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Geotechnical Investigation and Geologic Hazards Review

Four Single-Family Homes
Ticonderoga Drive
San Mateo, California

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation and geologic hazards review for four single-family homes to be located at Ticonderoga Drive in San Mateo, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions, update the engineering geology at the site, and provide geotechnical recommendations for design and construction of the proposed residential development.

For our use, we were provided with the following:

- A set of tentative maps, prepared by BKF Engineers, dated October 2005.
- A set of development plans, prepared by BKF Engineers, dated December 2, 2005.

1.1 Project Background

We previously prepared a geotechnical feasibility investigation for the Highland Estates Residential Development and presented the findings in our September 13, 2002 report titled "Geotechnical Feasibility, Highland Estates Residential Development, San Mateo County, California." The Highland Estates residential development consisted of approximately 97 acres; however, about 3.2 acres, which we are currently performing the geotechnical investigation and geologic hazards review, were previously designated as open space.

We understand that Soil Foundation Systems, Inc. (SFSI) performed geotechnical investigations for the Highland Estates Residential Development and presented the findings in their July 1993 and November 1994 reports. SFSI's report included a prior investigation by others. This prior investigation included logs from 49 test pits, as part of their geotechnical investigation for an approximately 11.9-acre site (a portion of the 97-acre Highland Estates parcel), in 1980. Eight test pits were located within the approximately proposed 3.2-acre parcel; however, only test pits TP-14, TP-15, and TP-16 were located within the planned development areas. The approximate locations of these test pits, relative to the planned development areas, are shown on the Site Plan, Figure 2.

Dr. Darwin Myers, C.E.G., previously performed engineering geologic work for the Highland Estates Residential Development. This work has been included in our report where it affects the proposed residences.

1.2 Project Description

The approximately 3.2-acre site is currently vacant with scattered trees, shrubs and vegetation. We understand that The Chamberlain Group plans to subdivide the site into four lots and develop them with single-family homes. We anticipate that the homes will consist of wood-framed structures with raised wood floors.

Based on the plans provided to us, the first floor finished elevation for the homes will be at about 507.5 to 510 feet above mean sea level (MSL), and second floor finished elevation will be at about 518 to 520 feet MSL. Due to the sloping terrain at the site, we anticipate that these homes will be supported on pier and grade beam foundations that step down the hillside. Associated underground utilities and driveways are also planned as part of the site development. Retaining walls will also be constructed to retain the proposed cuts and fills. The layout of the proposed homes is shown on the Site Plan, Figure 2.

Structural loads are yet to be determined. We assume that structural loads will be representative for this type of construction and will be relatively light on the order of 10 to 20 kips per pier. Site grading will consist of establishing the various pad and driveway grades. Additionally, grading to mitigate the shallow landslide deposits and reduce the steepness of the existing slope and to provide site access for the proposed homes is discussed in this report. Based on the plans provided, cuts up to 16 feet and fills up to 8 feet to establish level building pads and driveways are anticipated. Site retaining walls on the order of 5 to 9 feet high will also be constructed to retain the proposed cuts and fills.

1.3 Scope of Services

Our scope of services was presented in our agreement with you dated February 7, 2005. To accomplish this work, we provided the following services:

- Review of previous reports prepared for the site by SFSI, Berlogar, Long & Associates, and Lowney Associates.
- Exploration of subsurface conditions by drilling three borings and retrieving soil and bedrock samples for observation and laboratory testing.
- Evaluation of some of the physical and engineering properties of the subsurface soils and bedrock by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate building foundation, site earthwork, slabs-on-grade and retaining walls.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations for the proposed project.

2.0 GEOLOGIC SETTING

The site is located on the northwest side of Ticonderoga Drive approximately 75 feet to 700 feet from the intersection of Ticonderoga Drive and Allegheny Way. The site is situated along the western boundary of the City of San Mateo, on unincorporated land in San Mateo County, California at Latitude 37.5155° and Longitude 122.3385°. The proposed development and the topography of the area are shown on Figure 1.

The San Francisco peninsula is a relatively narrow band of rock at the north end of the Santa Cruz Mountains separating the Pacific Ocean from San Francisco Bay. It represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous to Jurassic age (70 to 200 million years old) rocks of the Franciscan Complex. Locally, these basement rocks are capped by younger sedimentary and volcanic rocks. Most of the Coast Ranges are covered by younger surficial deposits that reflect geologic conditions of the last million years or so.

Lateral and vertical movement along the many splays of the San Andreas Fault system and other secondary faults has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west. The San Andreas Fault system is about 40 miles wide in the Bay area and extends from the San Gregorio fault at the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified as well.

The proposed Highlands Estates development is situated approximately 0.9 miles northeast of the San Andreas Fault on the northeast side of Pulgas Ridge, which has generally northwest-aligned topography. This ridge and surrounding slopes are underlain by Franciscan Complex bedrock as shown on Figure 3. The site is situated on a southeast facing, moderate to steeply sloping hillside where shallow slope failures have been observed (Figure 2). To the southeast, directly across Ticonderoga Drive, a steep ravine provides drainage to the northeast into Polhemus Creek, which flows along the northeast face of Pulgas Ridge (along Polhemus Drive) towards the northwest.

Concepts of the origin of the Franciscan Complex have changed dramatically over the years. Present concepts have Franciscan rocks originating as oceanic plate material that was subducted and metamorphosed resulting in the mixed lithologies in a melange of harder, more intact blocks in a matrix of sheared shale as mapped by Pampeyan (1994) and shown on Figure 3.

3.0 SITE CONDITIONS

3.1 Exploration Program

Subsurface exploration was performed on March 8 and 9, 2005, using portable Minuteman drilling equipment to investigate, sample, and log subsurface soils and bedrock. Three exploratory borings were drilled to a maximum depth of 20 feet. The borings were backfilled with cement grout in accordance with San Mateo County Environmental Health Division guidelines. The approximate locations of the borings are shown on the Site Plan, Figure 2. Geotechnical cross sections A-A′, B-B′, C-C′, and D-D′, summarizing pertinent geotechnical data at the proposed homes, are presented in Figures 4A through 4D. The logs of our borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B. Previous test pits logs found in the SFSI report are attached in Appendix C.

3.2 Surface

Our Registered Geologist performed a reconnaissance of the site on February 2, 2005. This followed a review of aerial photographs (listed in the References) spanning the period from 1943 to 1973. At the time of this reconnaissance, the site consisted of a southeast-facing moderate to steep slope. Recent shallow slope failures were observed near the middle of the lot along Ticonderoga Road. Sandstone outcrops were observed along the Ticonderoga Road cut-slope near the middle and northeastern portions of the site. One seep was also noted along the road cut near the middle of the site.

3.3 Subsurface

Our borings encountered about 7 to 8 ½ feet of medium stiff to very stiff clay. Previous Plasticity Index (PI) test performed by SFSI for the near-surface clayey soil samples at the site area exhibited PI's ranging from 6 to 12 and Liquid Limit (LL) ranging from 32 to 42, indicating the near-surface soils have low to moderate plasticity and expansion potential.

As shown on Figure 2, Boring EB-2 was drilled in the active shallow landslide area. The test results indicate that the clay landslide deposits have a relatively lower density and penetration resistance compared to the surficial clay in Borings EB-1 and EB-3.

Below the surficial clay and/or clay landslide deposit layer, very soft to soft, moderately to severely weathered sheared bedrock of the Franciscan Formation was encountered to a maximum depth explored of 20 feet. A PI test was performed on a clayey sheared bedrock sample in Boring EB-2 at a depth of about 19½ feet. The test results exhibited a PI of 13, indicating that the sheared bedrock has low plasticity and expansion potential. This correlates with previous PI test results performed by SFSI, which exhibited PI's ranging from 4 to 10.

Test Pits TP-14, TP-15, and TP-16 excavated by Berlogar, Long & Associates (1980) encountered about 5½ to 10½ feet of clayey soils over bedrock of Franciscan Formation to a maximum depth explored of 11½ feet. In general, their test pits encountered similar materials to our borings.

3.4 Ground Water

No free ground water was encountered in any of our borings to a maximum depth of 20 feet. However, seepage of ground water was noted along the cut-slope for Ticonderoga Drive. Fluctuations in the level of the ground water may occur due to variations in rainfall, irrigation, and other factors not in evidence at the time our measurements were made. Please note that perched ground water conditions may be encountered in the bedrock fractures and the overlying soil.

4.0 GEOLOGIC HAZARDS

A qualitative evaluation of some geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

4.1 Fault Rupture

A regional fault map showing active faults relative to the site is presented in Figure 5. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, known formerly as a Special Studies Zone (CDMG, 2001), and no known surface expression of active faults is believed to exist within the site. The closest active fault to the site is the San Andreas, which passes beneath Crystal Springs Reservoir to the southwest. The site is about 0.9 miles from the San Andreas Fault. Primary fault rupture through the site, therefore, is not anticipated.

4.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to all developments in the San Francisco Bay Area. The "Seismicity" section that follows summarizes potential levels of ground shaking at the site.

4.3 Liquefaction

Liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands or silts. The Franciscan Complex bedrock underlying the site has a "very low" susceptibility to liquefaction (Knudsen et al., 2000). It is our judgment that the potential for liquefaction occurring in soil and bedrock at the site during seismic shaking is very low.

4.4 Differential Compaction

If near-surface materials at the site vary in composition either vertically or laterally, major earthquake shaking can cause non-uniform compaction, resulting in settlement of the materials and overlying facilities. This can also occur gradually over a long period of time. The site is located within a Franciscan melange zone (Pampeyan, 1994), and varying rock types typical of Franciscan melange zones were observed within the site. These differing rock types each have different strength characteristics. Structures should be found on similar materials or designed to accommodate differential compaction. Provided that the shallow landslide deposits are moved and replaced, it is our judgment that the potential for differential compaction occurring in soil and bedrock at the site to impact the proposed development during seismic shaking is low.

4.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying material toward an open face such as an excavation, channel, or body of water. Generally in soils, this movement is due to failure along a weak plane and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free.

At the time of this writing, the proposed site consisted of moderate to steep slopes near Ticonderoga Drive with a narrow band of relatively level topography along the northwestern boundary. Since the potential for liquefaction to occur at the site is considered to be low, it is our judgment that the potential for lateral spreading occurring in soil and bedrock of the site during seismic shaking is low.

4.6 Landsliding

The site is located within a hilly area with slopes described by Pampeyan (1994) as "unstable, especially when wet," and where small isolated landslides were mapped nearby by Brabb and Pampeyan (1972) and Leighton (1973). Although the site is located on the upper elevations of a steep sided ravine, no geomorphic indications of recent slope movement were identified on aerial photographs. During the site reconnaissance the minor slope failures that were previously mapped at the site were noted and are likely associated with the road-cut created for Ticonderoga Drive (Figure 2). Brabb et al. (1978) compiled a landslide susceptibility map for San Mateo County. This map shows the site within an area designated as moderately susceptible to landsliding generally based on slopes of greater than 30%, but also includes areas with 15% to 30% that are underlain by unstable rock units. Wieczorek et al. (1985) published a map depicting slope stability during earthquakes in San Mateo County. Most of the site is located in an area mapped as having moderate susceptibility, and the northwest portion of the site is shown as having very low susceptibility to landsliding triggered by a major earthquake (Figure 6). While this is a regional map and is not intended for site specific planning, it does show the hazard needs to be addressed in future developments located on these slopes. Since the proposed site is currently planned on the moderate to steep slopes near the crest of Pulgas Ridge, which is underlain by sheared rocks of the Franciscan Formation, we judge the potential for landsliding to be low in the bedrock material and moderate to high in the mapped landslide deposit areas. The existing shallow slope failures are deemed to be the result of slope over steepening associated with the construction of Ticonderoga Drive and can be mitigated during the grading phase of development. Detailed recommendations for mitigating shallow slope failures are presented in the "Fill Slopes and Drainage" section of this report.

4.7 Seismically Induced Waves

The site is situated more than 3 miles from San Francisco Bay at elevations ranging from 450 to 530 feet. This location is well above and beyond the maximum projected runnup by seismically generated tsunamis. The site is also not located next to any major drainage areas that would be affected by or generate a seismically induced wave. Therefore, this potential hazard is not anticipated to be a problem at the site.

4.8 Flooding

The proposed development is located on a hilltop so the only surface waters are the result of rain falling on the site itself or import water for irrigation. While either of these sources is capable of minor local flooding caused by plugged drains, adequate design and maintenance should reduce this hazard to a minor problem.

5.0 SEISMICITY

5.1 Regional Active Faults

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the Bay area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The San Andreas Fault, which passes approximately 0.9 miles southwest of the site, generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989. Two other major active faults in the site region are the San Gregorio fault, located 8.3 miles southwest of the site, and the potentially active Monte Vista - Shannon fault, located 7.4 miles to the southeast.

5.2 Maximum Estimated Ground Shaking

The Probabilistic Seismic Hazard Analysis (PSHA) performed by the California Geological Survey (2003) estimates a peak horizontal ground acceleration of 0.68g with a 10 percent probability of exceedance in 50 years for the site.

5.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. Our current understanding of earthquake activity indicates, however, the site will likely be subject to at least one moderate to severe earthquake within the next 30 years. During such an earthquake the risk of fault offset at the site is slight, but strong shaking of the site is likely to occur.

The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003), referred to as WG02, estimates that there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2002 and 2031. The probability of a magnitude 6.7 earthquake on the Peninsula segment of the San Andreas Fault which is closest to the site is believed to be 21%. During such an earthquake the danger of fault ground rupture at the site is slight, but strong ground shaking would occur. This result is the most important outcome of WG03's work, because any major earthquake can cause damage throughout the region.

This was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can inflict damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

5.4 California Building Code (CBC) Site Seismic Coefficients

The CGS issued maps locating "Active Fault Near-Source Zones" to be used with the 2001 CBC ("Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," CDMG/ICBO February 1998). Faults are classified as either "A," "B," or "C" as shown below. Only faults classified as "A" or "B" are mapped since faults classified as "C" do not increase the near-source factor.

Table 1. Seismic Source Definitions

Seismic Source Type	Seismic Source Description	Seismic Source Definition*	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/yr)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C.	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$	$SR \leq 2$

*Note: Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining seismic source type.

The following table lists Type A and Type B faults within 25-kilometers of the site.

Table 2. Approximate Distance to Seismic Sources

Fault	Seismic Source Type	Distance (miles)	Distance (kilometers)
*San Andreas (1906)	A	0.9	1.4
**Monte Vista - Shannon	B	7.4	11.8
San Gregorio	A	8.3	13.3

*Nearest Type A fault
**Nearest Type B fault

The 2001 CBC describes the procedure for determining soil profile types S_A through S_F in accordance with Section 1636.2 and Table 16-J. Based on our borings and published geologic maps, the site consists of about 7 to 8½ feet of medium stiff to very stiff clay overlying soft bedrock of Franciscan Formation. Based on our experience in the site vicinity, the hardness of the Franciscan is bedrock likely increasing with depth. For this reason, we judge that a soil profile type S_C, generally described as very dense soil or soft rock, is appropriate for design. Based on this information and local seismic sources, the site may be characterized for design based on Chapter 16 of the 2001 CBC using the information in Table 3 below.

Table 3. 2001 CBC Site Categorization and Site Seismic Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_c
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	San Andreas
Seismic Source Type (Table 16-U)	A
Distance to Seismic Source (kilometers)	1.4
Near Source Factor N_a (Table 16-S)	1.5
Near Source Factor N_v (Table 16-T)	2.0
Seismic Coefficient C_a (Table 16-Q)	0.60
Seismic Coefficient C_v (Table 16-R)	1.12

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

From a geotechnical engineering viewpoint the proposed homes may be constructed as planned, provided design and construction are performed in accordance with the recommendations presented in this report.

The primary geotechnical concerns at the site are as follows:

- Presence of active shallow landslides
- Undocumented backfill of former test pit excavations
- Differential settlement of cut and fill transitions

We have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the development.

6.1.1 Presence of Active Shallow Landslides

As previously discussed, most of the site is located in an area mapped as having moderate susceptibility, and the portion to the northwest is shown as having very low susceptibility to landsliding triggered by a major earthquake (Wieczorek et al., 1985). In addition, the existing shallow slope failures are the result of slope over steepening associated with the construction of Ticonderoga Drive. The approximate limits of the landslide are shown on the Site Plan, Figure 2. Mitigation measures for the shallow landslide are discussed in the "Earthwork" section below.

6.1.2 Undocumented Backfill of Former Test Pit Excavations

Test Pits TP-14, TP-15 and TP-16 are located within the proposed development area and were excavated to depths between 7 to 11½ feet below the existing site grade. These test pits should be removed and replaced with engineered fill. Test pits outside the planned development area need not to be removed and recompacted. Detailed recommendations are presented in the "Earthwork" section of this report.

6.1.3 Cut and Fill Transitions

The development plans indicate that fills on the order of 5- to 7-feet-thick are proposed for the driveway of two parcels to the east. To reduce the potential for differential movement beneath the driveway, the cut and fill transition area should be over-excavated to create a uniform pad. Detailed recommendations are presented in the "Earthwork section of this report.

6.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and test the geotechnical aspects of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation, and if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

7.0 EARTHWORK

7.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing landslide deposits, fills, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. We recommend that trees and shrubs designated to be removed should include the entire rootball and all roots larger that ½-inch in diameter. Depressions resulting from removal of trees and shrubs should be cleaned of loose soils and roots, and properly backfilled in accordance with the "Compaction" section of this report. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures be carried out under our observation and that backfill be tested during placement. Our geologist should confirm that the landslide materials have been removed during site grading.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 2 to 4 inches would be required. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

7.2 Removal of Shallow Landslide Deposits

Based on our borings and field observations, we estimate that the shallow landslide deposits extend to a depth up to about 7 feet at Lots 6, 7 and 8. To reduce damage to the planned structures, we recommend all shallow landslide deposits within the proposed development area be removed down to the native competent material and replaced as engineered fill. The landslide deposits beyond the development area (behind the retaining wall in the rear of the properties) may remain in-place provided that the retaining wall is designed in accordance with Table 4 of the "Retaining Walls" section of this report.

7.3 Undocumented Backfill of Former Test Pit Excavations

Test Pits TP-14, TP-15 and TP-16 are located within the proposed development area and were excavated to depths between 7 to 11½ feet below the existing site grade. Since there are no records of test pit backfill and compaction, we recommend that these test pits be over-excavated and recompacted with engineered fill if they are encountered during grading. Please note that these test pits will likely be remediated during construction of the fill slope at the site. Test pits outside the planned development area need not to be removed and recompacted.

7.4 Cut and Fill Transitions

To minimize the effects of differential movement beneath the driveway, we recommend that the cut and fill transition area be over-excavated to a depth of at least 2 feet below the proposed finished subgrade to create a uniform pad. However, adjustments to the depth of the over-excavation may need to be made at the time of construction depending on the actual conditions encountered during grading.

7.5 Abandoned Utilities

Abandoned utilities within the proposed building areas should be removed in their entirety. Utilities within the proposed building areas would only be considered for in-place abandonment provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structures.

Utilities outside the building areas should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for small diameter pipes (4 inches or less) abandoned in-place and increasingly higher with increasing diameter.

7.6 Subgrade Preparation

After the new construction areas have been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment. If the subgrade consists entirely of sandstone bedrock, scarifying of the exposed subgrade may not be required, as directed by our field engineer.

7.7 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension. Rocks or lumps larger than 4 inches should not be allowed to nest together. Rocks that nest together can cause bridging effects resulting in inadequate compaction.

It is noted that excavation of the sandstone at the site may produce rocks of lumps greater than 6-inches in largest dimension. The contractor should anticipate some "breaking down" time to reduce the rock size below 6-inches for reuse as fill. If a significant amount (i.e., more than 25 percent) of rocks or lumps greater than 6 inches is encountered, we should be contacted to evaluate the material and provide supplemental recommendations, if needed.

Imported and non-expansive fill materials should be inorganic and should have a Plasticity Index of 15 or less. Imported fill should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of proposed import fill should be submitted to us at least 10 days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials.

7.8 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness at a moisture content slightly above the laboratory optimum. Each successive lift should be firm and non-yielding under the weight of construction equipment. Fill greater than 5 feet in thickness should be compacted to at least 92 percent compaction for the portion below the upper 5 feet.

In pavement areas subject to vehicular traffic, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

7.9 Wet Weather Conditions

Earthwork such as fill placement and trench backfill may be very difficult during wet weather, especially for fill materials with a significant amount of clay. If the percent water in the fill increases significantly above the optimum moisture content, the soils will become soft, yielding, and difficult to compact. Saturated soils may require aerating or blending with drier soils to achieve a workable moisture content. Therefore, we recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

7.10 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas.

7.11 Temporary Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

7.12 Drainage

7.12.1 General Site Surface Drainage

Surface water should not be allowed to flow over retaining walls. Ponding of surface water should not be allowed at the top or bottom of slopes, adjacent to retaining walls, or on pavement. Positive surface gradients of at least 2 percent in unimproved areas and at least 1 percent on pavements should be provided to direct surface water toward suitable discharge facilities. Level areas above slopes should be graded to a 2 percent gradient or greater to direct surface water away from the top of slopes toward a suitable point of discharge such as concrete lined ditches or surface drain inlets. At a minimum, we recommend the surface drainage be designed in accordance with the latest edition of the 2001 CBC.

We recommend that a concrete-lined V-ditch be constructed behind the retaining wall in the rear of the properties to intercept surface run-off water. The V-ditch should be appropriately sized for maximum storm water flows based on the upslope tributary area and should discharge to appropriately sized drainage inlets. The concrete-lined V-ditch should be adequately reinforced and have adequate construction and control joints. Forming and backfilling around the concrete-lined V-ditch should not be allowed.

If irrigation of open-space areas or properties adjacent to the upslope side of the development occurs, both short-term and long-term drainage impacts to the development may occur and may not be observed for several years. If irrigation of property located upslope of the development occurs (including open-space area parts of the development), then additional surface and subsurface drainage measures may need to be installed. These measures may include installing or increasing the size of the drainage ditch. TRC Lowney should be consulted if irrigation will occur near the upslope side of the development or within adjacent open-space areas.

7.12.2 Lot Surface Drainage

Positive surface water drainage gradients (1 percent minimum in hardscape areas and 2 percent minimum in native soil areas) should be provided within 5 feet of the buildings to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to buildings or slabs-on-grade. Roof gutters should be used on all buildings. Roof downspouts should be connected to solid pipes that transmit storm water onto paved roadways, into drainage inlets, or into storm drains.

In order to minimize water induced impacts, we recommend that the homeowner's be advised in the projects CC&R's to perform regular maintenance of their lots and the open-space areas, including maintenance prior and after rainstorms. Maintenance should include the re-compaction of loosened soils, collapsing and infilling holes and burrows with compacted soils or low strength sand/cement grout, removal and control of burrowing animals, modifying storm water drainage patterns to allow for sheet flow into drainage inlets or ditches rather than concentrated flow, removal of debris within drainage ditches and inlets, and immediately repairing any erosion or soil flow. The inspection should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made.

If desired to minimize surface run-off water from migrating into the building perimeters, a subdrain system may be installed around the perimeter of the proposed homes to intercept the water. This subdrain system should consist of a 4-inch minimum diameter perforated pipe (perforations placed downward) placed about 1-foot from the perimeter of the foundation and extend at least 2 feet below the finished grade. The perforated pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 1-foot of backfill should consist of relatively low permeability compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet.

7.13 Erosion Control

Based on the development plans provided by BKF Engineers, no fill slopes will be constructed at the site. Cut slopes to approximately 4:1 (horizontal:vertical) will be performed on the two parcels to the west. As with any hillside development, exposed slopes require periodic maintenance due to minor sloughing and erosion as well as protection if grading during the winter. To minimize this potential for erosion, we recommend that permanent erosion control measures be placed on all slopes. The establishment of permanent erosion control measures is beneficial for long-term aesthetics, reduces erosion by slowing runoff velocities, enhances infiltration and transpiration, traps sediment and other particles and protects soil from raindrop impact.

A Storm Water Pollution Prevention Plan (SWPPP) should be prepared with the grading plans to fulfill the requirements of the State of California's General Permit to Discharge Storm Water Associated with Industrial Activity (General Permit). Federal Regulations for controlling pollutants in storm water run-off discharges, as described in Title 40, Code of Federal Regulations (CFR) Parts 122, 123, 124. TRC Lowney can provide the SWPPP preparation and monitoring services during the winter months.

7.14 Construction Observation

A representative from our company should observe and test the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including, site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule, we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

8.0 FOUNDATIONS

Provided that the site is prepared in accordance with the "Earthwork" section of this report, the proposed homes may be supported on a drilled cast-in-place, straight-shaft friction pier and grade beam foundation system.

8.1 Friction Piers

We recommend that the proposed structure be supported on drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of at least 16 inches and extend to a depth of at least 10 feet below the adjacent finished grade or 5 feet into bedrock, whichever is greater. Piers for site retaining wall may be extended to 5 feet below the adjacent finished grade or 3 feet into bedrock, whichever is greater. Piers may be designed for an allowable skin friction of 500 pounds per square foot for combined dead plus live loads with a one-third increase allowed for either transient wind or seismic loading. Piers should have a minimum center-to-center spacing of at least three pier diameters. Grade beams should be designed to span between piers in accordance with structural requirements.

Resistance to uplift loads will be developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 400 pounds per square foot be used.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the excavation of all piers be performed under our direct observation to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

If ground water is encountered and cannot be removed from pier holes prior to concrete placement, then concrete will need to be placed by tremie pipe. The concrete should be tremied to the bottom of the hole, keeping the tremie pipe below the surface of the concrete, to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole.

Total settlement for the recommended pier foundations should not exceed ½-inch and post construction differential settlement across the building founded on pier foundations should be less than ½-inch due to static loads.

8.2 Lateral Loads

Lateral loads exerted on structures supported on piers and grade beams may be resisted by a passive resistance based on an equivalent fluid pressure of 300 pounds per cubic foot acting against twice the projected area of the individual pier shaft below rough pad grade, with a maximum of 2,000 pounds per square foot at depth. The upper 12 inches of piers should be neglected when determining the lateral capacity of the piers.

9.0 CONCRETE SLABS-ON-GRADE

9.1 Interior Slabs-On-Grade

Since the expansion of the clayey soil and bedrock may vary across the site, we recommend that interior slab-on-grade floors be at least 4 inches thick and supported on at least 4 inches of ¾-inch crushed rock or Class 2 aggregate base to reduce the likelihood of slab damage from heave. Garage slabs should be at least 5 inches thick. We also recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil is expansive. In our opinion, consideration should be given toward a maximum control joint spacing of 2 feet per inch of concrete thickness in both directions for the interior slab-on-grade construction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Protection Considerations" section below should be considered.

9.2 Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. We note that some of these guidelines are different from local practice, and emphasize that they should be considered as the owner's option.

The purpose of these guidelines is to aid in producing concrete slabs of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with concrete construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 10-mil thick vapor barrier meeting minimum ASTM E 1745, Class C requirements should be placed directly below the slab (no sand). The vapor barrier should extend to the edge of the slab. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break. The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations. The crushed rock may be included as the upper 4 inches of non-expansive fill thickness.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels should not be permitted.

- When using Type I cement, all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 7 days. When using Type II cement, all concrete surfaces to receive any type of floor covering should be moist cured for a minimum of 14 days.

- Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. The application of these guidelines does not affect the geotechnical aspects of the foundation performance.

9.3 Exterior Concrete Flatwork and Sidewalks

We recommend that exterior concrete flatwork and sidewalk be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. The subgrade should also be compacted to at least 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If concrete flatwork and sidewalks are subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.

10.0 RETAINING WALLS

10.1 Lateral Earth Pressures

Any proposed conventional retaining walls, such as block masonry, wood walls, or cast-in-place concrete should be designed to resist lateral earth pressures from adjoining natural materials and/or backfill as well as from any surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls be designed to resist the lateral earth pressures presented in Table 4 below.

Table 4. Conventional Retaining Wall Lateral Earth Pressures

Backfill Inclination (horizontal:vertical)	Equivalent Fluid Pressure*		
	Unrestrained	Restrained	Landslide Deposit
Level	45 pcf	45 pcf + 8H psf	85 pcf
3:1	55 pcf	55 pcf + 8H psf	95 pcf
2.5:1	60 pcf	60 pcf + 8H psf	100 pcf
2:1	65 pcf	65 pcf + 8H psf	105 pcf

* Assumes drained conditions, add 40 pcf to the above values for undrained conditions.
H is the distance in feet between the bottom of the footing and the top of the retained soil.

Unrestrained walls should also be designed to resist an additional uniform pressure equivalent to one-third of any uniform surcharge loads applied at the surface; restrained retaining walls should also be designed to resist an additional uniform pressure equivalent to one-half of any uniform surcharge loads.

The above lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended in Table 4 for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture would be undesirable.

As previously discussed, the shallow landslide deposits beyond the development area (behind the retaining wall in the rear of the properties) may remain in-place provided that the retaining wall is designed to resist an equivalent soil unit weight as shown in Table 4 above. If the landslide deposits are removed and replaced as engineered fill, the unrestrained and/or restrained design parameters provided in Table 4 may be used for design.

10.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low pervious compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.3 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 92 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

Basement wall backfill should not proceed until interior walls and floors are poured and cured; otherwise, sufficient bracing will need to be provided. We recommend that our firm provide more frequent observation and testing during wall backfill for basement walls, which can be critical to the performance of surface improvements.

10.4 Foundation

Retaining walls may be supported on a drilled pier foundation system designed in accordance with the recommendations presented in the "Friction Piers" section of this report. An allowable passive resistance based on an equivalent fluid pressure of 300 pounds per cubic foot acting against twice the projected area of the individual pier shaft below finished grade, with a maximum of 2,000 pounds per square foot at depth, may be used for design. The upper 12 inches of piers should be neglected when calculating the lateral passive capacity of the piers.

11.0 PAVEMENTS

11.1 Asphalt Concrete

Because surface soils vary across the site, we judged an R-value of 5 to be applicable for design. If desired, an R-value testing may be performed once cuts for the driveways are made. If laboratory testing indicates a significantly higher R-value, it may be feasible to reduce the design pavement sections. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 5.

Table 5. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R-Value = 5

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	7.5	10.0
	4.5	2.5	9.5	12.0
Automobile Parking Channel	5.0	3.0	10.0	13.0
	5.5	3.0	12.0	15.0
Truck Access & Parking Areas	6.0	3.5	12.5	16.0
	6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study.

11.2 Portland Cement Concrete Pavements

Based on the design procedure developed by the Portland Cement Association, we recommend that Portland Cement Concrete (PCC) pavements subjected to automobile wheel loads be at least 5 inches thick and supported on at least 6 inches of aggregate base compacted to at least 95 percent relative compaction. Where heavier loads are expected, we recommend a minimum 6-inch-thick slab be used. Our design assumes a laterally restrained, unreinforced concrete section with a 28-day compressive strength of at least 3,500 pounds per square inch, and a subgrade soil R-value of 5. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

11.3 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

12.0 LIMITATIONS

This report has been prepared for the sole use of The Chamberlain Group, specifically for design and construction of four single-family homes at Ticonderoga Drive in San Mateo, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between explorations do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC Lowney cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC Lowney's report by others. Furthermore, TRC Lowney will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or 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 the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

13.0 REFERENCES

13.1 Literature

Berlogar, Long & Associates, 1980, *Geotechnical Investigation, 11.9 Acre Parcel, Polhemus Road and Ticonderoga Drive, San Mateo County for Ponderosa Homes,* Job No. 805-10.

Brabb, E.E., Pampeyan, E.H., and Bonilla, M.G., 1972, *Landslide susceptibility in San Mateo County, California*: U.S. Geological Survey Basic Data Contribution 43, scale 1:62,500.

Brabb, E.E., Pampeyan, E.H., and Bonilla, M.G., 1978, *Landslide susceptibility in San Mateo County, California*: U.S. Geological Survey Miscellaneous Field Studies Map MF-360, scale 1:62,500.

California Building Code, 2001, *Structural Engineering Design Provisions*, Vol. 2.

California Division of Mines and Geology [CDMG], 2000, *Official Map of Alquist-Priolo Earthquake Fault Zones, Special Studies Zones,* San Mateo Quadrangle-July 1, 1974, scale 1:24,000. CD-ROM 2000-004.

California Geological Survey [CGS], 2003, *Probabilistic Seismic Hazards Mapping, Ground Motion Page,* Seismic Shaking Hazards in California, April.

Knudsen, K.L., Sowers, J.M., Witter, R.C., and Helley, E.J., 2000, *Maps showing Quaternary geology and liquefaction susceptibility nine-county San Francisco Bay area, California*: National Earthquake Hazards Reduction Program, U.S. Geological Survey Award no. 1434-97-GR-03121, scale 1:275,000.

Leighton and Associates, 1973, *Geotechnical Hazards Synthesis Map for San Mateo County*, Official Map, County of San Mateo, Sheet 2 of 5, scale 1:24,000.

Lowney Associates, 2002, *Geotechnical Feasibility, Highland Estates Residential Development, San Mateo County, California,* Report No. 1291-2A.

Pampeyan, E.H., 1994, *Geologic map of the Montara Mountain and San Mateo 7-1/2′ quadrangles, San Mateo County, California*: U.S. Geological Survey Miscellaneous Investigations Series Map I-2390, scale 1:24,000.

Portland Cement Association, 1984, *Thickness Design for Concrete Highway and Street Pavements:* report.

Soil Foundation Systems, Inc., 1994, *Supplemental Geotechnical Report Responding to Geotechnical Review Comments for Highland Estates, San Mateo County, California,* File No. S24-634-2S.

Soil Foundation Systems, Inc., 1993, *Geotechnical Investigation Report for Highland Estates, San Mateo, California,* File No. S22-634-2.

State of California Department of Transportation, 1990, *Highway Design Manual*, Fifth Edition, July.

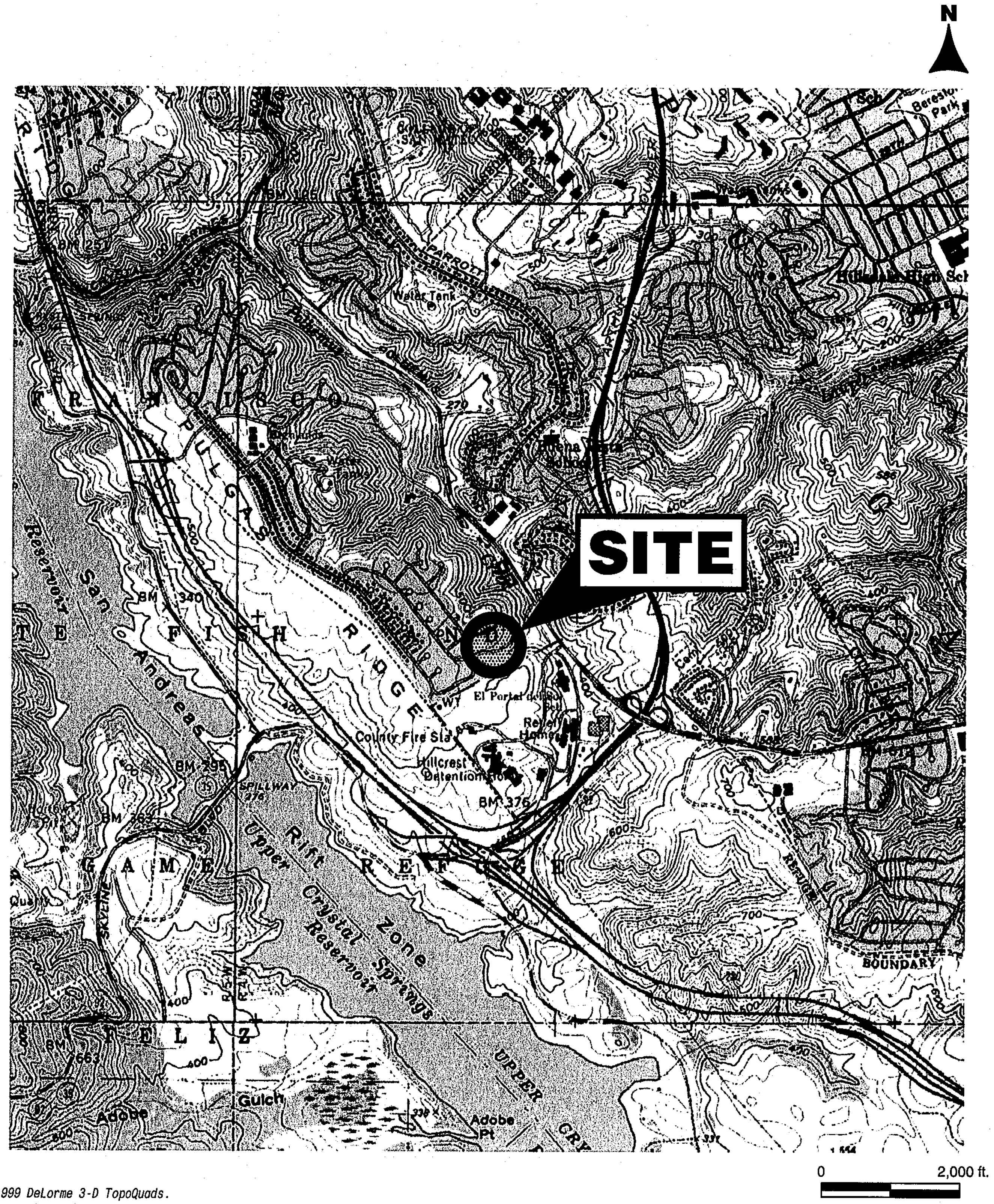
Weiczorek, G.F., Wilson, R.C., and Harp, E.L., 1985, *Map showing slope stability during earthquakes in San Mateo County, California*: U.S. Geological Survey Miscellaneous Investigations Series Map I-1257-E. scale 1:62,500.

Working Group on California Earthquake Probabilities [WGCEP], 2003, *Earthquake Probabilities in the San Francisco Bay Region: 2002 to 2031 - a summary of findings*: U.S. Geological Survey Open File Report 03-214.

13.2 Aerial Photographs

Geomorphic features on the following aerial photographs were interpreted at the U.S. Geological Survey in Menlo Park as part of this investigation:

Date	Flight	Frames	Scale	Type
October 11, 1943	DDB-2B	56, 57	1:20,000	black & white
May 27, 1956	DDB-2R	59, 60	1:20,000	black & white
May 9, 1973	3567-1	020, 021	1:12,000	black & white



© 1999 DeLorme 3-D TopoQuads.

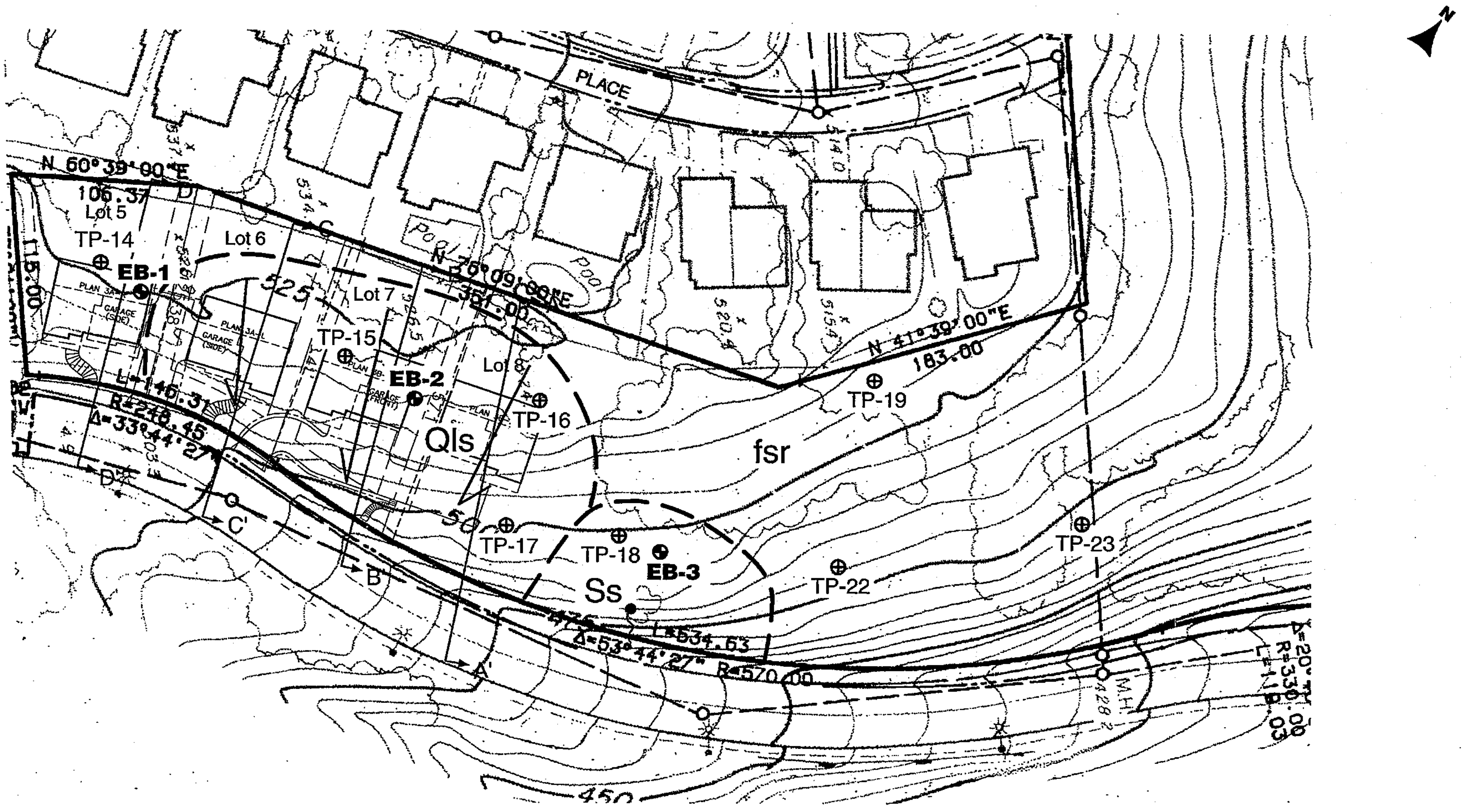
3/05*EB

VICINITY MAP

FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 1
1291-2B



0 60 ft.

Scale: 1" = 60'

LEGEND

- ● - Approximate location of exploratory boring
- ⊕ - Approximate location of previous test pits (Berlogar, 1980)
- ↑_↑ - Approximate location of geologic cross section
- ● - Approximate location of seep

EXPLANATION

Qls	Landslide Deposits
fsr	Sheared Rock (melange) Predominantly graywacke, siltstone and shale, substantial portions have been sheared, but includes hard blocks of all other Franciscan rock types.
Ss	Sandstone Mainly fine to medium grained graywacke sandstone which is very hard and massive

Base by BKF Engineers.

SITE PLAN

FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE

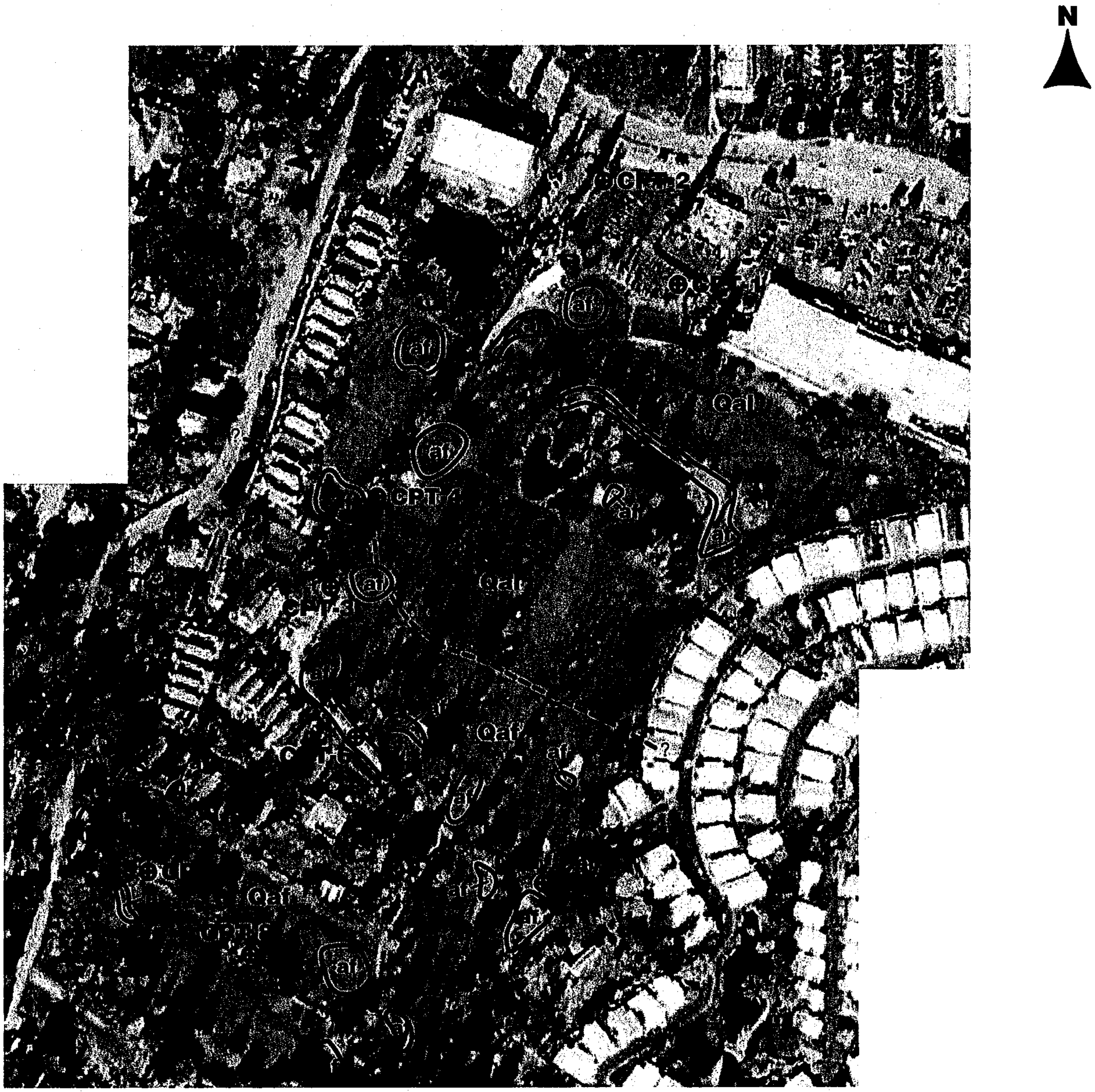
San Mateo, California

TRC Lowney

FIGURE 2

1291-2B

1/06*EB



LEGEND

⊕ - Approximate location of cone penetration test

◕ - Approximate location of exploratory hand auger boring

● - Approximate location of water well

——— – – – - Geologic contact; dashed where approximately located; queried where uncertain

MAP UNITS

af	Artificial fill, where obvious
Qal	Alluvium
Qaf	Alluvial fan deposits

Base approximated from Lowney Associates field notes.

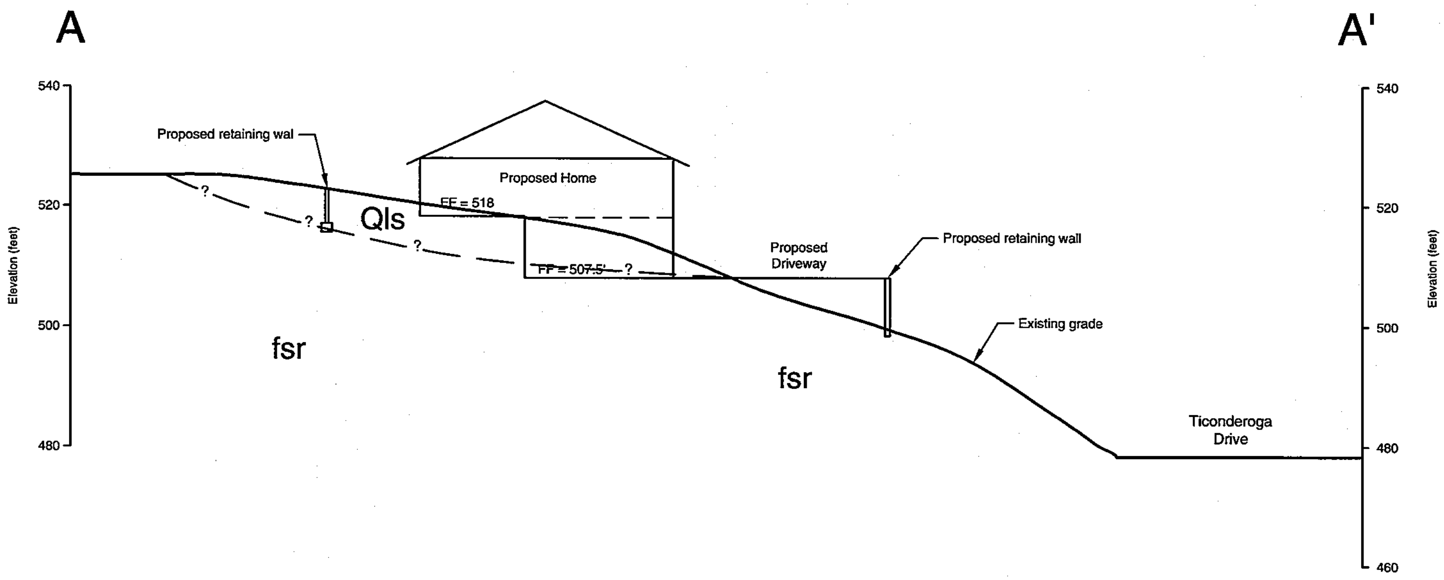
11/05*EB

SITE PLAN

VALLEY GARDENS GOLF COURSE
Scotts Valley, California

TRC Lowney

FIGURE 3
247-26A



LEGEND

Qls	Landslide deposits
fsr	Sheared rock of Franciscan formation

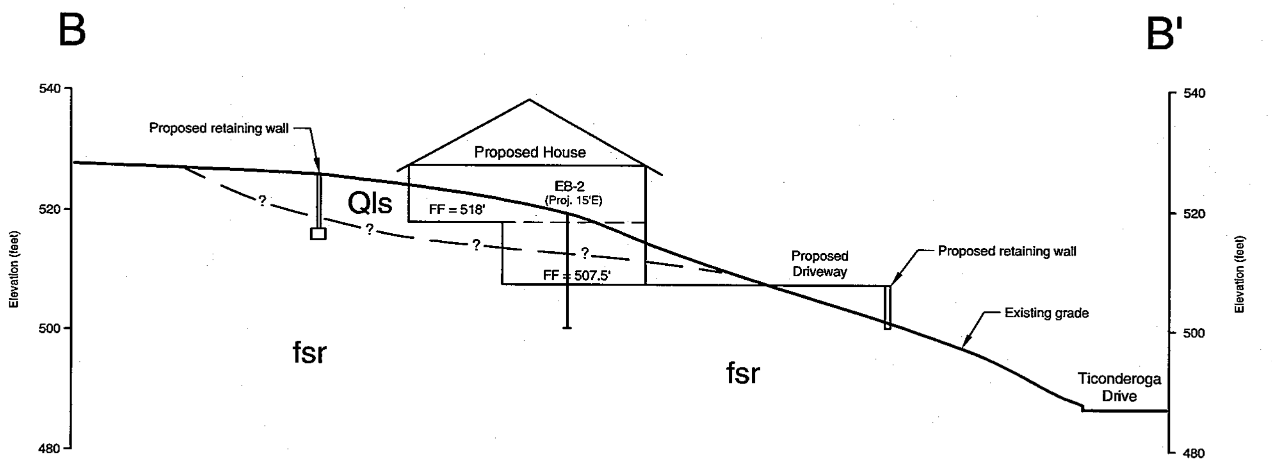
Horizontal = Vertical
0 20 ft.

GEOTECHNICAL CROSS SECTION A-A'
FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 4A
1291-2B

1/06*EB



LEGEND

Qls	Landslide deposits
fsr	Sheared rock of Franciscan formation

Horizontal = Vertical
0 20 ft.

GEOTECHNICAL CROSS SECTION B-B'

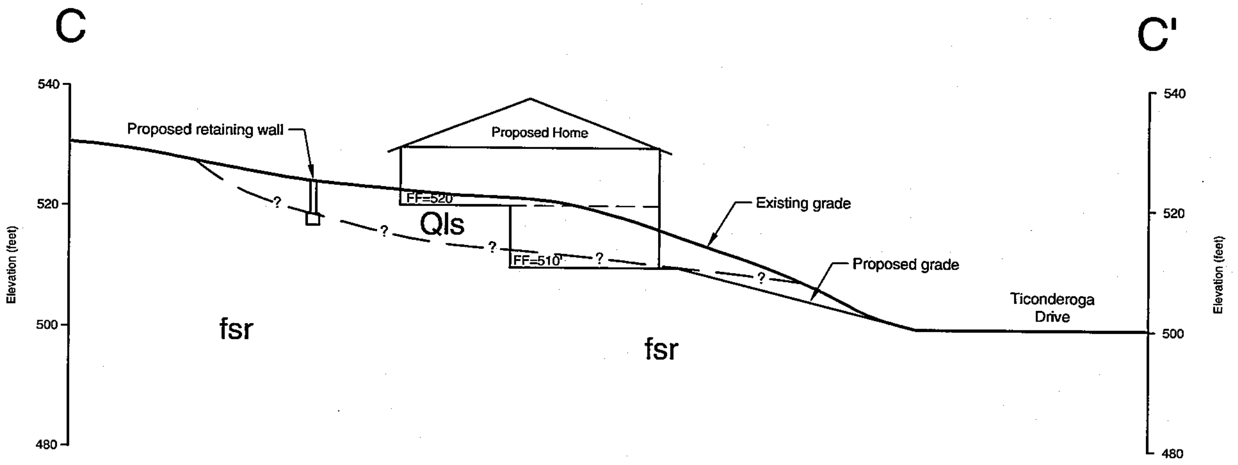
FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 4B

1291-2B

1/05*EB



LEGEND

Qls	Landslide deposits
fsr	Sheared rock of Franciscan formation

Horizontal = Vertical

0 20 ft.

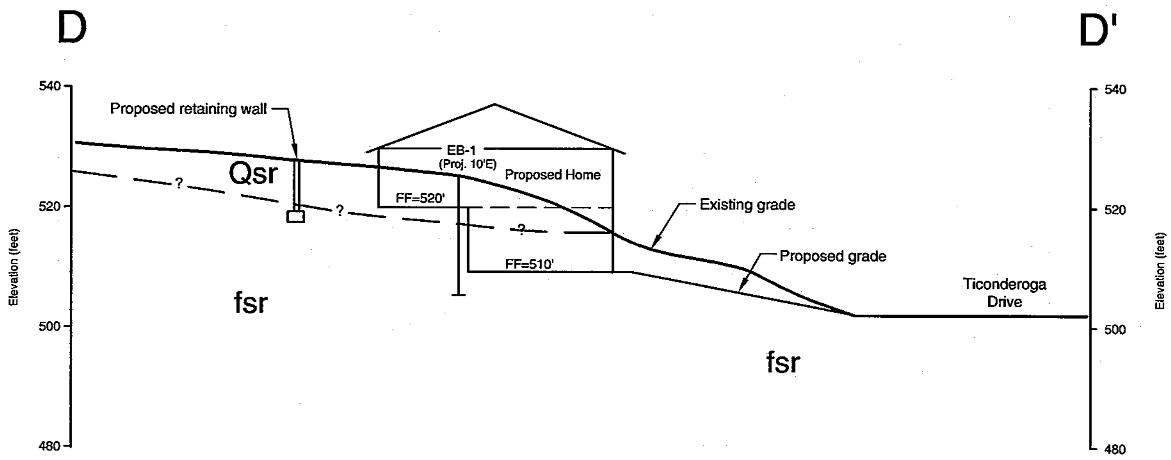
GEOTECHNICAL CROSS SECTION C-C'
FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 4C

1291-2B

1/06*EB



LEGEND

Qsr	Colluvium
fsr	Sheared rock of Franciscan formation

Horizontal = Vertical
0 20 ft.

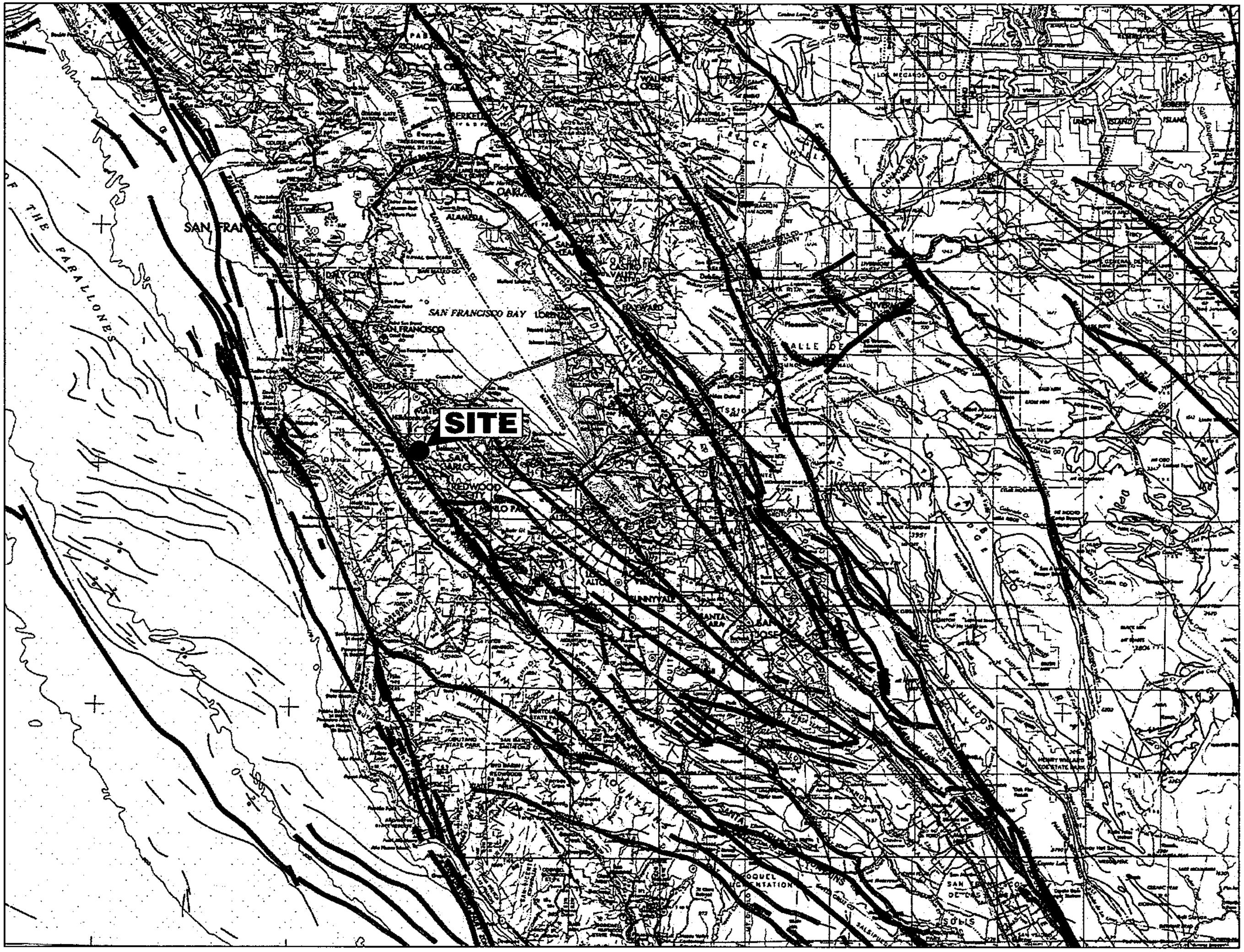
GEOTECHNICAL CROSS SECTION D-D'

FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 4D
1291-2B

1/06*EB



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore¹	DESCRIPTION
Quaternary	Late Quaternary	Historic		▬		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200	▬		Displacement during Holocene time.¹
		Pleistocene	10,000	▬		Faults showing evidence of displacement during late Quaternary time.¹·¹
	Early Quaternary		700,000	▬		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000	～		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

REGIONAL FAULT MAP

FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

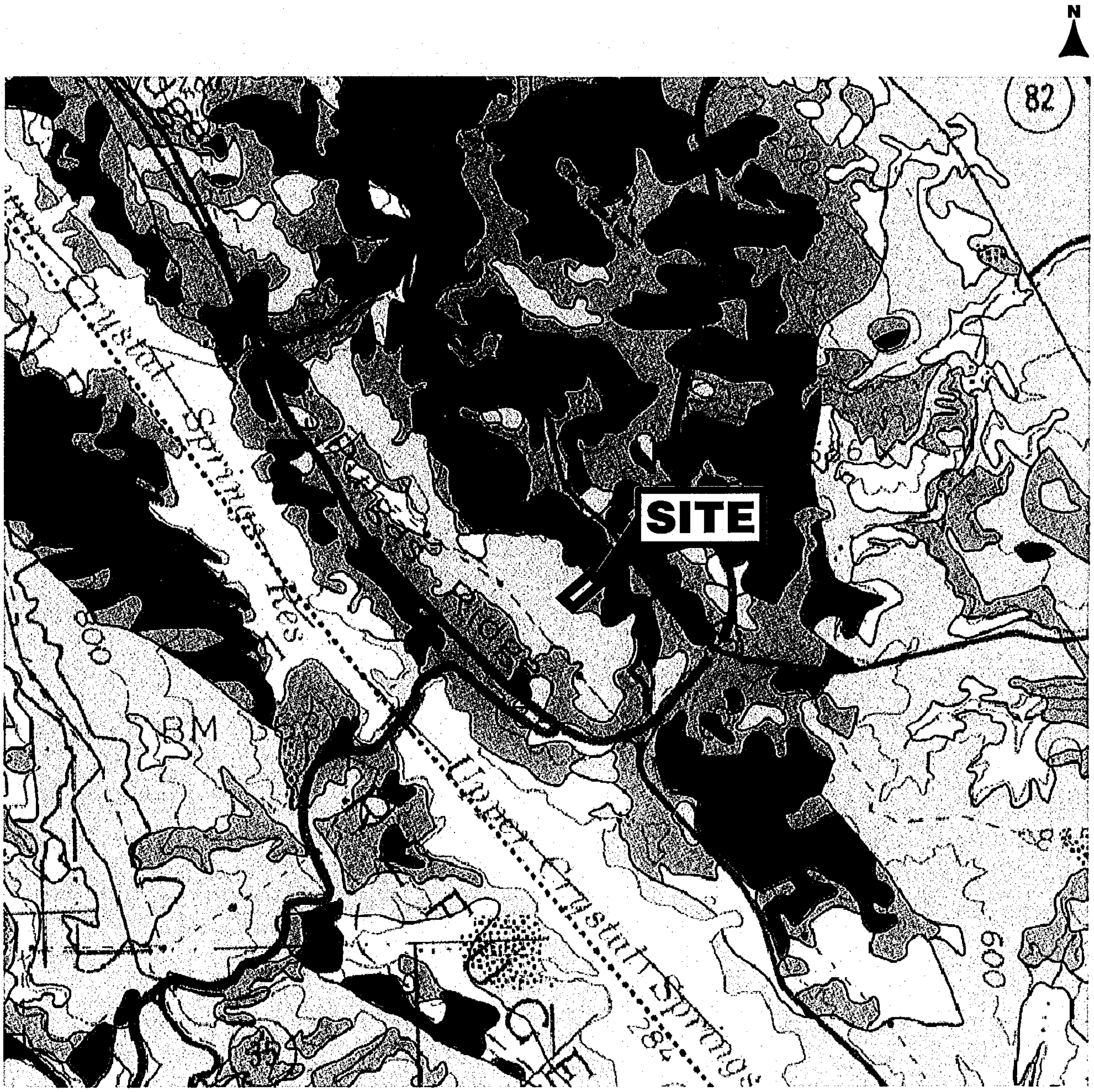
TRC Lowney

FIGURE 5

1291-2B

Division of Mines and Geology, James F. Davis, State Geologist

1/06'EB



EXPLANATION

Susceptibility and percentage of area likely to fail in a major earthquake

- ■ High..less than or equal to 25%
- ▨ Moderate ....approximately 15%
- □ Low ....approximately 5%
- □ Very Low ....less than 3%
- ▨ Area of low, moderate, or high liquefaction susceptibility
- —— Fault zone, approximately located

0 3,000 ft.

From: Wieczorek & others (1985)

1.06*EB

SLOPE STABILITY DURING EARTHQUAKES

FOUR SINGLE-FAMILY HOMES - TICONDEROGA DRIVE
San Mateo, California

TRC Lowney

FIGURE 6
1291-2B

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using portable minuteman solid-flight auger drilling equipment. Three 4-inch-diameter exploratory borings were drilled on March 8 and 9, 2005, to a maximum depth of 20 feet. The approximate locations of the borings are shown on the Site Plan, Figure 2. The soils and bedrock encountered were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

The locations of borings were approximately determined by pacing from existing structures and site boundaries. Elevations of the borings were estimated from a topographic map of the site. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil and bedrock samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring logs and related information depict subsurface conditions at the location indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the log represent the approximate boundary between soil or rock types and the transition may be gradual.

* * * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC		Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% 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	SM		Silty sands, sand-silt-mixtures, non-plastic fines
			SC		Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS 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 IS 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

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200 40 10 4 3/4" 3" 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 0.4 2 5 19 76mm

GRAIN SIZES

☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION

⧗ MODIFIED CALIFORNIA

▯ ROCK CORE

■ PITCHER TUBE

○ NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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FIGURE A-1

EXPLORATORY BORING: EB-1

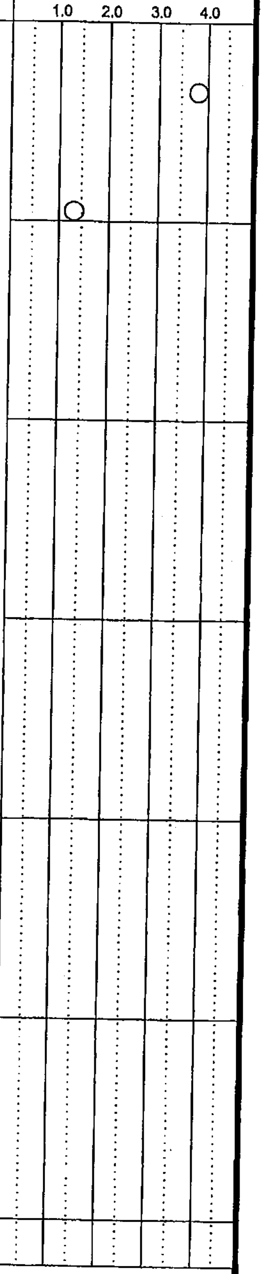
Sheet 1 of 1

DRILL RIG: MINUTE MAN
BORING TYPE: 4 INCH FLIGHT AUGER
LOGGED BY: BM
START DATE: 3-9-05 FINISH DATE: 3-9-05

PROJECT NO: 1291-2B
PROJECT: TICONDEROGA DRIVE
LOCATION: SAN MATEO, CA
COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION: 525 FT. (+/-)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression
525.0	0	▨	**LEAN CLAY WITH SAND (CL) [COLLUVIUM]** very stiff, moist, brown with reddish brown mottles, fine sand, some fine and coarse gravel, low plasticity	CL	17	⧗	14	101		
		▨			14	⧗	21	100		
519.8	5	▨	**LEAN CLAY (CL) [COLLUVIUM]** medium stiff to stiff, moist, gray, some fine and coarse gravel, moderate plasticity	CL						
517.0		⁘	**SANDSTONE [FRANCISCAN FORMATION (fsr)]** moderately to severely weathered, very soft, olive to brown	fsr	18	⧗	11	127		
	10	⁘								
	15	⁘			27	⧖	6			
		⁘	completely weathered, soft with hard seams, gray with bluish gray mottles		32	⧗	25	93		
505.0	20		Bottom of Boring at 20 feet							
	25									
	30									



GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 2/2/06 MV* FLL

TRC Lowney

EXPLORATORY BORING: EB-2

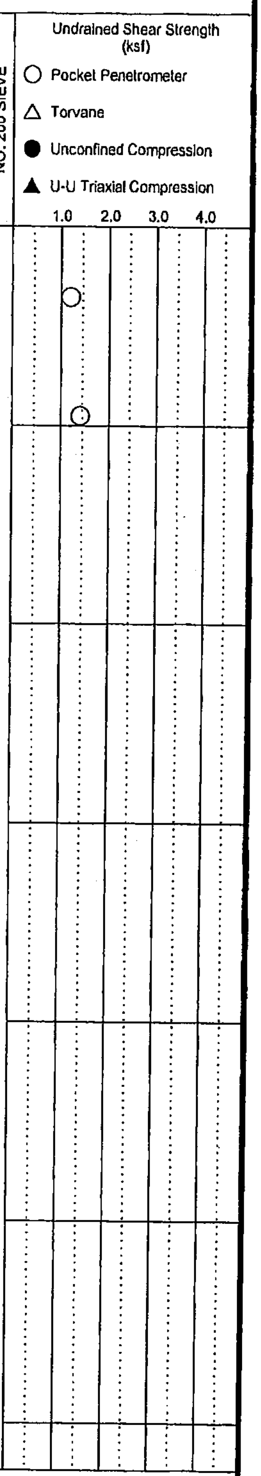
Sheet 1 of 1

DRILL RIG: MINUTE MAN
BORING TYPE: 4 INCH FLIGHT AUGER
LOGGED BY: BM
START DATE: 3-8-05 FINISH DATE: 3-8-05

PROJECT NO: 1291-2B
PROJECT: TICONDEROGA DRIVE
LOCATION: SAN MATEO, CA
COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by TRC Lowney, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION: 524 FT. (+/-)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
524.0	0	▨	**LEAN CLAY WITH SAND (CL) [LANDSLIDE DEPOSIT]** stiff, moist to wet, brown, fine sand, some fine and coarse gravel, low plasticity	CL	8	⧗			
						☐	21	90	
	5				9	⧗	22		
517.0		⁘	**SANDSTONE [FRANCISCAN FORMATION (fsr)]** moderately to severely weathered, soft, dark brown, friable, some clay seams	fsr					
					50/6"	⧗	5	113	
	10								
					24	⊠	7		
	15								
					19	⊠			
			Plasticity Index = 13, Liquid Limit = 29		47	⊠	10		
504.0	20		Bottom of Boring at 20 feet						
	25								
	30								



GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 2/6/06 MV" FLL

TRC Lowney

EXPLORATORY BORING: EB-3

Sheet 1 of 1

DRILL RIG: MINUTE MAN
BORING TYPE: 4 INCH FLIGHT AUGER
LOGGED BY: BM
START DATE: 3-8-05 FINISH DATE: 3-8-05

PROJECT NO: 1291-2B
PROJECT: TICONDEROGA DRIVE
LOCATION: SAN MATEO, CA
COMPLETION DEPTH: 20.0 FT.

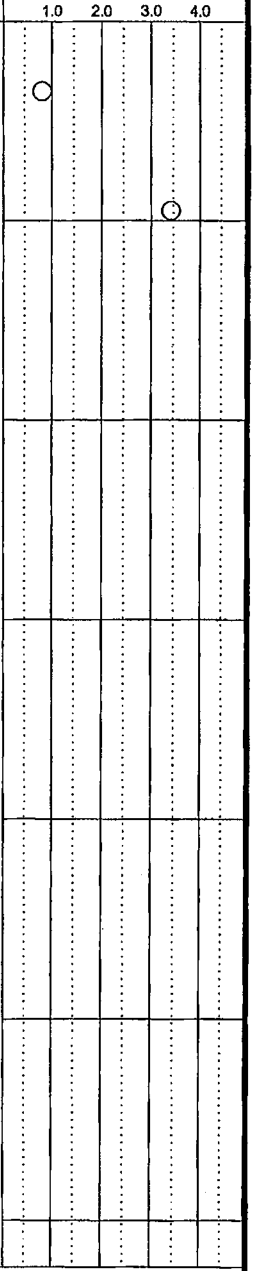
This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

SURFACE ELEVATION: 500 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
500.0	0	▨	**LEAN CLAY WITH SAND (CL) [LANDSLIDE DEPOSIT]** medium stiff, moist to wet, brown, fine sand, some fine and coarse gravel, trace organics, low plasticity	CL	13	⧗	20	100	
496.5	5	▨	**SANDY LEAN CLAY (CL) [COLLUVIUM]** very stiff, moist, gray, fine to coarse sand, some fine and coarse gravel, low plasticity	CL	24	⧗	13	113	
491.5		⁙	**SANDSTONE [FRANCISCAN FORMATION (fsr)]** moderate to severely weathered, soft, dark brown, friable, some fine sand	fsr	50/3"	⧗	6	124	
	10	⁙			49	⧖			
		⁙			59	⧖	8		
		⁙			32	⧖			
	15	⁙	increasing clay						
		⁙	abruptly severely weathered silty yellowish olive graywacke						
		⁙			55	⧖	3		
480.0	20		Bottom of Boring at 20 feet						
	25								
	30								

Undrained Shear Strength (ksf)

○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression



GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 2/2/06 MV* FLL

TRC Lowney

APPENDIX B

LABORATORY PROGRAM

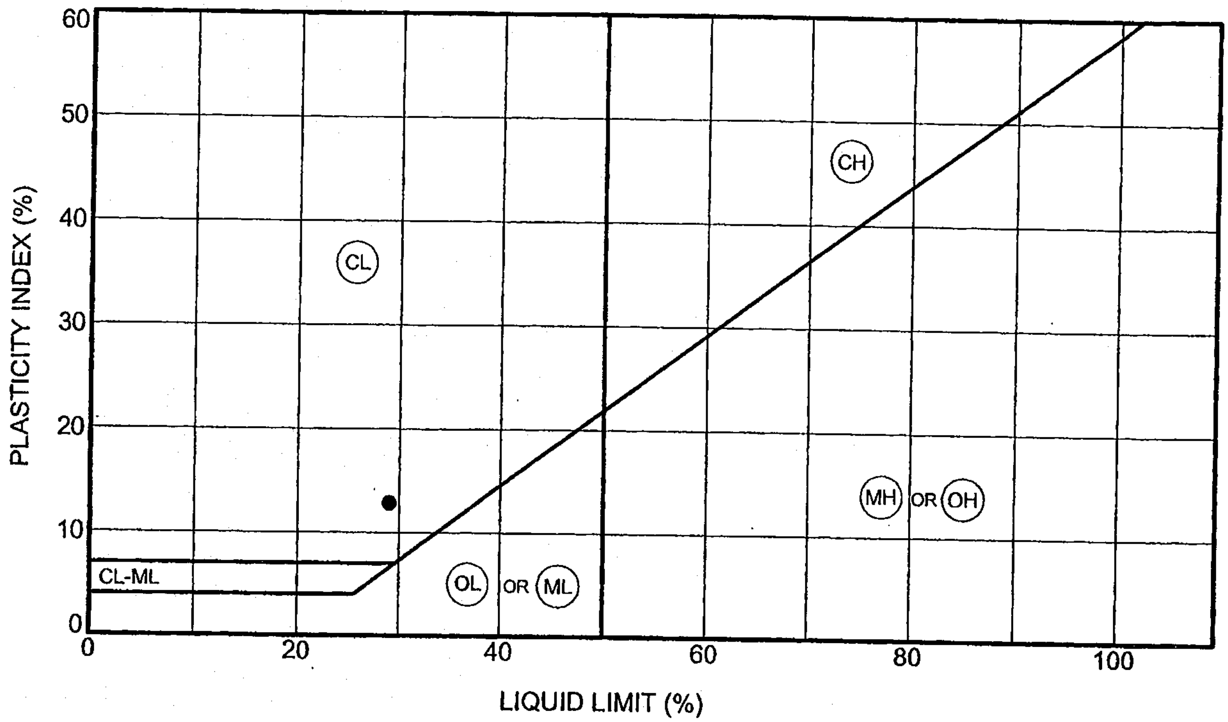
The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils and bedrock underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 15 soil and bedrock samples recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on nine samples to measure the unit weight of the subsurface soils and bedrock. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index (PI) test (ASTM D4318) was performed on a sample of the subsurface sheared bedrock to measure the range of water contents over which this material exhibits plasticity. The PI was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on the Plasticity Chart of this appendix and on the log of Boring EB-2 at the appropriate sample depth.

* * * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-2**	**19.5**	**10**	**29**	**16**	**13**		**Franciscan Formation [fsr]**

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PLASTICITY CHART AND DATA

Project: TICONDEROGA DRIVE
Location: SAN MATEO, CA
Project No.: 1291-2B

FIGURE B-1

LA_CORP.GDT 1/20/06 MV* FLL

APPENDIX C

PREVIOUS TEST PITS LOGS
BY BERLOGAR, LONG & ASSOCIATES (1980)

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-13	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-3½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'.
		Total depth 3½'; no free groundwater.
TP-14	0-4½	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4½-6	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	6-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 7'; no free groundwater.
TP-15	0-2½	Fill?: sandy clay, mottled dark brown and reddish-brown, slightly damp (W<PL), medium stiff to stiff, medium plasticity, layered structure (horizontal).
	2½-4	Soil: sandy clay, dark brown, damp (W<PL), firm to 3½', low plasticity; medium stiff below 3½'.
	4-5½	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	5½-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 7'; no free groundwater.

TEST PIT LOGS

Test Pit Number	Depth (ft.)	Description
TP-16	0-6	Fill: sandy clay as in TP-15; mottled light and dark brown, damp (W≤PL), and soft in top 2'; dark gray to dark brown, firm to very stiff, slightly damp (W<PL) below 2'.
	6-9	Soil: sandy clay, dark brown, damp (W<PL), firm to 7', low plasticity; medium stiff below 7'.
	9-10½	Subsoil: silty clay with minor sand, gray, damp to moist (W≥PL), medium stiff to stiff, high plasticity.
	10½-11½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 11½'; no free groundwater.
TP-17	0-2	Soil: silty clay with minor sand, gray, damp to moist (W<PL), medium stiff to stiff, high plasticity, soft in top foot.
	2-5½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Damp from 2 to 3½' (W<PL), slightly damp (W≤PL) below 3½'; large block of very fractured but hard greenstone at 5'.
		Total depth 5½'; no free groundwater.