APPENDIX A

GEOTECHNICAL INVESTIGATION

American Geotechnical, Inc.
SOIL, FOUNDATION AND GEOLOGIC STUDIES

August 27, 2008 File No. 33054-01

Alexander & Hibbs, AIA, Inc.
1200 N. Jefferson Street, Suite A
Anaheim, CA 92807

Attention: Mr. Allen Hibbs

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Proposed 401 South Tustin Renovation Project
Orange County Healthcare Agency
401 South Tustin Street
Orange, California

References: See Appendix A

Dear Mr. Hibbs:

In accordance with your request, American Geotechnical performed a geotechnical investigation at the subject site. The purpose of our investigation was to evaluate the geotechnical conditions at the site in the areas of proposed construction and to provide geotechnical parameters for design and construction. This report is prepared in general accordance with our proposal dated July 1, 2008, and your written authorization.

Based on our investigation, it is our opinion that the proposed construction is feasible from the geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development. When final plans for the proposed construction become available, they should be forwarded to this office for review and comment.

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We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.

Sincerely,

AMERICAN GEOTECHNICAL

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AA/CC/MJ:dd

Enclosures: Appendix A – References
Appendix B – Boring Logs
Appendix C – Summary of Laboratory Data
Appendix D – Geotechnical Guidelines for Grading Projects

Distribution: 4 – Addressee (UPS Overnight)

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1.0 INTRODUCTION

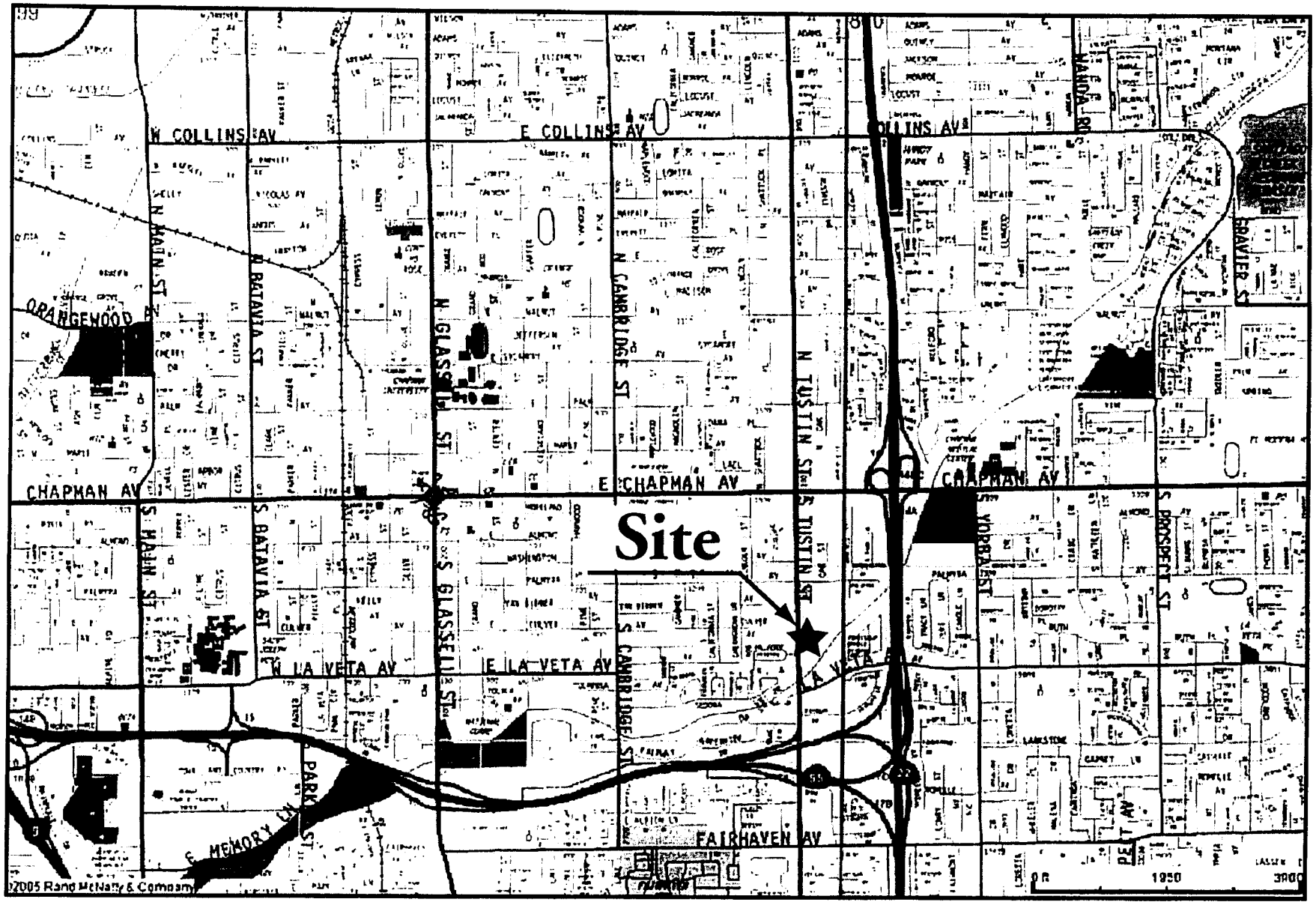
1.1 PURPOSE

The purpose of this geotechnical investigation was to evaluate the existing geotechnical conditions at the subject site and to provide design and construction criteria for the construction of the proposed 401 South Tustin Renovation project buildings and the associated parking areas, driveways, and walkways.

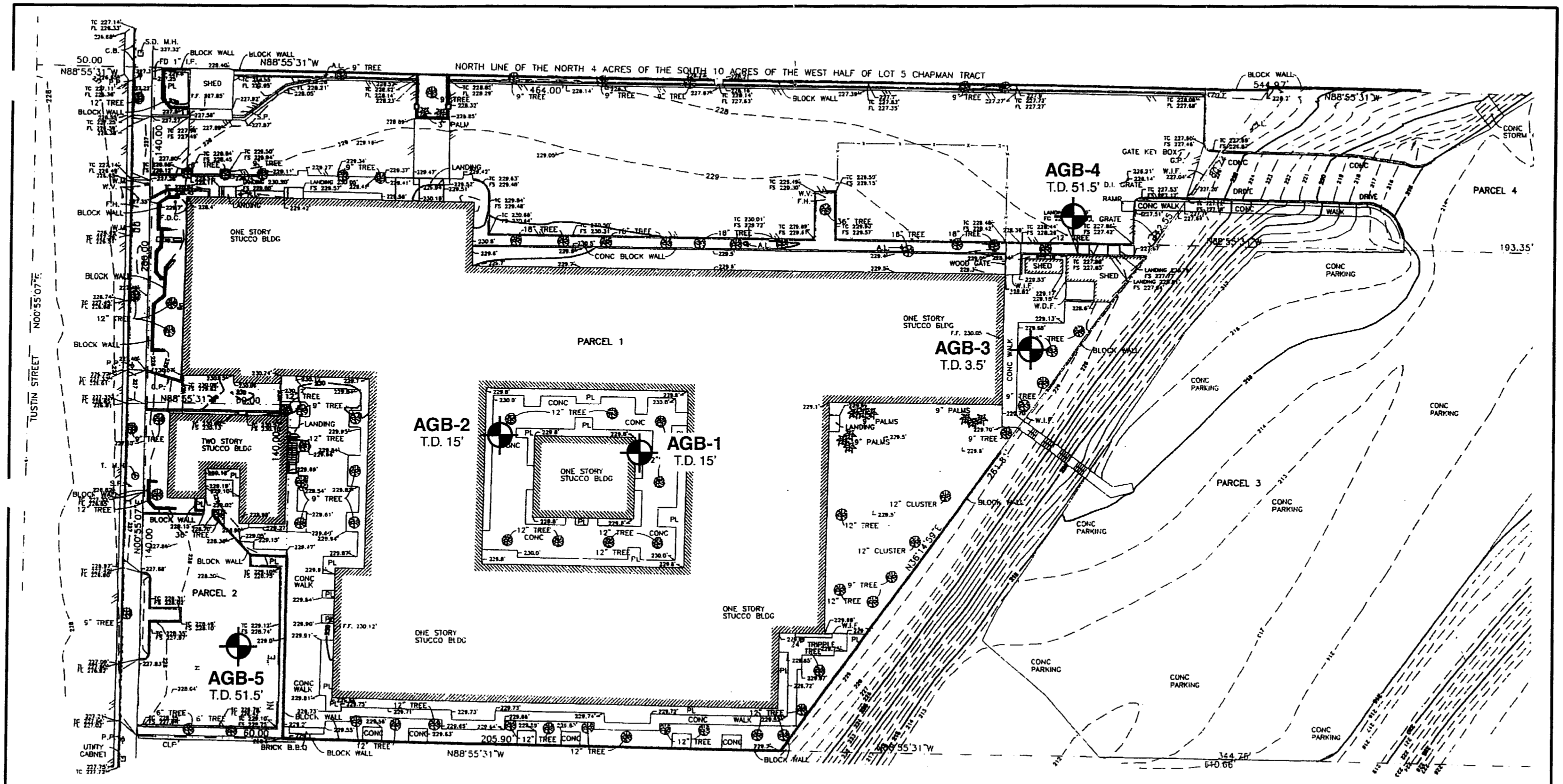
1.2 SCOPE OF SERVICES

The scope of services performed during this investigation consisted of the following:

- Research and review of available pertinent geotechnical literature;
- Subsurface exploration consisting of the excavation of three (3) small-diameter hollow stem-auger borings to a maximum depth of 15 feet below the existing ground; and excavation of two (2) air rotary hammer borings to a maximum depth of 51.5 feet below the existing ground surface in areas of the proposed building construction;
- Sampling and logging of the subsurface soil;
- Laboratory testing of the selected soil samples collected from the subsurface exploration to determine the engineering properties of the soil;
- Engineering and geologic analysis of the field and laboratory data; and
- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed construction.



American Geotechnical, Inc.	**Tustin Renovation** 401 S Tustin Street Orange, California	**Site Location Map** F.N. 33054.01 AUG 2008	Figure 1



Explanation

AGB-5 Approximate Location of AG Boring

NORTH

Scale: 1" = 40'

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Site Plan / Field Exploration Map

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Figure 2

1.3 PROJECT DESCRIPTION

The project site is located at 401 South Tustin Street in the city of Orange, California. The area of the proposed construction is a relatively-flat, approximately two acres, roughly rectangular parcel bounded by Tustin Street to the west, residential properties to the north, a water district property to the south and the Santiago Creek channel to the east. The abandoned County of Orange Interim Care Facility building occupies a majority of the site. Former asphalt surface drive and parking areas are located in the northern and southwestern portion of the site and concrete walkways and flatworks surround the building perimeter and central atrium. An approximately 12 to 15 foot high slope descends to the Santiago Creek channel along the eastern property boundary. A concrete ramp descends from the parking lot in the northeastern portion of the property to a concrete overflow parking lot within the Santiago Creek channel. The approximate location of the subject site is shown on the Site Location Map, Figure 1. A site plan including the existing building foot print is shown in Figure 2.

1.4 PROPOSED CONSTRUCTION

Based on the information provided, it is our understanding that the proposed site development will consist of the demolition of the existing buildings and improvements at the site and construction of three new single-story buildings, each approximately 7,500 square feet. The proposed buildings will house a Crisis Residential Center, a Wellness Center, and a Vocational Training Center. Other proposed improvements will consist of an approximately 1,000 square foot, single-story maintenance building and a miscellaneous structure to house a generator pad/electrical equipment. The buildings will utilize conventional spread-type footings. Construction will also consist of associated parking, driveways, underground utilities, walls, appurtenances, and other improvements. It is our understanding that data obtained from this investigation will be used in the development of grading and foundation plans. When grading and construction plans for the specific type(s) of construction become available, the plans must be submitted to this office for review and comments. The recommendations provided herein may be revised upon our review of the grading/construction plan(s) and further site analyses.

2.0 INVESTIGATION

2.1 FIELD EXPLORATION

The field investigation was conducted on July 15 and July 29, 2008. Subsurface conditions were explored by drilling five, small-diameter borings, AGB-1 through AGB-5. Borings AGB-1, 2 and 3 were drilled by 8-inch diameter, hollow-stem auger drilling equipment. Borings AGB-4 and 5 were drilled by 8-inch diameter air rotary hammer drilling equipment.

The borings were drilled to depths ranging from 3.5 to 51.5 feet below the existing ground surface. The soil materials encountered in our borings consisted of approximately 1 to 5 feet of fill soils encountered at the ground surface underlain by alluvial soils to the maximum explored depth of 51.5 feet. All of the borings drilled at the site were logged by a field professional using both visual and tactile means. Bulk and relatively-undisturbed drive samples, considered representative of subsurface materials, were collected and forwarded to the laboratory for testing. Drive samples from the exploratory borings were obtained with:

1. A 2.5-inch inside diameter (3.0-inch outside diameter) split barrel, thin wall sampler equipped with brass liner rings.

2. A standard split-spoon sampler with dimensions in accordance with ASTM D1586, equipped with brass liner rings.

Both samplers were driven with a 140-pound weight falling 30 inches. The blow counts to drive the thin wall sampler were noted on the logs only as an indication of relative driving resistance in the various layers and are not Standard Penetration Test ("N") values. The Standard Penetration Test values were obtained with the standard split-spoon sampler and are also shown on the logs. Logs of the exploratory borings are presented in Appendix B.

The approximate locations of the borings are shown on the Site/Field Exploration Map, Figure 2. Detailed boring logs are presented in Appendix B

2.2 LABORATORY TESTING

Samples collected in the field were examined in the laboratory to check the field classification and to assign laboratory testing. Selected samples were tested to determine soil properties for use in the subsequent engineering analyses. Laboratory tests included moisture-density, grain-size distribution, expansion potential, and corrosion potential (PH, electrical resistivity, soluble sulfates, and soluble chlorides). A summary of laboratory test results is presented in Appendix C.

3.0 SUBSURFACE CONDITIONS

3.1 SITE GEOLOGY

Information obtained from our subsurface investigation, as well as existing reports for the surrounding areas, indicate that alluvial deposits underlie the site covered by a thin veneer of fill soils.

3.1.1 Artificial Fill (Qaf)

Artificial fill soil was encountered at the ground surface, or immediately underlying the pavement section, in all five of our exploratory borings. The minimum fill thickness encountered was approximately 1 foot in boring AGB-3 and the maximum fill thickness encountered was approximately 5 feet in boring AGB-2. The fill soil was generally observed to consist of light reddish-brown or light grayish-brown, dry to slightly moist, medium dense gravelly sandy silt.

3.1.2 Alluvium (Qal)

Alluvial deposits were encountered in all five exploratory borings immediately underlying the fill soil. The alluvial deposits were generally observed to consist of interbedded layers of silty to sandy gravel, sandy clayey gravel, gravelly sand, clayey sand, gravelly sandy silt, clayey silt and gravelly clay to the maximum depth explored.

3.1.3 Groundwater

Groundwater was not encountered in any of the exploratory borings during our site investigation. Based on a review of the State of California, Seismic Hazard Zone report and Seismic Hazard Zone map for the Orange Quadrangle, the highest historic groundwater level at the site is shown as approximately 20 feet below the existing ground surface. Based on a review of recent published groundwater information (OCWD, 2005), groundwater is at a depth of approximately 170 feet below the existing ground surface at the site.

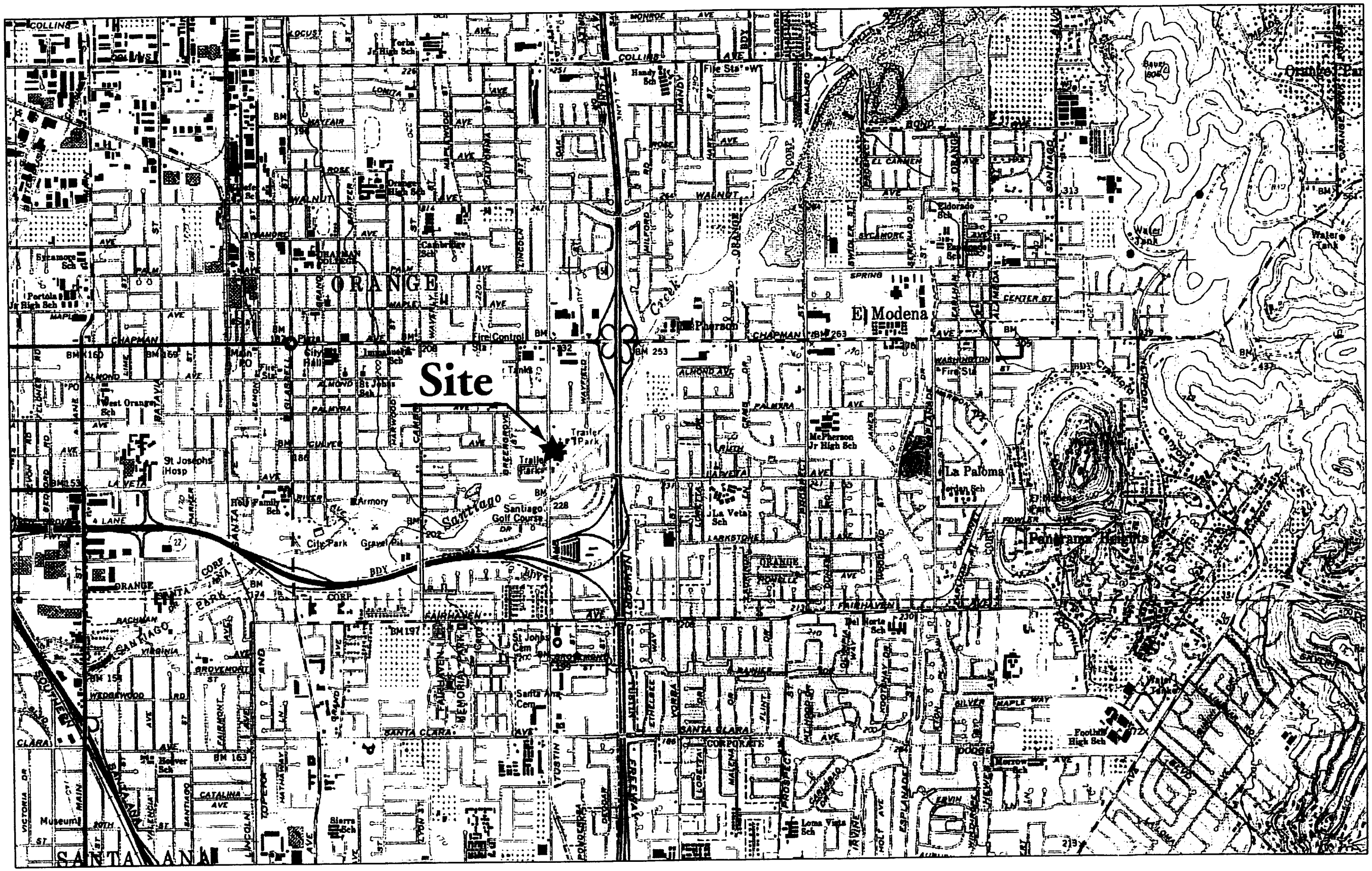
3.2 GEOLOGIC SETTING

The proposed building site is located at approximate site coordinates of 33.7818 degrees north latitude and 117.8349 degrees west longitude. The subject property is located within Section 33 of Township 4 South, Range 9 West as depicted on the Orange, CA 7.5-Minute Topographic Quad Map dated 1964, and photo revised 1981. A Site Location Map utilizing the 7.5-Minute Quad Map is attached as Figure 3.

Regionally, the subject site is located within the northern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges, valleys, and similarly oriented faults. The province extends from the Transverse Ranges southward to the tip of the Baja California Peninsula. The width of the province in southern California extends from the Pacific Ocean in the west to the Colorado Desert in the east.

The Peninsular Ranges province is a seismically active region characterized by structural blocks separated by a series of northwest-trending strike-slip faults. These faults are a part of or related to the San Andreas Fault System. The most prominent of these faults near the site area are the Whittier, Elsinore, and Newport-Inglewood Fault Zones, all of which area known to be active during the Quaternary time.

Locally, the subject site is situated within the Orange County coastal plain. The Orange County coastal plain is considered a part of the larger Los Angeles Basin. The Orange County Coastal Plain is a relatively-flat, alluvial plain located between upland regions, the Puente Hills on the north and northeast, the Santa Ana Mountains to the east and the San Joaquin Hills to the southeast. Several hundred feet of alluvial deposits underlie the site. These relatively recent alluvial sediments were generally deposited by local streams as active channel deposits and during times of flooding across the flood plains. These deposits generally consist of various mixtures of sand, silt, clay, and gravels. A Regional Geologic Map is attached as Figure 4.



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Site Topographic Map

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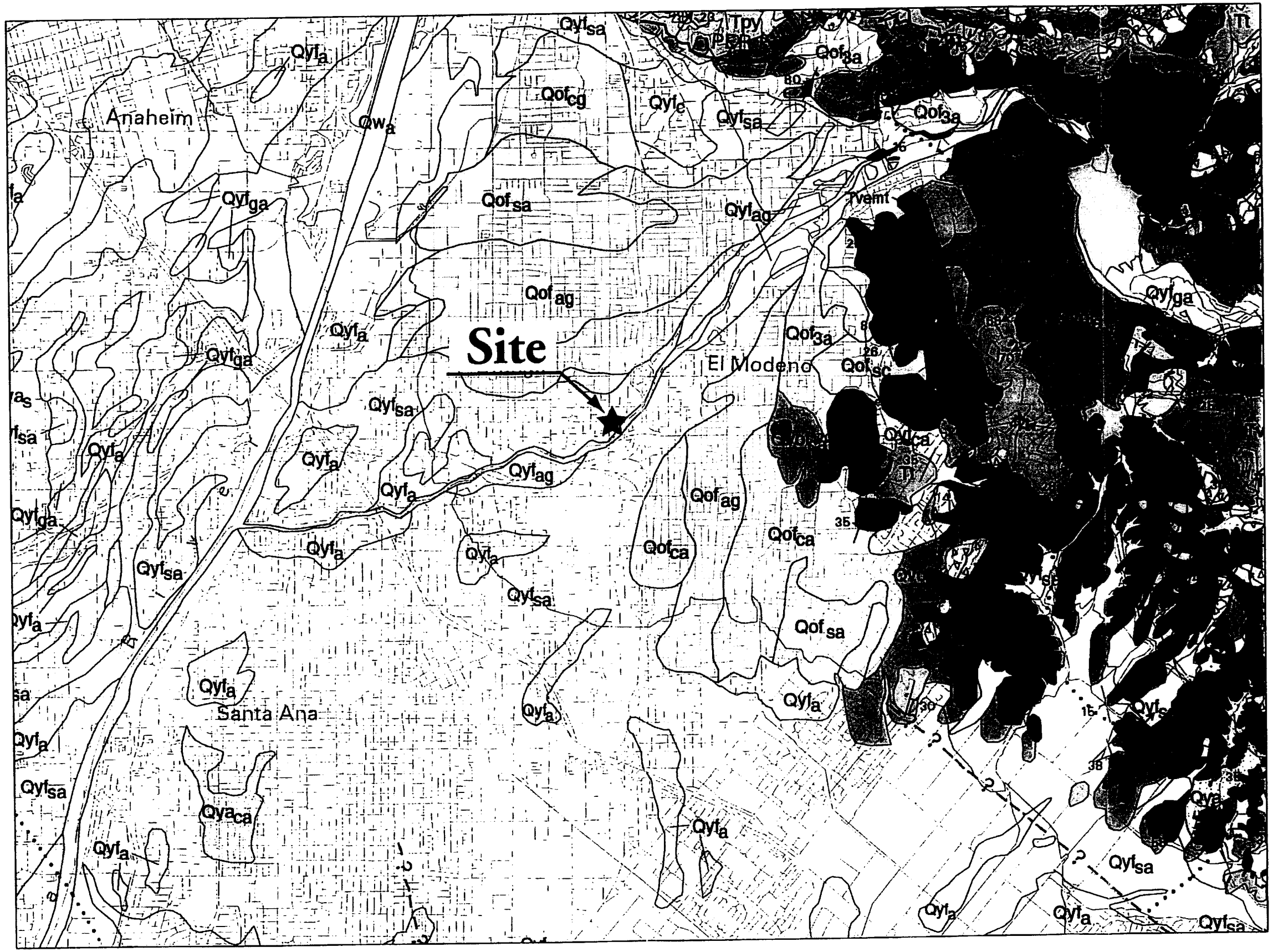
Figure 3



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Site Geologic Map

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Figure 4

3.5 FAULTING & SEISMICITY

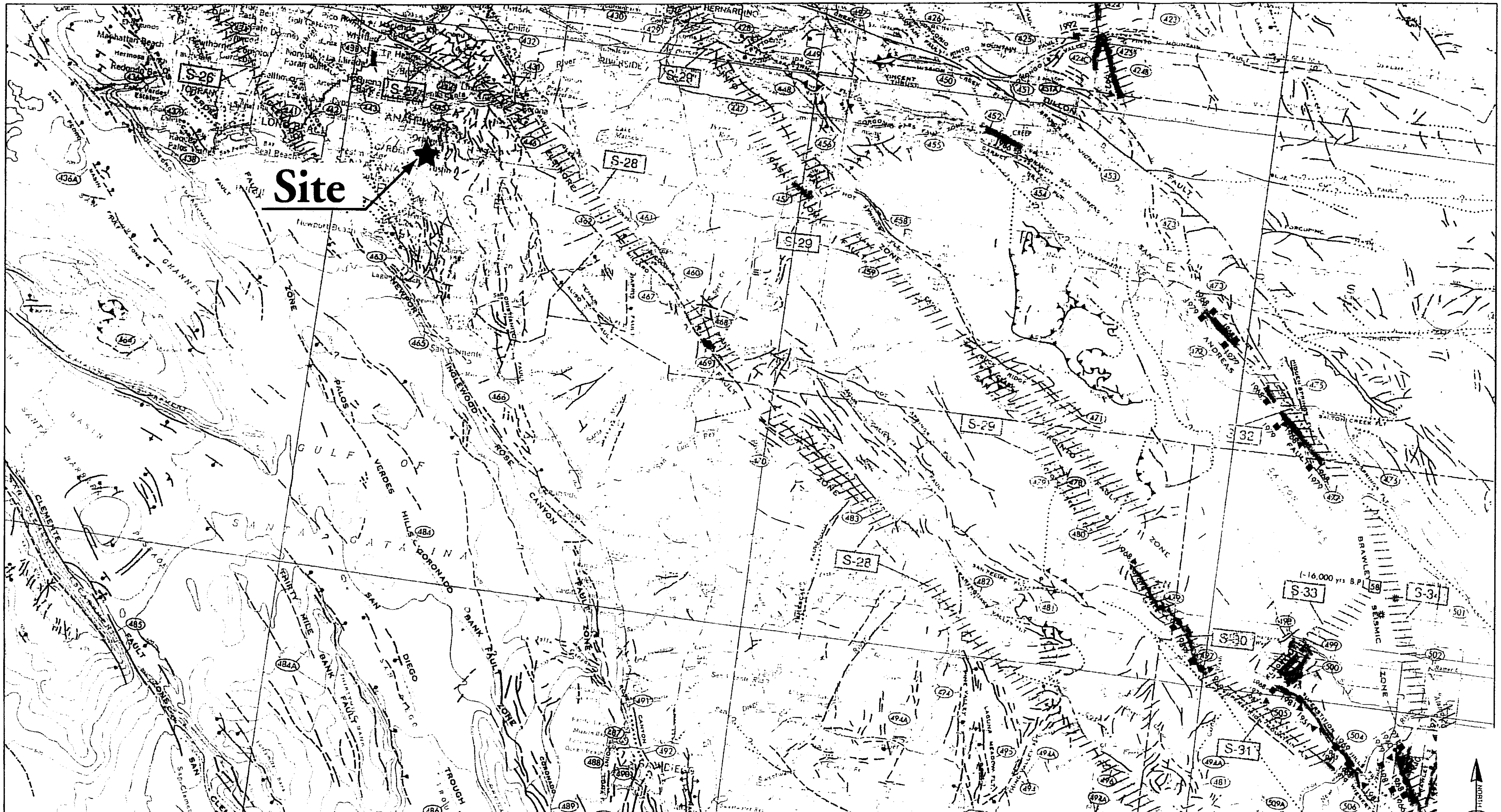
A risk common to all areas of southern California that should not be overlooked is the potential for damage resulting from seismic events (earthquakes). The site is located within a seismically active area, as is all of southern California. Although we are not aware of any active or potentially active faults on or within the immediate vicinity of the site, earthquakes generated on large regional faults such as the Newport-Inglewood, Elsinore, San Andreas, etc., could affect this site. The closest known active fault to the site is the San Joaquin Hills Blind Thrust Fault. The San Joaquin Hills Blind Thrust Fault is a buried thrust ramp located several miles below the ground surface and is located approximately 6.8 miles (10.9 km) southeast of the site. This fault is thought to be capable of a maximum 6.6 moment magnitude earthquake.

The closest historically recorded moderate to large earthquake is a moment magnitude 4.0 earthquake that had an epicenter located approximately 1.4 miles (2.3 km) from the site that occurred on August 22, 1936. This earthquake generated a peak ground acceleration of approximately 0.09g at the site.

The largest recorded historical earthquake to affect the site was the March 11, 1933 Long Beach Earthquake which had an epicenter located offshore from Newport Beach, approximately 13.7 miles (22 km) south of the site and probably occurred on the Newport-Inglewood Fault Zone. This earthquake had a moment magnitude of approximately 6.3 and generated an estimated peak ground acceleration of about 0.14g at the subject site.

The subject site is not located within a currently designated State of California Earthquake Fault Zone. Based on a review of the existing geologic information, no major surface fault crosses through or extends toward the site. The potential for surface rupture resulting from the movement of nearby major faults is not known with certainty but is considered low.

A Regional Fault Map is presented as Figure 5.



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Regional Fault Map

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Figure 5

4.0 CONCLUSIONS & RECOMMENDATIONS

4.1 BASIS

Conclusions and recommendations contained in this report are based upon information provided, information gathered, laboratory testing, engineering and geologic evaluations, experience, and professional judgment. Recommendations contained herein should be considered minimums consistent with industry practice. More rigorous criteria could be adopted if lower risk of future problems is desired. Where alternatives are presented, regardless of what approach is taken, some risk will remain, as is always the case. Usually, the lowest risk is associated with the greatest cost.

4.2 ALTERNATIVES

Where alternatives are presented in this report, the client should consider all information in the report, as well as the advice of other design team members, including the contractor. The client should also request any other information on design alternatives that he deems necessary. After consideration of costs, benefits, and risks, the client should identify the approach that he desires the project design and construction to take. It should be kept in mind that the governing authority may dictate the manner in which the project proceeds.

4.3 SITE SUITABILITY

Geotechnical exploration, analyses, experience, and judgment result in the conclusion that the proposed development is geotechnically feasible. It is our opinion that the site can be improved without hazard of landslide, slippage, or damaging settlement, and improvement can occur without similar adverse impact on adjoining properties. Realizing this expectation will require adherence to good construction practice, agency and code requirements, the recommendations in this report, and possible addendum recommendations made after plan review and at the time of construction.

4.4 SEISMIC DESIGN CONSIDERATION

4.4.1 CBC Design Parameters

The active fault closest to the site is the San Joaquin Hills Thrust Fault, located approximately 6.8 miles (10.9 km) southeast of the subject site. The site is not within an Alquist-Priolo Earthquake Fault Zone and no active faults with ground rupture potential have been mapped at the site. Based on this available information about the fault zone and the site soil conditions, the following seismic design parameters are recommended according to the 2007 Edition of the California Building Code (CBC), Chapter 16, Section 1613.

1. Site Class : D

2. Site Coefficients:
   1) F_a : 1.0
   2) F_v : 1.5

3. The mapped spectral accelerations:
   1) S_s (for short periods) : 1.397
   2) S_1 (for 1-second period) : 0.504

4. The maximum considered earthquake spectral response accelerations:
   1) $S_{MS} = F_a S_S$ (for short periods) : 1.397
   2) $S_{M1} = F_v S_1$ (for 1-second) : 0.756

5. The 5-percent damped design spectral response accelerations:
   1) $S_{DS} = 2/3\ S_{MS}$ (for short periods) : 0.931
   2) $S_{D1} = 2/3\ S_{M1}$ (for 1-second) : 0.504

It should be realized that the purpose of the seismic design utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of a large earthquake. This is the case for essentially all structures in Southern California.

4.4.2 Liquefaction Potential

Liquefaction is a sudden loss of strength of a saturated, cohesionless soil caused by cyclic loading (e.g. earthquake shaking). Generally, liquefaction occurs in predominantly poorly consolidated granular soil where the groundwater depth is less than 50 feet. However, our investigation indicated that no shallow ground water is present at the site and as such, there is no potential for liquefaction.

4.5 EARTHWORK

4.5.1 General

It is anticipated that the proposed construction will require grading at the site. When grading is conducted, it should be done in accordance with good construction practices, minimum code requirements, and the recommendations to follow. General guidelines for grading projects are also provided in Appendix D.

4.5.2 Site Preparation & Grading

Prior to the start of grading operations, utility lines within the project area, if any, should be located and marked in the field so they can be rerouted or protected during site development. All tree roots as well as existing asphalt pavement and buried foundations of existing building should be properly removed at the time of grading. All debris and perishable material should be removed from the site. From the geotechnical standpoint, it is acceptable that the asphalt

concrete removed from existing driveway and parking area be recycled, and reused in the construction as a part of base material. If reused, asphalt concrete should be crushed and the resulting base should conform to the applicable code and standard requirements.

It is our recommendation that all of the existing fill soil be excavated to the contact with the firm underlying alluvium deposits and placed back as properly compacted fill. Based on the information obtained from our investigation, a minimum over-excavation of 5 feet is recommended throughout the entire pad area (i.e. under the building, as well as other critical improvements such as generator pad, attached patios, etc.). In areas where less critical improvements such as parking/driveway, walkways, are planned, minimum depth of over-excavation and re-compaction is 2 feet or depth to existing fill/firm alluvium contact, whichever is greater. The area of site preparation/grading should extend five feet beyond any proposed improvements. The actual removal depth may also be locally increased based on the observation of the soil conditions encountered during grading. When excavations deeper than 5 feet are made, temporary construction slopes should be no steeper than 1.5:1 (horizontal:vertical). Sheeting and bracing should be provided by the contractor, as necessary, to protect workers in the excavation. Where excavations undermine existing improvements, such as the existing walls, etc., temporary structural support should be provided to reduce the risk of damage resulting from undercutting. Permanent cut and fill slopes should not be constructed steeper than 2:1, and should be considered subject to review by the geotechnical consultant at the time of grading. These slopes should possess sufficient compacted fines to limit erosion risk. If upon construction, relatively clean, cohesionless sands are encountered, reconstruction by blending in fines to compacted fill and/or flattening of slopes will be advised.

Where fill is to be placed, the upper 6 to 8 inches of the surface exposed by the excavation should be scarified, moisture-conditioned to 2 percent to 3 percent over optimum moisture content, and compacted to 90 percent relative compaction.[1] If localized areas of relatively loose soil prevent proper compaction, over-excavation, and re-compaction will be necessary.

[1] Relative compaction refers to the ratio of the in-place dry density of soil to the maximum dry density of the same material as obtained by the "modified proctor" (ASTM D1557) test procedure.

The on-site soil is generally suitable for use as compacted fill and backfill. Backfill around footings and in utility trenches within the building and paved areas should conform to the same material and compaction requirements as for the fill.

All grading should be conducted in accordance with the applicable codes, agency requirements, the aforementioned recommendations, and the grading guidelines which accompany this report as mentioned above.

4.5.3 Import Fill Material

The import fill material, if utilized, should meet the following criteria:

1. No particles larger than 6 inches in largest dimension.
2. Free of perishable material.
3. Plasticity Index of 20 or less and Liquid Limit of 40 or less.
4. Expansion Index of 20 or less.

4.6 FOUNDATION/SLAB DESIGN

4.6.1 General

Minimum criteria for the design of foundations and slabs applicable to the project are provided below. The criteria should not be considered as substitutes for actual designs by the structural engineer. The structural engineer should analyze the actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, strength, stiffness, and deflections in the various slab, foundation, and other elements of the structure, in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer should consider all applicable codes and authoritative sources where needed. If analyses by the structural

engineer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required. If the structural engineer has any questions or requires further assistance, please do not hesitate to call or otherwise transmit the engineer's requests.

4.6.2 Foundation Design

Foundation support for the new building could be derived by utilizing a conventional, shallow foundation system embedded within the newly placed compacted fill. A minimum three feet of properly compacted fill blanket below the bottom of the footing is recommended. Allowable design parameters for foundations are provided below. It should be noted that the following design parameters are provided, assuming that the near-surface soil has low expansive soil potential. These design parameters may be revised once additional testing is performed during rough grading of the project.

Minimum depth for interior and exterior footing
(measured from lowest adjacent grade)..... 24 inches

Minimum width..... 1.5 feet

Bearing pressure

a. Sustained loads (pounds per square foot).....1,500 psf
b. Total loads (including wind or seismic).....2,000 psf

Resistance to lateral loads

a. Passive soil resistance (pounds per cubic foot)
within compacted fill.....200 pcf
b. Coefficient of sliding friction..... 0.35

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 3.0.

Footings can be designed to resist lateral loads by using a combination of sliding friction and passive resistance. The coefficient of friction should be applied to dead load forces only. The upper one-foot of passive resistance should be neglected where the soil is not confined by slabs.

Footings designed and constructed in accordance with the foregoing criteria are expected to settle less than one-inch. Differential settlement of approximately half of the total settlement is expected.

It should be noted that a slope descends from the east side of the project area. In footings within 10 feet horizontal distance to descending slope faces, vertical and lateral resistance in the direction of slope should be ignored above the 10-foot setback line. In addition, an active Equivalent Fluid Pressure (EFP) of 50 pounds per cubic feet (pcf) should be applied from the surface to the 10 feet setback line to account for creep forces. This active pressure should resisted by passive pressure developed below the setback line.

4.6.3 Slab-on-Grade

The slab-on-grade in the interior of the building should be a minimum five inches thick and reinforced with No. 4 bars at 16 inches on center, both ways. The slab should be underlain by a 10-mil Visqueen plastic membrane sandwiched between two, 2-inch thick layers of sand. The sand should contain sufficient fines to allow light compaction (e.g. drum roller) to an unyielding condition. The plastic Visqueen barrier should be sealed at all splices, around plumbing, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier and care should be taken not to puncture the membrane. The splices between layers should be generously staggered. Below the sand layer and on top of the subgrade, a 4-inch thick, clean, crushed rock layer (e.g. 3/4-inch) is advised.

4.7 RETAINING WALL

Design

Wherever retaining walls are planned, they should be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot for level backfill and 65 pounds per cubic foot for 2:1 (horizontal:vertical) backfill. Appropriate allowances should be made for anticipated surcharge loading. It is assumed that the project engineer will incorporate an appropriately designed wall backdrain system for the purpose of mitigating potential for hydrostatic and/or seepage forces. It is also assumed that a predominantly granular, non-expansive backfill is provided. The minimum widths recommended for granular backfill are 1.5 feet at the base of the wall and 0.6 times that height of the wall at the top. The uppermost 1.0 to 1.5 feet should be backfilled with native, more cohesive material to minimize surface infiltration.

If walls are restrained against free movement by structural detailing such as basement walls, the active values given above should be increased. Although actual increase depends on the degree of restraint, 50 percent is recommended for design purposes.

It should be pointed out that the use of heavy compaction equipment in close proximity to retaining walls can result in excess wall movement and/or soil loadings exceeding design values. In this regard, care should be taken during backfilling operations. The foundation for the retaining walls should be designed according to the recommendations given in Section 4.6.

Waterproofing

If retaining walls form portions of the building interiors, special consideration should be given to waterproofing of the walls to prevent damage to the building interior. Unless dampness is acceptable on exterior wall faces, waterproofing should also be incorporated into the exterior retaining wall design.

Even though groundwater is not expected to be a problem at the site, extreme care should be exercised in sealing walls against water and water vapor migration. Where retaining walls are planned against interior space, continuity should be provided between the aforementioned wall moisture proofing on the back of the retaining wall and the moisture barrier typically placed under slab areas. This waterproofing is necessary to prevent the foundation concrete from acting as a wick through which moisture migrates to the interior space despite wall moisture proofing. The architect or structural engineer should develop the actual waterproofing details.

4.8 EXTERIOR FLATWORK

Wherever planed, the exterior flatwork should be placed over the newly placed compacted fill. The compacted fill thickness under the slabs should conform to the recommendations given in Section 4.5. Recommendations provided above for interior slabs can also used for exterior flatwork, but without sand layer or Visqueen moisture barrier. At the exterior edges of the flatwork, a thickened edge is recommended. The thickened edges should be a minimum of 12 inches deep and 8 inches wide and reinforced with two, No. 4 bars at top and bottom.

Movement of slabs adjacent to structures can be mitigated by doweling slabs to perimeter footings. As an option to doweling, an architectural separation could be provided between the main structures and the abutting appurtenant improvements.

4.9 PAVEMENT DESIGN

Based on the assumed R-value of ten for the near-surface soil, the following pavement sectional dimensions are provided for different values of Traffic Index (TI). It should be noted that the pavement design provided below should be verified by the R-value testing of the soil samples collected during rough grading.

Traffic Index (TI)	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5	4	8
6	4	10

For light traffic and parking areas, sectional parameters corresponding to the Traffic Index of 5 may be utilized. Pavement thickness corresponding to TI of 6 should be used for the fire truck access lanes and the areas of heavy vehicular use. It is also recommended that the existing loose surficial soil in the pavement areas be removed and properly placed. A minimum 24 inches of over-excavation should be anticipated. The over-excavation material could be utilized as fill. The fill should be placed to a minimum relative compaction of 90 percent. The aggregate base for the pavement should meet the requirements of rock base as described by the Standard Specifications for Public Works Construction, latest edition. The aggregate base should be compacted to at least 95 percent relative compaction.

As an alternative to asphalt pavements, Portland Cement Concrete (PCC) pavements may be considered in heavy truck areas (e.g. trash collection area). If used, the concrete pavements should be at least 7 inches thick and reinforced with a minimum of No. 5 bars at 12 inches on center, each way. The concrete used for the pavement should have a minimum 28-day

compressive strength of 3,500 psi. A minimum of 4 inches of 3/4-inch clean crushed rock is also recommended below the pavements. Similar to the asphalt pavement recommendations provided above, at least 24 inches of the subgrade should be over-excavated and recompacted to a minimum relative compaction of 90 percent.

The pavement recommendations presented above are based on the existing surface soil conditions. After the final grading is completed, actual subgrade materials will be collected and tested to confirm site conditions. If post-grading soil conditions vary, modified pavement sections will be recommended.

4.10 PROPERTY WALLS

The foundation for the property line walls should be designed based on the same recommendations provided for buildings (see Section 4.6). For walls planned along the top of the descending slopes, setback criteria provided in Section 4.6.2 is applicable and should be followed. A pile foundation system can also be utilized for walls along top of the slope as an alternative to shallow foundations. If utilized, minimum 18 inch diameter piles spaced not more than 6 feet on-center is recommended. The piles should be designed for the active and passive pressures provided in section 4.6.2. previously. Vertical load carrying capacity of piles embedded at least 2 feet into firm alluvial material can be design as end bearing piles using an allowable bearing capacity of 3,000 psf. The property walls, if masonry, should also be provided with vertical joints at a regular spacing, at about 20 feet on center.

4.11 CONCRETE

Laboratory testing indicated that the surface soil at the site has low levels of sulfates and as such, no special sulfate resistant concrete mix design is required. However, we recommend that low permeable concrete be utilized at the site to limit moisture transmission through slab and foundations. For this purpose, the water to cement ratio in the concrete should be limited to 0.5 (0.45 preferred). Limited use (subject to approval of mix designs) of a water-reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. The code dictates at least seven days of moist curing. Two to three weeks is preferred to minimize cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g. flatwork) and other improvements provided the mix designs consider limiting shrinkage. Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Alternatively, the concrete mix can also incorporate W.R. Grace/Eclipse shrinkage reducing admixture dosed for maximum benefit. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

4.12 SOIL CHEMICAL PROPERTIES

In addition to sulfate tests, the Chloride, pH and resistivity tests on near-surface site soil were also performed. The results of theses tests, presented in Appendix C, suggest that the site soil poses moderate risk of damage to buried metal objects. Appropriate design considerations should be made for this risk.

4.13 FOUNDATION DETAILS

4.13.1 Reinforcement Placement

Care should be taken in the placement of foundation and slab reinforcement. Placement details should be in conformance with ACI specifications. Unless otherwise specified by the structural engineer, continuous footing reinforcement should be placed in the upper and lower 1/3 portions of the foundation sections. The bottom foundation steel should not be closer than three inches to the underlying excavation. Slab reinforcement should be placed in a positive fashion between the midpoint and upper 1/3 point of the slab section. "Lifting" slab steel into place following concrete placement is not recommended.

4.13.2 Foundation & Slab Concrete Workmanship

The contractor should be responsible for supplying to the owner concrete mix designs for both slab and foundation concrete. The contractor should provide designs, place, finish, and cure concrete in accordance with all code and ACI recommended procedures. The contractor is referred to the ACI publication "Slabs on Grade."

Suitable joints should be provided to control cracking in flatwork.

4.14 DRAINAGE

Drainage recommendations should be provided by the project civil engineer. However, the following criteria could be utilized as general guidelines. Drainage should be directed away from structures via non-erodible conduits to suitable disposal areas. Five percent drainage is recommended directly away from structures. At least two percent minimum is recommended for drainage over soil areas. Four percent to five percent minimum is advised for drainage away from structures and for planter and groundcover areas. In pipes or paved swales, one percent should be adopted as the minimum unless otherwise recommended by the project

civil engineer. All enclosed planters should be provided with a suitably located drain or drains and/or flooding protection in the form of weep holes or similar.

4.15 PROJECT SAFETY

The contractor is the party responsible for providing a safe site. This consultant will not direct the contractor's operations and cannot be responsible for the safety of personnel other than his own representatives on site. The contractor should notify the owner if he is aware of and/or anticipates unsafe conditions. If the geotechnical consultant at the time of construction considers conditions unsafe, the contractor, as well as the owner's representative, will be notified. Within this report, the terminology safe or safely may have been utilized. The intent of such use is to imply low risk. Some risk will remain, however, as is always the case.

5.0 REMARKS

Only a portion of subsurface conditions have been reviewed and evaluated. Conclusions, recommendations, and other information contained in this report are based upon the assumptions that subsurface conditions do not vary appreciably between and adjacent to observation points. Although no significant variation is anticipated, it must be recognized that variations can occur.

This report has been prepared for the sole use and benefit of our client. The intent of the report is to advise our client on geotechnical matters involving the proposed construction. It should be understood that the geotechnical consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspect of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive the advice. Subsequent use of this report can only be authorized by the client. Any transferring of information or other directed use by the client should be considered "advice by the client."

Geotechnical engineering is characterized by uncertainty. Geotechnical engineering is often described as an inexact science or art. Conclusions and recommendations presented herein are partly based upon the evaluations of technical information gathered, partly on experience, and partly on professional judgment. The conclusions and recommendations presented should be considered "advice." Other consultants could arrive at different conclusions and recommendations. Typically, "minimum" recommendations have been presented. Although some risk will always remain, lower risk of future problems would usually result if more restrictive criteria were adopted. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

APPENDIX A

References

REFERENCES

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American Geotechnical, Inc.

File No. 33054-01
August 27, 2008

APPENDIX B

Boring Logs

BORING LOG No. AGB-1

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 1 of 1

Location: 401 S. Tustin Street, Orange

Start: Jul 15, 08

Estimated Surface Elevation: 230' ± Total Depth: 15.0' Rig Type: 8" HSA

End: Jul 15, 08

Field Description

By: CC

Surface Conditions:

Atrium planter - northeast corner of center building - Interim Care Facility

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight Pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description
0						ML		**ARTIFICIAL FILL (Qaf) (0-4')** Sandy SILT (ML) - light grayish-brown, dry to slightly moist, medium dense, medium to coarse sand, some fine to medium gravel, non-plastic.
								4.00
						GM SM		**4' ALLUVIUM (Qaf) (4'?-15')** Medium to coarse gravel lense from 4'-6' - difficult drilling
5								@6' Gravelly SAND (SM) to Sandy GRAVEL (GM) - Light grayish-brown, slightly moist, medium to coarse sand, fine to coarse gravel, sample disturbed.
		63						
								@8' Gravel lense.
10								@10' Attempted ring sample - unable to sample due to gravel.
								@12' Increased gravel size, minimal fines, generally 1-2" subrounded to subangular gravel, near refusal.
								@14' Pea gravel to 1/2" diameter, easier drilling.
15								15.00 @15' Increased gravel size, near refusal, boring terminated. Total Depth = 15 feet
20								

NOTES: **140 lb. hammer, 30" drop, backfilled with native soils.**

Legend: Bag, Ring

American Geotechnical

Plate

BORING LOG No. AGB-2

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 1 of 1

Location: 401 S. Tustin Street, Orange

Start: Jul 15, 08

Estimated Surface Elevation: 230' ± Total Depth: 15.0' Rig Type: 8" HSA

End: Jul 15, 08

Field Description

By: CC

Surface Conditions:

Atrium planter - northwest portion of atrium - Interim Care Facility

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight Pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description
0						ML		**ARTIFICIAL FILL (Qaf) (0-5')** Gravelly sandy SILT (ML) - light reddish-brown, dry to slightly moist, loose, medium to coarse sand, fine gravel, trace clay.
5		38				ML GP		5.00 @5' Gravel lense/layer ends at 6'. **ALLUVIUM (Qaf) (5' - 15')** Gravelly Sandy SILT (ML) - light reddish-brown, slightly moist, loose, nonplastic, trace gravel, medium to coarse sand.
		17						@8'-12' Thin gravel layers/lenses - slight to moderate drilling difficulty.
10								@12' Attempted sample - unable to drive due to gravel content, silt/sand matrix. @12'-14' Increased gravel, difficult drilling advancement.
								@14'-15' Easier drilling, more fines.
15								15.00 @15' Increased gravel, very difficult drilling, near refusal. Total Depth = 15 feet
20								

NOTES: **140 lb. hammer, 30" drop, backfilled with native soils.**

Legend: Ring

American Geotechnical

Plate

BORING LOG No. AGB-3

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 1 of 1

Location: 401 S. Tustin Street, Orange

Start: Jul 15, 08

Estimated Surface Elevation: 230' ± Total Depth: 3.5' Rig Type: 8" HSA

End: Jul 15, 08

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight Pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description (By: CC)
0						ML		Surface Conditions: Rear planter/Patio area above Santiago Creek Channel - Interim Care Facility Subsurface Conditions: FORMATION: Classification, color, moisture, tightness, etc. ARTIFICIAL FILL (Qaf) (0-1') ALLUVIUM (Qal) (1'-3.5') @1' Gravelly Sandy SILT (ML) - light grayish-brown, same as previous.
3.50								@3.5' Refusal due to gravel. Total Depth = 3.5 feet
5								
10								
15								
20								

NOTES: Backfilled with native soils.

Legend: ☒ Bag

American Geotechnical

Plate

BORING LOG No. AGB-4

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 1 of 3

Location: 401 S. Tustin Street, Orange

Start: Jul 29, 08

Estimated Surface Elevation: 230' ± Total Depth: 51.5' Rig Type: Air Rotary Hammer

End: Jul 29, 08

Field Description

By: KB

Surface Conditions:

AC pavement on right-rear portion of north parking lot.

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight Pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description
0						GM		0.30 **AC PAVEMENT (0-0.3')** **@0.3' ARTIFICIAL FILL (Qaf) (0.3'-4')** Silty Sandy GRAVEL (GM) - grayish-brown, dry to slightly moist, medium to coarse.
						GM		4.00 **@4' ALLUVIUM (Qal) (4'-51.5')** Silty Sandy GRAVEL (GM) - grayish-brown, slightly moist, medium to coarse.
5								
		50 for 4"						
10								
15								
		50 for 3"						
20								

NOTES: 140 lb. hammer, 30" drop, backfilled with native soils.

Legend: SPT, Bag

American Geotechnical

Plate

BORING LOG No. AGB-4

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 2 of 3

Field Description

By: KB

Surface Conditions:

AC pavement on right-rear portion of north parking lot.

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description
25								
		100						
30								@32' Becomes dark brown, increased fines.
35								
		100						
40						CL		40.00 @40' Sandy CLAY (CL) - dark brown, moist, medium stiff, medium plasticity.

Legend: SPT, Bag

American Geotechnical

Plate

BORING LOG No. AGB-4

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 3 of 3

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description (By: KB)
								Surface Conditions: AC pavement on right-rear portion of north parking lot.
								Subsurface Conditions: FORMATION: Classification, color, moisture, tightness, etc.
45								
50		37						51.50
								Total Depth = 51.5 feet
55								
60								
65								

Legend: SPT
Bag

American Geotechnical

Plate

BORING LOG No. AGB-5

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 1 of 3

Location: 401 S. Tustin Street, Orange

Start: Jul 29, 08

Estimated Surface Elevation: 230' ± Total Depth: 51.5' Rig Type: Air Rotary Hammer

End: Jul 29, 08

Field Description

By: KB

Surface Conditions:

AC pavement on right-rear portion of southwest parking lot.

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight Pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Field Description
0								0.30 **AC PAVEMENT (0-0.3')**
						GM		**ARTIFICIAL FILL (Qaf) (0.3' - 7')** Silty Sandy GRAVEL (GM) - light grayish-brown, dry to slightly moist.
								@3' Cobble encountered, broken, then drilling advanced.
5								
								7.00
		50 for 2"				GM		**ALLUVIUM (Qal) (7'-51.5')** Sandy GRAVEL (GM) - light grayish-brown, slightly moist, medium to coarse.
10								
15		50 for 5"						
20								

NOTES: 140 lb. hammer, 30" drop, backfilled with native soils.

Legend: SPT, Bag

American Geotechnical

Plate

BORING LOG No. AGB-5

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 2 of 3

Field Description

By: KB

Surface Conditions:

AC pavement on right-rear portion of southwest parking lot.

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Description
25		50 for 2"						
30								
35						GC		35.00 @35' Sandy Clayey GRAVEL (GC) - brown, moist, increased fines.
		50 for 5"						
40								

Legend: SPT, Bag

American Geotechnical

Plate

BORING LOG No. AGB-5

F.N. 33054-01

Project Name: 401 South Tustin

Sheet: 3 of 3

Field Description

By: KB

Surface Conditions:

AC pavement on right-rear portion of southwest parking lot.

Subsurface Conditions: **FORMATION: Classification, color, moisture, tightness, etc.**

Depth-Feet	Sample Type (INTACT / BULK)	Blow Counts Per Foot	Dry Unit Weight pcf	Moisture Content %	Laboratory Tests	USCS Symbol	Graphic Log	Description
45						GM SM		45.00 @45' Gravelly SAND (SM) to Sandy GRAVEL (GM) - brown, slightly moist.
50		55				SC		50.00 @50' Gravelly Clayey SAND (SC) - brown, moist, dense. 51.50
								Total Depth = 51.5 feet
55								
60								
65								

Legend: SPT, Bag

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Plate

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APPENDIX C

Summary of Laboratory Data

Exca. No.	Depth (feet)	In-Situ Dry Density (Lbs/cu. ft.)	In-Situ Moisture Content (%)	Maximum Dry Density (Lbs/cu. ft.)	Optimum Moisture Content (%)	% Passing #200 Sieve (%)	Expansion Index	Soil Chemical Testing			
								Chloride (mg/kg)	pH (-)	Resistivity (Ohm-cm)	Sulfate (mg/kg)
AGSB-1	0.0-4.0	-	-	130	8	38	31	17	8.17	2370	55
AGSB-1	6.0-7.5	113.2	2.1	-	-	-	-	-	-	-	-
AGSB-1	10-15	-	-	-	-	14	-	-	-	-	-
AGSB-2	6.0-7.5	89.0	3.4	-	-	-	-	-	-	-	-
AGSB-2	8.0-9.5	113.0	1.7	-	-	-	-	-	-	-	-
AGSB-3	0.0-3.5	-	-	-	-	-	14	25	8.66	2150	60
AGSB-4	7	-	1.0	-	-	-	-	-	-	-	-
AGSB-4	17	-	0.6	-	-	-	-	-	-	-	-
AGSB-4	27	-	2.0	-	-	-	-	-	-	-	-
AGSB-4	37	-	3.0	-	-	-	-	-	-	-	-
AGSB-4	50	128.7	3.6	-	-	-	-	-	-	-	-
AGSB-5	7	-	0.9	-	-	-	-	-	-	-	-
AGSB-5	17	-	1.3	-	-	-	-	-	-	-	-
AGSB-5	27	-	0.8	-	-	-	-	-	-	-	-
AGSB-5	37	-	3.4	-	-	-	-	-	-	-	-
AGSB-5	50-51.5	111.7	11.8	-	-	-	-	-	-	-	-

SUMMARY OF LABORATORY TESTING DATA			TABLE
AMERICAN GEOTECHNICAL	F.N. 33054.01	AUG. 2008	C

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APPENDIX D

Geotechnical Guidelines for Grading Projects

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GEOTECHNICAL GUIDELINES FOR GRADING PROJECTS

1.0 GENERAL

These guidelines should be considered to be a portion of the project specifications. The contractor should not vary from these guidelines without prior recommendation by the geotechnical consultant and approval of the client or his authorized representative. Recommendations by the geotechnical consultant and/or client should not be considered to preclude requirements for approval by the controlling agency. If disputes arise out of the interpretation of these grading guidelines, the geotechnical consultant shall provide the governing interpretation.

2.0 OBLIGATIONS OF PARTIES

The geotechnical consultant should be authorized by the client to provide observation and testing services during grading. The geotechnical consultant will report his findings and recommendations to the client or his authorized representative. The client should be chiefly responsible for all aspects of the project. The client or his authorized representative has the responsibility of reviewing the findings and recommendations of the geotechnical consultant. During grading, the client or his authorized representative should remain on site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

3.0 SITE PREPARATION

3.1 Prior to any site preparation or grading, the client should arrange and attend a meeting among the grading contractor, the design engineer, the geotechnical consultant, and representatives of the appropriate governing authorities as well as any other concerned parties. All parties should be given at least 48 hours notice.

3.2 Clearing and grubbing should consist of the removal of vegetation and otherwise deleterious natural materials from the areas to be graded. Clearing and grubbing should extend to the outside of all proposed excavation and fill areas.

3.3 Demolition should include removal of buildings, structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, mining shafts, tunnels, etc.), and other man-made surface and subsurface improvements from the areas to be graded. Demolition of utilities should include proper capping and/or rerouting of pipelines at the project perimeter and cutoff and capping of wells in accordance with the requirements of the governing authorities. Trees, plants, or man-made improvements not planned to be removed or demolished should be protected by the contractor. Debris generated during clearing, grubbing, and/or demolition operations should be removed from areas to be graded and disposed of off-site.

4.0 SITE PROTECTION

4.1 Protection of the site during the period of demolition and grading should be the responsibility of the contractor. The contractor should be responsible for the stability of all temporary excavations including buttress and retaining wall cuts.

4.2 The contractor should control dust at all times.

4.3 Precautions should be taken during the performance of site clearing, excavating, and grading to protect the work site from flooding, ponding, erosion, and all other damage by poor or improper surface drainage.

4.4 During periods of rainfall, the geotechnical consultant should be kept informed by the contractor as to the nature of remedial or preventative work being performed. Following periods of rainfall, the contractor should contact the geotechnical consultant and arrange a visual inspection of the site in order to assess rain-related damage. Adversely affected soil will be classified as unsuitable material and should be subject to overexcavation and replacement with compacted fill or other remedial grading.

5.0 EXCAVATIONS

5.1 Material which is unsuitable should be excavated under the observation and recommendations of the geotechnical consultant. Unsuitable material includes, but may not be limited to, dry, loose, soft, wet, organic, compressible, natural soil, and fractured, weathered, soft bedrock and non-engineered or otherwise deleterious fill material.

5.2 Unless otherwise recommended by the geotechnical consultant and approved by regulating agencies, permanent cut slopes should not be steeper than 2:1 (horizontal to vertical).

6.0 COMPACTED FILL

6.1 Prior to placement of compacted fill, the contractor should request a review by the geotechnical consultant of the exposed ground surface. Unless otherwise recommended, the exposed ground surface should be scarified (six inches minimum), watered or dried as needed, thoroughly blended to achieve near optimum moisture conditions, then thoroughly compacted to a minimum of 90 percent of the maximum density.

6.2 Compacted fill should then be placed in thin horizontal lifts not exceeding eight inches in loose thickness prior to compaction. Each lift should be watered or dried as needed, thoroughly blended to achieve near optimum moisture conditions, then thoroughly compacted by mechanical methods. Depending on the type of soil and compaction equipment, thinner lifts could be recommended. Each lift should be treated in a like manner until the desired finished grades are achieved. The contractor should have suitable and sufficient mechanical compaction equipment, watering apparatus and mixing capability on the job site to handle the amount of fill being placed. If necessary, excavation equipment should be "shut down" temporarily in order to permit proper compaction of fills. Earth moving equipment should only be considered a supplement and not substituted for conventional compaction equipment.

6.3 Unless otherwise specified, the minimum degree of compaction required is 90 percent of the laboratory maximum density. Depending on the soil type, depth of fill and/or other conditions, higher relative compaction could be recommended. Maximum density should be determined in accordance with ASTM Method of Test D1557, Method A or C.

6.4 Fill should be tested for compliance with the recommended relative compaction and moisture conditions. Field density testing should conform to ASTM Method of Test D1556, D2922 and/or D2937. Tests should be taken for about every two vertical feet or 1,000 cubic yards of fill placed. Where gravel is encountered, dry density should be corrected by the subtraction/elimination method. Actual test intervals may vary as field conditions dictate. Fill not found to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the geotechnical consultant. The contractor should assist the geotechnical consultant in digging test pits for testing compacted fill. As recommended by the geotechnical consultant, the contractor should "shut down" or remove grading equipment from an area being tested.

Maximum density testing may be required of samples obtained directly from the fill areas in order to verify conformance with the specifications. Processing of these additional samples may take two or more working days. The contractor may elect to move the operation to other areas within the project, or may continue placing compacted fill pending laboratory and field test results. Should he elect the second alternative, fill placed is done so at the contractor's risk.

6.5 For field testing purposes, near optimum moisture will vary with material type and other factors including compaction procedure. Near optimum moisture may be specifically recommended in Preliminary Investigation Reports and/or may be evaluated during grading. As a preliminary guideline, near optimum moisture should be considered from one percent below to three percent above optimum.

Prior to placement of additional compacted fill following an overnight or other grading delay, the exposed surface or previously compacted fill should be processed by scarification, watered, or dried as needed, thoroughly blended to near optimum moisture conditions, then recompacted. Where wet, dry, or other unsuitable material exists to depths of greater than eight inches, the unsuitable material should be overexcavated.

Excavated on-site material which is acceptable to the geotechnical consultant may be utilized as compacted fill. Where import materials are required for use on site, the geotechnical consultant should be notified at least 72 hours in advance of importing in order to sample and test materials from proposed borrow sites.

6.6 Rocks up to a maximum diameter of 12 inches may be utilized within the compacted fill provided they are placed in such a manner that nesting of the rock is avoided and compaction is achieved. Rocks greater than six inches in maximum diameter should not be placed in the upper four feet of compacted fill. Fill should be placed and thoroughly compacted over and around all rock. The amount of rock should not exceed 40 percent by dry weight passing the 3/4-inch sieve size.

During the course of grading operations, rocks or similar irreducible materials greater than 12 inches maximum diameter (oversized material) may be generated. These rocks should not be placed within the compacted fill unless accepted and placed in conformance with the geotechnical consultant's recommendations.

6.7 Unless otherwise recommended by the geotechnical consultant and approved by the regulating agencies, permanent fill slopes should not be steeper than 2:1 (horizontal to vertical).

7.0 TRENCH BACKFILL

7.1 Utility trench backfill should, unless otherwise recommended, be compacted by mechanical means. The degree of compaction should be a minimum of 90 percent of the laboratory maximum density unless otherwise recommended. As an alternative, where agreed to in advance by the geotechnical consultant, granular material (sand equivalent greater than 30) may be thoroughly jetted in place. Jetting should only be considered to apply to trenches no greater than two feet in width and four feet in depth. Following jetting operations, trench backfill should be thoroughly mechanically compacted and/or wheel-rolled from the surface.

7.2 Backfill of exterior and interior trenches extending below a 1:1 projection from the outer edge of foundations should be mechanically compacted.

7.3 Clean granular backfill and/or bedding are not recommended in slope areas unless provisions are made for a drainage system to mitigate the potential buildup of seepage forces.

8.0 STATUS OF GRADING

Prior to proceeding with any grading operation, the geotechnical consultant should be notified at least two working days in advance in order to schedule the necessary observation and testing services. Prior to any significant expansion or cutback in the grading operation, the geotechnical consultant should be provided with adequate notice (i.e., two days) in order to make appropriate adjustments in observation and testing services. Following completion of grading operations, and/or between phases of a grading operation, the geotechnical consultant should be provided with at least two working days notice in advance of commencement of additional grading operations.

wpdata\stddocs\grading

American Geotechnical, Inc.
SOIL, FOUNDATION AND GEOLOGIC STUDIES

July 9, 2009 File No. 33054-01R

Alexander and Hibbs AIA, Inc.
1200 N. Jefferson Street, Suite A
Anaheim, CA 92807

Attention: Mr. Allen Hibbs

Subject: **RESPONSE TO THE COUNTY OF ORANGE, PLANNING AND DEVELOPMENT SERVICES/GRADING SECTION, GEOTECHNICAL REPORT REVIEW SHEET, DATED JUNE 10, 2009**
Grading Plan Check No.: GB090039
Proposed 401 South Tustin Renovation Project
Orange County Healthcare Agency
401 South Tustin Avenue
Orange, California

Reference: **GEOTECHNICAL INVESTIGATION REPORT**
Proposed 401 South Tustin Renovation Project
By: American Geotechnical, Inc.
Dated: August 27, 2008 (File No. 33054-01)

Dear Mr. Hibbs:

This report presents our response to the review comments by the County of Orange given in the above-referenced review sheet dated June 10, 2009, regarding development of the subject project. For ease of referencing, review comments are repeated herein followed by our responses. A copy of the review sheet is attached with this response letter.

Item 1:

Please review the grading plan to be used. Provide additional recommendations as necessary.

Response:

When available, grading plans will be reviewed for conformance with our recommendations. Additional recommendations will be provided at that time if necessary.

22725 Old Canal Road, Yorba Linda, CA 92887 (714) 685-3900 - FAX (714) 685-3909
5764 Pacific Center Blvd., Suite 112, San Diego, CA 92121 (858) 450-4040 - FAX (858) 457-0814
10398 Rockingham Drive, Suite 7, Sacramento, CA 95827 - (916) 368-2088 - FAX (916) 368-2188
5600 Spring Mountain Road, Suite 201, Las Vegas, NV 89146 - (702) 562-5046 - FAX (702) 562-2457

Item 2:

Cooperate with civil engineer to see that your recommendations are incorporated into the plans.

Response:

We will communicate and cooperate with the project civil engineer.

Item 3:

Please provide soil design parameters for slab-on-grade per Section 1805.8.2 of the 2007 CBC.

Response:

Our investigation indicated that the near surface soil consists of low expansive materials (Expansion Index up to 31). In order to reduce expansive soil influences, we recommend that the upper two feet of the slab subgrade be presaturated in addition to the recommendations provided in our report. In our opinion, the foundation/slab recommendations provided in our report along with the subgrade presaturation are adequate for the site conditions. However, per CBC Section 1805.8.2, the following soil parameters based on the Post-Tension Institute methodology (PTI, 3rd Edition) are provided for further analysis.

Allowable Soil Bearing Pressure	1,500 psf
Edge Moisture Variation Distance (e_m), Center Lift	9 feet
Edge Moisture Variation Distance (e_m), Edge Lift	5 feet
Differential Soil Movement (y_m), Center Lift	0.3 inch
Differential Soil Movement (y_m), Edge Lift	0.6 inch

The structural engineer should utilize these parameters to analyze the slab/foundation system. Based on this analysis, the slab/foundation system can be upgraded, but not less than the recommendations given our report. In other words, our recommendations should be considered as a minimum.

Item 4:

Please address site grading per Section 1803.3 of the 2007 CBC. Provide appropriate recommendations and justification for any exceptions.

Response:

We recommend that all the requirements of the CBC Code Section 1803.3 be followed. Per this code scenario, impervious surfaces such as concrete flatwork within 10 feet from the buildings should be sloped at a minimum 2 percent ratio away from the buildings. As we understand, concrete flatwork is proposed around the entire perimeter of the building.

Item 5:

The historical high groundwater levels should be used to evaluate potential for liquefaction at the site, unless there is other compelling information showing otherwise. Please submit your published groundwater information (OCWD, 2005) showing otherwise and justify which level is correct.

Response:

As requested, a copy of the OCWD November 2005 groundwater elevation map is attached at the back of this report. According to this map, the groundwater is located at about 60 feet MSL. For the estimated site elevation of 230 MSL, the groundwater is located at about 170 feet below the ground surface. It should also be realized that our site specific investigation indicated no static or perched groundwater table up to the maximum explored depth (up to 50 feet below the ground surface).

As indicated in our report, the highest historic groundwater level was approximately 20 feet below the ground surface. This highest historic groundwater may be due to poor flood control measures available at that time. Within the last 40 years, significant flood control structures have been constructed along Santiago Creek and no significant flooding has occurred since 1969 With the present more advanced flood control measures it is highly unlikely that the water table will reach the highest historic level. However, to be conservative, a liquefaction analysis of the site was performed under the assumption that the groundwater is located at the historic level of 20 feet below the ground surface. It should be noted that the site is underlain by alluvial material consisting of predominantly gravel. The SPT blow counts obtained during our borings for this

predominantly gravel layer varied from about 50 to 100. To be conservative, knowing the possible influence by the gravel, the minimum SPT value of 50 was utilized uniformly throughout the gravel layer in the analysis. The liquefaction results are presented in the attached Appendix A. As shown, the site is not liquefiable.

Item 6:

If historical high groundwater levels need to be considered, re-address liquefaction potential and provide appropriate analysis/calculations. Please see second paragraph on Page 43 of Revised SP117 (2008). If blow counts are used in your calculations, justify that gravel has not erroneously skewed the results. Provide recommendations for correction as necessary.

Response:

Please see our response to Item 5. Even though we believe that the historic groundwater level is not applicable, a site specific liquefaction analysis has been performed and the results are included in Appendix A. Since our analysis predicts no liquefaction, lateral spreading at the site is unlikely. As indicated above, the least SPT blow count obtained in field testing was utilized in the analysis considering possible influences from gravel.

Item 7:

Update your report to reflect current site conditions.

Response:

There is no significant change of the site conditions from the time of investigation, and as such, the conclusions and recommendations provided in our original report and this letter are still applicable and appropriate. No revisions other than indicated herein are necessary.

We hope that this is the information you require at this time. Please contact us should you have any questions.

Respectfully submitted,

AMERCIAN GEOTECHNICAL, INC.

Arumugam Alvappillai, Ph.D.
Principal Engineer
G.E. 2504

Craig C. Chase
Principal Geologist
C.E.G. 2225

AA/CCC:dd

Enclosures: Copy of the OCWD November 2005 groundwater elevation map
Review Sheet dated June 10, 2009
Appendix A – Liquefaction Analysis

Distribution: 2 – Addressee (Regular Mail and Email: ahibbs@alexanderandhibbs.com)
1 – Willdan Engineering (Email: pmeng@willdan.com)

wpdata/oc/33054-01 aa.ccc.dd.july.2009.ResponseToReviewSheet

Planning & Development Services/Grading Section
300 North Flower Street, Santa Ana, California 92703
P.O. Box 4048, Santa Ana, California 92702

TELEPHONE: (714) 834-3326 FAX: (714) 834-5413

PC

sheet 1 of 2

GEOTECHNICAL REPORT REVIEW SHEET

Review of Geotechnical Reports Including:

	REPORT DATE	DATE RECEIVED	TYPE OF REPORT
[ ] Soil Report			[✓] Preliminary (Initial
[ ] Geologic Report			[ ] Ingrading or Interim
[✓] Geotechnical	8-27-08	5-20-09	[ ] Roughgrade Compacting
[ ] Response			[ ] Supplemental or Addendum
[ ] Other			[ ] Grading Plan Review
			[ ] Paving Design
			[ ] Final
			[ ]

By the Consulting Firm: American Geotechnical Their Job No. 33054-01

Tract Number: Lots:
Site Address: 401 South Tustin St., Orange
[✓] Grading Plan Check Number: GB090039
[ ] Preliminary Grading Permit No. [ ] Precise Grading Permit No.

Developer/Owner: Alexander & Hibbs AIA
Address: 1200 N. Jefferson St. Suite A Anaheim, CA 92807
Soil Engineer: Arumugam Alvappillai & Mohammad Joolazadeh
Engineering Geologist: Craig Chase

[ ] Report Approved [ ] Report Approved Subject to Conditions Below:
[✓] Prior to Approval of report attend to items listed below:
[ ] Prior to County Rough Grade approval and issuance of Building Permits submit:

1) Please review the grading plan to be used. Provide additional recommendations as necessary.
2) Cooperate with civil engineer to see that your recommendations are incorporated into the plans.
3) Please provide soil design parameters for slab-on-grade per section 1805.8.2 of the 2007 CBC.
4) Please address site grading per section 1803.3 of the 2007 CBC. Provide appropriate recommendations and justification for any exceptions.
5) The historical high groundwater levels should be used to evaluate potential for liquefaction at the site, unless there is other compelling information showing otherwise. Please submit your published groundwater information (OCWD, 2005) showing otherwise and justify which level is correct.
6) If historical high groundwater level needs to be considered readdress liquefaction potential and provide appropriate

Report Reviewed By: Nick Bebek, Engineering Geologist Date: 6-10-09

Planning & Development Services/Grading Section
300 North Flower Street, Santa Ana, California 92703
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FAX: (714) 834-5412

Sheet 2 of 2

GEOTECHNICAL REPORT REVIEW SHEET

Review of Geotechnical Reports Including:

	REPORT DATE	DATE RECEIVED	TYPE OF REPORT
[ ] Soil Report			[ ] Preliminary (Initial
[ ] Geologic Report			[ ] Ingrading or Interim
[ ] Geotechnical			[ ] Roughgrade Compacting
[ ] Reponse			[ ] Supplemental or Addendum
[ ] Other			[ ] Grading Plan Review
			[ ] Paving Design
			[ ] Final
			[ ]

By the Consulting Firm:

Their Job No.

Tract Number: Lots:
Site Address:
[✓] Grading Plan Check Number: GB090039
[ ] Preliminary Grading Permit No. [ ] Precise Grading Permit No.

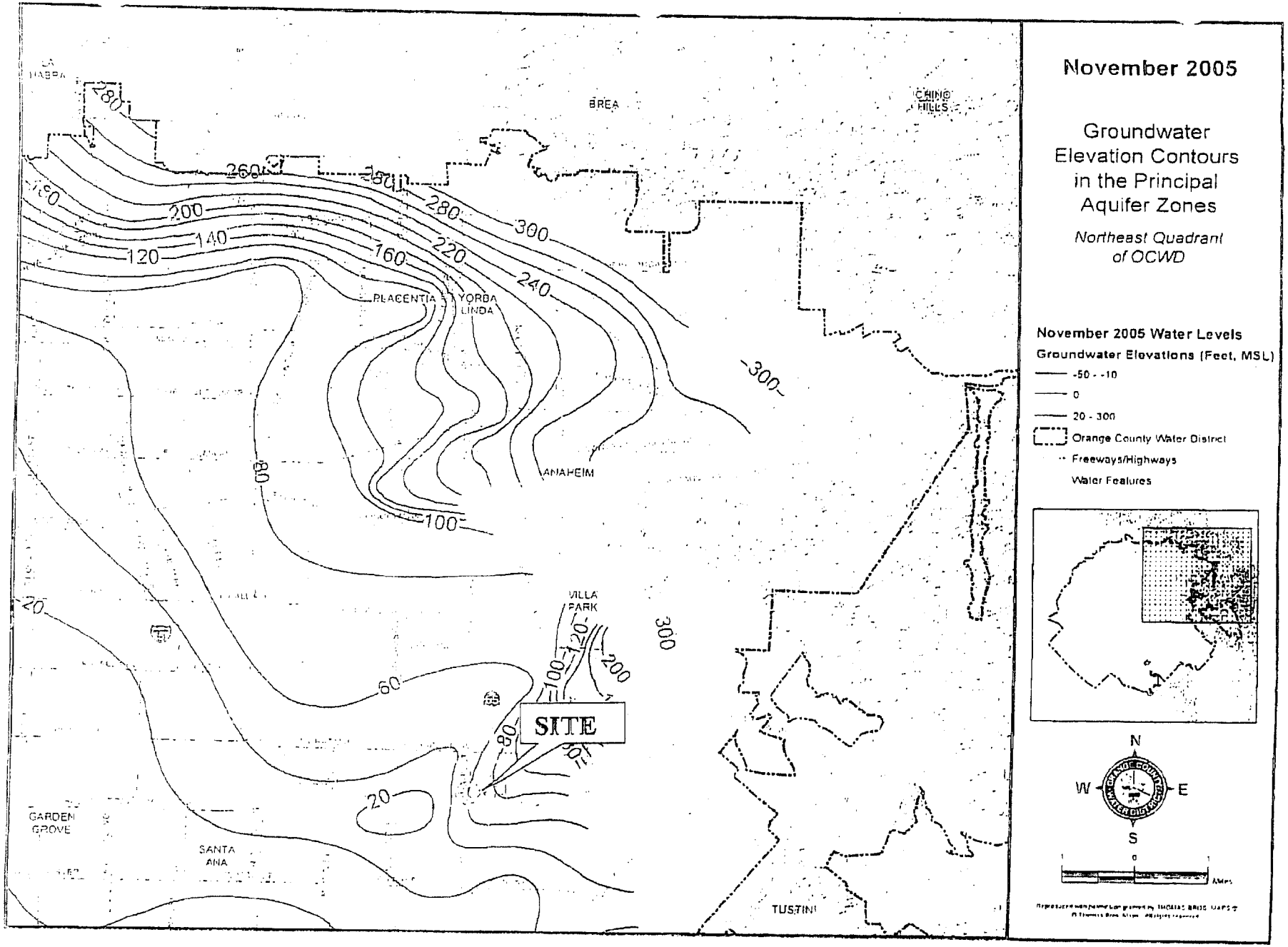
Developer/Owner:
Address:
Soil Engineer:
Engineering Geologist:

[ ] Report Approved [ ] Report Approved Subject to Conditions Below:
[ ] Prior to Approval of report attend to items listed below:
[ ] Prior to County Rough Grade approval and issuance of Building Permits submit:

analysis/calculations. Please see second paragraph on page 43 of Revised SP 117 (2008). If blow counts are used in your evaluation, justify that gravel has not erroneously skewed the results. Provide recommendations for correction as necessary.
7) Update your report to reflect current site conditions.

Report Reviewed By:
Nick Bebek, Engineering Geologist

Date: 6-10-09


November 2005
Groundwater Elevation Contours in the Principal Aquifer Zones
Northeast Quadrant of OCWD
November 2005 Water Levels
Groundwater Elevations (Feet, MSL)
-50 - -10
0
20 - 300
Orange County Water District
Freeways/Highways
Water Features
N
W
E
S
SITE
BREA
CHINO HILLS
PLACENTIA
YORBA LINDA
ANAHEIM
VILLA PARK
GARDEN GROVE
SANTA ANA
TUSTIN
300
280
260
240
220
200
160
140
120
100
80
60
20

File No. 33054-01
July 9, 2009

APPENDIX A

Liquefaction Analysis

SPT-Based Liquefaction Triggering Analysis for a single boring

Input parameters

Peak ground accel. (g) = 0.372 g

Earthquake magnitude M = 7.8

Water table depth (m) = 6.1 m = 20 feet

Average γ above water table = 16.9 kN/m³ = 107.6 lbs/cu ft

Average γ below water table = 24.7 kN/m³ = 157 lbs/cu ft.

Borehole diameter (mm) = 203.2 mm = 8 inch

Requires correction for sampler liners (YES/NO) NO

Rod lengths assumed equal to the depth plus 1.5 m (for the above ground extension)

SPT Sample Number	Depth (m)	Measured N	Soil type (USCS)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)	Energy ratio ER (%)	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ'_{vc} (kPa)	C_N	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60\text{-}cs}$	Stress reduct. Coeff, r_d	CSR	MSF for sand	K_σ for sand	CRR for M=7.5 & σ_{vc}'=1 atm	CRR	Factor of Safety
1	1.52	50	GM		14	75	1.25	1.15	0.80	1.00	57.5	25.8	25.8	1.70	97.8	2.9	100.66	1.00	0.241	0.92	1.10	2.00	2.00	2.00
2	3.05	50	GM		14	75	1.25	1.15	0.85	1.00	61.1	51.6	51.6	1.40	85.5	2.9	88.42	0.99	0.239	0.92	1.10	2.00	2.00	2.00
3	4.57	50	GM		14	75	1.25	1.15	0.95	1.00	68.3	77.3	77.3	1.14	78.0	2.9	80.94	0.97	0.236	0.92	1.08	2.00	1.96	2.00
4	6.10	50	GM		14	75	1.25	1.15	0.95	1.00	68.3	103.1	103.1	0.99	67.6	2.9	70.48	0.96	0.232	0.92	0.99	2.00	1.84	2.00
5	7.62	50	GM		14	75	1.25	1.15	0.95	1.00	68.3	140.7	125.8	0.90	61.2	2.9	64.09	0.94	0.256	0.92	0.94	2.00	1.73	2.00
6	9.14	50	GM		14	75	1.25	1.15	1.00	1.00	71.9	178.3	148.4	0.82	59.3	2.9	62.19	0.93	0.270	0.92	0.89	2.00	1.64	2.00
7	10.67	50	GM		14	75	1.25	1.15	1.00	1.00	71.9	216.0	171.1	0.77	55.2	2.9	58.13	0.91	0.278	0.92	0.84	2.00	1.56	2.00
8	12.19	50	GC		14	75	1.25	1.15	1.00	1.00	71.9	253.6	193.8	0.72	51.9	2.9	54.80	0.89	0.282	0.92	0.81	2.00	1.49	2.00
9	13.72	50	GC		14	75	1.25	1.15	1.00	1.00	71.9	291.2	216.4	0.68	49.1	2.9	52.01	0.87	0.284	0.92	0.78	2.00	1.43	2.00
10	15.24	50	SC		14	75	1.25	1.15	1.00	1.00	71.9	336.3	243.6	0.64	46.3	2.9	49.18	0.85	0.283	0.92	0.74	2.00	1.37	2.00

N = measured blow count

ER = the measured energy ratio as a percentage of the theoretical maximum

C_E = SPT correction factor for energy ratio; $C_E = ER_m/60$

C_B = SPT correction factor for borehole diameter; C_B = 1.0 for borehole diameter of 65-115 mm; C_B = 1.05 for borehole diameter of 150 mm; C_B = 1.15 for borehole diameter of 200 mm

C_R = SPT correction factor for rod length; C_R = 0.75 for rod length < 3 m; C_R = 0.80 for rod length 3-4 m; C_R = 0.85 for rod length 4-6 m; C_R = 0.95 for rod length 6-10 m; C_R = 1.00 for rod length 10-30 m

C_S = SPT correction factor for omitting sampler liners; C_S = 1.1 for N_{60} <= 10; C_S = 1 + N_{60}/100 for 10 <= N_{60} <= 30; C_S = 1.3 for N_{60} >= 30

N_{60} = the blow count for an energy ratio of 60%; $N_{60} = C_E C_B C_R C_S N_m$

σ_{vc} = overburn stress

σ'_{vc} = effective overburn stress

C_N = SPT overburden correction factor

$(N_1)_{60}$ = SPT blow count corrected to ER = 60% and an effective overburden stress of 1 atm; $(N_1)_{60} = C_N N_{60}$

r_d = stress reduction coefficient

CSR = cyclic stress ratio

MSF = magnitude scaling factor; $MSF = CRR_M/CRR_{M=7.5}$

K_σ = overburden correction ratio

CRR = cyclic resistance ratio

Factor of safety = CRR/CSR, maximum factor of safety = 2.0