APPENDIX E – GEOLOGY AND SOILS

D&M Consulting Engineers, Inc. Geological and Geotechnical Feasibility-Level. August 6, 2001.

GEOLOGICAL AND GEOTECHNICAL FEASIBILITY STUDY PROPOSED ENCINA HILLS SUBDIVISION MONTEREY COUNTY, CALIFORNIA

FOR: Harper Canyon LLC
c/o Whitson Engineers
2600 Garden Road, Suite 230
Monterey, California 93940

Attention: Mr. Ken Whitson

This document was prepared for use only by Harper Canyon LLC (owner/applicant for development) and their Civil Engineering Consultant, Whitson Engineers, only for the purposes stated, and within a reasonable time from its issuance. Please read the "Additional Services" and "Limitations" sections of this report.

August 6, 2001

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D&M CONSULTING ENGINEERS, INC.
Geotechnical/Environmental/Materials Testing
A URS CORPORATION COMPANY

August 6, 2001
File: 0001892

Harper Canyon LLC
c/o WHITSON ENGINEERS
2600 Garden Road, Suite 230
Monterey, California 93940

Attention: Mr. Ken Whitson

Subject: Geological and Geotechnical Feasibility Study for the Proposed Encina Hills Subdivision in Unincorporated Monterey County, California

Gentlemen:

D&M Consulting Engineers, Inc./Terratech, Inc. (D&M/Terratech) is pleased to submit a draft of our geotechnical and geological feasibility study report for the proposed Encina Hills Subdivision, formerly referred to as the Harper Canyon project, in Monterey County, California. The attached draft report provides a description of the research and site studies performed, and our conclusions and recommendations regarding the site conditions as they related to the proposed project.

In summary, it is our opinion that the site may be developed as presently proposed, provided that the recommendations presented in our report are followed. The majority of the site is covered by a mantle of medium dense to loose colluvial soils generally ranging from about 3 to 5 feet thick. These sandy soils are highly erodible and potentially unstable where they exist on slopes, and should be removed and replaced as engineered fill prior to filling or roadway construction. These soils will also need to be considered in development of the 17 custom home sites. Other issues that need to be addressed as they relate to the development of the project include two mapped landslides, existing erosion gullies, high erosion potential of the soils found on the site once disturbed, and landslide or debris flow potential of the weaker near surface soils. Methods to address and mitigate the issues highlighted herein are presented in our report.

Preliminary geotechnical recommendations are also presented which address development of the custom home sites. These include discussions of surface and subsurface water, weak surficial soils, potential impacts of the geologic and topographic regime and possible methods of mitigation where adverse or unfavorable conditions are present.

12 Thomas Owens Way | Monterey, California 93940 | 831 372 3716 Tel | 831 372 7481 Fax
3194 De La Cruz Blvd., Suite 19 | Santa Clara, California 95054 | 408 297 6969 Tel | 408 297 7716 Fax
1500 Graham Hill Road, Unit B | Santa Cruz, California 95060 | 831 458 4383 Tel | 831 458 4388 Fax

We appreciate the opportunity to provide our services to you on this project. This draft report is provided to you at this time for review, comment and discussion. Upon completion of your review we will be more than happy to discuss your comments and make revisions as appropriate.

Sincerely,

D&M CONSULTING ENGINEERS, INC.,/TERRATECH
A URS CORPORATION COMPANY

REGISTERED PROFESSIONAL ENGINEER
GREGORY J. RUF
No. 35389
Exp. 9-30-~~01~~03
CIVIL
STATE OF CALIFORNIA

8/8/01

Gregory J Ruf, P.E.
Principal Engineer

Christopher M. Palmer

REGISTERED GEOLOGIST
CHRISTOPHER M. PALMER
№ 1262
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

Christopher M. Palmer, R.G., C.E.G., H.G.
Senior Engineering Geologist

cc: Addressee (4)
Whitson Engineers (3)
Ms. Liz Farwell (1)

GEOLOGICAL AND GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ENCINA HILLS SUBDIVISION
MONTEREY COUNTY, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geological and geotechnical feasibility study of the Proposed Encina Hills Development site in unincorporated Monterey County, California. In addition to addressing the feasibility of developing the site, preliminary geotechnical recommendations are presented where mitigation of poor site conditions is needed. The project site is located southeast of Highway 68 and northeast of San Benancio Road. Access to the site is from Meyer Road by way of San Benancio Road as indicated on the Geologic Site Plan, Figure 2.

1.1 PROJECT DESCRIPTION

The proposed development will provide 17 custom home sites within about 164 acres of the 343.92-acre site. Lots will range in size from 5.1 to 34.3 acres. Approximately 180 acres of the parcel will remain as open space. Initial development of the project will include the construction of roadways and underground utilities. After this phase of the project is complete, lots will be sold for individual development as custom home sites. The proposed development is shown on the Geologic Site Map, Figure 2. The basis for the geologic site map is the topographic map of the property prepared by Whitson Engineers.

1.2 PURPOSE AND SCOPE OF WORK

The purpose of this study was to evaluate the geologic and geotechnical feasibility of developing the site, including the 17 single-family residential lots shown on the geologic site map as well as the roadways to allow for site access. This entailed evaluating the potential for regional and site-specific geologic conditions to adversely impact or preclude development of the site or selected areas of the property. In addition to evaluating the geologic conditions, we also considered the geotechnical aspects of the geologic deposits and the soils encountered in our subsurface studies with respect to future development. Our conclusions regarding the geologic and geotechnical aspects of site development and preliminary recommendations for use in future development of the site development planning process have been developed and are presented in this report.

Our investigation was performed in general agreement with the scope of work outlined in our proposal dated February 5, 2001, with direction from the project owner/applicant's agent and the project Civil Engineer, Whitson Engineers, as our study progressed. The California Division of Mines and Geology guidelines for geologic studies was considered in our performance of this study as required by the Monterey County Planning Department.

The scope of services provided during this study included the following:

1. Review of geologic and geotechnical maps, and other information in our files pertinent to the site and vicinity.
2. Review of four stereo sets of aerial photographs of the site area to view the geologic and geomorphic setting, and possible evidences of faulting or landsliding, and to evaluate the geologic units on the site.
3. Geologic reconnaissance and field mapping of the subject property by a Certified Engineering Geologist.
4. Site reconnaissance by Registered Professional Engineers.
5. Excavation of 23 test pits on the site with a backhoe to explore the geologic units and structural conditions on the site. The test pits were excavated to depths of 4 to 12 feet below existing ground surfaces and were logged by a D&M/Terratech geologist. Locations of the test pits are shown on the Geologic Site Plan (Figure 2). Test pit logs are included in the Appendices.
6. Drilling of 12 exploratory borings for geotechnical purposes using truck-mounted drill rigs to investigate subsurface conditions.
7. Soil samples were collected during the drilling of the exploratory borings at minimum depth intervals of 5 feet to allow for examination and logging. A D&M/Terratech geologist logged all of the borings. Locations of the soil borings are shown on the Geologic Site Plan (Figure 2). A Boring Log Legend and the logs of the borings are included in the Appendix.
8. Evaluation of the information collected, identification of any potential geologic constraints to the proposed development, and development of geologic recommendations for addressing any constraints identified.
9. Engineering analysis of the geologic data and subsurface data obtained from the borings and test pits.

10. Evaluation of the research and field data regarding possible geologic and seismic hazards affecting the site and the proposed project.

11. Preparation of this report addressing both the geotechnical and geologic aspects of future site development. The report includes our geologic map, test pit logs, boring logs and geologic cross-sections of the subject property.

Evaluation and recommendations for final design and construction, including mass grading, of subdivision lots and custom homes is beyond the scope of this report and will require further site studies. In addition, this study specifically excluded the assessment of environmental characteristics, particularly those involving hazardous substances at the site.

2.0 GEOLOGY

2.1 REGIONAL GEOLOGY

The Monterey Bay Area occurs along the edge of the Coast Range Geomorphic Province, and is comprised of a discontinuous series of northwest trending mountain ranges, ridges and intervening valleys characterized by intense, complex folding and faulting. The general geologic framework of the Monterey Bay Area is reported in regional studies prepared by the California Division of Mines and Geology (1959, 1977) and Clark and others (2000).

The region lies adjacent to the San Andreas Fault System, which has created predominantly northwest-southeast trending geologic structure and topographic features. The San Andreas Fault System constitutes the boundary between the Pacific and North American tectonic plates, and active faults are abundant in the region.

The property is situated near the northern end of the Sierra de Salinas Mountain Range. The crystalline basement rock, at depths of as much as 3,600 feet, consists of granitic rocks of the Salinian block and the older Sur Series metasedimentary rocks. The oldest on-site geologic unit exposed in the area is Plio-Pleistocene Continental Deposits that have been identified as the Paso Robles Formation (Clark and others, 2000). It is estimated that the maximum thickness of the Paso Robles Formation in the area is approximately 500 feet. The Paso Robles Formation underlies by the Santa Margarita Sandstone, a very fine to coarse-grained arkosic sandstone up to 1,300 feet thick (Clark and others, 2000). The Paso Robles Formation and the underlying Santa Margarita Formation constitute significant regional aquifers. Quaternary alluvium is mapped overlying the Paso Robles Formation in the major regional drainages (Dibblee, 1999).

Regional mapping by Clark and others (2000) shows the bedrock in the region is gently to moderately inclined, and is folded into a series of alternating synclines and anticlines with complex structural trends of both north-south and east-west geomorphic ridge expressions associated with variations of the bedding inclinations across the site.

2.2 SITE GEOLOGY

2.2.1 Bedrock Units

A recent geologic study of the Monterey and Seaside Quadrangles area has been published by Clark and others (2000), which also incorporates the work of several prior studies, including amongst others, Clark and others (1974, 1997), Dupre (1990), Greene and others (1973), and Rosenberg and Clark (1994). Based on the recent study of Clark and others (2000), four geologic units have been mapped at the site area. These include the surficial units which include: Colluvium, Qc; Alluvial Deposits, Qal; and questionable or possible Quaternary Landslide Deposits, Qls; and the underlying bedrock unit, the Paso Robles Formation/Continental Deposits Undivided, QTc. Descriptions of these units at the site area from Clark and others (2000) are presented below.

- Paso Robles Formation/Continental Deposits, QTc – A series of nonmarine, semiconsolidated, oxidized, poorly sorted, fine to coarse grained sand beds with pebble and cobble interbeds. These deposits have been correlated to the Paso Robles Formation in southern Salinas Valley, and stratigraphic relations suggest these deposits are Pleistocene and possibly Pliocene in part and thus younger than the type Paso Robles Formation.
- Colluvium, Qc (Holocene) - Unconsolidated, heterogeneous deposits of moderately to poorly sorted silt, sand, and gravel, which where explored on the subject site consisted mostly of sand and silty sand. These materials on the site have been derived from the Paso Robles Formation and generally ranged from a loose to very loose condition in surface exposures, to compact and medium dense where underlying relatively level valley areas. Where these materials are in a loose to very loose condition along slopes, they are subject to local sand runs and a slow downslope creep.
- Alluvial Deposits, Qal (Holocene) - Unconsolidated, heterogeneous deposits of moderately sorted silt and sand with discontinuous lenses of clay and silty clay, and locally gravel. These materials were generally in a compact, consolidated condition where explored in the generally level valley portions of the site.

* Landslide Deposits, Qls - Based on our aerial photo analysis, field reconnaissance of exposed units, field exploration pits, and a review of data obtained in prior studies at the site, we believe there is sufficient information to conclude that two slides may be present on-site. Also, the slope is mantled by colluvial soils that have been deposited from weathering and erosion of the Paso Robles Formation which underlie the slope.

2.2.2 Surficial Units

Residual soils on-site are derived from complete weathering of the underlying bedrock or other geologic material. Colluvial soils develop by the same processes but migrate downslope by creep, slopewash, and other gravity-induced processes. Our subsurface investigation revealed a soil mantle between 2 and 5 feet thick, derived from the underlying materials, overlying the Paso Robles Formation. The soil consists primarily of medium dense to dense clayey to silty sand. Granitic cobbles were encountered at a depth of about 3 feet in the Lot 1 test pit, and 1- to 2-foot diameter boulders of arkosic sandstone were found at a depth of about 1 to 2 feet in the Lot 11 test pit.

Two relatively recent small landslides with distinct head scarps were observed on lots 14 and 15. Two apparent older large landslides are mapped on lots 11 and 13, adjacent to the eastern property line (see Figure 2, Geologic Site Plan). They were mapped based upon aerial photo interpretation and a surface reconnaissance. Subsurface exploration of these features was not undertaken at this time, as available equipment could not access the two lots due to the saturated condition of the near-surface soils in March 2001 when the initial field study by D&M was performed. In addition, the dense brush and trees also preclude access to these areas. If future subsurface exploration is to be undertaken, a dozer cut road will likely be required to gain site access.

Hummocky topography was observed on Lots 8 and 9, but no scarp was evident in aerial photos. The hummocky topography combined with shallow subsurface water found in the colluvium overlying the Paso Robles Formation in the test pits on Lots 8 and 9 suggest that the slopes prone to creep. The hummocky topography may also be indicative of smaller surficial slides, similar to those observed on Lots 14 and 15, with the effects of erosion now obscuring the scrap and toe areas. Several erosion gullies and scarps were observed within the site boundaries on the aerial photos and during our reconnaissance of the site. These are depicted on Figure 2.

3.0 FAULTING AND SEISMICITY

3.1 REGIONAL AND LOCAL FAULTING

An overview of the fault setting of Central California and the Monterey Bay area is presented in several regional studies including Wallace (1990), Jennings (1994), Rosenberg and Clark (1994), and Clark and others (1997). The regional faults setting is shown on Figure 4, from Jennings (1994), while the local fault setting and Area Geology Figure 4, taken from Clark and others (2000).

This site is located in the seismically active San Francisco-Monterey Bay region but outside the earthquake fault zones established in accordance with the State of California Earthquake Fault Zones (Hart and Bryant, 1997) established by the Alquist-Priolo Earthquake Fault Zoning Act of 1972.

Small areas along the southern end of the site, including portions of Lot 17 and the remainder parcel, are located in County of Monterey Geotechnical Hazard Zone VI, the zone with the greatest prevalence of seismic hazards (Burkland & Associates, 1974). The remainder of the site is located in Hazard Zone IV.

Faults listed on maps produced by California Department of Conservation, Division of Mines and Geology (CDMG, 1998) and published by the International Conference of Building Officials have been classified as Type A and Type B faults for the purpose of evaluating potential seismic impacts associated with these faults. A review of the ICBO published maps indicates that the site is about 33 km southwest of the San Andreas (Pajaro) Fault, the closest Type A fault. The maximum expected magnitude of an earthquake for this segment of the San Andreas Fault is 7.9 (CDMG, 1998). The closest Type B faults are the Rinconada Fault located 4.5 km northeast of the site and the Monterey Bay-Tularcitos Fault located 10.2 km southwest of the site. The maximum expected magnitude (M_W = moment magnitude) of an earthquake generated by the Rinconada Fault is 7.3 (CDMG, 1998).

In addition to the fault zones mapped by CDMG, fault mapping by Dupre (1990) indicates the presence of other local faults. The active fault closest to the site is the Chupines Fault, located about 4 km southwest of the site. Vaughan and others (1991) calculated a "maximum slip rate of about 2 mm per year over the last 12,000 to 13,000 years" based on evidence from a trench about 4 km from the site (Clark and others, 1994). Rosenburg and Clark, (1994) report that epicenter data plotted within 1 km of the mapped fault trace suggests that the western part of the Chupines Fault zone is active. A maximum expected magnitude was not available for the Chupines Fault.

The Harper Fault is mapped trending northwest southeast about 2,000 feet east of the site (Dibblee, 1999; Burkland, 1974). Burkland (1974) also shows the Harper Canyon Fault trending northwest-southeast along San Benancio Canyon Road, about 1,300 feet southwest of the site. Burkland considered both the Harper and Harper Canyon Faults to be potentially active, as they offset Pleistocene strata. However, Clark and others (2000) note that there is no evidence of Holocene activity on the Harper Fault; these authors do not discuss the Harper Canyon Fault activity.

Existing geologic maps of the site and surrounding area do not show any faults on the property. Our review of aerial photographs and our geologic mapping did not reveal any photolineaments or evidence of faulting on the site.

Active and potentially active faults that may result in significant ground shaking at the site are listed on Table 1, which includes regional faults such as the San Andreas, and others, as well as local faults. The locations of the faults and associated parameters presented on Table 1 are based on data presented by Hart and others (1984), Wesnousky (1986), Wong and others (1988), Working Group on California Earthquake Probabilities (1990), Schwartz (1994), Jennings (1994), Mualchin (1995), Frankel and others (1996), Petersen and others (1996), and Clark and others (1997). The information on faults contained in Table 1 are based on the fault's distance from the site, fault length, slip rate, and maximum earthquake moment magnitude determined from the program EQFAULT version 3.0 (Thomas Blake and California Division of Mines and Geology, 1998). This was supplemented with a search with epicenter data from 1994-1999 from the University of California at Berkeley, Northern California Earthquake Data Center. The approximate site center coordinates, taken from the 7.5 minute Spreckels, California Quadrangle Map (USGS, 1947; rev. 1968; 1975), are:

Latitude: 36.5750 North   Longitude: 121.6980 West

Table 1
Significant Faults

Fault Name	Fault Length (km)	Closest Distance to Site (km)	Magnitude of Maximum Earthquake	Slip Rate (mm/yr)
Rinconada	105	4.5	7.3	1.0
Monterey Bay	99	10.4	7.1	0.5
Palo Colorado (Sur)	80	26.5	7.0	3
Zayante-Vergeles	56	27.6	6.8	0.1
San Andreas (creeping)	470	32.3	6.5	24
San Andreas (Pajaro)	470	32.5	6.8	24
San Andreas (1906)	470	32.5	7.9	24
Calaveras (so. of Calaveras Res.)	100	38.6	6.2	15
Sargent	53	13.6	6.5	3
San Andreas (Santa Cruz Mtn.)	470	42.2	7.0	24
Quien Sabe	14	47.0	6.4	1
San Gregorio	129	47.2	7.3	5
Hosgri	172	47.6	7.3	2.5
Monte Vista-Shannon	41	68.1	6.8	0.4
Ortigalita	83	71.3	6.9	1
San Andreas (peninsula)	470	72.3	7.1	24
Great Valley (segment 9)	39	75.5	6.6	1.5
Great Valley (segment 8)	41	76.5	6.6	1.5
Hayward SE Extension	26	77.9	6.4	3
Great Valley 10	22	78.0	6.4	1.5
Great Valley 11	25	86.9	6.4	1.5
Greenville-Marsh Creek	56	95.7	6.9	2
Calaveras (northern)	52	97.3	6.8	6
Hayward South	26	77.9	6.4	3
Hayward (Total Length)	80	23.6	7.1	9
Great Valley (segment 7)	45	53.7	6.7	1.5
Great Valley 12	17	104.9	6.3	1.5
Great Valley 13	30	115.8	6.5	1.5
San Andreas (Parkfield)	470	119.9	6.7	24
San Andreas (1857 Rupture)	470	119.9	7.8	24
Great Valley (segment 6)	45	122.5	6.7	1.5
Hayward (north)	26	134.5	6.4	3
Great Valley 14	24	142.9	6.4	1.5
Concord-Green Valley	26	148.1	6.9	6
San Andreas (Cholame)	470	155.5	7.8	24
San Andreas (north coast)	470	159.4	7.1	24
Los Osos	44	159.8	6.8	0.5
San Juan	68	160	7.0	1

3.2 SEISMICITY

3.2.1 Historical Seismicity

Monterey is in a region of Central California, which is traditionally characterized by high seismic activity. Some of the significant nearby historic earthquakes include the 1906 (M8+) San Francisco earthquake, the 1838 (M7) San Francisco earthquake, the 1989 (M6.9) Loma Prieta earthquake, the 1836 (M6.8) Hayward earthquake, the 1868 (M7) Hayward earthquake, the 1858 (M6.3) San Jose earthquake, the 1864 (M6) South Santa Cruz Mountains earthquake, the 1865 (M6.5) South Santa Cruz Mountains earthquake, the 1870 (M6) Los Gatos earthquake, the 1884 (M6) Santa Cruz Mountains earthquake, the 1897 (M6.3) Gilroy earthquake, the 1911 (M6.5) Calaveras fault earthquake, the two 1926 (M6.1) Monterey Bay earthquakes, and the 1984 (M6.1) Morgan Hill earthquake.

Studies by McCrory and others (1977) indicate that during the 1906 San Francisco Earthquake on the San Andreas Fault that the site occurs in an area that generally experienced Rossi-Forel Intensity values of VII to VIII. They also report that Modified Mercalli Intensity values (MM) of between VI and VIII occurred in the site area over a dozen times during a 159 year period.. Youd and Hoose (1978) indicate a report of 1906 earthquake shaking damage to tracks along the coast the between Seaside and Del Monte, although no damages are described for the immediate site area.

The Loma Prieta Earthquake (October 17, 1989, magnitude 7.1) occurred along the San Andreas Fault with the epicentral area about 40 kilometers north of the site. Plafker and Galloway (1989) and Stover (1990) indicate that the site rests in the general vicinity of Modified Mercalli Scale earthquake shaking intensity distributions of 6. The magnitude of ground motion measured instrumentally shows that the earthquake is reported to have triggered over 130 strong motion instruments operated by the U.S. Geological Survey and the California Division of Mines and Geology (EERI, 1989, Benuska, 1990). An instrumentation station in the Monterey area showed an acceleration value of 0.07 (EERI, 1989, Shakal and others, 1989).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km of the site), dates (1800 through February 2000), and magnitudes (M>4). A summary of the results of the historical search is presented below.

(1) Time Period (1800 to February 2000)	201 years
(2) Maximum Magnitude	M8.3
(3) Approximate distance to nearest historical M>4 earthquake	2.6 km
(4) Number of events exceeding magnitude 4 within search area	763

3.2.2 Design Level Earthquake

We have developed peak ground accelerations for Upper Bound Earthquake (UBE) and the Design Basis Earthquake (DBE). As defined in the 1997 UBC, the UBE is defined as the ground motion that has a 10 percent probability of being exceeded in 100 years (return period of about 950 years). The DBE is defined in the 1997 UBC as the ground motion that has a 10 percent probability of being exceeded in 50 years (return period of about 475 years).

A probabilistic seismic hazard analysis was used to estimate the peak ground accelerations for the UBE and DBE, as discussed above. This analysis involves the selection of an appropriate predictive relationship to estimate the ground motion parameters, and, through probabilistic methods, estimate of peak accelerations. The results of these analyses are presented in Section 4.2.1 of this report.

3.2.3 Attenuation Relationship

The types of faulting, magnitudes of the earthquakes, and the local soil conditions can influence site-specific ground motions. The attenuation relationships used to estimate ground motion from an earthquake source at some distance from the site need to consider these effects.

Many attenuation relationships have been developed to estimate the variation of peak ground surface acceleration with earthquake magnitude and distance from the site to the source of an earthquake. We have used relationships presented by Boore et al. (1993, 1994 and 1997), and Abramson and Silva (1997) because of their wide acceptance by seismologists. These relationships have also been used in developing recent Interim National Seismic Hazard Maps (Frankel et al., 1996) for the State of California. These relationships use an estimate of shear wave velocity of the soil profile in the analysis. An average shear wave velocity of material 280 m/s was selected. The predictive relationships by Boore et al. (1993, 1994, 1997) were developed from statistical analyses of recorded earthquakes from Western North America, including the records from the 1989 Loma Prieta earthquake and 1992 Landers earthquake. The attenuation relationships provide mean values of ground motions associated with one set of parameters: magnitude, distance, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis.

4.0 SITE INVESTIGATION

4.1 SITE DESCRIPTION

The subject property consists of an approximately 343.92-acre parcel located northeast of the terminus of Meyer Road, within the 7.5-minute Spreckels, California Quadrangle. Current and historic site usage is for cattle grazing. Surrounding land uses include public lands (Toro Park) and rural residential.

Past site development appears to have been limited to the grading of several unpaved, narrow roads and trails. Construction of these roadways across the slopes entailed cutting into the sides slopes and placing (likely by side casting) fill on the downslope side of the roadway. Fill slopes were observed to be as steep as 1.5:1. Fills were also noted to be obstructing natural drainages in a few locations.

The property consists of a series of rounded hills and ridges with intervening drainages, with approximately 700 feet of elevation change within the bounds of the site. Elevations on the parcel range from about 1,020 feet near the southeast corner to about 330 feet near the northeast end. The terrain is highly varied with natural slope gradients range from about 6:1 (horizontal:vertical) in the southern portion of the property to about 1.9:1. Steeper slopes are found within eroded drainages in several areas of the site as depicted on Figure 2. Slope inclinations of 1.5:1 to vertical, with some undercutting observed, are common in the erosional features. The Paso Robles Formation bedrock exposed in these scarps commonly exhibits vertical erosion rills.

Two large landslide features are mapped along the west face of the ridge along the eastern boundary of the site at Lots 11 and 13. In addition, two smaller slides are mapped at Lots 14 and 15, with hummocky terrain observed on the slope at Lots 8 and 9.

Although the regional tectonics and geomorphology of the region trend in a northwest-southeasterly direction, ridges on the site vary indirection from the trend. A north-south trending ridge is located above Lots 2, 11 and 13-15. This ridge intersects an east-west trending ridge located to the south of Lots 15-17. Erosion, and possibly regional folding, appears to have resulted in the formation of several smaller ridges on the site that also trend east-west.

Lots 1-6 of the proposed subdivision are located within the northern-most section of the site. The proposed house sites within these lots are within a small valley, with Lots 1 and 3-6 located on the eastern flank of a ridge. Lot 2 is located in the central area of this valley. The building site at Lot 1 is relatively level and is at the base of a narrow ridge with 2.5:1 and 1.5:1 side slopes to the west and east, respectively. These slopes are in excess of 100 feet high. The ridge line to the south of the pad extends up about 170 feet.

The pad site at Lot 2 is in an area above a well–defined active erosion gully and below a slope that extends up behind the lot about 240 feet. The slope has an inclination of about 2.5:1 in the upper 200 feet, flattening to about 3.3:1, with the pad area at about 5:1.

The designated area for the future houses at Lots 3-6 are below a ridge with slopes of about 2.8:1 to 2.5:1 with slope heights ranging from 80 to about 140 feet above the envelopes. The slope flattens to about 5:1 at the building areas.

Lot 7 is at the western side of the property, on the western flank of the ridge at Lots 1 and 3 through 6, and is west northwest of Lot 6. The selected building area has a surface gradient of about 5:1, with a 3.8:1 slope extending about 100 feet up to the east and a descending 150 foot slope at about 5.7:1 to west.

Lots 8-9 are located south of Lots 6 and 7 on a south-facing slope. Lot 10 is south of Lots 8 and 9 in the flatter areas below the slope and above a prominent erosion gully. This area was observed to be saturated in March 2001, indicating the presence of shallow (perched) groundwater. The slope at Lots 8 and 9 has a gradient of about 2.4:1 with a height of about 120 feet. The area was noted to be distinctively hummocky. The designated building areas are below the break in slope and slope at about 5:1. The area below Lot 8 flattens and is the selected location for the building at Lot 10. A vertical-walled erosion gully is present at the southern side of the Lot 10.

Lot 11 is located at the eastern side of the portion of the proposed Encina Hills development, on the western flank of a major ridge. The lot encompasses one of the two larger landslide areas mapped on the site. The proposed home site is just below a break in gradient, with a 2.3:1 to 2.5:1 slope to the east rising above the site 90 and 130 feet. The slope at the home site is about 4:1.

The Lot 12 home site has been placed at the site of a small, more resistant ridge. The area is relatively flat with a gradient of about 6.5:1 or flatter. The slopes that essentially wrap around the site on three sites have a gradient of about 4:1. These slopes appear to be related to erosion of less resistant materials than those at the selected site.

Lot 13 encompasses the second of the two larger landslides, as shown on Figure 2. The general topography at Lot 13 is also found at Lots 14-16, located to the south of Lot 13. The slopes above these four lots extend up from the proposed home sites about 130 feet. The upper 100 feet or so of the slopes have a gradient of about 2:1, flattening slightly to 2.3:1 in some areas. These is an apparent break in slope below this, with slopes at about 3:1 for a vertical height of about 30 feet; additional flattening of the slopes to about 4.5:1 to 5:1 occurs at the home sites.

The majority of Lot 17 lies on the north-facing slope of a west-trending ridge at the southern portion of the proposed development. The balance of the site wraps around the end of the ridge. The proposed home site is located at the western end of the ridge at the ridge top. The north face of the ridge slopes down at about 1.9:1 for a vertical distance of over 100 feet, flattens slightly and continues on down of over 200 feet. The western end of the ridge generally slopes down at about 2.5:1 for about 100 feet. A swale on the eastern flank of the site slopes down at about 4:1 and is representative of the proposed home site.

Surface runoff on the property is by sheet flow to the northwest along erosion gullies toward El Toro Creek. No flowing water was observed in drainages on the property during our surface reconnaissance and subsequent field investigation. Seven minor springs were found, however, two along the unpaved road near the boundary of Lot 17, two on Lot 15, one on Lot 13, and two on Lot 11. These last three springs were located along the existing unpaved road, at the base of apparent landslide deposits. One of the springs on Lot 15 is associated with a small, shallow landslide.

Vegetation observed consisted of grasses, bush and trees. The bushes and trees are often concentrated in the drainages, apparently due to moist conditions from seasonal surface water flow. Dense brush and mature trees are also prevalent on the steeper portions of the slopes above Lots 11 and 13-16.

4.2 FIELD INVESTIGATION

D&M/Terratech performed 23 exploratory test pits across the site. The test pits were excavated to depths ranging from about 3 to 16 feet below existing ground surface using a rubber-tired backhoe. Materials encountered in each boring were visually classified in the field by our representatives and a continuous log of each excavation was made. Visual classifications of soils encountered were made in accordance with the Unified Soil Classification System as shown on the Boring Log Legend presented in Appendices. Classifications of bedrock encountered also included geologic description where applicable. The test pit logs are also presented in the Appendices.

Twelve geotechnical exploratory test borings were also drilled at the site. The test borings were made to depths ranging from about 15 to 50-1/2 feet below existing ground surface using a Mobil B-24 drill rig equipped with 4-inch diameter continuous flight hollow-stem augers and a CME 45 drill rig equipped with 8-inch diameter hollow-stem augers. These borings ranged in depth from about 10 feet to about 50-1/2 feet below the existing ground surfaces. Relatively undisturbed soil samples were obtained at the boring locations by driving a 2-inch inside diameter tube sampler to a depth of 18 inches into the underlying soil using a 140-pound hammer falling 30 inches. Similarly, disturbed soil samples were also obtained with a 2-inch outside diameter Standard Penetration Test (SPT) sampler that was driven into the granular soils with a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers were recorded for each 6-inch penetration interval. The number of blows required to drive the samplers the last 12 inches (unless otherwise noted) are included on the boring logs. Visual classifications of soils encountered and a continuous log of each geotechnical boring were made in accordance with the Unified Soil Classification System as shown on the Boring Log Legend presented in the Appendices.

In addition to drilling borings for geotechnical purposes, two additional borings were excavated at each site where percolation tests were performed. This is discussed in more detail below.

The locations of the points of explorations were estimated in the field based on rough alignment with the existing site features. The locations of the borings and test pits should be considered accurate only to the degree implied by the locating method used.

4.3 SUBSURFACE CONDITIONS

The Paso Robles Formation underlies the property as mapped in road cuts along the access roads, in outcrops across the property, in erosion gullies and scarps, and where encountered in our test pits and exploratory borings. Clark and others (2000) describes the Paso Robles Formation as semiconsolidated, poorly sorted fine- to coarse-grained sands with interbedded pebble and cobble gravels. Dibblee (1999) shows the Paso Robles Formation horizontal or dipping between 3 and 15 degrees to the west in the vicinity of the site. The boring data indicates that the underlying Paso Robles deposits are very dense. Duripan horizons, or cemented soil layers, within these deposits commonly form prominent ledges and may prevent shallow water infiltration, resulting in debris flows. This unit, as observed during this investigation, consists of dense to very dense silty sand, gravelly sand, and clayey sand. Weakly to moderately cemented layers and varies from about 2 inches to about 4 feet thick were encountered locally.

The materials encountered in the test pits and the borings, as well as those mapped during our site reconnaissance support the previous mapping by Clark, Dibblee and others. Data obtained form the test pits reveals the presence of colluvial deposits over the site. These deposits are comprised of loose to medium dense sands, silts and gravels derived form the Paso Robles Formation.

No ponds were visible on the property or in any of the aerial photographs we reviewed. Seasonal perched groundwater conditions can form locally in the colluvium. During the initial stages of our site investigation between March 12 and 22, 2001, perched water was found at depths of about 1 to 2 feet in borings B-2 and B-5, and in test pits on Lots 2, 8, 9, 13 and 16. Areas with seasonal perched groundwater conditions noted in March 2001 are shown on Figure 2.

Groundwater (within a true aquifer) was not encountered in the Paso Robles Formation in any of the relatively shallow test pits or borings, (within a depth of about 50 feet below ground surface) at the time of our subsurface investigation in March and May, 2001. Dibblee (1999) shows a 500-foot-deep water well occurring at an elevation of about 600 feet on a neighboring property.

5.0 CONCLUSIONS - GEOLOGIC HAZARDS

SEISMIC RELATED GEOLOGIC HAZARDS

A discussion of specific seismic related geologic hazards that could impact the site is included below. The hazards considered include: Fault Ground Rupture; Seismic Shaking; Liquefaction, Lateral Spreading and Differential Compaction/Seismic Settlement; Seismically Induced Landslides and Ground Failures.

No seismic hazard zone maps have been published yet for Monterey County under the State's program of mapping areas potentially susceptible to seismically induced landsliding and liquefaction.

5.1 SURFACE FAULT RUPTURE

A compilation of data on historic seismically induced ground failures in northern California (Youd and Hoose, 1978) shows no recorded ground failure in the vicinity of the site, but the area was sparsely populated at the time of the 1906 "San Francisco" earthquake, from which most of the data in the publication were derived.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no known active faults are believed to traverse the site. Based on our literature review, site reconnaissance and aerial photo-analysis we did not identify any tonal lineations, geomorphic features or other features which could be suggestive of faulting, active or potentially active, crossing the site or the immediately adjacent area. Therefore, it is our opinion that the potential for fault-related surface rupture on the site is very low.

5.2 SEISMIC SHAKING

5.2.1 Estimated Peak Ground Acceleration

The estimated peak horizontal ground acceleration (in units of gravity), calculated using the method discussed above for the UBE and DBE are presented in Table 2. The corresponding return period and annual probability of occurrence are presented in Table 2.

Table 2
Estimated Peak Ground Acceleration

Event	Return Period	Probability of Occurrence	Annual Probability of Exceedance	Peak Horizontal Acceleration (g)
DBE	475	10% in 50 years	0.0022	0.55[(1)]
UBE	950	10% in 100 years	0.0011	0.68[(2)]

(1) EQFAULT
(2) U.S. Geological Survey, 2001, Earthquake Hazards Program, Seismic Hazard Mapping Project, Probabilistic Hazard Lookup by Latitude Longitude

Probabilistic modeling procedure was used to estimate the peak ground motion corresponding to the UBE and DBE. The probabilistic analysis approach is based on the characteristics of the earthquake and of the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake, distance from the site to the causative fault, length and activity of the fault. The effects of site soil conditions and mechanism of faulting are accounted for in the attenuation relationships.

5.2.2 Site Soil Profile

The characteristics of the soils and sediments underlying the site are important site-specific seismic design criteria to evaluate the site response. Site soil observations are based upon the geologic exposures in cuts and erosional features on the site, the materials encountered in our test pits. The observations also include materials encountered in our borings, which penetrated the site to a maximum depth of about 50-1/2 feet. A relatively thin mantle of colluvial deposits overlying bedrock of the sedimentary Paso Robles Formation generally underlies the site. The Paso Robles is comprised of semi-consolidated, locally weakly cemented, dense to dense clayey sand, silty sand, sand and gravel.

In our opinion, based on the materials encountered in our subsurface exploration of the site and our knowledge of the subsurface geology in the vicinity, we classify the upper 100 feet of soil profile as type S_C. Soil classified as S_C per Table 16-J of the 1997 UBC, S_C is defined as a soil profile consisting of very dense soil with shear wave velocity between 360 and 760 meters per second (m/s) or SPT-N = >50, or Su = >2000 psf for the upper 100 feet or 30 meters.

5.3 NEAR-FAULT CONSIDERATIONS IN STRUCTURAL DESIGN

Due to potential near-fault motion resulting from activity on local and regional faults, near-source effects should be considered in the structural design of the proposed project. For a code equivalent lateral force design, procedures from the 1997 UBC will need to be considered. The seismic design parameters that follow are as defined by the 1997 UBC and are determined based on the assigned Seismic Hazard Zone, proximity of the site to Type A and Type B faults and the soil profile type. Type A fault zones within 15 kilometers and Type B fault zones within 10 km are to be considered for near-source effects.

For this site, the Rinconada Fault (distance of approximately 4.5 kilometers NE) is defined as the closest significant Type B fault as defined by CDMG. With consideration of the type and proximity of the Rinconada Fault, and the location of the site in Seismic Hazard Zone 4, the Near-Source Factors N_a and N_v can be obtained from Tables 16-S and 16-T, respectively, of 1997 UBC. The Near-Source Factors in the Code are incorporated into the seismic coefficients C_a and C_v which are both used to determine the total design lateral force or shear at the base of the building or structure. Near-Source Factor N_a and N_v are 1.05 and 1.27, respectively, for this governing fault. The seismic coefficients C_a and C_v are equal to $0.4N_a$ and $0.56N_v$, respectively.

5.4 SEISMICALLY INDUCED GROUND FAILURE

5.4.1 Liquefaction and Lateral Spreading

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. As a result of the loss of strength, the soils gain mobility that can result in significant deformation, including both horizontal and vertical movement where the liquefied soil is not confined. Factors affecting the potential for a soil to liquefy include: 1) intensity and duration of earthquake shaking; 2) soil type and relative density; 3) presence of a confining layer allowing for build-up of excessive pore pressure, 4) overburden pressure; and 5) presence or absence of groundwater. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, Holocene age, fine grained sand deposits. Silts and silty sands have also been proven to be susceptible to liquefaction or partial liquefaction. The occurrence of liquefaction is generally limited to soils located within about 50 feet of the ground surface. Loss of bearing capacity and/or ground settlement can result as a result of liquefaction.

As previously discussed, the near-surface soils blanketing most of the slopes, in the less inclined portions of the site below the slopes and within erosional features, consists of medium dense to loose colluvial deposits of silts, sands and gravel. These deposits are derived from and overly the Paso Robles Formation. Our observation of these deposits indicates that the soils are relatively weak when saturated and that perched groundwater conditions are common on the site. The perched groundwater occurs at the contact between the more permeable colluvium and the dense, often cemented, Paso Robles. Regional mapping by Dupre (1990) shows the site area is classified as very low liquefaction potential.

Based on the observed conditions, it is our opinion that during the time of year when perched groundwater conditions are present, should strong shaking occur, that the potential for liquefaction or at least partial liquefaction to occur is low to moderate. Where the loose sand layer is present at or very near the ground surface and is free-draining, it tends to preclude the development of excess pore pressure, the potential is judge to be low. The dense, weakly indurated Paso Robles Formation that underlies the property is judged not to be susceptible to liquefaction.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of liquefied subsurface materials beneath a slope, or even beneath level ground if an open topographic face is nearby. With the potential for liquefaction judged to be low to moderate, there is a corresponding low to moderate potential for seismic induced lateral spreading to occur on this site.

5.4.2 Dynamic Compaction and Seismic Settlement

Another type of seismically induced ground failure that can occur as a result of seismic shaking is dynamic compaction or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. The potential impact of dynamic compaction is settlement of the ground surface. The loose to medium dense colluvial soils have a low to moderate potential to undergo some settlement where building loads are applied or fills are constructed and strong ground shaking occurs.. Mitigation measures are discussed later in this report. Based on the high apparent density of the Paso Robles Formation, it is our opinion that this material has a very low potential to undergo seismic induced settlement

5.4.3 Landslides and Seismically Induced Slope Failures

While there are two large landslides mapped on the site, one of which is queried, the Paso Robles materials when intact are not considered to be landslide prone. These materials are dense to very dense and are often locally cemented. Although we cannot completely rule out past landslide activities at the sites of the two larger mapped slides, there is also a possibility that these features are associated with or are now more pronounced as a result of erosion of site. The "headscarp" areas of the two mapped slides, as well as the slopes above Lots 14-17, are presently at inclinations of about 2:1. Based on the apparent relative density of the granular deposits (bedrock), these soils have a friction angle at peak strength on the order of 40 to 48 degree with a corresponding slope angle of repose of 1.6:1 to 1.4:1 (horizontal:vertical). Based on these values it is our opinion that the slopes are stable under static conditions.

As discussed below, the Paso Robles is highly susceptible to erosion, resulting in relatively loose colluvial deposit below the slopes. These materials, when founded on sloping bedrock surfaces, are prone to surficial sliding where seasonal perched groundwater conditions developed and, where there is a sufficient quantity of fines occur within the soil matrix to preclude rapid drainage. Where these types of slides occur in more granular deposits, the mobilized soils tend to drop out or deposit rather rapidly as the excess pore pressure within the soil mass dissipates. The slides tend to act as a shallow flow as can be seen at the two smaller, relatively recent slides mapped by our geologists at Lots 14 and 15. This process also appears to have occurred which lead-to the long term development of the hummocky terrain present on the slope at Lots 8 and 9, and to a lesser extent at Lot 17. As noted in the geology section of this report, the Paso Robles is mapped to have a west-trending dip-slope bedding that can result in soil creep or surficial landsliding. This is consistent with our observations of the slopes at Lots 11 and 13-16.

Mitigation of the surficial slide potential and for the potential impacts of ongoing sloughage or erosion of the faces of the two larger slides are discussed in subsequent sections of this report.

When considering slope stability with seismic shaking, there is a potential for sloughing of the face of the steeper slopes resulting in deposition of loose materials at the base of the slopes. This would increase the potential for slides to occur below the slopes within the Paso Robles Formation. The potential for randomly oriented ground cracking affecting the site and surrounding areas due to strong seismic shaking cannot be precluded.

5.5 EROSION

With the exception of the local cemented zones in the Paso Robles, the subsurface materials on the site lack cohesion and are very susceptible to erosion. However, where left undisturbed, even where vegetation appears to be relatively sparse, the materials appear to be somewhat resistant to erosion. There is evidence of some downslope creep where the surface is not eroded. However, once the surface is disturbed the potential for, and the rate of erosion, appear to increase significantly. Surface flows cut the materials deeply with incised erosion gullies often having near-vertical sidewalls. Where resistant or cemented layers are present and erosion gullies are being formed, undercutting of the sidewalls occurs thus increasing the lateral extent of the area affected by the erosional process. Erosion in the cemented soils also results in vertical rills ("badlands" topography) as can be seen along the access road south of the property. Mitigation of the effects of on-going and future erosion at the site will need to be addressed through development of controlled surface drainage plans and/or avoidance of erosion affected areas in site planning.

5.6 WEAK OR EXPANSIVE SOILS

Expansive soils were not encountered on the site and are not a consideration in future development of the site. As described throughout this report, the near-surface granular colluvial deposits are generally medium dense to loose. As such, these soils are found to be weak or compressible. This was clearly demonstrated by our inability to access several areas of the site during the month of March 2001 when the soils were wet. Mitigation of this condition will be required for areas to be mass graded and where road and houses are to be constructed.

5.7 TSUNAMI, SEICHE AND FLOODING

The site is located several miles from the shore of Monterey Bay, is a minimum of 330 feet above sea level, and is not located adjacent to or downslope of any lakes, creeks or water storage facilities. Consequently, the potential for Tsunamis, Seiche or seismic induced flooding due to storage facility failure is considered very low.

6.0 PRIMARY CONSIDERATIONS AND MITIGATION RECOMMENDATIONS

6.1 GENERAL

Based upon the data collected during this investigation, and from our geologic and geotechnical engineering analysis, it is our opinion that the site may be developed as discussed in this report provided that the recommendations presented in this report are incorporated into the final design and construction of the project. These opinions, conclusions, and recommendations are based on our field and office studies, and the properties of the materials encountered in our borings and test pits.

The primary geological and geotechnical considerations for design and construction of the project include the following: 1) landslides previously identified at the site; 2) seismic hazards (liquefaction, lateral spreading, lurching; seismic induced settlement), 3) control of surface erosion; 4) weak surface soils; 5) existing uncontrolled fills; and 6) stability of erosion ravine walls. Each of these items is discussed in detail below with recommendations for mitigation presented including grading activities and setbacks.

We note that the current study provides preliminary geotechnical recommendations for earthwork associated with the proposed roadways and infrastructure improvements, as well as future development of the home sites. We anticipate that work beyond the scope of this current study includes geotechnical evaluation and recommendations for overall site mass grading, site features, foundations for individual development sites, and associated elements such as retaining wall, driveways and pools will be required. Additional work that is recommended and is beyond the scope of our current services includes continued consultation with the Civil Engineer as project plans are developed, review of the completed site earthwork and grading plans, and specifications once they become available.

We recommend that D&M/Terratech be retained to provide observation and testing services during site earthwork associated with this report and any subsequent supplemental studies which include mass grading, construction of site infrastructure including but not limited to the on-site roads and utilities. This will allow us the opportunity to compare actual conditions with those encountered in our investigation and, if necessary, to expedite supplemental recommendations if warranted by the exposed surface and subsurface conditions

6.2 LANDSLIDES AND SLOPE STABILITY -

Landslides have been identified at the site as reported above and shown on Figure 2. Although the mass of soil that would be associated with the two larger slides shown on Figure 2 appears to have been eroded out from below the slides, there is some potential that shallow failures will occur on the faces of the slopes at the headscarp areas and on the flatter slopes below. Due to the granular nature of the soils, where sliding does occur, it tends to be surficial and the slide materials tend to deposit out fairly rapidly. Based on the observed conditions below the two larger slides, as well as at Lots 14 through 17, we recommend that appropriate site specific method(s) of mitigation be included in the development of plans for individual homes on these lots. Potential methods of mitigation include but or not necessarily limited to construction of the houses at Lots 11 and 13 through 16 be as far down the slope as possible (at the roadway setback), construction of debris walls or energy dissipation structures just below the mapped slides. Placement of the houses forward on the lots will allow for deposit of materials upslope of the planned development should sliding occur on the steeper slopes above these sites. Debris wall or dissipation structures would cause the materials to be diverted or to lose energy, thus depositing on the site prior to reaching the occupied structures below. Although there are no slides mapped at Lots 5 and 6, a similar siting approach is recommended at these lots as well. This should be considered in future assessment of the lots during the preparation of lot-specific development plans.

The potential for surficial sliding of the colluvial soils at Lots 11 and 13 through 16 can be reduced through the installation of subsurface drains. The drains will alleviate the build-up of a perched groundwater condition, believed to be the triggering mechanism for the shallow slides observed at Lots 14 and 15. Drains should be installed along the contact between the steeper 2:1 slopes above and the flatter slopes trending from 3:1 to 5:1 below. This is generally along the contact between the grass-covered slopes and those vegetated with a dense growth of brush and trees at Lots 13 through 16. The contact is less visible at Lot 11 but can be identified on the topographic map. Additional subdrains should be constructed across the flatter slopes as well.

The specific depths and locations of the subdrains should be established during development of site-specific improvement plans.

Due to the close proximity to the steeper slopes at Lots 8 and 9, and the unstable condition of the slope as demonstrated by the undulating terrain, mitigation will likely require reconstruction of the slope as a fill slope with internal drainage. It may be feasible to leave the slope intact, install subsurface drainage and to protect the planned structures with a debris wall. This will require further evaluation in consultation with the developers of these lots.

Based on the steep to very steep condition of the north facing slope at Lot 17, we recommend against siting of a house at the top of the ridge. Ideally, the house should be sited just south of the break at the south side of the top of the ridge, placing the house on south facing slope. A multi-story house notched into the south slope will still allow for a north view, would place the structure on more stable ground and could aid in allowing for development of the driveway.

6.3 LIQUEFACTION AND LATERAL SPREADING

As discussed above, these are loose sand deposits blanketing the flatter portions of the site. Where there is an absence of cohesive fines and/or significant coarse material (gravel), and a build-up of a perched water condition occurs, there is a potential that liquefaction or partial liquefaction could occur. With the relatively thin section of loose soils, pore pressure would likely dissipate quickly, alleviating the condition. However, where this occurs on a sloping surface, the soils could mobilize and spread downslope before the pore pressure is released.

The installation of subsurface drains, as discussed above under "Landslides" would aid in mitigating the potential for this occurrence. In addition, removal and reconstruction of the loose soils at the locations of the structures, with integral subsurface drainage, will aid in protecting the structures. Lots where this is of greater concern, based on the data available at this time, include Lots 2, 9, 10, and 13-16.

6.4 SURFACE EROSION

As clearly demonstrated on-site with the presence of significant erosion gullies, the soils are highly erodible once the surface soils have been disturbed. Limiting disturbance of the areas outside of the future grading and development areas will be essential in controlling erosion. The site development plans must include extensive erosion protection features to mitigate the increased flows and concentration of surface flows as a result of development. This will include the need to install lined ditches above and below any engineered slopes (cut or fill), and above existing erosion gullies if their continued development is to be slowed. Where surface soils and vegetation are disturbed, development of erosion mitigation plans should include, as a minimum, consideration of the use of vegetative matting where hydroseeding is to be used. The hydroseed mix should be developed as a site-specific mix which will produce a dense growth of grasses.

6.5 WEAK SURFACE SOILS

The surficial colluvial deposits are general comprised of silty and gravel sands and are generally medium dense to loose. These soils generally extended to depths of about 3 to 5 feet below existing grade and were encountered both in hillside and level areas of the site. In their current condition, these soils are not adequate to support embankments, roadways or houses. In addition, where roadways are planned for hillside areas, potentially unstable conditions could exist if these soils are left in place.

To provide support for the proposed embankments and roadways some rework (removal and compaction) of the surface soils will be necessary. The rework will include removal of surface soils followed by replacement and compaction of the excavated material. This will entail the construction of horizontal benches to alleviate potential soil movement associated with adverse dipping (dipslope) contacts between the colluvium and the underlying bedrock. Where roadways are planned for hillside areas, the removal depth and lateral extent will be greater and will include the excavation of keyways and benches into competent soil or bedrock where encountered.

Removal and replacement of these soils as engineered fill will improve their engineering characteristic, lessening their potential to undergo liquefaction where saturated or dynamic compaction where above groundwater. This will also improve bearing capacity, a consideration where retaining walls, house or other earth-supported structures are to be constructed.

6.6 Existing Uncontrolled Fills

Construction of the dirt roadways found in several areas of the site required the construction of fills. These fills are often found traversing slopes and are associated with cut/fill roadway construction. In at least one location, the fill for a roadway crosses a drainage ravine. This is at the main road in to the site. In all cases, these fills, and any other fill encountered on the site, are considered to be uncontrolled and will need to be removed prior to further development of these roadways as part of the development of the site. The material may be reused as fill as discussed below in the "Earthwork" section of this report.

Uncontrolled fill also exists at the locations of our exploratory test pits. The backfill materials will need to be re-excavated at these locations during site grading activities. The materials, when properly moisture conditioned may be replaced as compacted (engineered) fill in accordance with the recommendations for fill placement presented below.

6.7 Erosion Gully Wall Stability – Development Setbacks

The previously discussed there are a number of erosion gullies at the site with very steep to vertical wall conditions. These walls or slopes are not considered to be stable and thus should be avoided. Where erosion protection and/or slope stability improvements cannot be made due to jurisdictional constraints, the minimum setback for roadways should be equal to 2 times the height of the vertical or near-vertical feature. Where slopes within the erosion feature are 2:1 or steeper, the setback should be based on a plane projecting up from the base of the feature at an inclination of 2:1 to the ground surface behind the gully.

Setbacks for houses should be equal to 4 times the height of the vertical or near vertical feature. Where slopes within the erosion feature are at an inclination of 4:1 or steeper, the setback should be based on a plane projecting up from the base of the feature at 4:1 to the ground surface behind the gully. Where further erosion of the gullies can be controlled or prevented, a reduction in setback may be feasible. This should be evaluated on a case-specific basis.

7.0 PRELIMINARY DESIGN AND CONSTRUCTION RECOMMENDATIONS

7.1 EARTHWORK

7.1.1 General

All site preparation and grading should be performed in accordance with the preliminary recommendations contained in this report, and any subsequent site and project recommendations developed current with development plans. We recommend that all earthwork be observed by the Geotechnical Engineer. Final depths of stripping, rework, benching and keying should be assessed in the field by the Geotechnical Engineer at the time of grading. In addition, the conditions of exposed soils or geologic units at cut slopes should be evaluated by the Geotechnical Engineer and Engineering Geologist. The need for, and the extent of, additional slope stabilization measures should be determined during the grading of the site.

7.1.2 Site Clearing & Stripping

Prior to start of construction, the sites for proposed roadways and infrastructure improvements should be cleared of designated trees. Root balls associated with the existing trees should also be removed. Due to the large size of some of the mature trees at the site, excavations on the order of three to five feet deep may be required at these locations. Excavations resulting from the clearing operations should be backfilled with engineered fill placed and compacted in accordance with "Subgrade Preparation, Fill Placement and Compaction," below.

All surface vegetation should be removed from the areas of proposed construction. Stripping should include both the above grade vegetation as well as any associated dense root zone, such as would be expected where dense brush is present. Where vegetation is limited to grass and weeds in areas of cut, it may be feasible to limit stripping to removal of the surface vegetation by cutting at or just below the ground surface. The actual depth of the required stripping should be determined by the Geotechnical Engineer in the field at the time of grading. The stripping may be stockpiled for possible later use as topsoil fill in landscape areas.

To allow us to substantiate that our recommendations for site clearing and stripping have been adhered to, the site clearing work should be performed under the observation of a representative from D&M/Terratech.

7.1.3 Rework Of Surface Soils

To provide support for future mass fills, embankments and roadways, some rework of the surface soils should be performed. Where mass fills, embankments or roadways are planned in level areas or where slope inclinations are 8 horizontal to 1 vertical (8:1) or flatter, as a minimum, we recommend that the upper three feet of colluvium be removed, that the exposed soils be further evaluated by the Geotechnical Engineer, and that the removed soils be replaced as engineered fill once all loose soils have been removed. This may require excavation of the colluvial surface soils be excavated down to the underlying bedrock. Laterally, the soils should be removed out to a minimum distance of 5 feet from the edges of the proposed roadway section. Exposed subgrade should than be prepared in accordance with our recommendations presented below under "Subgrade Preparation, Fill Placement & Compaction." Following subgrade preparation, the stockpiled soil may be reused as fill provided it meets the requirements for fill presented under "Material For Fill."

Where fills, embankment or roadways are planned for areas of the site with slope inclinations steeper than 8:1, the depth and extent of the excavation will be more extensive. In these areas we recommend that material be entirely removed down to competent medium dense or denser soil, or to bedrock whichever is shallower and the excavated material stockpiled. After removal of the surface material, the exposed competent surface should be keyed and benched. The construction of keyways and benches, and subdrain installation are discussed below. Depths of removal, keying, benching and subdrain installation will all require observation by the Engineering Geologist and Geotechnical Engineer prior to fill placement and rebuilding of these rework areas.

7.1.4 Keyways & Benches

Where fill is to be placed as part of rework of loose soils on hillside slopes steeper than 8:1, keyways and benches should be excavated into the exposed competent medium dense soil or bedrock to provide support for the fill. Typically, keyways should be excavated at least 4 feet below the surface of the competent material and have a minimum width of 10 feet. Horizontal benches should be excavated into competent materials typically at 3-foot vertical intervals as the fill placement progresses up the slope. Benches should be excavated at least 1 foot below the surface of the competent material.

Subsurface drainage should be provided in keyways, on intermediate benches as appropriate, and in natural seepage areas and existing drainage courses to be filled. Recommendations regarding subsurface drainage are presented below.

7.1.5 Subgrade Preparation, Fill Placement & Compaction

After the stripping, clearing and loose soil removal, the exposed ground surface in areas to be filled, with the exception of intact bedrock or excavated benches, should be scarified to a depth of about 8 inches, moisture conditioned and compacted.

All sandy subgrade and fill soils should be moisture conditioned to between 2 percent above or below optimum moisture content and compacted to at least 95 percent relative compaction. Similarly, aggregate base should be compacted to at least 95 percent relative compaction in accordance with and ASTM procedures. Fill materials should be placed in horizontal lifts not exceeding 8 inches in uncompacted thickness. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended levels of compaction.

The poorly graded silty and sandy materials encountered at the site are judged to be relatively sensitive to compaction moisture content. Compaction of these materials can be difficult if the moisture content is not adequately controlled. Grading operations during the wet season or in areas where the soils are saturated may require provisions for drying the soil prior to compaction. If the project necessitates fill placement and compaction in wet conditions, we can provide alternative recommendations for drying the soil. Conversely, additional moisture may be required during the dry months. Water trucks should be available in sufficient number to provide adequate water during compaction.

7.1.6 Material For Fill

In general, the on-site soils without excessive visible organic matter as judged by the Geotechnical Engineer, and free of any deleterious materials or hazardous substances, may be used as engineered fill to achieve project grades.

Rocks or concrete fragments larger than 4 inches in maximum dimension should not be placed in fill areas within five feet of finished rough subgrade. Rocks larger than 12 inches in maximum dimension should not be placed in the upper 12 feet of fill areas. Oversized rocks, larger than 12 inches, may be placed in the deeper fills. Rocks larger than 2 feet in diameter should be removed from fill material and disposed of as directed by the Civil Engineer.

7.1.7 Trenches

Based on our experience in excavating our exploratory test pits, we anticipate that excavations for utility trenches can be readily made with either a conventional backhoe or excavator throughout most of the site. There are demonstrated zones of cemented sols that may impact production or require the use of larger more powerful equipment. The soils found at the site are predominately granular. Where cemented and surface runoff is controlled the bedrock Paso Robles materials will stand vertical. However, the degree of cementing and the gradation of the materials are variable. These soils can be prone to sloughage as they dry when cut at steep inclinations. All trenches should be constructed in accordance with OSHA and Cal-OSHA Safety Standards. Safety in and around utility trenches is the responsibility of the underground contractors.

All underground utility trenches should be backfilled with compacted engineered fill. The silty to sandy site soils or imported sand may be used for backfilling utility trenches. The trench backfill should be compacted to at least 95 percent relative compaction and capped with a minimum 12-inch thick layer of compacted, on-site fill soil similar to that of the adjoining subgrade. The trench backfill material should be placed in lifts not exceeding six inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting to attain compaction should not be permitted.

For purposes of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe; bedding and shading is all material placed in a trench below the backfill. With the exception of specific requirements of the local utility companies or building department, pipe bedding and shading should consist of clean medium-grained sand. The sand should be placed in a damp state and should be compacted by mechanical means prior to the placement of backfill soils. The sand should be compacted to at least 90 percent relative compaction. Above the pipe zone, underground utility trenches may be backfilled with either free-draining sand, on-site soil or imported soil. The trench backfill should be compacted to at least 90 percent relative compaction. Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to at least 95 percent relative compaction. The backfill material should be placed in lifts not exceeding 6 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water-jetting or flooding to attain compaction of backfill should not be permitted.

Where trenches are to be located down the face of a slope, there is a potential for seepage water to collect in the trench and to cause trench failure (blow-out) where water pressure builds-up. The potential for this to occur can be mitigated through the use of subsurface drainage or check dams or trench plugs within the trench. Trench plugs may consist of controlled density fill (2 sack/cubic yard cement/sand slurry), compacted clay soils, cross-trench filter fabric wrapped gravel. This should be considered in civil design.

To maintain the desired support for foundations, such as may be required for retaining walls, utility trenches should be located such that the base of the trench excavation is located above an imaginary plane having an inclination of 1.5 horizontal to 1.0 vertical, extending downward from the bottom edge of the adjacent footing.

7.1.8 Site Slopes

7.1.8.a General

Project planning was at the feasibility stage with development of a site plan but not grading plans at the time this report was prepared. Based on our understanding of the project, we would not anticipate cut or fill slopes constructed as a part of this development to exceed 30 feet in height. Deep-seated slope failures should not occur in cut and fill slopes that are designed and constructed in accordance with the recommendations presented in this report. However, shallow slope failures could still occur as the result of erosion and/or water infiltration. Therefore, it is important that the drainage and erosion control recommendations presented in this report are implemented into the design and construction of the site. Furthermore, it is essential that these measures be maintained on a regular basis after construction. All cut and fill slopes should be constructed with drainage and intermediate benches in accordance with the Uniform Building Code.

7.1.8.b Cut Slopes

We recommend that cut slopes above proposed roadways be designed and constructed no steeper than 2.5 horizontal to 1 vertical (2.5:1) in medium dense to dense sands. Cut slope areas where fragmented bedrock, out of slope bedding, or unstable soils are encountered during construction could require the construction of earth buttresses, stability fills (slope reconstruction) or other remedial measures. Cut slopes below colluvial soil deposits will expose less dense soils and may possibly expose a dipslope condition in some areas. Where colluvial soils are exposed in a cut slope, a flatter gradient or remedial grading above the cut slope will be required. This may entail over-excavation of the cut slope followed by the construction of a slope buttress. This may adversely impact trees or other features that are to be left undisturbed. This must be considered in civil design.

All cut slopes should be observed by our Engineering Geologist and Geotechnical Engineer at the time of grading to assess the applicability of our recommendations and to make supplemental recommendations, if necessary. Supplemental recommendations may include slope-flattening, installation of subsurface drainage, or slope reconstruction in areas where geologic weaknesses or local anomalies are encountered during site earthwork.

7.1.8.c Fill Slopes

We recommend that site fill slopes associated with proposed roadways be designed and constructed no steeper than 2.5:1 (horizontal to vertical). Slopes may be constructed if the fill slopes are internally reinforced during reconstruction. Internal reinforcement generally consists of geogrid. Slopes steeper than 2.5:1 and any slope to be stabilized with geogrid will require site and project specific design recommendations. These can be discussed after site planning has been developed further.

Construction of fill slopes above existing slopes will require a bench at the base of the fill slope. Where there is insufficient room to construct a bench, mechanical stabilization of the natural slope may be required prior to fill construction. Where a bench or mechanical stabilization is not provided, there may not be adequate support for the slope and failure could result in the natural slope below the fill.

Fill material must be compacted to the face of the slopes. To accomplish this, we recommend that slopes be over-built a minimum of four feet horizontally and then trimmed to design grades. The construction of a bench below the fill slope will aid in this process. Other methods may also provide the desired compaction. Proposed alternative methods should be submitted to us for review. Although deep-seated failures should not occur in properly compacted fill slopes, even properly designed and constructed fill slopes have a potential for shallow failures or surficial sloughage, particularly during periods of wet weather.

All fill slopes should be keyed and benched into the foundation and backslope soils. Vertical distance between benches should not exceed 3 feet. The construction of internal drainage is a critical element of the long-term stability of fill slopes. All keyways and intermediate benches on fill slopes should be drained. Additional subsurface drainage will be required in areas where seepage is evident or suspected and in areas where surface drainage exists. Subdrains should be constructed in accordance with the recommendations presented below under "Drainage."

7.2 DRAINAGE

7.2.1 Subsurface Drainage

Where fill slopes are to be constructed at existing slopes, subsurface drainage will be required at the contact between the native soils and the fill. This will require the installation of drains on the benches cut to support the fill as discussed in the preceding section. In general, bench drain spacing should be no more than 10 feet vertically and more than 30 feet horizontally. The drainage systems should consist of a combination of interceptors consisting of perforated pipe surrounded by drain rock (free draining ½- to ¾-inch crushed gravel) wrapped in filter fabric (Mirafi 140 or equivalent, lapped 12" at joints), and drainage blankets. Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68) may be an acceptable alternative to drain rock wrapped in filter fabric in areas were the volume of seepage is anticipated to be low. Class 2 permeable material may only be used where specifically approved by the Geotechnical Engineer during site grading.

Subdrain pipes should consist of rigid ABS (SDR-23.5) or PVC Schedule 40 minimum for locations where cover is up to 50 feet in height, and ABS (SDR-15.3) or PVC Schedule 80 minimum for fills greater than 50 feet in height. The lateral drains should have a minimum diameter of four inches. Laterals should be connected to a main line with a minimum diameter of six inches. The actual locations, depth and extent of the subsurface drainage systems should be assessed by us in the field at the time of construction.

Keyways and benches cut into the hillside at the back-cut of the slope should have a minimum slope of 1 percent along the length of the slope. A subdrain pipe and drain rock wrapped in a geotextile filter fabric should be installed at the back of the base of the slope excavation. The fabric wrapped gravel section should extend up the back of the cut a minimum of 3 feet (as with a chimney drain). A clean-out should be provided at the high end of the keyway subdrain line.

The drainage systems should consist of lateral drains or interceptors consisting of perforated pipe surrounded by drain rock wrapped in geotextile filter fabric (Mirafi 700X or equivalent, or Mirafi 140N or equivalent, lapped 12" at joints), connected to riser pipes also surrounded by drain rock wrapped in filter fabric.

Subdrains are also recommended in the colluvial deposits at Lots 8-10, 11 and 13-16 to aid in mitigating perched groundwater conditions and slope instability at these locations. Where roads are to be constructed across slopes, subdrains should be constructed at the toe of the slope to aid in protecting the baserock and subgrade soils from saturation and softening.

Subdrains installed to provide drainage of the colluvial soils at Lots 11, 8-10 and 13-16 should extend through the colluvium and in the Paso Robles Formation a minimum of two feet. This will result in a minimum depth of trench of about 6 feet. Subdrains along roadways should extend two feet below finished subgrade. These drains should be a minimum of 12 inches wide. Drains should be sloped at a minimum of 1 percent to a suitable discharge point.

Drain rock should be clean, crushed rock or gravel conforming to the following gradation:

Sieve Size	Percent Passing
1 - 1/2 inch	100
3/4 inch	90 - 100
No. 4	0 – 5

An alternate to the use of fabric wrapped gravel is Caltrans Class 2 permeable backfill (Caltrans Section 68). Where Class 2 permeable backfill is used, the filter fabric may be deleted. Subdrain pipes be bedded on at least 4 inches of drain rock or Class 2 permeable backfill. Pipe should be installed with perforations down and sloped to drain toward an appropriate collection facility. Each subdrain should be provided with at least one near vertical clean out of non-perforated plastic pipe which extends to the surface. Clean-outs should be installed on all main line subsurface drains and lateral lines.

Also, we recommend that subsurface drainage be provided in any natural drainage areas to be filled and in areas of observed or suspected seepage. The need for additional subdrains in other areas of the site should be evaluated by our Engineering Geologist and Geotechnical Engineer during the grading of the roadways and detention basins.

7.2.2 Surface Drainage & Erosion Control

Good surface drainage is essential to intercept and control surface water runoff to reduce slope erosion and subsurface infiltration. Effective erosion-control landscaping is also important. Measures to control surface water and erosion include placement of drains on and above cut and fill slopes, reduction of ponding of water, proper grading to prevent surface water flow over the tops of slopes, construction of berms at the top of slopes, installation of concrete V-ditches, landscaping of slopes, and control of irrigation on slopes. These items are discussed below.

Concentrated water should not be allowed to flow uncontrolled across slope faces. Areas above slopes should be graded to a 2 percent gradient or greater to direct surface water away from the top of the slopes and toward a suitable point of discharge such as erosion controlled ditches or surface drain inlets. Straw bale dikes and/or siltation basins should be constructed to reduce siltation during construction. Erosion control V-ditches, brow ditches, or intermediate benches, should be constructed on slopes where substantial surface water runoff is expected. Lined ditches and temporary silt fences should also be considered at the toe of both cut and fill slopes. Where benches are used for slopes greater than 30 feet in height, V-ditches are recommended to intercept surface water flowing down from above the benches. These types of surface drainage features should also be installed around the perimeter of the erosional ravine to reduce continued erosion. This would likely include either V-ditches combined with construction of a bench at the top of the ravine walls or the installation of a subdrain to capture water flowing along the soil/bedrock interface.

Slopes adjacent to proposed roadways should be protected from erosion by utilizing a system of erosion matting such as Excelsior blanketing or other erosion control matting combined with plantings of appropriate ground cover vegetation to reduce the potential for future erosion and possible slope deterioration. Planting should occur sometime prior to the start of the rainy season. Additional planting may be needed if the initial planting is partially or totally unsuccessful. A professional landscaper should provide specific details regarding erosion matting and planting. Areas of erosion should be anticipated even with erosion planting. If planting is unsuccessful, other mitigating measures such as temporary silt fences might be necessary, depending upon the susceptibility of the exposed materials to erosion.

We recommend that irrigation of all slopes be kept to a minimum. Over-watering of slope surfaces could result in surficial instability and/or downward creep of the near surface soils. For slopes that must be irrigated we recommend the use of a low volume system such as drip irrigation.

7.3 STRUCTURES

7.3.1 Foundations

Foundations will be required for support of retaining walls, houses and other site improvements. With proper site improvement including removal of all loose, weak, compressible or landslide-prone materials, the use of conventional shallow foundations may be feasible. The presence of deep fills or fills with differential thickness of more than 15 feet may adversely impact the performance of a shallow foundation system and will need to be considered in future planning. Where structures are to be constructed on a cut/fill type building pad, or on or immediately above slopes, drilled piers and grade beams may needed. Foundation design parameters will be structure dependent due to the variations in soil types, site preparation alternatives selected, grading activities, proximity to slopes and possibly other factors not evident at this time. Structure specific foundation recommendations should be developed after the structure type and siting, as well as preliminary loading information have been determined.

7.3.2 Earth Retaining Structures.

There are a number of viable alternatives for support of earth fills or where steep cuts are required. These include conventioanl cast-in-place concrete or masonry retaining walls supported by spread footings or drilled cast-in-place piers, soldier piles with lagging, segmental block walls with integral geogrid reinforcment the the retained soil mass (i.e. Keystone, Versa-Lok and others),mechanically stabilized earth (MSE) walls and soil nailed walls among others.

Soil nailing is well-suited to retaining cuts because no excavation is required behind the wall. Soil-nail walls have been used in a variety of civil engineering projects including stabilization of relatively high cut slopes. This technique has been employed to stabilize or support cut slopes on countless private developemnt as well as federal and state highway projects. The system works as a gravity retaining wall and helps restrain the movement of the soil mass behind the wall. Soil-nail walls will serve two purposes in their application on this project. The soil-nail structure will provide temporary shoring during excavations and will remain in place as part of the permanent lateral support system after completion of the construction. A soil-nail wall system involves a number of steel reinforcing bars (soil nails) installed in closely spaced drilled holes and grouted into the soil as the soil face is exposed during excavation. Construction is performed in vertical steps starting at the top of the excavation and proceeding downward in approximate 1.5 to 2-m high lifts. Permanent facings for soil-nail walls often includes reinforced shotcrete and may consist of cast-in-place and precast concrete panels to support the excavation face. A shotcrete facing that is colored and textured to blend in with the surrounding soil and rock can be constructed for aesthetic reasons.

Where large fills are to be constructed with a retained face, gravity walls including segmental block and MSE walls are often found to be very cost effective. Construction of these types of walls entails erection of a rigid facing material (concrete block or concrete panels) with geogrid reinforcement or steel straps mechanically connected to the facing and embedded in the engineered fills as the fill progresses up.

Where site constraints do not allow for placement of geogrid or straps, construction of a soldier pile and lagging wall or concrete/concrete block wall may be required. Selection of the foundations for these types of walls will be influenced by both site lateral constrains and proximity to descending slopes below the wall.

Recommendations for preliminary wall design can be prepared once information regarding wall location, height and anticipated loading with surcharge becomes available. Site-specific subsurface data may be required before formal design level recommendations can be prepared.

7.4 PAVEMENTS

Pavements for this project are expected to consist of private road providing access to the 17 custom lot sites. Traffic will ultimately consist primarily of light passenger vehicles cars, with truck traffic limited to garbage trucks and occasional delivery or moving trucks, or fire trucks. The potential for school bus traffic has not been discussed with us. Based on our experience with similar projects, we suggest using a Traffic Index (TI) of at least 5.5 for minor roadways with a TI of at least 6.0 for the main roadway. The actual traffic indexes to be used in site development should be assigned by the Civil Engineer in accordance with local requirements. For roadways of this type, a minimum asphalt concrete section of 4 inches is recommended over the appropriate aggregate base section for the given TI and subgrade soil.

Laboratory testing of the soils at the site has not been performed as a part of this preliminary study. We recommend that once site grades and roadway locations have been determined that soil sampling be performed. Resistance or R-value testing should then be performed to develop site-specific pavement sections in conjunction with the assignment of the appropriate Traffic Indexes by the Civil Engineer. Asphalt Concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type B asphalt concrete. These materials should comply with the specifications presented in Section 39 of the Caltrans Standard Specifications, latest edition. Class 2 aggregate base shall also conform to the materials specifications as presented in the Caltrans Standard Specifications, latest edition. ASTM Test procedures and should be used to assess the percent relative compaction of soils, aggregate base and asphalt concrete. Asphalt concrete should be compacted to a minimum of 96 percent of the maximum laboratory compacted (Hveem) unit weight.

Ideally, pavement areas should be sloped at a minimum of 2 percent and drainage gradients maintained to carry all surface water off the site due to the slightly porous or permeable nature of asphalt concrete. Surface water ponding should not be allowed anywhere on the site during or after construction.

8.0 ADDITIONAL SERVICES & LIMITATIONS

8.1 ADDITIONAL SERVICES

As noted above, we anticipate that work beyond the scope of the current study would include a plan review of the site specific mass grading plan once that plan becomes available, development of site specific grading recommendations based on the plan, supplemental subsurface exploration as required to more fully address future site plans, and ultimately the review of grading plan and specifications. The review of plans and specifications, and the observation and testing by D&M/Terratech of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report.

The required tests, observations, and consultation during construction include, but are not limited to:

- observation of site clearing and stripping.
- observation of over-excavation and replacement of existing uncontrolled fills and weak surficial soils (colluvial deposits) in fill and roadway areas.
- construction observation and density testing during subgrade preparation, placement and compaction of fill material, backfilling of utility trenches and finished pavement subgrades, and aggregate base.
- observation of subdrain installations.
- observation of site surface drainage improvements.
- construction observation and density testing during backfilling of retaining walls.

Further geotechnical studies and a final geotechnical report will be required to evaluate geotechnical conditions and provide final recommendations with respect to development lots, the design and construction of tract improvements including utilities, sidewalks, and streets, possible subsurface drainage in relation to these improvements, landscaping features that may have a geotechnical effect, finished lot drainage and erosion control, exterior flatwork, lot specific retaining walls, swimming pools and spas, and other items.

8.2 LIMITATIONS

Recommendations contained in this report are for the construction of proposed roadways, infrastructure improvements, and detention basins as described in this report. Our recommendations are based upon field observations, data from our exploratory borings and test pits, laboratory tests, and our present knowledge of the proposed construction. It is possible that subsurface conditions could vary between or beyond the points explored. If soil and groundwater conditions are encountered during construction which differ from those described herein, our firm should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including proposed grades, or roadway locations change from that described in this report, our recommendations should also be reviewed.

Our firm has prepared this report for exclusive use of Michael Wilson and Dana Broccoli Wilson (owner/applicant for development), and their Civil Engineering Consultant, Whitson Engineers, in substantial accordance with the generally accepted geotechnical engineering and engineering geology practices as they exist in the site area at the time of our study. No warranty, expressed or implied, is made. The recommendations provided in this report are based on the assumption that D&M/Terratech will be retained to consult with the Civil Engineer as site improvement plans are further developed and during the construction phase in order to evaluate compliance with our recommendations. It is the client's responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety including the Additional Services and Limitations section, as well as any subsequent reports or documents providing supplemental recommendations.

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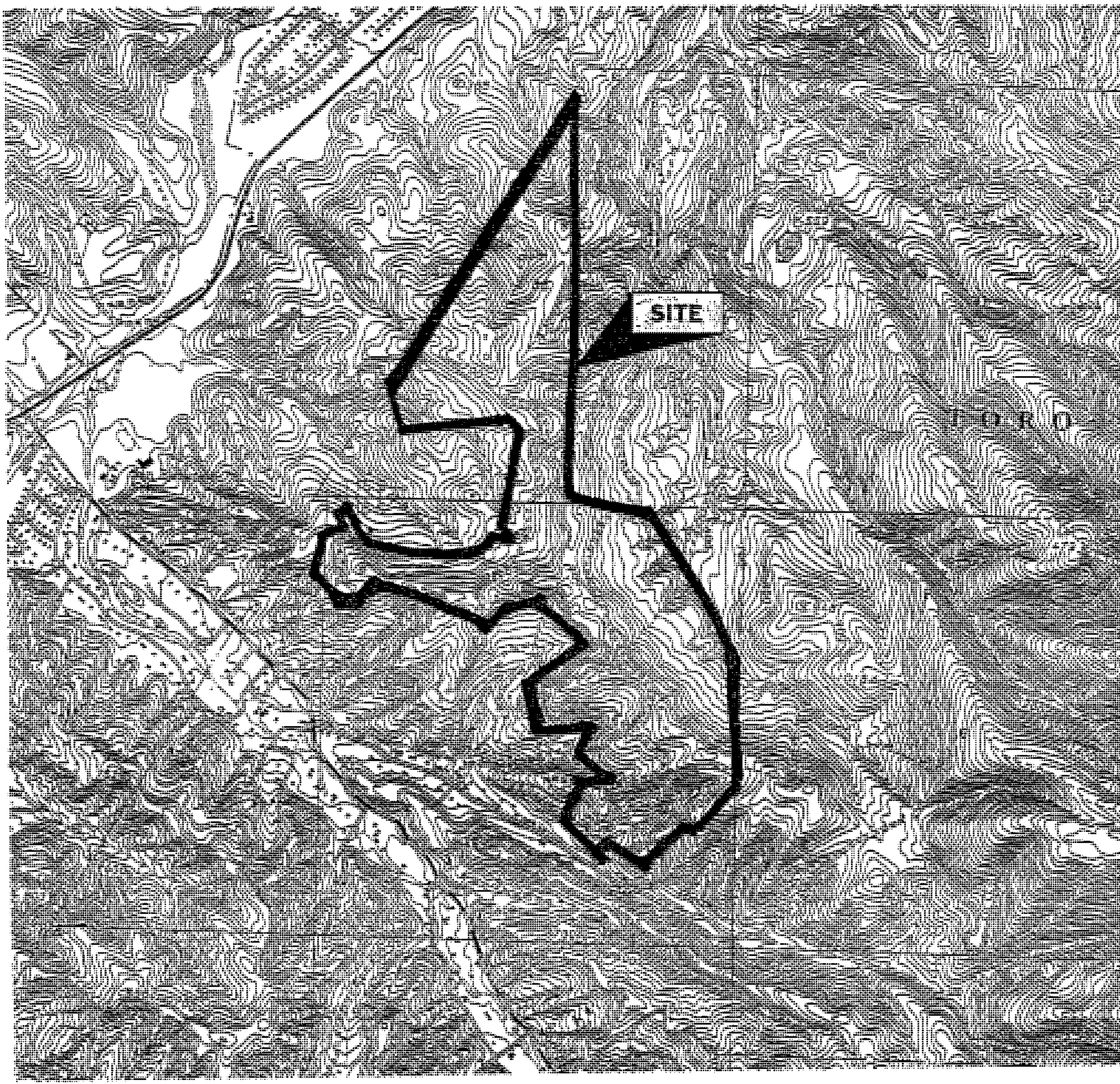
Aerial Photographs

Date	Source	Type	Frame	Scale
8/17/49	UCSC	black & white	ABG-17F-15, 16, 17	1:20,000
6/13/68	UCSC	black & white	GS-VBZK 2-138, 139, 140	1:15,000
8/28/81	UCSC	black & white	CDF-ALL-MO 18-11, 12, 13	1:24,000
9/20/97	UCSC	black & white	WAC-97CA 14-9, 10, 11, 65, 66	1:24,000



MAP REFERENCE: USGS 7.5–Minute Topographic Quadrangle, Spreckles, CA, 1947, photorevised 1984.

0 2,000'

SCALE: 1" = 2,000'

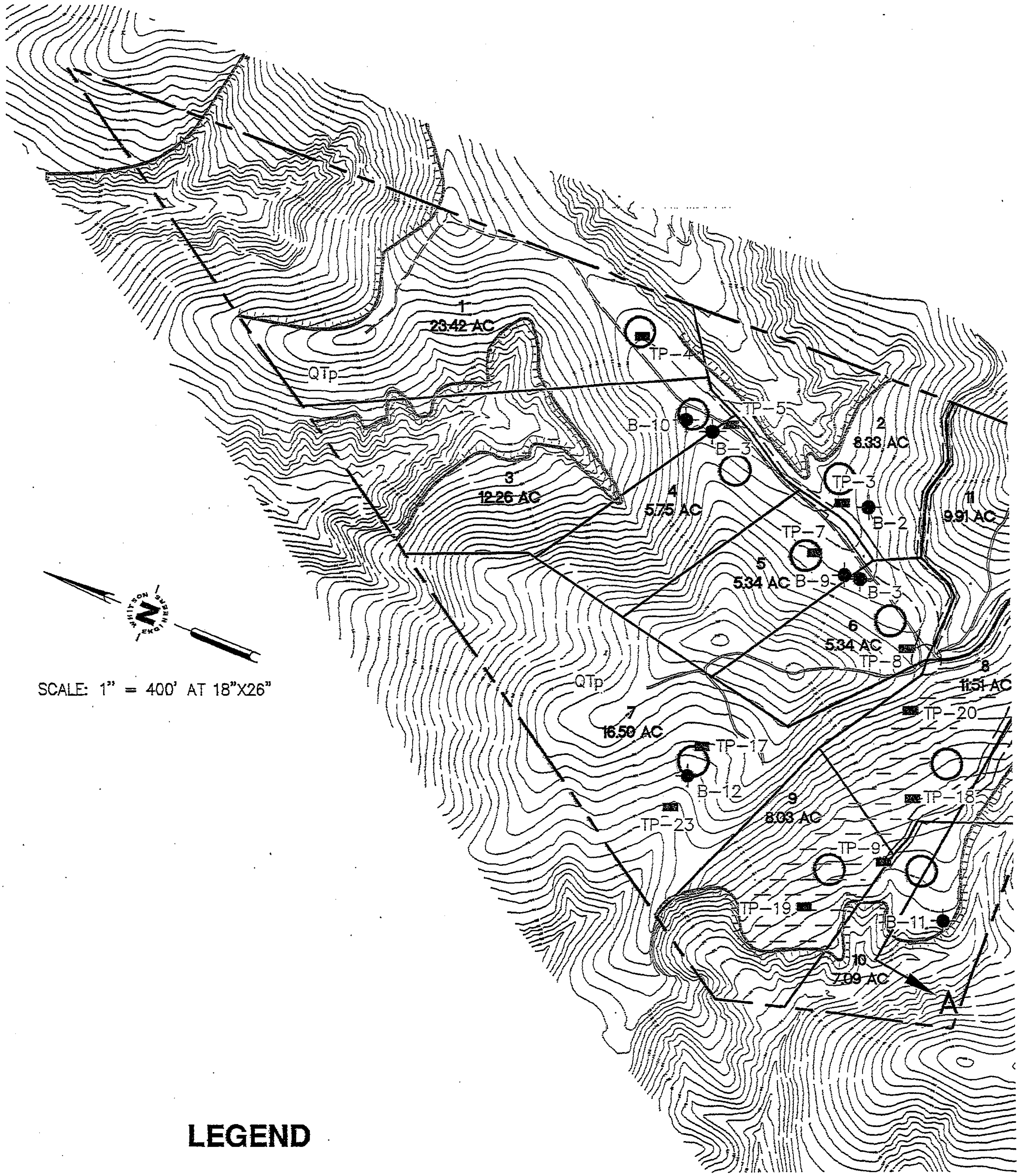
N

JUNE 2001

D&M CONSULTING ENGINEERS, INC.
A URS CORPORATION COMPANY

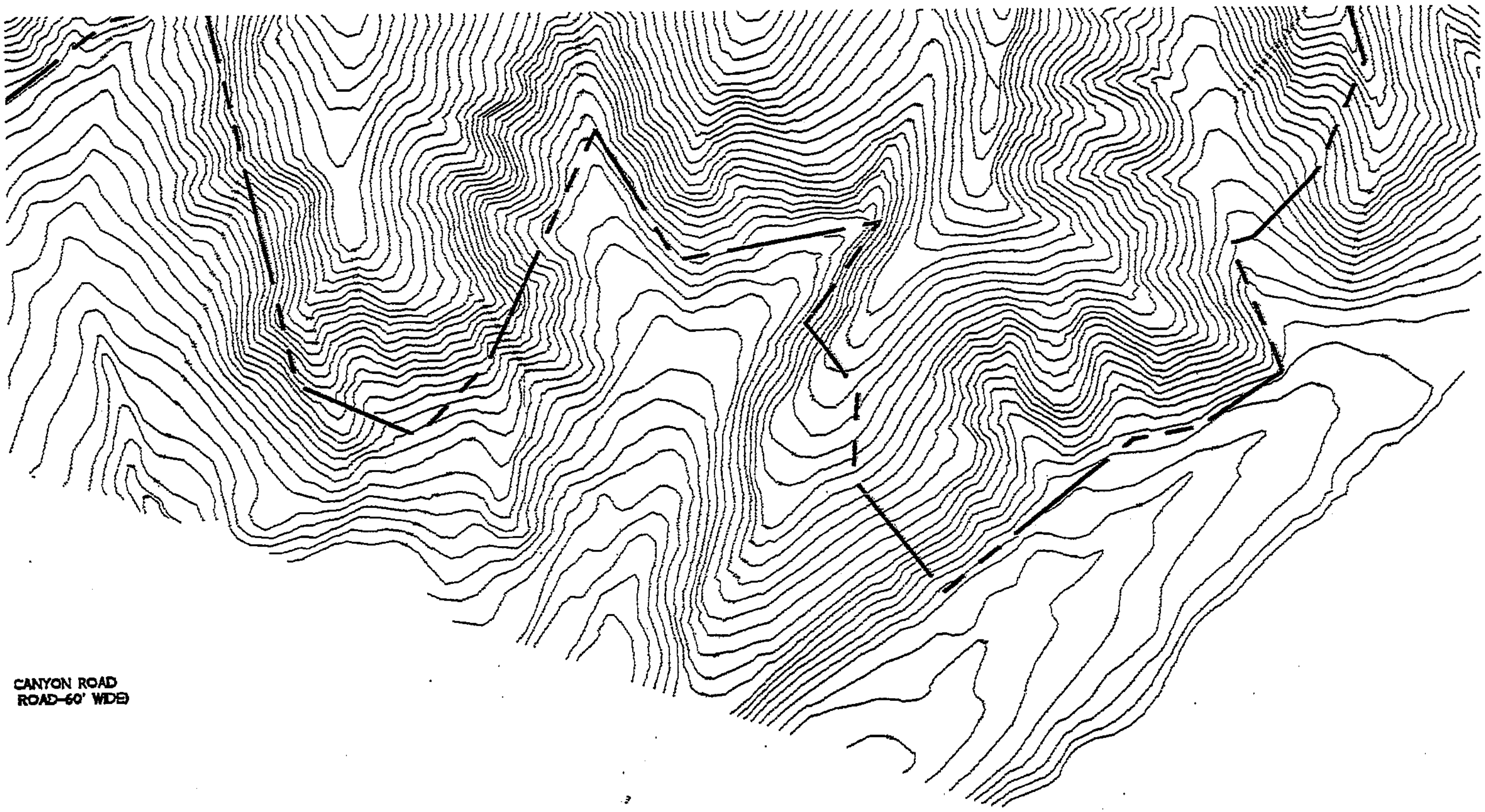
VICINITY MAP
ENCINA HILLS SUBDIVISION
MEYER ROAD
MONTEREY COUNTY, CALIFORNIA

FIGURE
1
PROJECT
1892



SCALE: 1" = 400' AT 18"X26"

LEGEND



ENCINA HILLS SUBDIVISION

FIGURE 2 - GEOLOGIC SITE MAP

PROJECT 1892

D&M CONSULTING ENGINEERS, INC.

A URS CORPORATION COMPANY

JULY 2001

BASE MAP REFERENCE: WHITSON ENGINEERS (MAY 31, 2000)

LEGEND

A A LINE OF GEOLOGIC CROSS SECTION (SEE FIGURE 4)

B-12 SOIL BORING/ PERCOLATION TEST LOCATION

TP-22 TEST PIT LOCATION

LANDSLIDE DEPOSIT AND SCARP, SHOWING DIRECTION OF MOVEMENT

AREA WITH SEASONAL PERCHED GROUND WATER CONDITIONS

SPRING OR SEEP

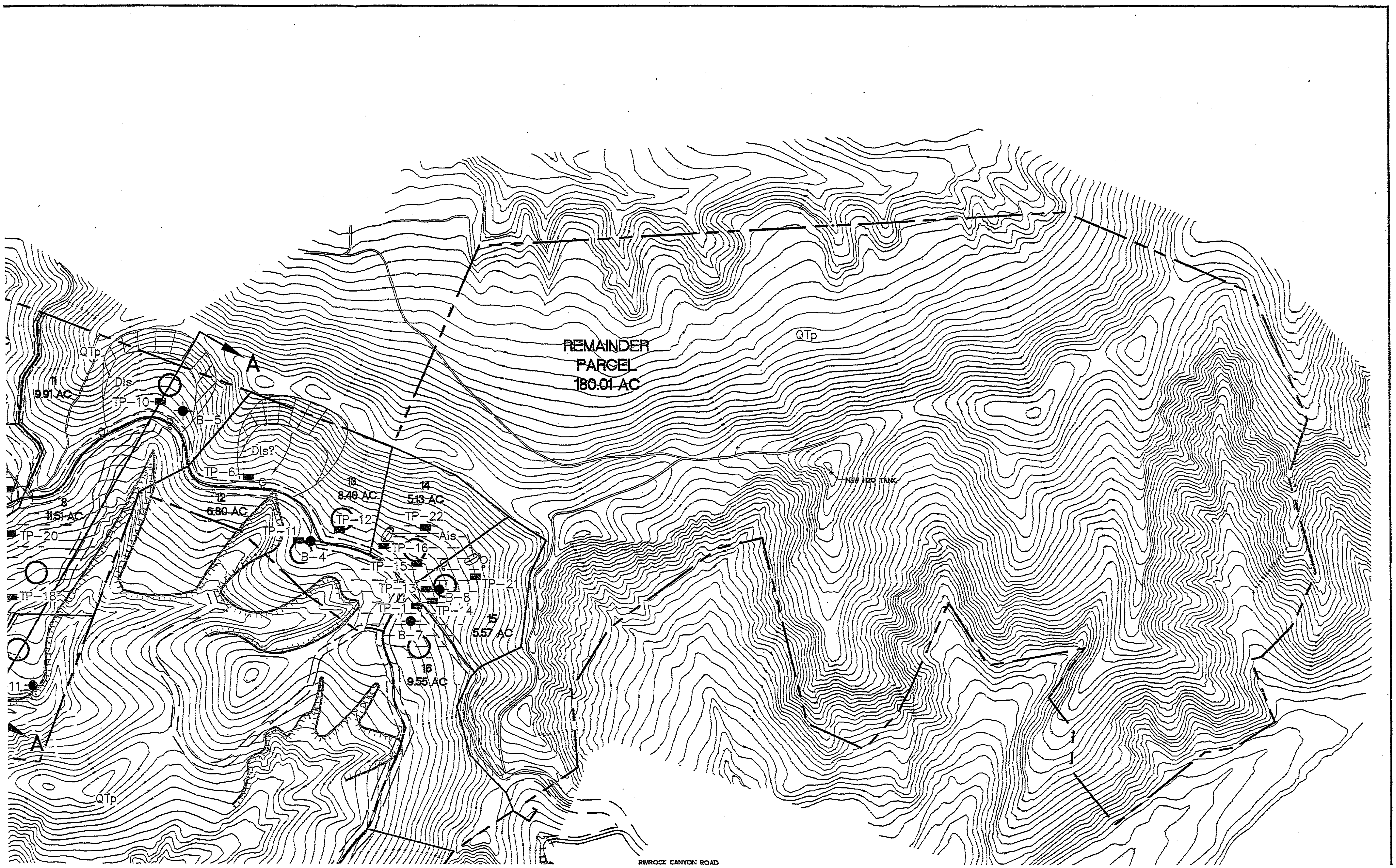
QTp PASO ROBLES FORMATION– SEMICONSOLIDATED, SILTY TO CLAYEY SAND AND SANDY GRAVEL; LOCALLY CEMENTED. GENERALLY OVERLAIN BY 2 TO 5 FEET OF LOOSE, CLAYEY SAND COLLUVIUM (NOT SHOWN)

Als ACTIVE LANDSLIDE– PRESENTLY, SEASONALLY, OR RECENTLY MOVING

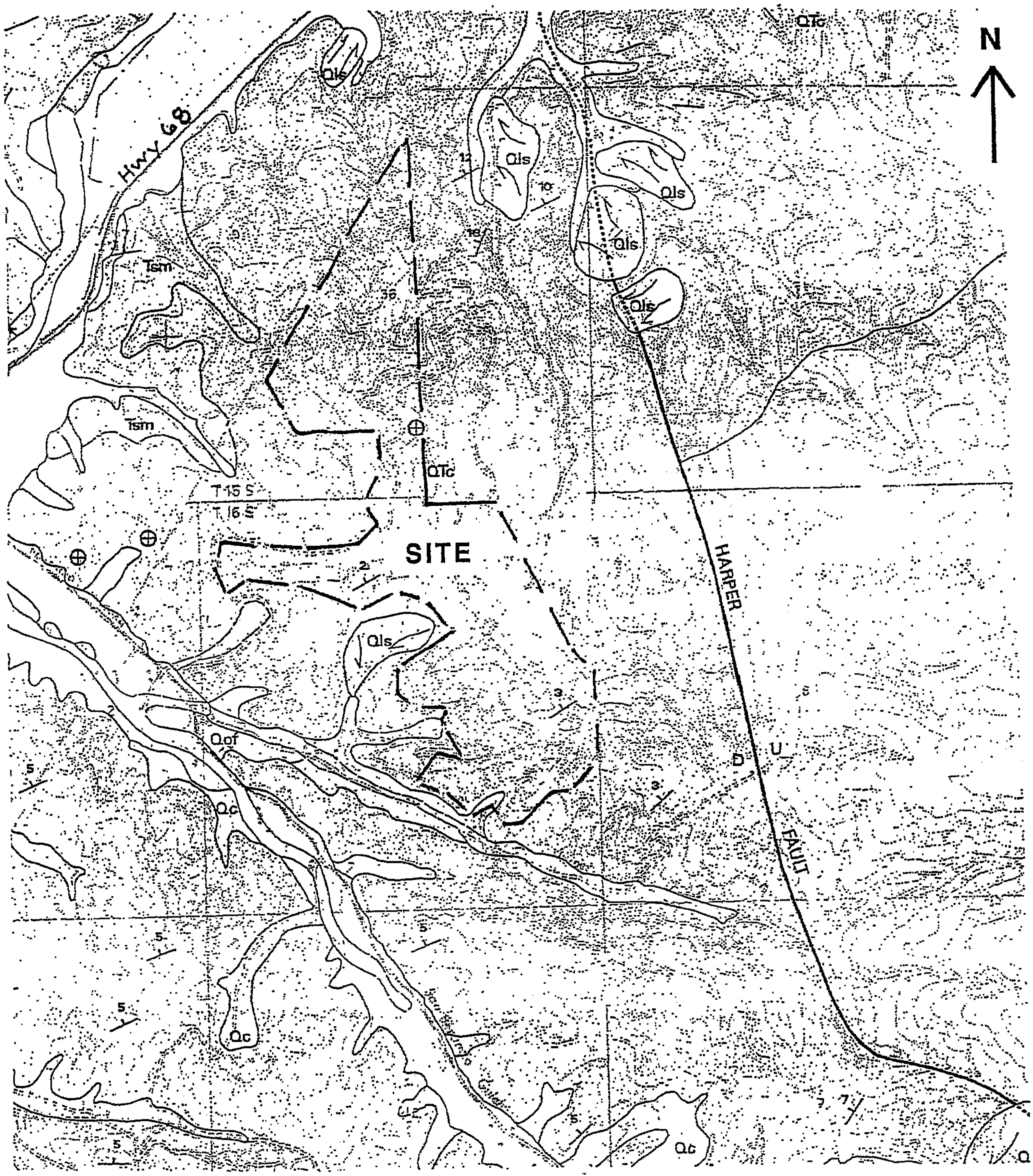
Dls DORMANT LANDSLIDE– PRESENTLY NOT MOVING LANDSLIDE BUT POSSIBLY SUSCEPTIBLE TO REACTIVATION BY ONGOING GEOLOGIC PROCESSES

PROPOSED HOME SITES

EROSION GULLY



REMAINDER
PARCEL
180.01 AC
QTp
A
11
9.91 AC
DIs
TP-10
B-5
DIs?
TP-6
8
11.51 AC
12
6.80 AC
13
8.40 AC
14
5.13 AC
TP-12
TP-22
TP-20
TP-11
B-4
AIs
TP-16
TP-15
TP-21
TP-13
B-8
TP-18
TP-1
TP-14
15
5.57 AC
B-7
16
9.55 AC
NEW H2O TANK
RIMROCK CANYON ROAD

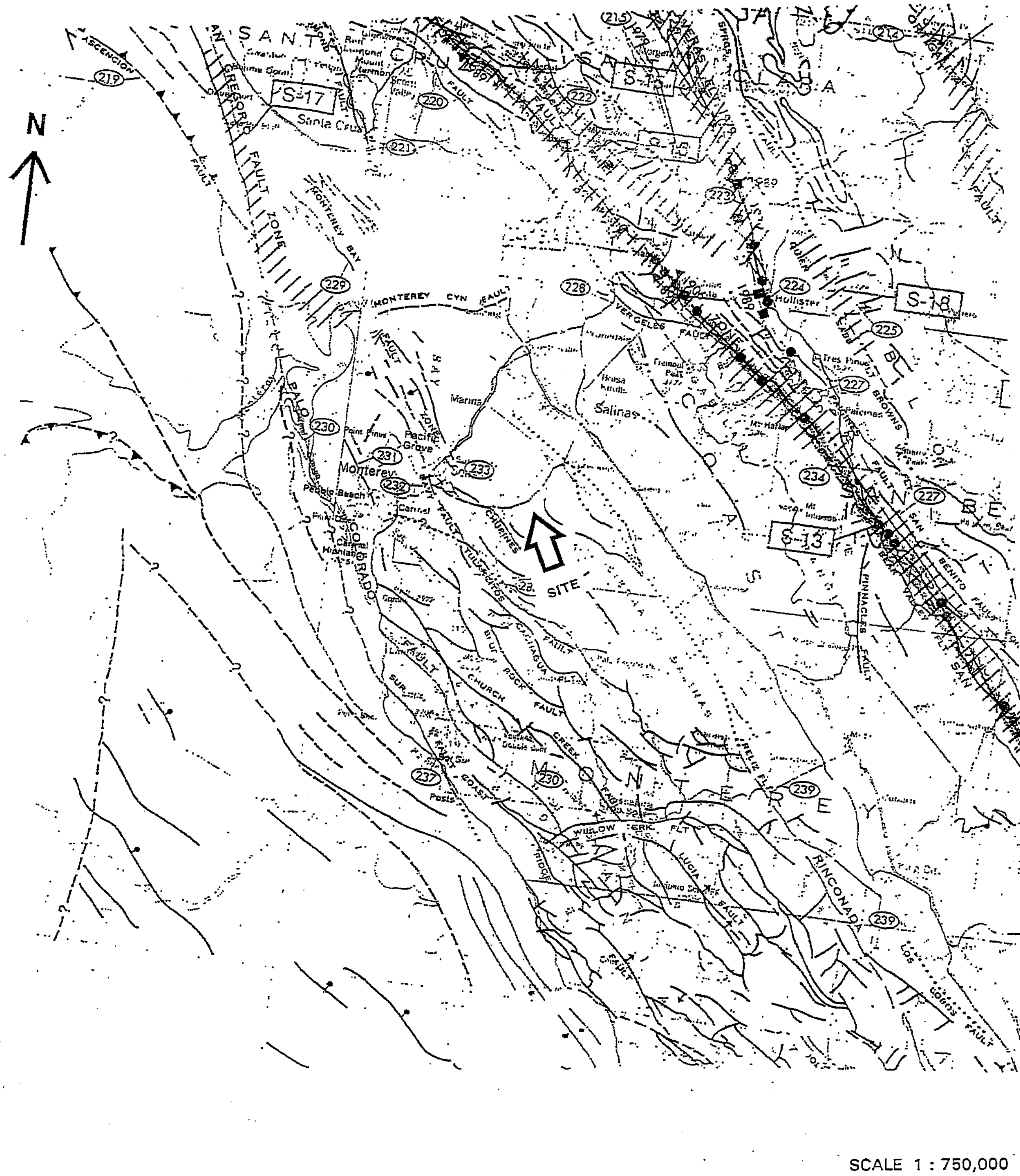


Qls – Quaternary Landslides
Qc – Quaternary Colluvium
Qof – Quaternary Older Fill
Qtc – Paso Robles Formation
Tsm – Tertiary Santa Margarita Sandstone

SCALE 1" = 2000'

Base: USGS Spreckels 7.5' Quadrangle;
Geology from Clark et. al., 2001.

D&M CONSULTING ENGINEERS July, 2001	Regional Geologic Map **Proposed Encina Hills Subdivision** Monterey County, California	FIGURE 3 PROJECT 1892



Source: Fault Activity Map of California and Adj. Areas, DMG, 1994.

D&M CONSULTING ENGINEERS

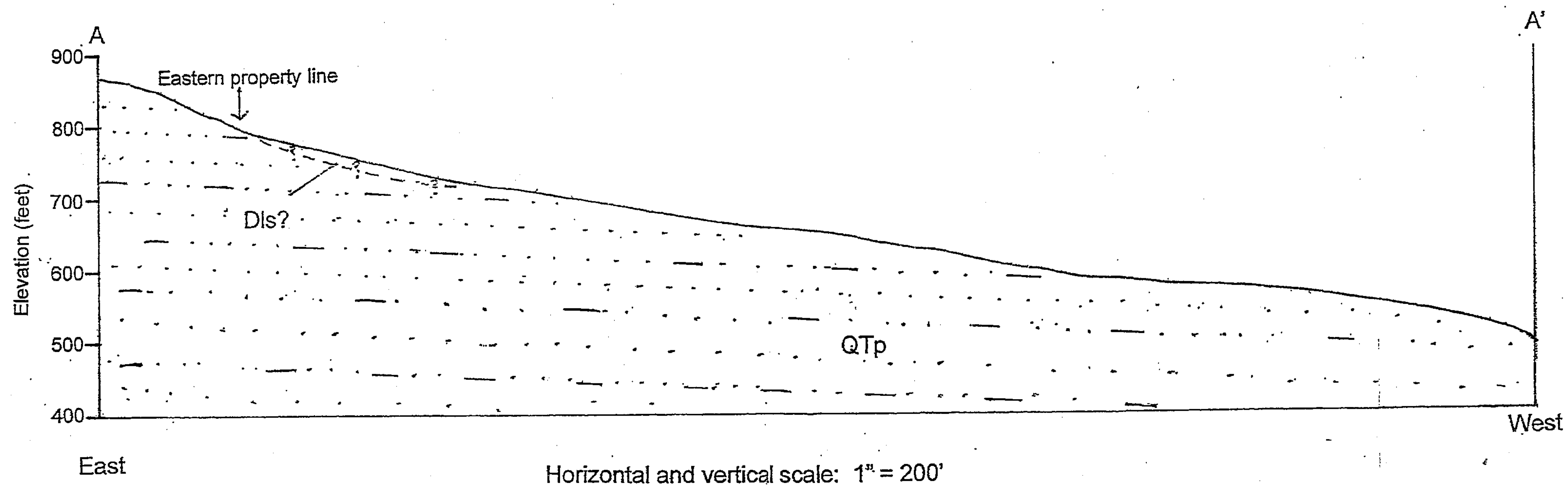
July, 2001

Regional Fault Map

Proposed Encina Hills Subdivision
Monterey County, California

FIGURE
4

PROJECT
1892



Horizontal and vertical scale: 1" = 200'

See Figure 3 for explanation

JUNE 2000

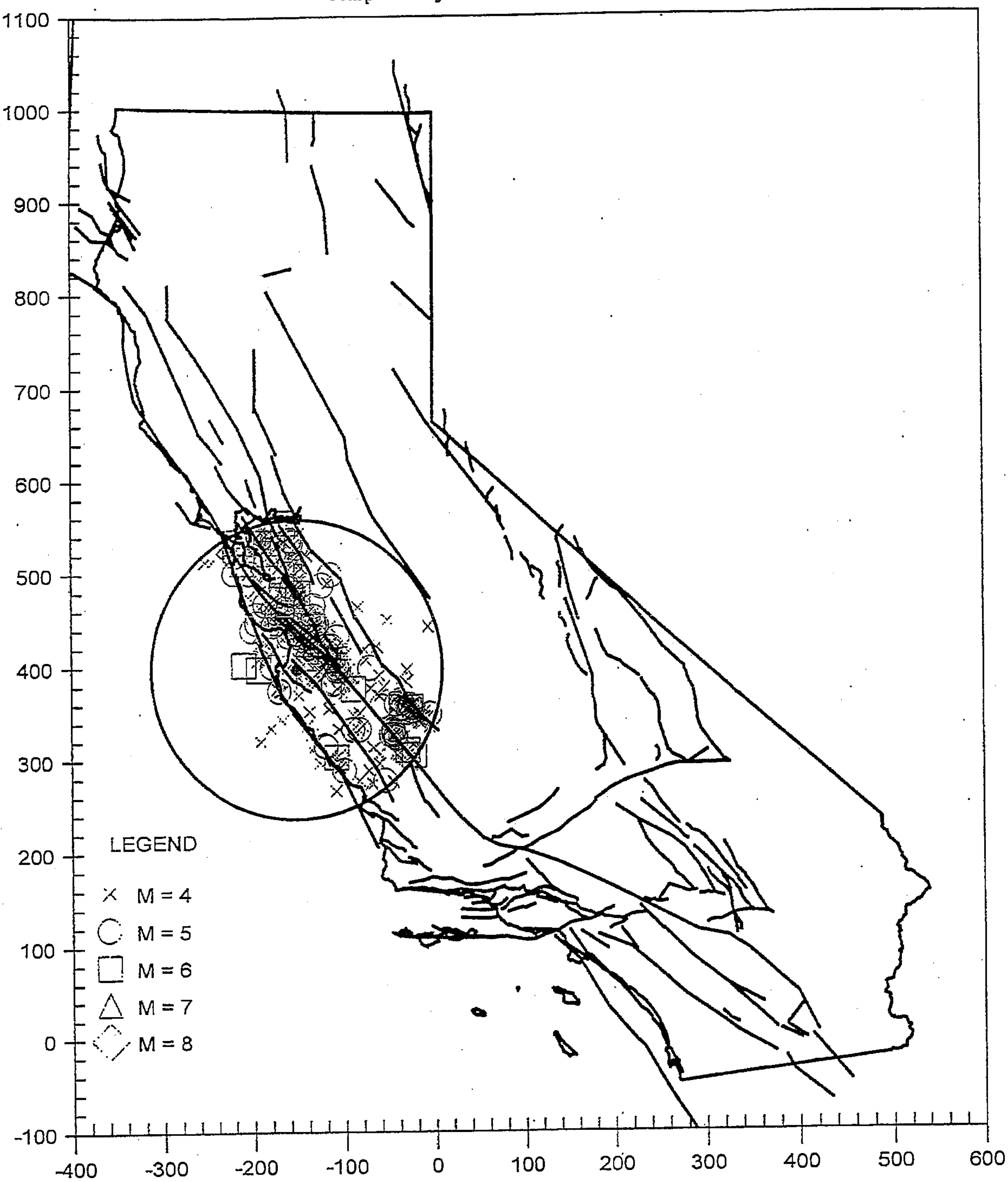
D&M CONSULTING ENGINEERS, INC.

CROSS SECTION A–A'
ENCINA HILLS SUBDIVISION
MEYER ROAD
MONTEREY COUNTY, CALIFORNIA

FIGURE
5
PROJECT
1892

EARTHQUAKE EPICENTER MAP

Harper Canyon



Source: Thomas Blake Computer Services, EQSEARCH, April, 2000.

D&M CONSULTING ENGINEERS

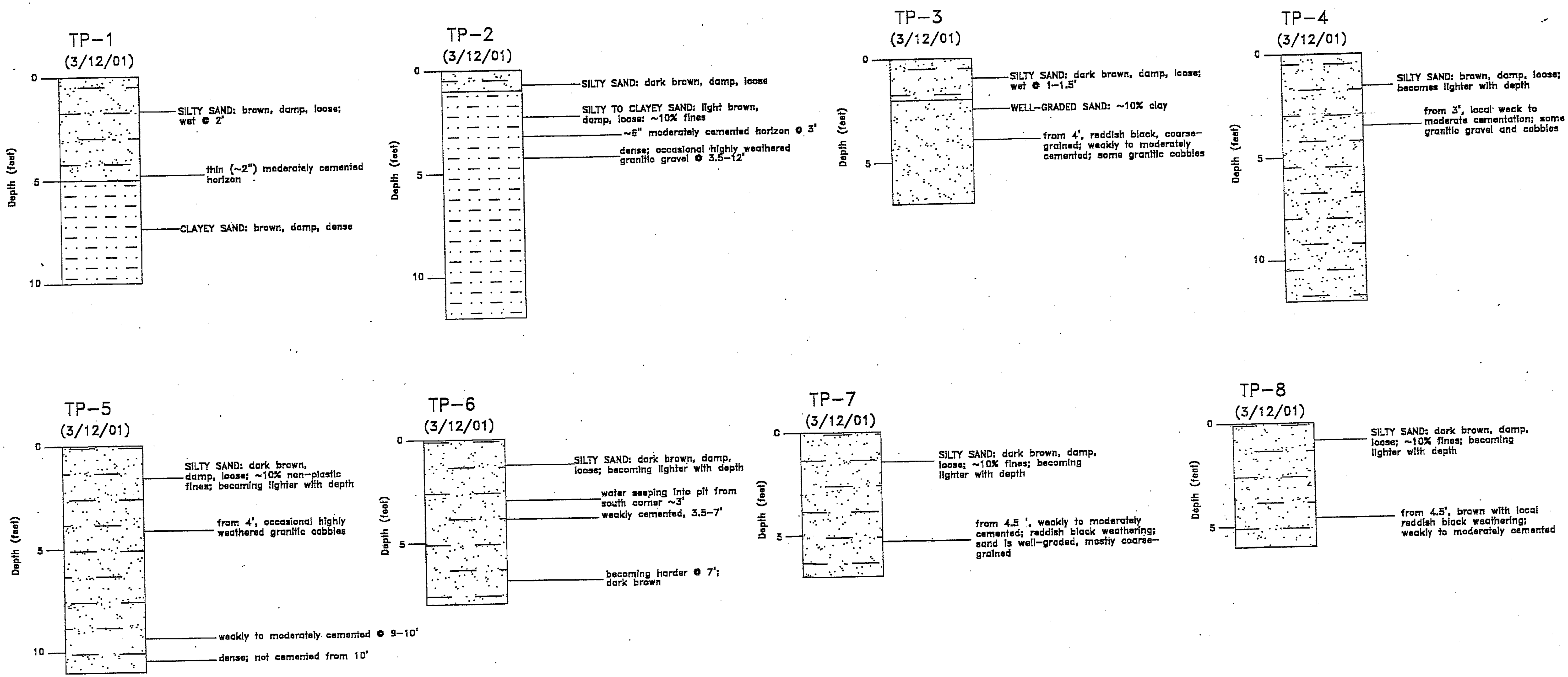
July, 2001

Regional Seismicity Map

Proposed Encina Hills Subdivision
Monterey County, California

FIGURE
6

PROJECT
1892

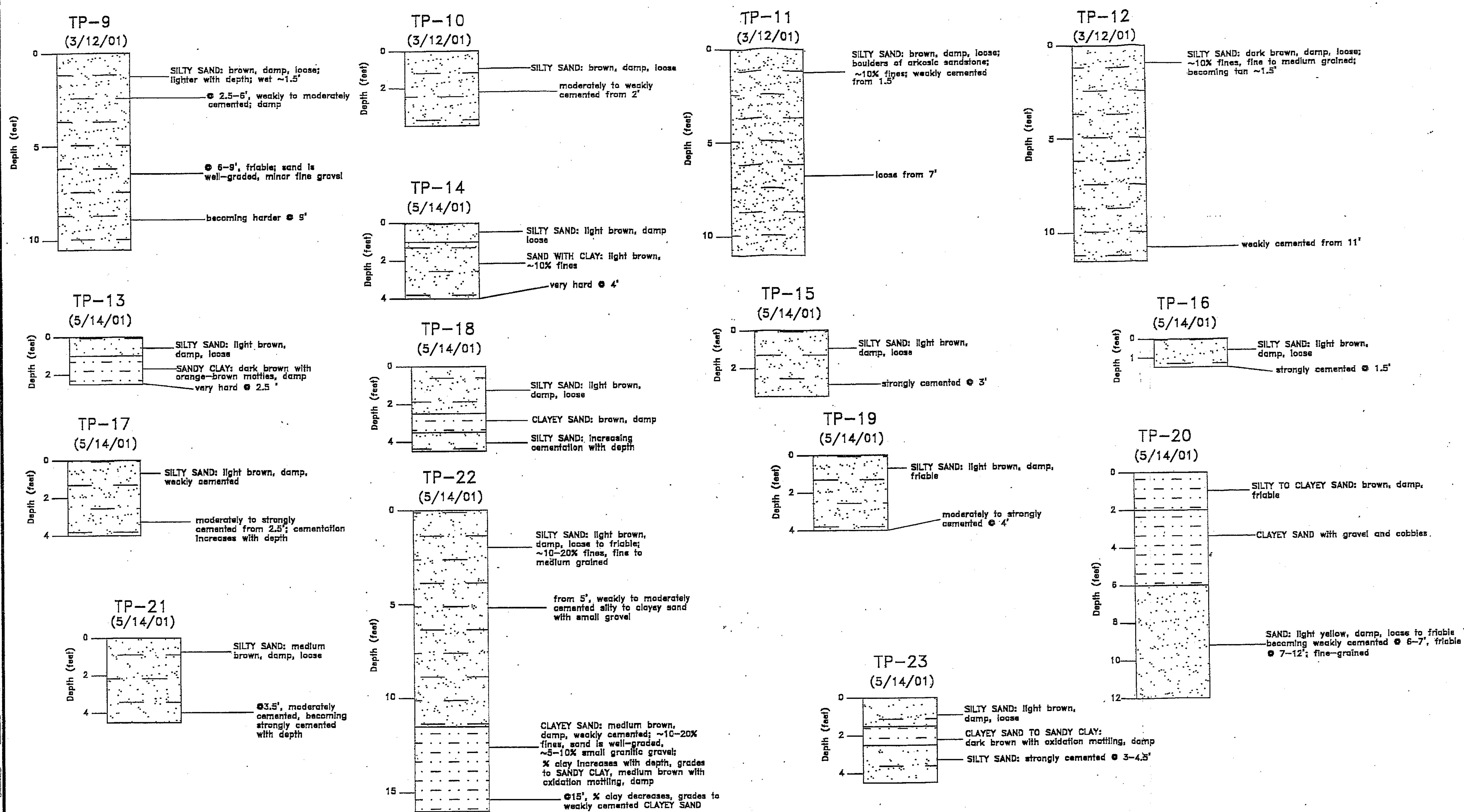


JUNE 2000

D&M CONSULTING ENGINEERS, INC.

EXPLORATORY TEST PITS (TP-1 - TP-8)--GENERALIZED SECTIONS
ENCINA HILLS SUBDIVISION
MEYER ROAD
MONTEREY COUNTY, CALIFORNIA

SHEET
A-1
PROJECT
1892



JUNE 2001

D&M CONSULTING ENGINEERS, INC.

EXPLORATORY TEST PITS (TP-9 – TP-23)—GENERALIZED SECTIONS
ENCINA HILLS SUBDIVISION
MEYER ROAD
MONTEREY COUNTY, CALIFORNIA

SHEET
A-2
PROJECT
1892

e

KEY TO EXPLORATORY BORING LOGS

PRIMARY DIVISIONS			GROUP1 SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of material is larger than No. 200 sieve size	GRAVELS More than half coarse fraction is larger than No.4 sieve	Clean Gravels (less than5% fines*)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravel with fines*	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS More than half 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, silt-sand mixtures, non-plastic fines
			SC	Clayey sand, 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 35		ML	Inorganic silts, clayey silts, rock flour, silty very fine sands
			CL	Inorganic clays of low plasticity, gravelly clay of low plasticity
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit is between35 and 50		MI	Inorganic silts, clayey silts and silty fine sand with intermediate plasticity
			CI	Inorganic clays, gravely clays, sandy clays and silty clays of intermediate plasticity
			OI	Inorganic clays and silty clays of intermediate plasticity
	SILTS AND CLAYS Liquid limit is greater than 50		MH	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soil
			CH	Inorganic clays of high plasticity
			OH	Organic clays and silts of high plasticity
HIGHLY ORGANIC SOILS			Pt	Peat, meadow mat, highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE	200	40	10	4	CLEAR SQUARE SIEVE OPENINGS ¾"	3"	12"
Silts and Clays	Fine	Medium	Coarse	Fine	Coarse	Cobbles	Boulders
	SAND			GRAVEL			

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 – 4
LOOSE	4 – 10
MEDIUM DENSE	10 – 30
DENSE	30 – 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH**	BLOWS/FOOT*
VERY SOFT	0 – ¼	0 – 2
SOFT	¼ – ½	2 – 4
FIRM	½ – 1	4 – 8
STIFF	1 – 2	8 – 16
VERY STIFF	2 – 4	16 – 32
HARD	OVER 4	OVER 32

SYMBOLS

▽	Initial Ground Water Level
▼	Final Ground Water Level
T	Standard Penetration Sampler
M	Modified California Sampler
I	Shelby Pitcher Tube Sampler

NOTES

*BLOWS per FOOT – Resistance to the advancement of the soil sampler-number of blows of a 140-pound hammer falling 30 inches to drive a split spoon sampler.

**Stratification lines on the logs represent the approximate boundary between soil types, and the transition may be gradual.

Modified California Sampler – 2 ½ O.D. (1 7/8 Inch I.D.) sampler

Standard Pennetration Sampler (Teizaghi) – 2 inch O.D. (1 3/8 Inch I.D.) split spoon sampler (ASTM D1586-84).

Shelby/Pitcher Tube Sampler – 3 ½ inch O.D. (3 inch I.D.) CME brand split spoon sampler (5 foot long); advances with augers and provides a 3 foot long continuous core.

BORING LOG

No. B-1

PROJECT: ENCINA HILLS SUBDIVISION | DATE: 3/21/01 | LOGGED BY: ACK

DRILL RIG: MOBILE B-24 | HOLE DIA.: 4" | SAMPLER: SPT

GROUND WATER DEPTH INITIAL: — | FINAL: — | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAYEY SAND: medium brown, damp, medium dense; ~10-20% fines, predominantly fine to medium grained; COLLUVIUM	SC	1										
		2										
			T	30								
		3										
SILTY SAND: light gray-brown, damp, very dense; ~20% fines, mostly fine to medium grained; weakly to moderately cemented; PASO ROBLES FORMATION	SM	4	T	50/4-1/2								
		5										
		6										
		7										
		8										
		9										
~20-30% fines, sand is poorly graded, fine grained, ~5% coarse sand and fine, rounded gravel		10	T	71								
		11										
		12										
		13										
		14										
~10% fines, well graded, ~15-20% rounded shale gravel to 1"; moderately cemented			T	50/6								
		15										
		16										
		17										
		18										
		19										
weakly cemented; minor gravel to ~1/2"		20	T	73								

BORING LOG

No. B-1

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: -- | FINAL: -- | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
	SM	21										
		22										
		23										
poorly graded, fine to medium grained, ~5% coarse sand, no gravel		24	T	50/6								
Bottom of boring @ 24.5'. No ground water encountered.		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

BORING LOG

No. B-2

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: medium brown, damp (wet @ 1-2'); ~20% fines, sand is poorly graded, fine to medium grained, ~5% coarse sand; COLLUVIUM	SM	1										
		2										
SILTY SAND: light brown, damp, dense; ~10-20% fines, minor fine, rounded gravel; weakly cemented; PASO ROBLES FORMATION	SM		T	37								
		3										
very dense		4	T	50/4								
		5										
		6										
		7										
		8										
		9										
			T	50/6								
		10										
		11										
		12										
		13										
		14										
			T	50/6								
		15										
		16										
CLAYEY SAND: light gray-brown, damp, very dense; ~20% fines, poorly graded, medium grained; PASO ROBLES FORMATION	SC	17										
		18										
auger refusal		19										
Bottom of boring @ 19'. No ground water encountered		20										

BORING LOG

No. B-3

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psi)
SILTY SAND: medium brown, damp, dense; ~10-20% fines, well graded; COLLUVIUM	SM	1										
		2										
SILTY SAND: gray-brown, damp, very dense; ~10-20% fines, well graded, moderately cemented; PASO ROBLES FORMATION	SM	3	T	39								
		4	T	50/2								
		5										
		6										
		7										
		8										
predominantly fine to medium grained, ~10-20% coarse sand		9	T	50/3								
		10										
		11										
		12										
		13										
		14										
~10% fines, trace fine, rounded gravel		15	T	77								
Bottom of boring @ 15'. No ground water encountered.		16										
		17										
		18										
		19										
		20										

BORING LOG

No. B-4

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: -- | FINAL: -- | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: medium brown, damp, medium dense; ~10-20% fines, mostly fine grained, ~5-10% coarse sand; COLLUVIUM	SM	1										
		2										
light yellow brown			T	24								
		3										
mottled brown, yellow-brown, and orange-brown, loose; minor sandstone gravel		4										
		5	T	9								
POORLY GRADED SAND: tan, damp, dense; <5% fines, fine grained; PASO ROBLES FORMATION	SP	6										
		7										
		8										
		9										
		10	T	31								
		11										
		12										
		13										
		14										
tan with pink mottles, very dense; ~5-10% low-plasticity fines		15	T	75								
		16										
		17										
		18										
WELL GRADED SAND WITH CLAY: tan, damp, very dense; ~5-10% fines; PASO ROBLES FORMATION	SW-SC	19										
			T	48/6								
		20										

BORING LOG

No. B-4

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
	SW-SC	21										
		22										
		23										
weakly cemented		24	T	50/4-1/2								
Bottom of boring @ 24.5'. No ground water encountered		25										
		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										

BORING LOG

No. B-5

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
WELL GRADED SAND WITH SILT: brown, wet; ~10% fines, predominantly fine to medium grained; COLLUVIUM	SW-SM	1										
		2										
WELL GRADED SAND WITH SILT: brown, damp, very dense; ~10-20% fines; cemented; PASO ROBLES FORMATION	SW-SM		T	60								
		3										
		4	T	50/3-1/2								
~10% fine gravel		5										
		6										
		7										
		8										
		9										
dense; ~5% fine gravel		10	T	45								
		11										
		12										
auger refusal @ 13'		13										
		14										
dense/very dense			T	50								
Bottom of boring @ 14.5'. No ground water encountered.		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. B-6

PROJECT ENCINA HILLS SUBDIVISION | DATE 3/21/01 | LOGGED BY ACK

DRILL RIG MOBILE B-24 | HOLE DIA. 4" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: medium brown, damp; ~10-20% fines, mostly fine to medium grained, minor fine gravel; COLLUVIUM	SM	1										
		2										
SILTY SAND: light brown, damp, very dense; ~20-30% fines, fine to medium grained, minor coarse sand; PASO ROBLES FORMATION	SM	3	T	67								
		4										
minor fine, granitic gravel		5	T	54								
		6										
		7										
		8										
		9										
no gravel		10	T	92								
Bottom of boring @ 10'. No ground water encountered.		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
		20										

BORING LOG

No. B-7

PROJECT ENCINA HILLS SUBDIVISION DATE 5/29/01 LOGGED BY CMP

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SANDY SILT: dark yellowish brown, damp; 55% silt, 45% fine sand	ML	1										
		2										
SANDY CLAY: dark brown, damp, hard; 20-30% fine to medium sand, low plasticity, massive	CL	3										
		4	T	40	1.0							
clay decreases with depth												
SANDY SILTY TO CLAYEY SAND: brown, damp, stiff/dense; 30-60% fine sand (varies), slightly plastic; sandier with depth to 5'	ML-SC	5										
		6										
		7										
		8										
CLAYEY SAND: very dark gray brown, damp, medium dense; 20-30% clay fines, 60-80% fine to coarse sand; clay content varies in sample, locally very sandy; massive	SC	9			3.2	2.5						
		10	T	23								
		11										
		12										
		13										
CLAYEY SAND TO POORLY GRADED SAND: brownish yellow, damp to slightly moist, medium dense; fine to medium sand mixed with low-plasticity clay and as thin beds; crudely bedded overall	SC-SP	14			1.5	1.0						
		15	T	18								
		16										
		17										
		18										
very thin clayey beds to laminae @ 19', containing disseminated fine sand		19			2.25	2.3						
POORLY GRADED SAND	SP	20	T	28								

BORING LOG

No. B-7

PROJECT ENCINA HILLS SUBDIVISION DATE 5/29/01 LOGGED BY CMP

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
POORLY GRADED SAND: yellowish brown, damp, medium dense: 10-15% non-plastic fines, 85-90% fine to coarse sand; well-bedded; slightly moist 18.5-19.5'	SP	21										
		22										
		23										
		24										
dense; <10% fines, sand locally very poorly sorted and clean		25	T	34	<0.5							
		26										
		27										
		28										
		29										
5-10% fines, rare fine gravel; massive to crudely bedded, local graded sand beds with coarse lag sands		30	T	36	1.0							
		31										
		32										
		33										
WELL GRADED SAND: very pale brown, damp, dense; 5% fines, 95% fine to coarse sand; massive to crudely bedded, local coarse lenses; possible reduced plant matter	SW	34										
		35	T	40	0.5							
		36										
drills as gravel		37										
		38										
very dense, ~10% fines, less coarse sand 39-40'; bedded		39										
		40	T	52	1.5							

BORING LOG

No. B-7

PROJECT ENCINA HILLS SUBDIVISION DATE 5/29/01 LOGGED BY CMP

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
	SW	41										
		42										
		43										
POORLY GRADED TO WELL GRADED SAND: yellow, damp, very dense; 5% non-plastic fines, 95% fine to coarse sand; variably bedded, grades in and out, some iron oxide-defined bedding	SP-SW	44										
		45	T	50	0.5							
		46										
		47										
		48										
POORLY GRADED SAND: yellow, damp, dense; ~10% non-plastic fines, fine to medium sand; crudely bedded, locally fine sand beds	SP	49										
		50	T	44	0.7							
Bottom of boring @ 50'. No ground water encountered.		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

BORING LOG

No. B-8

PROJECT ENCINA HILLS SUBDIVISION DATE 5/30/01 LOGGED BY ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: -- FINAL: -- HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: gray-brown, damp, loose; COLLUVIUM	SM	1										
		2										
		3										
CLAYEY SAND: light brown, damp, very dense; ~15-20% fines, sand is well-graded; minor fine, angular granitic gravel; PASO ROBLES FORMATION	SC	4										
		5	T	50/5								
		6										
		7										
		8										
		9										
		10										
locally, small granitic gravel, subangular to rounded			T	68								
		11										
		12										
		13										
SILTY SAND: light gray-brown, damp, very dense; ~30% low-plasticity fines, poorly graded, fine-grained	SM	14										
		15										
Drilling very slow from 15'			T	9/11								
		16										
		17										
		18										
WELL-GRADED SAND WITH CLAY: light yellow-brown, damp, very dense; ~10% fines, predominantly fine to medium-grained, ~10% coarse sand	SW-SC	19										
		20	T	50/6								

BORING LOG

No. B-8

PROJECT ENCINA HILLS SUBDIVISION | DATE 5/30/01 | LOGGED BY ACK

DRILL RIG CME 65 | HOLE DIA. 8" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		21										
		22										
		23										
		24										
~20% fines, predominantly medium to coarse sand		25	T	50/5								
		26										
		27										
		28										
		29										
~10-15% fines		30	T	90								
		31										
		32										
		33										
		34										
POORLY GRADED SAND WITH SILT: light yellow-brown, damp, very dense; ~5-10% fines, fine to medium grained	SP-SM	35	T	50/4								
		36										
		37										
		38										
POORLY GRADED SAND: light yellow-brown, damp, very dense; <5% fines, predominantly fine to medium sand, ~10% coarse sand	SP	39										
		40	T	50/4								

BORING LOG

No. B-8

PROJECT ENCINA HILLS SUBDIVISION | DATE 5/30/01 | LOGGED BY ACK

DRILL RIG CME 65 | HOLE DIA. 8" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: – | FINAL: – | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		41										
		42										
		43										
		44	T	50/5								
Drilling very slow 45-50'		45										
		46										
		47										
CLAYEY SAND: light yellow-brown, damp, very dense: ~15-20% fines, well graded, slightly cemented	SC	48										
		49	T	50/5								
Bottom of boring @ 49'. No ground water encountered.		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

BORING LOG

No. B-9

PROJECT ENCINA HILLS SUBDIVISION | DATE 5/31/01 | LOGGED BY ACK

DRILL RIG CME 65 | HOLE DIA. 8" | SAMPLER SPT

GROUND WATER DEPTH INITIAL: — | FINAL: — | HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
See log for B-3		1										
		2										
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
CLAYEY SAND: medium brown, damp, dense; ~20-30% fines, well-graded sand, ~5% small gravel, occasional larger, completely weathered granitic gravel		18										
		19										
		20										

BORING LOG

No. B-9

PROJECT ENCINA HILLS SUBDIVISION DATE 5/31/01 LOGGED BY ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		21	T	46								
		22										
WELL-GRADED SAND WITH CLAY: medium brown, damp, dense; ~10-15% fines, predominantly medium to coarse, trace fine gravel	SW-SC	23										
		24										
		25										
			T	35								
		26										
		27										
		28										
		29										
		30										
locally moist			T	27								
		31										
		32										
		33										
		34										
		35										
~15-20% fines, ~5% granitic gravel, some completely weathered clasts >1"		36	T	54								
		37										
		38										
WELL GRADED SAND WITH CLAY AND GRAVEL: light brown, damp, very dense; ~10% fines, mostly medium to coarse sand, ~20% well-graded gravel	SW-SC	39										
		40	T	50/6								

BORING LOG

No. B-9

PROJECT ENCINA HILLS SUBDIVISION DATE 5/31/01 LOGGED BY ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		41										
		42										
		43										
SANDY CLAY TO CLAYEY SAND: light gray-brown with local pink mottles, damp, hard/very dense; low-plasticity fines, sand is well-graded Slow drilling 45-50'	SC/ CL	44										
		45										
			T	64								
		46										
		47										
		48										
CLAYEY SAND: light brown, damp, very dense; ~30-40% fines, well-graded, locally up to ~5% fine, granitic gravel	SC	49										
		50	T	50/5								
Bottom of boring @ 50'. No ground water encountered		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

BORING LOG

No. B-10

PROJECT ENCINA HILLS SUBDIVISION DATE 6/1-5/2001 LOGGED BY CMP/ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: — FINAL: — HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
See log for B-1		1										
		2										
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
		11										
		12										
		13										
		14										
		15										
		16										
		17										
		18										
		19										
Drills slow/hard to 20'		20										

BORING LOG

No. B-10

PROJECT ENCINA HILLS SUBDIVISION DATE 6/1-5/2001 LOGGED BY CMP/ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: – FINAL: – HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		21										
Drills very hard, gravel chatter 22 to 23'		22										
		23										
SANDY SILT: brown, damp, hard; non-plastic to very slightly plastic fines, 30% sand; crudely bedded; appears to grade to more clayey silt @24.5'	ML	24										
		25	T	39	4.0							
		26										
		27										
		28										
		29										
SANDY SILT WITH GRAVEL: pale brown, damp, hard; non-plastic fines, 5% coarse gravel; massive, damp	ML	30	T	66	3.5							
		31										
becomes sandier, locally 30-40% fine sand with thin coarse sand lenses Hard drilling 31-33'		32										
		33										
WELL-GRADED GRAVELLY SAND TO WELL-GRADED SAND: very pale brown, damp, very dense; 5-10% non-plastic fines, 60-90% fine to coarse sand, up to 20% fine gravel; massive; contains fine to coarse gravel lenses	SW	34	T	50/6								
		35										
		36										
		37										
very hard drilling		38										
CLAYEY SAND: medium brown, damp, dense; ~30-40% fines, well-graded	SC	39										
Auger refusal @ 40'		40			>4.5							

BORING LOG

No. B-10

PROJECT	ENCINA HILLS SUBDIVISION	DATE	6/1-5/2001	LOGGED BY	CMP/ACK
DRILL RIG	CME 65	HOLE DIA. 8"	SAMPLER	SPT	
GROUND WATER DEPTH INITIAL:	--	FINAL:	--	HOLE ELEVATION:	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
	SC		T	44								
Bottom of boring @ 40.5'. No ground water encountered		41										
		42										
		43										
		44										
		45										
		46										
		47										
		48										
		49										
		50										
		51										
		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

BORING LOG

No. B-11

PROJECT: ENCINA HILLS SUBDIVISION	DATE: 6/5/01	LOGGED BY: ACK	
DRILL RIG: CME 65	HOLE DIA. 6"	SAMPLER: SPT	
GROUND WATER DEPTH INITIAL: –	FINAL: –	HOLE ELEVATION:	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
SILTY SAND: light brown, damp, loose; ~30% fines, predominantly fine to medium grained, some coarse sand; COLLUVIUM	SM	1										
		2										
		3										
		4										
Drilling harder @ 5'		5										
CLAYEY SAND: light brown, damp, very dense; ~20% fines, mostly fine to medium grained, minor fine gravel; PASO ROBLES FORMATION	SC	6										
			T	54	2.0							
		7										
		8										
		9										
		10										
~10-20% fines, sand is well graded		11										
			T	64	3.0							
		12										
SILTY SAND: light yellow-brown, damp, very dense; ~30% fines, mostly fine to medium grained, some coarse sand, ~5% fine gravel	SM	13										
		14										
		15										
		16			2.0							
CLAYEY SAND: light brown, damp, very dense; ~30-40% fines, well-graded	SC		T	80	>4.5							
		17										
Drilling becomes harder		18										
POORLY GRADED SAND: light yellow-brown, damp, very dense; ~5% fines, mostly fine to medium grained, trace coarse sand	SP	19										
		20										

BORING LOG

No. B-11

PROJECT	ENCINA HILLS SUBDIVISION	DATE	6/5/01	LOGGED BY	ACK
DRILL RIG	CME 65	HOLE DIA.	6"	SAMPLER	SPT
GROUND WATER DEPTH INITIAL:	--	FINAL:	--	HOLE ELEVATION:	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		21										
WELL-GRADED SAND WITH CLAY: light brown, damp, very dense; ~10-20% fines	SW-SC		T	79	1.0							
		22										
		23										
		24										
		25										
very hard drilling		26			3							
CLAYEY SAND TO SANDY CLAY: light brown, damp, dense-very dense/hard;	SC/CL		T	55	>4.5							
		27										
		28										
		29										
		30										
		31			2.5							
mostly fine to medium grained sand; trace fine gravel			M	50	1.5							
		32										
		33										
		34										
light brown with darker brown beds, very dense ~20% fines, crudely laminated		35										
		36										
			T	60	>4.5							
easier drilling		37										
WELL-GRADED SAND WITH CLAY: light brown, damp, very dense; ~10-20% fines, mostly medium to coarse, trace fine gravel	SW-SC	38										
		39										
		40										

BORING LOG

No. B-11

PROJECT ENCINA HILLS SUBDIVISION DATE 6/5/01 LOGGED BY ACK

DRILL RIG CME 65 HOLE DIA. 8" SAMPLER SPT

GROUND WATER DEPTH INITIAL: — FINAL: — HOLE ELEVATION:

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
		41	T	50/6	2.5							
		42										
CLAYEY SAND: mottled light to medium brown, damp, dense; ~30% fines, well-graded, minor charcoal	SC	43										
		44										
		45										
		46										
			T	42	4.5							
		47										
		48										
		49										
		50										
very dense		51										
			T	68	4.5							
Bottom of boring @ 51.5'. No ground water encountered.		52										
		53										
		54										
		55										
		56										
		57										
		58										
		59										
		60										

BORING LOG

No. B-12

PROJECT	ENCINA HILLS SUBDIVISION	DATE	6/6/01	LOGGED BY	ACK
DRILL RIG	CME 65	HOLE DIA.	6"	SAMPLER	SPT
GROUND WATER DEPTH INITIAL:	--	FINAL:	--	HOLE ELEVATION:	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
CLAYEY SAND: light brown, damp; ~40% fines, well-graded; COLLUVIUM	SC	1										
		2										
		3										
		4										
WELL-GRADED SAND WITH CLAY: mottled light and medium brown, damp, very dense; ~10-15% fines, ~15% small, subangular to rounded granitic gravel; PASO ROBLES FORMATION	SW-SC	5										
		6	T	50/3	2.75							
		7										
		8										
CLAYEY SAND: light gray-brown with local darker brown mottles, damp, very dense; ~20-30% fines, sand is well graded, trace fine gravel	SC	9										
		10										
		11	T	58	2.5							
		12										
		13										
		14										
light yellow-brown		15										
		16	T	54	3.0							
		17										
		18										
dense; ~10-20% fines, occasional subangular granitic gravel to ~1"		19										
		20										

BORING LOG

No. B-12

PROJECT	ENCINA HILLS SUBDIVISION	DATE	6/6/01	LOGGED BY	ACK
DRILL RIG	CME 65	HOLE DIA. 8"	SAMPLER	SPT	
GROUND WATER DEPTH INITIAL:	–	FINAL:	–	HOLE ELEVATION:	

DESCRIPTION	SOIL TYPE	DEPTH	SAMPLE	BLOWS PER FOOT	POCKET PEN (tsf)	TORVANE (tsf)	LIQUID LIMIT	WATER CONTENT	PLASTIC LIMIT	DRY DENSITY (pcf)	FAILURE STRAIN (%)	UNCONFINED COMPRESSIVE STRENGTH (psf)
			T	44								
		21										
		22										
		23										
very dense; ~30% fines, charcoal @ 24.5'		24										
WELL-GRADED SAND WITH CLAY: light yellow-brown, damp, very dense; ~10% fines, mostly medium to coarse	SW-SC	25			>4.5							
			T	64	1.25							
Bottom of boring @ 25.5'. No ground water encountered.		26										
		27										
		28										
		29										
		30										
		31										
		32										
		33										
		34										
		35										
		36										
		37										
		38										
		39										
		40										