SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

June 6, 2007

Red Mountain Retail Group, Inc.
1234 East 17th Street
Santa Ana, California 92701

Project No.: 07G154-1

Subject: **Geotechnical Investigation**
Proposed Mixed Use Development
Proposed Walgreens and Residential Development
SWC Harbor Boulevard and Hamilton Street
Costa Mesa, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Site Preparation

- It is expected that all of the existing structures and associated above- and below-ground improvements will be demolished prior to redevelopment of the subject site. All foundations, floor slabs, utilities, and other subsurface elements should be removed in their entirety and properly disposed of off-site.
- Portions of the subject site are underlain by 3 to 8± feet of undocumented fill and possible fill soils and native alluvial soils. The fill, possible fill and near-surface alluvial soils possess somewhat variable strengths and consolidation/collapse characteristics. No documentation regarding placement or compaction of the fill soils is available, and they are therefore considered to represent undocumented fill. It is recommended that remedial grading be performed within the proposed building areas in order to remove the existing fill soils and portions of the variable strength native soils.
- The existing soils within the proposed building pad areas should be overexcavated to a depth of 3 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation. The foundation influence zones should be overexcavated to a depth of 3 feet below the proposed building pad subgrade elevation.
- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned as necessary, and recompacted. The resulting soils may be replaced as compacted structural fill.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slabs

- Conventional Slab-on-Grade, 4½ inches thick.
- Reinforcement consisting of at least No. 3 bars at 18 inches on center, in both directions.
- The actual floor slab reinforcement to be determined by the structural engineer. Additional reinforcement may be necessary for structural considerations.

Pavements

ASPHALT PAVEMENTS				
Materials	**Thickness (inches)**			
	Parking Stalls (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)
Asphalt Concrete	3	3	3½	4
Aggregate Base	3	6	7½	10
Compacted Subgrade	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Automobile Parking and Drive Areas (TI=5.0)	Light Truck Traffic Areas (TI =6.0)	Moderate Truck Traffic Areas (TI =7.0)
PCC	5	5½	6½
Compacted Subgrade (95% minimum compaction)	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 07P201, dated April 16, 2007. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations and building floor slabs along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southwest corner of Harbor Boulevard and Hamilton Avenue in Costa Mesa, California. The site is bordered to the north by Hamilton Avenue, to the west by Charle Street and by adjacent properties, to the south by adjacent retail/commercial properties, and to the east by Harbor Boulevard. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The subject site consists of two adjacent previously developed parcels. These two parcels are irregularly-shaped, and together comprise an area of approximately 1.5± acres. The subject site is located within an area of existing mixed commercial/retail and residential development in Costa Mesa, California. The northeastern portion of the site is developed with one (1) two-story dental office building and two (2) one-story auto repair buildings. It appeared that these structures were of wood frame construction. The building footprints of these existing structures range from 1,000± to 6,000± ft^2.

Ground surface cover consists of exposed soil with negligible grass and weed growth, asphaltic concrete, or Portland concrete cement areas. Isolated stockpiles of construction debris including concrete soil, asphalt and steel pipes were observed in several areas of the site. Several steel drums with undetermined contents were observed in the southern portion of the site. Remnants of buried fluid lines from the previous auto shop were observed in several portions of the site. Several groundwater monitoring wells are also observed on the property. Several trailers and other vehicles were present on the site during our subsurface exploration. In addition, several stockpiles of soil, up to approximately 6± feet in height were observed in the southern portion of the site.

Detailed topographic information was not available at the time of this report. Visually, site topography is relatively level, with no obvious pattern of surface drainage. Visually, site topography slopes slightly downward to the west, at an estimated gradient of 1± percent. There was estimated to be less than 2± feet of elevation differential across the site.

3.2 Proposed Development

The preliminary site plans for the proposed development was obtained from the client. Based on these plans, the eastern portion of the site will be occupied with a new Walgreens store, approximately 10,900± ft^2 in size. It is anticipated that the Walgreens drugstore building will be a one-story structure of tilt-up concrete construction with limited areas of mezzanine construction and a truck dock areas. The western portion of the site will be developed with two (2) buildings, of wood-frame multi-story construction, comprised of seven (7) residential units each. All buildings are expected to be surrounded by asphaltic concrete pavements. Some areas of Portland cement concrete pavements are expected to be constructed in the loading dock areas of the Walgreens site.

Detailed structural information is not currently available. However, based on the assumed construction, maximum column and wall loads for the proposed Walgreens are expected to be on the order of 90 kips and 3 to 4 kips per linear foot, respectively. Based on the assumed construction of the residential structures, these wood frame buildings are expected to generate maximum column loads of 30 to 40 kips and 1 to 2 kips per linear foot, respectively. With the exception of some small retaining walls in the area of the truck loading docks, the proposed structures are not expected to incorporate any significant below grade construction.

Preliminary grading plans were not available at the time of the geotechnical investigation. Based on the existing topography, and assuming a relatively balanced site, cuts and fills on the order of 1 to 2± feet are expected to be necessary to achieve the proposed site grades within the building areas. Retaining walls of up to 4 to 5± feet in height are also expected to be necessary in the areas of the new truck loading docks.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of ten (10) borings advanced to depths of 5 to 25± feet below currently existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. Representative bulk and in-situ soil samples were taken during drilling. Relatively undisturbed in-situ samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Borings B-3 and B-4 were drilled through the existing pavements. At these boring locations, pavements consist of 4 to 5 inches of asphaltic concrete underlain by 0 to 6 inches of underlying aggregate base.

Artificial Fill

Artificial fill soils were encountered at the ground surface, at most of the boring locations. These fill soils extend to depths of 3 to 6½± feet below existing grade. These fill soils generally consist of loose to very dense silty fine sands and clayey sands with occasional asphalt debris. The fill soils possess variable strengths, a disturbed appearance and moderate debris content, resulting in their classification as fill. Additional soils classified as possible fill were encountered at Borings B-3 and B-4, extending from ground surface to depths of 4½ to 5½± feet and at Boring B-2 extending from beneath the fill soils to a depth of 8± feet. These possible fill soils possess some indicators of fill but also resemble the underlying native soils.

Alluvium

Native soils were encountered at all of the boring locations at the site and consist of sands, silty sands, and clayey sands extending to the maximum explored depth of 25± feet. These soils were generally medium dense to dense with occasional very dense soils. A layer of very stiff sandy clays ranging in thickness from 2 to 3½ feet, was observed at several of the borings. Based on in-situ moisture contents, these soils were generally damp to moist. Several samples obtained at depths ranging from 5½ to 25± feet possessed a strong hydrocarbon odor.

Groundwater

Very moist to wet soils were encountered during drilling of the deepest borings at depths of 18½ to 25± feet. Delayed readings taken within the open boreholes identified free water at depths of 18 to 18½± feet. Based on the water level measurements, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 18 to 18½± feet at the time of the subsurface exploration.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

In-situ Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-6 in Appendix C of this report.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**UBC Classification**
B-2 @ 0 to 5 feet	<0.001	Negligible
B-7 @ 0 to 5 feet	0.013	Negligible

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with Uniform Building Code (UBC) Standard 18-2. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	**Expansion Index**	**Expansive Potential**
B-4 @ 0 to 5 feet	1	Very Low
B-6 @ 0 to 5 feet	6	Very Low

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample taken from the subject site has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plate C-7 in Appendix C of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The completion of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The proposed development must be designed in accordance with the requirements of the latest edition of the Uniform Building Code (UBC) and/or the California Building Code (CBC). The UBC and CBC provide procedures for earthquake resistant structural design that include considerations for on-site soil conditions, seismic zoning, occupancy, and the configuration of the structures including the structural system and height. The seismic design parameters presented below are based on the seismic zone, soil profile, and the proximity of known faults with respect to the subject site.

The 1997 UBC and 2001 CBC Design Parameters have been generated using UBCSEIS, a computer program published by Thomas F. Blake (January 1998). The table below is a compilation of the data provided by UBCSEIS, and represents the largest design values presented by each type of fault. A copy of the output generated from this program is included in Appendix E of this report. A copy of the Design Response Spectrum, as generated by UBCSEIS is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

- Nearest Type A Fault: Cucamonga (54 km)
- Nearest Type B Fault: Newport-Inglewood (4.8 km)
- Soil Profile Type: S_D
- Seismic Zone Factor (Z): 0.40
- Seismic Coefficient (C_a): 0.45
- Seismic Coefficient (C_v): 0.78
- Near-Source Factor (N_a) 1.0
- Near-Source Factor (N_v) 1.2

The design procedures presented by the UBC and CBC are intended to protect life safety. Structures designed using these minimum design procedures may experience significant cosmetic damage and serious economic loss. The use of more conservative seismic design parameters would provide increased safety and a lower potential for cosmetic damage and economic loss during a large seismic event. Ultimately, the structural engineer and the project owner must determine what level of risk is acceptable and assign appropriate seismic values to be used in the design of the proposed structures.

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The subsurface conditions encountered at the subject site are not conducive to liquefaction. These conditions generally consist of dense to very dense granular soils, which are not susceptible to earthquake-induced liquefaction. Furthermore, the site is not located within a designated liquefaction zone as designated by the California Geological Survey (CGS). Based on these considerations, liquefaction is not considered to be a significant design concern for this project.

6.2 Geotechnical Design Considerations

General

The subject site is generally underlain by fill and possible fill soils, extending to depths of 3 to 8± feet, underlain by native alluvial soils. These fill soils were likely placed during previous development of the subject site. No data regarding the placement or compaction of these fill soils is currently available. These materials are not considered suitable for support of the new structure. These fill materials possess variable strengths and consolidation/collapse characteristics. Based on these factors, the fill materials are considered to represent undocumented fill. Based on these considerations, remedial grading is warranted within the proposed building area in order to remove and replace the near surface soils as compacted structural fill. Within the footprint of the proposed buildings, we recommend that additional grading be performed to remove all fill and possible fill soils.

The demolition of the existing structures and surrounding above- and below-ground improvements will result in significant disturbance of the near surface soils. Based on these considerations, remedial grading is considered warranted within the proposed building areas to remove these soils and replace them as compacted structural fill.

Several soil samples taken from the subject site exhibited strong hydrocarbon odors. We recommend that an environmental consultant experienced with these conditions be retained to review the site conditions to determine if remedial efforts are required. Any excavations created as a result of environmental remediation should be backfilled with compacted structural fill. Southern California Geotechnical should be provided with copies of any reports documenting future environmental remediation activities to determine their impact on the recommendations presented in this report.

Settlement

The proposed remedial grading will remove and replace the existing fill and possible fill soils. The underlying soils are not considered to be significantly compressible when exposed to load increases in the range of those that will be exerted by the foundations of the new structures. Therefore, following completion of the recommended grading, the post-construction settlements are expected to be within tolerable limits.

Expansion

Laboratory testing performed on a representative sample of the near surface soils indicates that these materials possess a very low expansion potential (EI's = 1 and 6). The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pad.

Grading and Foundation Plan Review

Detailed grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

It is our understanding from conversations with the client, that the existing buildings and associated above- and below-ground improvements are planned to be demolished. Following demolition of the existing buildings, all subsurface remnants, including foundations, floor slabs, lifts, utilities, etc., should be removed in their entirety. All existing structures and associated improvements should be demolished and the resulting debris disposed of in accordance with all applicable local, state and federal requirements. A representative of the geotechnical engineer should be present during the concluding stages of demolition, to verify adequate removals, and to monitor backfill of any excavations resultant from demolition activities. The existing concrete flatwork may be removed from the site, or alternatively pulverized to a maximum 2-inch particle size for later use as structural fill. It is our understanding that the existing stockpiles of soil and debris will be properly disposed of off-site.

Initial site preparation should include stripping of any vegetation on the site. No significant vegetation or topsoil was encountered at the boring locations. However, topsoil may be present at other locations on the site. Any such materials should be disposed of off-site, or in nonstructural areas of the property. The actual extent of stripping should be determined in the field by a representative of the geotechnical engineer, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Proposed Buildings

Remedial grading should be performed within the proposed buildings in order to remove the unsuitable, variable strength, existing fill and possible fill soils. Based on conditions encountered at the boring locations, these materials extend to depths of 3 to 8± feet. In order to provide a relatively uniform support condition for the new structures, it is recommended that the existing soils within the proposed buildings be overexcavated to a depth of 3 feet below existing grade and to a depth of 3 feet below proposed building pad subgrade elevation. The depth of overexcavation should also be sufficient to provide at least 3 feet of newly placed compacted structural fill below the bearing grade of all foundations.

The overexcavation areas should extend at least 5 feet beyond the building perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. This evaluation should include probing and proofrolling to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if dry, loose, porous, low density or otherwise unsuitable materials are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture treated to 2 to 4 percent above optimum moisture content, and compacted. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads. The subgrade soils within any areas of non-retaining site walls should be overexcavated to a depth of 1 foot below proposed foundation bearing grade. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork, Driveway, Parking and Driveway Areas

Based on economic considerations, overexcavation of the existing soils in flatwork, driveway, parking and driveway areas is not considered warranted, with the exception of areas where lower strength, or unstable soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength fill soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, driveway, parking and driveway areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not completely mitigate the extent of undocumented fill soils in these areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork, driveway, parking and driveway areas should be graded in a manner similar to that described for the building areas.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 1997 UBC/2001 CBC and the grading code of the City of Costa Mesa.
- Fill soils comprising the basement floor subgrade should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- All other fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of non to very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the City of Costa Mesa. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of sands, silty sands and clayey sands. These materials will be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

As discussed previously, possible environmental issues related to excavation exist at the site. These issues include but may not be limited to the hydrocarbon odors observed during drilling as well as the likely removal of existing above- and below-ground improvements such as lifts, pipes, tanks, etc., related to the on-site auto repair facility. We recommend that an environmental consultant be retained to review the site conditions and provide recommendations pertaining to environmental remediation activities for the site. Southern California Geotechnical should be provided with copies of any reports documenting future environmental remediation activities to determine their impact on the recommendations presented in this report.

Moisture Sensitive Subgrade Soils

Most of the near surface soils possess occasional silt and clay content. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas.

Groundwater

Based on the soil conditions encountered during drilling, the measurements taken within the open boreholes at the completion of drilling, and the in-situ moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 18 to 18½± feet below grade, at the time of the subsurface exploration. Therefore, groundwater is not expected to impact the proposed grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils extending to a depth of at least 3 feet below foundation bearing grade. Based on this subsurface profile, the proposed structures may be supported on conventional shallow foundations.

Proposed Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Within the new office building and garage areas, soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slab and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against suitable compacted structural fill. The maximum allowable passive pressure is 2500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 18 inches on-center, in both directions. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

- Slab underlayment: 2 inches of sand overlain by a 10 mil visqueen vapor barrier, overlain by 2 inches of additional sand.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Trash Enclosure Design Parameters

Although not clearly shown on the site plan provided to our office, the proposed development is expected to include at least one trash enclosure. It is expected that the trash enclosure as well as the approach slab will be subjected to relatively heavy wheel loads, imposed by trash removal equipment.

The subgrade soils in the area of the trash enclosure and the approach slab should be prepared in accordance with the recommendations for the parking areas, presented in Section 6.3 of this report. As such, it is expected that the trash enclosure will be underlain by structural fill soils, extending to a depth of 3 feet below proposed subgrade elevation. Based on geotechnical considerations, the following recommendations are provided for the design of the trash enclosure and the trash enclosure approach slab:

- The trash enclosure may consist of a 6-inch thick concrete slab incorporating a perimeter footing or a turned down edge, extending to a depth of at least 12 inches below adjacent finished grade. If the trash enclosure will incorporate rigid walls such as masonry block or tilt-up concrete, the perimeter foundations should be designed in accordance with the recommendations previously presented in Section 6.5 of this report.

- Reinforcement within the trash enclosure slab should consist of at least No. 3 bars at 18-inches on-center, in both directions.

- The trash enclosure approach slab should be constructed of Portland cement concrete, at least 6 inches in thickness. Reinforcement within the approach slab should consist of at least No. 3 bars at 18-inches on-center, in both directions.

- The trash enclosure and approach slab subgrades should be moisture conditioned to 2 to 4 percent above the optimum moisture content to a depth of 12 inches. The trash enclosure slab and the approach slab should be structurally connected, to reduce the potential for differential movement between the two slabs.

- The actual design of the trash enclosure and the trash enclosure approach slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.8 Landscape/Planter Wall Construction

Foundations

Foundations for landscape walls should be founded at a minimum depth of 12 inches below the lowest adjacent final grade. The footings should also be reinforced with a minimum of two No. 4 bars, one top and one bottom.

Construction Joints

In order to minimize the potential for unsightly cracking related to the effects of differential settlement, construction joints should be provided in the walls at horizontal intervals of approximately 20± feet, and at each corner. The separations should be provided in the blocks and should not extend through the foundation. Foundations should be poured monolithically with continuous reinforcement along the entire length of the wall. A joint to provide positive separation between the wall face and adjacent flatwork is also recommended. A ½± inch thick felt joint may be used for this application.

Drainage

Area drains should be extended into all planters that are located within 5 feet of building walls, foundations, retaining walls and landscape walls to minimize infiltration of water into the adjacent foundation soils. The surface of the ground in these areas should also be sloped at a minimum gradient of 2 percent away from the walls and foundations. A drip irrigation system is also recommended to prevent overwatering and subsequent saturation of the foundation walls.

Planter walls should be supported by continuous concrete footings designed and constructed in accordance with the recommendations presented for landscape walls.

6.9 Retaining Wall Design and Construction

Construction of the proposed residence basement may require below grade walls in the vicinity of the truck dock loading areas. Some small retaining walls may also be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters for two different types of wall backfill: on-site granular soils and imported select granular material. The on-site soils generally consist of sands, clayey sands and silty sands. Based on their composition, the on-site soils have been assigned a friction angle of 30 degrees. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60°.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type	
		Imported Aggregate Base	On-Site Soils
Internal Friction Angle (ϕ)		38°	30°
Unit Weight		130 lbs/ft^3	125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	30 lbs/ft^3	42 lbs/ft^3
	Active Condition (2h:1v backfill)	44 lbs/ft^3	67 lbs/ft^3
	At-Rest Condition (level backfill)	50 lbs/ft^3	63 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly, such as the perimeter walls of any basement levels.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Retaining Wall Foundation Design

The non-basement retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. Basement retaining wall foundations should be supported in accordance with the recommendations presented in a previous section of this report. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

It is recommended that a minimum 1 foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.10 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be supported on the existing fill soils. These materials generally consist of sands, silty sands and clayey sands. These materials are expected to exhibit fair to good pavement support characteristics, with estimated R-values of 30 to 50. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 30. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It may be desirable to perform

R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas. However, truck traffic must be excluded from areas where the thinner pavement section is used; otherwise premature pavement distress may occur. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes.

ASPHALT PAVEMENTS				
Materials	**Thickness (inches)**			
	Parking Stalls (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)	Moderate Truck Traffic (TI = 7.0)
Asphalt Concrete	3	3	3½	4
Aggregate Base	3	6	7½	10
Compacted Subgrade	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within Portland cement concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS			
Materials	**Thickness (inches)**		
	Automobile Parking and Drive Areas (TI=5.0)	Light Truck Traffic Areas (TI =6.0)	Moderate Truck Traffic Areas (TI =7.0)
PCC	5	5½	6½
Compacted Subgrade (95% minimum compaction)	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-1

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: 18.5 feet
CAVE DEPTH: 21 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	50/4"			FILL: Red Brown fine Sand, little Clay, trace Silt, trace fine root fibers, cemented, very dense-dry	105	2					
	X	54			FILL: Red Brown Silty fine Sand, dense-damp	109	6					
5	X	57			FILL: Brown Clayey fine Sand, trace medium Sand, dense-damp	112	6					
	X	19			ALLUVIUM: Dark Brown fine to medium Sand, medium dense-damp to moist	103	8					
10	X	28			Dark Gray Brown fine Sand, trace medium Sand, trace Clay, medium dense-damp to moist	110	9					
					Light Gray fine to medium Sand, little Silt, strong Hydrocarbon odor, medium dense to dense-moist to wet							
15	X	26					11					
20	X	32			@ 18½ feet, water encountered during drilling		28					
25	X	42					20					
					Boring Terminated at 25'							

TBL 07G154.GPJ SOCALGEO.GDT 6/5/07

TEST BORING LOG

PLATE B-1

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-2

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					FILL: Red Brown Silty fine Sand, trace Clay, cemented, very dense-damp							
	X	50/6"					6					
	X	61					5					
5												
					POSSIBLE FILL: Red Brown Silty fine Sand, trace Clay, cemented, very dense-dry							
	X	37					8					
					ALLUVIUM: Brown fine to medium Sand, trace Silt, medium dense to dense-damp							
	X	30					5					
10												
					Light Brown fine Sand, trace Silt, medium dense-damp							
	X	25					6					
15												
					Boring Terminated at 15'							

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-3

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: 18 feet
CAVE DEPTH: 19 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					5± inches Asphaltic concrete underlain by 6± inches Aggregate base							
	X	18			POSSIBLE FILL: Dark Red Brown Clayey fine Sand, trace Clay, medium dense-damp	122	10					
	X	20	4.5+		Brown fine Sandy Clay, very stiff-damp to moist	120	13					
5	X	23			ALLUVIUM: Brown Clayey fine Sand, trace Silt, medium dense-damp to moist	107	13					
	X	23			Gray fine to medium Sand, medium dense-damp	105	5					
10	X	17			Orange Brown fine to medium Sand, trace Silt, strong Hydrocarbon odor, medium dense-damp to moist	100	7					
15	X	22			Brown fine to coarse Sand, strong Hydrocarbon odor, medium dense-moist		8					
20	X	45			Green Gray fine to coarse Sand, strong Hydrocarbon odor, dense-wet @ 18 feet, water encountered during drilling		17					

Boring Terminated at 20'

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-4

JOB NO.: 07G154	DRILLING DATE: 5/11/07	WATER DEPTH: Dry
PROJECT: Proposed Mixed Use Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 13 feet
LOCATION: Costa Mesa, California	LOGGED BY: Tim Smith	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4± inches Asphaltic concrete underlain by no discernible Aggregate base							
	X	8			POSSIBLE FILL: Dark Red Brown Silty fine Sand, trace Clay, loose to medium dense-moist		12					EI = 1 @ 0 to 5'
	X	16					12					
5												
	X	16	4.5+		ALLUVIUM: Dark Brown fine Sandy Clay, trace Silt, strong Hydrocarbon odor, very stiff-moist		13					
	X	12			Green Gray fine to coarse Sand, strong Hydrocarbon odor, medium dense-moist		7					
10												
	X	21					6					
15												
					Boring Terminated at 15'							

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-5

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 20 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	24		FILL: Red Brown Silty fine Sand, medium dense-damp	96	8					
	50/4"		FILL: Brown Silty fine Sand, little Clay, dense to very dense-damp	117	9					
5	22	4.5+	ALLUVIUM: Brown fine Sandy Clay, medium stiff-damp to moist	106	14					
	28		Brown fine to medium Sand, trace Silt, medium dense-moist	116	14					
10	14		Orange Brown fine to medium Sand, trace coarse Sand, trace fine root fibers, loose-damp	98	4					
15	19		Light Brown fine Sand, trace thin Clay layers, medium dense-moist		7					
20	10	3.0	Green Gray Silty Clay, trace Silt, strong Hydrocarbon odor, medium stiff-very moist		31					
25	13	2.75			31					

Boring Terminated at 25'

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-6

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California
DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith
WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
	X	7			FILL: Black Silty fine Sand, trace Clay, little Asphalt fragments, loose-damp to moist		15					EI = 6 @ 0 to 5'
5	X	15			ALLUVIUM: Brown fine Sandy Clay, stiff-damp to moist with interbeds of Brown fine Sand, stiff-damp		12					
	X	18			Brown Clayey fine Sand, trace Silt, medium dense-damp to moist		11					
10	X	27			Red Brown fine to medium Sand, trace Silt, medium dense-damp to moist		8					
15	X	28					6					
					Boring Terminated at 15'							

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TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-7

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 11 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Dark Red Brown Silty fine Sand, trace Asphalt debris, medium dense-damp							
	X	18					6					
	X	24	4.5+		ALLUVIUM: Brown fine Sandy Clay, trace Silt, stiff-damp to moist		14					
5												
	X	17			Light Brown to Orange Brown fine Sand, trace Silt, trace medium to coarse Sand, medium dense-damp		7					
	X	16					4					
10												
					Light Brown fine to coarse Sand, medium dense-damp							
	X	13					3					
15												
					Boring Terminated at 15'							

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-8

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 17 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Dark Red Brown Silty fine Sand, trace Clay, medium dense-damp to moist							
	X	14				136	11					
	X	31				126	10					
5	X	25			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, medium dense-damp to moist	108	10					
	X	27				113	8					
	X	28			Orange Brown fine coarse Sand, little Clay, medium dense-damp to moist	103	8					
10												
					Brown fine Sand, trace Silt, medium dense-moist							
	X	20					12					
15												
					Green Gray Clayey Silt, trace fine Sand, medium stiff-very moist							
	X	12	4.0				33					
20												

Boring Terminated at 20'

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-9

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
	X	16			ALLUVIUM: Red Brown Silty fine Sand, trace Clay, medium dense-damp		6					
5	X	30	4.5+		Red Brown fine Sandy Clay, trace Silt, medium dense-damp		8					
					Boring Terminated at 5'							

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

PLATE B-9

SOUTHERN CALIFORNIA GEOTECHNICAL

BORING NO. B-10

JOB NO.: 07G154
PROJECT: Proposed Mixed Use Development
LOCATION: Costa Mesa, California

DRILLING DATE: 5/11/07
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Tim Smith

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

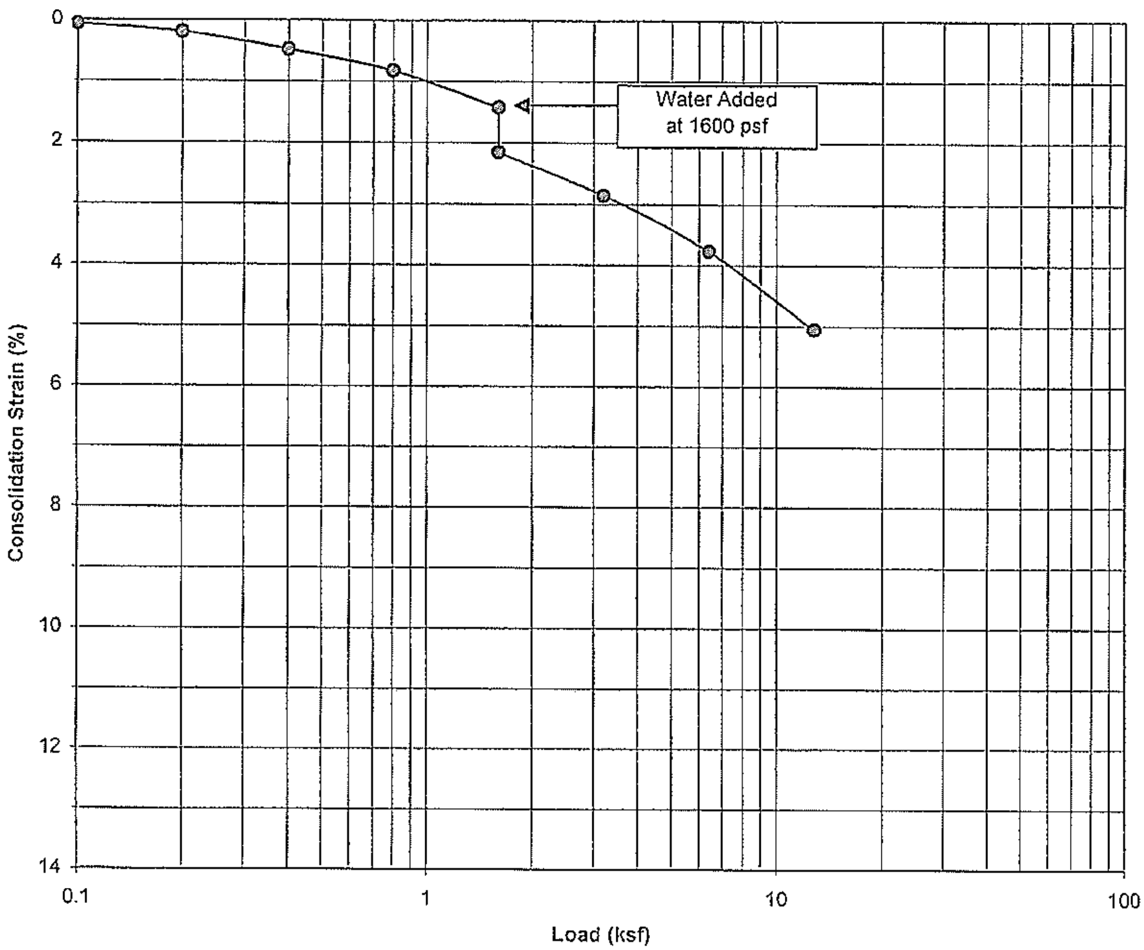
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					FILL: Dark Brown Silty fine Sand, trace Asphalt debris, loose-damp to moist							
	X	6					12					
					ALLUVIUM: Brown Silty fine Sand, trace Clay, loose-damp to moist							
	X	8					10					
5	X	16				117	10					

Boring Terminated at 6'

TBL 07G154.GPJ SOCALGEO.GDT 6/6/07

TEST BORING LOG

Consolidation/Collapse Test Results

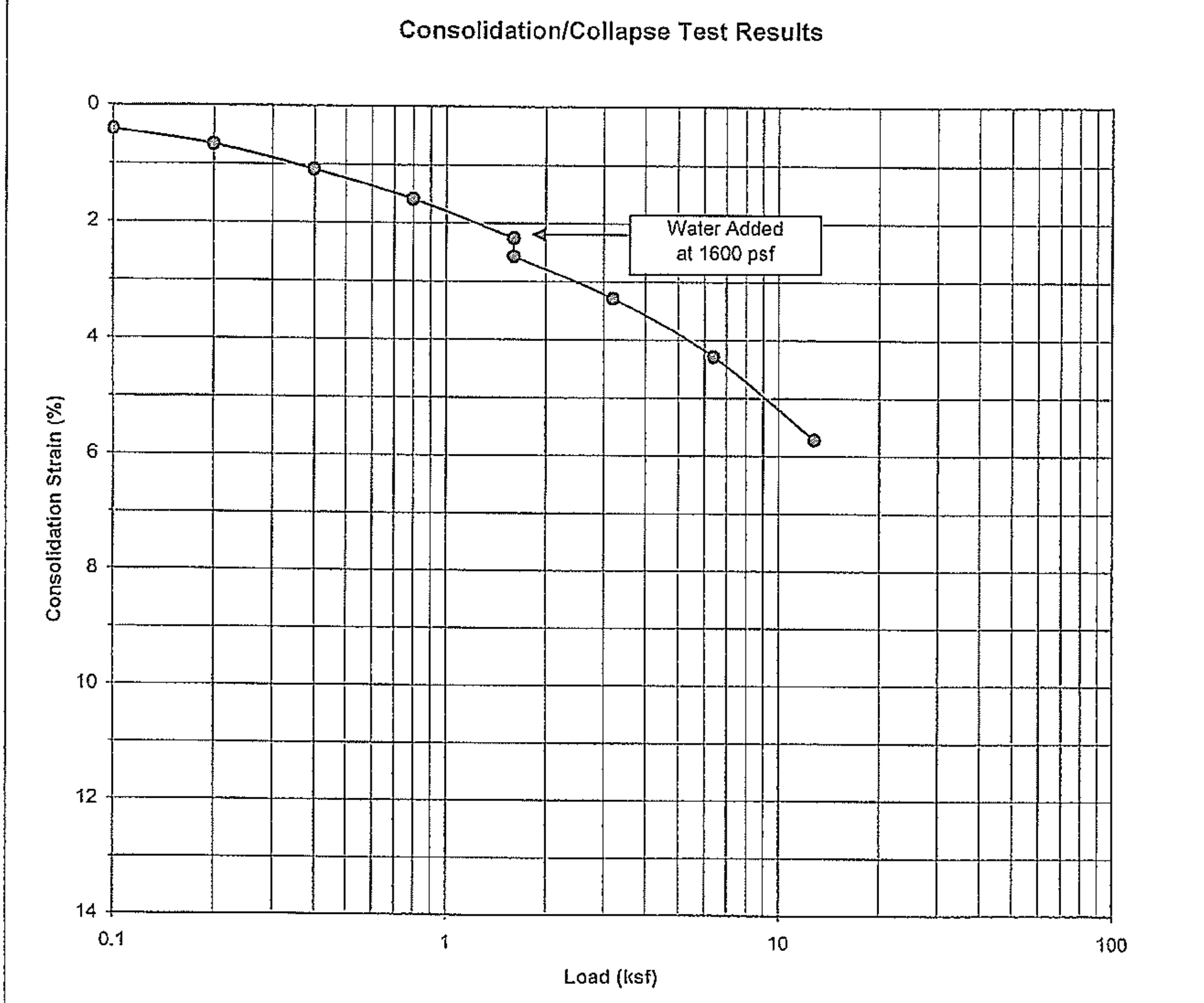


Classification: POSSIBLE FILL: Dark Red Brown Clayey fine Sand, trace Clay

Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	10
Depth (ft)	1 to 2	Initial Dry Density (pcf)	123.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	130.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.73

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C- 1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

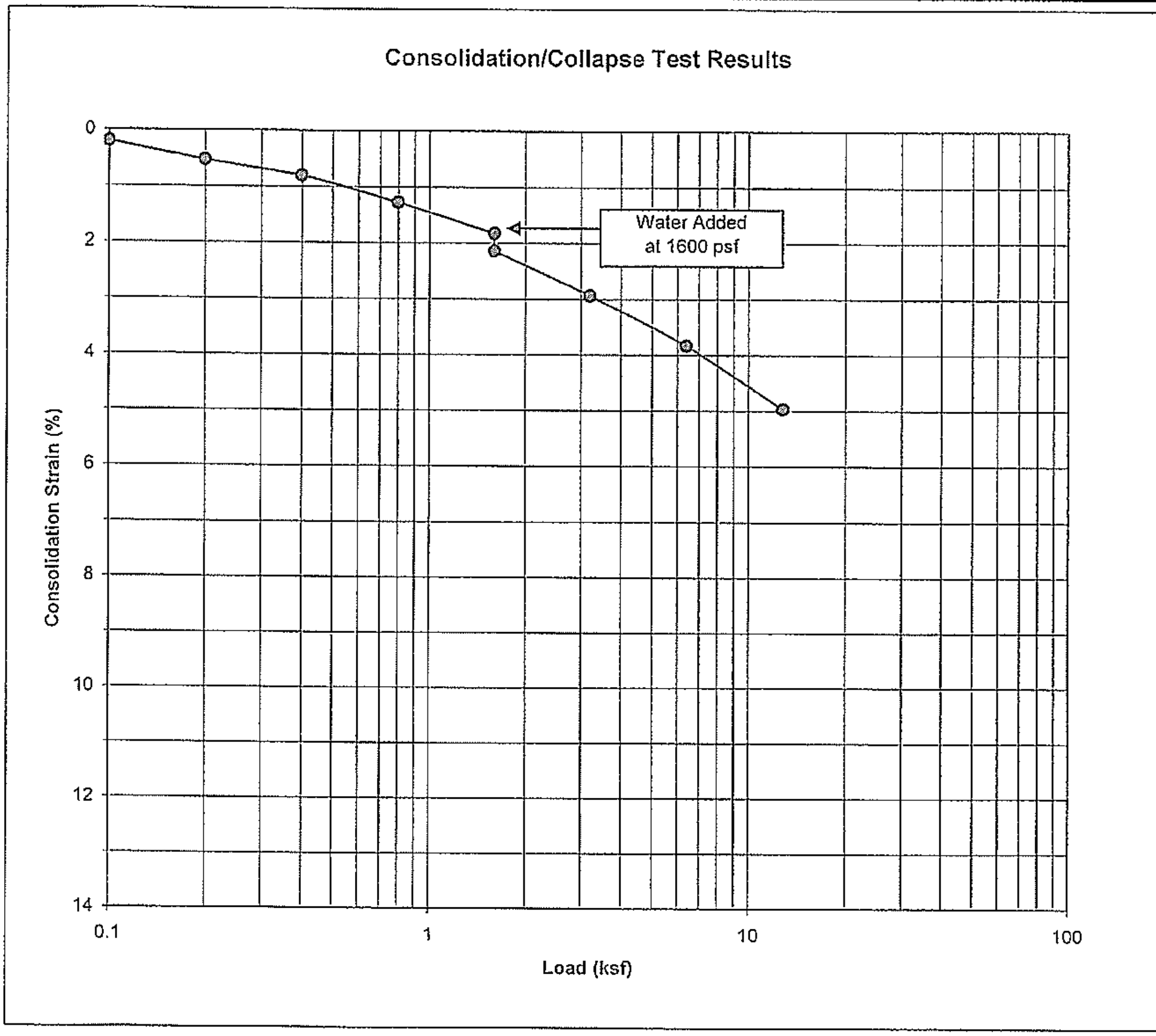


Classification: POSSIBLE FILL: Brown fine Sandy Clay

Boring Number:	B-3	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	3 to 4	Initial Dry Density (pcf)	122.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	130.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.31

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



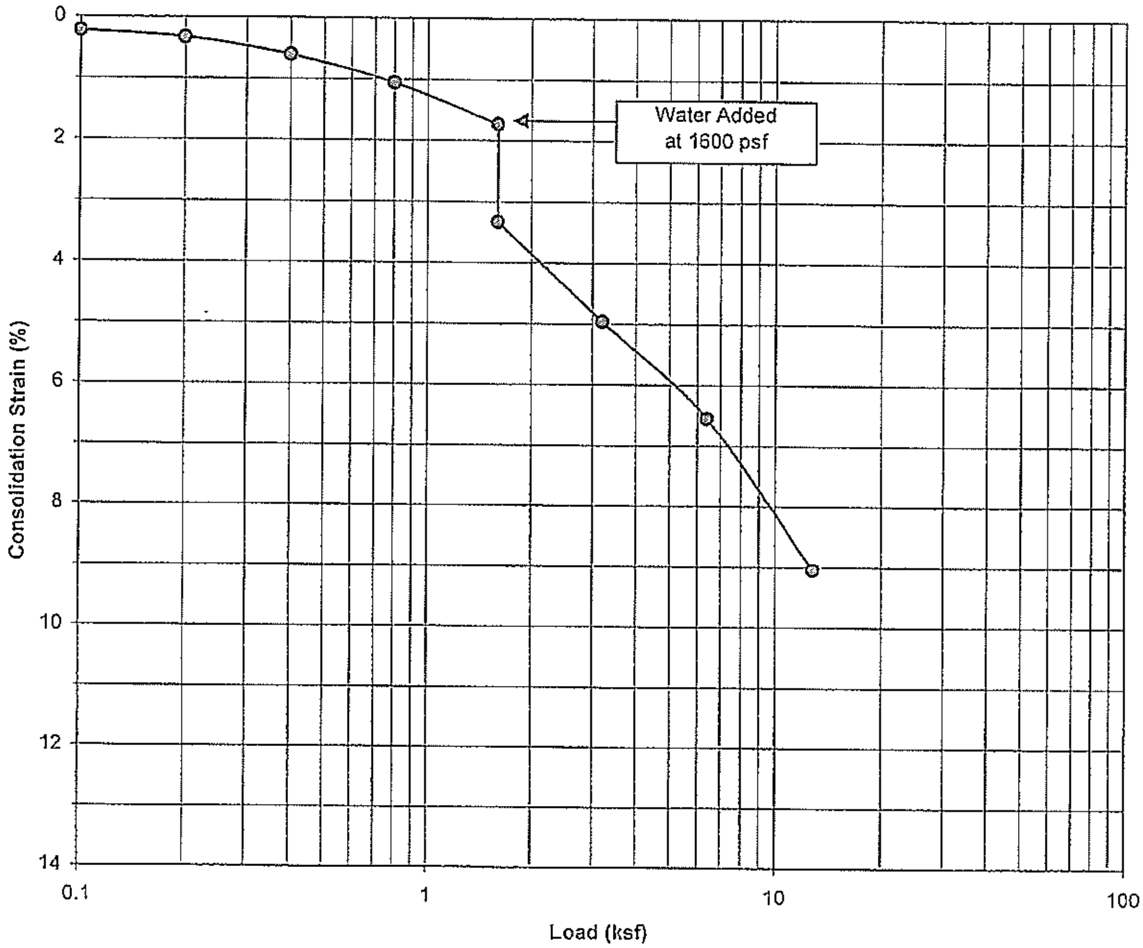
Classification: ALLUVIUM: Gray fine to medium Sand

Boring Number:	B-3	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	7 to 8	Initial Dry Density (pcf)	103.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.31

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C- 3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

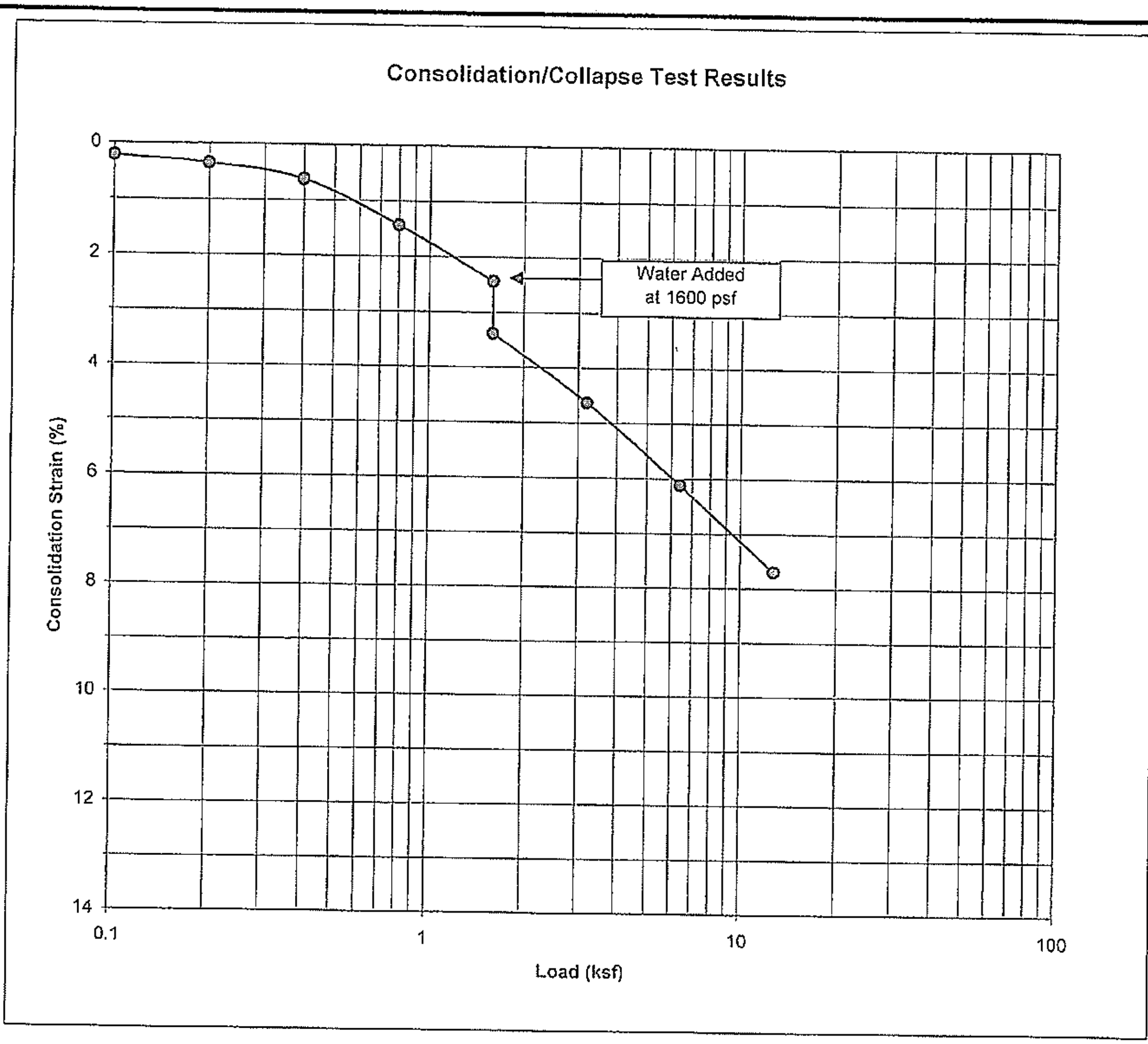


Classification: FILL: Dark Red Brown Silty fine Sand, trace Clay

Boring Number:	B-8	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	1 to 2	Initial Dry Density (pcf)	114.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	126.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.60

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

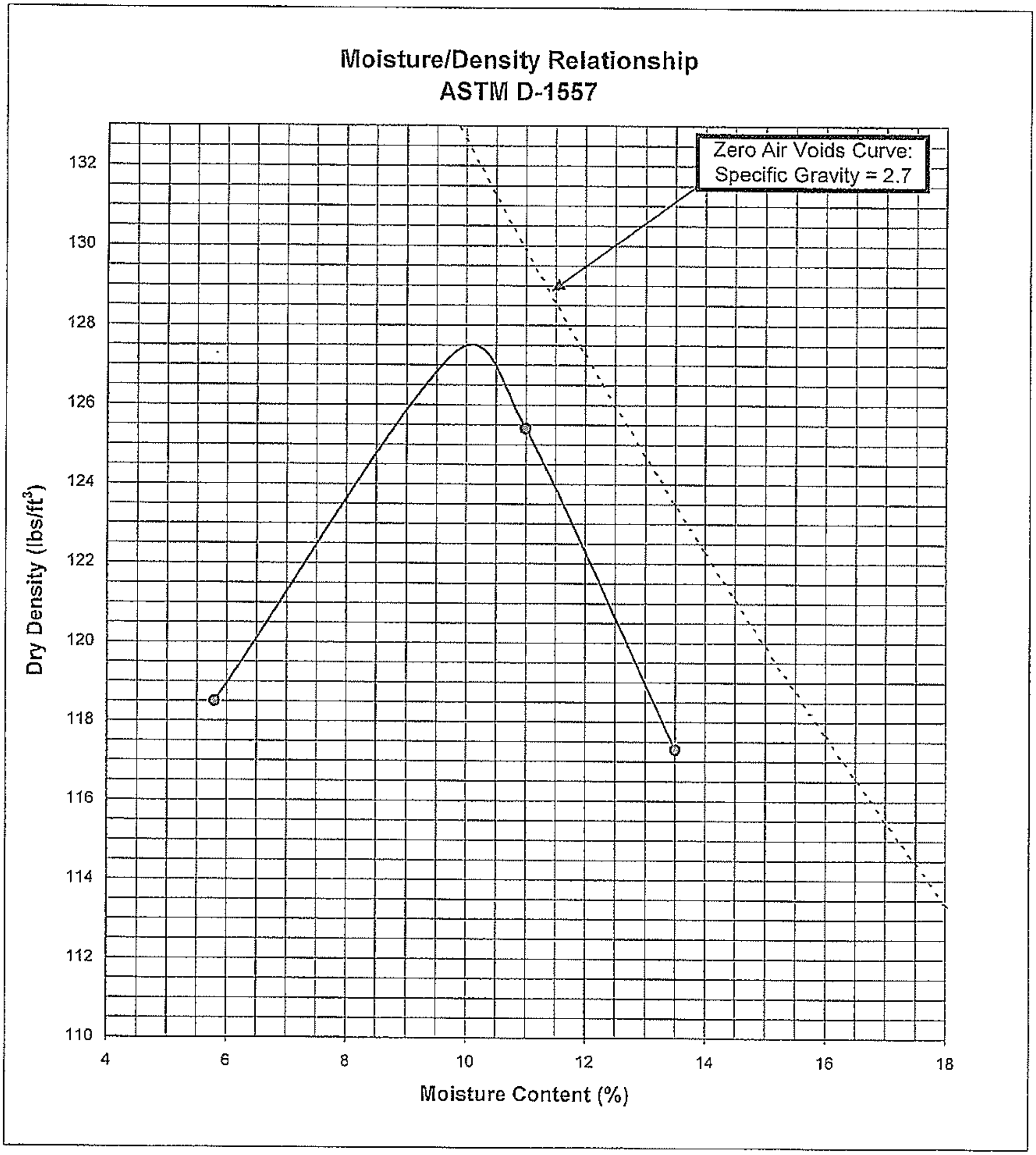


Classification: ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-8	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	7 to 8	Initial Dry Density (pcf)	114.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.95

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-4 @ 0 to 5'
Optimum Moisture (%)	10
Maximum Dry Density (pcf)	127.5
Soil Classification	Dark Red Brown Silty fine Sand

Proposed Mixed Use Development
Costa Mesa, California
Project No. 07G154
PLATE C-7

SOUTHERN CALIFORNIA GEOTECHNICAL

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and Uniform Building Codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.
- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 6 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. Acceptable methods typically include windrows. Oversize materials should not be placed within the range of excavation for foundations, utilities, or pools to facilitate excavations. Rock placement should be kept away from slopes (minimum distance: 15 feet) to facilitate compaction near the slope.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates G-2, G-4, and G-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate G-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate G-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate G-2).

<u>Cut Slopes</u>

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate G-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates G-6.

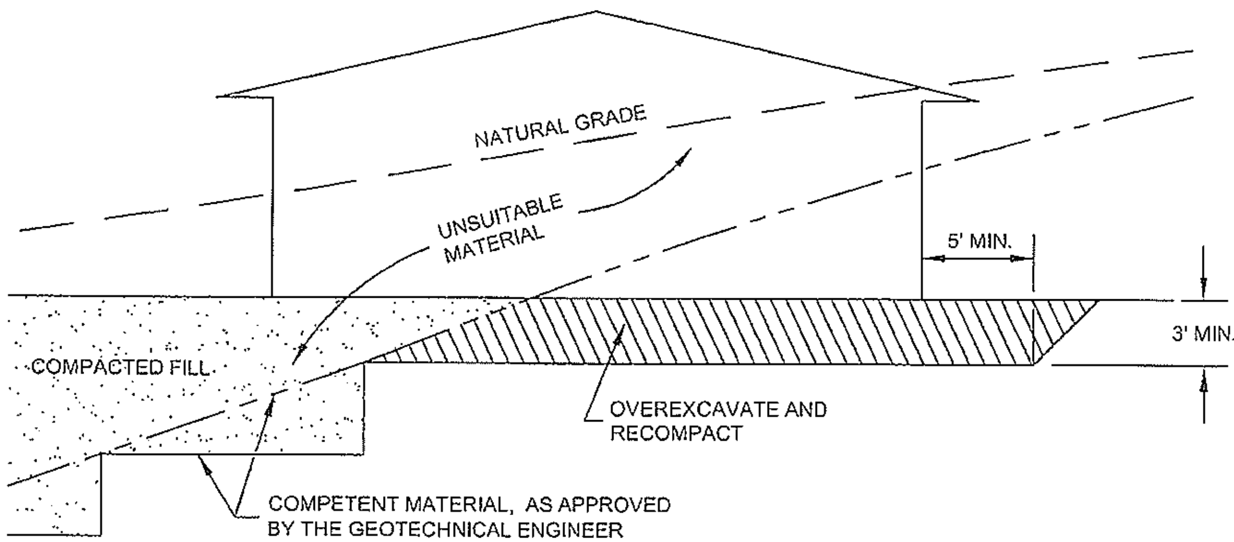
<u>Subdrains</u>

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate G-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

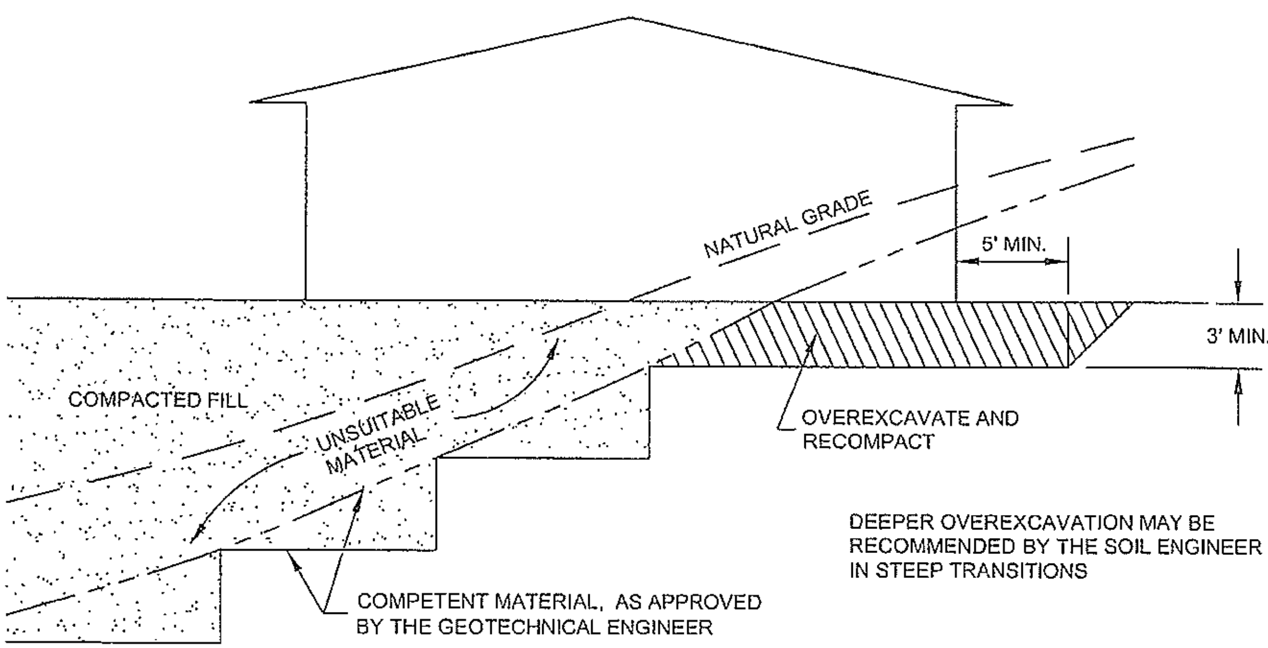
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



TRANSITION LOT DETAIL

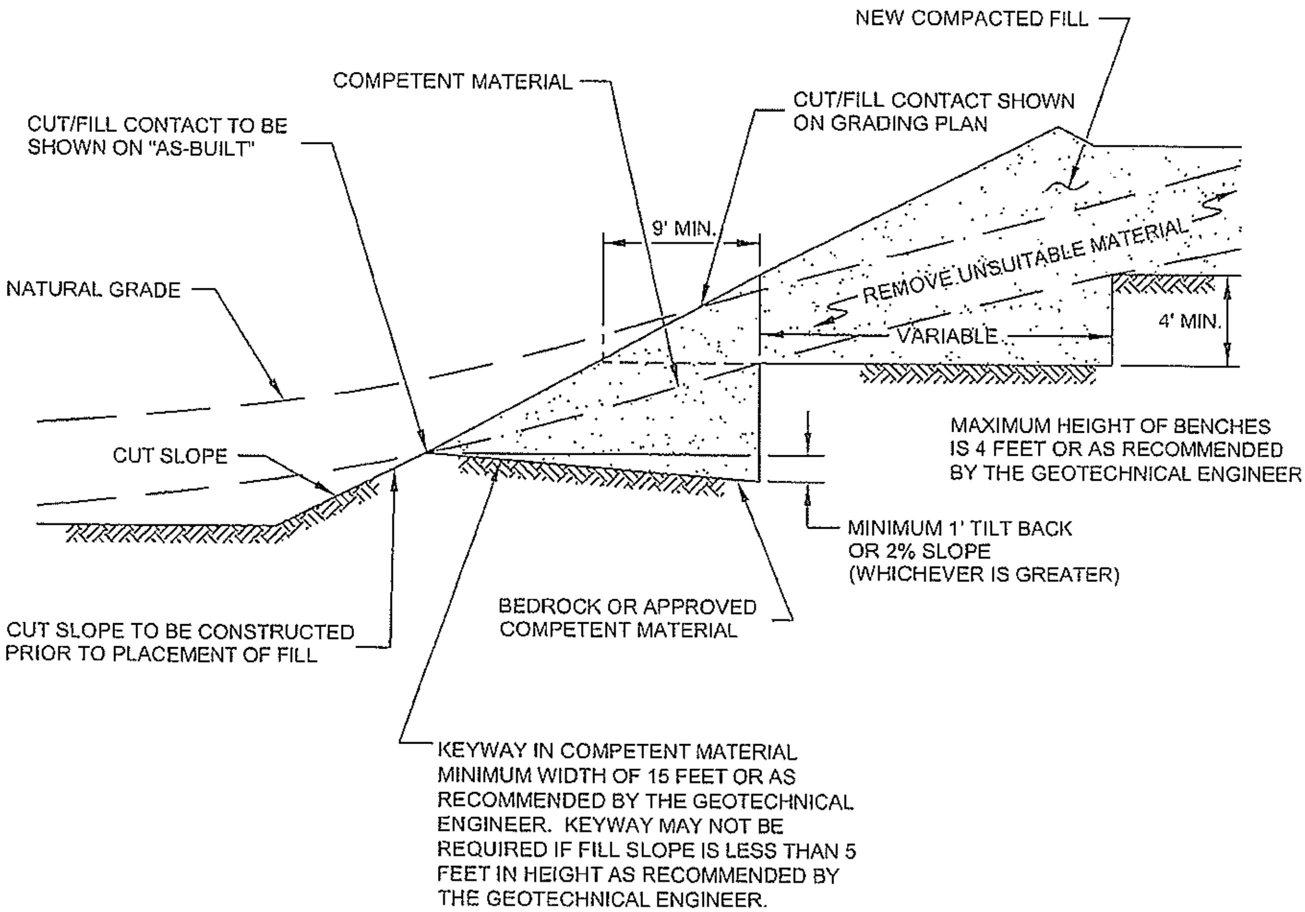
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD. GKM

PLATE D-1

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

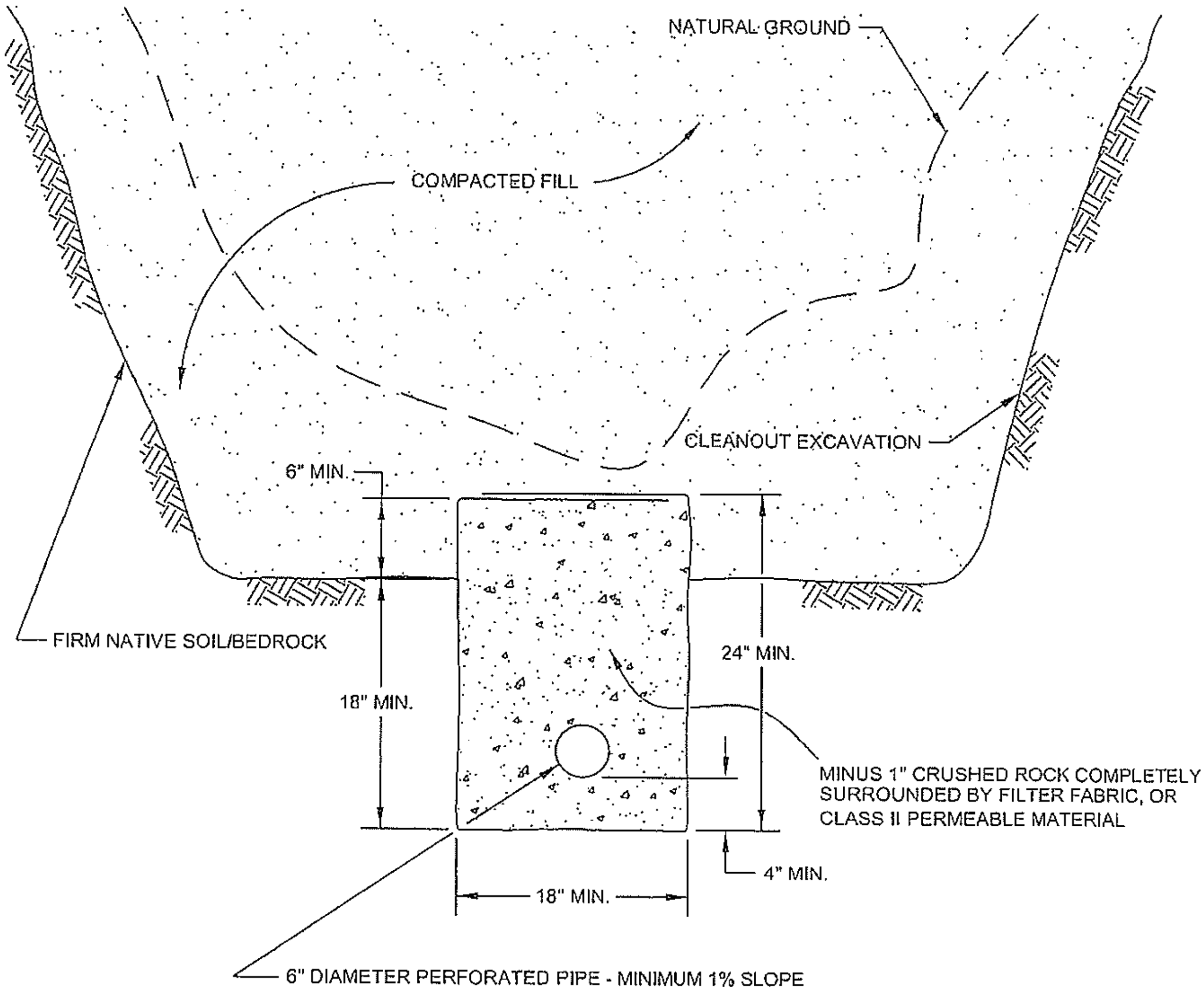
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo

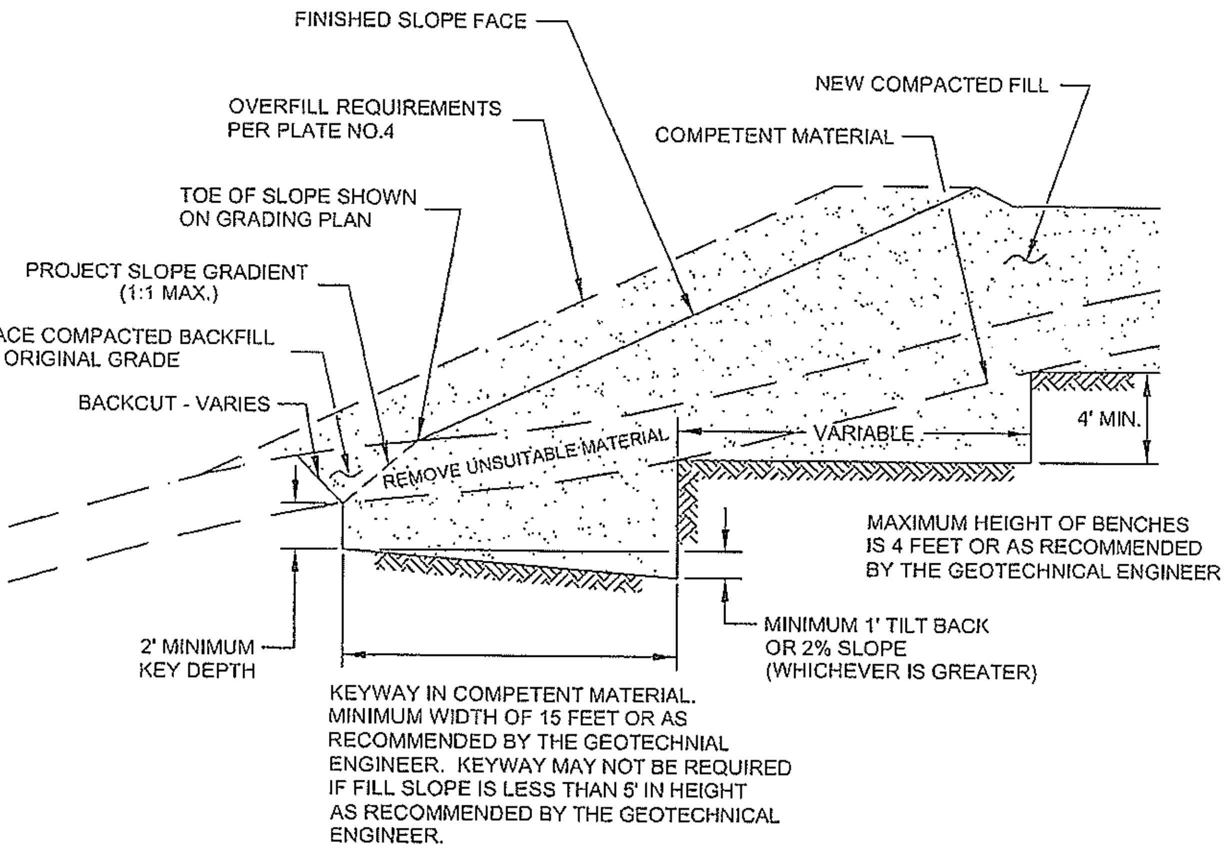
SOUTHERN
CALIFORNIA
GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

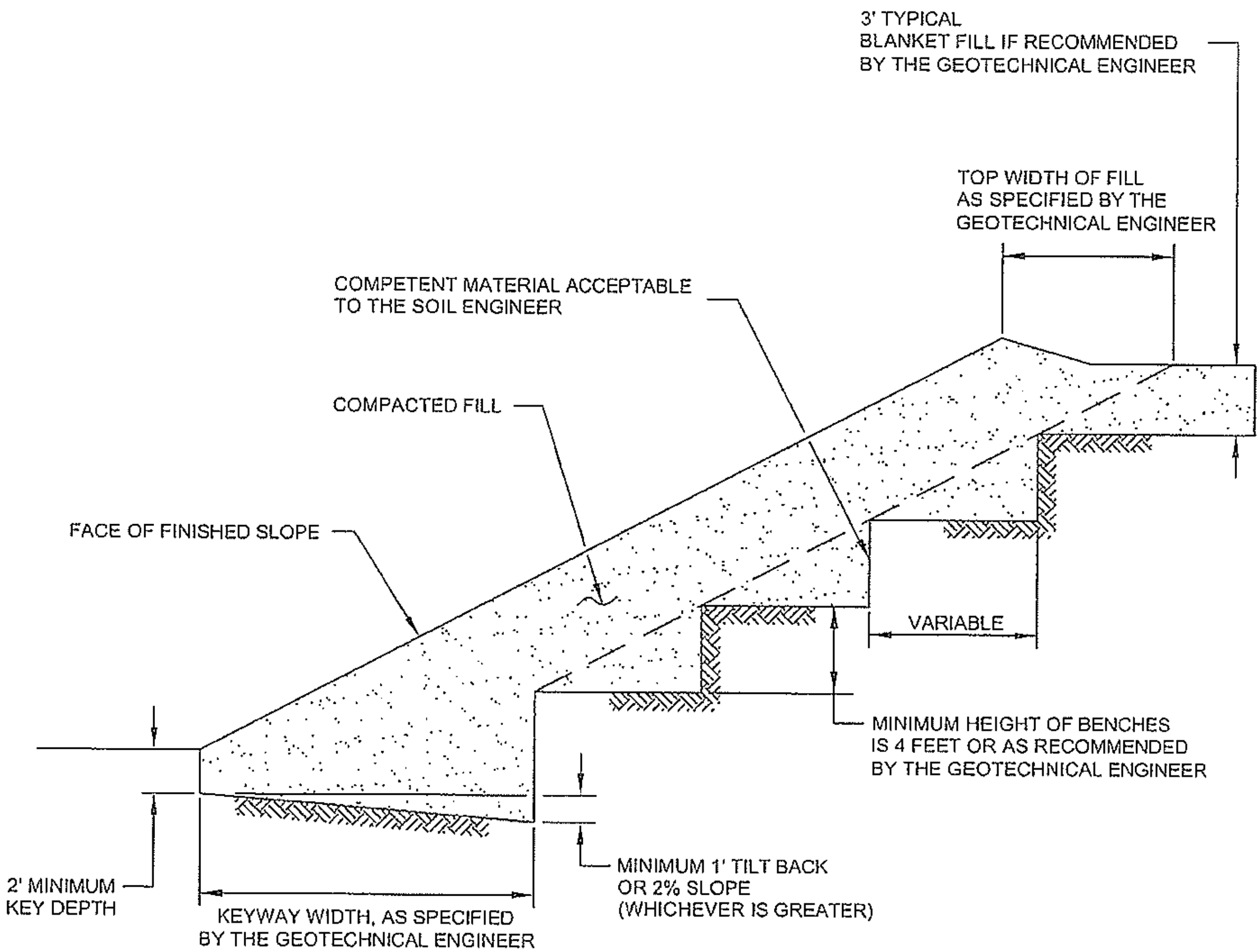
SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-3	



NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-4	



STABILIZATION FILL DETAIL

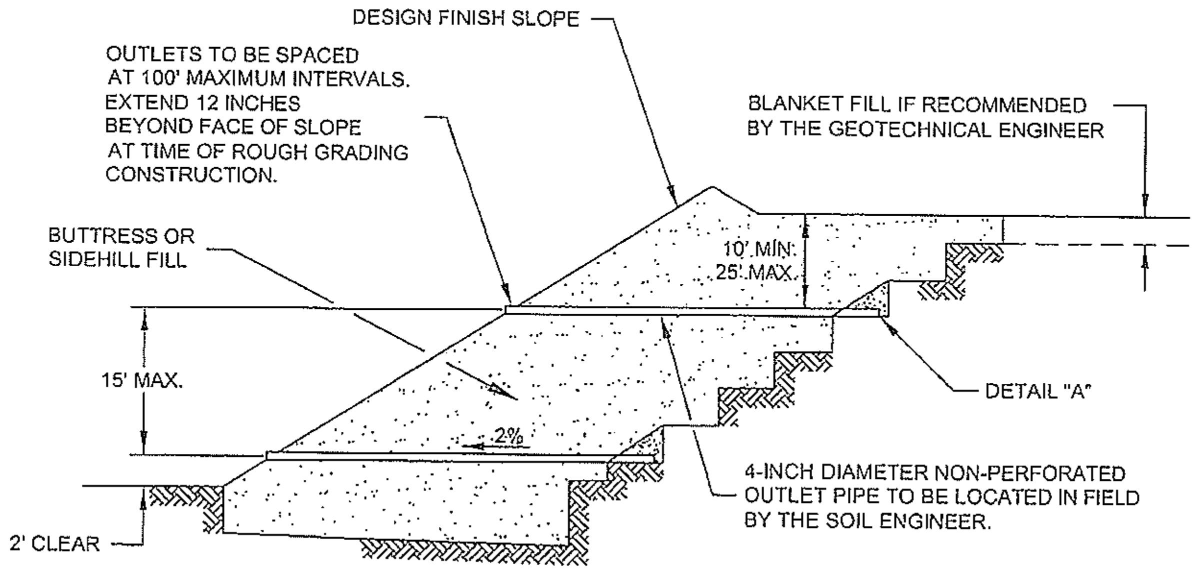
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

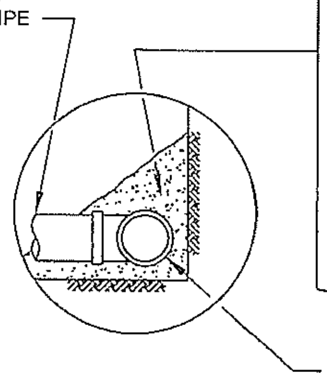
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

NOTES:

1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

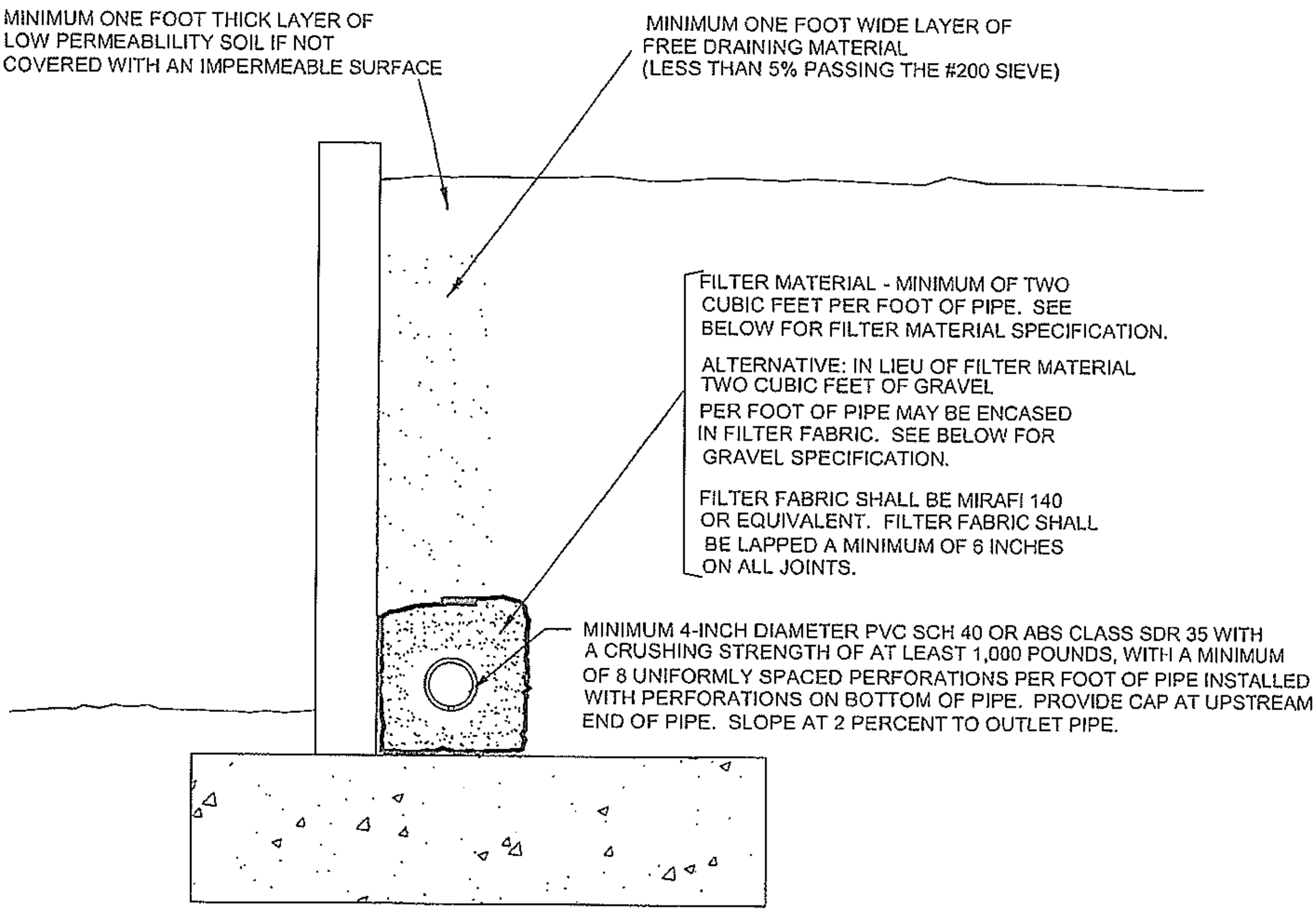
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL

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**********************
*                    *
*    U B C S E I S   *
*                    *
*    Version  1.03   *
*                    *
**********************
```

COMPUTATION OF 1997
UNIFORM BUILDING CODE
SEISMIC DESIGN PARAMETERS

JOB NUMBER: 07G154-1   DATE: 05-25-2007

JOB NAME: Proposed Walgreen's and Residential Development

FAULT-DATA-FILE NAME: CDMGUBCR.DAT

SITE COORDINATES:
SITE LATITUDE: 33.6650
SITE LONGITUDE: 117.9196

UBC SEISMIC ZONE: 0.4

UBC SOIL PROFILE TYPE: SD

NEAREST TYPE A FAULT:
NAME: CUCAMONGA
DISTANCE: 54.2 km

NEAREST TYPE B FAULT:
NAME: NEWPORT-INGLEWOOD (L.A.Basin)
DISTANCE: 4.8 km

NEAREST TYPE C FAULT:
NAME:
DISTANCE: 99999.0 km

SELECTED UBC SEISMIC COEFFICIENTS:
Na: 1.0
Nv: 1.2
Ca: 0.45
Cv: 0.78
Ts: 0.699
To: 0.140

```
*******************************************************************
* CAUTION:  The digitized data points used to model faults are     *
*           limited in number and have been digitized from small-  *
*           scale maps (e.g., 1:750,000 scale).  Consequently,     *
*           the estimated fault-site-distances may be in error by  *
*           several kilometers.  Therefore, it is important that   *
*           the distances be carefully checked for accuracy and    *
*           adjusted as needed, before they are used in design.    *
*******************************************************************
```

SUMMARY OF FAULT PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
NEWPORT-INGLEWOOD (L.A.Basin)	4.8	B	6.9	1.00	SS
NEWPORT-INGLEWOOD (Offshore)	8.2	B	6.9	1.50	SS
PALOS VERDES	22.7	B	7.1	3.00	SS
ELSINORE-WHITTIER	29.5	B	6.8	2.50	SS
CHINO-CENTRAL AVE. (Elsinore)	32.3	B	6.7	1.00	DS
ELSINORE-GLEN IVY	33.6	B	6.8	5.00	SS
SAN JOSE	41.8	B	6.5	0.50	DS
CORONADO BANK	44.1	B	7.4	3.00	SS
SIERRA MADRE (Central)	52.8	B	7.0	3.00	DS
ELSINORE-TEMECULA	53.0	B	6.8	5.00	SS
CUCAMONGA	54.2	A	7.0	5.00	DS
RAYMOND	54.8	B	6.5	0.50	DS
VERDUGO	56.2	B	6.7	0.50	DS
CLAMSHELL-SAWPIT	57.5	B	6.5	0.50	DS
HOLLYWOOD	58.1	B	6.5	1.00	DS
SANTA MONICA	64.7	B	6.6	1.00	DS
MALIBU COAST	69.8	B	6.7	0.30	DS
SAN JACINTO-SAN BERNARDINO	71.9	B	6.7	12.00	SS
SAN JACINTO-SAN JACINTO VALLEY	74.3	B	6.9	12.00	SS
ROSE CANYON	75.1	B	6.9	1.50	SS
SIERRA MADRE (San Fernando)	76.5	B	6.7	2.00	DS
SAN GABRIEL	79.9	B	7.0	1.00	SS
SAN ANDREAS - Southern	80.0	A	7.4	24.00	SS
ANACAPA-DUME	80.1	B	7.3	3.00	DS
SAN ANDREAS - 1857 Rupture	80.3	A	7.8	34.00	SS
CLEGHORN	83.3	B	6.5	3.00	SS
ELSINORE-JULIAN	89.8	A	7.1	5.00	SS
SANTA SUSANA	90.6	B	6.6	5.00	DS
NORTH FRONTAL FAULT ZONE (West)	92.7	B	7.0	1.00	DS
SAN JACINTO-ANZA	93.2	A	7.2	12.00	SS
HOLSER	99.4	B	6.5	0.40	DS
SIMI-SANTA ROSA	107.0	B	6.7	1.00	DS
OAK RIDGE (Onshore)	107.5	B	6.9	4.00	DS
SAN CAYETANO	115.9	B	6.8	6.00	DS
PINTO MOUNTAIN	119.1	B	7.0	2.50	SS
NORTH FRONTAL FAULT ZONE (East)	121.6	B	6.7	0.50	DS
HELENDALE - S. LOCKHARDT	124.8	B	7.1	0.60	SS
SAN JACINTO-COYOTE CREEK	132.7	B	6.8	4.00	SS
EARTHQUAKE VALLEY	135.0	B	6.5	2.00	SS
SANTA YNEZ (East)	135.6	B	7.0	2.00	SS
VENTURA - PITAS POINT	136.0	B	6.8	1.00	DS
LENWOOD-LOCKHART-OLD WOMAN SPRGS	141.4	B	7.3	0.60	SS
M.RIDGE-ARROYO PARIDA-SANTA ANA	146.0	B	6.7	0.40	DS
BURNT MTN.	146.2	B	6.5	0.60	SS
JOHNSON VALLEY (Northern)	149.3	B	6.7	0.60	SS
LANDERS	149.7	B	7.3	0.60	SS

SUMMARY OF FAULT PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
EUREKA PEAK	150.2	B	6.5	0.60	SS
RED MOUNTAIN	150.2	B	6.8	2.00	DS
SANTA CRUZ ISLAND	152.0	B	6.8	1.00	DS
GARLOCK (West)	156.4	A	7.1	6.00	SS
PLEITO THRUST	158.1	B	6.8	2.00	DS
EMERSON So. - COPPER MTN.	160.2	B	6.9	0.60	SS
GRAVEL HILLS - HARPER LAKE	162.4	B	6.9	0.60	SS
BIG PINE	163.1	B	6.7	0.80	SS
ELSINORE-COYOTE MOUNTAIN	163.8	B	6.8	4.00	SS
SAN JACINTO - BORREGO	167.9	B	6.6	4.00	SS
CALICO - HIDALGO	169.9	B	7.1	0.60	SS
BLACKWATER	172.7	B	6.9	0.60	SS
PISGAH-BULLION MTN.-MESQUITE LK	177.5	B	7.1	0.60	SS
GARLOCK (East)	180.4	A	7.3	7.00	SS
WHITE WOLF	181.5	B	7.2	2.00	DS
SANTA YNEZ (West)	182.9	B	6.9	2.00	SS
SANTA ROSA ISLAND	188.2	B	6.9	1.00	DS
SUPERSTITION MTN. (San Jacinto)	199.4	B	6.6	5.00	SS
ELMORE RANCH	203.8	B	6.6	1.00	SS
SUPERSTITION HILLS (San Jacinto)	205.8	B	6.6	4.00	SS
So. SIERRA NEVADA	206.7	B	7.1	0.10	DS
BRAWLEY SEISMIC ZONE	207.8	B	6.5	25.00	SS
ELSINORE-LAGUNA SALADA	215.6	B	7.0	3.50	SS
LITTLE LAKE	217.6	B	6.7	0.70	SS
LOS ALAMOS-W. BASELINE	225.7	B	6.8	0.70	DS
TANK CANYON	228.3	B	6.5	1.00	DS
IMPERIAL	232.9	A	7.0	20.00	SS
PANAMINT VALLEY	236.1	B	7.2	2.50	SS
OWL LAKE	236.4	B	6.5	2.00	SS
LIONS HEAD	243.1	B	6.6	0.02	DS
SAN JUAN	250.7	B	7.0	1.00	SS
SAN LUIS RANGE (S. Margin)	251.7	B	7.0	0.20	DS
DEATH VALLEY (South)	257.6	B	6.9	4.00	SS
CASMALIA (Orcutt Frontal Fault)	260.7	B	6.5	0.25	DS
OWENS VALLEY	280.9	B	7.6	1.50	SS
LOS OSOS	281.4	B	6.8	0.50	DS
DEATH VALLEY (Graben)	285.1	B	6.9	4.00	DS
HOSGRI	289.1	B	7.3	2.50	SS
RINCONADA	301.3	B	7.3	1.00	SS
INDEPENDENCE	315.8	B	6.9	0.20	DS
HUNTER MTN. - SALINE VALLEY	315.8	B	7.0	2.50	SS
DEATH VALLEY (Northern)	334.6	A	7.2	5.00	SS
SAN ANDREAS (Creeping)	356.7	B	5.0	34.00	SS
BIRCH CREEK	370.2	B	6.5	0.70	DS
WHITE MOUNTAINS	376.6	B	7.1	1.00	SS
DEEP SPRINGS	396.7	B	6.6	0.80	DS

SUMMARY OF FAULT PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
ROUND VALLEY (E. of S.N.Mtns.)	403.3	B	6.8	1.00	DS
DEATH VALLEY (N. of Cucamongo)	406.3	A	7.0	5.00	SS
FISH SLOUGH	413.3	B	6.6	0.20	DS
HILTON CREEK	428.8	B	6.7	2.50	DS
ORTIGALITA	441.2	B	6.9	1.00	SS
CALAVERAS (So.of Calaveras Res)	446.4	B	6.2	15.00	SS
MONTEREY BAY - TULARCITOS	449.1	B	7.1	0.50	DS
PALO COLORADO - SUR	450.5	B	7.0	3.00	SS
HARTLEY SPRINGS	451.8	B	6.6	0.50	DS
QUIEN SABE	459.7	B	6.5	1.00	SS
ZAYANTE-VERGELES	478.1	B	6.8	0.10	SS
SAN ANDREAS (1906)	483.3	A	7.9	24.00	SS
SARGENT	483.4	B	6.8	3.00	SS
MONO LAKE	487.2	B	6.6	2.50	DS
ROBINSON CREEK	518.0	B	6.5	0.50	DS
SAN GREGORIO	524.2	A	7.3	5.00	SS
MONTE VISTA - SHANNON	533.3	B	6.5	0.40	DS
GREENVILLE	533.4	B	6.9	2.00	SS
HAYWARD (SE Extension)	533.5	B	6.5	3.00	SS
CALAVERAS (No.of Calaveras Res)	553.3	B	6.8	6.00	SS
HAYWARD (Total Length)	553.3	A	7.1	9.00	SS
ANTELOPE VALLEY	557.7	B	6.7	0.80	DS
GENOA	581.9	B	6.9	1.00	DS
CONCORD - GREEN VALLEY	601.1	B	6.9	6.00	SS
RODGERS CREEK	639.7	A	7.0	9.00	SS
WEST NAPA	640.7	B	6.5	1.00	SS
POINT REYES	658.1	B	6.8	0.30	DS
HUNTING CREEK - BERRYESSA	663.5	B	6.9	6.00	SS
MAACAMA (South)	702.4	B	6.9	9.00	SS
COLLAYOMI	719.4	B	6.5	0.60	SS
BARTLETT SPRINGS	723.3	A	7.1	6.00	SS
MAACAMA (Central)	743.8	A	7.1	9.00	SS
MAACAMA (North)	803.4	A	7.1	9.00	SS
ROUND VALLEY (N. S.F.Bay)	810.0	B	6.8	6.00	SS
BATTLE CREEK	837.1	B	6.5	0.50	DS
LAKE MOUNTAIN	868.3	B	6.7	6.00	SS
GARBERVILLE-BRICELAND	885.1	B	6.9	9.00	SS
MENDOCINO FAULT ZONE	940.9	A	7.4	35.00	DS
LITTLE SALMON (Onshore)	948.2	A	7.0	5.00	DS
MAD RIVER	951.3	B	7.1	0.70	DS
CASCADIA SUBDUCTION ZONE	954.3	A	8.3	35.00	DS
McKINLEYVILLE	961.7	B	7.0	0.60	DS
TRINIDAD	963.3	B	7.3	2.50	DS
FICKLE HILL	963.6	B	6.9	0.60	DS
TABLE BLUFF	968.6	B	7.0	0.60	DS

LITTLE SALMON (Offshore)	982.0	B	7.1	1.00	DS

SUMMARY OF FAULT PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE (km)	SOURCE TYPE (A,B,C)	MAX. MAG. (Mw)	SLIP RATE (mm/yr)	FAULT TYPE (SS,DS,BT)
BIG LAGOON - BALD MTN.FLT.ZONE	1000.3	B	7.3	0.50	DS

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