REPORT OF GEOTECHNICAL INVESTIGATION UPDATE

Alvarado Creek Project
7407 Alvarado Road
La Mesa, California

JOB NO. 04-8598
15 February 2019

Prepared for:

Mr. Michael Brekka
RV COMMUNITIES, LLC

Geotechnical Exploration, Inc.
SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

15 February 2019

RV COMMUNITIES, LLC
7855 Herschel Avenue, Suite 201
La Jolla, CA 92037-4429
Attn: Mr. Michael Brekka

Job No. 04-8598

Subject: **Report of Geotechnical Investigation Update**
Alvarado Creek Project
7407 Alvarado Avenue
La Mesa, California

Dear Mr. Brekka:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has performed an updated investigation of the soil and geologic conditions at the location of the subject site. Exploratory cone penetrometer testing was performed at the site on March 26, 2004, exploratory trenches were placed on the site on April 26, 2004, and supplemental drilling was performed on July 24, 2018.

In our opinion, if the conclusions and recommendations presented in this report are implemented during site preparation, the site will be suited for the proposed apartment structures, partially subterranean parking and associated improvements from a geotechnical perspective.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 04-8598** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project (for site location, refer to Figure No. I).

I. PROJECT SUMMARY

It is our understanding, based on communications with Mr. Michael Brekka, that the existing RV Park is to be removed, and the site is to be developed to receive a new residential project consisting of four 5-story apartment buildings constructed over three levels of parking, with the lowest level partially subterranean. Architectural and civil plans were available for our review.

Our investigation revealed that the site is underlain by a thin veneer of silty sandy fill soil that is, in turn, underlain by sandy stream deposits along the southern perimeter of the property. Cobble conglomerate formational materials of the Stadium Conglomerate underlie the entire site at relatively shallow depths. We anticipate the structures will utilize either CMU or poured-in-place concrete for the basement walls, and standard building materials for the above-grade portions of the structures. The foundations will utilize deepened continuous perimeter and isolated spread footings with a slab on-grade foundation system.

Appropriate subdrains and waterproofing will be required for proposed basement/subgrade areas. We note that the water table was encountered at a depth of 3 feet below existing grade along the southern perimeter of the property during our field investigation. Dewatering will most likely be necessary during excavation for the basement levels. Dewatering during construction and sealing of potentially submerged building areas will be required in the proposed partial basement areas.

With the above in mind, the Scope of Work is briefly outlined as follows:

1. Identify and classify the surface and subsurface soils in the area of the proposed construction, in conformance with the Unified Soil Classification System (see Figure Nos. III and IV and Appendix A).

2. Make note of any faults or significant geologic features that may affect the site (see Figure No. VI).

3. Evaluate the existing and proposed bearing soil material.

4. Recommend the allowable bearing capacities for the existing on-site medium dense to dense natural soils and existing fill (if applicable).

5. Recommend site preparation procedures.

6. Evaluate the settlement potential of the bearing soils under the proposed structural loads.

7. Recommend preliminary foundation design information and provide active and passive earth pressures to be utilized in design of retaining walls or foundation structures.

II. SITE DESCRIPTION

The subject site is identified as Assessor's Parcel No. 469-021-01 and is comprised of 11.87 acres according to Map 346, in the City of La Mesa, County of San Diego, State of California. Presently, the site consists of a relatively level developed pad with approximate elevations ranging from 408 feet above mean sea level (MSL) to 425 feet above MSL. Site elevations were obtained from a topographic survey prepared by Fusce Engineering, dated August 28, 2018.

The site is bordered to the north by Alvarado Road and Interstate 8; to the south by Alvarado Creek and the right-of-way for the San Diego Trolley system and to the east and west by existing commercial properties. Concrete pads for recreational vehicle parking and adjacent asphalt-covered parking and driveway areas currently exist on the site. Overhead electric lines with power poles extending in an east-west direction are located through the central portion of the site.

III. FIELD INVESTIGATION

Three exploratory trenches and two bucket rig borings were placed on April 26, 2004, and July 24, 2018, respectively. They were placed specifically in accessible areas where the new structures and improvements are to be located (and where feasible due to existing structures and utilities on the site). Refer to Figure No. II for the exploratory excavation locations. The soil in the trenches was logged by our field representative, and bulk and in situ soil samples were taken of the predominant soils throughout the field operation. Excavation logs have been prepared on the basis of our observations and the results have been summarized on Figure No. III. The predominant soils have been classified in conformance with the Unified Soil Classification System (refer to Appendix A).

Nine cone penetrometer tests (CPT) were placed on the property on March 26, 2004. The CPT soundings were located in the field by referring to a site plan of the existing San Diego RV Park. CPT logs have been provided by Holguin, Fahan & Associates, Inc., and are included as Figure No. IV of this report. The predominant soils have been classified in conformance with the Unified Soil Classification System (refer to Appendix A) based on correlations obtained by Robertson and Campanella between soil types and tip and friction resistance measured by the cone penetrometer. Cone penetrometer measurements were extended up to a maximum depth of 8 feet at the location of CPT-5 and CPT-7.

Soil behavior interpretations are based on research performed by Robertson, P.K. and Campanella, R.C., 1989 *"Guidelines for* Geotechnical *Design Using the Cone Penetrometer Test and CPT with Pore Pressure Measurement*," Soil Mechanics Series No. 120, Civil Engineering Department, University of British Columbia, Vancouver, B.C., V6T1Z4, Sept. 1989.

The cone penetration tests were performed to aid in evaluating soil types, basic strength parameters, and the liquefaction potential of the existing subsurface soils, and to aid in developing appropriate site preparation and foundation design recommendations.

IV. GENERAL GEOLOGIC DESCRIPTION

The San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 84 miles from La Mesa) and westward to the San Clemente Fault (approximately 72 miles off-shore from La Mesa) (Berger and Schug, 1991).

During recent history, the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes have been experienced in historic time within the Chula Vista area. Since earthquakes have been recorded by instruments (since the 1930s), the Chula Vista area has experienced scattered seismic events with Richter magnitudes (M) generally less than 4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay; three of these earthquakes had recorded magnitudes of M4.0 to M4.2. In addition, the Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, had a magnitude of M5.3 (Hauksson, 1988).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (Hart, E.W., 1980), an "active" fault is one that has had ground surface displacement within Holocene time (about the last 11,000 years). Additionally, faults along which major historical earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Division of Mines and Geology defines a "potentially active" fault as one that has had ground surface displacement during Quaternary time, that is, during the past 11,000 to 1.6 million years (Hart, E.W., 1980).

V. SITE-SPECIFIC GEOLOGIC DESCRIPTION

A geologic investigation of the site was conducted to evaluate the on-site geology and potential of geologic hazards that might affect the site. Our investigation drew upon information gathered from published and unpublished geologic maps and reports, as well as results of our recent exploratory trenches and electronic cone penetration soundings.

A. Stratigraphy

Our field investigation and review of pertinent geologic maps and reports indicate that some artificial fill soils, stream deposits and dense formational materials underlie the site. Construction of the subterranean parking below the structures would result in partial removal of the encountered fill soils and stream deposits beneath the proposed structures.

Artificial Fill (Qaf): A limited amount of fill (up to approximately 2 to 3 feet) was encountered on the surface of the site. The encountered fill is loose to medium dense and consists of damp, red-brown to gray-brown, silty, fine to medium and fine to coarse sand with pebbles and cobbles. The shallow fill soils are considered to have a low expansion potential. These fill soils are not suitable in their current condition for bearing support. To be utilized as fill soils they require excavation and recompaction. Refer to Figure Nos. III and IV for details.

Stream Deposits: The fill soils along the southern perimeter of the property are underlain by stream deposits to an approximate depth of 9 feet below the present surface grade. As encountered on this site, the stream deposits consist of a medium dense, wet, tan-gray and orange-brown, fine to coarse sand with abundant cobbles and boulders (to 14 inches in diameter). These materials are considered to be of low expansion potential. These stream deposit soils are not suitable in their current condition for bearing support. To be utilized as bearing soils they require excavation and recompaction. Refer to Figure Nos. III and IV for details.

Stadium Conglomerate Formation (Tst): The site is underlain at depth by dense cobble conglomerate formational material of the Tertiary Stadium Conglomerate Formation. These formational soils are considered to have a negligible to very low liquefaction potential and low consolidation and expansion potential characteristics. Refer to Figure Nos. III and IV.

B. Structure

Slopes and roadcuts nearby allowed observation of bedding and geologic structural features of the Stadium Conglomerate Formation in the vicinity of the subject lot. The observed Stadium Conglomerate formational material appears to be massively bedded with no indication of measured strike and dip. Previous investigations performed in the vicinity by our firm, as well as review of a geologic map of the area (Kennedy and Tan, 2008), indicate that the area, in general, is commonly underlain by generally flat bedding.

VI. GEOLOGIC HAZARDS

The following is a discussion of the geologic conditions and hazards common to the La Mesa area, as well as project-specific geologic information relating to development of the subject property.

A. Local and Regional Faults

It is our opinion that a known "active" fault presents the greatest seismic risk to the subject site during the lifetime of the proposed structure. To date, the nearest known "active" faults to the subject site are the northwest-trending Rose Canyon Fault, Coronado Bank Fault and the Elsinore Fault.

Rose Canyon Fault: The Rose Canyon Fault Zone (Mount Soledad and Rose Canyon Faults), located approximately 7.4 miles west of the subject site, is mapped trending north-south from Oceanside to downtown San Diego, from where it appears to head southward into San Diego Bay, through Coronado and offshore. The Rose Canyon Fault Zone is considered to be a complex zone of onshore and offshore, en echelon strike slip, oblique reverse, and oblique normal faults. The Rose Canyon Fault is considered to be capable of causing a M7.5 earthquake and considered microseismically active, although no significant recent earthquake is known to have occurred on the fault. Investigative work on faults (believed to be part of the Rose Canyon Fault Zone) at the Police Administration and Technical Center in downtown San Diego and at the SDG&E facility in Rose Canyon, has encountered offsets in Holocene (geologically recent) sediments. These findings have been accepted as confirmed Holocene displacement on the Rose Canyon Fault and this previously classified "potentially active" fault has now been upgraded to an "active" fault as of November 1991 (California Division of Mines and Geology -- Fault Rupture Hazard Zones in California, 1994).

Coronado Bank Fault: The Coronado Bank Fault is located approximately 21.3 miles southwest of the site. Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). The Oceanside earthquake of M5.3, recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a M7.0 earthquake and is of great interest due to its close proximity to the greater Oceanside metropolitan area.

Elsinore Fault: The Elsinore Fault is located approximately 34 miles northeast of the site. The fault extends approximately 200 km (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, Oceanside, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

Although the Elsinore Fault Zone belongs to the San Andreas set of active, northwest-trending, right-slip faults in the southern California area (Crowell, 1962), it has not been the site of a major earthquake in historic time, other than a M6.0 earthquake near the town of Elsinore in 1910 (Richter, 1958; Toppozada and Parke, 1982). However, based on length and evidence of late-Pleistocene or Holocene displacement, Greensfelder (1974) has estimated that the Elsinore Fault Zone is reasonably capable of generating an earthquake as large as M7.5. Faulting evidence exposed in trenches placed in Glen Ivy Marsh across the Glen Ivy North Fault (a strand of the Elsinore Fault Zone between Corona and Lake Elsinore), suggest a maximum earthquake recurrence interval of 300 years, and when combined with previous estimates of the long-term horizontal slip rate of 0.8 to 7.0 mm/year, suggest typical earthquakes of M6.0 to M7.0 (Rockwell, 1985). More recently, the California Geologic Survey (2002) considers the Elsinore Fault capable of producing an earthquake of M6.8 to M7.1.

Newport-Inglewood Fault: The Newport-Inglewood Fault Zone is located approximately 35 miles northwest of the site. A significant earthquake (M6.4) occurred along this fault on March 10, 1933. Since then no additional significant events have occurred. The fault is believed to have a slip rate of approximately 0.6 mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.4 (SCEC, 2004).

San Jacinto Fault: The San Jacinto Fault is located 55 miles to the northeast of the site. The San Jacinto Fault Zone consists of a series of closely spaced faults, including the Coyote Creek Fault, that form the western margin of the San Jacinto Mountains. The fault zone extends from its junction with the San Andreas Fault in San Bernardino, southeasterly toward the Brawley area, where it continues south of the international border as the Imperial Transform Fault (Earth Consultants International [ECI], 2009).

The San Jacinto Fault zone has a high level of historical seismic activity, with at least 10 damaging earthquakes (M6.0 to M7.0) having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto Fault in 1899 and 1918 caused fatalities in the Riverside County area. Offset across this fault is predominantly right-lateral, similar to the San Andreas Fault, although some investigators have suggested that dip-slip motion contributes up to 10% of the net slip (ECI, 2009).

The segments of the San Jacinto Fault that are of most concern to major metropolitan areas are the San Bernardino, San Jacinto Valley and Anza segments. Fault slip rates on the various segments of the San Jacinto are less well constrained than for the San Andreas Fault, but the available data suggest slip rates of 12 ±6 mm/yr for the northern segments of the fault, and slip rates of 4 ±2 mm/yr for the southern segments. For large ground-rupturing earthquakes on the San Jacinto fault, various investigators have suggested a recurrence interval of 150 to 300 years. The Working Group on California Earthquake Probabilities (WGCEP, 2008) has estimated that there is a 31 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault. Maximum credible earthquakes of M6.7, M6.9 and M7.2 are expected on the San Bernardino, San Jacinto Valley and Anza segments, respectively, capable of generating peak horizontal ground accelerations of 0.48g to 0.53g in the County of Riverside, (ECI, 2009). A M5.4 earthquake occurred on the San Jacinto Fault on July 7, 2010.

The United States Geological Survey has issued the following statements with respect to the recent seismic activity on southern California faults:

> The San Jacinto fault, along with the Elsinore, San Andreas, and other faults, is part of the plate boundary that accommodates about 2 inches/year of motion as the Pacific plate moves northwest relative to the North American plate. The largest recent earthquake on the San Jacinto fault, near this location, the M6.5 1968 Borrego Mountain earthquake April 8, 1968, occurred about 25 miles southeast of the July 7, 2010, M5.4 earthquake.
>
> This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, M7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same

area on June 12th at 8:08 pm (Pacific Time). Thus, this section of the San Jacinto fault remains active.

Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologists continue to monitor the ongoing earthquake activity using the Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

B. Other Geologic Hazards

Ground Rupture: Ground rupture is characterized by bedrock slippage along an established fault and may result in displacement of the ground surface. For ground rupture to occur along a fault, an earthquake usually exceeds M5.0. If a M5.0 earthquake was to take place on a local fault, an estimated surface-rupture length 1 mile long could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known fault trace and, therefore, the risk of ground rupture is remote.

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of M5.0 or greater are generally associated with significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structures. The anticipated ground accelerations at the site from earthquakes on faults within 100 miles of the site are provided in Tables 1 and 2 of Appendix B. For structural design purposes, the calculated site acceleration corresponding to a 10 percent probability of exceedance in 50 years is 0.20g.

Liquefaction: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a dense fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are shaken by an earthquake of sufficient magnitude.

On this site, the risk of liquefaction of foundation material due to seismic shaking is considered to be minimal due to the dense nature of the natural-ground material. No loss of strength is anticipated to occur to the on-site soils due to an anticipated seismic event.

Flooding: A review of San Diego County flood hazard maps indicates that the site is located within a relatively low risk area. Due to the property's location adjacent to Alvarado Creek, it should be anticipated that flooding may occur on the site if left at the existing grade should a 100-year flood or greater occur in the San Diego area. The grading plans should show the 100-year flood event elevation and the project should be designed to comply with the City of La Mesa potential flooding requirements.

Geologic Hazard Summary: It is our opinion, based upon a review of the available maps and our site investigation, that the site is underlain by relatively stable formational materials, and appears suited for the proposed residential construction. No significant geologic hazards are known to exist on the site that would prevent the proposed apartment structures and associated improvements.

VII. GROUNDWATER

Free groundwater was encountered at a depth of 3 feet at the location of exploratory trench T-1 (2004) located along the south perimeter of the property adjacent to the Alvarado Creek and borings B-1 and B-2 (2018) in the western portion of the site. If moisture-related effects such as water seeps, flooding, efflorescence or high vapor emissions in the proposed subterranean garages are not acceptable, measures should be taken to reduce these issues. Remedial measures may include designing the below-grade parking structures to be watertight, anti-vapor membranes below the slab on-grade and/or utilizing a concrete mix having a maximum water-to-cement ratio of 0.45. Dewatering will most likely be necessary during excavation and grading for the subterranean garage levels of the structures.

Design considerations should be made to alleviate this situation. Subsurface drainage with a properly designed and constructed subdrain system will be required along with continuous back drainage behind the below-grade garage walls and any proposed elevator pits. Furthermore, the subterranean garages should be provided with the proper cross-ventilation to help reduce the potential for moisture-related problems as previously stated. The garage slabs should also be properly protected by proper sealing and waterproofing to help reduce potential for moisture intrusion. At the time of grading, subgrade observations may require the placement of subdrains under the garage slabs if groundwater is observed.

It should be kept in mind that any required grading operations may change surface drainage patterns and/or reduce soil permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

It must be understood that unless discovered during initial site exploration or encountered during site grading operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Water conditions encountered during grading operations should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and future property owners must realize that unanticipated post-construction appearances of groundwater may have to be dealt with on a site-specific basis.

VIII. LABORATORY TESTS AND SOIL INFORMATION

Laboratory tests were performed on the disturbed and relatively undisturbed soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed apartment structures. The following tests were conducted on the sampled soils:

1. Moisture Content (ASTM D2216-98)
2. Moisture/Density Relations (ASTM D1557-98, Method A)
3. Mechanical Analysis (ASTM D422-98)

The moisture content of a soil sample is a measure of the weight of water, expressed as a percentage of the dry weight of the sample.

The relationship between the moisture and density of remolded soil samples gives qualitative information regarding the soil strength characteristics and soil conditions to be anticipated during any future grading operation.

The expansion potential of soils is determined, when necessary, utilizing the Uniform Building Code Test Method for Expansive Soils (UBC Standard No. 29-2). In accordance with the UBC (Table 18-1-B), expansive soils are classified as follows:

EXPANSION INDEX	***POTENTIAL EXPANSION***
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very high

Based on our grain-size test results, our visual classification, and our experience with similar soils, the on-site silty fine to medium sand is considered to have a very low to low expansion potential (EI less than 50).

The Mechanical Analysis Test was used to aid in the classification of the soils according to the Unified Soil Classification System.

Based on laboratory test data, our experience with the formational materials in this area of La Mesa, our observations of the primary soil types on the project, and our previous experience with laboratory testing of similar formational soils, our Geotechnical Engineer has utilized conservative values for friction angle and cohesion for those formational soils and properly compacted fill soils that will have significant lateral support or bearing functions on the project. The assigned values have been utilized in determining the recommended allowable bearing capacity, as well as the active and passive earth pressures for wall and footing designs.

IX. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the practical field investigation conducted by our firm, and resulting laboratory tests, in conjunction with our knowledge and experience with the soils in the La Mesa area of the County of San Diego.

Our updated investigation revealed that the building sites are underlain by a thin veneer of fill soils that are, in turn, underlain by cobble conglomerate formational materials. Stream deposits underlie the fill soils along the southern perimeter of the site. The prevailing formational materials encountered on the site are dense and are of very low expansion and consolidation potential. These variable density fill soils and stream deposits will not provide a stable soil base for the proposed structures and associated improvements. As such, we recommend that these soils be removed and recompacted as part of site preparation prior to the addition of any new fill or structural improvements. It appears that excavation for the subterranean garage level will result in the removal of all of the fill soils and stream deposits in those areas. It is our opinion that the formational materials will provide adequate support for the proposed structures and improvements.

Excavation for the partially subterranean parking garages will result in the removal of the upper 3 to 4 feet of soil at the structure locations. Some areas will require removal of alluvium soils up to 8 feet in depth. The encountered sandy soil and cobble materials on the site are considered to be acceptable for export from the site. A shoring system may need to be installed in the areas of the planned garage basement excavations. In some areas of the site loose cohesionless soils may be encountered. Dewatering will most likely be necessary during excavation for the partially subterranean garage levels.

The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project.

> *If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept the responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the City Engineer in writing of such change prior to the recommencement of grading and/or foundation installation work.*

A. Seismic Design Criteria

1. *Seismic Design Criteria:* Site-specific seismic design criteria for the proposed residence are presented in the following table in accordance with Section 1613 of the 2016 CBC, which incorporates by reference ASCE 7-10 for seismic design. We have determined the mapped spectral acceleration values for the site, based on a latitude of 32.7725 degrees and longitude of -117.0378 degrees, utilizing a third-party tool provided by the USGS, which provides a

solution for ASCE 7-10 (Section 1613 of the 2016 CBC) utilizing digitized files for the Spectral Acceleration maps. Based on our experience with similar soil conditions, we have assigned a Site Soil Classification of C. Refer to the *"Seismic Design Map"* presented as Appendix B.

TABLE I

Mapped Spectral Acceleration Values and Design Parameters

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}
0.844	0.303g	1.2	1.5	1.013g	0.454g	0.675g	0.303g

B. Site Preparation

2. *Clearing and Stripping:* The existing improvements and vegetation observed on the site must be removed prior to the preparation of the site to receive new structural improvements.

3. *Treatment of Existing Fill and Colluvial Soils:* In order to provide a uniform, firm soils base for the proposed foundations and subterranean garage slabs on-grade, loose or insufficiently compacted soil materials (as evaluated by the project geotechnical engineer) should be excavated to expose dense formational materials. The excavation depth for surface soil removal and recompaction is anticipated to be approximately 3 to 4 feet. Surplus basement excavated materials are to be exported from the site. We recommend that shoring protection be installed in steep excavation areas as the basement grade is cut down to approximate finish subgrade elevation.

Any areas with loose soils exposed should be removed and recompacted to firm soils. Wet soils were encountered at an approximate depth of 3 feet under the majority of the site and should be anticipated during grading in this area.

It is anticipated that a subdrain system will be implemented during grading operations to control the groundwater that was encountered at a depth of 3 feet below the current ground surface elevation.

Any rigid improvements founded on the uncontrolled fill soils can be expected to undergo movement and possible damage and is therefore not recommended. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of the improvements. Any exterior area to receive concrete improvements should be verified for compaction and moisture within 48 hours prior to concrete placement.

4. *Subgrade Preparation for Basement Level:* Due to the presence of shallow groundwater under the site, especially along the south perimeter of the property, the grading contractor and general contractor should consider the need for sump pumps to pump out water anticipated to collect in the basement excavations. If the water flow is substantial, a subdrain system beneath the basement level, as well as permanent sump pumps, may need to be incorporated as part of the construction. Cut-off walls and subdrains would also aid in reducing the flow of groundwater into basements and other excavations deeper than 3 feet.

   Excavation for the subterranean garages should result in the removal of all of the fill soils and stream deposits at the garage locations. However, for ease of grading, any recompaction work of the surficial soils should be done before the basement excavations.

5. *Fill Compaction:* All structural fill and backfill should be compacted to a minimum degree of compaction of 90 percent at a minimum moisture content of 2 percent above the laboratory optimum based upon ASTM D1557-12. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either: 1) aerating and drying the fill if it is too wet; or 2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture.

   No uncontrolled fill soils should remain on the site after completion of any future site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils or left-in-place formational materials, the soils should be removed prior to completion of the grading operation.

   The soils in permanent access driveways to the lower-level garage areas should be watered to optimum moisture content and compacted to at least 90 percent of Maximum Dry Density, in accordance with ASTM D1557-12 standards.

6. *Trench and Retaining Wall Backfill:* Any buried objects or abandoned utility lines, etc. that might be discovered in the construction area should be removed and the excavation properly backfilled with compacted fill. New drainage and utility trenches should be backfilled in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means.

Any backfill soils placed behind retaining walls that support structures and other improvements (such as patios, sidewalks, etc.) should also be compacted to at least 90 percent of Maximum Dry Density.

Our experience has shown that even shallow, narrow trenches, such as for irrigation and electrical lines, which are not properly compacted, can result in problems, particularly with respect to shallow groundwater accumulation and migration.

C. Design Parameters for Proposed Foundations and Retaining Walls

7. *Footings:* Our preliminary foundation design of footings assumes that new footings will be placed at least 24 inches into dense natural (formational) soils and can be designed with an allowable soil bearing capacity of 2,500 pounds per square foot (psf). This applies to footings at least 12 inches in width. For wider and/or deeper footings, the allowable soil bearing capacity may be calculated based on the following equation:

$$Qa = 1000D+500W$$

where

"Qa" is the allowable soil bearing capacity (in psf);

"D" is the depth of the footing (in feet) as measured from the ***lowest*** adjacent grade; and

"W" is the width of the footing (in feet)

The allowable soil bearing capacity may be increased one-third for analysis including wind or earthquake loads. The total maximum allowable vertical bearing capacity for proposed shallow foundations should not exceed 5,000 psf.

8. *Lateral Loads:* The passive earth pressure of the encountered dense, natural-ground soils or properly compacted fill (to be used for design of shallow foundations and footings to resist the lateral forces) should be based on an equivalent fluid weight of 135 pounds per cubic foot (pcf) for footings below groundwater and 300 pcf for footings above groundwater. This passive earth pressure is only considered valid for design if the ground adjacent to the foundation structure is essentially level for a distance of at least three times the total depth of the foundation and is properly compacted or dense native soil. In addition, the lateral horizontal distance of properly compacted fill soils should extend at least three times the depth of foundation being considered.

   A Coefficient of Friction of 0.40 times the dead load may be used between the bearing soils and concrete foundations, walls, or floor slabs.

9. *Uplift:* Uplift pressure due to groundwater may need to be considered if the foundation is designed as a mat slab with basement walls (providing a "bath tub" effect).

10. *Settlement:* Based on our laboratory test results and our experience with the soil types on the subject site, the dense natural soils should experience differential angular rotation of approximately 1/300 under the allowable loads. The maximum differential settlement across the structure and footings when founded on dense natural formation should be on the order of 1½ inches.

11. *General Criteria for All Footings:* All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of four No. 5 reinforcing bars be provided in the footings (two near the top and two near the bottom. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 5 steel bars on 12-inch centers, both ways, with no less than three bars each way. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

    NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

D. Concrete Slab on-grade Criteria

12. *Minimum Floor Slab Thickness and Reinforcement:* Our experience indicates that, for various reasons, floor slabs occasionally crack, causing brittle surfaces such as ceramic tiles to become damaged. Therefore, we recommend that all slabs on-grade constructed on dense formational soils or properly compacted fill contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur.

12.1 The interior floor slabs for the basement garage areas should be a minimum of 5 inches actual thickness and be reinforced with No. 3 bars on 12-inch centers, both ways, placed at midheight in the slab. *Basement slabs should be underlain by a 6-inch-thick layer of crushed rock gravel or Class II base layer properly compacted. A waterproofing membrane may be included (such as Paraseal) if soil moisture is a concern to the owner/developer.* Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc.*** representative to have the proper moisture content within 48 hours prior to placement of a vapor barrier and pouring of concrete. The base layer should drain to a planned low point where a sump pump should be installed. A subdrain system, including several lines discharging into a collector subdrain pipe, will probably be required under the basement slabs.

12.2 Following placement of any concrete floor slabs, sufficient drying time must be allowed prior to placement of floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

13. *Concrete Isolation Joints:* We recommend the project Civil/Structural Engineer incorporate isolation joints and sawcuts to at least one-fourth the thickness of the slab in any floor designs. The joints and cuts, if properly placed, should reduce the potential for and help control floor slab cracking. It is recommended that concrete shrinkage joints be placed no further than 20 feet, approximately. However, due to a number of reasons (such as base preparation, construction techniques, curing procedures, and normal shrinkage of concrete), some cracking of slabs can be expected. ***Basement garage slabs***

should preferably be reinforced with enough steel to eliminate control joints if proper subdrain dewatering is a concern.

NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum safeguards to reduce possible crack separations.

14. *Slab Moisture Protection and Vapor Barrier Membrane:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested minimum protection criteria. Actual recommendations should be provided by the architect and waterproofing consultants.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls, and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-97 (2009) Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E154-88 (2005) Standard Test Methods for Water Vapor Retarders Used in Contact with Earth; ASTM E96-95 Standard Test Methods for Water Vapor Transmission of Materials; ASTM E1643-98 (2009) Standard Practice for Installation of Water Vapor Retarders Used in Contact Under Concrete Slabs; and ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

14.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and sub-paragraphs 7.1.1-7.1.5) should be less than 0.01 perms (grains/square foot/hour in Hg) and comply with the ASTM E1745 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643. The basis of design is 15-mil StegoWrap vapor barrier. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the vapor barrier be placed directly on properly prepared, smooth surface, properly compacted subgrade soils and the floor slabs be poured directly on the vapor barrier; no sand or gravel layers are used.

14.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped and sealed with mastic or the manufacturer's recommended tape or sealing products. In actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement.

14.3 Vapor retarders/barriers do not provide full waterproofing for structures constructed below free water surfaces. They are intended to help reduce or prevent vapor transmission and/or capillary migration through the soil and through the concrete slabs. Waterproofing systems must be designed and properly constructed if full waterproofing is desired. The owner and project designers should be consulted to determine the specific level of protection required.

15. *Exterior Slab Thickness and Reinforcement:* As a minimum for protection of on-site improvements, it is recommended that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be founded on properly compacted and tested fill or dense native formation and underlain by at least 24 inches of nonexpansive, properly compacted soils, with No. 3 bars at 18 inches on-center at the mid-height of the slab, and contain adequate isolation and control joints. The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observations and testing.

Control joints for exterior slabs should be placed no farther than 15 feet apart or the width of the slab, whichever is less, and also at reentrant corners. Control joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained.

E. ***Shoring Design Parameters***

16. *Basement Walls:* The basement excavations in areas where temporary slopes are not feasible will require shoring. Most likely a soldier pile and lagging option will be used. If this option is chosen, we recommend the following soil pressure distribution:

16.1 Any adjacent vertical surcharge may be converted to lateral pressure by multiplying by the 0.35 lateral pressure coefficient. For cantilever soldier pile and lagging, a linearly increasing soil pressure distribution may be used, with an active equivalent fluid weight equal to 35 pcf. In submerged portions, in addition to the water pressure of 62.4 pcf, the submerged soil pressure of 20 pcf should be added.

16.2 To calculate soldier pile passive force the following recommendation should be used: 300 pcf x 2.5 x diameter of drilled pier x depth of pile below the lowest adjacent excavation grade *above groundwater*. Use 135 pcf x 2.5 x diameter of soldier pile x depth of pile below lowest adjacent excavation grade for piles *below groundwater*.

Total allowable end-bearing resistance of drilled piles into formational soils should be calculated using the following: 1,000 x depth into formation x area of tip of drilled pile (to a maximum 20,000 psf). For shaft friction in submerged areas, use an average unit friction pressure (in psf) equal to 15Z, where Z is the depth below the lowest adjacent grade, in feet.

F. Retaining Wall Design Criteria

17. *Retaining Walls:* The active earth pressure above groundwater (to be utilized in the design of any cantilever walls allowed to rotate) should be based on an Equivalent Fluid Weight of 35 pcf (for level backfill only and low expansive ***imported soils***). We recommend that very low expansive to low expansive soils be used as wall backfill material. Therefore, retaining wall plans should indicate that walls be backfilled with very low to low expansive soils (i.e., expansion index less than 50). Unless the retaining walls are provided with an effective drainage system, an additional fluid pressure of 62.4 pcf should be added to the recommended soil pressure. Surcharge loads may be converted to a uniform horizontal soil pressure by using a conversion factor of 0.31.

    For design of restrained retaining walls, a triangular soil pressure increasing at a rate of 56 pcf in dry areas (or 28 pcf plus water pressure of 62.4 pcf for submerged areas) should be used. Any other surcharge loads applied within a horizontal distance measured from the face of the wall equal to its height should be considered in the structural design by converting the vertical surcharge to a horizontal uniform pressure by using a conversion factor of 0.47.

18. *Moisture/Water Seepage:* Due to presence of groundwater (derived primarily from the site's proximity to Alvarado Creek as well as rainfall and irrigation), excess moisture is a common problem in below-grade structures or behind

proposed basement level retaining walls. These problems are generally in the form of water seepage through walls, mineral staining, mold growth and high humidity.

Even without the presence of free water, the capillary draw characteristics, especially of fine-grained soils, can result in excessive transmission of water vapor through walls and floor slabs. In order to minimize the potential for moisture-related problems to develop at the site, proper ventilation and waterproofing should be provided for below-ground areas, and the backfill side of all structure retaining walls should be properly waterproofed and drained. As shown on Figure No. VII, the bottom of the perforated drain line should be at least 12 inches below the bottom of the interior floor slab. If the retaining wall subdrain is placed at the top of the foundation, water or moisture at slab subgrade may appear. In such cases, subdrains will need to be installed under the slab as previously discussed.

19. *Subdrains:* Proper subdrains and free-draining backwall material or geofabric drainage should be installed behind all retaining walls (in addition to proper waterproofing) on the subject project. ***Geotechnical Exploration, Inc.*** will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that the subdrains for any lower-level walls should be placed at an elevation at least 1 foot below the bottom of the lower level slabs. At least 0.5-percent fall should be provided for the subdrain. The subdrain should be placed in an envelope of crushed rock gravel up to 1 inch in maximum diameter, and be wrapped with Mirafi 140N filter or equivalent (see Figure No. VII). If a sump pump system is used to reduce the water pressure on basement/retaining walls, all subdrains should be directed toward sump pump locations.

G. *Slopes*

20. *Temporary Slopes:* Although shoring is proposed, we anticipate temporary slopes in formational material approximately 3 to 4 feet in height during the excavation process and construction of basement level garages. Based on the results of our field investigation, it is our opinion that the following temporary slope design criteria may be considered in areas where the proposed excavation slope top will be at least 10 feet away from any existing structures or improvements: The existing formational materials may be cut at an inclination of 0.75 horizontal to 1.0 vertical along the upper 5 feet (measured from present grade) and at 1.0:1.0 (horizontal to vertical) at lower elevations, for an unsupported period not to exceed eight weeks. After that time, our firm should evaluate the slope. No soil stockpiles or surcharge may be placed within a horizontal distance equal to the height of the excavation. Water seepage may require gentler temporary slopes be implemented.

Any plans for slopes in excess of the anticipated 12-foot maximum must be presented to our office prior to grading to allow time for review and specific recommendations, if warranted. Proper drainage away from the excavation should be provided at all times.

A representative of ***Geotechnical Exploration, Inc.*** must observe any steep temporary slopes *during construction*. In the event that soils comprising a slope are not as anticipated, any required slope design changes would be presented at that time.

H. ***Site Drainage Considerations***

21. *Erosion Control:* In addition, appropriate erosion control measures should be taken at all times during construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas or running uncontrolled over the tops of newly constructed cut or fill slopes. Particular care should be taken to prevent saturation of any temporary construction slopes.

22. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the building site after the structures and other improvements are in place. Drainage waters from this site and adjacent properties are to be directed away from the foundations, floor slabs, and footings, onto the natural drainage direction for this area or into properly designed and approved drainage facilities. Roof gutters and downspouts should be installed on the structure, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help minimize the potential for waters to seek the level of the bearing soils under the foundations, footings and floor slabs or further erosion of the adjacent natural slope. Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site. or running uncontrolled over the tops of newly constructed cut or fill slopes. Particular care should be taken to prevent saturation of any temporary construction slopes.

23. *Planter Drainage:* Planter areas, flower beds and planter boxes should be sloped to drain away from the foundations, footings, and floor slabs at a gradient of at least 5 percent within 5 feet from the perimeter walls. Planter

boxes should be constructed with a closed bottom and a subsurface drain, installed in gravel, with the direction of subsurface and surface flow away from the slopes, foundations, footings, and floor slabs, to an adequate drainage facility. Sufficient area drains and proper surface gradient should be provided throughout the project. Roof gutter and downspouts should be tied to storm drain lines.

24. *Drainage Quality Control*: It must be understood that it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

I. ***General Recommendations***

25. *Project Start Up Notification:* In order to minimize any work delays at the subject site during site development, this firm should be contacted 24 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

26. *Cal-OSHA:* Where not superseded by specific recommendations presented in this report, trenches, excavations and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by OSHA.

27. *Construction Best Management Practices (BMPs):* Construction BMPs must be implemented in accordance with the requirements of the controlling jurisdiction. At the very least, sufficient BMPs must be installed to prevent silt, mud or other construction debris from being tracked into the adjacent street(s) or storm water conveyance systems due to construction vehicles or any other construction activity. The contractor is responsible for cleaning any such debris that may be in the street at the end of each work day or after a storm event that causes breach in the installed construction BMPs.

    All stockpiles of uncompacted soil and/or building materials that are intended to be left unprotected for a period greater than 7 days are to be provided with erosion and sediment controls. Such soil must be protected each day when the probability of rain is 40% or greater. A concrete washout should be provided on all projects that propose the construction of any concrete improvements that are to be poured in place. All erosion/sediment control devices should be maintained in working order at all times. All slopes that are created or disturbed by construction activity must be protected against erosion and sediment transport at all times. The storage of all construction materials and equipment must be protected against any potential release of pollutants into the environment.

X. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be asked to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in the *"Report of Geotechnical Investigation Update"* for the project. In addition, the compaction of any fill soils placed during site grading work must be tested by a soil engineer. It is the responsibility of the grading contractor to comply with the requirements on the grading plans and the local grading ordinance. All retaining wall and trench backfill that will support structures or rigid improvements should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

XI. LIMITATIONS

Our conclusions and recommendations have been based on all available data obtained from our field investigation and laboratory analysis, as well as our experience with the soils and formational materials located in this area of the City of La Mesa. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time grading operations begin or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis which meet the contemporary standard of care in our profession within the County of San Diego. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the structural plans. We should be provided with the opportunity to review the project plans once they are available to verify that our recommendations are adequately incorporated.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 04-8598** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jay K. Heiser /pj

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Leslie D. Reed, President
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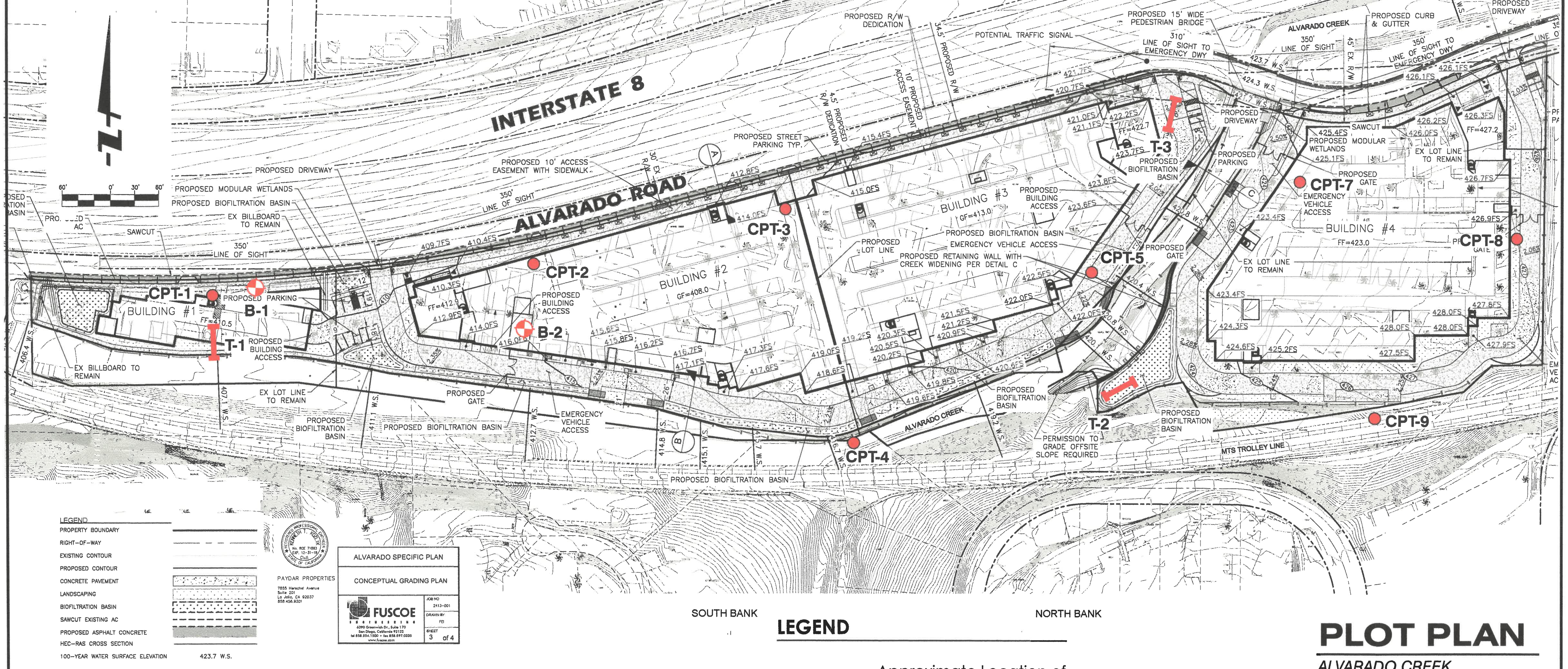
VICINITY MAP

Thomas Bros Guide San Diego County pg 1270

ALVARADO CREEK
7407 Alvarado Road
La Mesa, CA.

Figure No. I
Job No. 04-8598

Scale: 1" = 120'
(approximate)



LEGEND

● **CPT-9**	Approximate Location of Cone Penetrometer Test
⊕ **B-2**	Approximate Location of Exploratory Boring
⊢⊣ **T-3**	Approximate Location of Exploratory Trench

NOTE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

REFERENCE: This PLOT PLAN was prepared from an existing CONCEPTUAL GRADING PLAN by FUSCOE ENGINEERING dated 06/22/18 and from on-site field reconnaissance performed by GEI.

PLOT PLAN

ALVARADO CREEK
7407 Alvarado Road
La Mesa, CA.
Figure No. II
Job No. 04-8598

Geotechnical Exploration, Inc.

(February 2019)

04-8598-p2.ai

EQUIPMENT **Bucket/Auger Drill Rig**	DIMENSION & TYPE OF EXCAVATION **30-inch diameter Boring**	DATE LOGGED **7-24-18**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **3 feet**	LOGGED BY **JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			ASPHALT PAVEMENT OVER BASE , 3" thick.									
1 2			**SANDY CLAY** , with some cobbles. Firm. Moist. Dark gray-black. **FILL (Qaf)** -- perched water on contact.	CL								
3 4 5	▼		**SAND** , fine- to coarse-grained, with abundant cobbles (to 8" in diameter). Medium dense. Saturated. Gray-brown. **ALLUVIUM (Qal)**	GM								
6			**COBBLE CONGLOMERATE** , with fine- to medium-grained **SAND** matrix. Dense. Wet. Yellow-brown. **STADIUM CONGLOMERATE (Tst)**	GM								
7			Drilling Refusal @ 6.5'. Bottom @ 6.5'									

EXPLORATION LOG 8598 ALVARADO 2019.GPJ GEO_EXPL.GDT 2/13/19

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

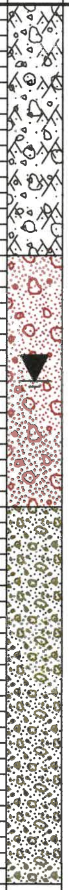
JOB NAME **Alvarado Creek Apartments**		
SITE LOCATION **7407 Alvarado Road, La Mesa, CA**		
JOB NUMBER **04-8598**	REVIEWED BY **LDR/JAC**	LOG No. **B-1**
FIGURE NUMBER **IIIa**	GEI Geotechnical Exploration, Inc.	

EQUIPMENT **Bucket/Auger Drill Rig**	DIMENSION & TYPE OF EXCAVATION **30-inch diameter Boring**	DATE LOGGED **7-24-18**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **3 feet**	LOGGED BY **JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
2			**SANDY CLAY** , with some roots and cobbles. Firm. Moist. Dark gray-brown. **FILL (Qaf)** -- perched water on contact.	CL								
4 6 8			**SAND** , fine- to coarse-grained, with abundant cobbles (to 8" in diameter). Medium dense. Saturated. Gray-brown. **ALLUVIUM (Qal)**	GM								
10			**COBBLE CONGLOMERATE** , with fine- to medium-grained **SAND** matrix. Dense. Wet. Yellow-brown. **STADIUM CONGLOMERATE (Tst)**	GM								
			Bottom @ 10'									

▼ PERCHED WATER TABLE ☒ BULK BAG SAMPLE 1 IN-PLACE SAMPLE ■ MODIFIED CALIFORNIA SAMPLE S NUCLEAR FIELD DENSITY TEST ▨ STANDARD PENETRATION TEST	JOB NAME **Alvarado Creek Apartments** SITE LOCATION **7407 Alvarado Road, La Mesa, CA**		
	JOB NUMBER **04-8598** FIGURE NUMBER **IIIb**	REVIEWED BY **LDR/JAC** **Geotechnical Exploration, Inc.**	LOG No. **B-2**

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**2' X 9' X 7' Trench**	**4-26-04**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 410' Mean Sea Level	**3 feet**	**JKH**



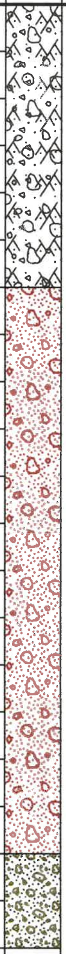
DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–2			**SILTY FINE TO COARSE SAND** , w/ some pebbles and cobbles. Medium dense. Damp. Light gray-brown. **FILL (Qaf)**	SM-GM								
2–4	▼ (3)		**FINE TO COARSE SAND** , w/ abundant pebbles and cobbles (to 6" in diameter). Medium dense. Moist to wet. Tan-gray and orange. **STREAM DEPOSITS**	GM								
4–7			**FINE TO COARSE SAND** , w/ abundant cobbles and boulders (to 12" - 14" in diameter). Medium dense. Wet (saturated). Tan-gray and brown. **STREAM DEPOSITS** East side of trench exposed: **COBBLE CONGLOMERATE**, w/ clayey sand and sandy clay matrix. Medium dense. Moist to wet. Tan-gray. **STADIUM CONGLOMERATE (Tst).**	GM								
7			Bottom @ 7'									

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME	**Alvarado Creek Apartments**
SITE LOCATION	**7407 Alvarado Road, La Mesa, California**
JOB NUMBER	**04-8598**
FIGURE NUMBER	**IIIc**
REVIEWED BY	**LDR/JAC**
LOG No.	**T-1**

Geotechnical Exploration, Inc.

EQUIPMENT **Rubber-tire Backhoe**	DIMENSION & TYPE OF EXCAVATION **2' X 10' X 10' Trench**	DATE LOGGED **4-26-04**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **JKH**



DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
2			**SILTY FINE TO COARSE SAND** , w/ some clay and abundant pebbles and cobbles (to 4" in diameter). Medium dense. Damp. Dark red-brown. **FILL (Qaf)** -- roots encountered.	SM-GM								
4 6 8			**FINE TO COARSE SAND** , w/ abundant pebbles and cobbles and boulders (to 12" - 14" in diameter). Medium dense. Dry to damp. Tan-gray and orange-brown. **STREAM DEPOSITS** -- roots encountered. -- roots encountered.	GM								
10			**COBBLE CONGLOMERATE** , w/ fine to medium sand and clay matrix. Dense. Damp. Tan-gray. **STADIUM CONGLOMERATE (Tst)**	GM								
			Bottom @ 10'									

▼	PERCHED WATER TABLE
☒	BULK BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	NUCLEAR FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

JOB NAME **Alvarado Creek Apartments**		
SITE LOCATION **7407 Alvarado Road, La Mesa, California**		
JOB NUMBER **04-8598**	REVIEWED BY **LDR/JAC**	LOG No. **T-2**
FIGURE NUMBER **IIId**	**Geotechnical Exploration, Inc.**	

EQUIPMENT **Rubber-tire Backhoe**	DIMENSION & TYPE OF EXCAVATION **2' X 10' X 4' Trench**	DATE LOGGED **4-26-04**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
1 2		☒	**SILTY FINE TO MEDIUM SAND** , w/ abundant roots and some cobbles and rock fragments (to 4" in diameter). Loose to medium dense. Damp to moist. Dark brown. **FILL (Qaf)**	SM			9.0	127.0				
3			**COBBLE CONGLOMERATE** , w/ fine to medium sand matrix. Dense. Damp. Tan-gray and orange. **STADIUM CONGLOMERATE (Tst)**	GM								
4 5 6 7 8 9			Bottom @ 4'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

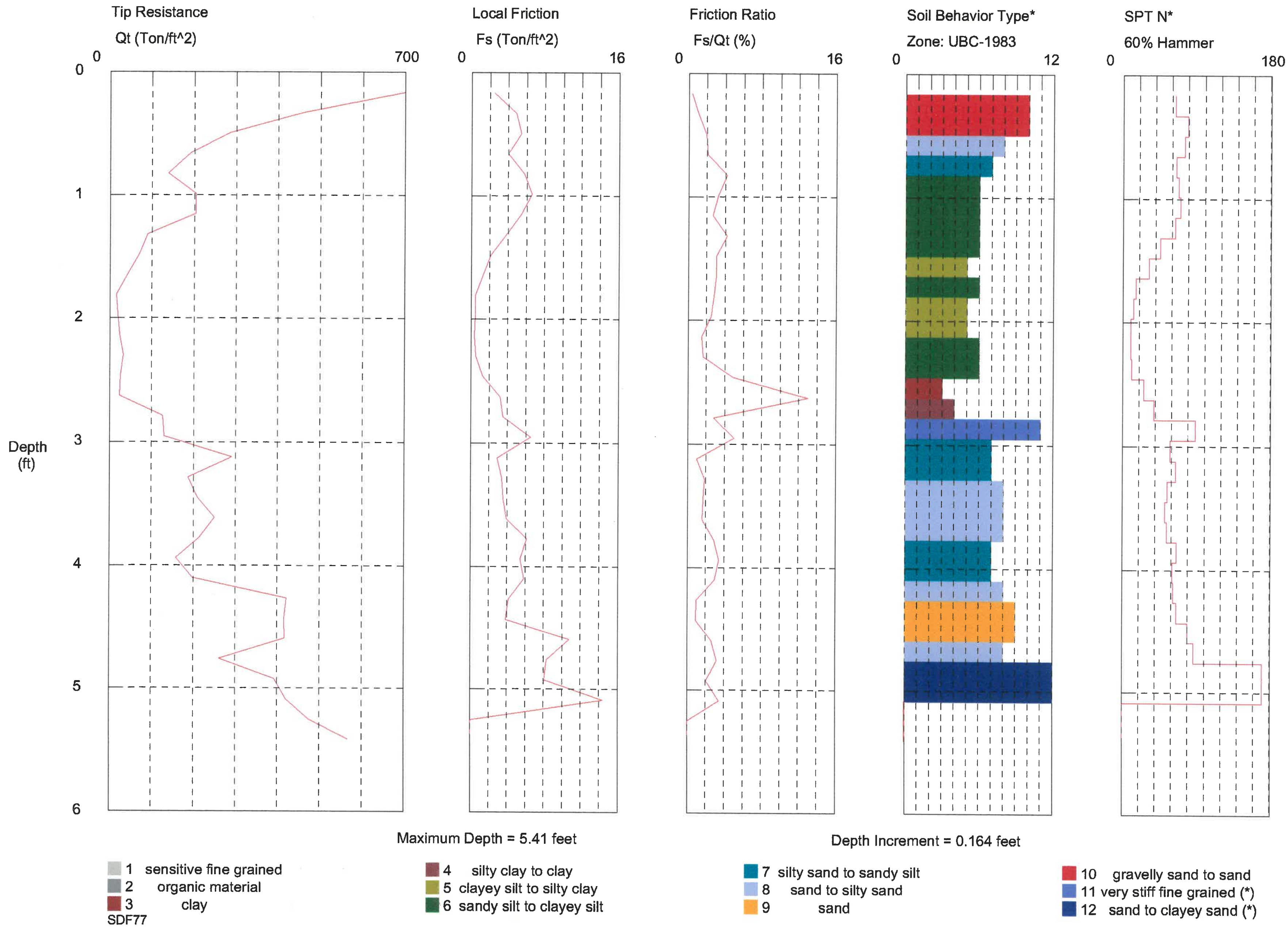
▨ STANDARD PENETRATION TEST

JOB NAME **Alvarado Creek Apartments**		
SITE LOCATION **7407 Alvarado Road, La Mesa, California**		
JOB NUMBER **04-8598**	REVIEWED BY **LDR/JAC**	LOG No. **T-3**
FIGURE NUMBER **IIIe**	**Geotechnical Exploration, Inc.**	

Geotechnical Explorations

Operator: Victor/Mike
Sounding: CPT-01
Cone Used: DSA0408

CPT Date/Time: 3/26/2004 9:15:
Location: San Diego RV Park
Job Number: 04-8598



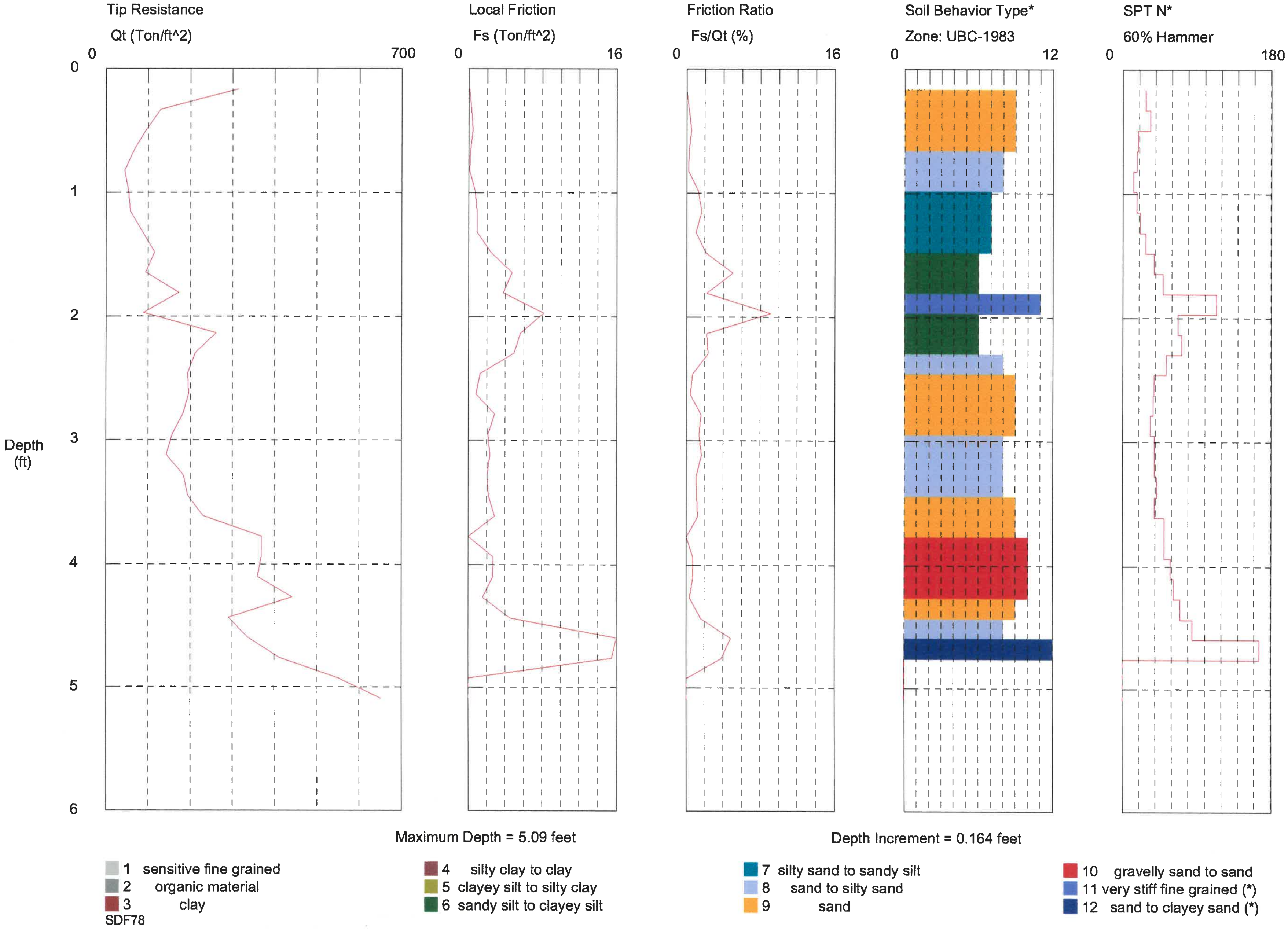
*Soil behavior type and SPT based on data from UBC-1983

Figure No. IVa

Geotechnical Explorations

Operator: Victor/Mike
Sounding: CPT-05
Cone Used: DSA0408

CPT Date/Time: 3/26/2004 11:57
Location: San Diego RV Park
Job Number: 04-8598

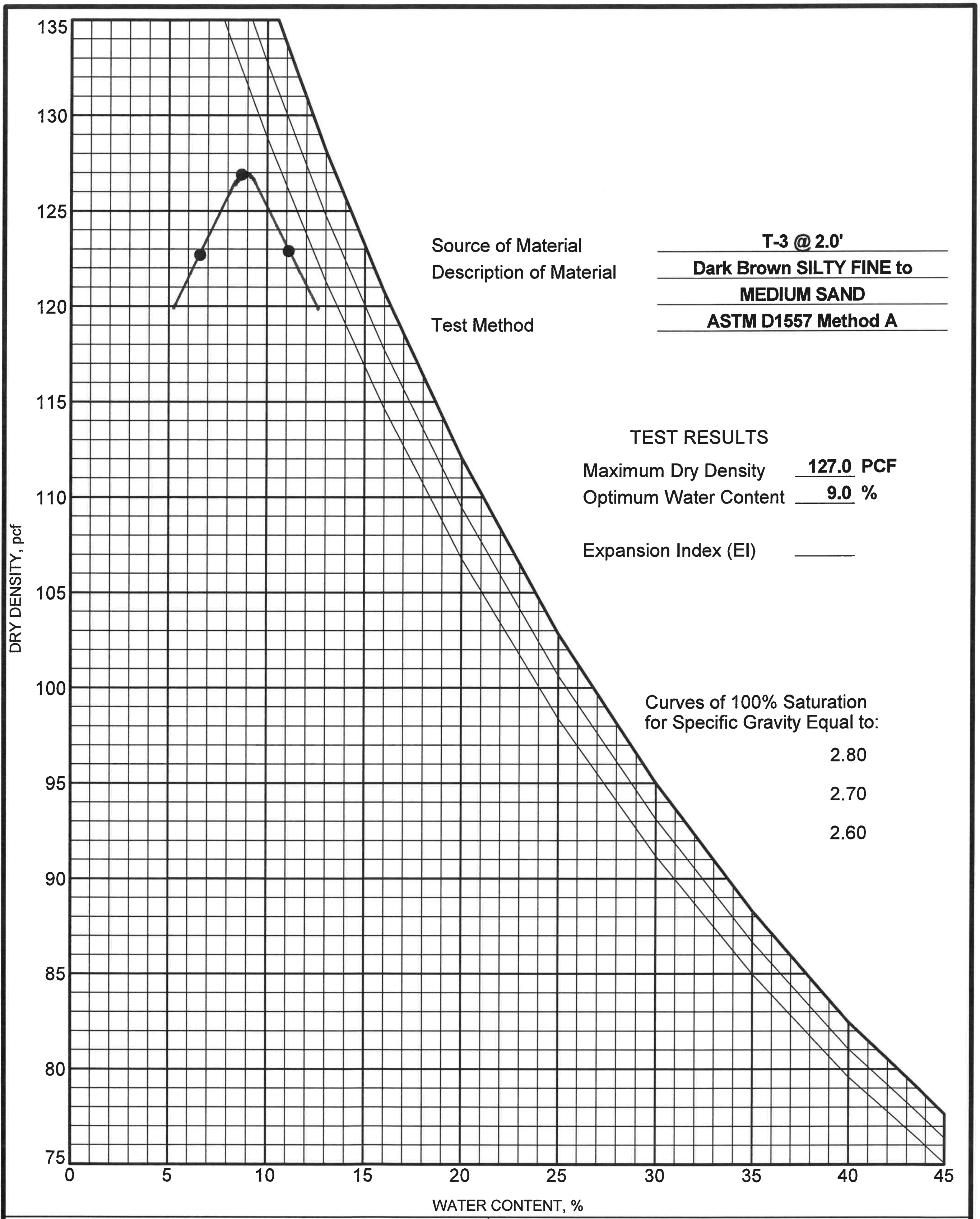


Maximum Depth = 5.09 feet

Depth Increment = 0.164 feet

*Soil behavior type and SPT based on data from UBC-1983

Figure No. IVb



Source of Material: T-3 @ 2.0'

Description of Material: Dark Brown SILTY FINE to MEDIUM SAND

Test Method: ASTM D1557 Method A

TEST RESULTS

Maximum Dry Density: 127.0 PCF

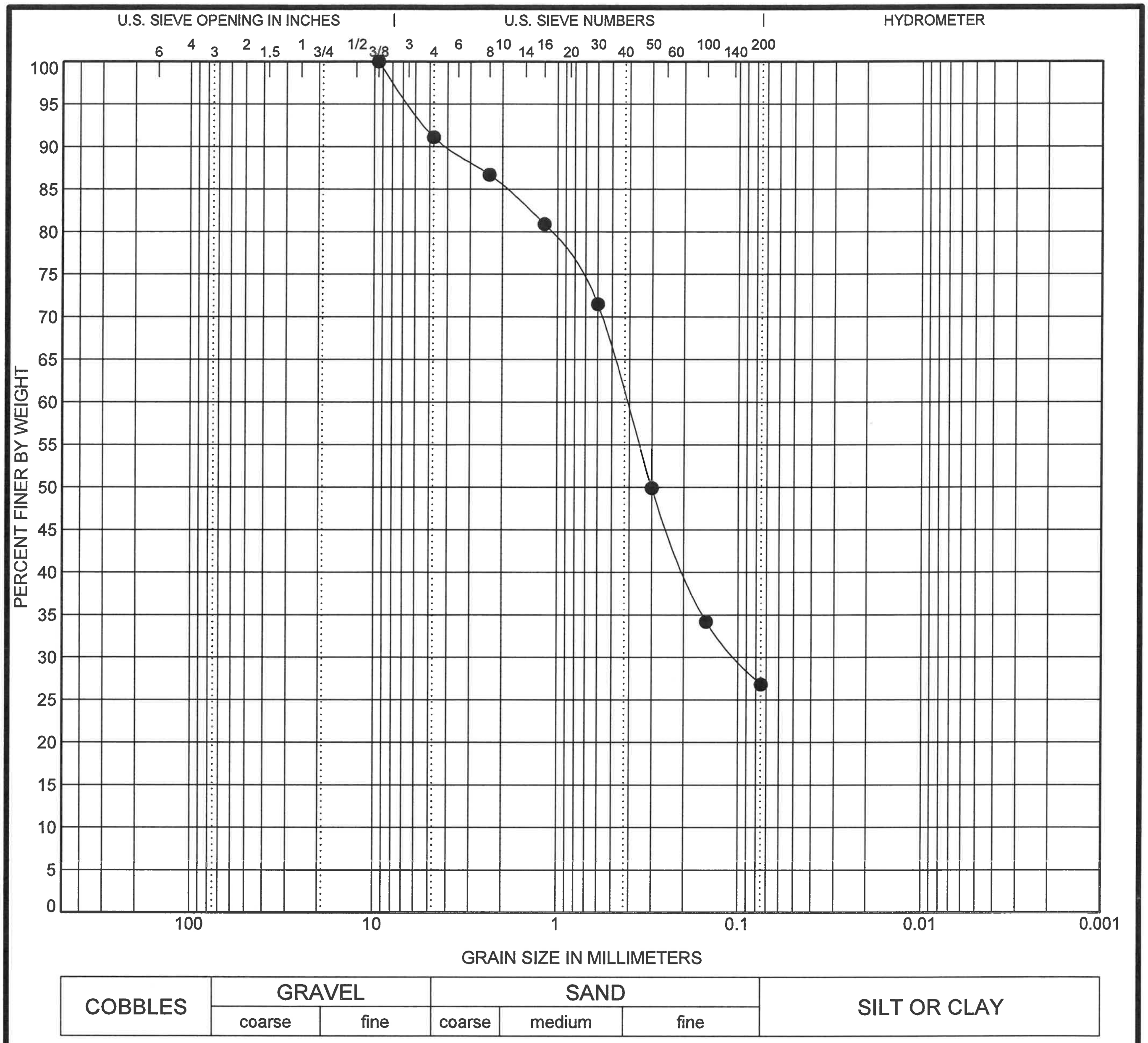
Optimum Water Content: 9.0 %

Expansion Index (EI): ______

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: Va
Job Name: Alvarado Creek Apartments
Site Location: 7407 Alvarado Road, La Mesa, California
Job Number: 04-8598



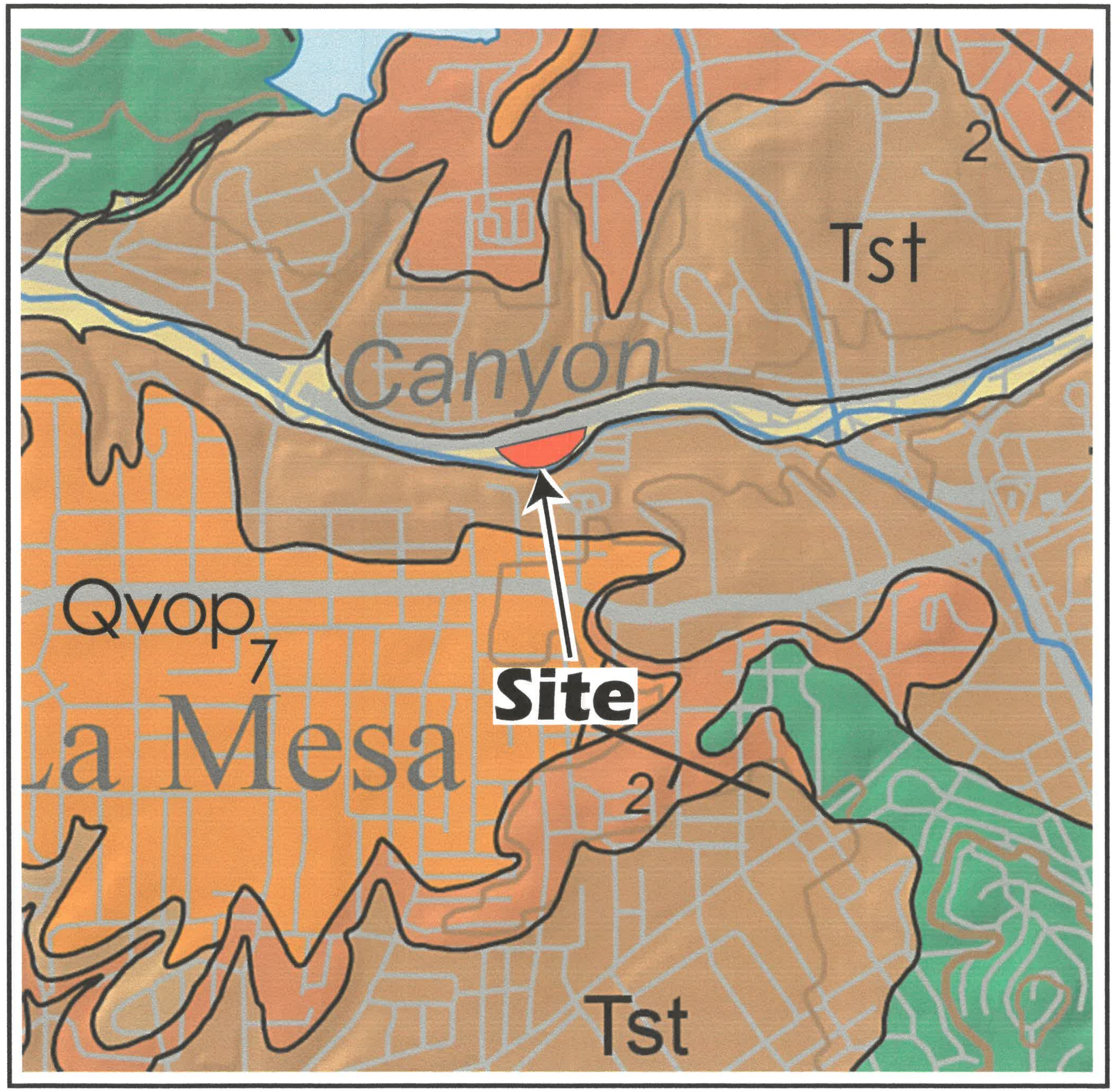
	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	**T-3 @ 2.0'**	**Dark Brown SILTY FINE to MEDIUM SAND**					

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**T-3 @ 2.0'**	**9.5**	**0.415**	**0.101**		**8.9**	**64.3**	**26.8**	

Geotechnical Exploration, Inc.

GRAIN SIZE DISTRIBUTION

Figure Number: Vb
Job Name: Alvarado Creek Apartments
Site Location: 7407 Alvarado Road, La Mesa, California
Job Number: 04-8598



ALVARADO CREEK
7407 Alvarado Road
La Mesa, CA.

EXCERPT FROM GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

1. Department of Conservation, California Geological Survey
2. U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

ONSHORE MAP SYMBOLS

Contact - Contact between geologic units; dotted where concealed.

70 U D Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Anticline - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow indicates direction of axial plunge.

Syncline - Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge.

Qls? Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

Strike and dip of beds

70 Inclined

Strike and dip of igneous joints

60 Inclined

Vertical

Strike and dip of metamorphic foliation

55 Inclined

DESCRIPTION OF UNITS

Tst	Stadium Conglomerate (middle Eocene)

Base Map

Onshore base (hypsography, hydrography, and transportation) from U.S.G.S. digital line graph (DLG) data, San Diego 30' x 60' metric quadrangle. Shaded topographic base from U.S.G.S. digital elevation models (DEM's). Offshore bathymetric contours and shaded bathymetry from N.O.A.A. single and multibeam data. Projection is UTM. zone 11, North American Datum 1927.

This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 98HQAG2049.

Prepared in cooperation with the U.S. Geological Survey, Southern California Areal Mapping Project.

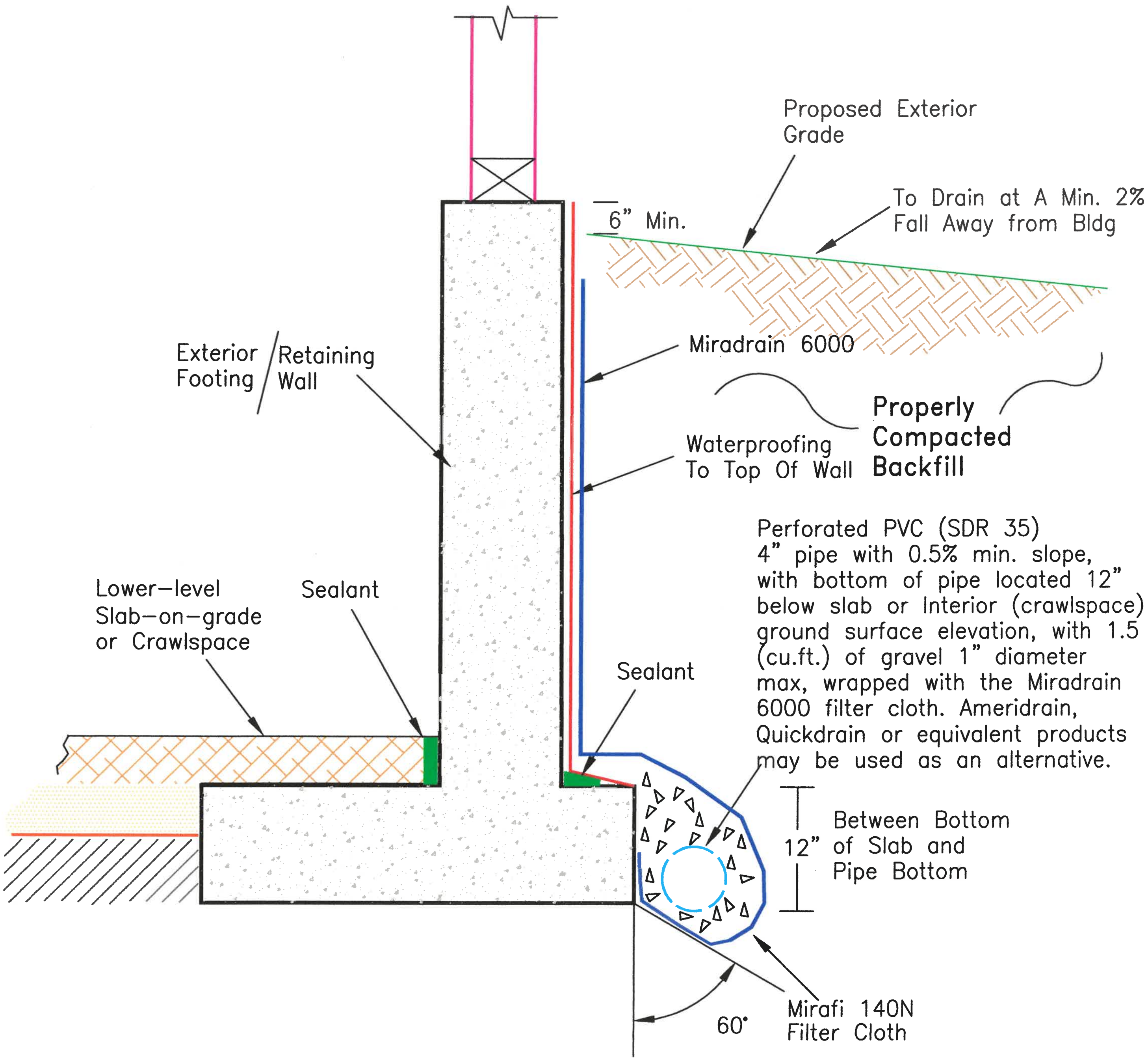
Alvarado-combo-2008-geo.ai

Figure No. VI
Job No. 04-8598

Geotechnical Exploration, Inc.

February 2019

TYPICAL SUBGRADE RETAINING WALL DRAINAGE RECOMMENDATIONS



Perforated PVC (SDR 35) 4" pipe with 0.5% min. slope, with bottom of pipe located 12" below slab or Interior (crawlspace) ground surface elevation, with 1.5 (cu.ft.) of gravel 1" diameter max, wrapped with the Miradrain 6000 filter cloth. Ameridrain, Quickdrain or equivalent products may be used as an alternative.

NOT TO SCALE

Figure No. VII

Job No. 04-8598

Geotechnical Exploration, Inc.

NOTE: As an option to Miradrain 6000, Gravel or Crushed rock 3/4" maximum diameter may be used with a minimum 12" thickness along the interior face of the wall and 2.0 cu.ft./ft. of pipe gravel envelope.

APPENDIX A
UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION

Coarse-grained (More than half of material is larger than a No. 200 sieve)

GRAVELS, CLEAN GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3″)	GW	Well-graded gravels, gravel and sand mixtures, little or no fines.
	GP	Poorly graded gravels, gravel and sand mixtures, little or no fines.
GRAVELS WITH FINES (Appreciable amount)	GC	Clay gravels, poorly graded gravel-sand-silt mixtures
SANDS, CLEAN SANDS (More than half of coarse fraction is smaller than a No. 4 sieve)	SW	Well-graded sand, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines.
SANDS WITH FINES (Appreciable amount)	SM	Silty sands, poorly graded sand and silty mixtures.
	SC	Clayey sands, poorly graded sand and clay mixtures.

FINE-GRAINED (More than half of material is smaller than a No. 200 sieve)

SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour, sandy silt and clayey-silt sand mixtures with a slight plasticity.
Liquid Limit Less than 50	CL	Inorganic clays of low to medium plasticity, gravelly clays, silty clays, clean clays.
	OL	Organic silts and organic silty clays of low plasticity.
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
Liquid Limit Greater than 50	CH	Inorganic clays of high plasticity, fat clays.
	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

APPENDIX B

SEISMIC DESIGN MAP

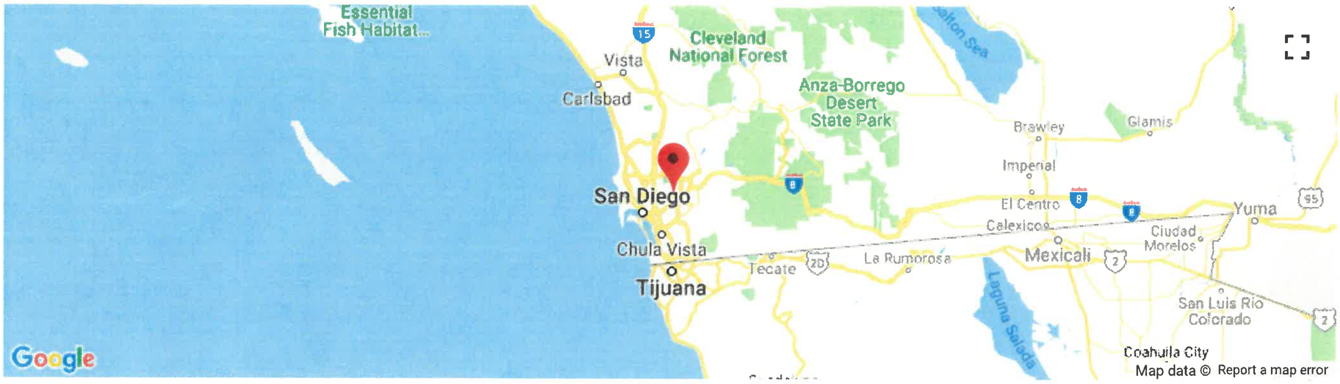
⚠ This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

ATC Hazards by Location

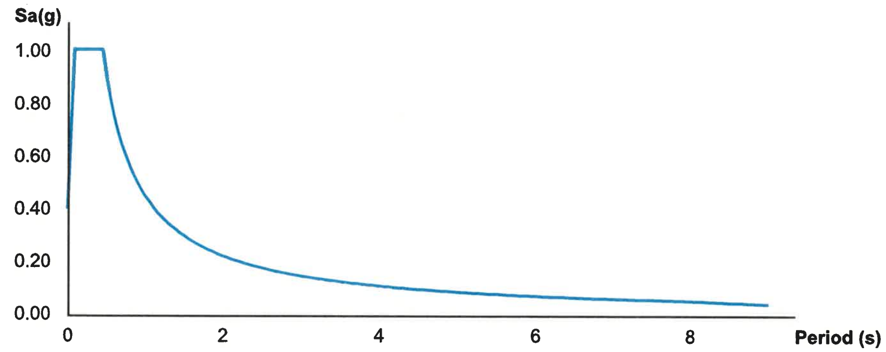
Search Information

Coordinates:	32.7725, -117.0378
Timestamp:	2019-02-14T15:39:38.112Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	C
Report Title:	7407 Alvarado Road, La Mesa, CA

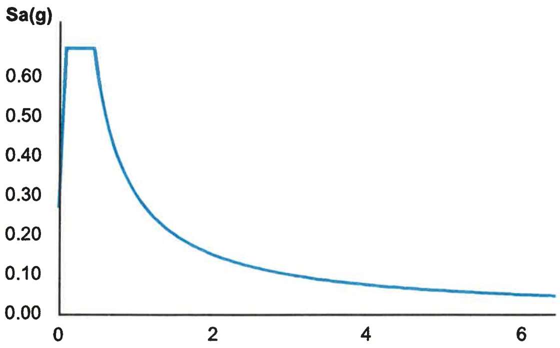
Map Results



MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Text Results

Basic Parameters

Name	Value	Description
S_S	0.844	MCE_R ground motion (period=0.2s)
S_1	0.303	MCE_R ground motion (period=1.0s)
S_{MS}	1.013	Site-modified spectral acceleration value
S_{M1}	0.454	Site-modified spectral acceleration value
S_{DS}	0.675	Numeric seismic design value at 0.2s SA
S_{D1}	0.303	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category

F_a	1.2	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s
PGA	0.366	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.44	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	0.844	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.935	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.303	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.331	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the United States Geological Survey Seismic Design Web Services.

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Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

08 March 2018

RV COMMUNITIES, LLC
7855 Herschel Avenue, Suite 201
La Jolla, CA 92037
Attn: Michael Brekka

Job No. 04-8598

Subject: **Interim Report of Site Conditions and Preliminary Opinions**
Alvarado Creek Apartments
7407 Alvarado Road
La Mesa, California

Dear Mr. Brekka:

In accordance with your request and our proposal for a Geotechnical Investigation Update, dated January 22, 2018, ***Geotechnical Exploration, Inc.*** is in the process of investigating the subsurface soil and geologic conditions at the subject property. Our work to date has included our previous investigation in 2004 consisting of three exploratory trenches and nine cone penetrometer tests (CPT) on the property to a maximum depth of 10 feet to evaluate the depth of the existing fill and stream deposits underlying the property. Our field work was conducted between March 26 and April 26, 2004. The update investigation will include seven exploratory borings to determine the current groundwater depth and depth to the underlying formational soils across the site.

Based on our previous exploratory trenching at the site, we found the site to be underlain at depth by dense formational materials of the Tertiary-age Stadium Conglomerate Formation. The existing surface fill soils have a thickness of approximately 2 to 3 feet at our trench locations along the southern perimeter of the property. The fill soils are underlain by natural ground materials (Stream Deposits consisting of fine to coarse sands with abundant cobbles from 12 to 14 inches in diameter) extending to a depth of approximately 3 to 9 feet. We note that the water table was encountered at a depth of 3 to 4 feet below existing grade during our previous field investigation.

The encountered natural ground materials below a depth of 3 to 9 feet have good load-bearing properties. As such, we will be recommending that the proposed apartment structures with partially subterranean parking be supported on deepened conventional foundations and slabs on grade founded on dense formational materials or properly compacted fill soils. Dewatering will most likely be necessary during excavation for the lower level parking areas and after construction if the lower level is going to be below the groundwater surface.

Based on the results of our previous field investigation, it is our opinion that the property can be developed with the proposed apartment structures with lower level parking utilizing conventional grading equipment. Specific foundation design soil parameters and site preparation recommendations will be provided in our updated report.

This opportunity to be of service is sincerely appreciated. Should you have any questions regarding this matter, please feel free to contact our office. Reference to our **Job No. 04-8598** will help to expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jay K. Heiser
Senior Project Geologist

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JAIME A. CERROS
No. 002007
Exp 9/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Paydar Apartment Project
7407 Alvarado Road
La Mesa, California

JOB NO. 04-8598
30 June 2004

Prepared for:

Mr. Reza Paydar
PAYDAR PROPERTIES, INC.

GEOTECHNICAL EXPLORATION, INC.
SOIL & FOUNDATION ENGINEERING • GROUNDWATER
HAZARDOUS MATERIALS MANAGEMENT • ENGINEERING GEOLOGY

30 June 2004

Job No. 04-8598

Mr. Reza Paydar
PAYDAR PROPERTIES, INC.
7855 Herschel Avenue, Suite 201
La Jolla, CA 92037-4429

Subject: **Report of Preliminary Geotechnical Investigation**
Paydar Apartment Project
7407 Alvarado Avenue
La Mesa, California

Dear Mr. Paydar:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has performed an investigation of the soil and geologic conditions at the location of the subject site. Exploratory cone penetrometer testing was performed at the site on March 26, 2004 and exploratory trenches were placed on the site on April 26, 2004.

In our opinion, if the conclusions and recommendations presented in this report are implemented during site preparation, the site will be suited for the proposed residential structures, subterranean parking and associated improvements from a geotechnical perspective.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 04-8598** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Leslie D. Reed, President
C.E.G. 999 exp. 3-31-05 / R.G. 3391

Jaime A. Cerros
No. 002007
Exp. 9/30/05

Leslie D. Reed
No. 999
Certified Engineering Geologist

7420 TRADE STREET • SAN DIEGO, CA 92121 • (858) 549-7222 • FAX: (858) 549-1604 • E-MAIL: geotech@ixpres.com

TABLE OF CONTENTS

REFERENCES

FIGURES

APPENDICES

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Paydar Apartment Project
7407 Alvarado Road
La Mesa, California

JOB NO. 04-8598

The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project (for site location, refer to Figure No. I).

I. EXECUTIVE SUMMARY

It is our understanding, based on communications with Mr. Kim Campbell, that the existing RV Park is to be removed and the site is to be developed to receive a new apartment project consisting of eight 4-story buildings with two levels of subterranean parking and associated improvements. Preliminary plans were not available for our review.

Our investigation revealed that the site is underlain by a thin veneer of silty sandy fill soil that is, in turn, underlain by sandy stream deposits along the southern perimeter of the property. Cobble conglomerate formational materials of the Stadium Conglomerate underlie the entire site at relatively shallow depths. We anticipate the structures will utilize either CMU or poured-in-place concrete for the basement walls, and standard building materials for the above-grade portions of the structures. The foundations will utilize continuous perimeter and isolated spread footings with a slab-on-grade foundation system.

Appropriate sub drains and waterproofing will be required for proposed basement/subgrade areas. We note that the water table was encountered at a depth of 3 to 4 feet below existing grade along the southern perimeter of the property during our field investigation. Dewatering will most likely be necessary during excavation for the basement levels.

With the above in mind, the Scope of Work is briefly outlined as follows:

1. Identify and classify the surface and subsurface soils in the area of the proposed construction, in conformance with the Unified Soil Classification System (see Figure Nos. III and IV and Appendix A).

2. Make note of any faults or significant geologic features that may affect the site (see Figure No. VI, and Appendices B, C and D).

3. Evaluate the existing and proposed bearing soil material.

4. Recommend the allowable bearing capacities for the existing on-site medium dense to dense natural soils and existing fill (if applicable).

5. Recommend site preparation procedures.

6. Evaluate the settlement potential of the bearing soils under the proposed structural loads.

7. Recommend preliminary foundation design information and provide active and passive earth pressures to be utilized in design of retaining walls or foundation structures.

II. SITE DESCRIPTION

The subject site is identified as Assessor's Parcel No. 469-021-01 and is comprised of 11.87 acres according to Map 346, in the City of La Mesa, County of San Diego, State of California. Presently, the site consists of a relatively level developed pad with an approximate elevation of 410 feet above mean sea level (MSL). No actual site survey elevations were available to our firm during our investigation.

The site is bordered to the north by Alvarado Road and Interstate 8; to the south by Alvarado Creek and the right-of-way for the San Diego Trolley system; to the east by existing commercial properties; and to the west by existing commercial properties. Concrete pads for recreational vehicle parking and adjacent asphalt-covered parking and driveway areas currently exist on the site. Overhead electric lines with power poles extending in an east-west direction are located through the central portion of the site.

III. FIELD INVESTIGATION

Three exploratory trenches were placed on the site, specifically in accessible areas where the new structures and improvements are to be located (and where feasible due to existing structures and utilities on the site). Refer to Figure No. II for the exploratory excavation locations. The soil in the trenches was logged by our field representative, and bulk and in situ soil samples were taken of the predominant soils throughout the field operation. Excavation logs have been prepared on the basis of our observations and the results have been summarized on Figure No. III. The predominant soils have been classified in conformance with the Unified Soil Classification System (refer to Appendix A).

Nine cone penetrometer tests (CPT) were randomly placed on the site. The CPT soundings were located in the field by referring to a site plan of the existing San Diego RV Park. CPT logs have been provided by Holguin, Fahan & Associates, Inc., and are included in Figure No. IV of this report. The predominant soils have been classified in conformance with the Unified Soil Classification System (refer to Appendix A) based on correlations obtained by Robertson and Campanella between soil types and tip and friction resistance measured by the cone penetrometer. Cone penetrometer measurements were extended up to a maximum depth of 8 feet at the location of CPT-5 and CPT-7.

Soil behavior interpretations are based on research performed by Robertson, P.K. and Campanella, R.C., 1989 *"Guidelines for Geotechnical Design Using the Cone Penetrometer Test and CPT with Pore Pressure Measurement,"* Soil Mechanics Series No. 120, Civil Engineering Department, University of British Columbia, Vancouver, B.C., V6T1Z4, Sept. 1989.

The cone penetration tests were performed to aid in evaluating soil types, basic strength parameters, and the liquefaction potential of the existing subsurface soils, and to aid in developing appropriate site preparation and foundation design recommendations.

IV. GENERAL GEOLOGIC DESCRIPTION

The San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 92 miles from Chula Vista) and westward to the San Clemente Fault (approximately 80 miles off-shore from Chula Vista) (Berger and Schug, 1991).

During recent history, the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes have been experienced in historic time within the Chula Vista area. Since earthquakes have been recorded by instruments (since the 1930s), the Chula Vista area has experienced scattered seismic events with Richter magnitudes generally less than 4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay; three of these earthquakes had recorded magnitudes of 4.0 to 4.2. In addition, the Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, had a magnitude of 5.3 (Hauksson, 1988).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (Hart, E.W., 1980), an "active" fault is one that has had ground surface displacement within Holocene time (about the last 11,000 years). Additionally, faults along which major historical earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Division of Mines and Geology defines a "potentially active" fault as one that has had ground surface displacement during Quaternary time, that is, during the past 11,000 to 1.6 million years (Hart, E.W., 1980).

V. SITE-SPECIFIC GEOLOGIC DESCRIPTION

A geologic investigation of the site was conducted to evaluate the on-site geology and potential of geologic hazards that might affect the site. Our investigation drew upon information gathered from published and unpublished geologic maps and reports, as well as results of our recent exploratory trenches and electronic cone penetration soundings.

A. *Stratigraphy*

Our field investigation and review of pertinent geologic maps and reports indicate that some artificial fill soils, stream deposits and dense formational materials underlie the site. Construction of the subterranean parking below the structures would result in the removal of all of the encountered fill soils and stream deposits beneath the proposed structures.

Artificial Fill (Qaf): A limited amount of fill (up to approximately 2 to 3 feet) was encountered on the surface of the site. The encountered fill is loose to medium dense and consists of damp, red-brown to gray-brown, silty, fine to medium and fine to coarse sand with pebbles and cobbles. The shallow fill soils are considered to have a low expansion potential. These fill soils are not suitable in their current condition for bearing support. To be utilized as fill soils they require excavation and recompaction. Refer to Figure Nos. III and IV for details.

Stream Deposits: The fill soils along the southern perimeter of the property are underlain by stream deposits to an approximate depth of 9 feet below the present surface grade. As encountered on this site, the stream deposits consist of a medium dense, wet, tan-gray and orange-brown, fine to coarse sand with abundant cobbles and boulders (to 14 inches in diameter). These materials are considered to be of low expansion potential. These stream deposit soils are not suitable in their current condition for bearing support. To be utilized as bearing soils they require excavation and recompaction. Refer to Figure Nos. III and IV for details.

Stadium Conglomerate Formation (Tst): The site is underlain at depth by dense cobble conglomerate formational material of the Tertiary Stadium Conglomerate Formation. These formational soils are considered to have a negligible to very low liquefaction potential and low consolidation and expansion potential characteristics. Refer to Figure Nos. III and IV.

B. ***Structure***

Slopes and roadcuts nearby allowed observation of bedding and geologic structural features of the Stadium Conglomerate Formation in the vicinity of the subject lot. The observed Stadium Conglomerate formational material appears to be massively bedded with no indication of measured strike and dip. Previous investigations performed in the vicinity by our firm, as well as review of a geologic map of the area (Kennedy, 1975), indicate that the area, in general, is commonly underlain by generally flat bedding.

VI. GEOLOGIC HAZARDS

The following is a discussion of the geologic conditions and hazards common to the La Mesa area, as well as project-specific geologic information relating to development of the subject property.

A. ***Local and Regional Faults***

It is our opinion that a known "active" fault presents the greatest seismic risk to the subject site during the lifetime of the proposed structure. To date, the nearest known "active" faults to the subject site are the northwest-trending Rose Canyon Fault, Coronado Bank Fault and the Elsinore Fault.

Rose Canyon Fault: The Rose Canyon Fault Zone (Mount Soledad and Rose Canyon Faults), located approximately 7.4 miles west of the subject site, is mapped trending north-south from Oceanside to downtown San Diego, from where it appears to head southward into San Diego Bay, through Coronado and offshore. The Rose Canyon Fault Zone is considered to be a complex zone of onshore and offshore, en echelon strike slip, oblique reverse, and oblique normal faults. The Rose Canyon Fault is considered to be capable of causing a 7.5-magnitude earthquake and considered microseismically active, although no significant recent earthquake is known to have occurred on the fault. Investigative work on faults (believed to be part of the Rose Canyon Fault Zone) at the Police Administration and Technical Center in downtown San Diego and at the SDG&E facility in Rose Canyon, has encountered offsets in Holocene (geologically recent) sediments. These findings have been accepted as confirmed Holocene displacement on the Rose Canyon Fault and this previously classified "potentially active" fault has now been upgraded to an "active" fault as of November 1991 (California Division of Mines and Geology -- Fault Rupture Hazard Zones in California, 1994).

Coronado Bank Fault: The Coronado Bank Fault is located approximately 21.3 miles southwest of the site. Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). The Oceanside earthquake of 5.3 magnitude, recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a 7.0-magnitude earthquake and is of great interest due to its close proximity to the greater Oceanside metropolitan area.

Elsinore Fault: The Elsinore Fault is located approximately 41.4 miles northeast of the site. The fault extends approximately 200 km (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, Oceanside, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

Although the Elsinore Fault Zone belongs to the San Andreas set of active, northwest-trending, right-slip faults in the southern California area (Crowell, 1962), it has not been the site of a major earthquake in historic time, other than a 6.0-magnitude quake near the town of Elsinore in 1910 (Richter, 1958; Toppozada and Parke, 1982). However, based on length and evidence of late-Pleistocene or Holocene displacement, Greensfelder (1974) has estimated that the Elsinore Fault Zone is reasonably capable of generating an earthquake with a magnitude as large as 7.5. Faulting evidence exposed in trenches placed in Glen Ivy Marsh across the Glen Ivy North Fault (a strand of the Elsinore Fault Zone between Corona and Lake Elsinore), suggest a maximum earthquake recurrence interval of 300 years, and when combined with previous estimates of the long-term horizontal slip rate of 0.8 to 7.0 mm/year, suggest typical earthquake magnitudes of 6 to 7 (Rockwell, 1985).

B. ***Other Geologic Hazards***

Ground Rupture: Ground rupture is characterized by bedrock slippage along an established fault and may result in displacement of the ground surface. For ground rupture to occur along a fault, an earthquake usually exceeds magnitude 5.0. If a 5.0-magnitude earthquake were to take place on a local fault, an estimated surface-rupture length 1 mile long could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known fault trace and, therefore, the risk of ground rupture is remote.

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of magnitude 5.0 Richter scale or greater are generally associated with significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structures. The anticipated ground accelerations at the site from earthquakes on faults within 100 miles of the site are provided in Tables 1 and 2 of Appendix B. For structural design purposes, the calculated site acceleration corresponding to a 10 percent probability of exceedance in 50 years is 0.20g.

Liquefaction: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a dense fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are shaken by an earthquake of sufficient magnitude.

On this site, the risk of liquefaction of foundation material due to seismic shaking is considered to be minimal due to the dense nature of the natural-ground material. No loss of strength is anticipated to occur to the on-site soils due to an anticipated seismic event.

Flooding: A review of San Diego County flood hazard maps indicates that the site is located within a relatively low risk area. Due to the property's location adjacent to Alvarado Creek, it should be anticipated that minor flooding may occur on the site, if left at the existing grade, should a 100-year flood or greater occur in the San Diego area.

VII. EARTHQUAKE RISK EVALUATION

Evaluation of earthquake risk requires that the effect of faulting on, and the mass stability of, a site be evaluated utilizing the M_{10} seismic design event, i.e., an earthquake event on an active fault with less than a 10 percent probability of being exceeded in 50 years. Further, sites are classified by UBC 1997 Edition into "soil profile types S_A through S_F." Soil profile types are defined by their shear velocities where shear velocity is the speed at which shear waves move through the upper 30 meters (approximately 100 feet) of the ground. These are:

$S_A \Rightarrow$ Greater than 1500 m/s
$S_B \Rightarrow$ 760 to 1500 m/s
$S_C \Rightarrow$ 360 m/s to 760 m/s
$S_D \Rightarrow$ 180 to 360 m/s
$S_E \Rightarrow$ Less than 180 m/s
$S_F \Rightarrow$ Soil requiring specific soil evaluation

By utilizing an earthquake magnitude M_{10} for a seismic event on an active fault, knowing the site class and ground type, a prediction of anticipated site ground acceleration, g, from these events can be estimated. The subject site has been assigned Classification "S_C."

An estimation of the peak ground acceleration and the repeatable high ground acceleration (RHGA) likely to occur at the project site based on the known significant local and regional faults within 100 miles of the site is also included in Appendix B. In addition, a listing of the known historic seismic events that have occurred within 100 miles of the site at a magnitude of 5.0 or greater since the year 1800, and the probability of exceeding the experienced ground accelerations in the future based upon the historical record, is provided in Appendix C. Both Appendix B and Appendix C are tables generated from computer programs EQFault and EQSearch by Thomas F. Blake (2001) utilizing a digitized file of late-Quaternary California faults (EQFault) and a file listing of recorded earthquakes (EQSearch). Estimations of site intensity are also provided in these listings as Modified Mercalli Index values. The Modified Mercalli Intensity Index is provided as Appendix D.

It is our opinion that a known "active" fault presents the greatest seismic risk to the subject site during the lifetime of the proposed residence. To date, the nearest known "active" faults to the subject site are the northwest-trending Rose Canyon Fault, Coronado Bank Fault and the Elsinore Fault.

The owner should understand that there is some risk associated with any construction in the San Diego area due to the proximity of the Rose Canyon Fault, which is considered "active". The maximum probable *repeatable horizontal ground acceleration* (RHGA) anticipated is 0.1707g. The maximum probable *peak horizontal ground acceleration* anticipated is 0.2749g. The structural design shall be based on a site acceleration of 0.20g, which has a 10 percent probability of exceedance in 50 years.

Summary: It is our opinion, based upon a review of the available maps and our site investigation, that the site is underlain by relatively stable formational materials, and appears suited for the proposed residential construction. No significant geologic hazards are known to exist on the site that would prevent the proposed apartment structures and associated improvements.

VIII. GROUNDWATER

Free groundwater was encountered at a depth of 3 feet at the location of exploratory trench T-1 located along the south perimeter of the property adjacent to the Alvarado Creek. If moisture-related effects such as water seeps, efflorescence or high vapor emissions in the proposed subterranean garages are not acceptable, measures should be taken to reduce these issues. Remedial measures may include designing the below-grade parking structures to be water "tight", anti-vapor membranes below the slab on-grade and/or utilizing a concrete mix having a maximum water-to-cement ratio of 0.45. Dewatering will most likely be necessary during excavation and grading for the subterranean garage levels of the structures.

Design considerations should be made to alleviate this situation. Subsurface drainage with a properly designed and constructed subdrain system will be required along with continuous back drainage behind the below-grade garage walls and any proposed elevator pits. Furthermore, the subterranean garages should be provided with the proper cross-ventilation to help reduce the potential for moisture-related problems as previously stated. The garage slabs shall also be properly protected by proper sealing and waterproofing to help reduce potential for moisture intrusion. At the time of grading, subgrade observations may require the placement of subdrains under the garage slabs if groundwater is observed.

It should be kept in mind that any required grading operations may change surface drainage patterns and/or reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

It must be understood that unless discovered during initial site exploration or encountered during site grading operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Whereas water conditions encountered during grading operations should be evaluated and remedied by the project civil and geotechnical consultants, the project developer and future property owners must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis.

IX. LABORATORY TESTS AND SOIL INFORMATION

Laboratory tests were performed on the disturbed and relatively undisturbed soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed apartment structures. The following tests were conducted on the sampled soils:

1. Moisture Content (ASTM D2216-98)
2. Moisture/Density Relations (ASTM D1557-98, Method A)
3. Mechanical Analysis (ASTM D422-98)

The moisture content of a soil sample is a measure of the weight of water, expressed as a percentage of the dry weight of the sample.

The relationship between the moisture and density of remolded soil samples gives qualitative information regarding the soil strength characteristics and soil conditions to be anticipated during any future grading operation.

The expansion potential of soils is determined, when necessary, utilizing the Uniform Building Code Test Method for Expansive Soils (UBC Standard No. 29-2). In accordance with the UBC (Table 18-1-B), expansive soils are classified as follows:

EXPANSION INDEX	***POTENTIAL EXPANSION***
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very high

Based on our grain-size test results, our visual classification, and our experience with similar soils, the on-site silty fine to medium sand is considered to have a very low to low expansion potential (EI less than 50).

The Mechanical Analysis Test was used to aid in the classification of the soils according to the Unified Soil Classification System.

Based on laboratory test data, our experience with the formational materials in this area of La Mesa, our observations of the primary soil types on the project, and our previous experience with laboratory testing of similar formational soils, our Geotechnical Engineer has utilized conservative values for friction angle and cohesion for those formational soils and properly compacted fill soils that will have significant lateral support or bearing functions on the project. The assigned values have been utilized in determining the recommended allowable bearing capacity, as well as the active and passive earth pressures for wall and footing designs.

X. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the practical field investigation conducted by our firm, and resulting laboratory tests, in conjunction with our knowledge and experience with the soils in the La Mesa area of the County of San Diego.

Our investigation revealed that the building site is underlain by a thin veneer of fill soils that are, in turn, underlain by cobble conglomerate formational materials. Stream deposits underlie the fill soils along the southern perimeter of the site. The prevailing formational materials encountered on the site are dense and are of very low expansion and consolidation potential. These variable density fill soils and stream deposits will not provide a stable soil base for the proposed structures and associated improvements. As such, we recommend that these soils be removed and recompacted as part of site preparation prior to the addition of any new fill or structural improvements. It appears that excavation for the subterranean garage level will result in the removal of all of the fill soils and stream deposits in those areas. It is our opinion that the formational materials will provide adequate support for the proposed structures and improvements.

Excavation for the subterranean parking garages will result in the removal of the upper 10 to 12 feet of soil at the structure locations. The encountered sandy soil and cobble materials on the site are considered to be acceptable for export from the site. A shoring system may need to be installed in the areas of the planned garage basement excavations. In some areas of the site loose cohesionless soils may be encountered. Dewatering will most likely be necessary during excavation for the subterranean garage levels.

A. ***Site Preparation***

1. The existing improvements and vegetation observed on the site must be removed prior to the preparation of the site to receive new structural improvements.

2. In order to provide a uniform, firm soils base for the proposed foundations and subterranean garage slabs on-grade, soil materials (as evaluated by the project geotechnical engineer) shall be excavated to expose dense formational materials. The excavation depth on the site is anticipated to be approximately 10 to 12 feet and all excavated materials are to be exported from the site. We recommend that shoring protection be installed as the basement grade is cut down to approximate finish subgrade elevation. Any areas with loose soils exposed should be removed and recompacted to firm soils. Wet soils were encountered at an approximate depth of 3 feet along the southern perimeter of the property and should be anticipated during grading in this area.

   Due to the presence of groundwater along the south perimeter of the property, the grading contractor and general contractor shall consider the need for sump pumps to pump out water anticipated to collect in the basement excavations. If the water flow is substantial, a permanent sump pump may need to be incorporated as part of the construction. It is possible that a cut-off wall or subdrain along the south side of the property would aid in reducing the flow of groundwater into excavations.

Excavation for the subterranean garages should result in the removal of all of the fill soils and stream deposits at the garage locations. However, for ease of grading, any recompaction work of the surficial soils shall be done before the basement excavations. The excavated loose soils shall be cleaned of any debris and deleterious materials, watered to the approximate optimum moisture content, placed where needed to reach planned grades, and compacted to at least 90 percent of Maximum Dry Density, in accordance with ASTM D1557-98 standards.

3. No uncontrolled fill soils shall remain on the site after completion of any future site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils or left-in-place formational materials, the soils shall be removed prior to completion of the grading operation.

   The permanent access driveways to the lower-level garage areas shall be watered to optimum moisture content and compacted to at least 90 percent of Maximum Dry Density, in accordance with ASTM D1557-98 standards.

4. Any buried objects or abandoned utility lines, etc., which might be discovered in the construction areas, shall be removed and the excavation properly backfilled with approved on-site soils and compacted to at least 90 percent of Maximum Dry Density.

5. Any backfill soils placed in utility trenches or behind retaining walls that support structures and other improvements (such as patios, sidewalks, etc.) shall be compacted to at least 90 percent of Maximum Dry Density.

B. ***Design Parameters for Foundation and Retaining Walls***

6. For preliminary foundation design of new footings, based on the assumption that new footings will be placed at least 24 inches into dense natural (formational) soils, we provide an allowable soil bearing capacity equal to 2,500 pounds per square foot (psf). This applies to footings at least 24 inches into the bearing soils and at least 12 inches in width. For wider and/or deeper footings, the allowable soil bearing capacity may be calculated based on the following equation:

$$Qa = 1000D+500W$$

where

"Qa" is the allowable soil bearing capacity (in psf);

"D" is the depth of the footing (in feet) as measured from the ***lowest*** adjacent grade; and

"W" is the width of the footing (in feet).

The allowable soil bearing capacity may be increased one-third for analysis including wind or earthquake loads. The total maximum allowable vertical bearing capacity for proposed shallow foundations shall not exceed 5,000 psf.

7. The passive earth pressure of the encountered dense, natural-ground soils (to be used for design of shallow foundations and footings to resist the lateral forces) shall be based on an Equivalent Fluid Weight of 135 pounds per cubic

foot for footings below groundwater and 300 pounds per cubic foot for footings above groundwater. This passive earth pressure shall only be considered valid for design if the ground adjacent to the foundation structure is essentially level for a distance of at least three times the total depth of the foundation and is properly compacted or dense native soil. In addition, the lateral horizontal distance of properly compacted fill soils shall extend at least three times the depth of foundation being considered.

8. A Coefficient of Friction of 0.40 times the dead load may be used between the bearing soils and concrete foundations, walls, or floor slabs.

9. The following table summarizes site-specific seismic design criteria to calculate the base shear needed for the design of the structure. The design criteria was obtained from the California Building Code (CBC 2001 edition).

Parameter	**Value**	**Reference**
Seismic Zone Factor, Z	0.40	Table 16-I
Soil Profile Type	S_C	Table 16-J
Seismic Coefficient, C_a	$0.40N_a$	Table 16-Q
Seismic Coefficient, C_v	$0.56N_v$	Table 16-R
Near-Source Factor, N_a	1.0	Table 16-S
Near-Source Factor, N_v	1.0	Table 16-T
Seismic Source Type	B	Table 16-U

10. Our experience indicates that, for various reasons, footings and slabs occasionally crack, causing tile and brittle surfaces to become damaged. Therefore, we recommend that all conventional shallow footings and slabs-on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur.

10.1 A minimum of steel for continuous footings should include at least four No. 5 steel bars continuous, with two bars near the bottom of the footing and two bars near the top.

10.2 Isolated square footings should contain, as a minimum, a grid of three No. 5 steel bars on 12-inch centers, both ways, with no less than three bars each way.

10.3 The interior floor slab (on-grade) for the basement garage areas should be a minimum of 5 inches actual thickness and be reinforced with No. 3 bars on 12-inch centers, both ways, placed at midheight in the slab. *Basement slabs shall be underlain by a 4-inch-thick layer of crushed rock gravel or Class II base layer properly compacted. A waterproofing membrane may be included (such as Paraseal) if soil moisture is a concern to the owner/developer.* Slab subgrade soil shall be verified by a ***Geotechnical Exploration, Inc.*** representative to have the proper moisture content within 48 hours prior to placement of a vapor barrier and pouring of concrete. The base layer should drain to a planned low point where a sump pump should be installed.

We recommend the project Civil/Structural Engineer incorporate isolation joints and sawcuts to at least one-fourth the thickness of the slab in any floor designs. The joints and cuts, if properly placed, should reduce the potential for and help control floor slab cracking. It is recommended that concrete shrinkage joints be placed no further than 20 feet, approximately. However, due to a number of reasons (such as base preparation, construction techniques, curing procedures, and normal shrinkage of concrete), some cracking of slabs can be expected. ***Basement garage slabs should preferably be reinforced with enough steel to eliminate control***

joints. *NOTE: The project Civil/Structural Engineer shall review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum safeguards to reduce possible crack separations.*

Based on our laboratory test results and our experience with the soil types on the subject site, the dense natural soils should experience differential angular rotation of approximately 1/300 under the allowable loads. The maximum differential settlement across the structure and footings when founded on dense natural formation shall be on the order of 1½ inches.

11. As a minimum for protection of on-site improvements, it is recommended that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be founded on properly compacted and tested fill or dense native formation and underlain by at least 12 inches of nonexpansive, properly compacted soils, with No. 3 bars at 18 inches on-center at the mid-height of the slab, and contain adequate isolation and control joints. The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observations and testing. Any rigid improvements founded on the uncontrolled fill soils can be expected to undergo movement and possible damage and is therefore not recommended. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of the improvements. Any exterior area to receive concrete improvements shall be verified for compaction and moisture within 48 hours prior to concrete placement.

Control joints for exterior slabs shall be placed at spaces no farther than 15 feet apart, or the width of the slab, whichever is less, and also at reentrant corners. Control joints in exterior slabs shall be sealed with elastomeric joint sealant. The sealant shall be inspected every 6 months and be properly maintained by the owner.

C. ***Shoring Design Parameters***

12. The basement excavations may extend from each property line and across the entire lot therefore shoring will be needed. Most likely a soldier pile and lagging option will be used. If this option is chosen, we recommend the following soil pressure distribution:

> Any adjacent vertical surcharge may be converted to lateral pressure by multiplying by the 0.35 lateral pressure coefficient. For cantilever soldier pile and lagging, a linearly increasing soil pressure distribution may be used, with an active equivalent fluid weight equal to 35 pcf.

13. To calculate passive force, use the following:

> 300 pcf x 2.5 x diameter of drilled pier x depth of pier below lowest adjacent excavation grade. Use 135 pcf for drilled piers below water.

Total allowable end-bearing resistance of drilled piers into formational soils shall be calculated as follows:

> 1000 x depth into formation x area of tip of drilled pier to a maximum 20,000 psf

For shaft friction, use a unit friction pressure equal to 18Z where Z is the depth below the lowest adjacent grade, in feet. The friction pressure is in psf.

D. *Retaining Walls*

14. The active earth pressure above groundwater (to be utilized in the design of any cantilever walls, allowed to rotate) shall be based on an **Equivalent Fluid Weight** of **35** pounds per cubic foot (for level backfill only and low-expansive ***imported soils***). We recommend that very low-expansive to low-expansive soils be used as wall backfill material. Therefore, retaining wall plans should indicate that walls be backfilled with very low to low expansive soils (EI less than 50). Unless the retaining walls are provided with an effective drainage system, an additional fluid pressure of 62.4 pcf shall be added to the soil pressure.

    For design of restrained retaining walls, a uniform pressure equal to 9xH (nine times the total height of retained wall, considered in pounds per square foot) shall be considered as acting everywhere on the back of the wall ***in addition*** to the design Equivalent Fluid Weight. Any other surcharge loads applied within a horizontal distance measured from the face of the wall equal to its height shall be considered in the structural design

15. Due to presence of groundwater (derived primarily from the site's proximity to Alvarado Creek, as well as rainfall and irrigation), excess moisture is a common problem in below-grade structures or behind proposed basement level retaining walls. These problems are generally in the form of water seepage through walls, mineral staining, mold growth and high humidity.

Even without the presence of free water, the capillary draw characteristics, especially of fine-grained soils, can result in excessive transmission of water vapor through walls and floor slabs. In order to minimize the potential for moisture-related problems to develop at the site, proper ventilation and waterproofing shall be provided for below-ground areas and the backfill side of all structure retaining walls should be properly waterproofed and drained. As shown on Figure No. VII, the bottom of the perforated drain line should be at least 12 inches below the bottom of the interior floor slab. If the retaining wall subdrain is placed at the top of the foundation, water or moisture at slab subgrade may appear. In such cases, subdrains will need to be installed under the slab.

16. Proper subdrains and free-draining backwall material or geofabric drainage shall be installed behind all retaining walls (in addition to proper waterproofing) on the subject project. ***Geotechnical Exploration, Inc.*** will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans shall clearly indicate that the subdrains for any lower-level walls shall be placed at an elevation at least 1 foot below the bottom of the lower-level slabs. At least 0.5-percent fall shall be provided for the subdrain. The subdrain shall be placed in an envelope of crushed rock gravel up to 1 inch in maximum diameter, and be wrapped with Mirafi 140N filter or equivalent (see Figure No. VII). If a sump pump system is going to be used to reduce the water pressure on the basements, all sub drainage should be directed toward the sump pump locations.

In general, guidelines and requirements of Chapter 18 and its Appendix (CBC 2001 Edition) shall be followed for wall and floor basement waterproofing.

E. *Slopes*

17. Although shoring is proposed, we anticipate that temporary slopes into the formational material of approximately 10 to 12 feet in height may be required during the excavation process and construction of the basement-level garages. Based on the results of our field investigation, it is our opinion that the following temporary-slope design criteria may be considered in areas where the excavation slope top will be at least 10 feet away from any existing structures or improvements. The existing formational materials may be cut at an inclination of 0.75 horizontal to 1.0 vertical along the upper 5 feet (measured from present grade) and at 1.0:1.0 (horizontal to vertical) at lower elevations, for an unsupported period not to exceed eight weeks. After that time, a slope condition evaluation shall be provided by our firm. No soil stockpiles or surcharge may be placed within a horizontal distance equal to the height of the excavation.

    Any plans for slopes in excess of the anticipated 12-foot maximum must be presented to our office prior to grading to allow time for review and specific recommendations, if warranted. Proper drainage away from the excavation shall be provided at all times.

18. A representative of ***Geotechnical Exploration, Inc.*** must observe any steep temporary slopes *during construction*. In the event that soils comprising a slope are not as anticipated, any required slope design changes would be presented at that time.

19. Where not superseded by specific recommendations presented in this report, trenches, excavations and temporary slopes at the subject site shall be

constructed in accordance with Title 8, Construction Safety Orders, issued by OSHA.

F. ***Floor Slab Vapor Transmission***

20. Vapor moisture can cause some problems on moisture sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mildew and staining on slabs, walls and carpets.

21. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. The thicker the plastic, the stronger the resistance against puncturing.

22. Although polyethylene (visqueen) products are most commonly used, products such as Vaporshield possess much higher tensile strength and are more specifically designed for and intended to retard moisture transmission into concrete slabs. The use of Vaporshield or equivalent is highly recommended for this structure.

23. The vapor retarders need to have joints lapped and sealed with mastic or manufacturer's recommended tape for additional protection. To provide some protection to the moisture retarder, a layer of at least 4 inches of clean sand on top and 2 inches at the bottom shall also be provided. No heavy equipment, stakes or other puncturing instruments shall be used on top of the liner before or during concrete placement. In actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled

across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness.

The vapor retarders are not waterproof. They are intended to help prevent or reduce capillary migration of vapor through the soil into the pores of concrete slabs. Other waterproofing systems must supplement vapor retarders if full waterproofing is desired. The owner should be consulted to determine the specific level of protection required.

G. ***Site Drainage Considerations***

24. Adequate measures shall be taken to properly finish-grade the building site after the structures and other improvements are in place. Drainage waters from this site and adjacent properties are to be directed away from the foundations, floor slabs, and footings, onto the natural drainage direction for this area or into properly designed and approved drainage facilities. Roof gutters and downspouts should be installed on the structure, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help minimize the potential for waters to seek the level of the bearing soils under the foundations, footings and floor slabs or further erosion of the adjacent natural slope. Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site.

In addition, appropriate erosion control measures shall be taken at all times during construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas.

25. Planter areas, flower beds and planter boxes shall be sloped to drain away from the foundations, footings, and floor slabs at a gradient of at least 5 percent within 5 feet from the perimeter walls. Planter boxes shall be constructed with a closed bottom and a subsurface drain, installed in gravel, with the direction of subsurface and surface flow away from the slopes, foundations, footings, and floor slabs, to an adequate drainage facility. Sufficient area drains and proper surface gradient shall be provided throughout the project. Roof gutter and downspouts shall be tied to storm drain lines.

H. ***General Recommendations***

26. Following placement of any concrete floor slabs, sufficient drying time must be allowed prior to placement of floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

27. In order to minimize any work delays at the subject site during site development, this firm should be contacted 24 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.)

XI. GRADING NOTES

Any required grading operations shall be performed in accordance with the General Earthwork Specifications (Appendix E) and the requirements of the City of La Mesa Grading Ordinance.

28. ***Geotechnical Exploration, Inc.*** recommends that we be asked to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in the *"Report of Geotechnical Investigation"* for the project. In addition, the compaction of any fill soils placed during site grading work must be tested by a soil engineer. It is the responsibility of the grading contractor to comply with the requirements on the grading plans and the local grading ordinance. All retaining wall and trench backfill that will support structures or rigid improvements shall be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

29. It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the structural plans. We shall be provided with the opportunity to review the project plans once they are available, to see that our recommendations are adequately incorporated in the plans.

30. This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if

he considered any of the recommended actions presented herein to be unsafe.

XII. LIMITATIONS

Our conclusions and recommendations have been based on all available data obtained from our field investigation and laboratory analysis, as well as our experience with the soils and formational materials located in this area of the City of La Mesa. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time grading operations begin or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis which meet the contemporary standard of care in our profession within the County of San Diego. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 04-8598** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jay K. Heiser
Senior Project Geologist

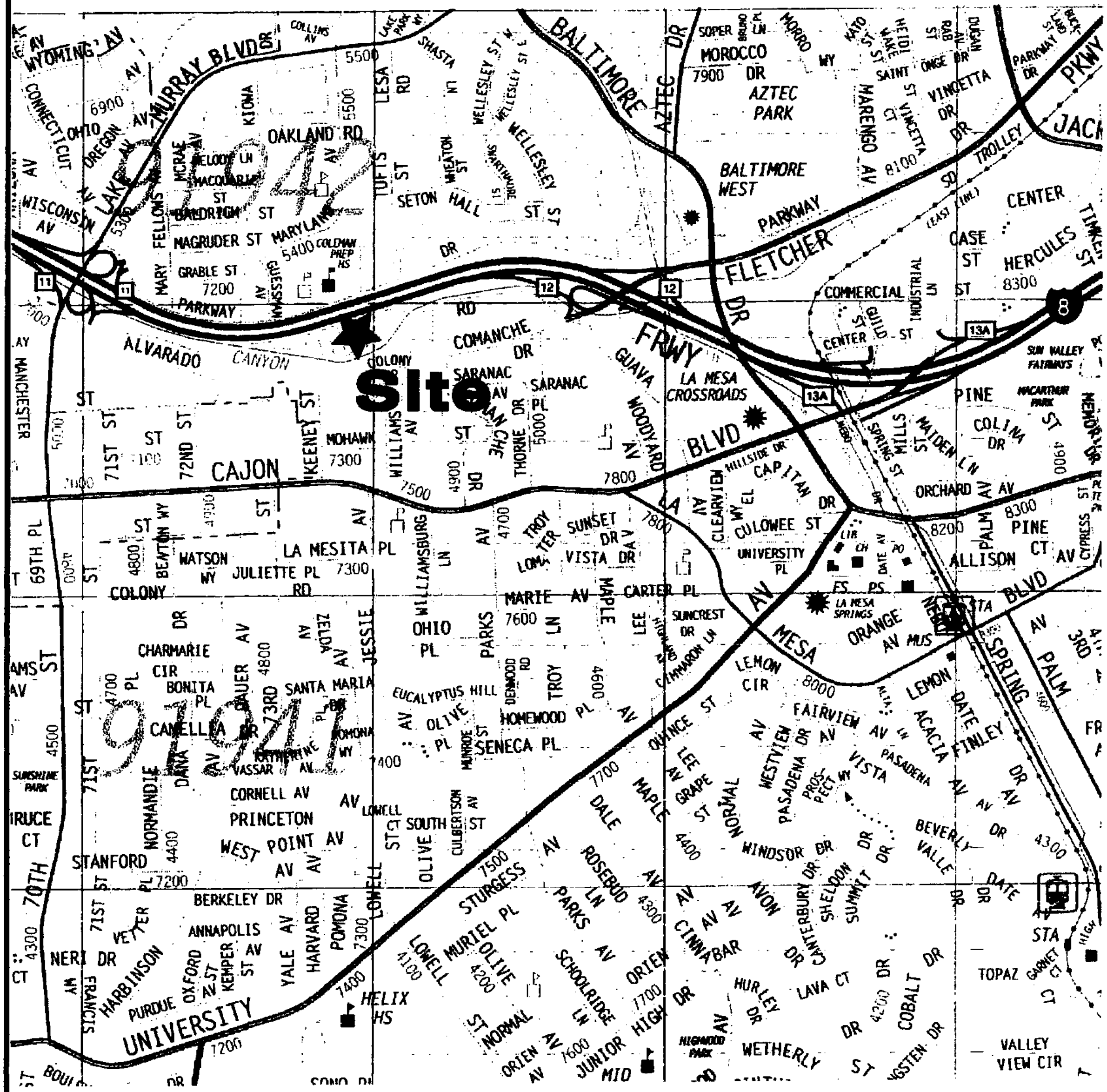
Leslie D. Reed, President
C.E.G. 999[exp. 3-31-05]/R.G. 3391

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Leslie D Reed
No. 999

Jaime A. Cerros
No. C02007
Exp. 9/30/05

VICINITY MAP

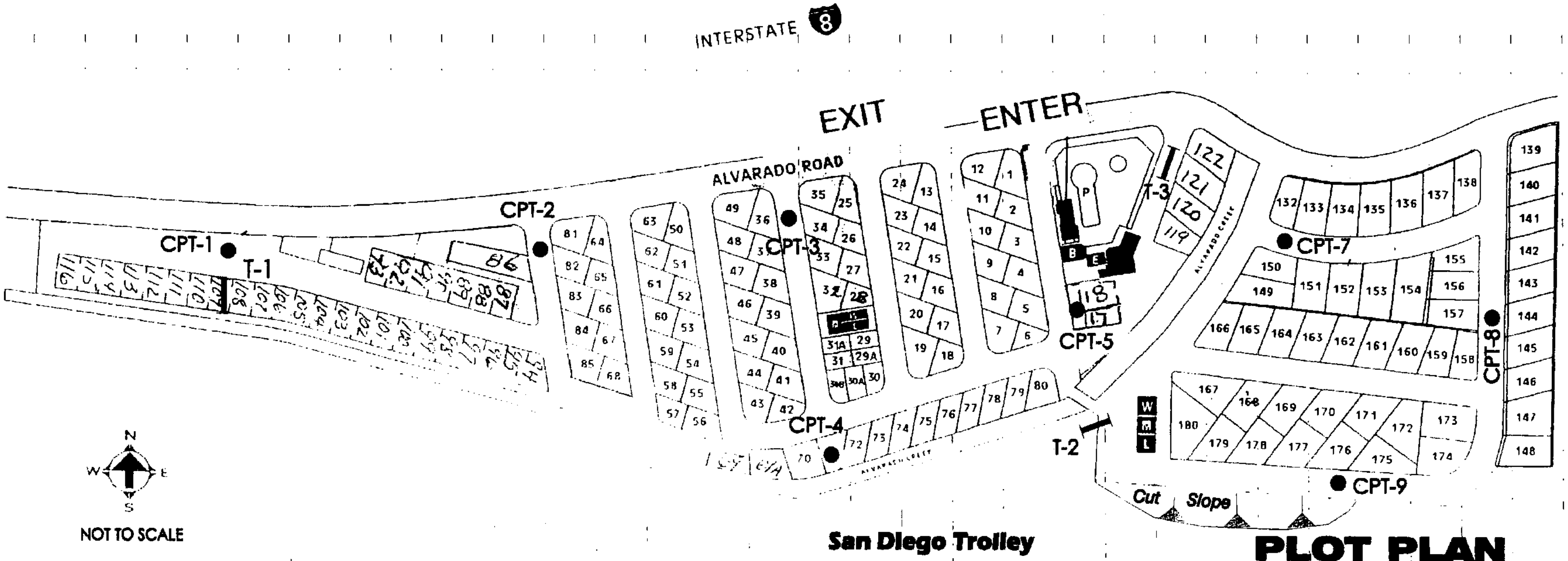


Thomas Bros Guide San Diego County pg 1270

Paydar Apartment Project
7407 Alvarado Road
La Mesa, CA.

Figure No. I
Job No. 04-8598

NOTE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.



GEI Legend

● CPT-9 APPROXIMATE LOCATION OF CONE PENETROMETER TEST

T-3 APPROXIMATE LOCATION OF EXPLORATORY TRENCH

REFERENCE: This Plot Plan was prepared from an existing udated Plan provided by the client and from on-site field reconnaissance performed by GEI.

PLOT PLAN

Paydar Apartment Project
7407 Alvarado Road
La Mesa, CA.
Figure No. II
Job No. 04-8598

Geotechnical Exploration, Inc.

June 2004

04-8598-p

EQUIPMENT **Rubber-tire Backhoe**	DIMENSION & TYPE OF EXCAVATION **2' X 9' X 7' Trench**	DATE LOGGED **4-26-04**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER DEPTH **at -3 feet**	LOGGED BY **JKH**

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.								
			SILTY FINE TO COARSE SAND, w/ some pebbles and cobbles. Medium dense. Damp. Light gray-brown.	SM-GM								
1			**FILL (Qaf)**									
2			**FINE TO COARSE SAND**, w/ abundant pebbles and cobbles (to 6" in diameter). Medium dense. Moist to wet. Tan-gray and orange.	GM								
3	▼		**STREAM DEPOSITS**									
4			**FINE TO COARSE SAND**, w/ abundant cobbles and boulders (to 12" - 14" in diameter). Medium dense. Wet (saturated). Tan-gray and brown.	GM								
5			**STREAM DEPOSITS**									
6			East side of trench exposed: **COBBLE CONGLOMERATE**, w/ clayey sand and sandy clay matrix. Medium dense. Moist to wet. Tan-gray.									
7			**STADIUM CONGLOMERATE (Tst).**									
			Bottom @ 7'									
8												

▼ WATER TABLE

☒ LOOSE BAG SAMPLE

[1] IN-PLACE SAMPLE

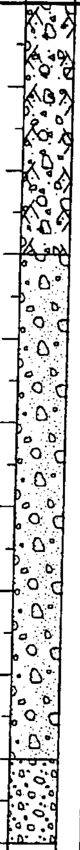
■ DRIVE SAMPLE

[S] SAND CONE/F.D.T.

▨ STANDARD PENETROMETER

JOB NAME **Paydar Apartment Site**		
SITE LOCATION **7407 Alvarado Road, La Mesa, California**		
JOB NUMBER **04-8598**	REVIEWED BY **LDR/JAC**	LOG No. **T-1**
FIGURE NUMBER **IIIa**	**Geotechnical Exploration, Inc.**	

EQUIPMENT **Rubber-tire Backhoe**	DIMENSION & TYPE OF EXCAVATION **2' X 10' X 10' Trench**	DATE LOGGED **4-26-04**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER DEPTH **Not Encountered**	LOGGED BY **JKH**



DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–3			**SILTY FINE TO COARSE SAND,** w/ some clay and abundant pebbles and cobbles (to 4" in diameter). Medium dense. Damp. Dark red-brown. **FILL (Qaf)** -- roots encountered.	SM-GM								
3–9			**FINE TO COARSE SAND,** w/ abundant pebbles and cobbles and boulders (to 12" - 14" in diameter). Medium dense. Dry to damp. Tan-gray and orange-brown. **STREAM DEPOSITS** -- roots encountered. -- roots encountered.	GM								
9–10			**COBBLE CONGLOMERATE,** w/ fine to medium sand and clay matrix. Dense. Damp. Tan-gray. **STADIUM CONGLOMERATE (Tst)**	GM								
10			Bottom @ 10'									

▼ WATER TABLE
☒ LOOSE BAG SAMPLE
1 IN-PLACE SAMPLE
■ DRIVE SAMPLE
S SAND CONE/F.D.T.
▨ POCKET PENETROMETER

JOB NAME **Paydar Apartment Site**		
SITE LOCATION **7407 Alvarado Road, La Mesa, California**		
JOB NUMBER **04-8598**	REVIEWED BY **LDR/JAC**	LOG No. **T-2**
FIGURE NUMBER **IIIb**	**Geotechnical Exploration, Inc.**	

EQUIPMENT **Rubber-tire Backhoe**	DIMENSION & TYPE OF EXCAVATION **2' X 10' X 4' Trench**	DATE LOGGED **4-26-04**
SURFACE ELEVATION **± 410' Mean Sea Level**	GROUNDWATER DEPTH **Not Encountered**	LOGGED BY **JKH**

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
1			**SILTY FINE TO MEDIUM SAND**, w/ abundant roots and some cobbles and rock fragments (to 4" in diameter). Loose to medium dense. Damp to moist. Dark brown. **FILL (Qaf)**	SM								
2		☒					9.0	127.0				
3			**COBBLE CONGLOMERATE**, w/ fine to medium sand matrix. Dense. Damp. Tan-gray and orange. **STADIUM CONGLOMERATE (Tst)**	GM								
4			Bottom @ 4'									
5												
6												
7												
8												
9												

▼ WATER TABLE
☒ LOOSE BAG SAMPLE
1 IN-PLACE SAMPLE
■ DRIVE SAMPLE
S SAND CONE/F.D.T.
▨ STANDARD PENETROMETER

JOB NAME
Paydar Apartment Site

SITE LOCATION
7407 Alvarado Road, La Mesa, California

JOB NUMBER
04-8598

FIGURE NUMBER
IIIc

REVIEWED BY **LDR/JAC**

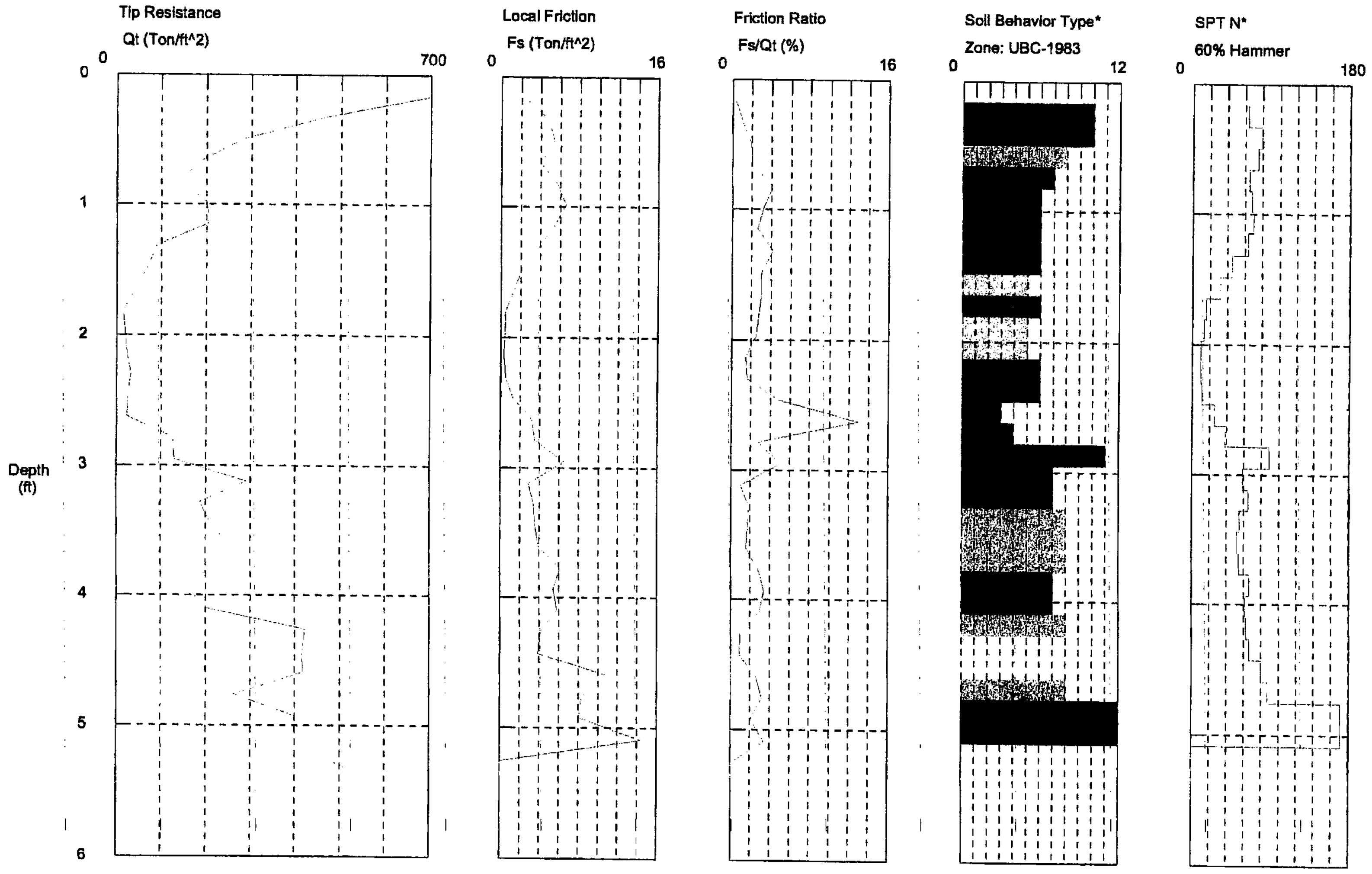
Geotechnical Exploration, Inc.

LOG No.
T-3

EXPLORATION LOG 8598 PAYDAR.GPJ GEO_EXPL.GDT 6/28/04

Geotechnical Explorations

Operator: Victor/Mike	CPT Date/Time: 3/26/2004 9:15:
Sounding: CPT-01	Location: San Diego RV Park
Cone Used: DSA0408	Job Number: 04-8598



Maximum Depth = 5.41 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
■ 2 organic material
■ 3 clay
SDF77

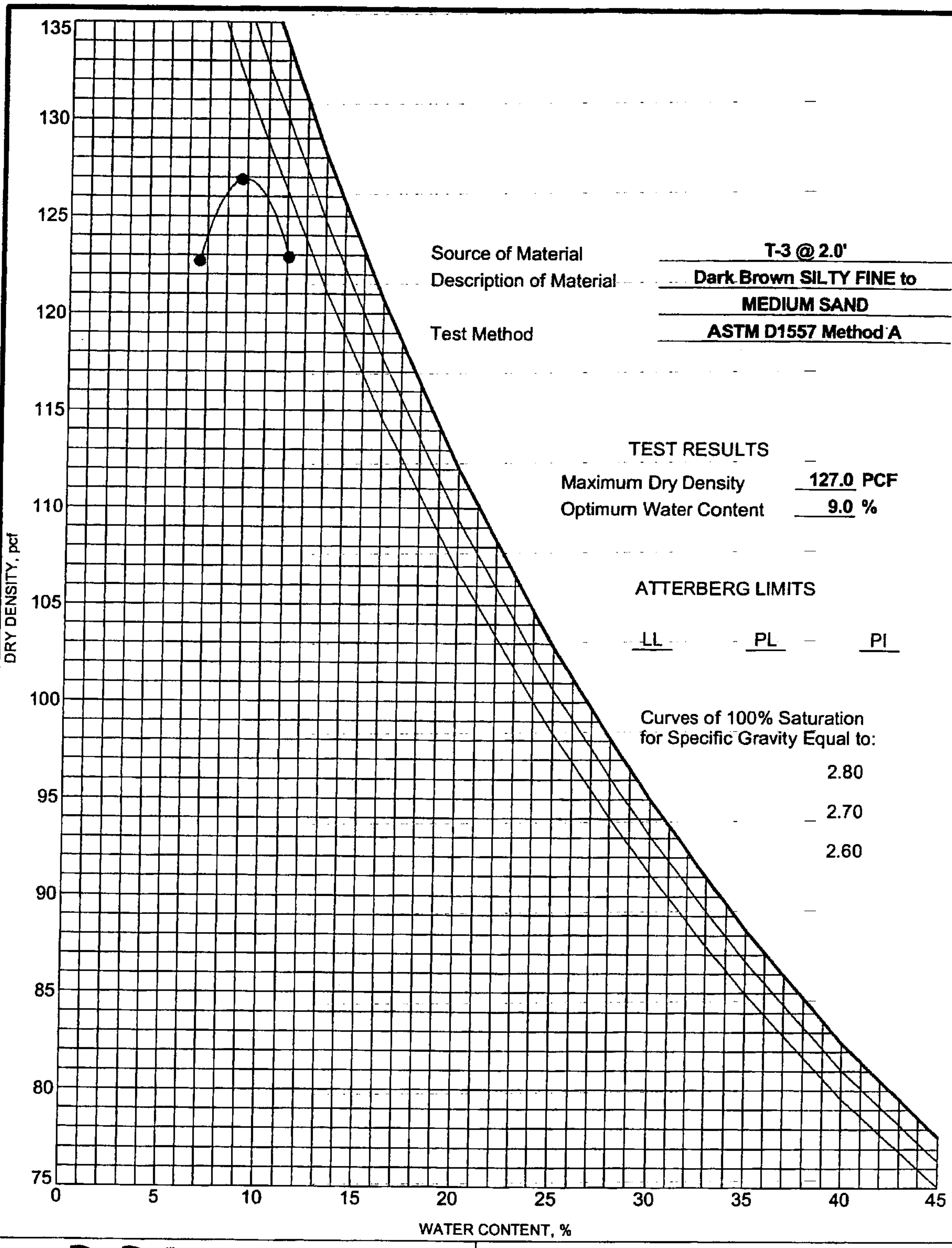
■ 4 silty clay to clay
5 clayey silt to silty clay
■ 6 sandy silt to clayey silt

■ 7 silty sand to sandy silt
▨ 8 sand to silty sand
9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

Figure No. IVa



Source of Material T-3 @ 2.0'

Description of Material Dark Brown SILTY FINE to MEDIUM SAND

Test Method ASTM D1557 Method A

TEST RESULTS

Maximum Dry Density 127.0 PCF

Optimum Water Content 9.0 %

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation for Specific Gravity Equal to:

2.80

2.70

2.60

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

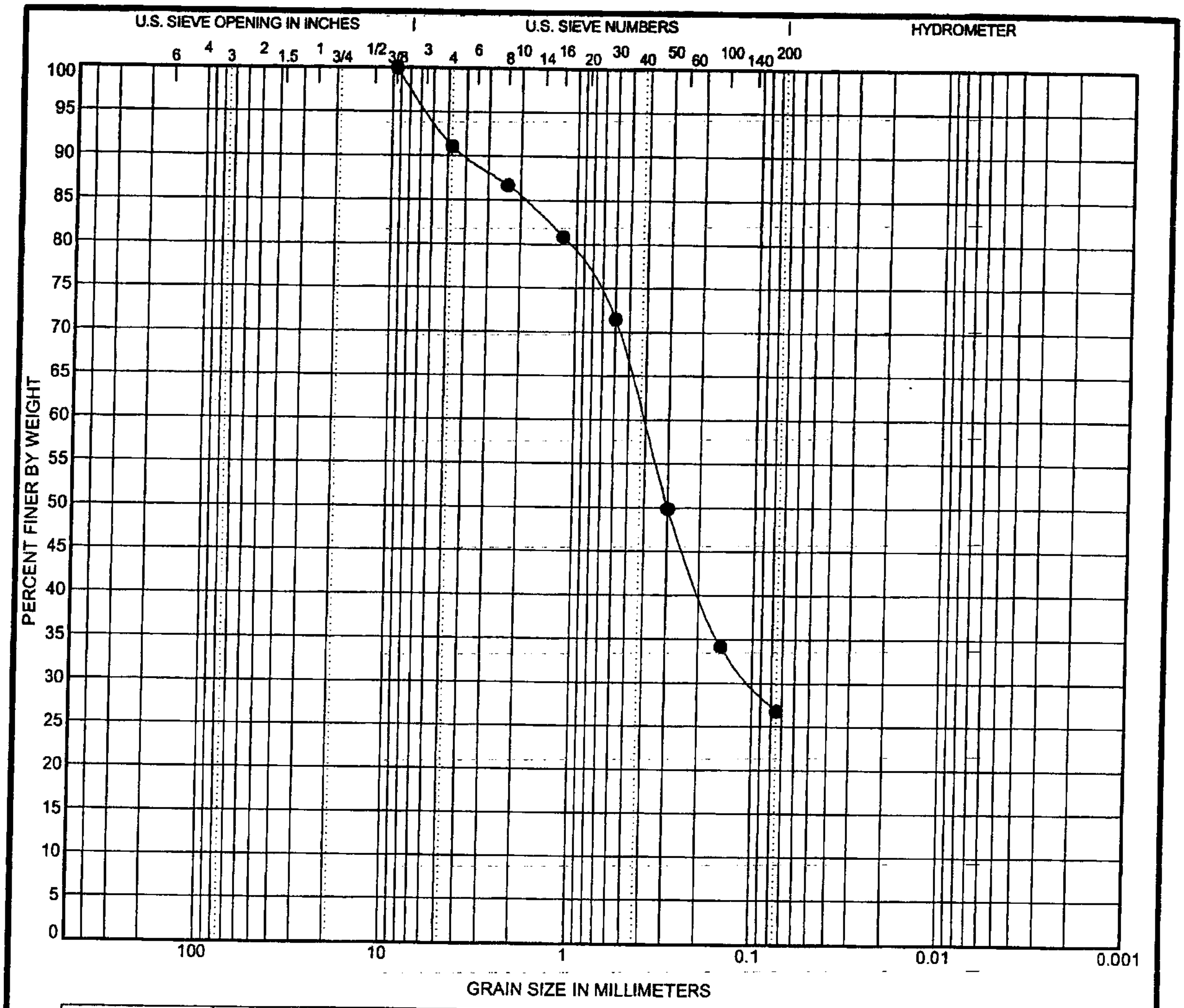
Figure Number: Va

Job Name: Paydar Apartment Site

Site Location: 7407 Alvarado Road, La Mesa, California

Job Number: 04-8598

US_COMPACTION 8598 PAYDAR.GPJ GEO_EXPL.GDT 8/28/04



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	T-3 @ 2.0'	Dark Brown SILTY FINE to MEDIUM SAND					

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	T-3 @ 2.0'	9.5	0.415	0.101		8.9	64.3	26.8	

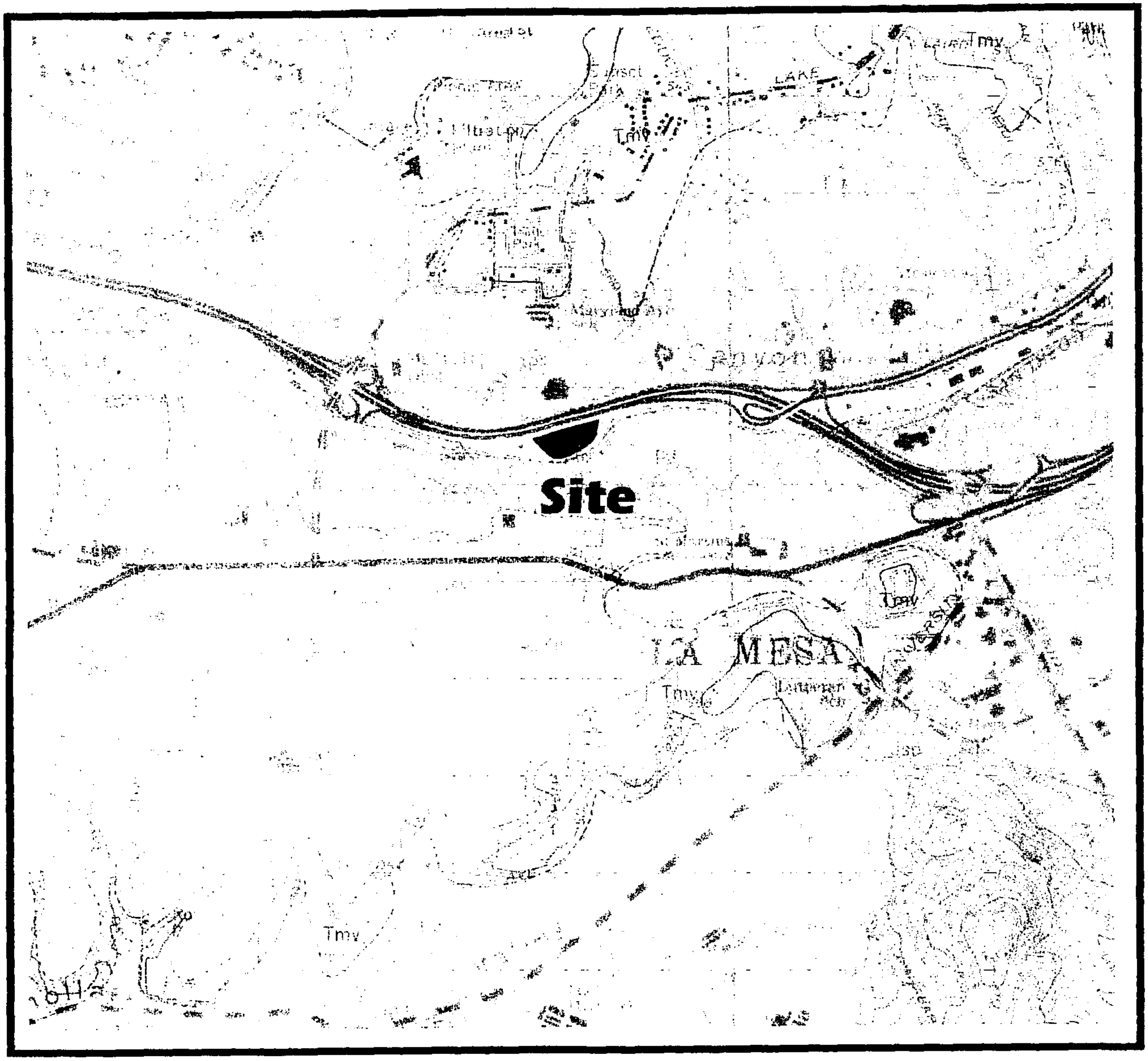
Geotechnical Exploration, Inc.

GRAIN SIZE DISTRIBUTION

Figure Number: Vb
Job Name: Paydar Apartment Site
Site Location: 7407 Alvarado Road, La Mesa, California
Job Number: 04-8598

US GRAIN SIZE 8598 PAYDAR.GPJ GEO EXPL.GDT 6/28/04

GEOLOGY MAP
1975
by Michael P. Kennedy and G.L. Peterson



Paydar Apartment Project
7407 Alvarado Road
La Mesa, CA.

Figure No. VIa
Job No. 04-8598

June 2004

GEOLOGY OF THE LA MESA QUADRANGLE SAN DIEGO COUNTY, CALIFORNIA

by Michael P. Kennedy and G. L. Peterson

1975

EXPLANATION

QUATERNARY

Holocene

Qsw

Qsw

Alluvium and Slopewash

Qsw, Slopewash; Qal & Qsw, Alluvium and Slopewash undifferentiated.

Qls

Landslide deposits

Pleistocene

Qt

Stream-terrace deposits

Ql

Lindavista Formation

TERTIARY

Pliocene

San Diego Formation

Eocene

Tmv

Poway Group

Tp, Pomerado Conglomerate; Tmv, Mission Valley Formation; Tst, Stadium Conglomerate. Conglomerate marked by circle pattern.

Tf

Friars Formation

Conglomerate marked by circle pattern.

CRETACEOUS

Kg

Granite rocks

Undifferentiated granitic rocks of the southern California batholith.

JURASSIC

Jsp

Santiago Peak Volcanics

SYMBOLS

Contact
(dashed where approximately located; dotted where concealed)

Fault, showing dip
(dashed where approximately located; dotted where concealed; U, upthrown side; D, downthrown side).

5

Strike and dip of bedding in sedimentary rocks.

40

Strike and dip of bedding in metasedimentary rocks.

Qls

Landslide with direction of movement indicated by arrows.

Δ 12-16

Clay sample locality.

× 12-23

Pit, quarry, or mine.

★ 5

Fossil coccolith locality.

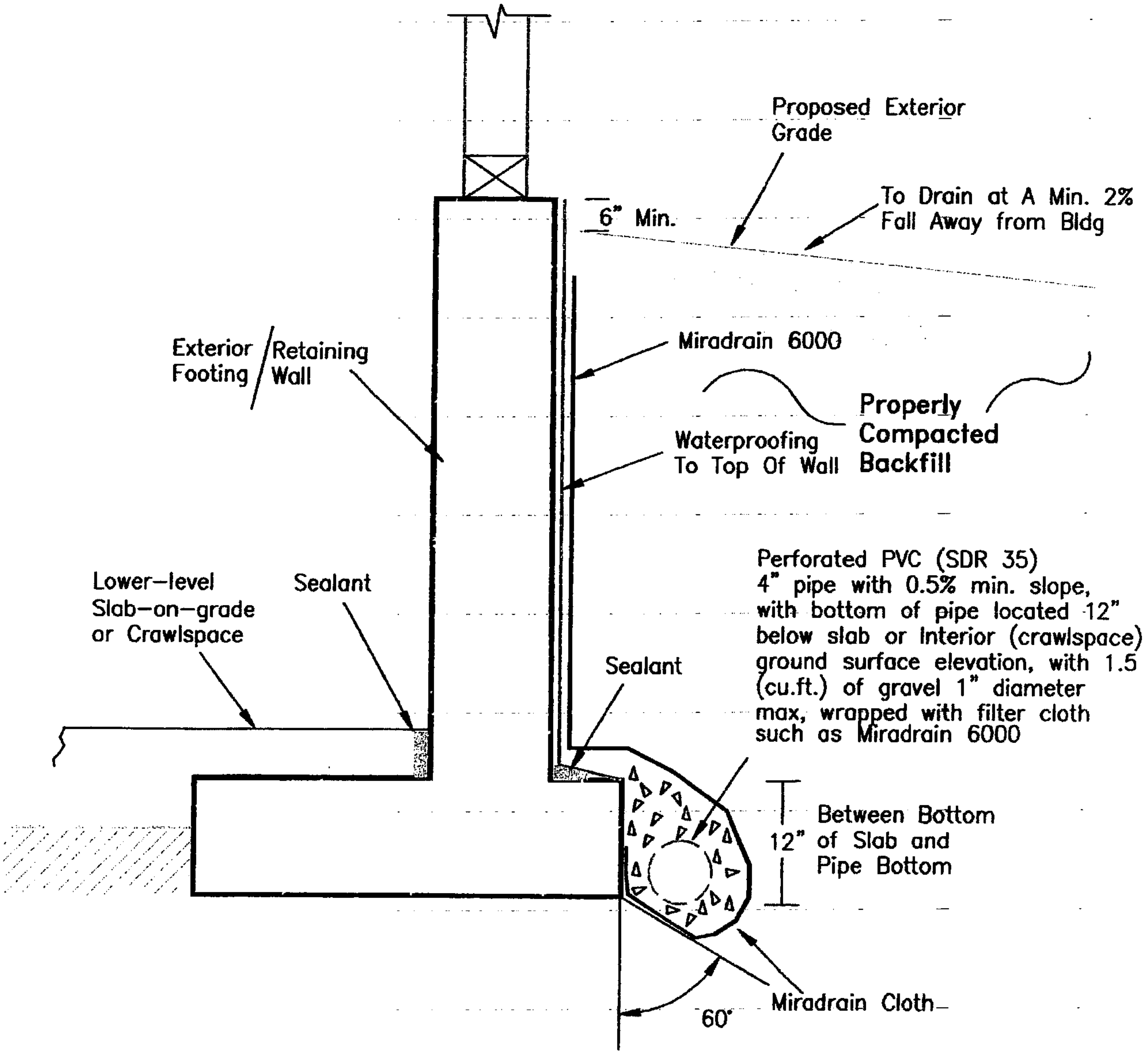
• 16

Fossil vertebrate locality.

Figure No. VIb
Job No. 04-8598

June 2004

TYPICAL SUBGRADE RETAINING WALL DRAINAGE RECOMMENDATIONS



NOT TO SCALE

Figure No. VII
Job No. 04-8598

NOTE: As an option to Miradrain 6000, Gravel or Crushed rock 3/4" maximum diameter may be used with a minimum 12" thickness along the interior face of the wall and 2.0 cu.ft./ft. of pipe gravel envelope.

GEi Geotechnical Exploration, Inc.

APPENDIX A
UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION

Coarse-grained (More than half of material is larger than a No. 200 sieve)

GRAVELS, CLEAN GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3")	GW	Well-graded gravels, gravel and sand mixtures, little or no fines.
	GP	Poorly graded gravels, gravel and sand mixtures, little or no fines.
GRAVELS WITH FINES (Appreciable amount)	GC	Clay gravels, poorly graded gravel-sand-silt mixtures
SANDS, CLEAN SANDS (More than half of coarse fraction is smaller than a No. 4 sieve)	SW	Well-graded sand, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines.
SANDS WITH FINES (Appreciable amount)	SM	Silty sands, poorly graded sand and silty mixtures.
	SC	Clayey sands, poorly graded sand and clay mixtures.

FINE-GRAINED (More than half of material is smaller than a No. 200 sieve)

SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour, sandy silt and clayey-silt sand mixtures with a slight plasticity.
Liquid Limit Less than 50	CL	Inorganic clays of low to medium plasticity, gravelly clays, silty clays, clean clays.
	OL	Organic silts and organic silty clays of low plasticity.
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
Liquid Limit Greater than 50	CH	Inorganic clays of high plasticity, fat-clays.
	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

APPENDIX B

EQ FAULT TABLES

```
                    ***********************
                    *                     *
                    *    E Q F A U L T    *
                    *                     *
                    *    Version 3.00     *
                    *                     *
                    ***********************

                 DETERMINISTIC ESTIMATION OF
              PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 04-8598
                                          DATE: 06-16-2004

JOB NAME: Paydar Test Run

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT


SITE COORDINATES:
   SITE LATITUDE:  32.7722
   SITE LONGITUDE:  117.0361

SEARCH RADIUS:   100  mi

ATTENUATION RELATION:  12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   DISTANCE MEASURE:  cdist
   SCOND:   1
   Basement Depth:  5.00 km     Campbell SSR:  1     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED:  CDMGFLTE.DAT


MINIMUM DEPTH VALUE (km):  3.0
```

```
                        ---------------
                        EQFAULT SUMMARY
                        ---------------


                 -----------------------------
                 DETERMINISTIC SITE PARAMETERS
                 -----------------------------

Page  1
-------------------------------------------------------------------------------
                              |              |ESTIMATED MAX. EARTHQUAKE EVENT
                              | APPROXIMATE  |-------------------------------
      ABBREVIATED             |  DISTANCE    | MAXIMUM  |  PEAK    |EST. SITE
      FAULT  NAME             |  mi   (km)   |EARTHQUAKE|  SITE    |INTENSITY
                              |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
==============================|==============|==========|==========|=========
ROSE CANYON                   |  7.4(  11.9)|   6.9    |  0.275   |   IX
CORONADO BANK                 | 21.3(  34.2)|   7.4    |  0.147   |  VIII
ELSINORE-JULIAN               | 34.2(  55.1)|   7.1    |  0.074   |  VII
NEWPORT-INGLEWOOD (Offshore)  | 35.0(  56.4)|   6.9    |  0.063   |  VI
EARTHQUAKE VALLEY             | 38.7(  62.3)|   6.5    |  0.043   |  VI
ELSINORE-COYOTE MOUNTAIN      | 41.4(  66.6)|   6.8    |  0.049   |  VI
ELSINORE-TEMECULA             | 41.9(  67.4)|   6.8    |  0.049   |  VI
SAN JACINTO-COYOTE CREEK      | 55.4(  89.1)|   6.8    |  0.036   |   V
SAN JACINTO-ANZA              | 56.7(  91.2)|   7.2    |  0.047   |  VI
SAN JACINTO - BORREGO         | 57.2(  92.0)|   6.6    |  0.031   |   V
ELSINORE-GLEN IVY             | 62.9( 101.2)|   6.8    |  0.032   |   V
PALOS VERDES                  | 63.2( 101.7)|   7.1    |  0.039   |   V
SUPERSTITION MTN. (San Jacinto) | 66.5( 107.1)|   6.6    |  0.026   |   V
LAGUNA SALADA                 | 67.2( 108.1)|   7.0    |  0.034   |   V
SAN JACINTO-SAN JACINTO VALLEY | 67.2( 108.2)|   6.9    |  0.032   |   V
ELMORE RANCH                  | 71.0( 114.2)|   6.6    |  0.024   |   V
SUPERSTITION HILLS (San Jacinto)| 71.6( 115.2)|   6.6    |  0.024   |   V
NEWPORT-INGLEWOOD (L.A.Basin) | 77.4( 124.5)|   6.9    |  0.027   |   V
CHINO-CENTRAL AVE. (Elsinore) | 78.9( 126.9)|   6.7    |  0.033   |   V
SAN ANDREAS - Coachella       | 82.3( 132.5)|   7.1    |  0.030   |   V
SAN ANDREAS - Southern        | 82.3( 132.5)|   7.4    |  0.037   |   V
WHITTIER                      | 82,5( 132.8)|   6.8    |  0.024   |  IV
SAN ANDREAS - San Bernardino  | 84.9( 136.6)|   7.3    |  0.033   |   V
IMPERIAL                      | 85.9( 138.3)|   7.0    |  0.026   |   V
BRAWLEY SEISMIC ZONE          | 86.7( 139.5)|   6.4    |  0.017   |  IV
SAN JACINTO-SAN BERNARDINO    | 86.8( 139.7)|   6.7    |  0.021   |  IV
```

```
COMPTON THRUST                 |  87.0( 140.0)|   6.8    |   0.032  |   V
BURNT MTN.                     |  89.7( 144.3)|   6.4    |   0.017  |  IV
ELYSIAN PARK THRUST            |  90.4( 145.5)|   6.7    |   0.029  |   V
PINTO MOUNTAIN                 |  90.7( 146.0)|   7.0    |   0.025  |   V
EUREKA PEAK                    |  92.1( 148.2)|   6.4    |   0.016  |  IV
SAN JOSE                       | 99.8 ( 160.6)|   6.5    |   0.022  |  IV
*******************************************************************************
```

```
-END OF SEARCH-   32 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON                  FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.4 MILES (11.9 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.2749 g
```

```
                    ***********************
                    *                     *
                    *     E Q F A U L T   *
                    *                     *
                    *     Version 3.00    *
                    *                     *
                    ***********************

                 DETERMINISTIC ESTIMATION OF
              PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 04-0598
                                             DATE: 06-16-2004

JOB NAME: Paydar Test Run

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT


SITE COORDINATES:
   SITE LATITUDE:  32.7722
   SITE LONGITUDE:  117.0361

SEARCH RADIUS:   100  mi

ATTENUATION RELATION:  12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   DISTANCE MEASURE:  cdist
   SCOND:   1
   Basement Depth:  5.00 km     Campbell SSR:  1     Campbell SHR:  0
   COMPUTE RHGA HORIZ. ACCEL. (FACTOR:   0.65    DISTANCE:   20  miles)

FAULT-DATA FILE USED:  CDMGFLTE.DAT


MINIMUM DEPTH VALUE (km):  3.0
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                    EQFAULT SUMMARY
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              -----------------------------
              DETERMINISTIC SITE PARAMETERS
              -----------------------------

Page  1
-------------------------------------------------------------------------------
                               |              |ESTIMATED MAX. EARTHQUAKE EVENT
                               | APPROXIMATE  |------------------------------
         ABBREVIATED           |   DISTANCE   | MAXIMUM  |   RHGA   |EST. SITE
         FAULT  NAME           |  mi    (km)  |EARTHQUAKE|   SITE   |INTENSITY
                               |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
===============================|==============|==========|==========|=========
ROSE CANYON                    |   7.4(  11.9)|   6.9    |   0.179  |  VIII
CORONADO BANK                  |  21.3(  34.2)|   7.4    |   0.147  |  VIII
ELSINORE-JULIAN                |  34.2(  55.1)|   7.1    |   0.074  |   VII
NEWPORT-INGLEWOOD (Offshore)   |  35.0(  56.4)|   6.9    |   0.063  |   VI
EARTHQUAKE VALLEY              |  38.7(  62.3)|   6.5    |   0.043  |   VI
ELSINORE-COYOTE MOUNTAIN       |  41.4(  66.6)|   6.8    |   0.049  |   VI
ELSINORE-TEMECULA              |  41.9(  67.4)|   6.8    |   0.049  |   VI
SAN JACINTO-COYOTE CREEK       |  55.4(  89.1)|   6.8    |   0.036  |    V
SAN JACINTO-ANZA               |  56.7(  91.2)|   7.2    |   0.047  |   VI
SAN JACINTO - BORREGO          |  57.2(  92.0)|   6.6    |   0.031  |    V
ELSINORE-GLEN IVY              |  62.9( 101.2)|   6.8    |   0.032  |    V
PALOS VERDES                   |  63.2( 101.7)|   7.1    |   0.039  |    V
SUPERSTITION MTN. (San Jacinto)|  66.5( 107.1)|   6.6    |   0.026  |    V
LAGUNA SALADA                  |  67.2( 108.1)|   7.0    |   0.034  |    V
SAN JACINTO-SAN JACINTO VALLEY |  67.2( 108.2)|   6.9    |   0.032  |    V
ELMORE RANCH                   |  71.0( 114.2)|   6.6    |   0.024  |    V
SUPERSTITION HILLS (San Jacinto)|  71.6( 115.2)|   6.6    |   0.024  |    V
NEWPORT-INGLEWOOD (L.A.Basin)  |  77.4( 124.5)|   6.9    |   0.027  |    V
CHINO-CENTRAL AVE. (Elsinore)  |  78.9( 126.9)|   6.7    |   0.033  |    V
SAN ANDREAS - Coachella        |  82.3( 132.5)|   7.1    |   0.030  |    V
SAN ANDREAS - Southern         |  82.3( 132.5)|   7.4    |   0.037  |    V
WHITTIER                       |  82.5( 132.8)|   6.8    |   0.024  |   IV
SAN ANDREAS - San Bernardino   |  84.9( 136.6)|   7.3    |   0.033  |    V
IMPERIAL                       |  85.9( 138.3)|   7.0    |   0.026  |    V
BRAWLEY SEISMIC ZONE           |  86.7( 139.5)|   6.4    |   0.017  |   IV
SAN JACINTO-SAN BERNARDINO     |  86.8( 139.7)|   6.7    |   0.021  |   IV
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COMPTON THRUST                 |  87.0( 140.0)|   6.8    |   0.032  |   V
BURNT MTN.                     |  89.7( 144.3)|   6.4    |   0.017  |  IV
ELYSIAN PARK THRUST            |  90.4( 145.5)|   6.7    |   0.029  |   V
PINTO MOUNTAIN                 |  90.7( 146.0)|   7.0    |   0.025  |   V
EUREKA PEAK                    |  92.1( 148.2)|   6.4    |   0.016  |  IV
SAN JOSE                       | 99.8 ( 160.6)|   6.5    |   0.022  |  IV
********************************************************************************




-END OF SEARCH-   32 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON                      FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.4 MILES (11.9 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.1787 g
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APPENDIX C

EQ SEARCH TABLES

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                    *                       *
                    *     E Q S E A R C H   *
                    *                       *
                    *     Version 3.00      *
                    *                       *
                    *************************

                         ESTIMATION OF
                      PEAK ACCELERATION FROM
                   CALIFORNIA EARTHQUAKE CATALOGS


 JOB NUMBER: 04-8598
                                                   DATE: 06-16-2004

 JOB NAME: Paydar Test Run

 EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


 MAGNITUDE RANGE:
    MINIMUM MAGNITUDE:  5.00
    MAXIMUM MAGNITUDE:  9.00

 SITE COORDINATES:
    SITE LATITUDE:  32.7722
    SITE LONGITUDE:  117.0361

 SEARCH DATES:
             START DATE:   1800
             END DATE:   2003

 SEARCH RADIUS:
             100.0 mi
             160.9 km

 ATTENUATION RELATION:  12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
    UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
    ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
    SCOND:   0  Depth Source:  A
    Basement Depth:  5.00 km     Campbell SSR:  1     Campbell SHR:  0
    COMPUTE PEAK HORIZONTAL ACCELERATION

 MINIMUM DEPTH VALUE (km):  3.0
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TEST.OUT

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                     EARTHQUAKE SEARCH RESULTS
                     -------------------------

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    |         |          |            |  TIME    |      |       |  SITE   |SITE|  APPROX.
FILE|  LAT.   |  LONG.   |    DATE    |  (UTC)   |DEPTH |QUAKE  |  ACC.   | MM |  DISTANCE
CODE| NORTH   |  WEST    |            | H M Sec  | (km) | MAG.  |   g     |INT.|  mi  [km]
----+---------+----------+------------+----------+------+-------+---------+----+-----------
MGI |32.8000|117.1000|05/25/1803| 0  0  0.0|   0.0| 5.00|  0.166 |VIII|  4.2(  6.7)
T-A |32.6700|117.1700|10/21/1862| 0  0  0.0|   0.0| 5.00|  0.088 | VII| 10.5( 16.9)
T-A |32.6700|117.1700|12/00/1856| 0  0  0.0|   0.0| 5.00|  0.088 | VII| 10.5( 16.9)
T-A |32.6700|117.1700|05/24/1865| 0  0  0.0|   0.0| 5.00|  0.088 | VII| 10.5( 16.9)
DMG |32.7000|117.2000|05/27/1862|20  0  0.0|   0.0| 5.00|  0.088 | VII| 10.5( 16.9)
DMG |32.8000|116.8000|10/23/1894|23  3  0.0|   0.0| 5.90|  0.154 |VIII| 10.7( 17.3)
MGI |33.0000|117.0000|09/21/1856| 730  0.0|   0.0| 5.70|  0.105 | VII| 13.8( 22.3)
DMG |33.0000|117.3000|11/22/1800|2130  0.0|   0.0| 5.00|  0.060 | VI | 15.9( 25.5)
DMG |33.2000|116.7000|01/01/1920| 235  0.0|   0.0| 6.50|  0.110 | VII| 21.9( 35.3)
DMG |33.0000|116.4330|06/04/1940|1035  8.3|   0.0| 5.00|  0.027 |  V | 35.4( 56.9)
MGI |33.2000|116.6000|10/12/1920|1748  0.0|   0.0| 5.10|  0.026 |  V | 38.3( 61.7)
DMG |32.7000|116.3000|02/24/1892| 720  0.0|   0.0| 6.70|  0.029 |  V | 38.9( 62.5)
T-A |32.2500|117.5000|01/13/1877|20  0  0.0|   0.0| 5.00|  0.062 | VI | 43.0( 69.3)
DMG |32.2000|116.5500|11/04/1949|204238.0|   0.0| 5.70|  0.021 | IV | 45.0( 72.5)
DMG |32.2000|116.5500|11/05/1949| 43524.0|   0.0| 5.10|  0.029 |  V | 48.6( 78.2)
PAS |32.9710|117.8700|07/13/1986|1347  8.2|   6.0| 5.30|  0.020 | IV | 48.6( 78.2)
DMG |32.0830|116.6670|11/25/1934| 818  0.0|   0.0| 5.00|  0.022 | IV | 50.3( 80.9)
DMG |33.3430|116.3460|04/28/1969|232042.9|  20.0| 5.80|  0.018 | IV | 52.2( 84.0)
DMG |33.2000|116.2000|05/28/1892|1115  0.0|   0.0| 6.30|  0.026 |  V | 56.1( 90.3)
PAS |33.5010|116.5130|02/25/1980|104738.5|  13.6| 5.50|  0.036 |  V | 56.7( 91.3)
DMG |33.5000|116.5000|09/30/1916| 211  0.0|   0.0| 5.00|  0.021 | IV | 58.7( 94.5)
DMG |32.0000|117.5000|05/01/1939|2353  0.0|   0.0| 5.00|  0.016 | IV | 59.0( 95.0)
DMG |32.0000|117.5000|06/24/1939|1627  0.0|   0.0| 5.00|  0.015 | IV | 59.8( 96.2)
DMG |33.1900|116.1290|04/09/1968| 22859.1|  11.1| 6.40|  0.015 | IV | 59.8( 96.2)
DMG |33.2170|116.1330|08/15/1945|175624.0|   0.0| 5.70|  0.036 |  V | 59.9( 96.4)
DMG |33.2830|116.1830|03/23/1954| 41450.0|   0.0| 5.10|  0.023 | IV | 60.6( 97.6)
DMG |33.2830|116.1830|03/19/1954| 95429.0|   0.0| 6.20|  0.016 | IV | 60.7( 97.7)
DMG |33.2830|116.1830|03/19/1954|182117.0|   0.0| 5.50|  0.031 |  V | 60.7( 97.7)
DMG |33.2830|116.1830|03/19/1954| 95556.0|   0.0| 5.00|  0.020 | IV | 60.7( 97.7)
                                                            0.015 | IV | 60.7( 97.7)
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DMG |33.4000|116.3000|02/09/1890|12 6 0.0|   0.0| 6.30| 0.033 |  V | 60.8( 97.8)
DMG |32.9670|116.0000|10/21/1942|162519.0|   0.0| 5.00| 0.015 | IV | 61.6( 99.1)
DMG |32.9670|116.0000|10/21/1942|162213.0|   0.0| 6.50| 0.037 |  V | 61.6( 99.1)
DMG |32.9670|116.0000|10/22/1942|181326.0|   0.0| 5.00| 0.015 | IV | 61.6( 99.1)
DMG |32.9670|116.0000|10/21/1942|162654.0|   0.0| 5.00| 0.015 | IV | 61.6( 99.1)
DMG |33.1130|116.0370|04/09/1968| 3 353.5|   5.0| 5.20| 0.016 | IV | 62.5(100.6)
DMG |33.4000|116.2610|03/25/1937|1649 1.8|  10.0| 6.00| 0.027 |  V | 62.7(101.0)
DMG |32.9830|115.9830|05/23/1942|154729.0|   0.0| 5.00| 0.015 | IV | 62.8(101.0)
DMG |33.7100|116.9250|09/23/1963|144152.6|  16.5| 5.00| 0.014 | IV | 65.1(104.7)
DMG |31.8110|117.1310|12/22/1964|205433.2|   2.3| 5.60| 0.019 | IV | 66.6(107.2)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|   0.0| 6.00| 0.025 |  V | 67.4(108.5)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|   0.0| 5.00| 0.014 | III| 67.4(108.5)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|   0.0| 5.00| 0.014 | III| 67.4(108.5)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|   0.0| 5.00| 0.014 | III| 67.5(108.7)
DMG |33.7500|117.0000|04/21/1918|223225.0|   0.0| 6.80| 0.042 | VI | 67.5(108.7)
DMG |33.2310|116.0040|05/26/1957|155933.6|  15.1| 5.00| 0.013 | III| 67.6(108.8)
DMG |31.8670|116.5710|02/27/1937| 12918.4|  10.0| 5.00| 0.013 | III| 68.1(109.6)
DMG |33.6990|117.5110|05/31/1938| 83455.4|  10.0| 5.50| 0.017 | IV | 69.6(112.0)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|   0.0| 6.40| 0.030 |  V | 71.0(114.2)
PAS |33.0130|115.8390|11/24/1987|131556.5|   2.4| 6.00| 0.023 | IV | 71.4(114.8)
DMG |33.0000|115.8330|01/08/1946|185418.0|   0.0| 5.40| 0.016 | IV | 71.5(115.1)
DMG |33.0330|115.8210|09/30/1971|224611.3|   0.0| 5.10| 0.013 | III| 72.7(117.0)
DMG |33.1830|115.8500|04/25/1957|222412.0|   0.0| 5.10| 0.013 | III| 74.3(119.6)
PAS |33.0820|115.7750|11/24/1987| 15414.5|   4.9| 5.00| 0.019 | IV | 76.1(122.5)
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                              EARTHQUAKE SEARCH RESULTS
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    |        |         |          |  TIME  |      |     |       |    |
FILE|  LAT.  |  LONG.  |   DATE   |  (UTC) |DEPTH|QUAKE| SITE  |SITE|  APPROX.
CODE| NORTH  |  WEST   |          | H M Sec| (km)| MAG.|  ACC. | MM |  DISTANCE
    |        |         |          |        |     |     |   g   |INT.|  mi   [km]
----+--------+---------+----------+--------+-----+-----+-------+----+------------
DMG |32.8170|118.3500|12/26/1951| 04654.0|   0.0| 5.90| 0.020 | IV | 76.3(122.8)
DMG |32.9830|115.7330|01/24/1951| 717 2.6|   0.0| 5.60| 0.017 | IV | 76.9(123.8)
DMG |31.7500|116.5000|04/29/1935|20 8 0.0|   0.0| 5.00| 0.012 | III| 77.2(124.2)
DMG |33.2160|115.8000|04/25/1957|215738.7|  -0.3| 5.20| 0.013 | III| 77.4(124.6)
DMG |32.9500|115.7170|06/14/1953| 41729.9|   0.0| 5.50| 0.016 | IV | 77.5(124.7)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|   0.0| 5.20| 0.013 | III| 77.9(125.3)
DMG |32.9000|115.7800|10/02/1928|19 1 0.0|   0.0| 5.00| 0.012 | III| 78.0(125.5)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|   0.0| 5.00| 0.012 | III| 78.1(125.6)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|   0.0| 6.00| 0.021 | IV | 78.4(126.2)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|   0.0| 6.30| 0.025 |  V | 79.3(127.7)
DMG |31.7960|116.2690|06/11/1963|152338.3|  -2.0| 5.80| 0.018 | IV | 80.9(130.2)
DMG |33.6170|118.0170|03/14/1933|19 150.0|   0.0| 5.10| 0.012 | III| 81.3(130.9)
DMG |33.9500|116.8500|09/28/1946| 719 9.0|   0.0| 5.00| 0.011 | III| 82.0(132.0)
DMG |33.2330|115.7170|10/22/1942| 15038.0|   0.0| 5.50| 0.015 | IV | 82.7(133.1)
DMG |32.2500|115.7500|12/01/1958| 32118.0|   0.0| 5.80| 0.017 | IV | 83.1(133.7)
DMG |32.2500|115.7500|12/01/1958| 6 2 0.0|   0.0| 5.50| 0.014 | IV | 83.1(133.7)
DMG |32.2500|115.7500|12/01/1958| 350 0.0|   0.0| 5.00| 0.011 | III| 83.1(133.7)
PAS |33.0980|115.6320|04/26/1981|12 920.4|   3.8| 5.70| 0.016 | IV | 84.4(135.8)
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DMG |33.9760|116.7210|06/12/1944|104534.7| 10.0| 5.10| 0.011 | III| 85.1(136.9)
GSG |31.8060|116.1280|03/23/1994|025916.2| 22.0| 5.00| 0.011 | III| 85.2(137.1)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.022 | IV | 85.7(137.9)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.014 | IV | 85.9(138.3)
T-A |33.5000|115.8200|05/00/1868| 0 0 0.0| 10.0| 5.30| 0.012 | III| 86.4(139.0)
DMG |31.8000|116.1000|10/10/1953|1849 6.0|  0.0| 6.30| 0.023 | IV | 86.4(139.1)
PAS |32.9270|115.5400|10/16/1979| 54910.2| 10.4| 5.00| 0.010 | III| 86.5(139.3)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.011 | III| 87.4(140.7)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.011 | III| 87.4(140.7)
PAS |33.0140|115.5550|10/16/1979| 65842.0|  0.0| 5.10| 0.011 | III| 87.4(140.7)
PAS |32.9280|115.5390|10/16/1979| 61948.7|  9.1| 5.50| 0.014 | III| 87.5(140.8)
GSP |33.8760|116.2670|06/29/1992|160142.0|  9.2| 5.10| 0.011 | III| 87.5(140.8)
PAS |33.9980|116.6060|07/08/1986| 92044.5|  1.0| 5.20| 0.011 | III| 88.2(141.9)
DMG |33.1170|115.5670|07/29/1950|143632.0| 11.7| 5.60| 0.014 | IV | 88.2(141.9)
DMG |33.1170|115.5670|07/28/1950|175048.0|  0.0| 5.50| 0.014 | III| 88.4(142.2)
DMG |33.9330|116.3830|12/04/1948|234317.0|  0.0| 5.40| 0.013 | III| 88.4(142.2)
DMG |31.8330|116.0000|05/10/1956|114854.0|  0.0| 6.50| 0.025 |  V | 88.6(142.5)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 5.00| 0.010 | III| 88.7(142.7)
DMG |32.8000|115.5000|06/23/1915| 456 0.0|  0.0| 7.00| 0.036 |  V | 88.9(143.0)
DMG |32.8000|115.5000|06/23/1915| 359 0.0|  0.0| 6.25| 0.021 | IV | 89.2(143.5)
DMG |32.7330|115.5000|05/19/1940| 43640.9|  0.0| 6.25| 0.021 | IV | 89.2(143.5)
GSP |33.9020|116.2840|07/24/1992|181436.2|  0.0| 6.70| 0.029 |  V | 89.2(143.6)
MGI |32.7000|115.5000|01/01/1927|13 0 0.0|  9.0| 5.00| 0.010 | III| 89.3(143.6)
DMG |32.5000|118.5500|02/24/1948| 81510.0|  0.0| 5.30| 0.012 | III| 89.3(143.8)
DMG |32.7670|115.4830|05/19/1940| 455 0.0|  0.0| 5.30| 0.012 | III| 90.0(144.8)
DMG |32.7670|115.4830|05/19/1940| 63320.0|  0.0| 5.50| 0.013 | III| 90.2(145.1)
DMG |32.7670|115.4830|05/19/1940| 63540.0|  0.0| 5.00| 0.010 | III| 90.2(145.1)
DMG |32.7670|115.4830|05/19/1940| 55134.0|  0.0| 5.50| 0.013 | III| 90.2(145.1)
DMG |33.0000|115.5000|02/26/1930| 230 0.0|  0.0| 5.50| 0.013 | III| 90.2(145.1)
DMG |33.0000|115.5000|12/17/1955| 6 729.0|  0.0| 5.00| 0.010 | III| 90.4(145.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.40| 0.012 | III| 90.4(145.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.010 | III| 90.6(145.8)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.10| 0.010 | III| 90.6(145.8)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.30| 0.012 | III| 90.6(145.8)
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                    EARTHQUAKE SEARCH RESULTS
                    ----------------------------

Page  3
------------------------------------------------------------------------------
    |        |         |          |  TIME    |      |      |       |    |
FILE|  LAT.  |  LONG.  |   DATE   |  (UTC)   |DEPTH |QUAKE | SITE  |SITE|  APPROX.
CODE| NORTH  |  WEST   |          | H M Sec  | (km) | MAG. | ACC.  | MM |  DISTANCE
----+--------+---------+----------+----------+------+------+   g   |INT.|  mi  [km]
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.010 | III| 90.6(145.8)
DMG |32.5000|115.5000|09/08/1921|1924 0.0|  0.0| 5.00| 0.010 | III| 91.3(146.9)
DMG |32.5000|115.5000|01/01/1927| 81645.0|  0.0| 5.75| 0.015 | IV | 91.3(146.9)
DMG |32.5000|115.5000|01/01/1927| 91330.0|  0.0| 5.50| 0.013 | III| 91.3(146.9)
DMG |32.5000|115.5000|11/05/1923|22 7 0.0|  0.0| 5.00| 0.010 | III| 91.3(146.9)
DMG |32.5000|115.5000|05/01/1918| 432 0.0|  0.0| 5.00| 0.010 | III| 91.3(146.9)
DMG |32.5000|115.5000|11/07/1923|2357 0.0|  0.0| 5.50| 0.013 | III| 91.3(146.9)
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MGI |32.5000|115.5000|04/16/1925| 330 0.0|  0.0| 5.00| 0.010 | III| 91.3(146.9)
MGI |32.5000|115.5000|04/16/1925| 520 0.0|  0.0| 5.30| 0.012 | III| 91.3(146.9)
DMG |32.5000|115.5000|04/19/1906| 030 0.0|  0.0| 6.00| 0.018 | IV | 91.3(146.9)
DMG |34.0170|116.5000|07/24/1947|221046.0|  0.0| 5.50| 0.013 | III| 91.3(147.0)
DMG |34.0170|116.5000|07/25/1947| 61949.0|  0.0| 5.20| 0.011 | III| 91.3(147.0)
DMG |34.0170|116.5000|07/26/1947| 24941.0|  0.0| 5.10| 0.010 | III| 91.3(147.0)
DMG |34.0170|116.5000|07/25/1947| 04631.0|  0.0| 5.00| 0.010 | III| 91.3(147.0)
GSP |33.9610|116.3180|04/23/1992|045023.0| 12.0| 6.10| 0.019 | IV | 91.9(147.9)
PAS |32.7660|115.4410|10/15/1979|231930.0|  9.3| 5.20| 0.011 | III| 92.6(149.0)
DMG |34.1000|116.8000|10/24/1935|1448 7.6|  0.0| 5.10| 0.010 | III| 92.7(149.1)
DMG |31.6250|116.2110|06/10/1969| 34132.7| -2.0| 5.00| 0.010 | III| 92.7(149.2)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.011 | III| 92.9(149.5)
DMG |33.1670|115.5000|12/20/1935| 745 0.0|  0.0| 5.00| 0.010 | III| 93.1(149.8)
DMG |31.5000|116.5000|10/17/1954|225718.0|  0.0| 5.70| 0.014 | IV | 93.3(150.1)
PAS |31.8900|115.8210|05/08/1985|234020.0|  6.0| 5.00| 0.010 | III| 93.5(150.4)
DMG |34.1000|116.7000|02/07/1889| 520 0.0|  0.0| 5.30| 0.011 | III| 93.7(150.8)
PAS |31.9270|115.7770|07/17/1975|182447.0| 17.3| 5.00| 0.010 | III| 93.8(150.9)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.012 | III| 94.2(151.6)
GSP |34.0290|116.3210|08/21/1993|014638.4|  9.0| 5.00| 0.009 | III| 96.1(154.6)
DMG |31.7500|115.9170|02/09/1956|165953.0|  0.0| 5.70| 0.014 | IV | 96.2(154.8)
DMG |31.7500|115.9170|03/09/1956| 03240.0|  0.0| 5.00| 0.009 | III| 96.2(154.8)
DMG |31.7500|115.9170|02/09/1956|143230.0|  0.0| 6.00| 0.029 |  V | 96.2(154.8)
DMG |31.7500|115.9170|02/11/1956| 25746.0|  0.0| 5.10| 0.010 | III| 96.2(154.8)
DMG |31.7500|115.9170|02/11/1956| 61124.0|  0.0| 5.00| 0.009 | III| 96.2(154.8)
DMG |31.7500|115.9170|02/09/1956|152426.0|  0.0| 6.10| 0.018 | IV | 96.2(154.8)
DMG |31.7500|115.9170|02/10/1956|181254.0|  0.0| 5.50| 0.012 | III| 96.2(154.8)
DMG |31.7500|115.9170|02/10/1956|15 929.0|  0.0| 5.00| 0.009 | III| 96.2(154.8)
DMG |31.7500|115.9170|02/09/1956|184845.0|  0.0| 5.70| 0.014 | IV | 96.2(154.8)
DMG |32.2500|115.5000|12/30/1934|1352 0.0|  0.0| 6.50| 0.023 | IV | 96.4(155.2)
GSP |34.1630|116.8550|06/28/1992|144321.0|  6.0| 5.30| 0.011 | III| 96.6(155.4)
GSP |34.0640|116.3610|09/15/1992|084711.3|  9.0| 5.20| 0.010 | III| 97.3(156.6)
DMG |34.1800|116.9200|01/16/1930| 034 3.6|  0.0| 5.10| 0.010 | III| 97.4(156.8)
DMG |34.1800|116.9200|01/16/1930| 02433.9|  0.0| 5.20| 0.010 | III| 97.4(156.8)
DMG |31.6000|116.1000|11/26/1955|1736 0.0|  0.0| 5.40| 0.012 | III| 97.7(157.2)
DMG |34.0670|116.3330|05/18/1940| 55120.2|  0.0| 5.20| 0.010 | III| 98.1(157.9)
DMG |34.0670|116.3330|05/18/1940| 72132.7|  0.0| 5.00| 0.009 | III| 98.1(157.9)
DMG |34.2000|117.1000|09/20/1907| 154 0.0|  0.0| 6.00| 0.016 | IV | 98.6(158.8)
GSP |34.1950|116.8620|08/17/1992|204152.1| 11.0| 5.30| 0.011 | III| 98.7(158.9)
GSP |31.7030|115.9100|12/03/1991|175435.0|  5.0| 5.30| 0.011 | III| 98.9(159.1)
DMG |33.7830|118.2500|11/14/1941| 04136.3|  0.0| 5.40| 0.011 | III| 98.9(159.1)
GSP |34.1000|116.4040|06/29/1992|141330.0|  9.0| 5.40| 0.011 | III| 99.2(159.6)
DMG |31.7000|115.9000|02/11/1956| 519 0.0|  0.0| 5.00| 0.009 | III| 99.4(160.0)
DMG |31.7000|115.9000|02/09/1956|1434 0.0|  0.0| 5.60| 0.013 | III| 99.4(160.0)
GSN |34.2030|116.8270|06/28/1992|150530.7|  5.0| 6.70| 0.026 |  V | 99.5(160.2)
DMG |34.0830|116.3000|05/18/1940| 5 350.5|  0.0| 5.40| 0.011 | III| 99.9(160.8)

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-END OF SEARCH-   158 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2003

LENGTH OF SEARCH TIME:   204  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 4.2 MILES (6.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.166 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.567
  b-value=  0.382
  beta-value=  0.879

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TABLE OF MAGNITUDES AND EXCEEDANCES:
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  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+-----------
     4.0     |      158        |  0.77451
     4.5     |      158        |  0.77451
     5.0     |      158        |  0.77451
     5.5     |       62        |  0.30392
     6.0     |       28        |  0.13725
     6.5     |       10        |  0.04902
     7.0     |        1        |  0.00490
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APPENDIX D

MODIFIED MERCALLI INTENSITY INDEX

APPENDIX D
MODIFIED MERCALLI INTENSITY SCALE OF 1931
(Excerpted from the California Division of Conservation Division of Mines and Geology DMG Note 32)

The first scale to reflect earthquake intensities was developed by deRossi of Italy, and Forel of Switzerland, in the 1880s, and is known as the Rossi-Forel Scale. This scale, with values from I to X, was used for about two decades. A need for a more refined scale increased with the advancement of the science of seismology, and in 1902, the Italian seismologist Mercalli devised a new scale on a I to XII range. The Mercalli Scale was modified in 1931 by American seismologists Harry O. Wood and Frank Neumann to take into account modern structural features.

The Modified Mercalli Intensity Scale measures the intensity of an earthquake's effects in a given locality, and is perhaps much more meaningful to the layman because it is based on actual observations of earthquake effects at specific places. It should be noted that because the damage used for assigning intensities can be obtained only from direct firsthand reports, considerable time -- weeks or months -- is sometimes needed before an intensity map can be assembled for a particular earthquake.

On the Modified Mercalli Intensity Scale, values range from I to XII. The most commonly used adaptation covers the range of intensity from the conditions of *"I -- not felt except by very few, favorably situated,"* to *"XII -- damage total, lines of sight disturbed, objects thrown into the air."* While an earthquake has only one magnitude, it can have many intensities, which decrease with distance from the epicenter.

It is difficult to compare magnitude and intensity because intensity is linked with the particular ground and structural conditions of a given area, as well as distance from the earthquake epicenter, while magnitude depends on the energy released at the focus of the earthquake.

I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in building of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI	Few, if any, masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Practically all works of construction are damaged greatly or destroyed. Waves seen on ground surface. Lines of sight and level are distorted. Objects thrown upward into the air.

APPENDIX E

GENERAL EARTHWORK SPECIFICATIONS

APPENDIX E
GENERAL EARTHWORK SPECIFICATIONS

General

The objective of these specifications is to properly establish procedures for the clearing and preparation of the existing natural ground or properly compacted fill to receive new fill; for the selection of the fill material; and for the fill compaction and testing methods to be used.

Scope of Work

The earthwork includes all the activities and resources provided by the contractor to construct in a good workmanlike manner all the grades of the filled areas shown in the plans. The major items of work covered in this section include all clearing and grubbing, removing and disposing of materials, preparing areas to be filled, compacting of fill, compacting of backfills, subdrain installations, and all other work necessary to complete the grading of the filled areas.

Site Visit and Site Investigation

1. The contractor shall visit the site and carefully study it, and make all inspections necessary in order to determine the full extent of the work required to complete all grading in conformance with the drawings and specifications. The contractor shall satisfy himself as to the nature, location, and extent of the work conditions, the conformation and condition of the existing ground surface; and the type of equipment, labor, and facilities needed prior to and during prosecution of the work. The contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications, must be brought to the engineer's attention in order to clarify the exact nature of the work to be performed.

2. A soils investigation report has been prepared for this project by GEI. It is available for review and should be used as a reference to the surface and subsurface soil and bedrock conditions on this project. Any recommendations made in the report of the soil investigation or subsequent reports shall become an addendum to these specifications.

Authority of the Soils Engineer and Engineering Geologist

The soils engineer shall be the owner's representative to observe and test the construction of fills. Excavation and the placing of fill shall be under the observation of the soils engineer and his/her representative, and he/she shall give a written opinion regarding conformance with the specifications upon completion of grading. The soils engineer shall have the authority to cause the removal and replacement of porous topsoils, uncompacted or improperly compacted fills, disturbed bedrock materials, and soft alluvium, and shall have the authority to approve or reject materials proposed for use in the compacted fill areas.

The soils engineer shall have, in conjunction with the engineering geologist, the authority to approve the preparation of natural ground and toe-of-fill benches to receive fill material. The engineering geologist shall have the authority to evaluate the stability of the existing or proposed slopes, and to evaluate the necessity of remedial measures. If any unstable condition is being created by cutting or filling, the engineering geologist and/or soils engineer shall advise the contractor and owner immediately, and prohibit grading in the affected area until such time as corrective measures are taken.

The owner shall decide all questions regarding: (1) the interpretation of the drawings and specifications, (2) the acceptable fulfillment of the contract on the part of the contractor, and (3) the matter of compensation.

Clearing and Grubbing

1. Clearing and grubbing shall consist of the removal from all areas to be graded of all surface trash, abandoned improvements, paving, culverts, pipe, and vegetation (including -- but not limited to -- heavy weed growth, trees, stumps, logs and roots larger than 1-inch in diameter).

2. All organic and inorganic materials resulting from the clearing and grubbing operations shall be collected, piled, and disposed of by the contractor to give the cleared areas a neat and finished appearance. Burning of combustible materials on-site shall not be permitted unless allowed by local regulations, and at such times and in such a manner to prevent the fire from spreading to areas adjoining the property or cleared area.

3. It is understood that minor amounts of organic materials may remain in the fill soils due to the near impossibility of complete removal. The amount remaining, however, must be considered negligible, and in no case can be allowed to occur in concentrations or total quantities sufficient to contribute to settlement upon decomposition.

Preparation of Areas to be Filled

1. After clearing and grubbing, all uncompacted or improperly compacted fills, soft or loose soils, or unsuitable materials, shall be removed to expose competent natural ground, undisturbed bedrock, or properly compacted fill as indicated in the soils investigation report or by our field representative. Where the unsuitable materials are exposed in final graded areas, they shall be removed and replaced as compacted fill.

2. The ground surface exposed after removal of unsuitable soils shall be scarified to a depth of at least 6 inches, brought to the specified moisture content, and then the scarified ground compacted to at least the specified density. Where undisturbed bedrock is exposed at the surface, scarification and recompaction shall not be required.

3. All areas to receive compacted fill, including all removal areas and toe-of-fill benches, shall be observed and approved by the soils engineer and/or engineering geologist prior to placing compacted fill.

4. Where fills are made on hillsides or exposed slope areas with gradients greater than 20 percent, horizontal benches shall be cut into firm, undisturbed, natural ground in order to provide both lateral and vertical stability. This is to provide a horizontal base so that each layer is placed and compacted on a horizontal plane. The initial bench at the toe of the fill shall be at least 10 feet in width on firm, undisturbed, natural ground at the elevation of the toe stake placed at the bottom of the design slope. The engineer shall determine the width and frequency of all succeeding benches, which will vary with the soil conditions and the steepness of the slope. Ground slopes flatter than 20 percent (5.0:1.0) shall be benched when considered necessary by the soils engineer.

Fill and Backfill Material

Unless otherwise specified, the on-site material obtained from the project excavations may be used as fill or backfill, provided that all organic material, rubbish, debris, and other objectionable material contained therein is first removed. In the event that expansive materials are encountered during foundation excavations within 3 feet of finished grade and they have not been properly processed, they shall be entirely removed or thoroughly mixed with good, granular material before incorporating them in fills. No footing shall be allowed to bear on soils which, in the opinion of the soils engineer, are detrimentally expansive -- unless designed for this clayey condition.

However, rocks, boulders, broken Portland cement concrete, and bituminous-type pavement obtained from the project excavations may be permitted in the backfill or fill with the following limitations:

1. The maximum dimension of any piece used in the top 10 feet shall be no larger than 6 inches.

2 Clods or hard lumps of earth of 6 inches in greatest dimension shall be broken up before compacting the material in fill.

3. If the fill material originating from the project excavation contains large rocks, boulders, or hard lumps that cannot be broken readily, pieces ranging from 6 inches in diameter to 2 feet in maximum dimension may be used in fills below final subgrade if all pieces are placed in such a manner (such as windrows) as to eliminate nesting or voids between them. No rocks over 4 feet will be allowed in the fill.

4. Pieces larger than 6 inches shall not be placed within 12 inches of any structure.

5. Pieces larger than 3 inches shall not be placed within 12 inches of the subgrade for paving.

6. Rockfills containing less than 40 percent of soil passing 3/4-inch sieve may be permitted in designated areas. Specific recommendations shall be made by the soils engineer and be subject to approval by the city engineer.

7. Continuous observation by the soils engineer is required during rock placement.

8. Special and/or additional recommendations may be provided in writing by the soils engineer to modify, clarify, or amplify these specifications.

9. During grading operations, soil types other than those analyzed in the soil investigation report may be encountered by the contractor. The soils engineer shall be consulted to evaluate the suitability of these soils as fill materials.

Placing and Compacting Fill Material

1. After preparing the areas to be filled, the approved fill material shall be placed in approximately horizontal layers, with lift thickness compatible to the material being placed and the type of equipment being used. Unless otherwise approved by the soils engineer, each layer spread for compaction shall not exceed 8 inches of loose thickness. Adequate drainage of the fill shall be provided at all times during the construction period.

2. When the moisture content of the fill material is below that specified by the engineer, water shall be added to it until the moisture content is as specified.

3. When the moisture content of the fill material is above that specified by the engineer, resulting in inadequate compaction or unstable fill, the fill material shall be aerated by blading and scarifying or other satisfactory methods until the moisture content is as specified.

4. After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than the density set forth in the specifications. Compaction shall be accomplished with sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other approved types of acceptable compaction equipment. Equipment shall be of such design that it will be able to compact the fill to the specified relative compaction. Compaction shall cover the entire fill area, and the equipment shall make sufficient trips to ensure that the desired density has been obtained throughout the entire fill. At locations where it would be impractical due

to inaccessibility of rolling compacting equipment, fill layers shall be compacted to the specified requirements by hand-directed compaction equipment.

5. When soil types or combination of soil types are encountered which tend to develop densely packed surfaces as a result of spreading or compacting operations, the surface of each layer of fill shall be sufficiently roughened after compaction to ensure bond to the succeeding layer.

6. Unless otherwise specified, fill slopes shall not be steeper than 2.0 horizontal to 1.0 vertical. In general, fill slopes shall be finished in conformance with the lines and grades shown on the plans. The surface of fill slopes shall be overfilled to a distance from finished slopes such that it will allow compaction equipment to operate freely within the zone of the finished slope, and then cut back to the finished grade to expose the compacted core. Alternate compaction procedures include the backrolling of slopes with sheepsfoot rollers in increments of 3 to 5 feet in elevation gain. Alternate methods may be used by the contractor, but they shall be evaluated for approval by the soils engineer.

7. Unless otherwise specified, all allowed expansive fill material shall be compacted to a moisture content of approximately 2 to 4 percent above the optimum moisture content. Nonexpansive fill shall be compacted at near-optimum moisture content. All fill shall be compacted, unless otherwise specified, to a relative compaction not less than 95 percent for fill in the upper 12 inches of subgrades under areas to be paved with asphalt concrete or Portland concrete, and not less than 90 percent for other fill. The relative compaction is the ratio of the dry unit weight of the compacted fill to the laboratory maximum dry unit weight of a sample of the same soil, obtained in accordance with A.S.T.M. D-1557 test method.

8. The observation and periodic testing by the soils engineer are intended to provide the contractor with an ongoing measure of the quality of the fill compaction operation. It is the responsibility of the grading contractor to utilize this information to establish the degrees of compactive effort required on the project. More importantly, it is the responsibility of the grading contractor to ensure that proper compactive effort is applied at all times during the grading operation, including during the absence of soils engineering representatives.

Trench Backfill

1. Trench excavations which extend under graded lots, paved areas, areas under the influence of structural loading, in slopes or close to slope areas, shall be backfilled under the observations and testing of the soils engineer. All trenches not falling within the aforementioned locations shall be backfilled in accordance with the City or County regulating agency specifications.

2. Unless otherwise specified, the minimum degree of compaction shall be 90 percent of the laboratory maximum dry density.

3. Any soft, spongy, unstable, or other similar material encountered in the trench excavation upon which the bedding material or pipe is to be placed, shall be removed to a depth recommended by the soils engineer and replaced with bedding materials suitably densified.

   Bedding material shall first be placed so that the pipe is supported for the full length of the barrel with full bearing on the bottom segment. After the needed testing of the pipe is accomplished, the bedding shall be completed to at least 1 foot on top of the pipe. The bedding shall be properly densified before backfill is placed. Bedding shall consist of granular material with a sand equivalent not less than 30, or other material approved by the engineer.

4. No rocks greater than 6 inches in diameter will be allowed in the backfill placed between 1 foot above the pipe and 1 foot below finished subgrade. Rocks greater than 2.5 inches in any dimension will not be allowed in the backfill placed within 1 foot of pavement subgrade.

5. Material for mechanically compacted backfill shall be placed in lifts of horizontal layers and properly moistened prior to compaction. In addition, the layers shall have a thickness compatible with the material being placed and the type of equipment being used. Each layer shall be evenly spread, moistened or dried, and then tamped or rolled until the specified relative compaction has been attained.

6. Backfill shall be mechanically compacted by means of tamping rollers, sheepsfoot rollers, pneumatic tire rollers, vibratory rollers, or other mechanical tampers. Impact-type pavement breakers (stompers) will not be permitted over clay, asbestos cement, plastic, cast iron, or nonreinforced concrete pipe. Permission to use specific compaction equipment shall not be construed as guaranteeing or implying that the use of such equipment will not result in damage to adjacent ground, existing improvements, or improvements installed under the contract. The contractor shall make his/her own determination in this regard.

7. Jetting shall not be permitted as a compaction method unless the soils engineer allows it in writing.

8. Clean granular material shall not be used as backfill or bedding in trenches located in slope areas or within a distance of 10 feet of the top of slopes unless provisions are made for a drainage system to mitigate the potential buildup of seepage forces into the slope mass.

Observations and Testing

1. The soils engineers or their representatives shall sufficiently observe and test the grading operations so that they can state their opinion as to whether or not the fill was constructed in accordance with the specifications.

2. The soils engineers or their representatives shall take sufficient density tests during the placement of compacted fill. The contractor should assist the soils engineer and/or his/her representative by digging test pits for removal determinations and/or for testing compacted fill. In addition, the contractor should cooperate with the soils engineer by removing or shutting down equipment from the area being tested.

3. Fill shall be tested for compliance with the recommended relative compaction and moisture conditions. Field density testing should be performed by using approved methods by A.S.T.M., such as A.S.T.M. D1556, D2922, and/or D2937. Tests to evaluate density of compacted fill should be provided on the basis of not less than one test for each 2-foot vertical lift of the fill, but not less than one test for each 1,000 cubic yards of fill placed. Actual test intervals may vary as field conditions dictate. In fill slopes, approximately half of the tests shall be made at the fill slope, except that not more than one test needs to be made for each 50 horizontal feet of slope in each 2-foot vertical lift. Actual test intervals may vary as field conditions dictate.

4. Fill found not to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the soils engineer.

Site Protection

It shall be the grading contractor's obligation to take all measures deemed necessary during grading to maintain adequate safety measures and working conditions, and to provide erosion-control devices for the protection of excavated areas, slope areas, finished work on the site and adjoining properties, from storm damage and flood hazard originating on the project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with the job specifications, all berms and benches have been properly constructed, and all associated drainage devices have been installed and meet the requirements of the specifications.

All observations, testing services, and approvals given by the soils engineer and/or geologist shall not relieve the contractor of his/her responsibilities of performing the work in accordance with these specifications.

After grading is completed and the soils engineer has finished his/her observations and/or testing of the work, no further excavation or filling shall be done except under his/her observations.

Adverse Weather Conditions

1. Precautions shall be taken by the contractor during the performance of site clearing, excavations, and grading to protect the worksite from flooding, ponding, or inundation by poor or improper surface drainage. Temporary provisions shall be made during the rainy season to adequately direct surface drainage away from and off the worksite. Where low areas cannot be avoided, pumps should be kept on hand to continually remove water during periods of rainfall.

2. During periods of rainfall, plastic sheeting shall be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the contractor shall install checkdams, desilting basins, rip-rap, sandbags, or other devices or methods necessary to control erosion and provide safe conditions.

3. During periods of rainfall, the soils engineer should be kept informed by the contractor as to the nature of remedial or preventative work being performed (e.g. pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).

4. Following periods of rainfall, the contractor shall contact the soils engineer and arrange a walk-over of the site in order to visually assess rain-related damage. The soils engineer may also recommend excavations and testing in order to aid in his/her assessments. At the request of the soils engineer, the contractor shall make excavations in order to evaluate the extent of rain-related damage.

5. Rain-related damage shall be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress, and other adverse conditions identified by the soils engineer. Soil adversely affected shall be classified as Unsuitable Materials, and shall be subject to overexcavation and replacement with compacted fill or other remedial grading, as recommended by the soils engineer.

6. Relatively level areas, where saturated soils and/or erosion gullies exist to depths of greater than 1.0 foot, shall be overexcavated to unaffected, competent material. Where less than 1.0 foot in depth, unsuitable materials may be processed in place to achieve near-optimum moisture conditions, then thoroughly recompacted in accordance with the applicable specifications. If the desired results are not achieved, the affected materials shall be over-excavated, then replaced in accordance with the applicable specifications.

7. In slope areas, where saturated soils and/or erosion gullies exist to depths of greater than 1.0 foot, they shall be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where affected materials exist to depths of 1.0 foot or less below proposed finished grade, remedial grading by moisture-conditioning in place, followed by thorough recompaction in accordance with the applicable grading guidelines herein presented may be attempted. If materials shall be overexcavated and replaced as compacted fill, it shall be done in accordance with the slope-repair recommendations herein. As field conditions dictate, other slope-repair procedures may be recommended by the soils engineer.