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GEOTECHNICAL
P201900287

GEOTECHNICAL INVESTIGATION AND MAJOR GEOLOGIC CONSTRAINTS REPORT UPDATE
FOR
PARCEL MAP 18954
SOUTHWEST CORNER OF SNOWDROP ROAD AND HAVEN AVENUE
RANCHO CUCAMONGA, CA

for

Marangston Inc.
300 E. Dakota Ave.
San Dimas, California 91773

July 23, 2018

18-0924-01

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July 23, 2018

Marangston Inc.
300 E. Dakota Ave.
San Dimas, California 91773

Attention: Mr. Kirk Wallace

Subject: Geotechnical Investigation and Major Geologic Constraints Report Update
Parcel Map 18954
Southwest corner of Snowdrop Road and Haven Avenue
Rancho Cucamonga, CA

Dear Mr. Wallace:

In accordance with your request, a geotechnical investigation and a major geologic constraints report update has been completed for the above referenced project. The report addresses both engineering geologic and geotechnical conditions. The results of the investigation are presented in the accompanying report, which includes a description of site conditions, results of our field exploration, laboratory testing, conclusions, and recommendations.

We appreciate this opportunity to be of service to you. If you have any questions regarding this report, please do not hesitate to contact us at your convenience.

Respectfully submitted,

RMA Group

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1.00 INTRODUCTION

1.01 Purpose

A geotechnical investigation has been completed for a proposed residential tract at the southwest corner of Haven Avenue and Snowdrop Road in Rancho Cucamonga, California. The purpose of the investigation was to summarize geotechnical and geologic conditions at the site, to assess their potential impact on the proposed development, and to develop geotechnical and engineering geologic design parameters.

1.02 Scope of the Investigation

The general scope of this investigation included the following:

- Review of published and unpublished geologic, seismic, groundwater and geotechnical literature.
- Review of prior geologic and geotechnical reports for the site and adjoining properties
- Examination of aerial photographs.
- Contacting of underground service alert to locate onsite utility lines.
- Logging and sampling of 7 exploratory trenches excavated and backfilled with a backhoe.
- Laboratory testing of a representative bedrock sample.
- Geotechnical evaluation of the compiled data.
- Preparation of this report presenting our findings, conclusions and recommendations.

Our scope of work did not include a preliminary site assessment for the potential of hazardous materials onsite.

1.03 Site Location and Description

The site consists of approximately 4.3 acres of land located in the foothills of the San Gabriel Mountains in an unincorporated area of San Bernardino County north of Rancho Cucamonga, California. It is situated at the southwest corner of Haven Avenue and Snowdrop Road. Its geographic position is at Latitude 34.1714° and Longitude -117.5763°. The approximate location of the site is illustrated on the accompanying Site Location and Earthquake Fault Zone Map (Figure 1).

At the time of investigation the site was vacant and undeveloped with the exception of an old concrete pad. Topography through the majority of the property slopes to the south at a gradient of about 7%, except where interrupted by east-west trend graded slopes of about 5 to 10 feet in height. The west and east sides of the site descend into ravines.

Vegetation on the site consists of chaparral and a few eucalyptus trees. Substantial portions of the site, especially on the western side of the property, are essentially devoid of vegetation as a result of prior grading.

1.04 Current and Past Land Usage

Our field investigation revealed there is no current land usage.

As documented in a 2003 geologic report we prepared, the prior property owner reported that soil and bedrock materials were previously exported from the site to the northeast of the site to construct a flood control levee for the Deer Creek Canyon Wash. Comparison of elevations shown of the U. S. Geological Survey quadrangle map of the area and current site elevations suggests that the original site topography was lowered by as much as about 25 feet. A 1966 aerial photograph shows that Snowdrop Road formerly passed through the site. We found no visible remnant of the road within the site, although there is a remnant road cut to the northwest of the site. Aerial photographs taken in 1980 and later show Snowdrop Road had been relocated to its present location north of the site. There are no man-made structures within the site other than a concrete pad which the previous owner reported was used as the floor for a large tent that had been set up on the property by a church. The tent is visible on a 1980 aerial photograph we reviewed.

Aerial photographs indicate that the site was partially graded in 2006 or 2007. The grading apparently consisted of placement of up to about 9 feet of fill in the western and southern portions of the site. The fill is thickest in the center of the site, where a slope of up to 9 feet in height is located. It appears the grading was never finished and we know of no soil report documenting the placement and compaction of fill soils or other grading activities.

1.05 Planned Usage

Based on a review of a rough grading plan prepared by Cubit Engineering, we understand that Parcel Map 18954 will be subdivided into four residential lots ranging in size from 1.00 to 1.24 acres. Three lots will have 5,000 sq. ft. building pads and one lot with have a 4,000 sq. ft. building pad. The lots will be created by cut and fill grading. Maximum depths of cut and fill will be on the order of 10 feet. Cut and fill slopes are proposed at a gradient of 2:1 (horizontal to vertical). Maximum cut and fill slopes heights will be on the order of 10 feet. The proposed grading will involve about 8,100 cubic yards of cut and 6,500 cubic yards of fill (assuming 20% shrinkage). Drainage retention basins are also proposed for each lot.

Our investigation was performed prior to the preparation of architectural or foundation plans. To aid in preparation of this report, we utilized the following assumption:

- Residential structures will be one to two stories in height built on graded pads without basements.
- Maximum foundation loads of 2 to 3 kips per linear foot for continuous footings and 60 kips for isolated spread footings.

1.06 Investigation Methods

Our investigation consisted of office research, field exploration, laboratory testing, review of the compiled data, and preparation of this report. It has been performed in a manner consistent with generally accepted engineering and geologic principles and practices, and has incorporated applicable requirements of California Building Code. Definitions of technical terms and symbols used in this report include those of the ASTM International, the California Building Code, and commonly used geologic nomenclature.

Technical supporting data are presented in the attached appendices. Appendix A presents a description of the methods and equipment used in performing the field exploration and logs of subsurface exploration. Appendix B presents a description of our laboratory testing and the test results. Standard grading specifications and references are presented in Appendices C and D, respectively.

2.00 Findings

2.01 Geologic Setting

The site is located in the foothills of the San Gabriel Mountains. The foothills are composed of a complex series of igneous and metamorphic rocks that are in places overlain by younger and older alluvial fan deposits. The Cucamonga Fault is the most significant structural feature in the area. It occurs along the southern front of the San Gabriel Mountains and it separates basement rocks of the San Gabriel Mountains from alluvial deposits. The general geologic setting of the site is illustrated on the accompanying Regional Geologic Map (Figure 2).

2.02 Summary of Prior Geologic and Geotechnical Reports

RMA Group prepared at geologic report for the site and adjoining land to the south and west in 2003. At that time the subject site was known as Parcel 1 of Tentative Parcel Map 15821. The adjoining land was known as Parcels 2, 3 and 4 of Tentative Parcel Map 15821. The report was prepared in response to County of San Bernardino intake review requirements contained in a December 9, 2002 Interoffice Memo prepared by Matthew Slowik, Senior Associate Planner III. The memo required a review of major geologic constraints.

The RMA Group report documented that the eastern side of Tentative Parcel Map 15821 is located within the boundaries of an Alquist-Priolo Earthquake Fault Zone for fault rupture hazard which was established along the regional trend of the Cucamonga fault. It should be noted that the Cucamonga fault is actually mapped to the east of the property and that the site is located within the buffer zone along the mapped fault trace. To evaluate the potential for future fault rupture within the site, two exploratory trenches were excavated across Parcel 1 in west to east directions. The trenches, which were 260 and 187 feet long extended to depths of about 4 to 8 feet. Both trenches were extended into bedrock. No faults were found. The report was reviewed by the County of San Bernardino Land Use Services Department 2004. The review requested additional information about lineament identified in Parcels 2 and 3, but additional information was not requested with respect to the subsurface investigation performed within Parcel 1 (the subject site).

The County review comments were addressed in a 2007 geologic surface fault rupture report prepared by RGS Engineering Geology. That investigation included logging of a 525-foot long trench excavated across the entire width of the Earthquake Fault Zone within Parcel 4 (adjoining to the south of the subject site). Faulting was not found in the RGS trench and RGS report concluded that the east side of Tract 15821 (the area of the Earthquake Fault Zone) is not traversed by faults.

A County review of the RGS report requested additional information about features found in Parcels 2 and 3 (west of the subject site), but no additional information was requested about the investigation of the Earthquake Fault Zone or Parcel 1. A response was prepared by RGS in 2007. There were no additional conclusions or recommendations pertaining to the Earthquake Fault Zone or Parcel 1.

RMA Group prepared a geotechnical investigation report for Parcels 2 and 3 in 2009. The report, which did not include Parcel 1, was conditionally approved by the County of San Bernardino.

Copies of the RMA fault trench logs within Parcel 1 and the RGS fault trench log within Parcel 4 (the Earthquake Fault Zone) are presented in Appendix B. The approximate locations of the previously excavated trenches within the site are shown on Figure 3. Our interpretations of subsurface conditions are presented on Figure 4. The approximate locations of the RMA and RGS fault investigation trenches are shown on Figure 5.

2.03 Earth Materials

Our subsurface investigation, mapping and review of geologic literature revealed that the site is underlain by artificial fill, alluvium, older alluvium and metamorphic bedrock.

- Artificial fill (map symbol af)

  Artificial fill was encountered in the center and western portions of the site, and along the face of the slope south of Snowdrop Road. The fill in the center and western portion of the site ranges from a few inches up to 9 feet in thickness and based upon aerial photographs was placed during uncompleted grading operations in 2006 or 2007. It is uncertain if the fill was placed onsite as stockpiled soil or to construct building pads. Also, fill was encountered around the existing concrete slab in the center of the site. This fill ranged up to 4 feet in thickness and was most likely placed to provide a level pad for the concrete slab.

  In our trenches the fill was found to consist of gray, red, reddish brown, brown and grayish brown silty sand. A small amount of man-made materials is present in the fills, mainly consisting of small pieces of concrete.

  The fills are judged to be non-engineered and will require removal to competent native ground where present within the grading area and elsewhere if required by the reviewing agency. The existing fill soils may be reused for compacted fill provided they are free of vegetation or other deleterious materials.

- Alluvium soils (map symbol Qal)

  Alluvial soils are present in ravine on the western side of the site. These soils were not encountered during our subsurface investigation, but were likely derived from nearby native materials. Alluvial soils are not present within the proposed grading area or building pads.

- Older alluvium (map symbol Qoal)

  Older alluvium is present in the eastern side of the site. It was encountered in Trench T-5 and is exposed in outcrops along the west side of Haven Avenue. The older alluvium consists of red and reddish brown silty sand with gravel and cobbles.

  Morton and Matti (2001) describe the older alluvium as a Unit 2 of a very old (early Pleistocene) alluvial fan having extremely dissected surfaces and Stage S2 soils (see Figure 2). Stage S2 soils have an age of approximately 300,000 to 800,000 years before the present per Figure 4.11 in Bull (1991).

- Metamorphic bedrock (map symbol gc)

  Morton and Matti (2001) classified the bedrock beneath the site as granulitic gneiss, mylonite and cataclasite of possible Proterozoic (Precambrian) age (see Figure 2). Because of the petrographic complexity of this unit, we have utilized a simplified classification of gneiss since rock classification does not impact geotechnical design recommendations.

  In trenches the bedrock consisted of brown and gray, foliated and fractured, moderately hard to hard gneiss. Foliation typically dips to the northwest between about 30 to 50 degrees. Joints are typically steeply inclined. Mineral lineations were noted within the bedrock at many locations.

The approximate distribution of the mapped geologic units is graphically depicted on the accompanying Geologic Map and Cross Sections (Figures 3 and 4).

The subsurface soils and bedrock encountered in the exploratory trenches excavated at the site are described in greater detail on the logs contained in Appendix A.

2.04 Expansive Soils

Expansion testing performed in accordance with ASTM D4829 indicates that earth materials underlying the site have an expansion classification of very low (E.I. = 1).

Results of expansion test and other soil index tests are presented in Appendix B. Since site grading will redistribute earth materials, potential expansive properties should be verified at the completion of rough grading.

2.05 Surface and Groundwater Conditions

No surface water was present within the site or within adjacent ravines at the time of this study was performed.

Surface water was observed flowing in the ravine on the west side of the site during prior investigations of Tentative Parcel Map 15821. In addition, the U.S. Geological Survey Cucamonga Peak quadrangle map shows an offsite spring to the west of the site where the ravine crosses Snowdrop Road.

Groundwater was not encountered within the site during current and prior subsurface exploration. Bedrock underlying the site is generally considered non-water bearing from a water production perspective.

2.06 Faults

The majority of the site is located within the boundaries of an Alquist-Priolo Earthquake Fault Zone for fault-rupture hazard that was established along the Cucamonga fault (see Figures 1, 3 and 5). The Earthquake Fault Zone map shows the site is situated within the buffer zone established to the west of the mapped trace of the Cucamonga fault. However, no faults are known to pass through the site (Figures 1 and 2). The County of San Bernardino and City of Rancho Cucamonga have adopted the State Earthquake Fault Zone without modification in their land planning documents.

As a part of our current investigation, we reviewed prior faulting investigation reports prepared by RMA Group and RGS Engineering Geology, as well as other associated documents. Both investigations included review of pertinent regional geologic data, examination of aerial photographs, geologic field mapping, excavation and logging of trenches within the Earthquake Fault Zone and preparation of a written report.

In summary, the 2003 RMA Group subsurface investigation consisted of two trenches orientated in a west/northwest to east directions. The trenches were extended through surface soil and older alluvium into metamorphic bedrock. The bedrock was observed to be foliated and jointed. Foliation was found to dip to the northwest at varying inclinations, typically between about 30 to 55 degrees. Joints were typically found to dip at high angles in various directions. Faults were not exposed in the trenches. In Trench T-1 a zone of rock striations was observed from Stations 190 to 200. The striations were described as slickensides, however based on additional observations during our current study and descriptions of such features in the subsequently released U.S. Geological Survey Open File Report 2006-1217, it is our opinion these striations are better described as moderately rough and stepped foliation and joint surfaces with linear mineral orientations. The conditions observed at the site are essentially the same as those shown by Photographs 249 and 250 in U.S. Geological Survey Open File Report 2006-1217. Logs of the RMA trenches are presented in Appendix A. Locations of the trenches are shown on Figures 3 and 5.

The RGS subsurface investigation included excavation of a trench that spanned the entire width of the Alquist-Priolo zone west of Haven Avenue. The trench (FT-3) was 529 feet long and was orientated in a southeast to northwest direction. In summary, the trench was extended through topsoil into Pleistocene age older fan deposits or bedrock, except for the last 20+ feet of the trench at its northwest end where Holocene age alluvium was exposed. According to the RGS log, faulting was not exposed by the trench. A copy of the RGS log is presented in Appendix A and the location of the RGS trench is shown on Figure 5.

In conclusion, faulting was not encountered within the Earthquake Fault Zone during the RMA and RGS investigations and neither report recommended a fault setback zone within the subject site.

There are of course many other faults in southern California that could generate earthquakes that could be felt at the site. The accompanying Regional Fault Map (Figure 6) illustrates the location of the site with respect to major faults in the region. The distance to notable faults within 100 kilometers of the site is presented on Table 1.

2.07 Historic Seismicity

The site is located in a seismically active area, as is the case throughout Southern California. Three historic strong earthquakes have been epicentered within about 15 miles of the site. The most recent of these events was the 1990 magnitude 5.5 Upland earthquake, epicentered about 8 miles to the southwest. The other earthquakes with magnitudes of about 6 to 6.4 were epicentered in Lytle Creek and Cajon Pass areas. These events occurred in 1894 and 1899, prior to the development of seismic monitoring networks, and thus their locations and magnitudes are only approximate. Strong earthquakes that have occurred in this region in historic time and their approximate epicentral distances are summarized in Table 2.

Seismic design parameters relative to the requirements of the 2016 California Building Code are presented in Section 3.09.

2.08 Flooding Potential

According to Federal Emergency Management Agency (2016), the site is located within Flood Zone X, which is defined as an "area of minimal flood hazard."

The incised drainage on the west side of the site could, of course, be subject to flooding. However, the proposed building pads will be constructed on a ridge elevated above the drainage course and thus will not be subject to flooding within the ravine. Control of surface runoff within building pads originating from onsite and offsite sources will need to be incorporated to site planning and grading.

2.09 Landslides

The San Bernardino County Geologic Hazards Overlay Map FH20C maps the site in a Generalized Landslide Susceptibility category of low to moderate. However, according to regional geologic maps by Morton (1974 and 1969), Morton and Matti (1997 and 2001) and Morton and Miller (2006) no landslides are known to exist within the site. Topographic landforms suggestive of landslides were not apparent in the field or on aerial photographs and landslides were not encountered during the current or prior subsurface investigations.

3.00 Conclusions and Recommendations

3.01 General Conclusion

Based on specific data and information contained in this report, our understanding of the project and our general experience in engineering geology and geotechnical engineering, it is our professional judgment that the proposed development is geologically and geotechnically feasible. This is provided that the recommendations presented below are fully implemented during design, grading and construction.

3.02 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications outlined in Appendix C, unless specifically revised or amended below. Recommendations contained in Appendix C are general specifications for typical grading projects and may not be entirely applicable to this project.

It is also recommended that all earthwork and grading be performed in accordance with Appendix J of the 2016 California Building Code and all applicable governmental agency requirements. In the event of conflicts between this report and Appendix J, this report shall govern.

3.03 Earthwork Shrinkage, Bulking and Subsidence

Shrinkage is the decrease in volume of soil upon removal and recompaction expressed as a percentage of the original in-place volume. Bulking is an increase in volume determined in the same manner. Subsidence occurs as natural ground is densified to receive fill. These factors account for changes in earth volumes that occur during grading. Our estimates are as follows:

Unit	Shrinkage %, Bulking %	Subsidence (ft)
Topsoil & Existing fill	5 - 10% shrinkage	nil
Older alluvium	0 - 5% shrinkage	0.1
Bedrock	≈5% bulking	nil

It is anticipated that topsoil and existing fill will be completely removed to competent bedrock, thus subsidence will be nil. Shrinkage and subsidence estimates are not provided for alluvium because currently no grading is proposed within areas underlain by alluvium.

The degree to which fill soils are compacted and variations in the insitu density of existing soils and bedrock will influence earth volume changes. Consequently, some adjustments in grades near the completion of grading could be required to balance the earthwork.

3.04 Removals and Overexcavation

All vegetation, trash and debris should be cleared from the grading area and removed from the site. Prior to placement of compacted fills, all non-engineered fills and loose, porous, or compressible soils within the grading area will need to be removed down to competent ground. Removal and requirements will also apply to cut areas, if the depth of cut is not sufficient to reach competent ground. Removed and/or overexcavated soils may be moisture-conditioned and recompacted as engineered fill, except for soils containing detrimental amounts of organic material. Estimated depths of removals are as follows:

- Non-engineered fill up to about 9 feet thick was encountered during our subsurface investigation. Complete removal of fill and underlying compressible native soils from grading areas will need to be performed. If other non-engineered fills are encountered during grading, they will also need to be removed along with any underlying compressible native soils.

- Loose, porous and compressible native soils needing removal within the grading area are expected to be only a few feet or less in thickness. Removal the these materials will need to extend to competent older alluvium or bedrock.

- The concrete currently onsite may be processed and placed in the compacted fill or hauled off the site. If the concrete is use as fill material, it must be broken down to approximately 4 to 8-inch particles and mixed thoroughly with on-site soils. No large and flat pieces are to be used for fill.

In addition to the above requirements, overexcavation will also need to meet the following criteria for the building pads, concrete flatwork and pavement areas:

- Footings shall extend to bedrock or shall be undercut, moistened, and compacted as necessary to produce soils compacted to a minimum of 90% relative compaction to a depth equal to the width of the footing below the bottom of the footing or to a depth of 3 feet below the bottom of the footing, whichever is less. Additional overexcavation of footing areas might be necessary because of cut to fill transitions. Should this occur, overexcavation depth will need to conform to the overexcavation detail in Appendix C. Footing areas shall be defined as the area extending from the edge of the footing for a distance of 5 feet. If footing areas are not identified at the time of grading, the entire building pad within each parcel should be overexcavated as recommended above.

- All floor slabs, concrete flatwork and paved areas shall be underlain by a minimum of 12 inches of soil compacted to a minimum of 90% relative compaction.

The exposed soils beneath all overexcavation should be scarified an additional 12 inches, moisture conditioned and compacted to a minimum of 90% relative compaction.

The above recommendations are based on the assumption that earth materials encountered during field exploration are representative of soils throughout the site. However, there can be unforeseen and unanticipated variations in soils between points of subsurface exploration. Hence, overexcavation depths must be verified, and adjusted if necessary, at the time of grading. The overexcavated materials may be moisture-conditioned and re-compacted as engineered fill.

3.05 Rippability and Rock Disposal

Our exploratory trenches were advanced without difficulty and no oversize materials (greater than 12 inches in maximum dimension) were encountered in our subsurface investigation. Accordingly we expect that all earth materials to the currently proposed depths of grading will be rippable with conventional heavy duty grading equipment and oversized materials are not expected. Since the site is underlain by surface and near surface bedrock, it is possible that excavation difficulties and generation of oversize materials could occur if deep excavations are made.

3.06 Subdrains

Groundwater was not encountered during our current or prior subsurface investigations at the site. Surface water was not present during our current site investigation, but was present in the ravine on the west side of the site during our prior investigation of the property. However, since the proposed grading will not place any fill in that ravine, installation of a canyon subdrain will not be necessary.

3.07 Natural, Fill and Cut Slopes

No landslides are known to exist within the site and no landslides were encountered during the current or prior subsurface investigations. Consequently, natural slopes within the site are judged to be grossly stable.

All fill and cut slopes should be inclined no steeper than 2:1 (horizontal to vertical). Fill slopes will need to be keyed and benched into competent bedrock or competent older alluvium as shown on the detail contained in Appendix C. The fill soils will need to be compacted to a least 90 percent relative compaction to the face of the slopes.

Field data indicates foliation will dip into proposed cut slopes. Therefore, it is anticipated cut slopes will be grossly stable, subject to verification of anticipated geologic conditions at the time of grading.

Typically slope stability calculations are performed for slopes steeper than 2:1 and/or higher than 30 feet. Since the proposed maximum slope height is on the order of 10 feet and slopes will be inclined at 2:1 or flatter, slope stability calculations were not performed.

3.08 Faulting

Prior geologic fault investigations within the site and adjoining property to the south included excavation and logging of trenches across the Alquist-Priolo Zone. The trenches extended into very old (early Pleistocene) alluvium and bedrock of probable Proterozoic age. Since the trenches spanned the entire width of the Alquist-Priolo Zone crossing the site and faults were not encountered within pre-Holocene geologic materials, we conclude the site is not impacted by active faulting (as defined by the State of California) and a fault setback zone is not recommended.

3.09 Seismic Design Parameters

The potential damaging effects of regional earthquake activity must be considered in the design of structures.

Mapped seismic design parameters have been developed in accordance with Section 1613 of the 2016 California Building Code (CBC) using the online U.S. Geological Survey Seismic Design Maps Calculator (ASCE 10 Standard), a site location based on latitude and longitude, and site class of C based on the U.S. Geological Survey online Vs30 value for Strong Motion Station 615RCU (Vs30 = 432 m/sec), which is located about 1,000 feet southwest of the site.

The parameters generated for the subject site are presented below:

2016 California Building Code (CBC) Seismic Parameters

Parameter	Value
Site Location	Latitude = 34.1714 degrees Longitude = -117.5763 degrees
Site Class	Site Class = C Soil Profile Name = very dense soil & soft rock
Mapped Spectral Accelerations (Site Class B)	S_s (0.2- second period) = 3.054g S_1 (1-second period) = 1.127g
Site Coefficients (Site Class C)	F_a = 1.000 F_v = 1.300
Risk-Targeted Maximum Considered Earthquake Spectral Accelerations (Site Class C)	S_{MS} (0.2- second period) = 3.054g S_{M1} (1-second period) = 1.465g
Risk-Targeted Design Earthquake Spectral Accelerations (Site Class C)	S_{DS} (0.2- second period) = 2.036g S_{D1} (1-second period) = 0.977g

The above table shows that the mapped spectral response acceleration parameter at 1-second period (S_1) ≥ 0.75g. Therefore, for Risk Category II, the Seismic Design Category is E. Consequently, as required for Seismic Design Categories C through F by CBC Section 1803.5.11, slope instability, liquefaction, total and differential settlement, and surface displacement by faulting or seismically lateral spreading or lateral flow have been evaluated. Applicable portions of CBC Section 1803.5.12 have also been evaluated including dynamic lateral loading of retaining walls.

Peak earthquake ground acceleration adjusted for site class effects (PGA_M) has been calculated in accordance with ASCE 7-10 Section 11.8.3 as follows: $PGA_M = F_{PGA} \times PGA = 1.000 \times 1.190 = 1.19g$.

3.10 Liquefaction and Secondary Earthquake Hazards

Potential secondary seismic hazards that can affect land development projects include liquefaction, tsunamis, seiches, seismically induced settlement, seismically induced flooding and seismically induced landsliding.

Liquefaction

Liquefaction is a phenomenon where earthquake-induced ground motions increase the pore pressure in saturated, granular soils until it is equal to the confining, overburden pressure. When this occurs, the soil can completely lose its shear strength and enter a liquefied state. The possibility of liquefaction is dependent upon grain size, relative density, confining pressure, saturation of the soils, and intensity and duration of ground shaking. In order for liquefaction to occur, three criteria must be met: underlying loose sandy soils, a groundwater depth of less than about 50 feet, and a potential for seismic shaking from nearby large-magnitude earthquake.

Because of the presence of shallow bedrock, liquefaction is not a hazard at this site.

It should be noted that the California Geological Survey has not yet prepared a Seismic Hazard Zone Map of potential liquefaction hazards for the quadrangle in which the site is located.

Tsunamis and Seiches

Tsunamis are sea waves that are generated in response to large-magnitude earthquakes. When these waves reach shorelines, they sometimes produce coastal flooding. Seiches are the oscillation of large bodies of standing water, such as lakes, that can occur in response to ground shaking. Tsunamis and seiches do not pose hazards due to the inland location of the site and lack of nearby bodies of standing water.

Seismically Induced Settlement

Seismically induced settlement occurs most frequently in areas underlain by loose, granular sediments. Damage as a result of seismically induced settlement is most dramatic when differential settlement occurs in areas with large variations in the thickness of underlying sediments. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Because of the presence of shallow bedrock, seismically induced settlement is not a concern at this site.

Seismically Induced Flooding

There are no water reservoirs or dams located up-gradient of the site. Therefore, the potential for seismically induced flooding at the site is nil.

Seismically Induced Landsliding

Due to the absence of existing landslides within the site and the relatively low inclinations of existing slopes, the potential for seismically induced landsliding is judged to be very low. This assumes that any slopes created during development of the site will be properly designed and constructed.

It should be noted that the California Geological Survey has not yet prepared a Seismic Hazard Zone Map of potential earthquake-induced landslide hazards for the quadrangle in which the site is located.

3.11 Foundations

Isolated spread footings and/or continuous wall footings are recommended to support the proposed structures. If the recommendations in the section on grading are followed and footings are established in bedrock or compacted fill materials, footings may be designed using the following allowable soil bearing values:

- Continuous Wall Footings:

  Footings having a minimum width of 12 inches and a minimum depth of 12 inches below the lowest adjacent grade have allowable bearing capacity of 1,500 pounds per square foot (psf). This value may be increased by 10% for each additional foot of width and/or depth to a maximum value of 3,500 psf.

- Isolated Spread Footings:

  Footings having a minimum width of 12 inches and a minimum depth of 18 inches below the lowest adjacent grade have allowable bearing capacity of 2,000 psf. This value may be increased by 10% for each additional foot of width or depth to a maximum value of 3,500 psf.

- Retaining Wall Footings:

  Footings for retaining walls should be founded a minimum depth of 12 inches and have a minimum width of 12 inches. Footings may be designed using the allowable bearing capacity and lateral resistance values recommended for building footings. However, when calculating passive resistance, the upper 6 inches of the footings should be ignored in areas where the footings will not be covered with concrete flatwork. This value may also be increased by 10% for each additional foot of width or depth to a maximum value of 3,500 psf. Reinforcement should be provided for structural considerations as determined by the design engineer.

The above bearing capacities represent an allowable net increase in soil pressure over existing soil pressure and may be increased by one-third for short-term wind or seismic loads. The maximum expected settlement of footings designed with the recommended allowable bearing capacity is expected to be on the order of ½ inch with differential settlement on the order of ¼ inch.

Expansion testing indicates near surface soils at the site have a very low expansion potential. Therefore, reinforcement of footings for expansive soil is not required from a geotechnical perspective. Due to the preliminary nature of the expansion tests performed for this study, we recommend additional testing be performed near the completion of rough grading to verify the test results and recommended foundation design criteria.

3.12 Foundation Setbacks from Slopes

Setbacks for footings adjacent to slopes should conform to the requirements of the California Building Code. Specifically, footings should maintain a horizontal distance or setback between any adjacent slope face and the bottom outer edge of the footing.

For slopes descending away from the foundation, the horizontal distance may be calculated by using h/3, where h is the height of the slope. The horizontal setback should not be less than 5 feet, nor need not be greater than 40 feet per the California Building Code. Where structures encroach within the zone of h/3 from the top of the slope the setback may be maintained by deepening the foundations. Flatwork and utilities within the zone of h/3 from the top of slope may be subject to lateral distortion caused by gradual downslope creep. Walls, fences and landscaping improvements constructed at the top of descending slopes should be designed with consideration of the potential for gradual downslope creep.

For ascending slopes, the horizontal setback required may be calculated by using h/2 where h is the height of the slope. The horizontal setback need not be greater than 15 feet per the California Building Code.

3.13 Slabs on Grade

We recommend that floor slabs have a nominal thickness of 4 inches. Because the underlying soils have a very low expansion potential, reinforcing of slabs on grade for structures is optional from a geotechnical perspective. Floor slabs should be divided into squares or rectangles using weakened plane joints (contraction joints), each with maximum dimensions not exceeding 15 feet. Contraction joints should be made in accordance with American Concrete Institute (ACI) guidelines. If weakened plane joints are not used, then the slabs shall be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab.

Special care should be taken on floors slabs to be covered with thin-set tile or other inflexible coverings. These areas may be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab, to mitigate drying shrinkage cracks. Alternatively, inflexible flooring may be installed with unbonded fabric or liners to prevent reflection of slab cracks through the flooring.

A moisture vapor retarder/barrier is recommended beneath all slabs-on-grade that will be covered by moisture-sensitive flooring materials such as vinyl, linoleum, wood, carpet, rubber, rubber-backed carpet, tile, impermeable floor coatings, adhesives, or where moisture-sensitive equipment, products, or environments will exist. We recommend that design and construction of the vapor retarder or barrier conform to Section 1805 of the 2016 California Building Code (CBC) and pertinent sections of American Concrete Institute (ACI) guidance documents 302.1R-04, 302.2R-06 and 360R-10.

The moisture vapor retarder/barrier should consist of a minimum 10 mils thick polyethylene with a maximum perm rating of 0.3 in accordance with ASTM E 1745. Seams in the moisture vapor retarder/barrier should be overlapped no less than 6 inches or in accordance with the manufacturer's recommendations. Joints and penetrations should be sealed with the manufacturer's recommended adhesives, pressure-sensitive tape, or both. The contractor must avoid damaging or puncturing the vapor retarder/barrier and repair any punctures with additional polyethylene properly lapped and sealed.

ACI guidelines allow for the placement of moisture vapor retarder/barriers either directly beneath floor slabs or below an intermediate granular soil layer.

Placing the moisture retarder/barrier directly beneath the floor slab will provide improved curing of the slab bottom and will eliminate potential problems caused by water being trapped in a granular fill layer. Concrete slabs poured directly on a vapor retarder/barrier can experience shrinkage cracking and curling due to differential rates of curing through the thickness of the slab. Therefore, for concrete placed directly on the vapor retarded, we recommend a maximum water cement ratio of 0.45 and the use of water-reducing admixtures to increase workability and decrease bleeding.

If granular soil is placed over the vapor retarder/barrier, we recommend that the layer be at least 2 inches thick in accordance with traditional practice in southern California. Granular fill should consist of clean fine graded materials with 10 to 30% passing the No. 100 sieve and free from clay or silt. The granular layer should be uniformly compacted and trimmed to provide the full design thickness of the proposed slab. The granular fill layer should not be left exposed to rain or other sources of water such as wet-grinding, power washing, pipe leaks or other processes, and should be dry at the time of concrete placement. Granular fill layers that become saturated should be removed and replaced prior to concrete placement.

An additional layer of sand may be placed beneath the vapor retarder/barrier at the developer's discretion to minimize the potential of the retarder/barrier being punctured by underlying soils.

3.14 Miscellaneous Concrete Flatwork

Miscellaneous concrete flatwork and walkways may be designed with a minimum thickness of 4 inches. Large slabs should be reinforced with a minimum of 6x6-10/10 welded wire mesh placed at mid-height in the slab. Control joints should be constructed to create squares or rectangles with a maximum spacing of 15 feet. Walkways may be constructed without reinforcement. Walkways should be separated from foundations with a thick expansion joint filler. Control joints should be constructed into non-reinforced walkways at a maximum of 5 feet spacing.

The subgrade soils beneath all miscellaneous concrete flatwork should be compacted to a minimum of 90 percent relative compaction for a minimum depth of 12 inches. The geotechnical engineer should monitor the compaction of the subgrade soils and perform testing to verify that proper compaction has been obtained.

3.15 Footing Excavation and Slab Preparations

All footing excavations should be observed by the geotechnical consultant to verify that they have been excavated into competent soils or bedrock. The foundation excavations should be observed prior to the placement of forms, reinforcement steel, or concrete. These excavations should be evenly trimmed and level. Prior to concrete placement, any loose or soft soils should be removed. Excavated soils should not be placed on slab or footing areas unless properly compacted.

Prior to the placement of the moisture barrier and sand, the subgrade soils underlying the slab should be observed by the geotechnical consultant to verify that all under-slab utility trenches have been properly backfilled and compacted, that no loose or soft soils are present, and that the slab subgrade has been properly compacted to a minimum of 90 percent relative compaction within the upper 12 inches.

Footings may experience and overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom of the trench.

Slabs on grade and walkways should be moist prior to the placement of concrete.

3.16 Lateral Load Resistance

Lateral loads may be resisted by soil friction and the passive resistance of the soil. The following parameters are recommended.

- Passive Earth Pressure = 470 pcf (equivalent fluid weight).
- Coefficient of Friction (soil to footing) = 0.43
- Retaining structures should be designed to resist the following lateral active earth pressures:

Surface Slope of Retained Materials (Horizontal:Vertical)	Equivalent Fluid Weight (pcf)
Level	35
5:1	36
4:1	38
3:1	40
2:1	49

These active earth pressures are only applicable if the retained earth is allowed to strain sufficiently to achieve the active state. The required minimum horizontal strain to achieve the active state is approximately 0.0025H. Retaining structures should be designed to resist an at-rest lateral earth pressure if this horizontal strain cannot be achieved.

- At-rest Lateral Earth Pressure = 55 pcf (equivalent fluid weight)

The Mononobe-Okabe method is commonly utilized for determining seismically induced active and passive lateral earth pressures and is based on the limit equilibrium Coulomb theory for static stress conditions. This method entails three fundamental assumptions (e.g., Seed and Whitman, 1970): Wall movement is sufficient to ensure either active or passive conditions, the driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the heel of the wall and extending to the free surface of the backfill, and the driving soil wedge and the retaining structure act as rigid bodies, and therefore, experiences uniform accelerations throughout the respective bodies (U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures).

- Seismic Lateral Earth Pressure = 22 pcf (equivalent fluid weight).

The seismic lateral earth pressure given above is an inverted triangle, and the resultant of this pressure is an increment of force which should be applied to the back of the wall in the upper 1/3 of the wall height.

Per CBC Section 1803.5.12 dynamic seismic lateral earth pressures shall be applied to foundation walls and retaining walls supporting more than 6 feet of backfill. Dynamic seismic lateral earth pressures may also be applied to shorter walls at the discretion of the structural engineer.

3.17 Drainage and Moisture Proofing

Surface drainage should be directed away from the proposed structure into suitable drainage devices. Neither excess irrigation nor rainwater should be allowed to collect or pond against building foundations or within low-lying or level areas of the lot. Surface waters should be diverted away from the tops of slopes and prevented from draining over the top of slopes and down the slope face.

Walls and portions thereof that retain soil and enclose interior spaces and floors below grade should be waterproofed and dampproofed in accordance with CBC Section 1805.

Retaining structures should be drained to prevent the accumulation of subsurface water behind the walls. Backdrains should be installed behind all retaining walls exceeding 3 feet in height. A typical detail for retaining wall back drains is presented in Appendix C. All backdrains should be outlet to suitable drainage devices. Retaining wall less than 3 feet in height should be provided with backdrains or weep holes. Dampproofing and/or waterproofing should also be provided on all retaining walls exceeding 3 feet in height.

3.18 Cement Type and Corrosion Potential

Soluble sulfate tests indicate that concrete at the subject site will have a negligible exposure to water-soluble sulfate in the soil. Our recommendations for concrete exposed to sulfate-containing soils are presented in the table below.

Recommendations for Concrete exposed to Sulfate-containing Soils

Sulfate Exposure	Water Soluble Sulfate (SO_4) in Soil (% by Weight)	Sulfate (SO_4) in Water (ppm)	Cement Type (ASTM C150)	Maximum Water-Cement Ratio (by Weight)	Minimum Compressive Strength (psi)
Negligible	0.00 - 0.10	0-150	--	--	2,500
Moderate	0.10 - 0.20	150-1,500	II	0.50	4,000
Severe	0.20 - 2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan or slag	0.45	4,500

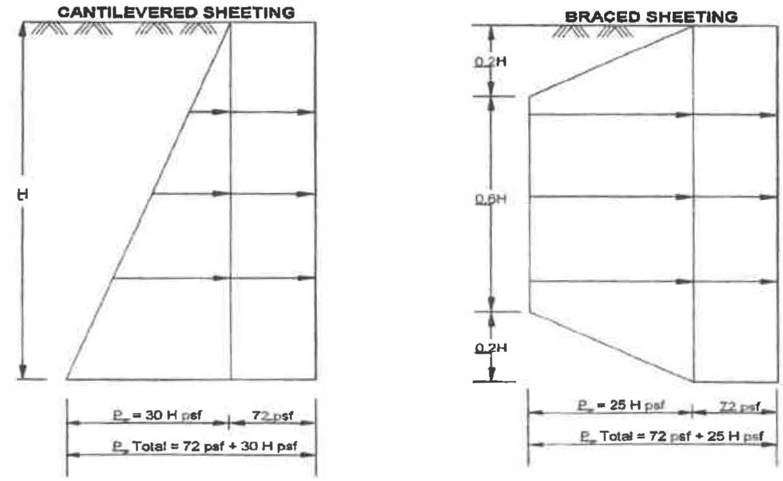
Use of alternate combinations of cementitious materials may be permitted if the combinations meet design recommendations contained in American Concrete Institute guideline ACI 318-11.

The soils were also tested for soil reactivity (pH), electrical resistivity (ohm-cm) and chloride content. The test results indicate that the on-site soils have a soil reactivity of 6.5, an electrical resistivity of 10,000 ohm-cm, and a chloride content of 123 ppm. A neutral or non-corrosive soil has a soil reactivity value ranging from 5.5 to 8.4. Generally, soils that could be considered moderately corrosive to ferrous metals have resistivity values of about 3,000 ohm-cm to 10,000 ohm-cm. Soils with resistivity values less than 3,000 ohm-cm can be considered corrosive and soils with resistivity values less than 1,000 ohm-cm can be considered extremely corrosive. Soil with a chloride content of 500 ppm or greater are generally considered corrosive.

Based on our preliminary analysis, it appears that the underlying onsite soils are slightly to moderately corrosive to ferrous metals. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. As RMA Group Inc. does not practice corrosion engineering, a qualified corrosion engineer may be consulted to further assess the corrosive properties of the soil.

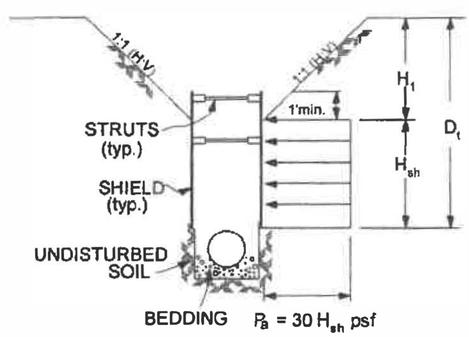
3.19 Temporary Slopes

Excavation of utility trenches will require either temporary sloped excavations or shoring. Temporary excavations in existing alluvial soils may be safely made at an inclination of 1:1 or flatter. If vertical sidewalls are required in excavations greater than 5 feet in depth, the use of cantilevered or braced shoring is recommended. Excavations less than 5 feet in depth may be constructed with vertical sidewalls without shoring or shielding. Our recommendations for lateral earth pressures to be used in the design of cantilevered and/or braced shoring are presented below. These values incorporate a uniform lateral pressure of 72 psf to provide for the normal construction loads imposed by vehicles, equipment, materials, and workmen on the surface adjacent to the trench excavation. However, if vehicles, equipment, materials, etc., are kept a minimum distance equal to the height of the excavation away from the edge of the excavation, this surcharge load need not be applied.



SHORING DESIGN: LATERAL SHORING PRESSURES

Design of the shield struts should be based on a value of 0.65 times the indicated pressure, Pa, for the approximate trench depth. The wales and sheeting can be designed for a value of 2/3 the design strut value.



HEIGHT OF SHIELD, H_{sh} = DEPTH OF TRENCH, D_t , MINUS DEPTH OF SLOPE, H_1

TYPICAL SHORING DETAIL

Placement of the shield may be made after the excavation is completed or driven down as the material is excavated from inside of the shield. If placed after the excavation, some overexcavation may be required to allow for the shield width and advancement of the shield. The shield may be placed at either the top or the bottom of the pipe zone. Due to the anticipated thinness of the shield walls, removal of the shield after construction should have negligible effects on the load factor of pipes. Shields may be successively placed with conventional trenching equipment.

Vehicles, equipment, materials, etc. should be set back away from the edge of temporary excavations a minimum distance of 15 feet from the top edge of the excavation. Surface waters should be diverted away from temporary excavations and prevented from draining over the top of the excavation and down the slope face. During periods of heavy rain, the slope face should be protected with sandbags to prevent drainage over the edge of the slope, and a visqueen liner placed on the slope face to prevent erosion of the slope face.

Periodic observations of the excavations should be made by the geotechnical consultant to verify that the soil conditions have not varied from those anticipated and to monitor the overall condition of the temporary excavations over time. If at any time during construction conditions are encountered which differ from those anticipated, the geotechnical consultant should be contacted and allowed to analyze the field conditions prior to commencing work within the excavation.

Cal/OSHA construction safety orders should be observed during all underground work.

3.20 Utility Trench Backfill

The onsite fill soils will not be suitable for use as pipe bedding for buried utilities. All pipes should be bedded in a sand, gravel or crushed aggregate imported material complying with the requirements of the Standard Specifications for Public Works Construction Section 306-1.2.1. Crushed rock products that do not contain appreciable fines should not be utilized as pipe bedding and/or backfill. Bedding materials should be densified to at least 90% relative compaction (ASTM D1557) by mechanical methods. The geotechnical consultant should review and approve of proposed bedding materials prior to use.

The on-site soils are expected to be suitable as trench backfill provided they are screened of organic matter and cobbles over 12 inches in diameter. Trench backfill should be densified to at least 90% relative compaction (ASTM D1557). On-site granular soils may be water densified initially. Supplemental mechanical compaction methods may be required in finer ground soils to attain the required 90% relative compaction.

All utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Cal/OSHA construction safety orders should be observed during all underground work.

3.21 Preliminary Pavement Section

An R-value test was performed on representative sample of the underlying bedrock in order to provide information for preliminary structural pavement design. A structural section was designed using the procedures outlined in Chapter 630 of the California Highway Design Manual (Caltrans, 2017). This procedure uses the principle that the pavement structural section must be of adequate thickness to distribute the load from the design traffic index (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R-value).

Development of the design traffic indexes on the basis of a traffic study is beyond the scope of this report. To performed calculations, we have used a traffic index of 5 which corresponds to a City of Rancho Cucamonga local street classification. Selection of the final pavement structural section should be based on economic considerations which are beyond the scope of this investigation. Recommended structural sections are as follows:

- <u>Local Street (TI=5, R-Value=59):</u>
  3 inches of asphaltic concrete over
  4 inches of crushed aggregate base

Portland cement concrete (PCC) pavements for areas which are not subject to traffic loads may be designed with a minimum thickness of 4.0 inches of Portland cement concrete on compacted native soils. If traffic loads are anticipated, PCC pavements should be designed for a minimum thickness of 6.0 inches of Portland cement concrete on 4.0 inches of crushed aggregate base.

Final pavement design should be performed upon completion of rough grading based on testing of subgrade soils and an agency specified traffic index value.

Prior to paving, the subgrade soils should be scarified and the moisture adjusted to within 2% of the optimum moisture content. The subgrade soils should be compacted to a minimum of 90% relative compaction. All aggregate base courses should be compacted to a minimum of 95% relative compaction.

3.22 Plan Review

Once a formal grading and foundation plans are prepared for the subject property, this office should review the plans from a geotechnical viewpoint, comment on changes from the plan used during preparation of this report and revise the recommendations of this report where necessary.

3.23 Geotechnical Observation and Testing During Rough Grading

The geotechnical engineer should be contacted to provide observation and testing during the following stages of grading:

- During the clearing and grubbing of the site.
- During the demolition of any existing structures, buried utilities or other existing improvements.
- During excavation and overexcavation of compressible soils.
- During all phases of grading including ground preparation and filling operations.
- When any unusual conditions are encountered during grading.

A final geotechnical report summarizing conditions encountered during grading should be submitted upon completion of the rough grading operations.

3.24 Post-Grading Geotechnical Observation and Testing

After the completion of grading the geotechnical engineer should be contacted to provide additional observation and testing during the following construction activities:

- During trenching and backfilling operations of buried improvements and utilities to verify proper backfill and compaction of the utility trenches.
- After excavation and prior to placement of reinforcing steel or concrete within footing trenches to verify

that footings are properly founded in competent materials.

- During fine or precise grading involving the placement of any fills underlying driveways, sidewalks, walkways, or other miscellaneous concrete flatwork to verify proper placement, mixing and compaction of fills.
- When any unusual conditions are encountered during construction.

4.00 CLOSURE

The findings, conclusions and recommendations in this report were prepared in accordance with generally accepted engineering and geologic principles and practices. No other warranty, either expressed or implied, is made. This report has been prepared for Marangston, Inc. to be used solely for design purposes. Anyone using this report for any other purpose must draw their own conclusions regarding required construction procedures and subsurface conditions.

The geotechnical and geologic consultant should be retained during the earthwork and foundation phases of construction to monitor compliance with the design concepts and recommendations and to provide additional recommendations as needed. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

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FIGURES AND TABLES

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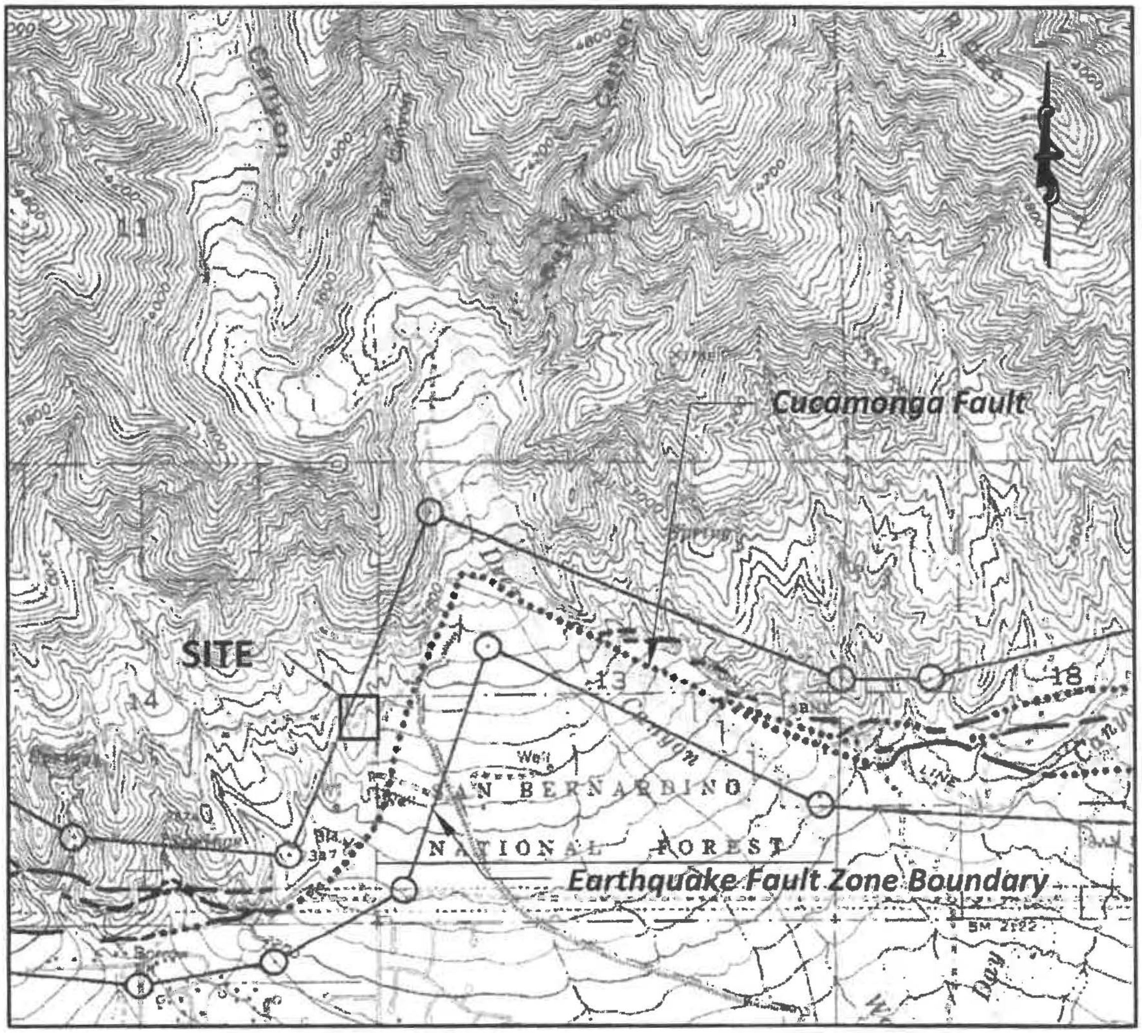
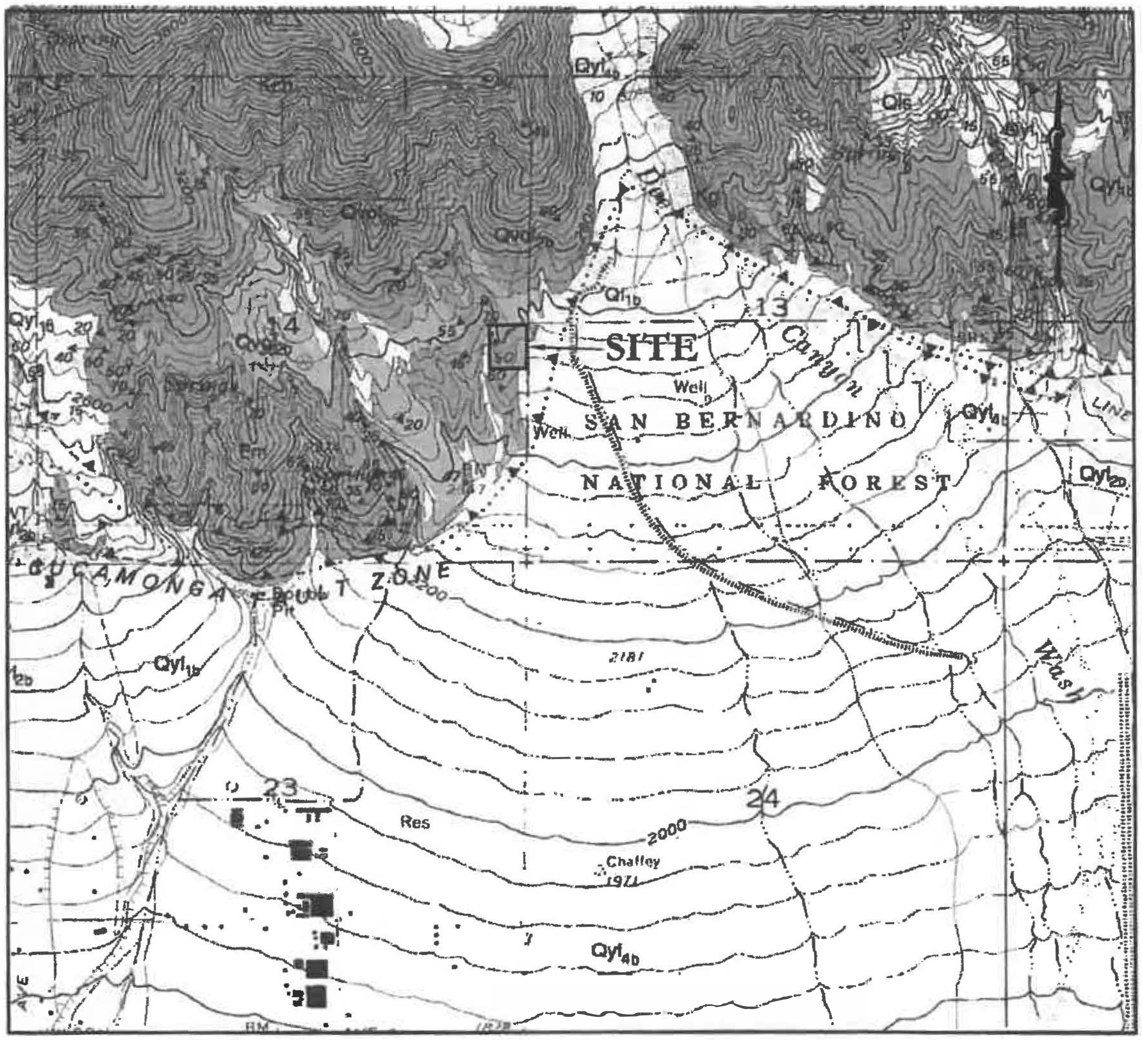
GEOTECHNICAL CONSULTANTS



SITE LOCATION AND EARTHQUAKE FAULT ZONE MAP
Scale: 1" = 2,000'

Base Map: CDMG, Earthquake Fault Zones Map for Cucamonga Peak Quadrangle, 1995.

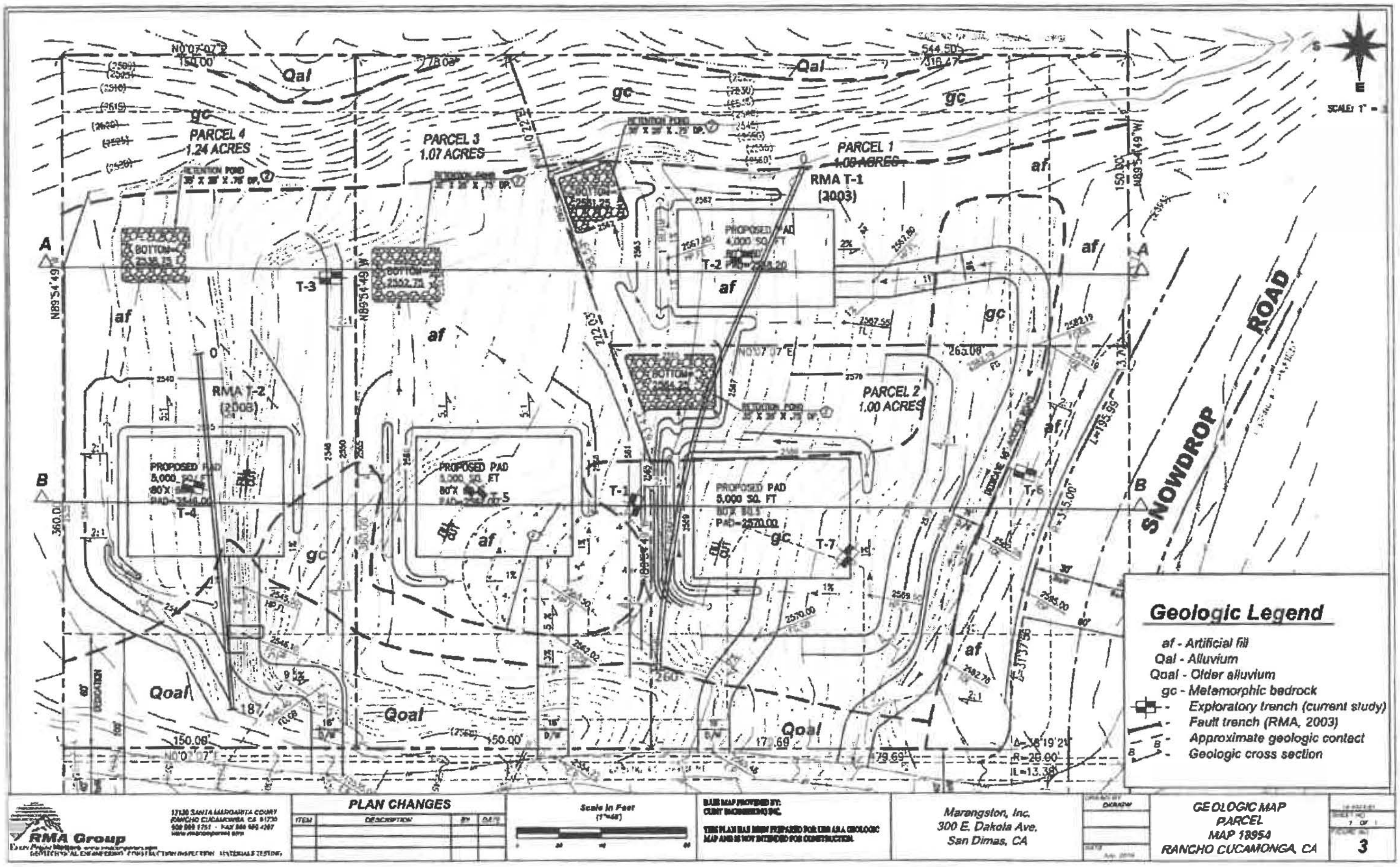


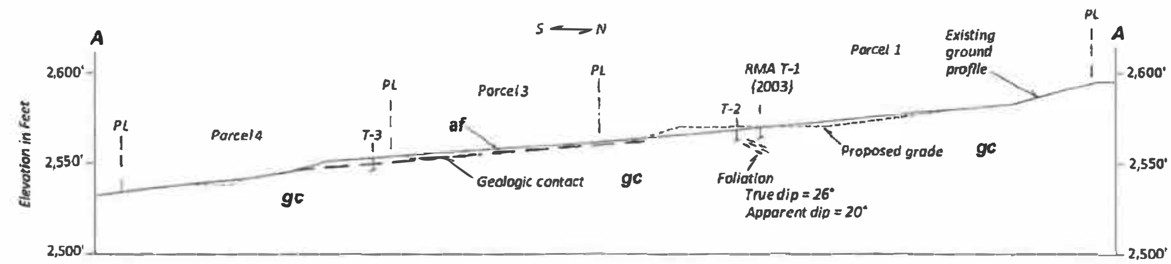
REGIONAL GEOLOGIC MAP
Scale: 1" = 2,000'

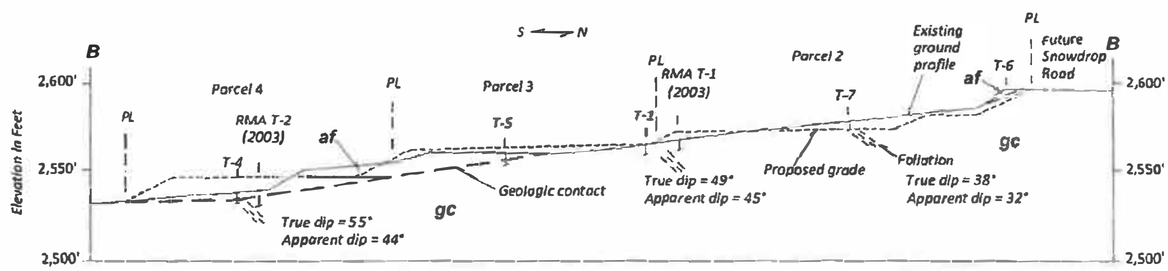
Partial Legend

Qyf - Young alluvial fan deposits
Qvof - Very old alluvial fan deposits
Pm - Proterozoic metamorphic bedrock

Source: Morton, D.M. and Matti, J.C., 2001, Geologic Map of the Cucamonga Peak 7.5' Quadrangle, San Bernardino, CA.


Geologic Legend
af - Artificial fill
Qal - Alluvium
Qoal - Older alluvium
gc - Metamorphic bedrock
Exploratory trench (current study)
Fault trench (RMA, 2003)
Approximate geologic contact
Geologic cross section
PARCEL 4
1.24 ACRES
PARCEL 3
1.07 ACRES
PARCEL 1
1.08 ACRES
PARCEL 2
1.00 ACRES
RMA T-1
(2003)
RMA T-2
(2003)
PROPOSED PAD
4,000 SQ. FT
PROPOSED PAD
5,000 SQ. FT
SNOWDROP
ROAD
RMA Group
PLAN CHANGES
ITEM
DESCRIPTION
Scale in Feet
Marengston, Inc.
300 E. Dakota Ave.
San Dimas, CA
GEOLOGIC MAP
PARCEL
MAP 18954
RANCHO CUCAMONGA, CA
3





RMA Group
12130 SANTA MARGARITA COURT RANCHO CUCAMONGA, CA 91730

GEOTECHNICAL ENGINEERING CONSTRUCTION INSPECTION MATERIALS TESTING

PLAN CHANGES

ITEM	DESCRIPTION	BY	DATE

Holzontal Scale in Feet

Marangaton, Inc.
300 E. Dakota Ave.
San Dimas, CA

GEOLOGIC CROSS SECTIONS
PARCEL
MAP 18954
RANCHO CUCAMONGA, CA

1 OF 1

4

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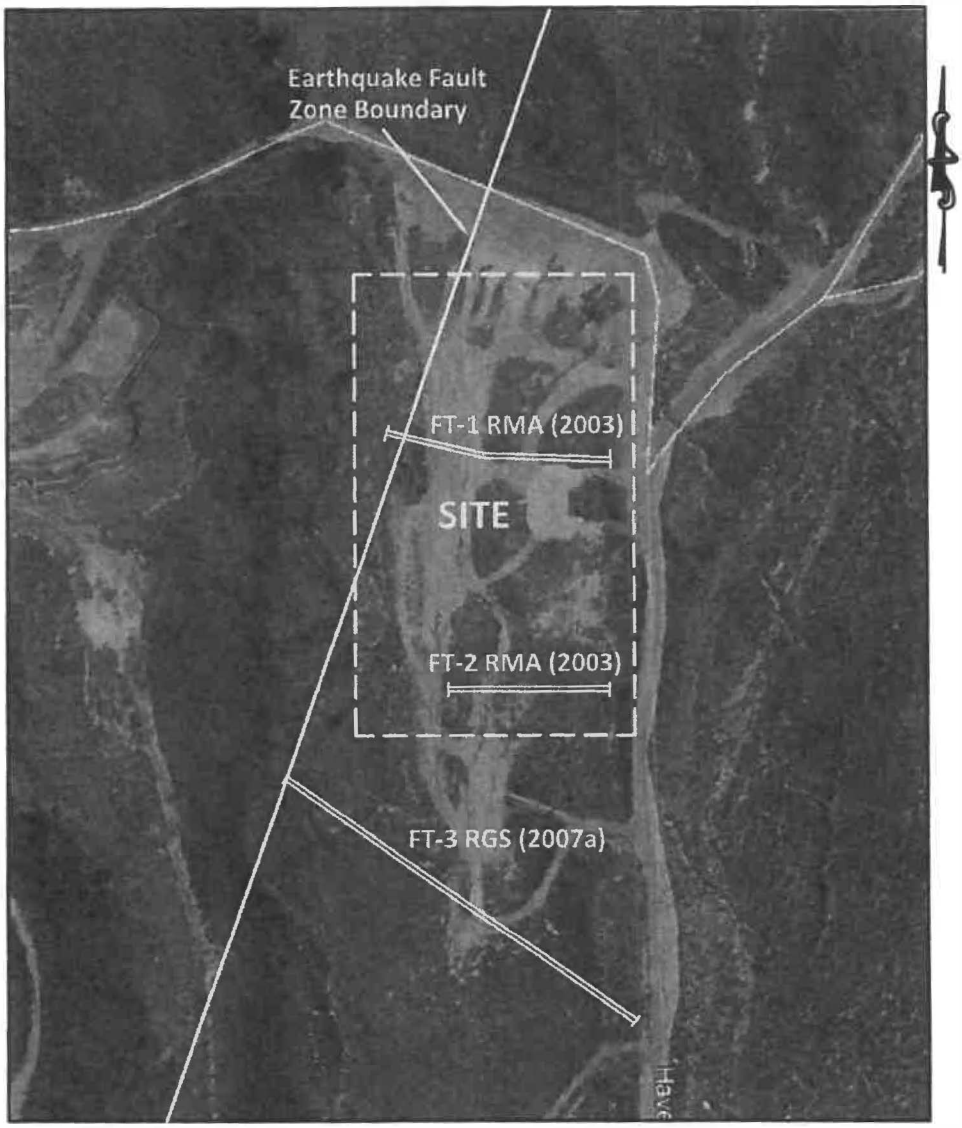
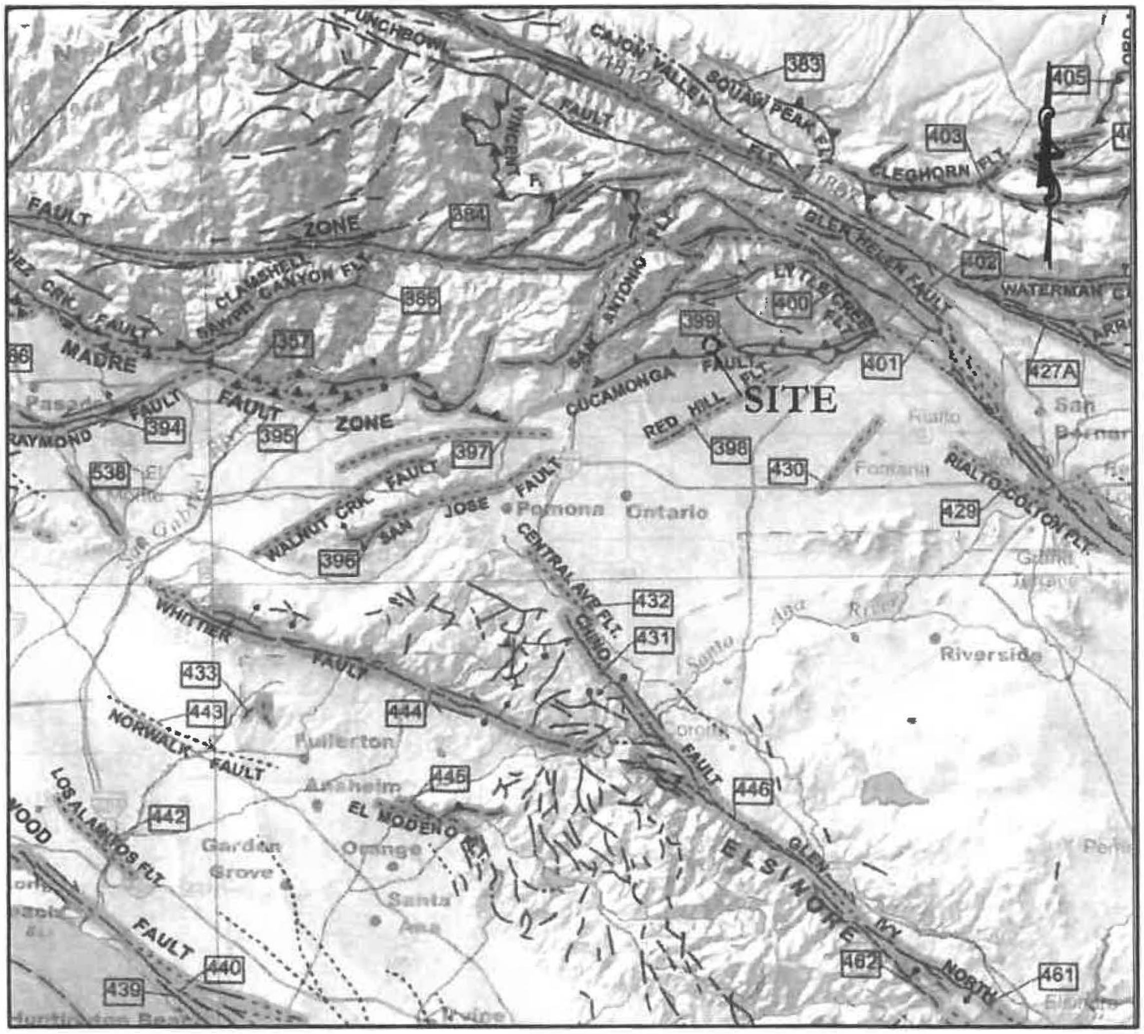
GEOTECHNICAL CONSULTANTS



FAULT TRENCH LOCATION MAP

- Approximate location of prior fault investigation trench

Base Map: Google Earth, 2018



REGIONAL FAULT MAP

Legend

399 -Fault Number in map database
Red - faults showing evidence of displacement during late historic time (last 200 years)
Orange - Fault with Holocene displacement
Green - Fault with late Holocene displacement
Purple - Quaternary fault
Black - Pre-Quaternary fault

Base Map: California Geological Survey, 2010, Fault Activity Map

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NOTABLE FAULTS WITHIN 100 KILOMETERS AND SEISMIC DATA

Fault Zone & geometry	Distance (km)	Distance (mi.)	Maximum Moment Magnitude	Slip Rate (mm/yr)
Chino-Central Ave. (rl-r-o)	22	14	6.7	1.0
Clamshell-Sawpit (r)	26	16	6.5	0.5
Cleghorn (ll-ss)	19	12	6.5	3.0
Cucamonga (r)	0.1	0.1	6.9	5.0
Elsinore (rl-ss)	36	22	6.8	5.0
Gravel Hills-Harper (rl-ss)	98	61	7.1	0.6
Helendale - S Lockhart (rl-ss)	62	39	7.3	0.6
Hollywood (ll-r-o)	61	38	6.4	1.0
Holser (r)	94	58	6.5	0.4
Johnson Valley (rl-ss)	91	57	6.7	0.6
Landers (rl-ss)	96	60	7.3	0.6
Lenwood-Lockhart (rl-ss)	86	53	7.5	0.6
Malibu Coast (ll-r-o)	90	56	6.7	0.3
Newport-Inglewood (rl-ss)	67	42	6.9	1.5
North Frontal - Western (r)	31	19	7.2	1.0
North Frontal - Eastern (r)	74	0	6.7	0.5
Northridge (r)	77	48	7.0	1.5
Palos Verde (rl-ss)	79	49	7.3	3.0
Puente Hills Blind Thrust (r)	38	24	7.1	0.7
Raymond (ll-r-o)	40	25	6.5	1.5
San Andreas (rl-ss)	15	9	7.5	24.0
San Gabriel (rl-ss)	67	42	7.2	1.0
San Jacinto (rl-ss)	10	6	6.7	12.0
San Joaquin Hills (r)	58	36	6.6	0.5
San Jose (ll-r-o)	12	7	6.4	0.5
Santa Monica (ll-r-o)	77	48	6.6	1.0
Santa Susana (r)	86	53	6.7	5.0
Sierra Madre (r)	16	10	7.2	2.0
San Fernando (r)	68	42	6.7	2.0
Upper Elysian Park (r)	50	31	6.4	1.3
Verdugo (r)	53	33	6.9	0.5
Whittier (rl-ss)	35	22	6.8	2.5

Notes:

Fault geometry - (ss) strike slip, (r) reverse, (n) normal, (rl) right lateral, (ll) left lateral, (o) oblique
Fault and Seismic Data - California Geological Survey (Cao), 2003

HISTORIC STRONG EARTHQUAKES IN SOUTHERN CALIFORNIA SINCE 1812

Date	Event	Causitive Fault	Magnitude	Epicentral Distance (miles)
Dec. 12, 1812	Wrightwood	San Andreas?	7.3	21
Jan. 9, 1857	Fort Tejon	San Andreas	7.9	237
Dec. 16, 1858	San Bernardino Area	uncertain	6.0	20
Feb. 9,1890	San Jacinto	uncertain	6.3	92
May 28, 1892	San Jacinto	uncertain	6.3	93
July 30, 1894	Lytle Creek	uncertain	6.0	9
July 22, 1899	Cajon Pass	uncertain	6.4	10
Dec.25, 1899	San Jacinto	San Jacinto	6.7	43
Sept. 20, 1907	San Bernardino Area	uncertain	5.3	28
May 15, 1910	Elsinore	Elsinore	6.0	34
April 21, 1918	Hemet	San Jacinto	6.8	45
July 23, 1923	San Bernardino	San Jacinto	6.0	20
March 11, 1933	Long Beach	Newport-Inglewood	6.4	41
April 10, 1947	Manix	Manix	6.4	83
Dec. 4, 1948	Desert Hot Springs	San Andreas or Banning	6.5	72
July 21, 1952	Wheeler Ridge	White Wolf	7.3	103
Feb. 9, 1971	San Fernando	San Fernando	6.6	51
July 8, 1986	North Palm Springs	Banning or Garnet Hills	5.6	58
Oct. 1, 1987	Whittier Narrows	Puente Hills Thrust	6.0	31
Feb. 28, 1990	Upland	San Jose	5.5	8
June 28, 1991	Sierra Madre	Clamshell Sawpit	5.8	25
April 22, 1992	Joshua Tree	Eureka Peak	6.1	76
June 28, 1992	Landers	Johnson Valley & others	7.3	67
June 28, 1992	Big Bear	uncertain	6.5	44
Jan. 17, 1994	Northridge	Northridge Thrust	6.7	57
Oct. 16, 1999	Hector Mine	Lavic Lake	7.1	83

Notes:

Earthquake data: U.S.G.S. P.P. 1515 & online data, Southern Calif. Earthquake Center & California Geological Survey online data
Magnitudes prior to 1932 are estimated from intensity.
Magnitudes after 1932 are moment, local or surface wave magnitudes.

Site Location:

Site Longitude: - 117.576
Site Latitude: 34.171

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APPENDIX A

FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

A-1.00 FIELD EXPLORATION

A-1.01 Number of Trenches

Our subsurface investigation consisted of 7 trenches excavated with a backhoe. Also included in this appendix are the logs of 2 fault investigation trenches from a previous investigation by RMA Group and the log of 1 fault investigation trench from a previous investigation by RGS Engineering Geology.

A-1.02 Location of Trenches

A Site Geologic Map showing the approximate locations of the trenches is presented as Figure 3.

A-1.03 Trench Logging

Logs of RMA trenches were prepared by RMA geologic staff and are attached in this appendix. The logs contain factual information and interpretation of subsurface conditions between samples. The strata indicated on these logs represent the approximate boundary between earth units and the transition may be gradual. The logs show subsurface conditions at the dates and locations indicated, and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing.

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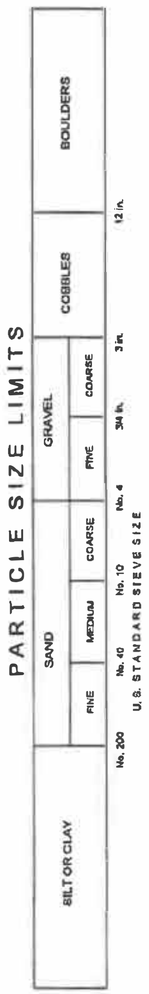

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravel gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	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.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

<u>BOUNDARY CLASSIFICATIONS:</u> Soils possessing characteristics of two groups are designated by combinations of group symbols.

UNIFIED SOIL CLASSIFICATION SYSTEM

I. SOIL STRENGTH/DENSITY

BASED ON STANDARD PENETRATION TESTS

Apparent density of sand		Consistency of clay	
Penetration Resistance N (blows/Ft)	Apparent density	Penetration Resistance N (blows/ft)	Consistency
0-4	Very Loose	<2	Very Soft
4-10	Loose	2-4	Soft
10-30	Medium Dense	4-8	Medium Stiff
30-50	Dense	8-15	Stiff
>50	Very Dense	15-30	Very Stiff
		>30	Hard

N = Number of blows of 140 lb. weight falling 30 in. to drive 2-in OD sampler 1 ft.

BASED ON RELATIVE COMPACTION

Compactness of sand		Consistency of clay	
% Compaction	Compactness	% Compaction	Consistency
<75	Loose	<80	Soft
75-83	Medium Dense	80-85	Medium Stiff
83-90	Dense	85-90	Stiff
>90	Very Dense	>90	Very Stiff

II. SOIL MOISTURE

Moisture of sands		Moisture of clays	
% Moisture	Description	% Moisture	Description
<5%	Dry	<12%	Dry
5-12%	Moist	12-20%	Moist
>12%	Very Moist	>20%	Very Moist, wet

SOIL DESCRIPTION LEGEND

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Exploratory Trench Log

Location: See Site Geologic Map

Elevation: 2,565'

Logged By: KD

Equipment: Backhoe w/24" bucket

Trench No. T-1

Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Gray to red-brown silty fine to coarse sand with minor gravel and small pieces of concrete and red brick, dry, dense.
					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliated with mineral lineations, moderately to highly fractured, dense. Joint attitudes N82E/58SE, N32E/78SE, N87W/63SW
5							Total depth 4 feet No groundwater Trench backfilled
10							
15							

Exploratory Trench Log

Location: See Site Geologic Map

Elevation: 2,567'

Logged By: KD

Equipment: Backhoe w/24" bucket

Trench No. T-2

Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Red-brown silty fine to coarse sand with minor gravel and few surface cobbles, dry, dense.
					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliated with mineral lineations, moderately to highly fractured, dense. Foliation attitudes: N48E/26NW, N532E/21NW Joint attitude: N25E/65SE
5							Total depth 3 feet No groundwater Trench backfilled
10							
15							

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Exploratory Trench Log

Location: See Site Geologic Map | Logged By: KD | **Trench No. T-3**

Elevation: 2,552' | Equipment: Backhoe w/24" bucket | Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Red silty fine to coarse sand with minor gravel, few cobbles, occasional small boulders and few small pieces of red brick, dry, dense. At 2 feet becomes reddish-brown in color.
5					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation, moderately to highly fractured, dense. Total depth 6.5 feet No groundwater Trench backfilled
10							
15							

Exploratory Trench Log

Location: See Site Geologic Map | Logged By: KD | **Trench No. T-4**

Elevation: 2,539' | Equipment: Backhoe w/24" bucket | Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Light reddish-brown silty fine to coarse sand with minor gravel, dry, dense.
5					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation, moderately to highly fractured, dense. Total depth 6 feet No groundwater Trench backfilled
10							
15							

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Exploratory Trench Log

Location: See Site Geologic Map
Elevation: 2,561'

Logged By: KD
Equipment: Backhoe w/24" bucket

Trench No. T-5
Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Grayish-brown silty fine to coarse sand with minor metamorphic bedrock derived gravels and a few cobbles, dry, dense.
					SM		Older alluvium (Qoal): Red silty fine to coarse sand, dry, trace of gravel
5					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation, moderately to highly fractured, dense.
							Total depth 6 feet No groundwater Trench backfilled
10							
15							

Exploratory Trench Log

Location: See Site Geologic Map
Elevation: 2,583'

Logged By: KD
Equipment: Backhoe w/24" bucket

Trench No. T-6
Date Excavated: 6-14-18

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					SM		Artificial fill (af): Light brown to grayish-brown silty fine to coarse sand with minor metamorphic bedrock derived gravels and a few cobbles, few small tree roots, dry, medium dense. Trench excavated on south facing slope on north side of site. Fill thickness ranges from 2 to 4 feet.
5					--		Metamorphic bedrock (gc): Light brown and gray gneiss, moderate to well developed foliation, moderately to highly fractured, dense.
							Total depth 5 feet No groundwater Trench backfilled
10							
15							

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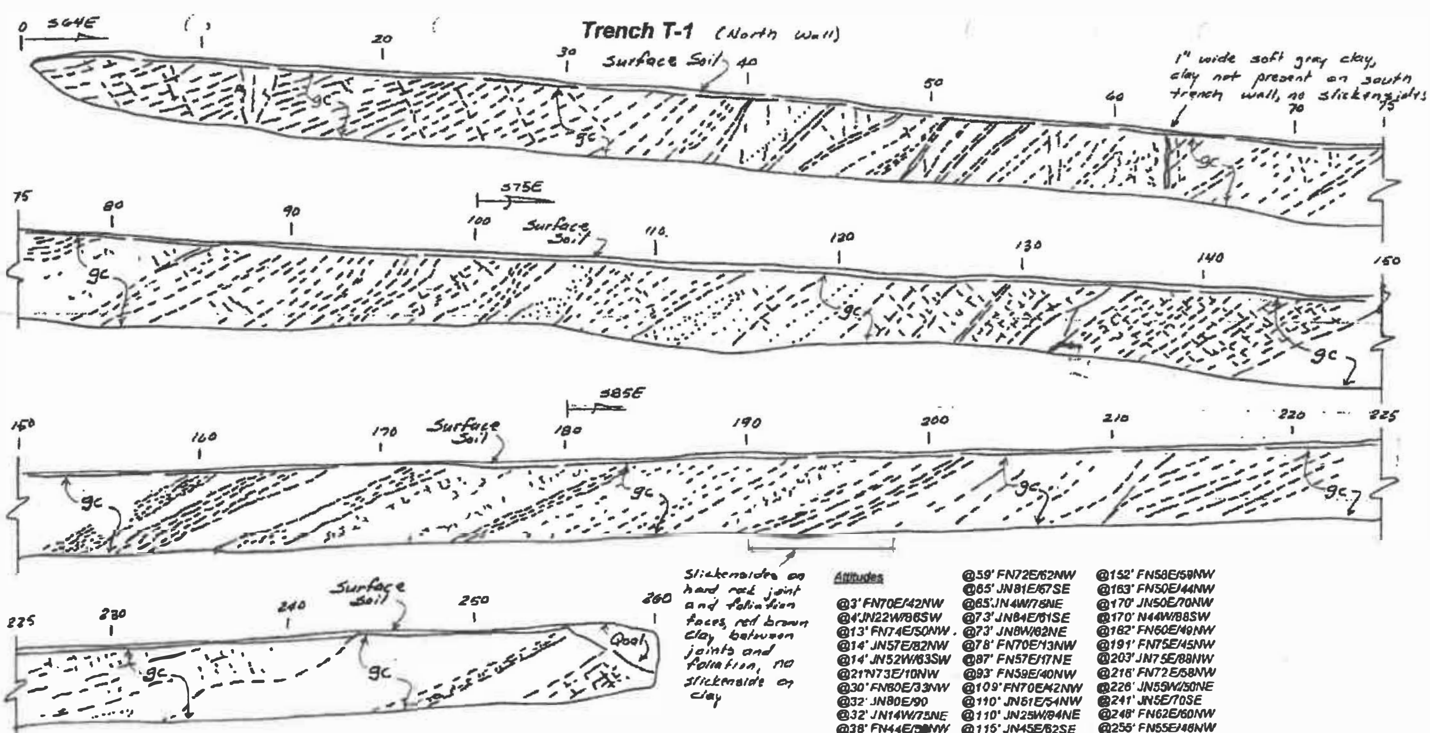
Exploratory Trench Log

Location: See Site Geologic Map
Elevation: 2,577'

Logged By: KD
Equipment: Backhoe w/24" bucket

Trench No. T-7
Date Excavated: 9-12-12

Depth (ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)		USCS	Graphic Symbol	Material Description This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.
					--		Metamorphic bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation with mineral lineations, moderately to highly fractured, dense. Joint attitudes: N4SW/70SW, N68E/62NW Foliation attitude: N53E/38NW
5							Total depth 4 feet No groundwater Trench backfilled
10							
15							



Geologic Units

Surface soil (ripped bedrock): Gray brown gravelly, silty sand

Older alluvium (Qoal): Reddish brown silty sand with gravel, cobbles and a few boulders

Bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation, moderately to highly fractured, dense, hard.

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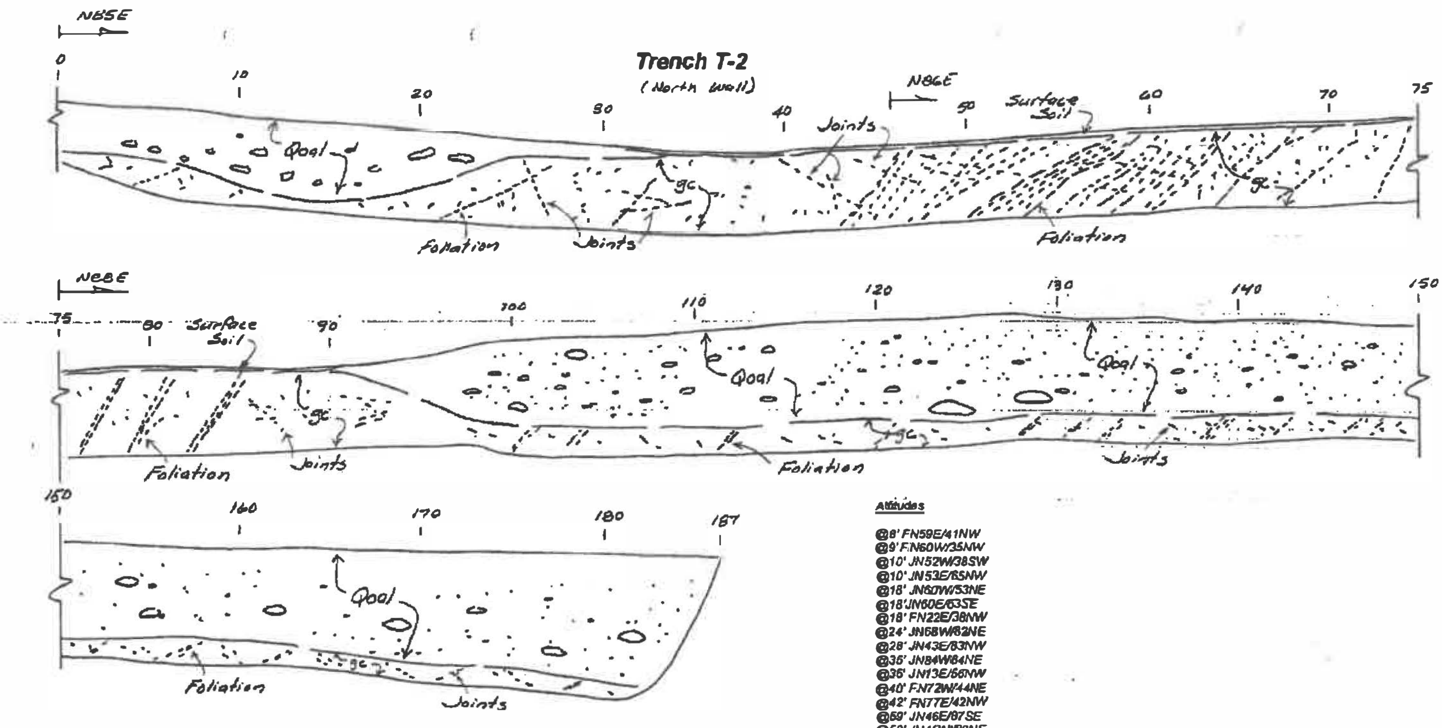
Geologic Trench Logs	
Client: Maricic	Job No.: 03-002-01
Scale: 1" = 5', H = V	Figure No. 4



Attitudes

@8' FN59E/41NW
@9' FN60W/35NW
@10' JN52W/38SW
@10' JN53E/65NW
@18' JN60W/53NE
@18' JN60E/63SE
@18' FN22E/38NW
@24' JN68W/82NE
@28' JN43E/83NW
@35' JN84W/84NE
@36' JN13E/56NW
@40' FN72W/44NE
@42' FN77E/42NW
@59' JN46E/87SE
@59' JN45W/82NE
@65' FN8E/81NE
@77' JN55W/86NE
@78' JN45W/82NE
@84' FN43E/55NW
@92' JN78E/73SE
@92' JN77E/73NW

J = Joints, F = Foliation

Geologic Units

Surface soil (ripped bedrock): Gray brown gravelly, silty sand

Older alluvium (Qoal): Reddish brown silty sand with gravel, cobbles and a few boulders

Bedrock (gc): Orange brown and gray gneiss, moderate to well developed foliation, locally massive, moderately fractured, dense, hard.

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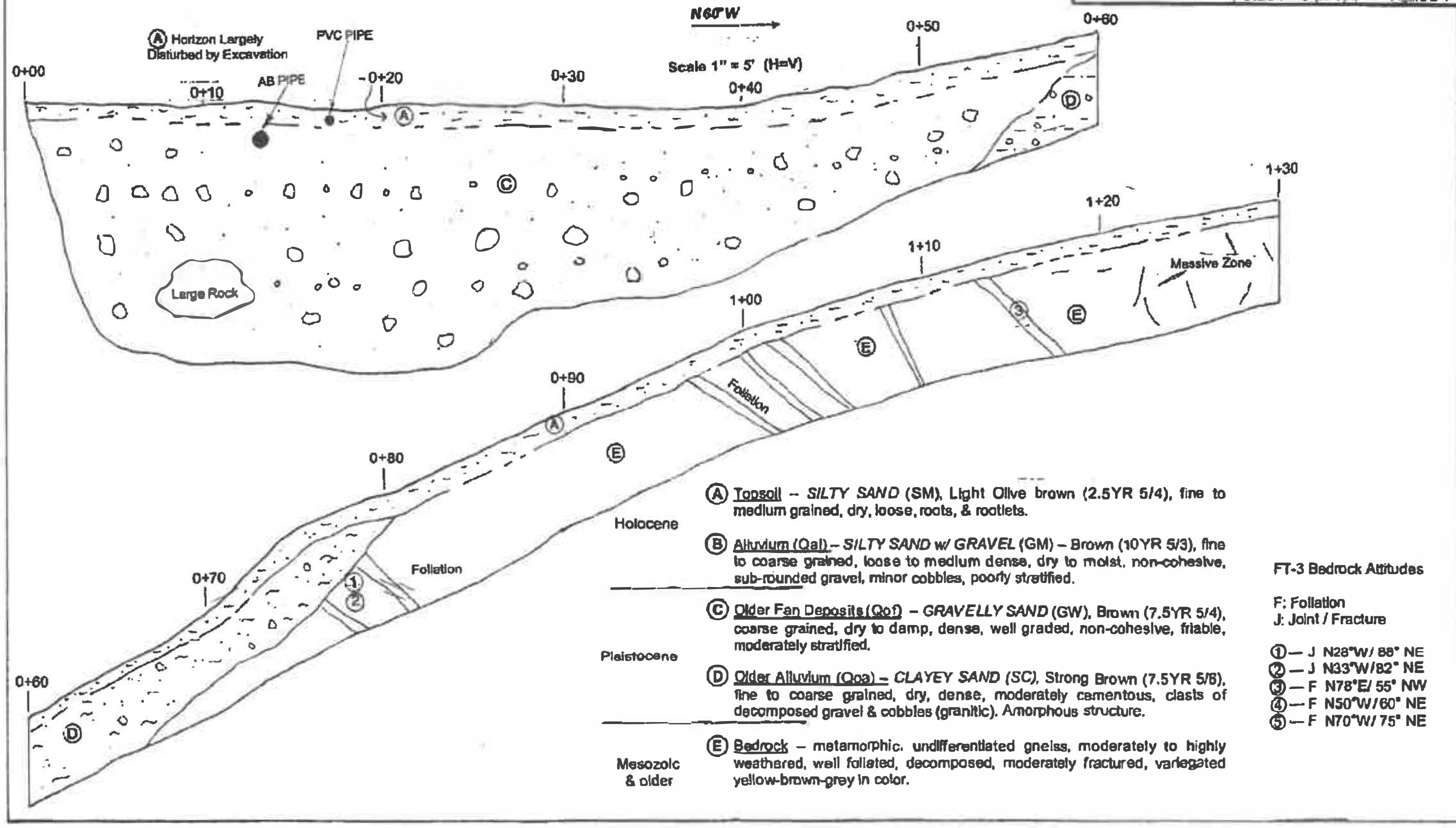
Geologic Trench Logs	
Client: Maricic	Job No.: 03-002-01
Scale: 1" = 5', H = V	Figure No. 5

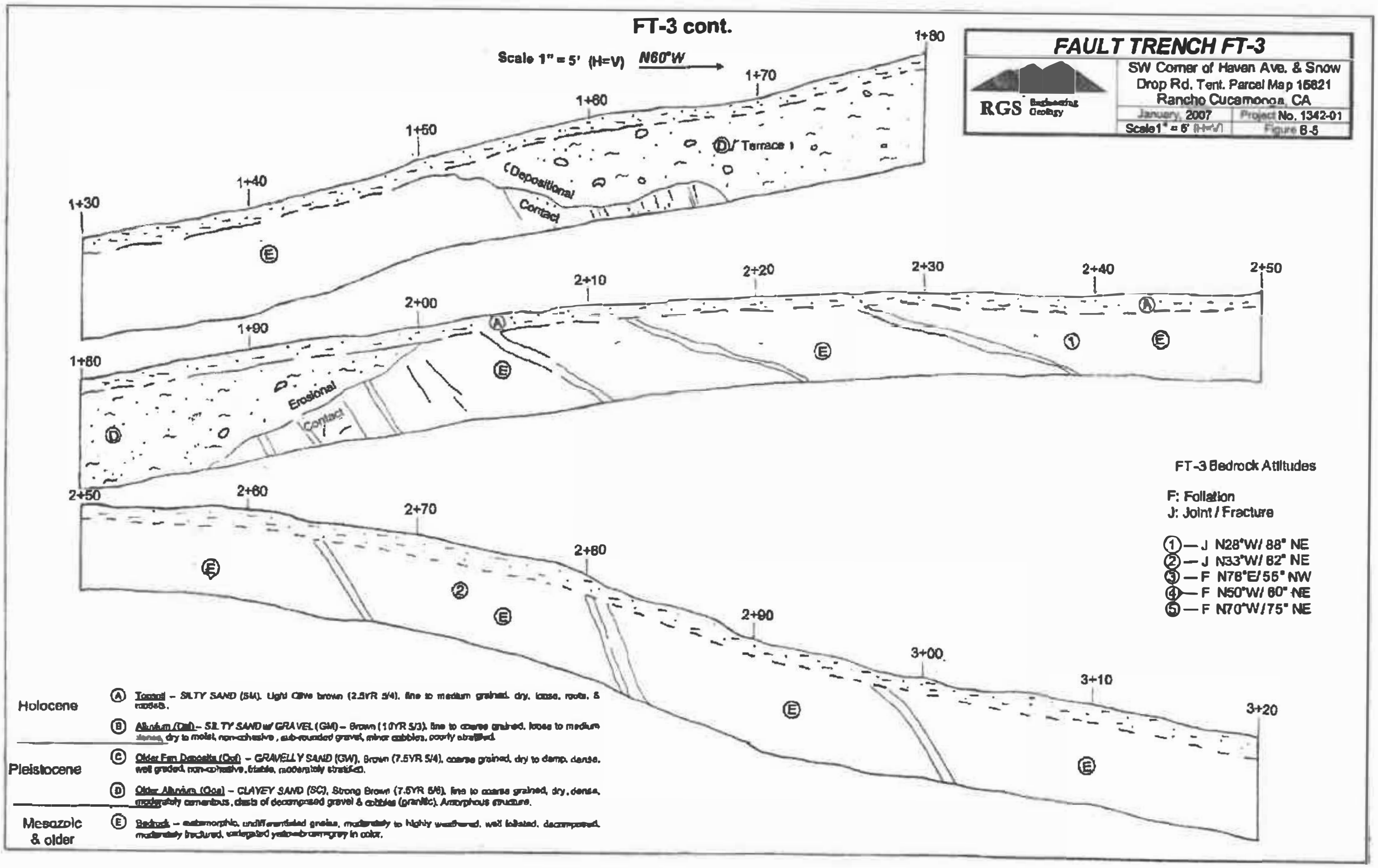
FAULT TRENCH FT-3

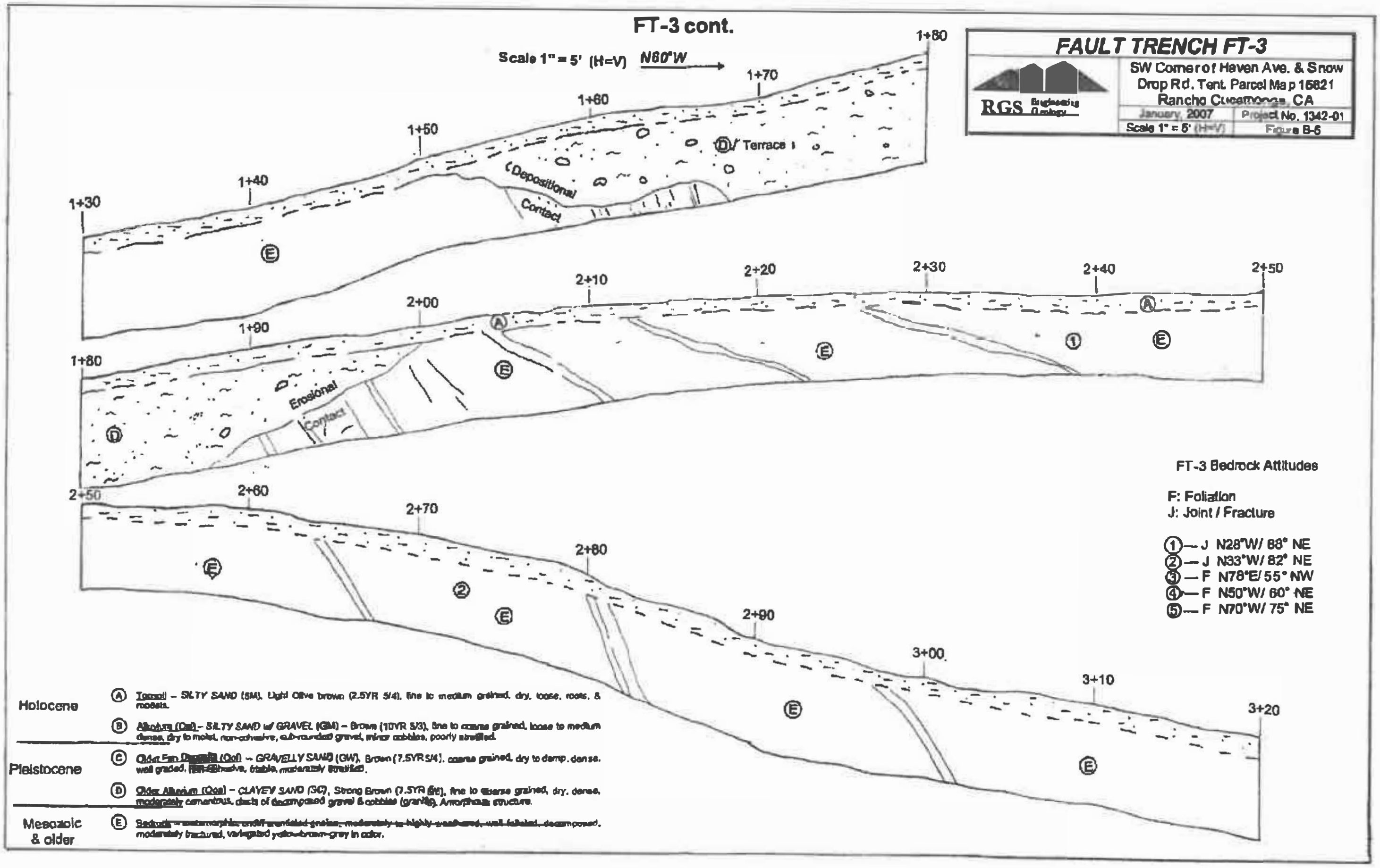
RGS Engineering Geology

SW Corner of Haven Ave. & Snow Drop Rd. Tent. Parcel Map 15821 Rancho Cucamonga, CA

January, 2007	Project No. 1342-01
Scale 1" = 5' (H=V)	Figure B-4

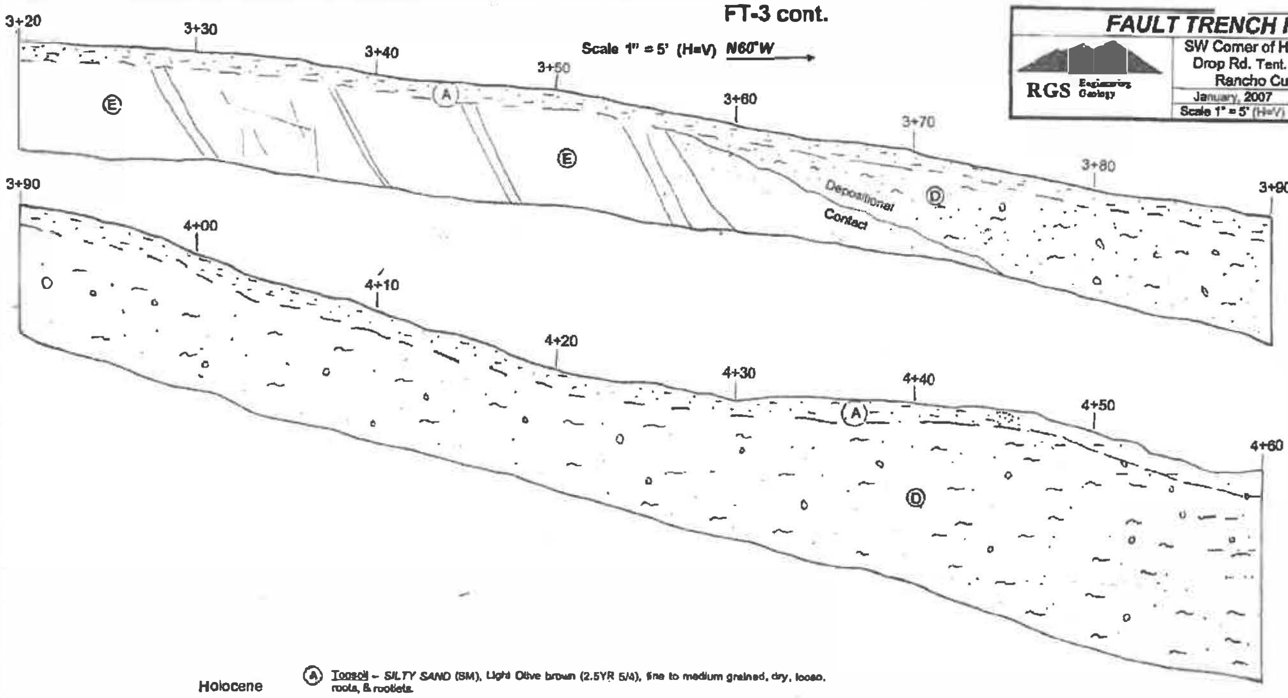



FT-3 cont.
Scale 1" = 5' (H=V)
N60°W
1+30
1+40
1+50
1+60
1+70
1+80
D Terrace
(Depositional Contact
E
1+80
1+90
2+00
2+10
2+20
2+30
2+40
2+50
A
E
1
Erosional Contact
D
2+50
2+60
2+70
2+80
2+90
3+00
3+10
3+20
2
FAULT TRENCH FT-3
RGS Engineering Geology
SW Corner of Haven Ave. & Snow Drop Rd. Tent. Parcel Map 15821
Rancho Cucamonga, CA
January, 2007
Project No. 1342-01
Scale 1" = 5' (H=V)
Figure B-5
FT-3 Bedrock Attitudes
F: Foliation
J: Joint / Fracture
1 — J N28°W/ 88° NE
2 — J N33°W/ 82° NE
3 — F N76°E/ 55° NW
4 — F N50°W/ 60° NE
5 — F N70°W/ 75° NE
Holocene
A Topsoil – SILTY SAND (SM). Light Olive brown (2.5YR 5/4), fine to medium grained, dry, loose, roots, & rootlets.
B Alluvium (Qal) – SILTY SAND w/ GRAVEL (GM) – Brown (10YR 5/3), fine to coarse grained, loose to medium dense, dry to moist, non-cohesive, sub-rounded gravel, minor cobbles, poorly stratified.
Pleistocene
C Older Fan Deposits (Qof) – GRAVELLY SAND (GW), Brown (7.5YR 5/4), coarse grained, dry to damp, dense, well graded, non-cohesive, friable, moderately stratified.
D Older Alluvium (Qoa) – CLAYEY SAND (SC), Strong Brown (7.5YR 5/6), fine to coarse grained, dry, dense, moderately cementous, clasts of decomposed gravel & cobbles (granitic). Amorphous structure.
Mesozoic & older
E Bedrock – metamorphic, undifferentiated gneiss, moderately to highly weathered, well foliated, decomposed, moderately fractured, variegated yellowish-brown-grey in color.


FT-3 cont.
Scale 1" = 5' (H=V)
N60°W
FAULT TRENCH FT-3
RGS
SW Corner of Haven Ave. & Snow Drop Rd. Tent. Parcel Map 15821
January, 2007
Project No. 1342-01
Scale 1" = 5' (H=V)
Figure B-6
1+30
1+40
1+50
1+60
1+70
1+80
D
Terrace
Depositional Contact
E
1+80
1+90
2+00
2+10
2+20
2+30
2+40
2+50
A
Erosional Contact
D
E
1
2+50
2+60
2+70
2+80
2+90
3+00
3+10
3+20
2
FT-3 Bedrock Attitudes
F: Foliation
J: Joint / Fracture
1 — J N28°W/ 88° NE
2 — J N33°W/ 82° NE
3 — F N78°E/ 55° NW
4 — F N50°W/ 60° NE
5 — F N70°W/ 75° NE
Holocene
A Topsoil – SILTY SAND (SM), Light Olive brown (2.5YR 5/4), fine to medium grained, dry, loose, roots, & rootlets.
B Alluvium (Qal) – SILTY SAND w/ GRAVEL (GM) – Brown (10YR 5/3), fine to coarse grained, loose to medium dense, dry to moist, non-cohesive, sub-rounded gravel, minor cobbles, poorly stratified.
Pleistocene
C Older Fan Deposits (Qof) – GRAVELLY SAND (GW), Brown (7.5YR 5/4), coarse grained, dry to damp, dense, well graded, non-cohesive, friable, moderately stratified.
D Older Alluvium (Qoa) – CLAYEY SAND (SC), Strong Brown (7.5YR 5/6), fine to coarse grained, dry, dense, moderately cementous, clasts of decomposed gravel & cobbles (granitic). Amorphous structure.
Mesozoic & older
E Bedrock – metamorphic and/or granitic gneiss, moderately to highly weathered, well foliated, decomposed, moderately fractured, variegated yellow-brown-grey in color.

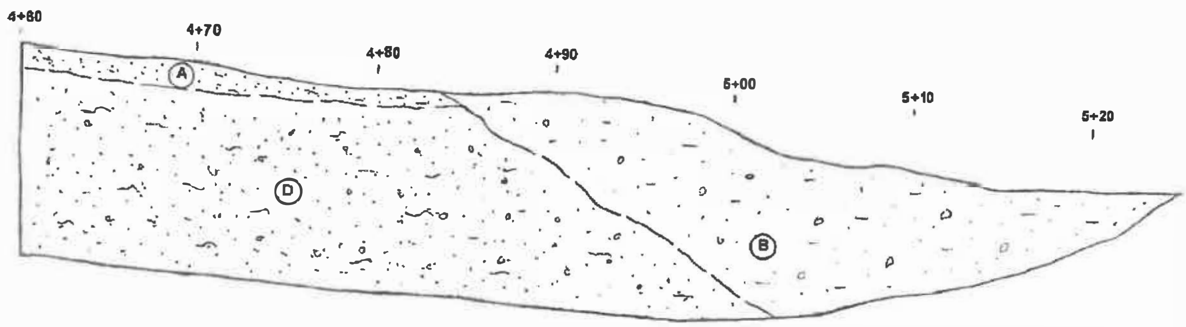
FT-3 cont.

Scale 1" = 5' (H=V) N60°W



FAULT TRENCH FT-3	
RGS Engineering Geology	SW Corner of Haven Ave. & Snow Drop Rd. Tent. Parcel Map 15821 Rancho Cucamonga, CA
January, 2007	Project No. 1342-01
Scale 1" = 5' (H=V)	Figure B-6

Holocene	(A) Topsoil – *SILTY SAND* (SM), Light Olive brown (2.5YR 5/4), fine to medium grained, dry, loose, roots, & rootlets.
	(B) Alluvium (Qal) – *SILTY SAND w/ GRAVEL* – Brown (10YR 5/3), fine to coarse grained, loose to medium dense, dry to moist, non-cohesive, sub-rounded gravel, minor cobbles, poorly stratified.
Pleistocene	(C) Older Fan Deposits (Qof) – *GRAVELLY SAND* (GW), Brown (7.5YR 5/4), coarse grained, dry to damp, dense, well graded, non-cohesive, friable, moderately stratified.
	(D) Older Alluvium (Qoa) – *CLAYEY SAND (SC)*, Strong Brown (7.5YR 5/6), fine to coarse grained, dry, dense, moderately cementous, clasts of decomposed gravel & cobbles (granitic). Amorphous structure.
Mesozoic & older	(E) Bedrock – metamorphic, undifferentiated gneiss, moderately to highly weathered, well foliated, decomposed, moderately fractured, variegated yellow-brown-gray in color.



Holocene	(A) Topsoil – *SILTY SAND* (SM), Light Olive brown (2.5YR 5/4), fine to medium grained, dry, loose, roots, & rootlets.
	(B) Alluvium (Qal) – *SILTY SAND w/ GRAVEL* – Brown (10YR 5/3), fine to coarse grained, loose to medium dense, dry to moist, non-cohesive, sub-rounded gravel, minor cobbles, poorly stratified.
Pleistocene	(C) Older Fan Deposits (Qof) – *GRAVELLY SAND* (GW), Brown (7.5YR 5/4), coarse grained, dry to ~~damp, dense, well graded, non-cohesive, friable, moderately stratified.~~
	(D) Older Alluvium (Qoa) – *CLAYEY SAND (SC)*, Strong Brown (7.5YR 5/6), fine to coarse grained, dry, dense, moderately cementous, clasts of decomposed gravel & cobbles (granitic). Amorphous structure.
Mesozoic & older	(E) Bedrock – metamorphic, undifferentiated gneiss, moderately to highly weathered, well foliated, decomposed, moderately fractured, variegated yellow-brown-grey in color.

FAULT TRENCH FT-3

RGS Engineering Geology

SW Corner of Haven Ave. & Snow Drop Rd. Tent. Parcel Map 15821
Rancho Cucamonga, CA

January, 2007	Project No. 1342-01
Scale 1" = 5' (H=V)	Figure B-7

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APPENDIX B

LABORATORY TESTS

APPENDIX B

LABORATORY TESTS

B-1.00 LABORATORY TESTS

B-1.01 Maximum Density

Maximum density - optimum moisture relationship for a representative soil sample encountered during the field exploration were performed in the laboratory using the standard procedures of ASTM D1557.

B-1.02 Expansion Tests

An expansion index tests were performed on a representative soil sample encountered by the test methods outlined in ASTM D4829.

B-1.03 Soluble Sulfates and Chlorides

A test was performed on a representative sample encountered during the investigation using the Caltrans Test Methods CTM 417 and CTM 422.

B-1.04 Soil Reactivity (pH) and Electrical Resistivity

A representative soil sample was tested for soil reactivity (pH) and electrical resistivity using California Test Method 643. The pH measurement determines the degree of acidity or alkalinity in the soils.

B-1.05 Particle Size Analysis

Particle size analysis was performed on a representative soil in accordance to the standard test methods of the ASTM D422. The hydrometer portion of the standard procedure was not performed and the material retained on the #200 screen was washed.

B-1.06 Direct Shear

A direct shear test was performed on a representative soil sample using the standard test method of ASTM D3080 (consolidated and drained). The test was performed on a sample remolded at 90 percent relative compaction.

The shear test was performed on a direct shear machine of the strain-controlled type. To simulate possible adverse field conditions, the sample was saturated prior to shearing. Several specimens were sheared at varying normal loads and the results plotted to establish the angle of the internal friction and cohesion of the tested sample.

B-1.07 Resistance Value (R-Value)

A Resistance Value tests was performed on a representative soil sample by the test methods outlined in California 301.

B-1.08 Test Results

Test results for all laboratory tests performed on the subject project are presented in this appendix.

SAMPLE INFORMATION

Sample Number	Sample Description	Sample Location Trench No.	Sample Location Depth (ft)
1	Orange brown to gray gneiss	1	2-3

MAXIMUM DENSITY - OPTIMUM MOISTURE

Test Method: ASTM D1557

Sample Number	Optimum Moisture (Percent)	Maximum Density (lbs/ft^3)
1	8.5	130.6

EXPANSION TEST

Test Method: ASTM D4829

Sample Number	Molding Moisture Content (Percent)	Final Moisture Content (Percent)	Initial Dry Density (lbs/ft^3)	Expansion Index	Expansion Classification
1	8.5	13.9	116.3	1	Very low

SOLUBLE SULFATES AND CHLORIDES

Test Method: CTM 417 and CTM 422

Sample Number	Soluble Sulfate (ppm)	Chlorides (ppm)
1	189	123

SOIL REACTIVITY (pH) AND ELECTRICAL RESISTIVITY

Test Method: CTM 643

Sample Number	pH	Resistivity (Ohm-cm)
1	6.5	10,000

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PARTICLE SIZE ANALYSIS

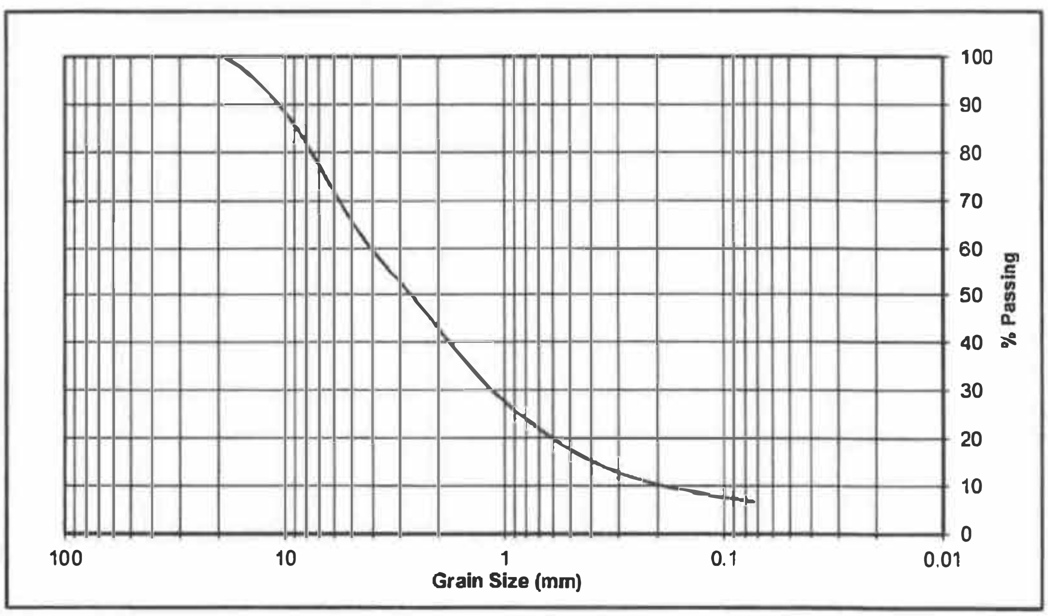
ASTM D422

Sample ID: 1

Fraction A: Dry Net Weight (gms): 1,534
Fraction B: Dry Net Weight (gms): 517.6

	Screen Size	Net Retained Weight (gms)	Net Passing Weight (gms)	% Passing
Fraction A:	3"	0	1534	100
	1-1/2"	0	1534	100
	3/4"	0	1534	100
	3/8"	198	1336	87
	#4	548	986	64

	Screen Size	Net Retained Weight (gms)	Net Passing Weight (gms)	% Passing
Fraction B:	#8	137.4	380.2	47
	#16	269.6	248.0	31
	#30	359.6	158.0	20
	#50	415.2	102.4	13
	#100	445.4	72.2	9
	#200	463.4	54.2	7



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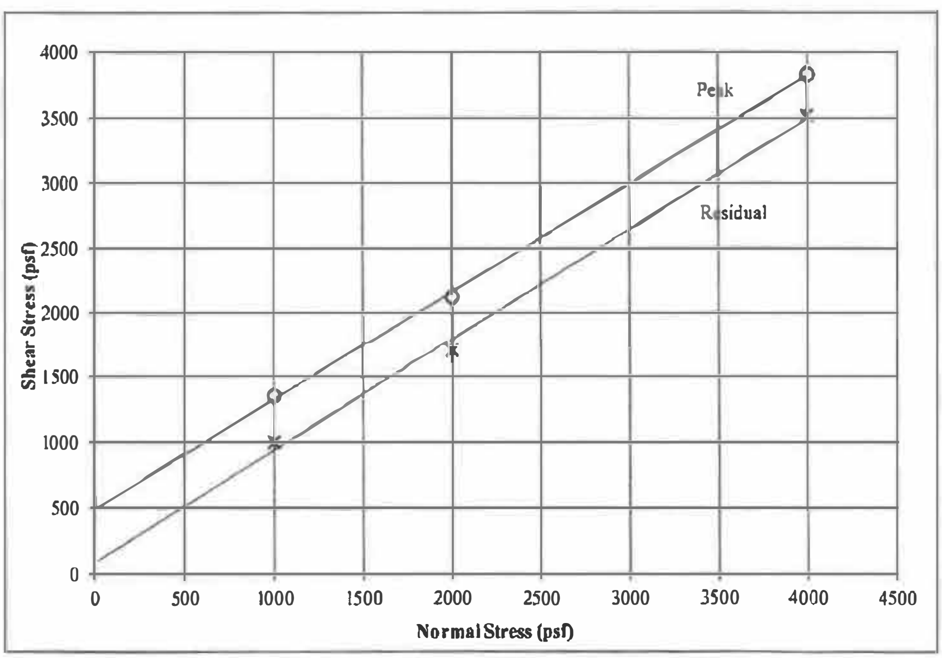
DIRECT SHEAR TEST
ASTM D3080

Sample ID: 1

Maximum Dry Density (pcf) = 130.6
Optimum Moisture Content (%) = 8.5
Initial Dry Density (pcf) = 117.5
Initial Moisture Content (%) = 8.5
Final Moisture Content (%) = 15.4

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	1351	1001
2000	2122	1700
4000	3835	3521

	Peak	Residual
Cohesion (psf) =	490	90
Friction Angle (deg) =	40	40



CTM 301 - DETERMINATION OF RESISTANCE "R" VALUE OF TREATED AND UNTREATED BASES, SUBBASES, AND BASEMENT SOILS BY THE STABILOMETER

Sample ID: 1

Specimen No	A	B	C
Moisture Content (%)	9.9	9.3	8.6
Dry Density (pcf)	127.5	128.3	128.5
Exudation Pressure (psi)	269	396	791
Stabilometer R Value	53	73	76
Expansion Pressure Dial	0	0	0

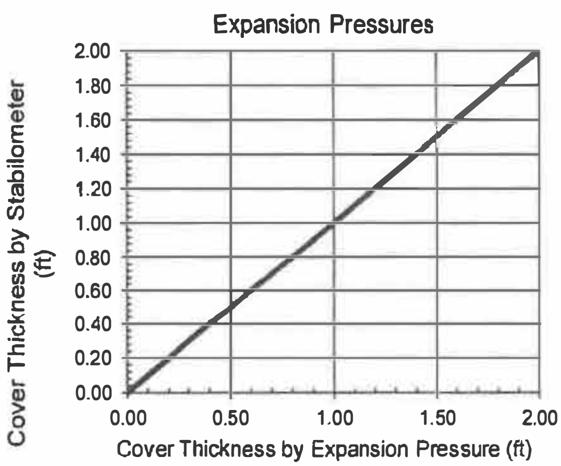
Use: Traffic Index = 5.0 Gravel Factor = 1.00

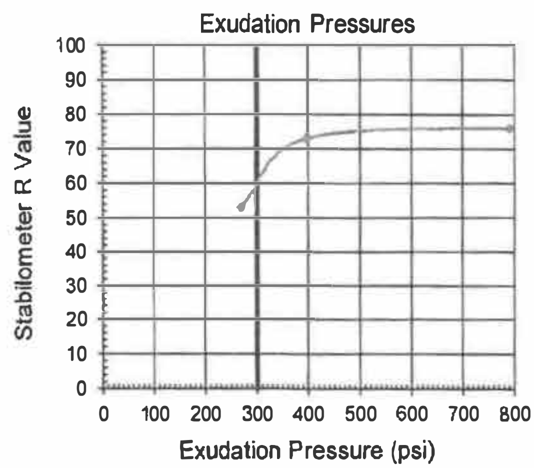
Thickness by Expansion (ft)			
Thickness by Stabilometer (ft)	0.75	0.43	0.38

Equilibrium Thick (ft) -

Equilibrium Pressure R Value **n/a**
Exudation Pressure R Value @ 300 psi **59**

Use Exudation R Value





Expansion Pressure R-Value is based on the following structural section:

Thickness of AC (ft)=	**0.25**	G_f(ac) =	2.50	W(ac) =	145
Thickness of Aggregate Base (ft)=	**0.42**	G_f(base) =	1.10	W(base) =	130
		G_f(avg) =	1.62	W(avg) =	136

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APPENDIX C

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX C

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

C-1.00 GENERAL DESCRIPTION

C-1.01 Introduction

These specifications present our general recommendations for earthwork and grading as shown on the approved grading plans for the subject project. These specifications shall cover all clearing and grubbing, removal of existing structures, preparation of land to be filled, filling of the land, spreading, compaction and control of the fill, and all subsidiary work necessary to complete the grading of the filled areas to conform with the lines, grades and slopes as shown on the approved plans.

The recommendations contained in the geotechnical report of which these general specifications are a part of shall supersede the provisions contained hereinafter in case of conflict.

C-1.02 Laboratory Standard and Field Test Methods

The laboratory standard used to establish the maximum density and optimum moisture shall be ASTM D1557.

The insitu density of earth materials (field compaction tests) shall be determined by the sand cone method (ASTM D1556), direct transmission nuclear method (ASTM D6938) or other test methods as considered appropriate by the geotechnical consultant.

Relative compaction is defined, for purposes of these specifications, as the ratio of the in-place density to the maximum density as determined in the previously mentioned laboratory standard.

C-2.00 CLEARING

C-2.01 Surface Clearing

All structures marked for removal, timber, logs, trees, brush and other rubbish shall be removed and disposed of off the site. Any trees to be removed shall be pulled in such a manner so as to remove as much of the root system as possible.

C-2.02 Subsurface Removals

A thorough search should be made for possible underground storage tanks and/or septic tanks and cesspools. If found, tanks should be removed and cesspools pumped dry.

Any concrete irrigation lines shall be crushed in place and all metal underground lines shall be removed from the site.

C-2.03 Backfill of Cavities

All cavities created or exposed during clearing and grubbing operations or by previous use of the site shall be cleared of deleterious material and backfilled with native soils or other materials approved by the soil engineer. Said backfill shall be compacted to a minimum of 90% relative compaction.

C-3.00 ORIGINAL GROUND PREPARATION

C-3.01 Stripping of Vegetation

After the site has been properly cleared, all vegetation and topsoil containing the root systems of former vegetation shall be stripped from areas to be graded. Materials removed in this stripping process may be used as fill in areas designated by the soil engineer, provided the vegetation is mixed with a sufficient amount of soil to assure that no appreciable settlement or other detriment will occur due to decaying of the organic matter. Soil materials containing more than 3% organics shall not be used as structural fill.

C-3.02 Removals of Non-Engineered Fills

Any non-engineered fills encountered during grading shall be completely removed and the underlying ground shall be prepared in accordance to the recommendations for original ground preparation contained in this section. After cleansing of any organic matter the fill material may be used for engineered fill.

C-3.03 Overexcavation of Fill Areas

The existing ground in all areas determined to be satisfactory for the support of fills shall be scarified to a minimum depth of 6 inches. Scarification shall continue until the soils are broken down and free from lumps or clods and until the scarified zone is uniform. The moisture content of the scarified zone shall be adjusted to within 2% of optimum moisture. The scarified zone shall then be uniformly compacted to 90% relative compaction.

Where fill material is to be placed on ground with slopes steeper than 5:1 (H:V) the sloping ground shall be benched. The lowermost bench shall be a minimum of 15 feet wide, shall be a minimum of 2 feet deep, and shall expose firm material as determined by the geotechnical consultant. Other benches shall be excavated to firm material as determined by the geotechnical consultant and shall have a minimum width of 4 feet.

Existing ground that is determined to be unsatisfactory for the support of fills shall be overexcavated in accordance to the recommendations contained in the geotechnical report of which these general specifications are a part.

C-4.00 FILL MATERIALS

C-4.01 General

Materials for the fill shall be free from vegetable matter and other deleterious substances, shall not contain rocks or lumps of a greater dimension than is recommended by the geotechnical consultant, and shall be approved by the geotechnical consultant. Soils of poor gradation, expansion, or strength properties shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils providing satisfactory fill material.

C-4.02 Oversize Material

Oversize material, rock or other irreducible material with a maximum dimension greater than 12 inches, shall not be placed in fills, unless the location, materials, and disposal methods are specifically approved by the geotechnical consultant. Oversize material shall be placed in such a manner that nesting of oversize material does not occur and in such a manner that the oversize material is completely surrounded by fill material compacted to a minimum of

90% relative compaction. Oversize material shall not be placed within 10 feet of finished grade without the approval of the geotechnical consultant.

C-4.03 Import

Material imported to the site shall conform to the requirements of Section 4.01 of these specifications. Potential import material shall be approved by the geotechnical consultant prior to importation to the subject site.

C-5.00 PLACING AND SPREADING OF FILL

C-5.01 Fill Lifts

The selected fill material shall be placed in nearly horizontal layers which when compacted will not exceed approximately 6 inches in thickness. Thicker lifts may be placed if testing indicates the compaction procedures are such that the required compaction is being achieved and the geotechnical consultant approves their use.
Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to insure uniformity of material in each layer.

C-5.02 Fill Moisture

When the moisture content of the fill material is below that recommended by the soils engineer, water shall then be added until he moisture content is as specified to assure thorough bonding during the compacting process.

When the moisture content of the fill material is above that recommended by the soils engineer, the fill material shall be aerated by blading or other satisfactory methods until the moisture content is as specified.

C-5.03 Fill Compaction

After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than 90% relative compaction. Compaction shall be by sheepsfoot rollers, multiple-wheel pneumatic tired rollers, or other types approved by the soil engineer.

Rolling shall be accomplished while the fill material is at the specified moisture content. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to insure that the desired density has been obtained.

C-5.04 Fill Slopes

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting of the slopes may be done progressively in increments of 3 to 4 feet in fill height. At the completion of grading, the slope face shall be compacted to a minimum of 90% relative compaction. This may require track rolling or rolling with a grid roller attached to a tractor mounted side-boom.

Slopes may be over filled and cut back in such a manner that the exposed slope faces are compacted to a minimum of 90% relative compaction.

The fill operation shall be continued in six inch (6") compacted layers, or as specified above, until the fill has been brought to the finished slopes and grades as shown on the accepted plans.

C-5.05 Compaction Testing

Field density tests shall be made by the geotechnical consultant of the compaction of each layer of fill. Density tests shall be made at locations selected by the geotechnical consultant.

Frequency of field density tests shall be not less than one test for each 2.0 feet of fill height and at least every one thousand cubic yards of fill. Where fill slopes exceed four feet in height their finished faces shall be tested at a frequency of one test for each 1000 square feet of slope face.

Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density reading shall be taken in the compacted material below the disturbed surface. When these readings indicate that the density of any layer of fill or portion thereof is below the required density, the particular layer or portion shall be reworked until the required density has been obtained.

C-6.00 SUBDRAINS

C-6.01 Subdrain Material

Subdrains shall be constructed of a minimum 4-inch diameter pipe encased in a suitable filter material. The subdrain pipe shall be Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent. Subdrain pipe shall be installed with perforations down. Filter material shall consist of 3/4" to 1 1/2" clean gravel wrapped in an envelope of filter fabric consisting of Mirafi 140N or approved equivalent.

C-6.02 Subdrain Installation

Subdrain systems, if required, shall be installed in approved ground to conform the approximate alignment and details shown on the plans or herein. The subdrain locations shall not be changed or modified without the approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in the subdrain line, grade or material upon approval by the design civil engineer and the appropriate governmental agencies.

C-7.00 EXCAVATIONS

C-7.01 General

Excavations and cut slopes shall be examined by the geotechnical consultant. If determined necessary by the geotechnical consultant, further excavation or overexcavation and refilling of overexcavated areas shall be performed, and/or remedial grading of cut slopes shall be performed.

C-7.02 Fill-Over-Cut Slopes

Where fill-over-cut slopes are to be graded the cut portion of the slope shall be made and approved by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope.

C-8.00 Trench Backfill

C-.01 General

Trench backfill within street right of ways shall be compacted to 90% relative compaction as determined by the ASTM D1557 test method. Backfill may be jetted as a means of initial compaction; however, mechanical compaction will be required to obtain the required percentage of relative compaction. If trenches are jetted, there must be a suitable delay for drainage of excess water before mechanical compaction is applied.

C-9.00 Seasonal Limits

C-9.01 General

No fill material shall be placed, spread or rolled while it is frozen or thawing or during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the soils engineer indicate that the moisture content and density of the fill are as previously specified.

C-10.00 Supervision

C-10.01 Prior to Grading

The site shall be observed by the geotechnical consultant upon completion of clearing and grubbing, prior to the preparation of any original ground for preparation of fill.

The supervisor of the grading contractor and the field representative of the geotechnical consultant shall have a meeting and discuss the geotechnical aspects of the earthwork prior to commencement of grading.

C-10.02 During Grading

Site preparation of all areas to receive fill shall be tested and approved by the geotechnical consultant prior to the placement of any fill.

The geotechnical consultant or his representative shall observe the fill and compaction operations so that he can provide an opinion regarding the conformance of the work to the recommendations contained in this report.

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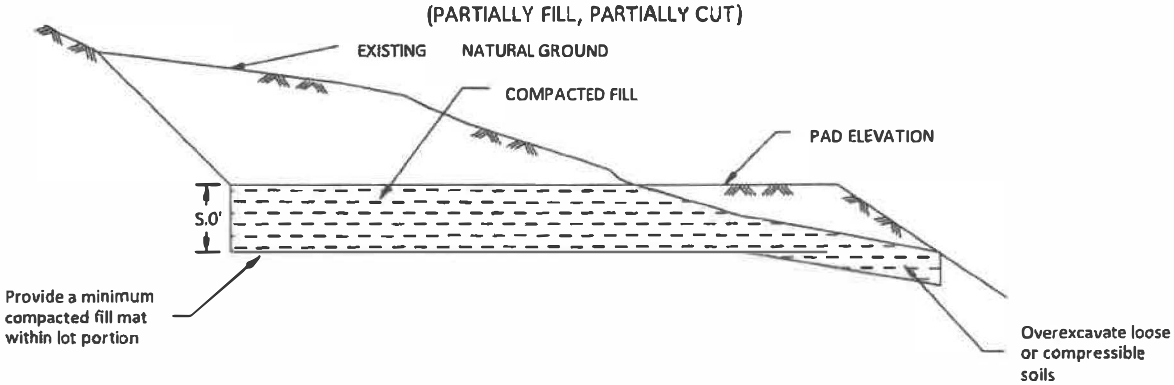
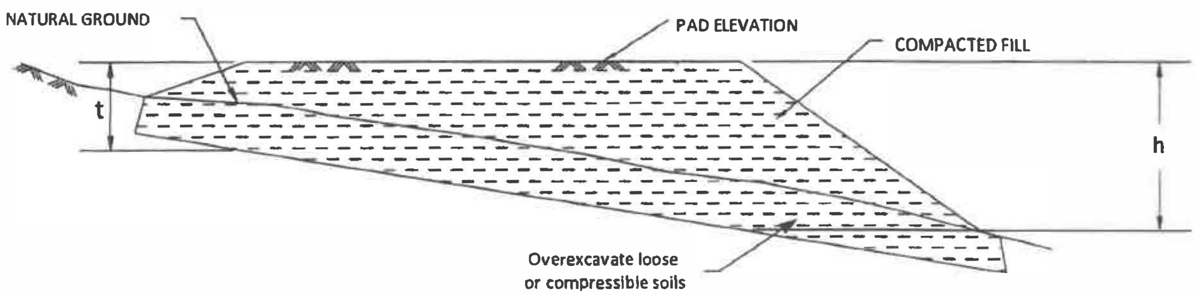
CUT TO FILL TRANSITION PADS

(PARTIALLY FILL, PARTIALLY CUT)

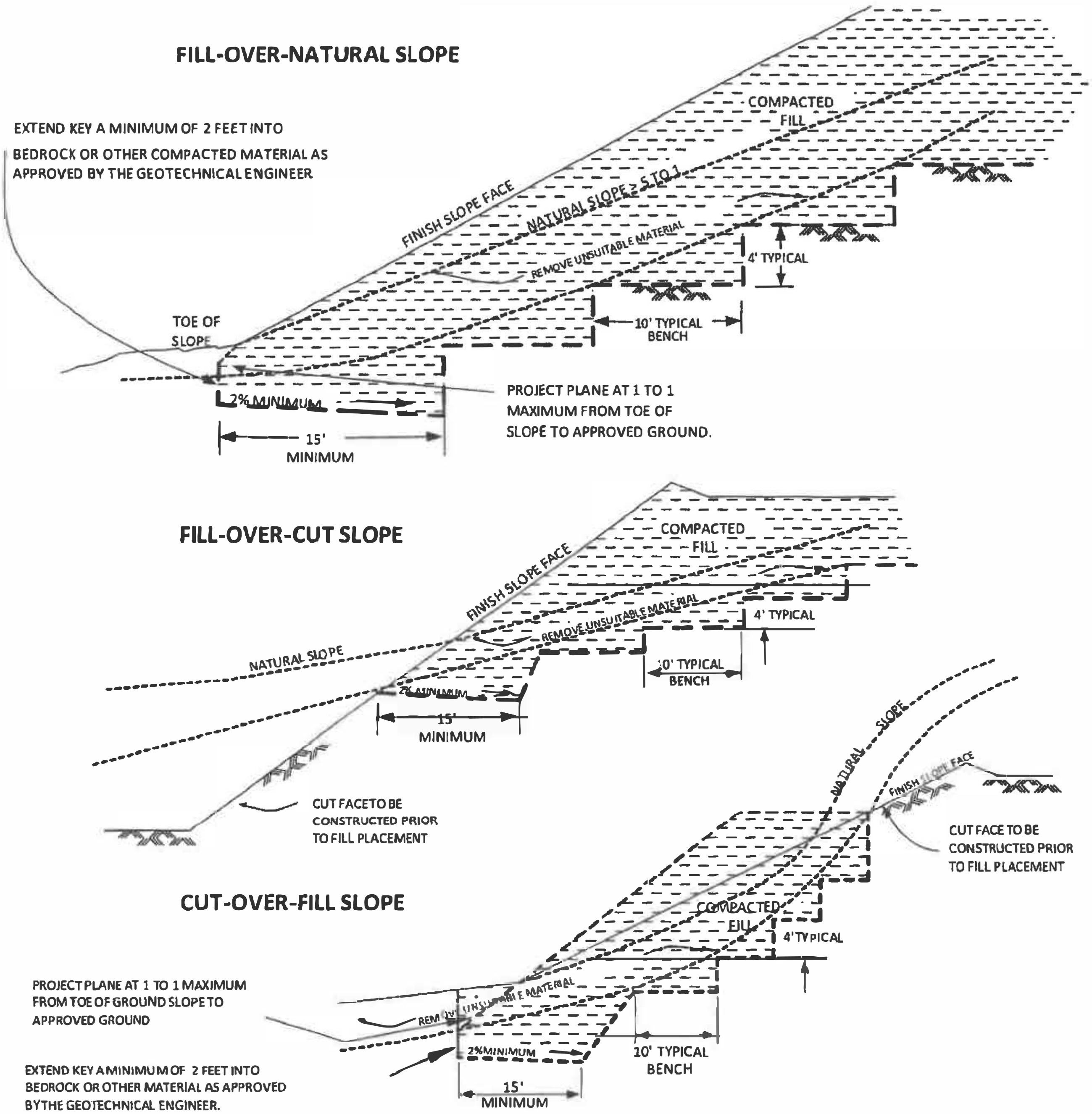


FILL PADS

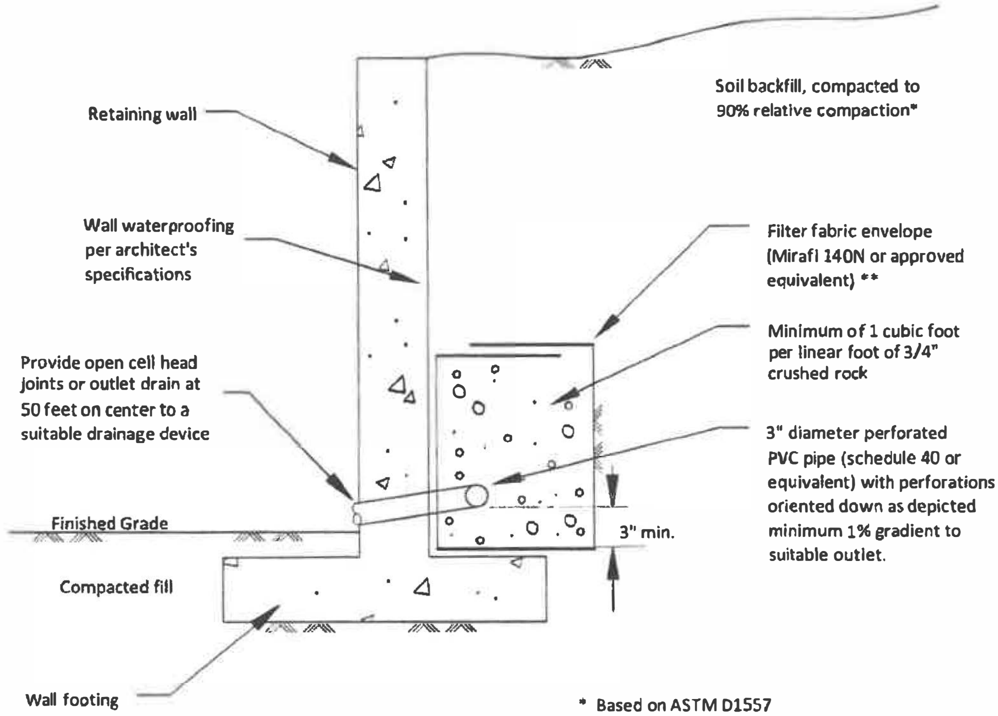


Minimum Mat Thickness t (ft)	Maximum Fill Depth h (ft)
5.0	0 to 15
one third the maximum fill depth not to exceed 15 feet	> 15

FILL TRANSITIONS OVEREXCAVATION REQUIREMENTS



KEY AND BENCHING DETAIL



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2
PERMEABLE MATERIAL
(CAL TRANS SPECIFICATIONS)

Sieve Size	% 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

RETAINING WALL DRAINAGE DETAIL

APPENDIX D

REFERENCES

APPENDIX D

REFERENCES

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