Anchor Testing

6.18.1 All of the anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches. The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15-minute period in order for the anchor to be approved for the design loading.

6.18.2 At least ten percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

6.18.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

6.18.4 For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

6.18.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. A representative of this firm should observe the installation and testing of the anchors.

6.19 Internal Bracing

6.19.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings.

For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 1,000 pounds per square foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

6.20 Surface Drainage

6.20.1 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.

6.20.2 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. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

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

6.21 Plan Review

6.21.1 Grading, foundation and shoring plans should be reviewed by the Geotechnical Engineer 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, if necessary.

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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APPENDIX A

FIELD INVESTIGATION

The scope of the field investigation, performed on October 27, 2006 consisted of excavating six, 7-inch diameter borings utilizing a hollow stem-auger drilling machine. The borings were conducted to depths between 20½ and 50½ 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 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. Standard Penetration Tests were performed in the 50-foot boring and bulk samples were also obtained.

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-6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are shown on the Site Plan, (Figure 2).

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, grain size, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B10. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.

Project No. A8481-06-01
April 25, 2008

VIA E-MAIL & GSO

Amstar / Red Oak Huntington Beach, LLC
2101 Business Center Drive, #230
Irvine, CA 92612

Attention: Mr. Alex Wong

Subject: ADDENDUM LETTER
UPDATE OF SEISMIC DESIGN CRITERIA
PROPOSED COLLEGE COUNTRY MIXED-USE DEVELOPMENT
7302-7400 CENTER AVENUE
HUNTINGTON BEACH, CALIFORNIA

References: Geotechnical Investigation, by Geocon Inland Empire, Inc., Project No. A8481-06-01 dated December 12, 2006;

City of Huntington Beach Department of Public Works, Conditional Use Permit No. 07-043, Design Review No 07-031, dated April 21, 2008;

PBS&J Geotechnical Report Peer Review – Ripcurl Mixed Use Development EIR, dated April 1, 2008.

Dear Mr. Wong:

This letter has been prepared in response to the April 21, 2008 City of Huntington Beach design review letter and the peer review comments by PBS&J, for the above referenced December 12, 2006 geotechnical report. The PBS&J letter indicates that the seismic design parameters were based on pre-2007 Building Code requirements and will need to be updated. The requested information is presented below.

The table on the following page summarizes site-specific design criteria obtained from the 2007 California Building Code (CBC; Based on the 2006 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.

IBC SEISMIC DESIGN PARAMETERS

Parameter	Value	IBC-06 Reference
Site Class	E	Table 1613.5.2
Spectral Response – Class B (short), S_S	1.539	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.554g	Figure 1613.5(4)
Site Coefficient, F_a	0.9	Table 1613.5.3(1)
Site Coefficient, F_v	2.4	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.385g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	1.330g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.923g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.887g	Section 1613.5.4 (Eqn 16-40)

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.

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

Sincerely,

GEOCON INLAND EMPIRE, INC.

REGISTERED PROFESSIONAL ENGINEER
NEAL D. BERLINER
No. 2576
Exp. 12/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Neal D. Berliner
Vice President

NDB:am

(2) Addressee