GEOTECHNICAL ENGINEERING STUDY
Leavesley Road Four-Parcel Subdivision
Leavesley Road [100± Acres; APN 898-04-002]
Gilroy, Santa Clara County, California

JULY 2007

For

Leavesley Road Partners, LLC
c/o MH Engineering Co.
Morgan Hill, California 95037

Prepared by

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E$_2$C, Inc. Project Number 2645SC01-GA

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c/o MH Engineering Co.
16075 Vineyard Boulevard
Morgan Hill, California 95037

Subject: **Leavesley Road Four-Parcel Subdivision**
Leavesley Road [100± Acres; APN 898-34-002]
Gilroy, Santa Clara County, California
GEOTECHNICAL ENGINEERING STUDY

Gentlemen:

As requested by Ms. Gloria Ballard of MH Engineering Co., on your behalf, E_2C has conducted a geotechnical investigation for the proposed four-parcel subdivision of the subject site in eastern Gilroy, California. The report documents the field investigation performed at the subject site, as well as the results of the laboratory testing of the relatively undisturbed earth materials retrieved during the field investigation. Based on these results, we are presenting geotechnical design criteria for the four proposed single-family residences and related improvements at the site.

Should you have any questions or require supplemental information, please do not hesitate to contact us.

Sincerely,
E_2C, Inc.

James E. Ball
Project Geologist

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APPENDIX A

Logs of Test Borings
Laboratory Test Results

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

LIST OF FIGURES

GEOTECHNICAL ENGINEERING STUDY

PROJECT: **Leavesley Road Four-Parcel Subdivision**
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Gilroy, California

CLIENT: **Leavesley Road Partners, LLC**

1.0 INTRODUCTION

1.1 Location and Description of Site

The subject site is located at approximately 37° 03' N latitude, 121° 31' W longitude on the northeast quarter of the southeast quadrant of the Gilroy 7.5-minute quadrangle (Figure 1). The parcel is most readily accessed from the City of Gilroy via Highway 152 (Leavesley Road) east to where it changes direction to the southeast and becomes Ferguson Avenue. The rural eastern section of Leavesley Road branches off of Highway 152 to the northeast and continues 0.95 miles to Crews Road. Leavesley Road follows Alamias Creek up former Ruby Canyon Road (as shown on the 1981 photorevised Gilroy quadrangle). The 100± acre parcel is approximately 0.6 miles (1 km) northeast of the intersection of Crews Road and Leavesley Road (Figure 1).

The U.S. Geological Survey's Gilroy quadrangle shows the lowest elevation at the site to be approximately 400 feet along Leavesley Road, rising to roughly 680 feet at the northern portion of the site. The site is characterized by a narrow, northwest-trending linear ravine which crosses the northern half of the site, and is the dominant topographic feature at the site. A small man-made pond occupies the ravine where it is closest to the southern hair-pin loop of Leavesley Road. Topography is predominantly gently sloping, except for two prominent rocky hilltops near the north and northeastern property lines and the Alamias Creek embankment. A smaller rocky knoll is located near the center of the site. Vegetation at the site consists of central California oak woodland and grasses and brush. Drainage is generally south toward Alamias Creek. A PG& E easement crosses the central portion of the site.

1.2 Purpose and Scope

The purpose of this geotechnical engineering study was to identify and evaluate surface and sub-surface soil and groundwater conditions at the site as they may affect the proposed development, and to provide appropriate geotechnical engineering design criteria. The report presents recommendations for site preparation and grading, foundation design criteria, site drainage, concrete slab-on-grade construction, and utility trench backfill. Conclusions in this report are based on the data acquired and evaluated from E_2C's exploratory drilling and laboratory testing programs.

1.3 Planned Development

The Cover Sheet (Sheet 1 of 6) dated September 2006 prepared by MH Engineering Co. indicates that the proposed development of the site will entail the subdivision of the overall parcel into four smaller parcels ranging from approximately 14 acres to 43 acres, with a proposed 2.5± acre building envelope on each of the new parcels (Figure 2). The remaining space of each new parcel will be designated open space. Four custom single-family residences are planned on the resultant parcels. It is assumed herein that the homes will be wood-frame, two-story structures with raised wood floors, with attached garages with concrete slab-on-grade floors. The residences will be serviced by leachfields and septic tank systems. Access to the homes will be via a shared driveway (proposed Yarak Court) off of Leavesley Road along the southeastern edge of the property. The portion of the site that is north of the deep ravine that marks the main trace of the Calaveras fault will remain undeveloped as open space.

2.0 INVESTIGATIVE PROCEDURES

2.1 Geologic Conditions

The following paragraphs summarize the geologic conditions at the site. The summary is derived from the Geologic Hazards Evaluation and subsequent Fault Investigation Report issued by Earth Systems Consultants Northern California (2005). The "main" trace of the Calaveras fault at the site is defined herein as the fault contact between Quaternary age volcanic rocks and Cretaceous sedimentary rocks as discussed below.

The geology of the site vicinity has been mapped by Dibblee (1973), Armstrong and Wagner (1976), and Helley and Nakata (1991). Dibblee (1973) shows the northwest-trending main trace of the Calaveras fault approximately coincident with the southwestern edge of the ravine which crosses the northeastern part of the site (Figure 2). The bedrock northeast of the Calaveras fault is mapped as possible Cretaceous age Berryessa Formation and unnamed early Tertiary or late Cretaceous sedimentary rocks. The orientation of bedding in the site vicinity is roughly northwest with a 25 to 30 degree northeast dip. Southwest of the Calaveras fault, the bedrock is mapped as Pliocene or Pleistocene age unnamed basalt. A sample of the basalt taken along the Alamias Creek drainage (Ruby Canyon) southwest of the site was dated at approximately 3.5 million years before present (Dibblee, 1973).

Helley and Nakata (1991) show the main trace of the Calaveras fault along the centerline of the ravine (Figure 2). The bedrock northeast of the Calaveras fault is mapped as Jurassic and Cretaceous age Great Valley Sequence sedimentary rocks. Volcanic rocks consisting predominantly of Pliocene age flow and tuff breccia are shown southwest of the fault, with some olivine basalt flows.

Seven traces, or strands, of the Calaveras fault were identified on the site by exploratory trenching (ESCNC, 2005). Fifty-foot setbacks from the fault traces were established as shown on Figure 2.

2.2 Subsurface Exploration Program

Eight exploratory borings were drilled under E_2C's direction on 13 June 2007 at the approximate locations shown on the Site Plan (Figure 2). The borings were drilled to depths of 4 to 15 feet with a truck-mounted B-24 drill rig equipped with solid-stem augers. The shallow borings were terminated due to auger refusal.

As the borings were advanced, relatively undisturbed samples were obtained at selected depths by driving a 3-inch diameter (O.D.) split tube sampler into the undisturbed soil mass by means of a 140-pound hammer with a 30-inch free fall. The sampler was driven 18 inches, and the number of blows for each 6 inches of penetration was recorded. The blow-count value for the final 12 inches of driving is indicated on the boring log at the appropriate depth.

The samples were classified in the field, sealed, and returned to the soils laboratory for testing. Our logs of the exploratory borings showing the vertical distribution of the soil units, the locations of the samples, blow-count values, and selected laboratory test results are presented in Appendix A.

2.3 Laboratory Testing Program

Subsequent to the subsurface exploration program, the soil samples were tested in the soils laboratory to aid in their classification and to determine some of the pertinent engineering properties of the soil types encountered at the site. The results of the various tests performed in our laboratory testing program are presented in Appendix A. A short general description of the tests performed is presented below.

Moisture-Density Determinations (ASTM D 2937) - Moisture content and dry density tests were performed on selected samples in order to evaluate the density and moisture variations through the explored soil profile. The moisture content is determined according to ASTM (American Society for Testing and Materials) Test Method D2216-80. For many soil types, the moisture content is one of the most significant index properties used in establishing a correlation between soil behavior and an index property. In fine-grained (cohesive) soils, for example, the consistency of a given soil type depends on its moisture content. The dry density of the soil is determined by a mathematical relationship between the moisture content and wet density of the soil sample. A summary of the measured moisture contents and dry densities of the tested samples is presented in Table A1.

Atterberg Limits and Plasticity Index (ASTM D 4318)- The Atterberg limits value was determined for a selected undisturbed soil sample. The Atterberg limits, the plastic limit and the liquid limit are defined as the moisture content, in percent, of a soil at the arbitrarily defined boundaries between the plastic and brittle states, and the liquid and plastic states, respectively. The plasticity index (P.I.) is the range of moisture content over which a soil behaves plastically. Numerically, it is the difference between the plastic limit and the liquid limit. The liquid and plastic limits, and the plasticity index, are an integral part of several soil engineering systems. These values are also used to correlate with engineering behavior such as compressibility, compactibility, permeability, expansiveness, and shear strength. The results of these tests are presented in Table A2.

Direct Shear Test (ASTM D308 -Modified)- Direct shear tests were performed on an undisturbed sample to determine the angle of internal friction and the cohesion of the tested sample. These strength characteristics are used, either individually or with other soil properties, to calculate engineering design parameters such as bearing capacity, earth pressures and slope stability. The test specimens were saturated under a 100 pound per square foot load prior to testing, and were sheared without allowing a significant amount of drainage during the shearing process. The results of these tests are presented in Table A1.

2.4 Engineering Analysis and Evaluation Procedures

An engineering analysis of the accumulated field and laboratory data was undertaken to provide geotechnical engineering recommendations for the design and construction of the proposed single-family residence and related improvements. These analyses included evaluation of the subsurface profile illustrated and described on the exploratory boring logs, evaluation of in-situ moisture content and dry density data, direct shear strength test results, and Atterberg Limits data. This information was utilized to provide recommendations for general site preparation and grading, for generation of foundation design criteria, and concrete slab-on-grade construction.

3.0 INVESTIGATIVE RESULTS

3.1 Subsurface Conditions

The site is overlain by a layer of brown to greyish brown and black clay to silty clay that is approximately 4 feet thick. The surficial clay was dry to slightly moist and in a stiff to very stiff condition. A Plasticity Index (P.I.) of 60 was derived from a sample of the clay, indicating very high expansive potential. The soil at the site is termed a *vertisol*, characterized by a high content of expansive clay that forms deep cracks when the soil shrinks in dry seasons. Cracks extending from the ground surface to the weathered bedrock contact were routinely observed in the exploratory trenches excavated at the site (ESCNC, 2005). The vertisol was approximately 4 feet thick. Cracks several inches wide were observed during the exploratory drilling for this study.

The surficial clay is underlain by Santa Clara Formation earth materials. The basalt is considered herein to be within the Santa Clara formation because the basalt found at the site has been dated at 3.5 million years (Dibblee, 1973), which is roughly coeval with the Plio-Pleistocene-age Santa Clara Formation. Drill refusal at less than five feet was experienced in the borings drilled on proposed Parcel 1 and Parcel 4, which are situated at the northern and southern limits of the developable area. Basalt is exposed at the ground surface on the low ridges on these parcels, and it is anticipated that basalt bedrock is at shallow depth at the proposed building locations on Parcels 1 and 4. Mudstone, siltstone, fine-grained sandstone, and possible weathered volcanic rocks (tuff?) were encountered in the remaining borings. The sedimentary rocks are weakly cemented, but well-consolidated and generally in a hard/very dense condition.

Groundwater was not encountered. It should be noted that fluctuations in the level of subsurface water can occur due to variations in rainfall, temperature, and other factors, and groundwater levels should not be considered constant. However, the presence of shallow groundwater is not expected to cause complications for the proposed single-family residences.

The logs of exploratory Borings B1 through B8, showing the approximate vertical distribution of the alluvial profile, are presented in Appendix A. The logs also show the vertical location of the samples obtained in the borings.

3.2 Liquefaction, Lateral Spreading

Soil liquefaction is a condition where saturated granular soils near the ground surface undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes and other vibrations. In this process, the soil acquires mobility sufficient to permit both vertical and horizontal movements if not confined. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands and loose silts with very low cohesion. It is generally acknowledged that liquefaction will not occur if the soils described above are located at depths greater than 40 to 50 feet below ground surface. In the deeper deposits the greater overburden pressure is sufficient to prevent liquefaction from occurring. Soils resistant to liquefaction include all soil types that are dry, cohesive, or sufficiently dense. The site is underlain by very dense/hard deposits of the Santa Clara Formation. The potential for earthquake-induced liquefaction and lateral spreading at the site is considered low.

3.3 Settlement (Static Foundation Loads)

The rather uniform and very stiff/hard nature of the Santa Clara Formation indicates that differential settlement due to static loading from building construction (including any fill placement) should not present a problem to the proposed development. E_2C estimates that less than 1/2 inch of total static settlement and 1/2 inch of differential settlement in 50 feet for combined structural fills and foundation loads of the proposed buildings at the site once construction is completed.

3.4 Seismically-Induced Differential Compaction

If the near-surface soil/alluvium vary in composition both vertically and laterally, strong seismic shaking can cause differential compaction of the near-surface earth materials. The resulting movement of the soil/alluvium may be expressed at the ground surface with detrimental effect on man-made structures. The Santa Clara Formation materials encountered at the site are very dense or hard mudstone, siltstone, sandstone, and possible volcanic rocks, and do not vary abruptly in thickness or lateral extent over short distances. It is our opinion that the potential for differential compaction at the site is low.

4.0 SITE DEVELOPMENT AND CONSTRUCTION CRITERIA

Grading and Site Development

4.1. *The most severe geotechnical constraint at the site is the presence of approximately 4 feet of highly expansive surficial clay soil.* The recommendations of this geotechnical engineering study are for the four proposed single-family residences and related improvements as described above. The following recommendations apply to grading and site preparation activities that may be required in order to achieve the final grades at the subject site.

4.2. It is recommended that the project geotechnical engineer review the grading plans, improvement plans and foundation plans for the project. The review is intended to determine compliance with the intent of the recommendations contained in this report.

4.3. Site grading and foundation construction shall be observed by the project geotechnical engineer and tested, as necessary, to determine general compliance with the following recommendations. In addition, it is recommended that the geotechnical engineer observe conditions exposed by the grading and record significant features and/or changes that may be exposed.

4.4. It is recommended that all aspects of grading be thoroughly covered in a pre-construction conference with representatives of the owner, grading contractor, civil engineer, and geotechnical engineer.

4.5. Site clearing, debris removal, and placement of fill and other grading operations at the site shall be conducted in accordance with the recommendations provided in this report.

4.6. Existing utility lines, where known, shall be located on the grading plans to assist the geotechnical engineer during the grading operations. The necessity of removing abandoned underground utility lines will be determined by the geotechnical engineer during site grading. The geotechnical engineer shall be notified if buried debris is encountered during grading operations in order to provide the necessary supplemental recommendations.

4.7. Ruts or depressions resulting from the removal of previous foundations and abandoned or buried structures/debris that may be encountered shall be cleaned down to native soil approved by our field representative. The bottom(s) of the resulting depression(s) shall be scarified and compacted. Depressions resulting from the removal of old foundation elements or utilities shall then be backfilled with compacted structural fill as described below. Clearing and backfilling operations shall be performed under the observation of the geotechnical engineer.

4.8. Prior to grading operations, areas which are to receive pavement sections or concrete slabs-on-grade shall be stripped and cleared of surface organic material. The exact depth of stripping will be determined in the field by the geotechnical engineer while the stripping operations are in progress. The exposed ground surface and the new fill shall be compacted to a minimum of 90 percent relative compaction. If loose or excessively dry areas are observed, these areas shall be scarified, moisture conditioned, and compacted to 90 percent relative compaction as well. Organically contaminated soil may either be stockpiled and later used as topsoil in landscaping areas, or be removed from the site.

4.9. "General" structural fill is defined herein as native soil or imported fill soil material which, when properly compacted, will support foundations, pavements, concrete slabs-on-grade or other fills without detrimental settlement.

4.10. The native soils which are free of deleterious materials may be reused as "general" structural fill, but not as select "structural" fill. If achievement of final grades requires the importation of structural fill, a sample of each proposed import material shall be delivered to the soil engineer or his representative for testing and approval at least three working days prior to being transported to the site.

4.11. Select imported structural fill soil material for this project shall:

a. have a plasticity index of less than 15 and/or an expansion index less than 50;
b. be free of organics, debris or other deleterious material;
c. have a maximum rock size of three (3) inches; and
d. contain sufficient clay binder to allow for stable foundation and utility trench excavations.

4.12. It is expected that the building pads will be above existing grade. *Due to the highly expansive soil at the site, <u>the upper 4 feet of the building pads shall be select imported structural fill compacted as described in this report</u>.* The building pads shall be subexcavated at least 2 feet below natural grade. The subexcavation shall extend at least 6 feet beyond the footprint of the building (Figure 3). The bottom of the excavation shall be moisture conditioned above optimum moisture content as necessary, and compacted to at least 90% relative compaction. Select imported fill may then be placed in thin, compacted lifts until the design grade is achieved.

4.13. The select import fill building pads shall extend at least 6 feet beyond the perimeter of the house foundations. The building pads must slope away (approximately ¼-inch per foot) from the foundation to provide drainage away from the footings. A French drain shall be installed around the perimeter of the residences. In addition, a 3-foot wide impermeable layer shall extend at least 3 feet beyond the foundations. Refer to Section 9 for site drainage recommendations.

4.14. Compaction of structural fills and subgrade soils under asphalt paving shall be to at least 90 percent relative compaction except as specifically stated in other paragraphs in this report. Compaction of all import aggregate baserock materials under proposed asphalt pavements shall be to at least 95 percent relative compaction. Compaction criteria is based on the laboratory procedure ASTM D1557-91. The subgrade under asphalt pavements may need to be stabilized prior to subgrade compaction if unsuitable areas are encountered during grading.

4.15. The geotechnical engineer shall be notified at least 48 hours prior to commencement of any grading operations so that he may coordinate the work in the field with the contractors.

5.0 Uniform Building Code Parameters

5.1. Available information on soil type and seismicity was used for design criteria for the Site based on Chapter 16 of the 1997 Uniform Building Code (UBC) 1997 and Map F-20, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, ICBO February 1998). This information is summarized in the following Table.

TABLE 1
Summary of Seismic Design Parameters (UBC 1997)

Seismic Zone	4
Seismic Source Type A	San Andreas
Distance to Seismic Source Type A	19.0 km
Seismic Source Type B	Calaveras (south)
Distance to Seismic Source Type B	<0.5 km
Subgrade Classification	S_C
Near Source Factor N_a	1.3
Near Source Factor N_v	1.6
Seismic Coefficient C_a	0.52
Seismic Coefficient C_v	0.72

6.0 Foundation Recommendations

6.1 The proposed residences may be supported by conventional continuous footings bearing on the properly constructed building pads. *Isolated interior footings are not recommended.* The design of the foundation, including detailed reinforcing requirements, shall be performed by the project structural engineer.

6.2 Interior and exterior strip and spread footings shall be founded a minimum of 36 inches trenching depth into the properly compacted select import fill building pads. For these conditions, the footings may be designed for an allowable bearing capacity of 2,500 pounds per square foot for combined dead plus live loads. This value may be increased by one-third to include short term wind and/or seismic loadings.

6.3 Resistance to lateral loads will be provided by friction between the bottom of the foundation footings and the underlying soils. A friction factor of 0.35 may be applied to the dead load portion of the footing load. Additional lateral resistance will be provided by passive pressures acting against the sides of foundations. A passive pressure based on an equivalent fluid weight of 300 pounds per cubic foot may be used in the design of the foundation. The combined passive and frictional resistance may be used as the total lateral resistance without applying a reduction to either value.

7.0 Concrete Slab-On-Grade Construction

7.1. It is recommended that at least 6 inches of compacted gravel or crushed rock be placed under exterior concrete slabs that will receive vehicular traffic. The top 6 inches of subgrade soil and the baserock layer shall be compacted to at least 90% relative compaction. These exterior traffic bearing slabs shall be reinforced with at least #3 bars on 18-inch centers. Construction joints in all slabs shall be doweled.

7.2. Exterior concrete slabs-on-grade that will not receive vehicular traffic (such as sidewalks and patios) shall be underlain by a minimum of 4 inches of compacted gravel or other approved granular base material. The subgrade soil shall be scarified and compacted to at least 90% as described above. The import base material shall be approved by the geotechnical engineer prior to being transported to the site.

7.3. In interior areas where dampness of floor slabs cannot be tolerated, the concrete floor slab-on-grade should be shall on 2 inches of clean sand over a plastic membrane at least 6 mils thick over 4 inches of imported clean 3/8-inch diameter pea gravel, bringing the total thickness of imported granular material under the slab to 6 inches.

8.0 Utility Trench Backfill

8.1. Trench backfill is defined herein as that portion of the backfill material placed above the import shading material. It is assumed that utility pipes are properly bedded and shaded at least 6 inches over the top of the pipe with an approved, imported, granular material. It is further assumed that the trenches will not exceed 10 feet in depth.

8.2. Utility trenches in pavement areas and on the building pads shall be backfilled with layers of compacted import materials, native soils, or combinations thereof.

8.3. The utility trench backfill material shall be compacted to at least 90% relative compaction. Trenches excavated in the area under foundations defined by a plane radiating at a 45 degree angle downward from the bottom edge of the footing shall be backfilled with material compacted to a minimum 90% relative compaction. Backfill in landscape areas shall be compacted to a minimum 85% relative compaction.

8.4. Jetting is not suitable for native clayey soils utilized as backfill in utility trenches on this project. When utilized as trench backfill, the native soil shall be mechanically compacted.

8.5. Import sand, gravel Class II aggregate, or quarry fines may be used as trench backfill rather than native soil, if so desired. The gravel shall be clean, 3/4-inch in maximum diameter, and will not require moisturizing or mechanical compaction. Native soil and import materials such as sand, Class II aggregate base, or quarry fines will require mechanical compaction, and may require moisturizing prior to compaction operations.

8.6. Within the proposed dwellings, sand or gravel may be used to the top of the trench. In landscape areas, at least the final 24 inches of backfill shall be compacted native soil to prevent surface rain or irrigation water from rapidly penetrating down into the trench. The top 12 inches shall be compacted native soil under exterior paved areas.

8.7. Where an opening is made under or through the perimeter foundations for items such as utility lines, the openings must be resealed so that they are water-tight to prevent possible entrance of outside irrigation or rain water into the portion of the soil beneath the structure. Utility trenches extending under the perimeter foundations shall be either backfilled with native or fill soils, compacted to 90% minimum relative compaction, or be sealed by concrete poured around the pipes. The waterproofing of the utility trenches shall extend at least two feet on both sides of where it passes under the perimeter foundations. Where the pipes pass through sleeves cast into the perimeter foundation stem wall, a plumber's mastic type sealant or poured concrete shall be used around the pipe.

8.8. Where trenches pass from landscape areas to pavement areas, at least a three foot length of trench, centered on the curb line, shall be backfilled with native soil to reduce the potential for lateral migration of water from the planter to the pavement area.

9.0 Surface Drainage

9.1. The control of surface drainage is critical for the satisfactory performance of the project. It is essential that surface drainage be controlled, especially adjacent to structures, pavements and slopes. No concentrated surface water shall be allowed to flow over the top of cut, fill, or natural slopes. Instead, such surface water shall be diverted by soil berms, concrete lined ditches, or be collected in catch-basins back from the slope edge.

9.2. Exterior soil grades adjacent to foundations shall be sloped away (approximately ¼-inch per foot) from the house in order to prevent surface water from ponding adjacent to the footings. Roof drainage shall be collected in a water tight pipe system and discharged onto the driveways or well away from the house at an approved location.

9.3. A French-drain system shall be constructed around each house. The pipes shall discharge at approved locations. It is anticipated that the French-drains and roof drains can be routed to the storm drain system constructed for proposed Yarak Court.

9.4. No vegetation shall be allowed to grow within 3 feet of the house foundations. A 3-foot wide concrete apron shall be constructed around the perimeter of the houses. The apron shall slope away from the foundations as described above.

9.5. The final subgrade under the wood floors of the house shall be shaped to drain toward sufficient catch basins installed under the wood floors to collect any irrigation or rainwater that may find its way under the house. These under-floor catch basins shall be connected to a 4-inch diameter non-perforated drain pipe, which extends to an approved discharge point.

10.0 Asphalt Pavement Designs

10.1. Due to the very high expansive potential of the clay soil at the site, a conservative design "R"-Value of 3 was utilized in the computations of the alternate asphalt pavement designs for the proposed private driveway.

10.2. Pavement designs for traffic indexes of 4.0 to 7.0 at 0.5 increments are provided for your use. A computer program based upon the State of California Highway Design Manual, 5th Edition (7/1/95) was used in computing the alternate asphalt pavement sections presented in Table A below. Alternate pavement sections are presented in Table A below.

10.3. Soil subgrade areas in the proposed asphalt pavement areas shall be scarified as recommended in other paragraphs of this report, and be compacted to at least 90% relative compaction. The soil subgrade shall be moisture conditioned and compacted at over optimum water content. Caltrans specification, Class II Aggregate Base, with an "R"-Value = 78 minimum, shall be compacted to at least 95% relative compaction. Relative compaction values are based on the laboratory test procedure ASTM D1557-91.

<u>RECOMMENDED ALTERNATE ON-SITE ASPHALT PAVEMENT SECTIONS</u>
<u>SUBGRADE DESIGN R-VALUE 3</u>

Traffic Index	4.0	4.5	5.0	5.5	6.0	6.5	7.0
AC Thickness	3.0"	3.0"	3.5"	3.5"	4.0"	4.5"	4.5"
Class 2 Aggregate Base Thickness R=78 min.	6.5"	8.5"	9.0"	11.0"	12.0"	13.5"	15.0"

10.4. If the soil subgrade is properly prepared as described, it is our opinion that any of the above pavement design sections will be capable of supporting a 65,000-pound emergency vehicle for the very limited use that is expected.

11.0 Plan Reviews

11.1. E_2C shall be provided the opportunity for a general review of the grading, foundation, and drainage plans prior to their being submitted to the appropriate agencies for their review. This review is to satisfy Santa Clara County Planning department requirements, and assess general compliance with the recommendations of this report and incorporation of these recommendations into the project plans and specifications.

12.0 Observation and Testing During Construction

12.1 It is recommended that E_2C be retained to provide observation and testing services during grading and foundation construction. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

13.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report has been prepared for the sole use of Leavesley Road Partners LLC specifically for the design of the proposed single-family residences and related improvements at APN 898-34-002) on Leavesley Road in Gilroy, California. The opinions presented in this report have been formulated in accordance with generally accepted geotechnical engineering practices that exist in the area at the time this report was prepared. No other warranty, expressed or implied, is made or should be inferred. We are not responsible for data presented by others.

The opinions, conclusions and recommendations contained in this report are based upon information obtained from explorations at widely separated locations, site reconnaissance, review of data made available to us, and upon local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. In addition, geotechnical issues may arise that are not apparent at this time.

The geotechnical engineer should be retained to review the final plans and specifications when they become available in order to verify that these documents are consistent with the geotechnical recommendations and design criteria presented in this report. E_2C, Inc. shall be retained to provide observation and testing services during construction in order to ascertain compliance with our recommendations. If we are not retained for these services, E_2C, Inc. cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of the E_2C, Inc. report by others. Furthermore, E_2C, Inc. will cease to be the Geotechnical Engineer of Record at the time another consultant is retained for follow-up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the soil boring logs regarding odors noted or unusual or suspicious items or condition observed, are strictly for the information of our client.

14.0 REFERENCES CITED

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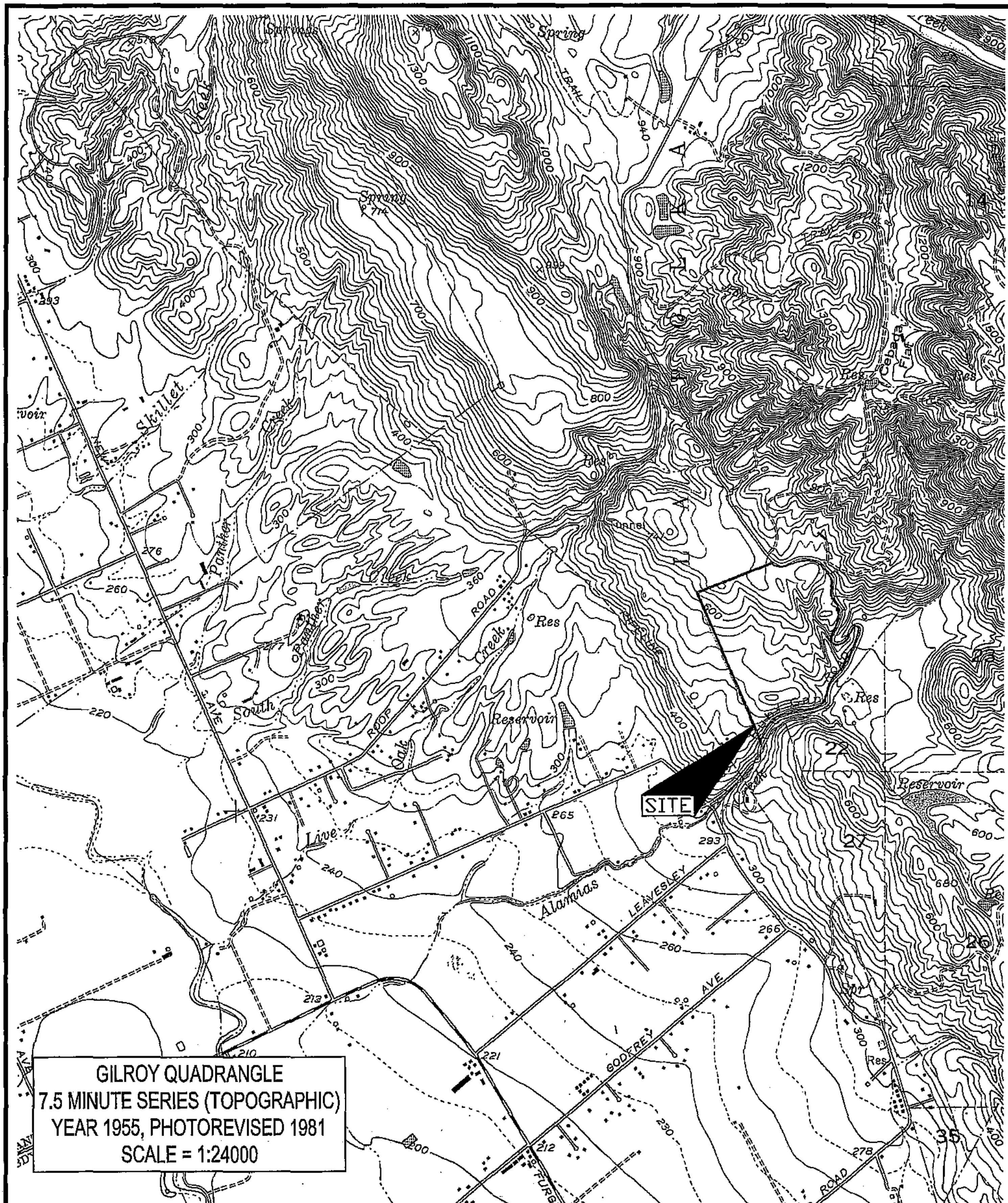
FIGURES



SITE LOCATION (U.S.G.S. BASE)

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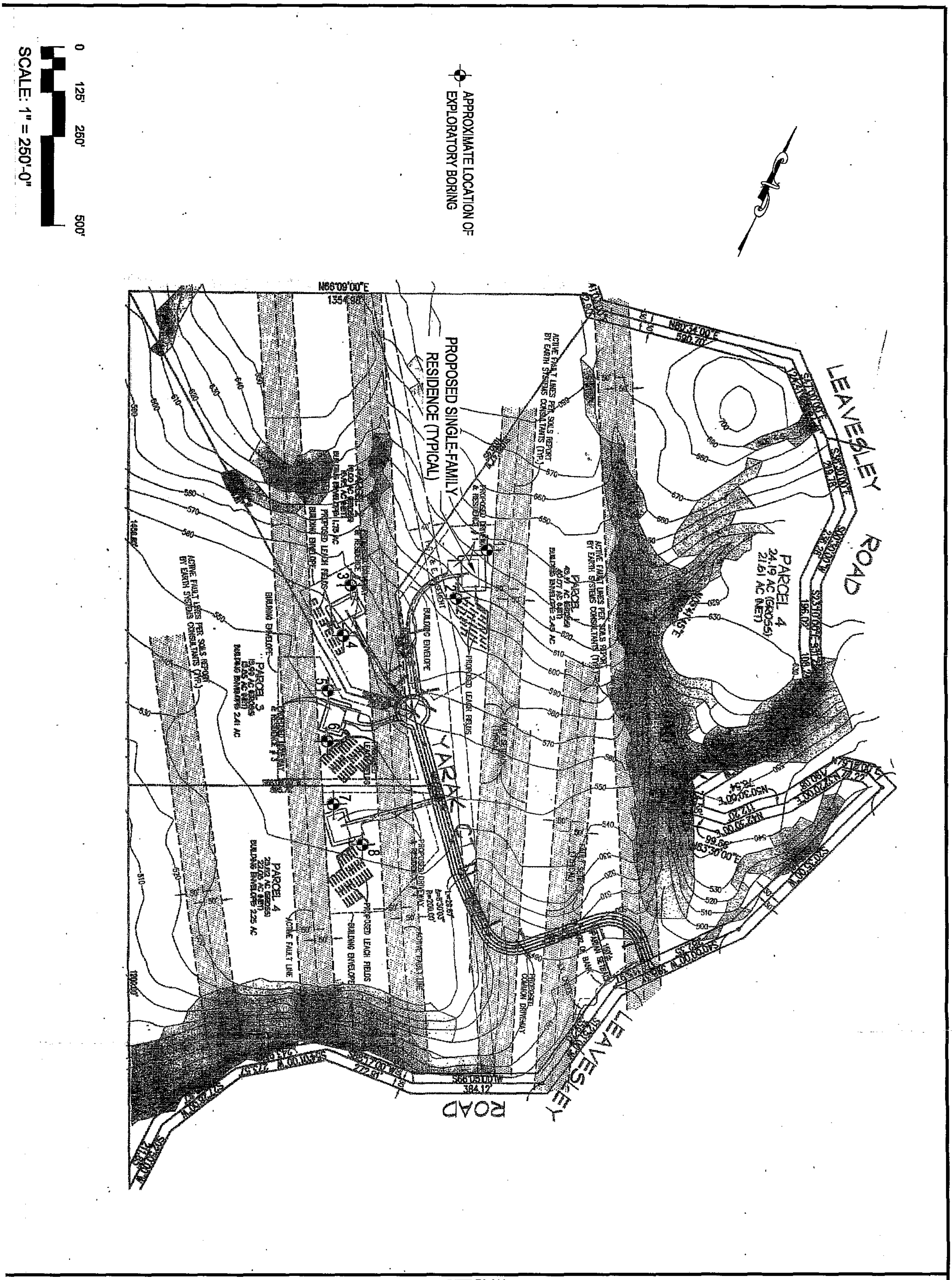
ENGINEERING CONSULTANTS
382 MARTIN AVENUE
SANTA CLARA, CALIFORNIA 95050-3112
TEL: 408.327.5700 FAX: 408.327.5707

LEAVESLEY ROAD FOUR-PARCEL SUBDIVISION
LEAVESLEY ROAD (APN 898-34-002)
GILROY, CA

FILENAME:	2645SC01-GA
DATE:	JULY 2007
CHECK BY:	JEB
DRAWN:	CAC

FIGURE
1



SITE PLAN

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GILROY, CA

FILENAME:	2645SC01-GA
DATE:	JULY 2007
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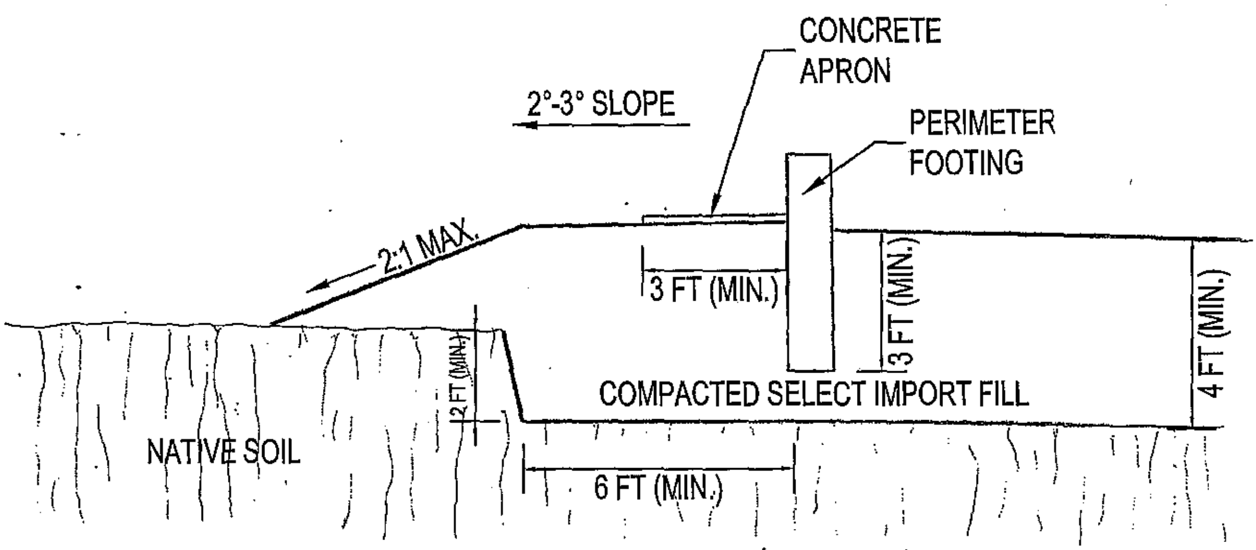
DRAWN:	CAC

FIGURE: 2



N.T.S.

BUILDING PAD CROSS-SECTION			
ENVIRONMENTAL / ENGINEERING CONSULTANTS 382 MARTIN AVENUE SANTA CLARA, CALIFORNIA 95050-3112 TEL: 408.327.5700 FAX: 408.327.5707	LEAVESLEY ROAD FOUR-PARCEL SUBDIVISION LEAVESLEY ROAD (APN 898-34-002) GILROY, CA	FILENAME: 2645SC01-GA DATE: JULY 2007 CHECK BY: JEB DRAWN: CAC	FIGURE: 3

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BORING LOG B - 1

Project No. 2645SC01-GA

Driller: Hillside Geotechnical
Drill Rig: B24
Depth to Groundwater: NGWE
Date Drilled: 13 June 2007
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
		1-1	50		4.5+	Silty clay; brown, dry, hard, basalt cobble in sampler shoe	15.8	72.6
				CH		hard drilling		
5								
						auger refusal at 8 feet (probable basalt boulder)		
						Boring terminated at 8 feet No groundwater encountered		
10								
15								
20								
25								
30								

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BORING LOG B - 2

Project No. 2645SC01-GA

Driller: Hillside Geotechnical
Drill Rig: B24
Depth to Groundwater: NGWE
Date Drilled: 13 June 2007
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
						Clay; black, moist, very stiff		
		2-1	25	CH		Clay; mixed black and pinkish brown, with occasional fine angular clast, calcareous, slightly moist, very stiff	26.5	87.7
5						**Santa Clara Formation**		
				ML		Clayey silt; pale brown and olive brown, iron oxide staining, with occasional fine angular volcanic clast, calcareous, friable, dry to slightly moist, hard [tuff?]		
		2-2	67				20.6	90.7
10						Boring terminated at 10 feet No groundwater encountered		
15								
20								
25								
30								

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BORING LOG B - 3

Project No. 2645SC01-GA

Driller: Hillside Geotechnical
Drill Rig: B24
Depth to Groundwater: NGWE
Date Drilled: 13 June 2007
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
		3-1	16	CH		Clay; dark greyish brown, very plastic, moist, stiff Plasticity Index = 60	--	--
5						**Santa Clara Formation**		
				CH		olive clay cuttings, harder drilling at 6 feet		
10		3-2	32			Clay; olive with grey silty inclusions, occasional fine angular to subangular clast, calcareous concretions, slightly moist, very stiff	38.6	81.1
		3-3	39	CH		Clay, mottled olive hues, iron oxide staining, occasional rounded coarse sand and fine gravel, slickensided, slightly moist, hard [highly weathered lacustrine volcanic deposit?]	37.4	83.3
15						Boring terminated at 15 feet No groundwater encountered		
20								
25								
30								

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BORING LOG B - 4

Project No. 2645SC01-GA

Driller: Hillside Geotechnical
Drill Rig: B24
Depth to Groundwater: NGWE
Date Drilled: 13 June 2007
Logged by: JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
		4-1	19	CH		Clay; very dark brown, with pale brown fine sandy calcareous inclusions,moist, very stiff phi = 31 degrees C = 415 p.c.f.	25.0	91.8
5						**Santa Clara Formation** light olive clay cuttings		
10		4-2	40	CH		Silty clay; light olive and brownish yellow with grey mottles, calcareous concretions, fractured, crumbly, slightly moist, hard [mudstone]	17.2	98.2
		4-3	71	SM/ SC		Silty fine sand; yellowish brown, with minor clay binder, thin interbeds grey silty fine sand with clay, moist, very dense [weakly cemented sandstone]	9.4	128.1
15						Boring terminated at 15 feet No groundwater encountered		
20								
25								
30								

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BORING LOG B - 5

Project No. 2645SC01-GA

Driller Hillside Geotechnical
Drill Rig B24
Depth to Groundwater NGWE
Date Drilled 13 June 2007
Logged by JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
		5-1	25	CH		Clay; dark brown and pale brown hues, pale hues calcareous, with trace very coarse angular sand, moist, very stiff	19.0	101.3
5						**Santa Clara Formation**		
		5-2	58			Siltstone and very fine sandstone; light yellowish brown and light grey, siltstone slightly calcareous, weakly cemented, friable, well consolidated, dry, very dense	17.6	111.2
10						Boring terminated at 10 feet No groundwater encountered		
15								
20								
25								
30								

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BORING LOG B - 6

Project No. 2645SC01-GA

Driller Hillside Geotechnical
Drill Rig B24
Depth to Groundwater NGWE
Date Drilled 13 June 2007
Logged by JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
		6-1	28	CH		Clay; very dark grey, with calcareous nodules and crusts, dry to slightly moist, very stiff	17.3	105.7
5						**Santa Clara Formation**		
		6-2	50			Mudstone; yellowish brown, faintly mottled with grey, calcareous on fracture surfaces, well consolidated, slightly moist, hard	15.4	114.5
10						Boring terminated at 10 feet No groundwater encountered		
15								
20								
25								
30								

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BORING LOG B - 7

Project No. 2645SC01-GA

Driller Hillside Geotechnical
Drill Rig B24
Depth to Groundwater NGWE
Date Drilled 13 June 2007
Logged by JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
				CH		Clay; dark yellowish brown, with abundant poorly sorted basalt clasts, dry, hard		
		7-1	71			auger refusal in basalt detritus	20.6	101.5
5						Boring terminated at 4 feet No groundwater encountered		
10								
15								
20								
25								
30								

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BORING LOG B - 8

Project No. 2645SC01-GA

Driller Hillside Geotechnical
Drill Rig B24
Depth to Groundwater NGWE
Date Drilled 13 June 2007
Logged by JEB

Depth (ft.)	Symbol	Sample No.	Blows / Ft.	U.S.C.S Soil Group	Pocket Pen. (t.s.f.)	DESCRIPTION	% Moisture	Dry Density (p.c.f.)
0						**Soil (Vertisol)**		
				CH		Clay; dark yellowish brown, with abundant poorly sorted basalt clasts, dry, hard		
		8-1	56				23.0	111.7
						auger refusal in basalt detritus		
5						Boring terminated at 4 feet No groundwater encountered		
10								
15								
20								
25								
30								

TABLE A1

Summary of Moisture, Density and Direct Shear Testing

Sample No.	Depth feet	In-Place Conditions: Moisture Content (% dry wt)	In-Place Conditions: Dry Density p.c.f.	Direct Shear Testing: Angle of Internal Friction (degrees)	Direct Shear Testing: Unit Cohesion p.s.f.
1-1	2-2.5	15.8	72.6		
2-1	3-3.5	26.5	87.7		
2-2	9.5-10	20.6	90.7		
3-1	3-3.5	--	--		
3-2	9.5-10	38.6	81.1		
3-3	14.5-15	37.4	83.3		
4-1	3-3.5	25.0	91.8	31	415
4-2	9.5-10	17.2	98.2		
4-3	14.5-15	9.4	128.1		
5-1	2-2.5	19.0	101.3		
5-2	9.5-10	17.6	111.2		
6-1	3-3.5	17.3	105.7		
6-2	9.5-10	15.4	114.5		
7-1	3-3.5	20.6	101.5		
8-1	3.3.5	23.0	111.7		

TABLE A2

Summary of Laboratory Atterberg Limits Test Results

Sample No.	Depth ft.	Description of Soil	Atterberg Limits: Liquid Limit %	Atterberg Limits: Plasticity Index (P.I.)
3-1	3-3.5	Dark greyish brown clay (CH)	81	60