ADDENDUM NO. 2

To the Contract Documents for:

Menlo Park City School District

Oak Knoll Elementary School & Laurel Elementary School

ELC Playgrounds
Bid #004-2019-03

June 21, 2019

This addendum shall supersede all previously issued specifications, addenda and drawings. All other conditions remain unchanged. The following clarifications, changes, modifications, corrections and/or additions as set forth herein shall apply to the above documents and shall be made a part thereof and shall be subject to all the requirements thereof as though originally specified and/or shown.

This addendum consists of 1 page, plus attachments.

Attachments:
Bid Announcement: *(None)*
Project Manual: Geotechnical Report dated June 2007 (60 pages)
Drawing Sheets: *(None)*
Clarifications: Questions and Responses (1 page)

CHANGES TO THE BID ANNOUNCEMENT: *(None)*

CHANGES TO THE PROJECT MANUAL:

Item No 1.1
Reference: Geotechnical Report dated June 2007
Description: **INSERT** the attached Geotechnical Report dated June 2007 (60 pages).

CHANGES TO THE DRAWINGS: *(None)*

CLARIFICATIONS:

Item No 3.1
Reference: Bid Clarifications
Description: Refer to the attached Clarifications (1 page).

END OF ADDENDUM #2

Clarification:	**OAK KNOLL ELEMENTARY SCHOOL:**
1	Sheet L4.1 Detail #3 indicates 30" of non-expansive fill per Geotechnical Report, typical. Please clarify. Also no Geotechnical report is included.
2	Sheet L4.1 Detail #4 & 5 states 4" perforated pipe 20' on center max. Please clarify 20' on center max.
	LAUREL ELEMENTARY SCHOOL:
1	Per L1.2, drawings call out for artificial turf under play house. Please clarify they want turf and not safety surface.

Solution and/or Suggestion:	
	OAK KNOLL ELEMENTARY SCHOOL:
1	Geotech Report from 2007 included. Native soils in this area tend to be expansive - please import non-expansive soil as indicated in the attached report.
2	Disregard that note. Install perf pipe as shown on Conceptual Grading and Drainage Plan
	LAUREL ELEMENTARY SCHOOL:
1	Artifical turf called out and detailed on plans is approved for use under play structures.

GEOTECHNICAL & ENGINEERING GEOLOGIC INVESTIGATION OAK KNOLL ELEMENTARY SCHOOL 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA

THIS REPORT HAS BEEN PREPARED FOR:

MENLO PARK CITY SCHOOL DISTRICT

ATTN: MR. AHMAD SHEIKHOLESLAMI

181 ENCINAL AVENUE

ATHERTON, CALIFORNIA 94027

JUNE 2007

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

June 19, 2007
Project No. 598-3R1

Menlo Park City School District
Attn: Mr. Ahmad Sheikholeslami
181 Encinal Avenue
Atherton, California 94027

RE: GEOTECHNICAL & ENGINEERING GEOLOGIC INVESTIGATION, MENLO PARK CITY SCHOOL DISTRICT OAK KNOLL ELEMENTARY SCHOOL, 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA

Ladies and Gentlemen:

We are pleased to present the results of our geotechnical engineering and engineering geologic investigation relating to design and construction of the proposed improvements to the Oak Knoll Elementary School campus located at 1895 Oak Knoll Lane in Menlo Park, San Mateo County, California. This report summarizes the results of our field, laboratory, geologic, and geotechnical engineering work, and presents conclusions and recommendations relating to the engineering geologic and geotechnical engineering aspects of the design and construction of the proposed improvements.

Our report is contingent upon our review of the project plans and our observation and testing, as necessary, of the foundation, earthwork, and drainage aspects of the construction.

If you have any questions concerning our investigation, please call.

Very truly yours,

MURRAY ENGINEERS, INC.

William P. Carter, E.I.T.
Staff Engineer

Mark F. Bauman, C.E.G. 1787
Principal Engineering Geologist

John Stillman, G.E 2523, C.E.G 1868
Principal Geotechnical Engineer

CTP:WPC:MFB:JAS

Copies: Addressee (6)

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INTRODUCTION

This report contains the results of our geotechnical engineering and engineering geologic investigation relating to design and construction of the proposed improvements to the Oak Knoll Elementary School campus located at 1895 Oak Knoll Lane in Menlo Park, California. The project location is indicated on the Vicinity Map, Figure A-1. The purpose of our investigation was to evaluate the subsurface conditions on the site; to assess the engineering geologic hazards potentially affecting the site and the proposed improvements; and to provide geotechnical design criteria and recommendations for the school project improvements.

Project Description

Based on our review of the proposed improvement plans, we understand the Oak Knoll Elementary School Campus project will include construction of a new two-story multi-use building in the northeast corner of the property along Oak Avenue. The first floor of the new building will include a multi-use room with an indoor basketball court and stage surrounded by five classrooms, a kitchen, a music room, and an art room. The second floor of the new building will include five classrooms, two offices, and a work room. In addition, the existing multi-use building at the western corner of the site will undergo modernization including a minor footprint expansion to incorporate a new learning center and conversion of existing building space to accommodate a new conference room and administration storage area. Other site improvements will include a new covered lunch patio area adjacent to the new multi-use building, a new parking area in the northeast corner of the site, covered walkways, a new play field in the southern corner of the site, and various landscaping improvements. The layout of the existing and proposed construction at the Oak Knoll Elementary School Campus is shown on Figure A-2, Site Plan & Engineering Geologic Map.

Scope of Services

We performed the following services in accordance with our agreement dated November 16, 2006:

- Reviewed aerial photographs, and geologic and seismic conditions in the site vicinity to evaluate the geologic hazards that could potentially impact the school site and the proposed improvements
- Reviewed a geotechnical investigation report and a seismic evaluation report prepared by Jo Crosby and Associates for previously proposed additions to the school (these reports were prepared and signed by our principal geotechnical engineer, Mr. John A. Stillman, G.E., C.E.G., while employed by Jo Crosby and Associates in the late 1990s)
- Performed a reconnaissance of the school property in the area of the proposed improvements
- Explored the subsurface conditions by advancing, sampling, and logging six exploratory borings in the vicinity of the proposed construction
- Performed laboratory analyses and testing on selected soil samples for soil classification and to evaluate engineering properties of the subsurface materials
- Performed engineering geologic analyses to evaluate site-specific engineering geologic hazards and their potential impact on the proposed improvements
- Performed geotechnical engineering analyses to develop geotechnical engineering design criteria for the proposed improvements
- Prepared this report presenting a summary of our investigation and our engineering geologic and geotechnical engineering conclusions and recommendations relating to the project

GEOLOGIC & SEISMIC CONDITIONS

Geologic Overview

The Oak Knoll School property is situated along the western margin of the Santa Clara Valley, a broad, sediment-filled basin bounded on the southwest by the Santa Cruz Mountains and on the northeast by the Diablo Mountain range. Based on the U.S. Geological Survey topographic map of the Palo Alto 7.5-Minute Quadrangle, the site is located at 37.431 North Latitude and 122.191 West Longitude, immediately west of San Francisquito Creek at an approximate elevation of 120 feet above mean sea level (see Figure A-1).

According to the Geologic Map of the Palo Alto and Part of the Redwood Point 7½-Minute Quadrangles (Pampeyan, 1993), the site is located in an area underlain by Holocene (?) age (younger than 11,000 years old) and Pleistocene age (approximately 11,000 to 1.6 million years old) stream terrace deposits (Qst). This material is generally described as poorly sorted, poorly to weakly consolidated, flat-lying deposits of gravel, sand, and silt deposited along San Francisquito Creek. Younger alluvium is mapped within the San Francisquito Creek channel and a broad zone of coarse-grained alluvium is mapped along the margins of the channel to the northeast of the site. The coarse-grained alluvium likely is inter-fingered with the stream terrace deposits, which underlie the site. A copy of the relevant portion of the geologic map is presented on Figure A-3, Vicinity Geologic Map.

According to the map, at depth, the stream terrace deposits are underlain by bedrock of the upper to lower Eocene age (approximately 34 to 55 million years old) Whiskey Hill formation and unnamed upper Crustaceous age (65 to 100 million years old) shale, both of which are mapped along the San Francisquito Creek channel approximately 150 to 200 feet east of the site. The Whiskey Hill formation is generally described as pale yellowish-orange to pale yellowish-brown silty claystone with interbeds of poorly to very well cemented, coarse-grained feldspathic sandstone, glauconitic sandstone, and tuffaceous siltstone. The claystone to sandstone ratio is estimated to be about 3:1. The unnamed shale is generally described as pale-olive to grayish-olive micaceous shale and silty shale with some thin limey lenses. According to the map, the Whiskey Hill formation is in faulted contact with the unnamed shale, which is depicted on the map as an approximately 600-foot wide, west-trending, lenticular body, bounded by faulted contacts. An expanded discussion of local faulting is provided in the Faulting and Seismicity section below. The Whiskey Hill formation and the unnamed shale are underlain by Franciscan Complex basement rock, which according to the geologic map is encountered at an elevation of approximately -90 feet (approximately 210 feet below the ground surface) (see Figure A-3).

Landsliding & Liquefaction

According to the State of California Seismic Hazard Zones Map for Palo Alto Quadrangle (CGS, 2006), the site is not located in an area considered susceptible to earthquake-induced landsliding or liquefaction (see Figure A-4, State Seismic Hazard Zones Map). Based on the preliminary maps prepared by the USGS for the Association of Bay Area Governments (Knudsen and others, 2000), the site is located in an area classified as being moderately susceptible to liquefaction. The younger alluvium mapped within the San Francisquito Creek channel, beyond the limits of the property is classified as being very highly susceptible to liquefaction. A detailed discussion of the liquefaction potential at the site is provided in the Liquefaction Analysis section, below.

Faulting & Seismicity

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. There are three major faults that trend in a northwest direction through the Bay Area, which have generated about 12 earthquakes per century large enough to cause significant structural damage. The faults along which these earthquakes occur are part of the San Andreas Fault system, a right-lateral, strike-slip fault system that extends for at least 700 miles along the California Coast, and primarily includes the San Andreas, Hayward, and Calaveras faults. The San Andreas fault is located approximately 5.7 kilometers southwest of the site. The Hayward and Calaveras faults are located approximately 21 and 29 kilometers northeast of the site, respectively (see Figure A-5, Bay Area Fault Map).

Seismologic and geologic experts convened by the U. S. Geological Survey concluded that there is a 62 percent probability for at least one "large" earthquake of magnitude 6.7 or larger in the Bay Area, before the year 2032. They also maintain that there could be more than one earthquake of this magnitude, and that numerous "moderate" earthquakes of about magnitude 6 are probable before the year 2032. The San Andreas fault is estimated to have a 21 percent probability of producing a magnitude 6.7 or larger earthquake by the year 2032. The Hayward fault is estimated to have a 27 percent probability of producing a similar size earthquake during the same time period (WGCEP, 2003).

The San Andreas fault has a regional trend of approximately N34ºW; however, to the southwest of the site, where the fault cuts through the Santa Cruz Mountains, the fault strikes approximately 10 degrees more to the west, forming a restraining bend. This restraining bend has resulted in a zone of compression along the east side of the Santa Cruz Mountains and the formation of a system of reverse and right-reverse faults within the eastern foothills and the alluvial plain adjacent to the foothills (Angell and others, 1997). Although not zoned as active faults by the State of California, thrust faults of this type have become a topic of significant concern as potential earthquake sources. Blind thrust faults or thrust faults that are buried beneath the ground surface have been recognized as a source of major earthquake events over the past several decades, including the 1983 Coalinga and 1994 Northridge earthquakes.

According to Pampeyan (1993), concealed (buried), bifurcated traces of the Pulgas Fault are located along the north and south sides of the Oak Knoll School property. These traces are mapped on the basis of an historic exposure of an unnamed Cretaceous shale in San Francisquito Creek to the east of the site. Pampeyan maps the southern trace as a faulted contact between the unnamed shale to the north and Whiskey Hill formation to the south. The northern trace is mapped as a buried trace near a depositional contact between the unnamed shale to the south and coarse-grained alluvium to the north. Both traces have a trend of approximately N85W in the area of the school property. (see Figure A-3). We note that earlier mapping by Graham and Church (1963), the authors who identified this fault, depict the trace as a single strand (not a bifurcated trace) with a trend of approximately N40W. Approximately 9,000 feet northwest of the site and approximately 11,500 feet to the southeast, Pampeyan maps this fault trace as a single strand trace concealed beneath the lower Pleistocene and upper Pliocene age (approximately 11,000 to 3.6 million years old) Santa Clara formation bedrock, suggesting an inactive fault with the latest fault movement occurring more than 11,000 years ago.

Seismotectonics immediately southeast of the Oak Knoll School property were evaluated by Kovach and Page (1995). Kovach and Page described a series of generally northwest-trending faults, both observed and inferred, extending along the base of the foothills along the west side of the Stanford University campus, including the Willow Road Bridge fault, as well as the Stock Farm monocline. The Willow Road Bridge fault was apparently exposed in the excavation for the bridge abutment in 1955. The fault was described as follows:

> *"…an ill-defined zoned of intense shearing several yards wide in argillaceous material. Definitive planes denoting strike and dip of the zone as a whole were absent, but multiple shears were generally steep."*

The Stock Farm monocline is described as a "subtle, northwest-southeast-trending topographic declivity," extending from Page Mill Road to the northeast to San Francisquito Creek, believed by the authors to be related to uplift along a southwest-dipping blind thrust fault. According to Kovach and Page, while they did not observe compelling surficial evidence to indicate that these features had experienced Holocene movement, microseismic events (between magnitude 0.8 and 3.5) recorded in the Stanford area between the 1970 and 1992 suggest that minor slip may occur intermittently at depth along unspecified reverse faults in the area. Focal plane solutions suggest an inclined zone of tectonic activity through the area dipping approximately 40 to 55 degrees to the southwest, toward the San Andreas fault. Kovach and Page attributed the lack of surficial expression of faulting to the possibility that surficial erosion is exceeding the very low strain rate, which was theorized to result in less than 1 millimeter of uplift per year along the monoclinal feature. Kovach and Page indicated that because of the limited length of the local faults, they may be capable of producing an earthquake with a magnitude of up to approximately 5.5. Although they indicated that confirmation is required, because the Willow Road Bridge fault does not appear to offset stream terrace deposits, Kovach and Page discount the Willow Road Bridge fault as a possible seismic source. Kovach and Page conclude that the San Andreas and Hayward faults provide a greater seismic risk in the area than the local fault features discussed.

Quaternary deformation in the foothills west of Palo Alto, California was evaluated by Angell and others (1997) by identification and classification of Quaternary surficial deposits, interpretation of aerial photographs, and field mapping. Specifically, Angell and the other investigators performed extensive geologic mapping of young terrace deposits along San Francisquito Creek in an effort to define the style, location, and rate of quaternary deformation across the frontal thrust belt system. The initial work by Angell and others (1997) combined with later investigations by Hansen and others (2001) and Bullard and others (2004), ultimately led to the identification of a broad zone of Quaternary deformation mapped as the Stanford fault.

As part of their investigation, Angell and others (1997) prepared a regional geologic cross-section from Woodside to Palo Alto, which is reproduced as Figure A-6, Regional Geologic Cross-Section (Angel and others, 1997). The section depicts the local geology and faulting and includes microseismicity data referenced by Kovach and Page (1995). Based on age estimates and vertical separation in alluvial terraces, Angell and others estimated an uplift rate across the Pulgas and Stanford faults of 0.7 to 1.6 meters per thousand years. Given the mapped length of the Pulgas-Stanford fault system of 16 kilometers, Angell and others concluded that the maximum credible earthquake for the system is M6.2; however, they indicated that coseismic secondary faulting during large magnitude earthquakes on the San Andreas fault may account for all of the uplift, precluding the need for an independent large magnitude earthquake within the Pulgas-Stanford fault system.

Bullard and others (2004) expanded the previous investigators work by performing more detailed measurements of vertical deformation in alluvial terraces and correlating terrace ages with archaeological data. Immediately east of San Francisquito Creek, they mapped the Stanford fault zone as a right-stepping, *en echelon* monocline. An extension of this fault zone passes immediately north of the Oak Knoll School property. The Pulgas fault was mapped approximately 400 meters southwest of the site. The results of their data are presented on Figure A-7, Local Geologic Map (Bullard and others, 2004). Bullard and others concluded that while latest Pleistocene terraces are folded across the Stanford fault zone, they do not appear to be displaced across the Pulgas fault, suggesting that the Pulgas fault is an inactive feature. Based on their measurements and age correlations, Bullard and others concluded that the uplift rate across the Stanford fault zone is approximately 0.6±0.05 mm/yr.

Aerial Photographs

Three sets of stereographic aerial photographs from 1939, 1963, and 1973 were reviewed to evaluate the geomorphic expression of local faulting in the area of the Oak Knoll School property. In all three sets of photographs, the school property is easily identifiable based on natural and cultural features.

In the 1939 photographs, the Oak Knoll School and Oak Knoll Lane had not yet been constructed; however, Vine Street to the southwest of the school property is present. There appears to be a subdued, northwest-trending topographic rise to the southwest of the school property near the Alameda de las Pulgas, which is consistent with the location of the Pulgas fault mapped by Angell and others (1997) and Bullard and others (2004). Open, dry grassland is located immediately northwest of the school site and to the southeast, on the opposite side of San Francisquito Creek. The grassland is uniform and not interrupted by tonal or vegetal changes. In the 1963 photographs, the school is present and most of the area is developed. In both the 1963 and 1973 photographs, the topographic rise to the southwest of the school site is discernable. There is no obvious evidence for topographic, vegetal, or tonal lineaments suggestive of past faulting trending towards or through the school site in any of the sets of photographs, which were reviewed.

Flooding

According to the Federal Emergency Management Agency's Flood Insurance Rate Map for the area (FEMA, 1981), the site is located outside of a detailed study area. However, to the northeast and downstream from the site, it appears that the 100-year flood is confined to the creek and the 500-year flood is confined by the western creek bank and extends a short distance to the east of the creek.

PRIOR GEOTECHNICAL INVESTIGATIONS

Jo Crosby & Associates (JCA) prepared a seismic evaluation report for the Oak Knoll School property on May 9, 1996. Based on a reconnaissance-level investigation, JCA provided a preliminary opinion that there is a potential for surface fault rupture along the two, bifurcated traces of the Pulgas fault mapped by Pampeyan (1993) and that fault rupture may likely be caused by sympathetic movement during a large earthquake on the San Andreas fault. To mitigate this potential hazard, JCA recommended a 50-foot building setback from the mapped traces.

Subsequently, JCA performed a geotechnical investigation for proposed additions to the Oak Knoll School. The results of their investigation are presented in their report dated

November 21, 1997. As part of their investigation, JCA referenced their prior seismic evaluation report, reviewed geologic literature, performed a site reconnaissance, and evaluated the subsurface conditions in the area of the proposed additions by advancing 5 exploratory borings to depths of 8 to 14 feet below grade. According to the report and the boring logs, the site is blanketed with up to 1 foot of artificial fill and clayey soil to a depth of approximately 5 feet. The native soil is underlain by alluvium composed of a mixture of gravel, sand, and clay. No groundwater was encountered in the borings, which were excavated on October 8, 1997. Based on their investigation, JCA concluded that the potential for liquefaction, lateral spreading, and differential compaction at the site is low because of the density of the subsurface materials and the absence of high groundwater. Although not discussed in the text of the report, the site plan attached to the report depicts 100-foot building setbacks from the concealed traces of the Pulgas fault, which appear to have been transferred onto the site plan by tracing the faults from a photocopy enlargement of the geologic map by Pampeyan (1993).

SITE EXPLORATION AND RECONNAISSANCE

Exploration Program

Our field investigation was performed on February 22, 2007, and included a site reconnaissance and the excavation, sampling, and logging of six exploratory borings to depths ranging from 5 to 50 feet at the approximate locations shown on Figure A-2. The boring locations were approximately determined by measuring distances from known points on the supplied site plan and should be considered accurate only to the degree implied by the mapping technique used.

The borings were advanced using a truck-mounted drill rig equipped with hollow-stem, continuous flight augers. Soil samples were collected with split-spoon samplers driven with a 140-pound hammer repeatedly dropped from a height of 30 inches with a wireline sampling system. The samplers included the 2-inch outside diameter (OD) Standard Penetration Test (SPT) sampler, as well as 2.5- and 3-inch OD split-spoon samplers. The sampler types used are indicated on the log at the appropriate depth. The number of hammer blows required to drive the samplers was recorded for each 6-inch increment and the sum of the second and third 6-inch increment is recorded on the log. The associated blow count data, which is presented on the boring logs, has not been corrected for sampler type and hammer efficiency. The logs of our borings are presented in Appendix B as Figures B-1 through B-6. Also included in Appendix B are Figure B-7, Key to Boring Logs; Figure B-8, Unified Soil Classification System; and Figure B-9, Key to Bedrock Descriptions.

Our staff engineer logged the borings in general accordance with the Unified Soil Classification System and Key to Bedrock Descriptions. The boring logs show our interpretation of the subsurface conditions at the location and on the date indicated and it is not warranted that these conditions are representative of the subsurface conditions at other locations and times. In addition, the stratification lines shown on the logs represent approximate boundaries between various soil materials and the transitions may be gradual.

Site Description

The Oak Knoll Elementary School property is located on the south side of Oak Knoll Lane near the intersection of Oak Knoll Lane and Oak Avenue in Menlo Park. The approximately triangular-shaped property measures approximately 8.9 acres in plan area and is bounded by Oak Avenue to the east, Vine Street to the southwestern, and developed residential properties and Oak Knoll Lane to the north. Drainage across the property is generally characterized as uncontrolled sheet flow to the northeast.

The school property is currently occupied by several single-story, wood-framed school buildings, situated in the central portion of the site, and consisting of six classroom buildings, a multi-use building, administration wing, library, and portable classroom buildings. Covered and non-covered walkways facilitate pedestrian access to the various school buildings. A paved driveway/drop-of/parking area is located in the northwest corner of the site and additional parking is located at the southern end of the site. The northern approximate ¼ of the school property contains paved play areas including basketball courts and ball-walls, a large play field area, a playground apparatus area, and a covered lunch area. The southern approximate ¼ of the site contains paved play areas including basketball and tetherball courts, a baseball field, and two tennis courts. The school is landscaped with scattered mature trees, decorative trees, shrubs, and lawns.

The school site was initially developed in 1951 by the Menlo Park City School District and presently accommodates approximately 700 students. A modernization of the school took place in the late 1990's and included footprint expansions of the three classroom buildings, addition of a library and media wing to the administration building, expansion of the parking area in the southwest corner of the site, and renovation of several classrooms and the multi-use building. Based on our review of portions of the existing foundation plans, it appears that the school buildings are founded on continuous perimeter spread footing foundations with concrete slab-on-grade floors. The footing depths are shown to vary, but are generally about 4 feet in depth from adjacent exterior grade.

Based on our site reconnaissance, it appears that the existing nearly 60-year old school building foundations have performed adequately for their intended use. Minor distress was observed in portions of the school buildings in the form of exterior stucco cracks along the walls and adjacent window frames and doorways. Cracks along portions of the exterior concrete slabs supporting the covered walkways were also observed as well as minor to significant cracking of the paved play areas and parking lots. Based on our subsurface investigation, it appears that the existing Asphaltic Concrete (AC) pavement sections underlying the paved play areas in the northern quadrant of the site include approximately 2 inches of AC over approximately 4 to 6 inches of Aggregate Baserock.

Subsurface

Six exploratory borings were advanced on the site in the area of the proposed improvements. Borings B-1 and B-2 were excavated to a depth of 5 feet in the area of the proposed new parking lot at the north end of the site. Borings B-3 through B-6 were excavated to depths of approximately 30 to 50 feet within and around the footprint of the proposed multi-use building in the northern portion of the site.

In general, our borings encountered alluvial topsoil or fill in the upper 2.5 to 3.5 feet, consisting of stiff to very stiff, moderately plastic silty to sandy clay to clayey silt. Below the surficial soils our borings encountered fine-grained alluvium; consisting of low to moderate plasticity, stiff to hard clays and silts with varying amounts of sand and fine gravel; and extending to depths of 11 to 23 feet. In Borings B-3 through B-6, the fine-grained alluvium was underlain by 4.5- to 22-foot thick layers of coarse-grained alluvium, consisting of medium dense to very dense clayey sand to sandy and clayey gravel. Severely weathered, gray siltstone, light yellowish brown sandstone, and bluish gray siltstone and conglomerate bedrock were encountered beneath the alluvium at depths of approximately 20, 27.5, 27, and 32 feet, respectively in Borings B-3 through B-6. The bedrock persisted to the bottom of the borings at depths between 30 to 50 feet. The siltstone unit appears to be moderately expansive and consistent with mapped exposures of the Whiskey Hill formation. We did not observe any evidence of highly sheared, pale-olive to grayish-olive micaceous shale and silty shale, such as was identified at the Willow Road Bridge abutment excavation.

Atterberg limits testing on samples of the alluvial topsoil encountered in the upper 2 feet in borings B-1 and B-5 yielded liquid limits between 31 and 29 percent and plasticity indices of 16 percent, indicating that this material has a low to moderate potential for expansion (see Figure C-1, Plasticity Chart). A sieve analysis of fine-grained alluvium encountered in Boring B-6 at a depth of 7 to 8.5 feet yielded 54 percent passing the No. 200 sieve. Sieve analyses on coarse-grained alluvium encountered in Borings B-3 through B-6 yielded between 12 and 42 percent passing the No. 200 sieve.

A direct shear test was conducted on a sample of the alluvium from Boring B-4 at a depth of 1 to 1.5 feet and included a cohesion value of 90 psf and an angle of internal friction of 23 degrees (see Figure C-2, Direct Shear Test Chart). R-value testing performed on the fine-grained alluvium encountered in the upper 5 feet in Boring B-1 yielded an R-value of 16 (see Figure C-3, R-Value Test Report).

Corrosivity testing was performed on a sample of the alluvium from Boring B-4 at a depth of approximately 1 to 1.5 feet. The testing included an evaluation of resistivity, chloride and sulfate content, and pH. A summary of the test results is presented in Appendix C on Figure C-4. The corrosion potential of the soil sample tested may be classified as moderately corrosive. We recommend that a corrosion engineer be consulted regarding corrosion protection for foundations and any underground metallic improvements proposed for the school project.

Groundwater

According to Plate 1.2 of the State Seismic Hazard Zone Report for the Palo Alto Quadrangle (CGS, 2006), the site is located in an area where historically high groundwater levels have been greater than 30 feet below the ground surface. Free groundwater was encountered in Boring B-6 at the time of drilling at a depth of 27.5 feet. It appears that this groundwater may be perched on top of the weathered bedrock; however, no groundwater was encountered in the other borings. We note that fluctuations in the level of groundwater can occur due to variations in rainfall, landscaping, and other factors not evident at the time our observations and measurements were made.

PROBABILISTIC SEISMIC HAZARD ANALYSIS

Oak Knoll Elementary School falls under the jurisdiction of the 2002 Statewide Probabilistic Seismic Hazard model required by the 2001 California Building Code (Chapters 16 through 18) and adopted by the California Geological Survey for review of public schools in California. As a consequence, a site-specific probabilistic seismic hazard analysis (PSHA) was performed as a part of our investigation to evaluate representative probable peak ground accelerations for the subject school site that may be utilized in the design of the building improvements. Specifically, the PSHA was conducted using the computer program FRISKSP (Version 4.0), which characterizes the ground motion at the site in terms of peak ground acceleration (PGA) for various probabilities and exceedance periods, taking into account historical seismicity, available paleoseismic data, the geologic slip rates of regional faults, and site-specific response characteristics. The computer program includes a database of potentially damaging earthquake sources, including known active faults and historic seismic source zones, their activity rates, and distances from the project site; employs published maximum moment magnitudes for earthquake sources and published curves for attenuation of PGA with distance from earthquake source, as a function of earthquake magnitude and travel path; and evaluates the likely effects of site-specific response characteristics.

A site-specific probability of exceedance diagram and return period verses acceleration graph have been included in this report as Figures D-1 and D-2, respectively, utilizing the FRSKSP computer program and basing the seismic criteria on equations developed by Campbell & Bozorgnia (1997) for an alluvial soil site. Given representative latitude of 37.431 and longitude of 122.191, as input parameters in the computer program, the following criteria were obtained by extrapolating the required PGA values from the attached probability curve:

> Upper Bound Earthquake (UBE or 950-year return period) - Ground motion for a 10% probability of exceedance in 100 years based on a predominant earthquake magnitude of 7.9 Mw = 0.82g
>
> Design Basis Earthquake (DBE or 475-year return period) - Ground motion for a 10 percent probability of exceedance in 50 years based on a predominant earthquake magnitude of 7.9 Mw = 0.70g

LIQUEFACTION ANALYSIS

Liquefaction is a process by which soil deposits below the water table temporarily lose strength and behave as a viscous liquid rather than a solid, typically during a moderate to large earthquake. In general, clean fine- to medium-grained sand and silts, which contain relatively little clay, are most susceptible to "classic" style liquefaction. Such a condition occurs when earthquake-induced ground shaking causes loose granular material to consolidate, resulting in increased pore water pressures and an upward movement of groundwater that may result in a temporary liquefied condition. Historical performance of buildings damaged during earthquakes has revealed that saturated soft to firm fine-grained soils (silts and clays) of low plasticity and limited clay content are also considered susceptible to liquefaction. Such soil types exhibit a loss of strength and stiffness from earthquake-induced ground shaking related to the phenomenon of strain-softening. The main concern with this hazard is that structures founded on or above temporarily liquefied soils may sink or tilt, potentially causing significant structural damage.

To address the potential for liquefaction at the school site and its impact on the planned improvements, we performed analyses using our subsurface information combined with site-specific design level earthquake values to estimate the magnitude of liquefaction-induced total and differential settlements. The basis of our data included a groundwater level of 27.5 feet below grade and incorporation of potentially liquefiable layers of medium dense to very dense clayey sands and sandy to clayey gravels at depths ranging from approximately 11 to 32 feet below existing grade. The majority of the finer grained soils encountered in our test borings were eliminated from the analysis based on engineering judgment and by the preliminary screening criteria outlined in CDMG Special Publication 117 (1997), entitled "Guidelines for Evaluating and Mitigating Seismic Hazards in California". Specifically, the screening method takes into account the amount of clay content (greater than 15%) and liquid limit (greater than 35% or in-place moisture content that is greater than 0.9 times the liquid limit) or by evaluating the density of the fine grained and granular soil from relatively high corrected blow count data.

Our analyses were performed using the computer program LiquefyPro (Version 5.2E), which calculates a factor of safety (FS) against soil liquefaction by comparing the cyclic resistance ratio (CRR), the ratio of the resistance of the identified soil to liquefaction during cyclic shaking, to the cyclic stress ratio (CSR), the seismic loading that would likely result from a design level earthquake at the study location. If the factor of safety for a soil layer is less than 1.0, there is a potential that the soil layer may liquefy during a moderate to large seismic event. The methods outlined in the above publications were also used to evaluate magnitude of anticipated soil settlement, calculated as the volumetric strain (qualified by the CSR) times the thickness of the liquefiable soil layer.

The CRR during a design-level earthquake is a function of groundwater level, earthquake magnitude, soil density, and the depth of the soil layer being evaluated. For the purpose of our liquefaction analyses, we considered a design groundwater level at a depth of 27.5 feet below the existing ground surface. According to the State Seismic Hazard Zone Report for Palo Alto 7.5-minute Quadrangle (2006), the estimated PGA in alluvial conditions in the site vicinity is approximately 0.70 g for a 10% probability of exceedance in 50 years (DBE of 475 year return period) based on a predominant earthquake magnitude of 7.9 Mw. However, as a conservative approach in our analysis, we utilized the upper bound curves developed by Campbell & Bozorgnia (1997 Revision) during our PHSA to estimate a site-specific PGA of approximately 0.82 g for a 10% probability of exceedance in 100 years (UBE or 949 year return period) based on a predominant earthquake magnitude of 7.9 Mw. The soil density values were estimated based on site-specific laboratory data collected during exploratory drilling. Our CRR calculations are based on normalized standard penetration blow counts corrected for field-testing procedures, such as hammer efficiency, borehole diameter, rod length, liner correction factor, and overburden pressures. In addition, because the CRR curves used in LiquefyPro are based on clean sand, we applied a fines content correction to the SPT data based on modified curves published by Stark and Olsen (1995).

LiquefyPro calculates liquefaction induced settlement by dividing the soil strata into thin layers and calculating settlement for each layer. The settlement in each layer was calculated by multiplying the volumetric strain by the thickness of each layer. Volumetric strain was calculated using the method published by Ishihara & Yosmine (1993), which uses the factor of safety against liquefaction against corrected SPT data.

The results of our analysis for Borings B-5 and B-6 indicate that the granular soils encountered below the highest projected groundwater level have a low potential for significant liquefaction resulting from a design-level earthquake. Specifically, the total seismic-induced settlement estimates generated using the LiquefyPro program and based on the input subsurface data is as follows:

Boring B-5 = 0.4 inches total seismic-induced settlement
Boring B-6 = 0.3 inches total seismic-induced settlement

Based on engineering judgment and utilizing a percentage of the total seismic-induced settlement values indicated above, an assumption can be made that there exists the potential for approximately 0.2 inches of seismic-induced differential settlement across the proposed multi-use building site areas (see Figures D-3 and D-4, Liquefaction Hazard Analyses, Boring B-5 and B-6, respectively).

In summary, it should be noted that computer-aided liquefaction analyses are mathematical models of subsurface materials, and they contain many assumptions. Liquefaction analyses and the generated factors of safety should only be used to indicate general ground stability trends. In general, factors of safety below 1.00 indicate a potential liquefaction hazard. However, a subsurface layer with a factor of safety of less than 1.00 may not necessarily liquefy but the probability of liquefaction will be greater than that in a layer with a higher factor of safety. Conversely, a subsurface layer with a factor of safety greater than 1.00 may liquefy but the probability of liquefaction is lower than that in a layer with a lower factor of safety.

Further findings that refute the potential for liquefaction occurring at the subject school site are as follows:

1. All potentially liquefiable zones identified in our analyses have been classified as medium dense to very dense based on normalized blow count resistance measured

during exploratory drilling. Such soil types are commonly not susceptible to liquefaction.

2. To our knowledge, no historical evidence of seismic induced ground deformation has been documented at this site or neighboring area (Knudsen and others, 2000).

3. According to the USGS geologic map of the area (Pampeyan, 1993), the site is located in an area underlain by Pleistocene age (approximately 10,000 to 1.6 million years old) stream terrace deposits (Qst). In general, pre-Holocene aged soil deposits, particularly gravels, are generally not considered susceptible to liquefaction.

4. According to the State of California Seismic Hazard Zones Map for Palo Alto Quadrangle (2006), the site is not located in an area considered susceptible to earthquake-induced liquefaction (see Figure A-4).

CONCLUSIONS

From a geotechnical perspective, it is our opinion that the Oak Knoll Elementary School site is suitable for the proposed improvements, provided the recommendations contained in this report are implemented in the design and construction of the proposed improvements. In our opinion, the primary geotechnical constraints to the proposed improvements are the low to moderately expansive surficial soils blanketing the school site; the slight potential for seismic induced settlement of moderately dense granular layers noted at depth beneath the site; and the potential for strong ground shaking at the site as a result of a moderate to large earthquake on the San Andreas Fault. In addition, based on our investigation, it is our opinion that the potential for surface fault rupture and coseismic ground deformation at the site is low. A more detailed discussion of potential engineering geologic hazards at the site is presented below.

Based on our investigation, the site appears to be blanketed by clayey to gravelly alluvium to depths of approximately 20 to 32 feet below grade. Minor amounts of fill generally less than approximately 2.5 feet thick are located within the proposed building site. Beneath the fill and alluvial cover, our exploratory borings encountered siltstone, sandstone, and conglomerate bedrock, which appear to be units of the Whiskey Hill formation. The Whiskey Hill formation bedrock persists to the maximum depth explored of 50 feet. According to Pampeyan (1993), Franciscan Assemblage basement rocks are encountered approximately 210 feet below the ground surface. In our opinion, the alluvial soil below a depth of approximately 3 feet should provide adequate support for the foundations of the proposed improvements, provided that they are designed and constructed in accordance with the recommendations presented in this report.

Engineering Geologic Hazards

As part of our investigation, we reviewed and evaluated the potential for engineering geologic hazards to impact the site and the proposed improvements, considering the geologic setting, topography, and the soils encountered during our investigation. The results of our review and evaluation follow:

- Surface Fault Rupture - Based on our investigation, it is our opinion that the potential for surface rupture through the school site and, specifically, in the area of the existing and proposed improvements is low. According to Pampeyan (1993), traces of the Pulgas fault are located through or very near the northern and southern ends of the property. The bifurcated nature and the alignment of this fault are in sharp contrast to the single-strand, N45W alignment presented by the original investigators, Graham and Church (1963). More recent mapping by Angell and others (1997) and Bullard and others (2004), depict a more westerly alignment of the Pulgas fault, approximately coincident with a topographic rise, which is visible in stereographic aerial photographs, located to the west of the school property and to the east of the Alameda de las Pulgas. Recent investigators detail a broad, right-stepping, *en echelon* monoclinal feature, known as the Stanford fault zone, passing immediately northeast of the school property. Based on the latest work by Bullard and others (2004), the Pulgas fault appears to be inactive; however, the Stanford fault zone appears to be possibly active, with vertically off-set alluvial terraces suggesting uplift on the order of approximately 0.6 mm/yr. Based on our subsurface investigation, it appears that the north end of the school property, in the area of the proposed improvements is underlain by bedrock of the Whiskey Hill formation. We did not observe any evidence of the unnamed Cretaceous shale, generally described as sheared, olive-colored, micaceous shale, which was exposed in the Willow Road Bridge abutment excavation. In our opinion, the absence of the unnamed Cretaceous shale at the site is consistent with the more northeasterly alignment of the Stanford fault zone described by Angell and others (1997) and Bullard and others (2004). We note that this more northeasterly alignment is also consistent with the original mapping of the fault zone by Graham and Church (1963). Additionally, Kovach and Page (1995) tentatively have discounted the Willow Road Bridge fault as a possible seismic source. Based on this collective data, it is our opinion that no active or potentially active faults cross the Oak Knoll School property in the area of the proposed or existing improvements. Therefore, in our opinion the potential for fault rupture to occur at the site is low.

- Coseismic Ground Deformation – Coseismic ground deformation, including ground uplift, warping, and fissuring can occur as a result of slip along faults during earthquake events. As noted above, the Stanford fault zone, a broad, monoclinal structure is mapped to the east of the Oak Knoll School property and appears to pass immediately northeast of the school property. This feature apparently has developed as a result of contractional faulting through the area over geologic time. Based on the location of this feature to the east and northeast of the property and the lack of evidence for deformation through the site, it is our opinion that the potential for structurally damaging coseismic deformation at the site, as a result of uplift along the Stanford fault zone is low.

- Ground Shaking - As noted in the Seismicity section above, moderate to large earthquakes are probable along several active faults in the greater Bay Area. Therefore, strong ground shaking should be expected several times during the design life of the proposed improvements. The improvements should be designed in accordance with current earthquake resistant standards, including the 2001 California Building Code and the design parameters presented in this report.

- Differential Compaction - During moderate and large earthquakes, soft or loose, natural or fill soils can become densified and settle, often unevenly across a site. Because the surficial soil encountered at the site is generally stiff to hard where fine-grained and medium dense where granular, in our opinion, the potential for differential compaction at the site is low.

- Liquefaction - Based on our investigation, which included a quantitative liquefaction hazard analysis, in our opinion the probability of liquefaction or ground displacement, ground lurching, differential settlement, or lateral spreading as a result of liquefaction at the site during a major seismic event in the area is low. Any potential differential foundation movement resulting from earthquake induced liquefaction in the area of the proposed building footprints, if it were to occur, has been estimated to be less than 0.5 inch and should therefore not adversely affect the building improvements provided the recommendations contained in this report are implemented in design and construction. Please refer to the Liquefaction Analysis section of our report for more detailed information concerning this potential geologic hazard.

RECOMMENDATIONS

In our opinion, the proposed multi-use building, covered lunch area, and addition to the existing multi-use building may be supported on conventional continuous spread footings bearing in the underlying competent alluvial deposits. Interior isolated spread footings should not be utilized in the design of new building foundations. Building slabs should be constructed with sufficient underlayment to mitigate the potential for differential floor movement resulting from the potentially expansive surficial soils encountered at the site.

Roof overhangs, landscape, and playground features, including signs, fences, basketball posts, flag poles, gates and other similar structures, may be supported on either drilled piers or conventional isolated or continuous spread footings. Our detailed recommendations are presented in the following sections of this report.

EARTHQUAKE DESIGN PARAMETERS

Based on the Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada (CDMG, 1998), the site is approximately 9.5 kilometers from the San Andreas fault (Type A) (see Figure A-8, Active Fault Near-Source Zones). In accordance with guidelines presented in the 2001 California Building Code, the following seismic design parameters will apply:

- Seismic Zone Factor (Z) = 0.4 (Zone 4)
- Soil Profile Type = S_C, Very Dense Soil and Soft Rock (Table 16-J)
- Near Source Factors: Na = 1.2 (Table 16-S)
  Nv = 1.5 (Table 16-T)

SPREAD FOOTINGS

We recommend the proposed multi-use building, covered lunch area, and additions to the existing multi-use building be supported on conventional continuous spread footing foundations. In addition, conventional continuous spread footing foundations may be used for any additions to and/or modernization of existing structures. Strong continuity should be provided where new spread footings abut existing foundations.

All continuous spread footings should have a minimum width of 20 inches and extend at least 36 inches below final exterior grade and at least 24 inches below bottom of slab, whichever is deeper. Footings for additions should also extend to at least the depth of the existing adjacent footings and, if new footing depths fall below existing footings, the existing footings should be underpinned where they abut new foundations. All footings located adjacent to utility trenches should bear below a 1:1 plane extended upward from the bottom edge of the utility trench.

Isolated spread footings should not be utilized in the design of building foundations. However, isolated spread footings (or drilled piers) may be used for roof overhangs, landscape, and playground features, including signs, fences, basketball posts, flag poles, gates and other similar structures. If used, isolated spread footings should be a minimum of 24 inches square and should extend at least 36 inches below lowest adjacent exterior grade.

We recommend an allowable bearing pressure of 2,500 pounds per square foot for dead plus live loads, with a one-third increase allowed for total loads including wind and seismic forces. The weight of the footings can be neglected for design purposes.

Lateral loads may be resisted by friction between the bottom of the footings and the supportive material using a friction coefficient of 0.30 for concrete formed on undisturbed soil. In addition to the preceding frictional resistance, lateral resistance may be provided by passive pressures acting against foundations poured neat in the footing excavations using an equivalent fluid pressure of 270 pounds per cubic foot acting below a depth of 1 foot from the lowest adjacent grade.

Footing reinforcing should be determined by the project structural engineering based on the geotechnical design parameters presented in this report and anticipated building loads. However, as a minimum, we suggest that all continuous footings be reinforced with six No. 5 reinforcing bars (two near the top, two at the middle, and two near the bottom), to provide structural continuity and to permit spanning of local irregularities. Actual

The footing excavations should be substantially free of all loose soil, prior to placing reinforcing steel and concrete. Our representative should observe the footing excavations prior to placing concrete forms and reinforcing steel to see that they are founded in competent bearing materials and have been properly prepared. In addition, any loose soil in the footing excavations resulting from the placement of forms and reinforcing steel should be removed prior to placing concrete.

Thirty year vertical differential movement of spread footings due to static loads is not expected to exceed approximately 1 inch across any 20-foot span of the foundation.

DRILLED CAST-IN-PLACE CONCRETE PIERS

Drilled cast-in-place concrete piers may be used as an alternative to conventional spread footings to support roof overhangs, landscape, and playground features, including signs, fences, basketball posts, flag poles, gates and other similar structures. All piers should be at least 16 inches in diameter and extend at least 10 feet below lowest adjacent grade for roof overhangs and at least 8 feet below the lowest adjacent grade for landscape and playground features. The piers should be designed to resist dead plus live loads using an allowable skin friction value of 450 pounds per square foot acting below a depth of 2 feet from the lowest adjacent grade with a one-third increase allowed for transient loads, including wind and seismic forces. The upper 2 feet of soil and any point-bearing resistance should be neglected for support of vertical loads. Drilled piers should be spaced no closer than three pier diameters, center-to-center.

All active lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 270 pounds per cubic foot, acting on 2 times the projected area for the depth of the piers in the alluvium. The passive resistance within the upper two feet of all piers should be neglected.

Pier reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements. However, as a minimum, piers should be reinforced with a cage of four No. 5 steel reinforcing bars for the full length of the piers.

The bottoms of the pier excavations should be substantially free of loose cuttings and soil slough prior to the installation of reinforcing steel and the placement of concrete. In addition, any significant amounts of accumulated water in the pier excavations should be pumped out prior to placing concrete or displaced using the tremie method when placing concrete. A representative of Murray Engineers, Inc. should observe the pier excavations to evaluate whether the piers are founded in the supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

SLABS-ON-GRADE

Interior Building Slabs

Interior building slabs-on-grade should be underlain by at least 18 inches of compacted select granular fill, such as Class 2 aggregate baserock, overlying compacted subgrade soils. Please refer to the Compaction section of this report for compaction specifications. Building slabs may be designed as free-floating slabs, structurally isolated from adjacent footings or they may be structurally tied to the adjacent footings. In general, slabs should be provided with control joints at spacings of not more than 10 feet. The project structural designer should determine slab reinforcement based on anticipated use and loading.

We recommend that the building slabs be underlain by a moisture barrier, including a vapor barrier consisting of a highly durable membrane not less than 10 mils thick (such as Stego Wrap Vapor Barrier by Stego Industries, LLC or equivalent), underlain by a capillary break consisting of at least 4 inches of ½- to ¾-inch crushed rock. The capillary break may be considered the equivalent thickness as the upper 4 inches of select granular fill recommended above. Please also refer to the Vapor Barrier Considerations section below for additional information. Please note that these recommendations do not comprise a specification for "waterproofing." For greater protection against concrete dampness, we recommend that a waterproofing consultant be retained.

Exterior Slabs

Patio and other exterior slabs should be underlain by at least 12 inches of select granular fill, although, generally, the greater the thickness of select granular fill, the less surface cracking of the slabs is likely to occur. In general, slabs should be provided with control joints at spacings of not more than 10 feet. The project structural designer should determine slab reinforcement based on anticipated use and loading.

Vapor Barrier Considerations

Based on our understanding, two opposing schools of thought currently prevail concerning protection of the vapor barrier during construction. Some believe that 2 inches of sand should be placed above the vapor barrier to protect it from damage during construction and also to provide a small reservoir of moisture (when slightly wetted just prior to concrete placement) to benefit the concrete curing process. Still others believe that protection of the vapor barrier and/or curing of concrete are not as critical design considerations when compared to the possibility of entrapment of moisture in the sand above the vapor barrier and below the slab. The presence of moisture in the sand could lead to post-construction absorption of the trapped moisture through the slab and result in mold or mildew forming at the upper surface of the slab.

We understand that recent trends are to use a highly durable membrane (at least 10 mils thick) without the protective sand covering for interior slabs surfaced with floor coverings including, but not limited to, carpet, wood, or glued tiles and linoleum. However, it is also noted that several special considerations are required to reduce the potential for concrete edge curling if sand will not be used, including slightly higher placement of reinforcement steel and a water-cement ratio not exceeding 0.5 (Holland and Walker, 1998). We recommend that you consult with other members of your design team, such as your structural engineer, architect, and waterproofing consultant for further guidance on this matter.

ASPHALTIC CONCRETE PAVEMENTS

Based on our investigation, the pavement subgrade conditions at the site are likely to consist of clayey subgrade soils that have a low to moderate potential for expansion. Laboratory testing on the clayey near surface soils encountered at the site revealed an R-value of 16. However, based on our local experience, for asphaltic pavement design purposes, we have estimated an R-value of 5 for the clayey surficial soil encountered at the site.

Procedure 608 of the Caltrans Highway Design Manual was used to develop the pavement sections presented in Table 1. Asphaltic concrete and aggregate base should conform to and be placed in accordance with the requirements of the California Department of Transportation, Standard Specifications, latest edition, except that the compaction standard should be ASTM D 1557 (latest edition).

Table 1. Asphaltic Concrete Pavement Sections
Subgrade Design R-Value = 5

Location	Design Traffic Index	Asphaltic Concrete (Inches)	Aggregate* Baserock (Inches)	Total Thickness (Inches)
Automobile Access & Parking	4.0	2.5	8.0	10.5
	4.5	3.0	8.5	11.5
Truck Access & Parking	5.0	3.0	10.0	13.0
	5.5	3.0	12.0	15.0
	6.0	3.0	14.0	17.0
	6.5	4.0	14.0	18.0

***Caltrans Class II Aggregate Base (minimum R-value of 78)**

The subgrade soil beneath asphaltic pavements should be scarified and moisture conditioned, as necessary, to a depth of 6 inches. The scarified subgrade soils and aggregate baserock should be compacted in accordance with the recommendations outlined in Table 2 of the Compaction section of this report. If soft subgrade conditions are encountered during construction, it may be necessary to place a geotextile strength fabric, such as Mirafi 500x or equivalent, on the subgrade soil prior to placement of aggregate baserock.

EARTHWORK

It is anticipated that the earthwork for the proposed project will be limited to the excavation of spread footings and drilled piers (if used), the potential raising of building pad grades and other associated site improvements, and the compaction of subgrade and select granular fill beneath slabs-on-grade. Any excavated soil should be disposed of off site or may be placed as fill in the relatively level portions of the property. All earthwork should be performed in accordance with the following recommendations.

Clearing & Site Preparation

All deleterious materials, topsoil, roots, vegetation, designated utility lines, foundations, etc. should be cleared from the areas to receive the planned construction. Excavations that extend below finished grade should be backfilled with structural fill, placed and compacted as discussed below.

Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent organic material by volume (ASTM D2974) are suitable for use as structural fill provided that the material has a plasticity index of less than 15 percent. In general, fill material should not contain rocks or pieces larger than 6 inches in greatest dimension, and contain no more than 15 percent larger than 2.5 inches. Any required import should have a plasticity index of less than 15 percent or be predominately granular. Class 2 aggregate baserock should meet the specifications outlined in the Caltrans Standard Specifications, latest edition. Crushed rock, if used below slabs or walkways, should be ½- to ¾-inch in particle size range and contain not more than 5 percent passing a No. 200 sieve.

Select granular fill material should consist of a predominantly granular soil with less than 5 percent fines (material passing a No. 200 sieve) and 100 percent of particle sizes smaller than 1-inch. Pea gravel will generally not be allowed. Any proposed fill for import should be approved by Murray Engineers, Inc. prior to importing to the site. Our approval process may require index testing to establish the expansivity of the soil; therefore, it is important that we receive any such samples at least 3 days prior to planned importing.

Trench Backfill

In general, bedding and shading materials to be used around underground utilities should be well-graded sand or gravel, conforming to the specifications of the local utility companies or City of Menlo Park specifications. Trench backfill placed above utility lines should be placed and compacted in accordance with the recommendations presented in the following Compaction section of this report or local regulations, if more stringent.

On-site soil may be used as trench backfill, provided that it meets the recommendations outlined above for fill material. Trench backfill should be placed and compacted in approximately 8-inch lifts and compacted in accordance with the compaction recommendations presented below. Water jetting of the trench backfill should not be allowed.

As noted above utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal to vertical) plane projected downward from the base of the footing. If deeper utility trenches are required, footing depths should be increased to conform to this recommendation.

Where utility trenches will cross beneath footings, we recommend that a trench plug be utilized through the bedding and shading zone to mitigate the potential for water migration through these permeable materials and beneath buildings. The trench plug may consist of compacted clayey soil, a 2-sack cement/sand slurry mix, or similar relatively impermeable material. The upper portion of these utility trenches should be backfilled with concrete, lean concrete, or a similar higher strength backfill beneath the foundation and for a distance of 2 feet on either side of the foundation.

Compaction

The scarified surface soils and all structural fill should be compacted in uniform lifts, no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted to the specifications listed in Table 2 below. The relative compaction and moisture content specified in Table 2 is relative to ASTM D 1557, latest edition. Compacted lifts should be firm and non-yielding under the weight of compaction equipment prior to the placement of successive lifts.

Table 2. Compaction Specifications

Fill Element	Relative Compaction*	Moisture Content*
General fill for raising site grades	90 percent	Near optimum
Upper 6 inches of subgrade beneath slabs-on-grade and asphaltic concrete for non-expansive soils (PI<20%)	90 percent	Near optimum
Upper 6 inches of subgrade beneath slabs-on-grade and asphaltic concrete for expansive soils (PI>20%), if encountered	87 to 90 percent	3-4% over optimum
Aggregate baserock under slabs-on-grade and asphaltic concrete	95 percent	Near optimum
Backfill of utility trenches using on-site soils	90 percent	Near optimum
Backfill of upper 12 inches of utility	95 percent	Near optimum

trenches with pavement areas		
Backfill of utility trenches using imported sand	95 percent	Near optimum
½- to ¾-inch Crushed Rock – compact with at least 3 passes of a vibratory plate with lift thickness ≤ 12 inches	See note at left	Not critical

***Relative to ASTM D 1557, latest edition.**

Temporary Slopes and Trench Excavations

The contractor should be responsible for the stability of all temporary slopes and trenches excavated at the site and the design and construction of any required temporary shoring or bracing. Shoring, bracing, or construction benching should be provided in accordance with all applicable local, state, and federal safety regulations, including the current OSHA excavation and trench safety standards. Because of the potential for variable soil conditions, field modifications of temporary cut slopes may be required.

SITE DRAINAGE

The finished grades should be designed to drain surface water away from buildings, adjoining slabs, and yard areas. Slopes of at least 3 percent are recommended within 5 feet of structures. Where such surface gradients are difficult to achieve, we recommend that area drains and/or surface drainage swales be installed to direct surface water to a suitable discharge location away from buildings. Buildings should be provided with roof gutters and downspouts. Downspout drainage should preferably be collected in closed pipe systems and routed to a suitable discharge outlet.

FUTURE SERVICES

Plan Review

To better assure conformance of the final design documents with the recommendations contained in this report, and to better comply with the State school requirements, Murray Engineers, Inc. must review the completed project plans prior to construction. The plans should be made available for our review as soon as possible after completion so that we can better assist in keeping your project schedule on track.

At a minimum, we recommend that the following project-specific note be added to the architectural, structural, and civil plans:

- All earthwork and site drainage, including foundation excavation, preparation of subgrade beneath slabs-on-grade, placement and compaction of engineered fill, and final surface drainage installation should be performed in accordance with the geotechnical report prepared by Murray Engineers, Inc., dated June 15, 2007. Murray Engineers, Inc. should be provided at least 48 hours advance notification of any earthwork operations and should be present to observe and test, as necessary, the earthwork and foundation installation phases of the project.

Construction Observation Services

Murray Engineers, Inc. should observe and test, as necessary, the earthwork and foundation phases of construction in order to a) confirm that subsurface conditions exposed during construction are substantially the same as those interpolated from our limited subsurface exploration, on which the analysis and design were based; b) observe compliance with the geotechnical design concepts, specifications and recommendations; and c) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on limited subsurface information. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it may be necessary to re-evaluate our recommendations.

LIMITATIONS

This report has been prepared for the sole use of the Menlo Park City School District, specifically for evaluating engineering geologic hazards and developing geotechnical design criteria relating to design and construction of the proposed improvements on the Oak Knoll School property located at 1895 Oak Knoll Lane in Menlo Park, California. The opinions presented in this report are based upon information obtained from borings at widely separated locations, site reconnaissance, review of field data made available to us, and upon local experience and engineering judgment, and have been formulated in accordance with generally accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was prepared. Further, our recommendations are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. It should be understood that geotechnical issues may arise that were not apparent at the time our investigation was conducted and this report prepared. In addition, it should be understood that we are not responsible for data provided or presented by others. No warranty, either expressed or implied, is made or should be inferred.

The recommendations provided in this report are based on the assumption that we will be retained to provide the Future Services described above in order to evaluate compliance with our recommendations. If we are not retained for these services, Murray Engineers, Inc. cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Murray Engineers, Inc. report by others. Furthermore, if another geotechnical consultant is retained for follow-up service to this report, Murray Engineers, Inc. will at that time cease to be the Engineer-of-Record.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any property other than that evaluated.

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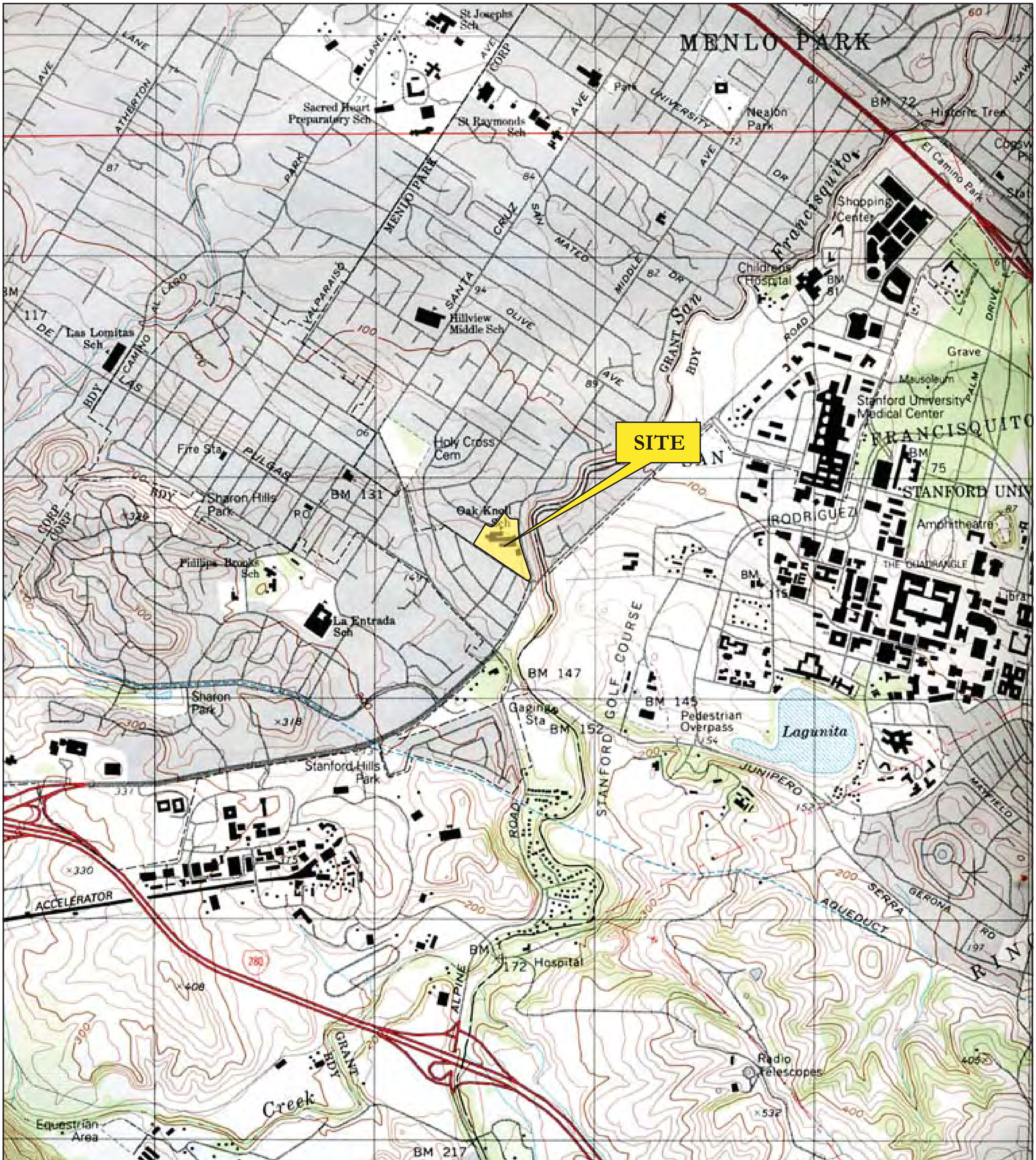
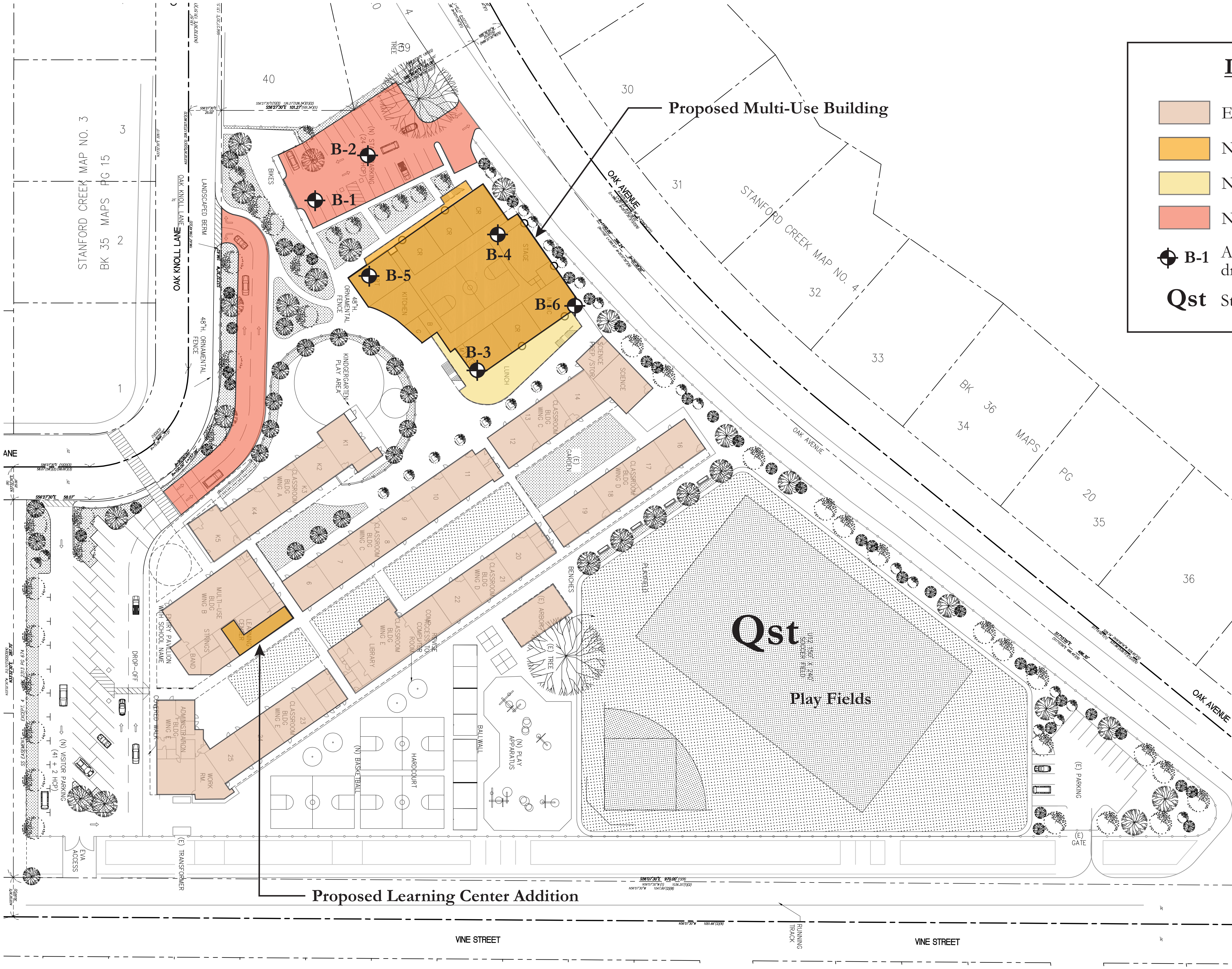


Base: USGS Topographic Map, Palo Alto 7.5-Minute Quadrangle, 1997 Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	**OAK KNOLL ELEMENTARY** **1895 OAK KNOLL LANE** **SAN MATEO COUNTY, CALIFORNIA**		**VICINITY MAP**
	PROJECT NO. 598-3R1	**JUNE 2007**	**FIGURE A-1**



Legend

- Existing Buildings
- New Construction
- New Covered Lunch Patio
- New Driveway and Parking
- B-1 Approximate Location of Boring, drilled by Murray Engineers, Inc.
- Qst Stream Terrace Deposits

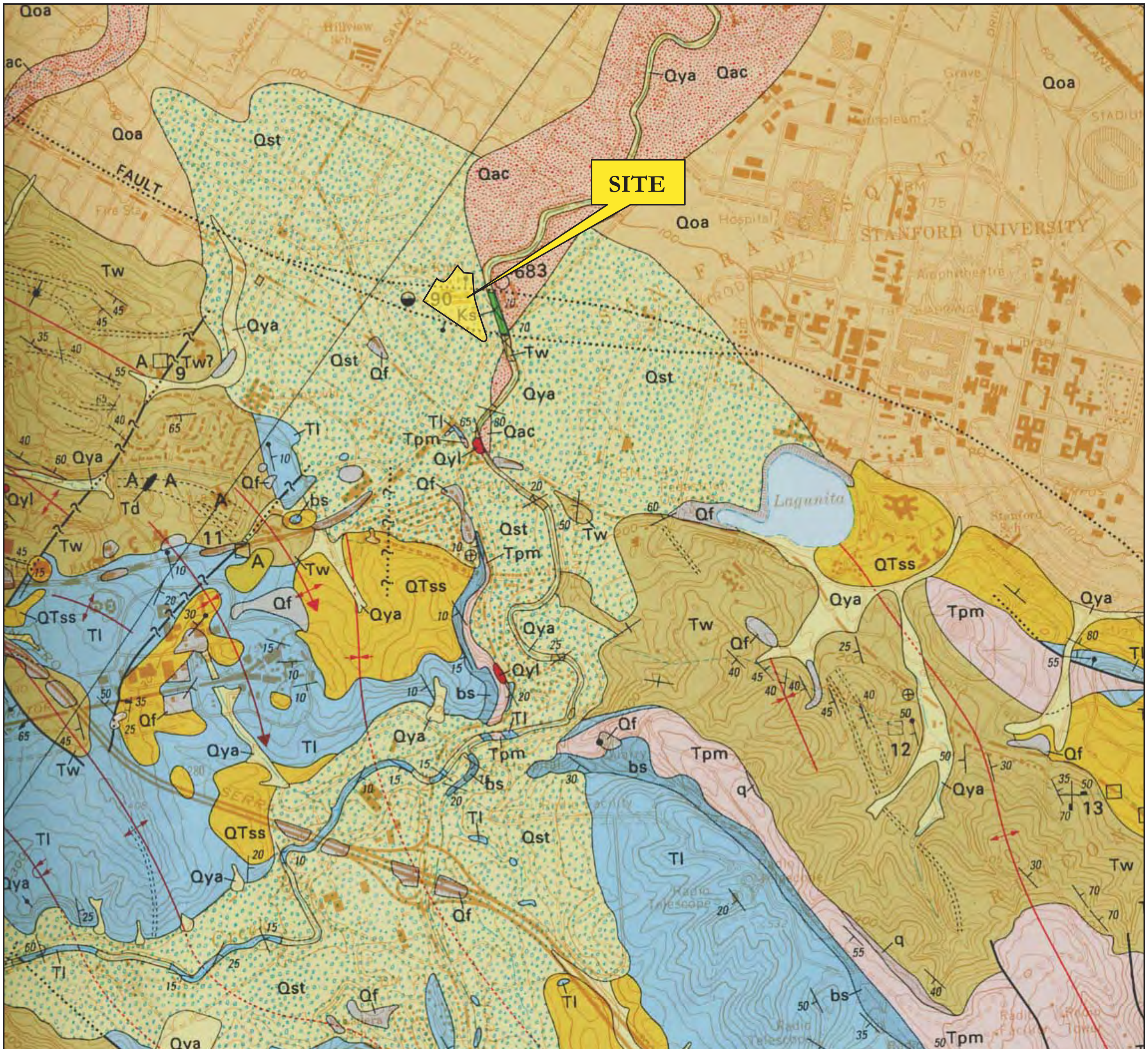
Base: Proposed Improvements Plan by Deems Lewis McKinley, dated March 7, 2007.
Approximate Scale: 1 inch = 80 feet

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

SITE PLAN & ENGINEERING GEOLOGIC MAP

OAK KNOLL ELEMENTARY
1895 OAK KNOLL LANE
SAN MATEO COUNTY, CALIFORNIA

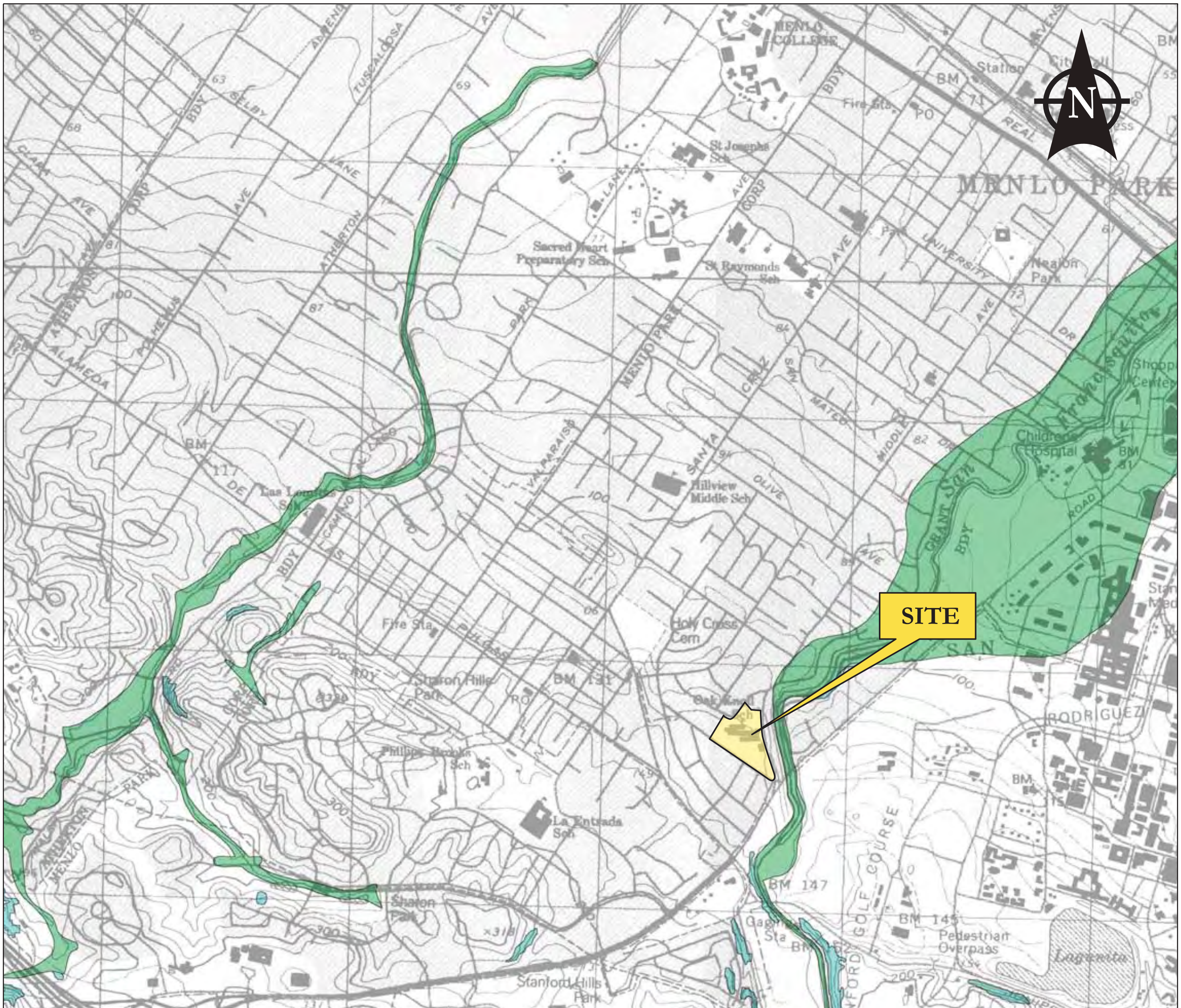
PROJECT NO. 598-3R2	JUNE 2007	FIGURE A-2



Legend & Selected Map Symbols

Qac	Coarse-Grained Alluvium		Contact
Qst	Stream Terrace Deposits	?	Fault, dashed where approx. located, dotted where concealed or inferred, queried where uncertain
Qoa	Older Alluvium	-256	Borehole to basement
Tw	Whiskey Hill Formation		Scale is 1 inch = 2,000 feet.
Ks	Shale (unnamed)		

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA		VICINITY GEOLOGIC MAP
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE A-3



Legend

Areas where historic occurrence of liquefaction, or local, geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Base: State of California Seismic Hazard Zone Map for the Palo Alto 7.5 Minute Quadrangle, CGS 2006
Scale: 1 inch = 2,000 feet.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA		STATE SEISMIC HAZARD ZONES MAP
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE A-4

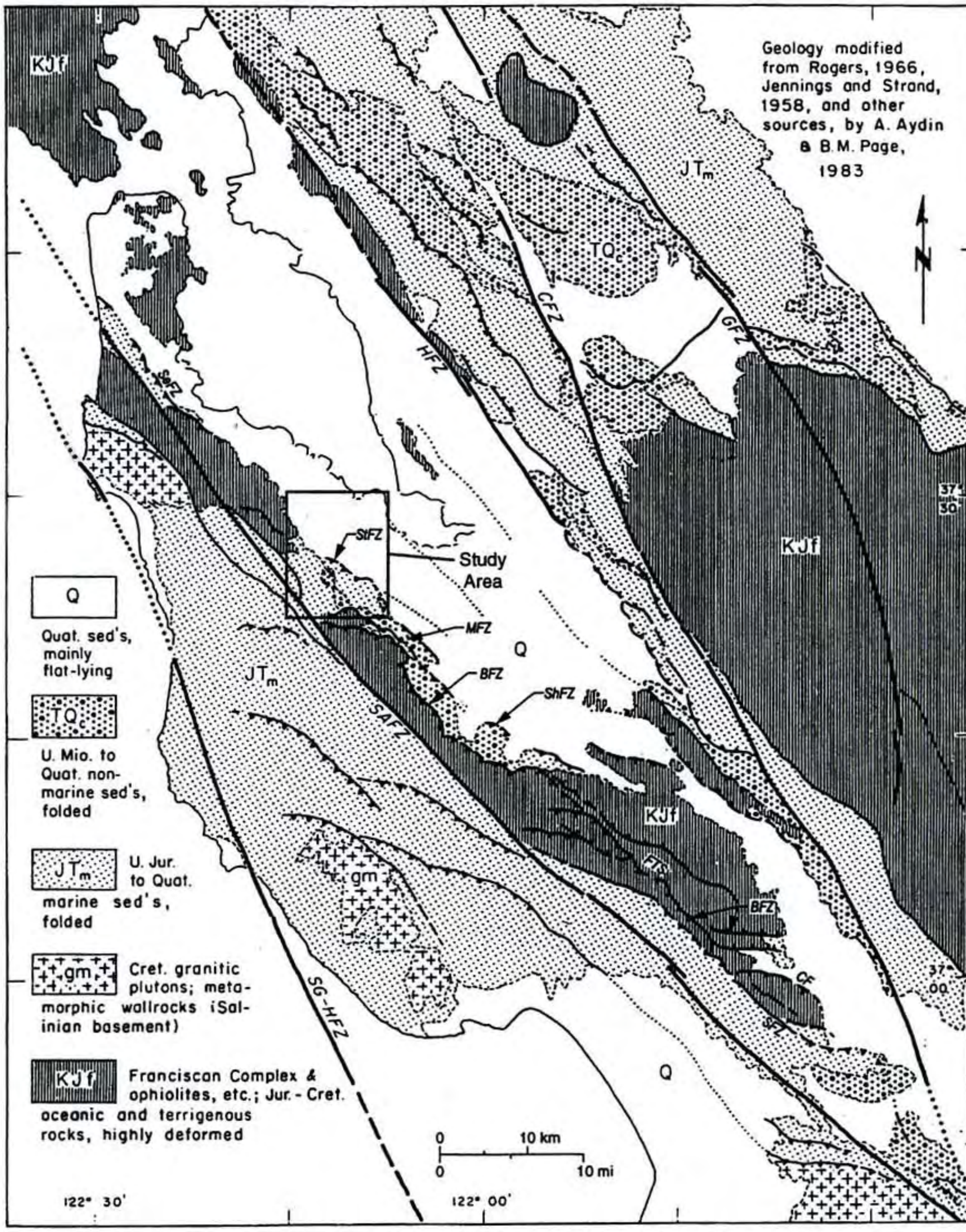
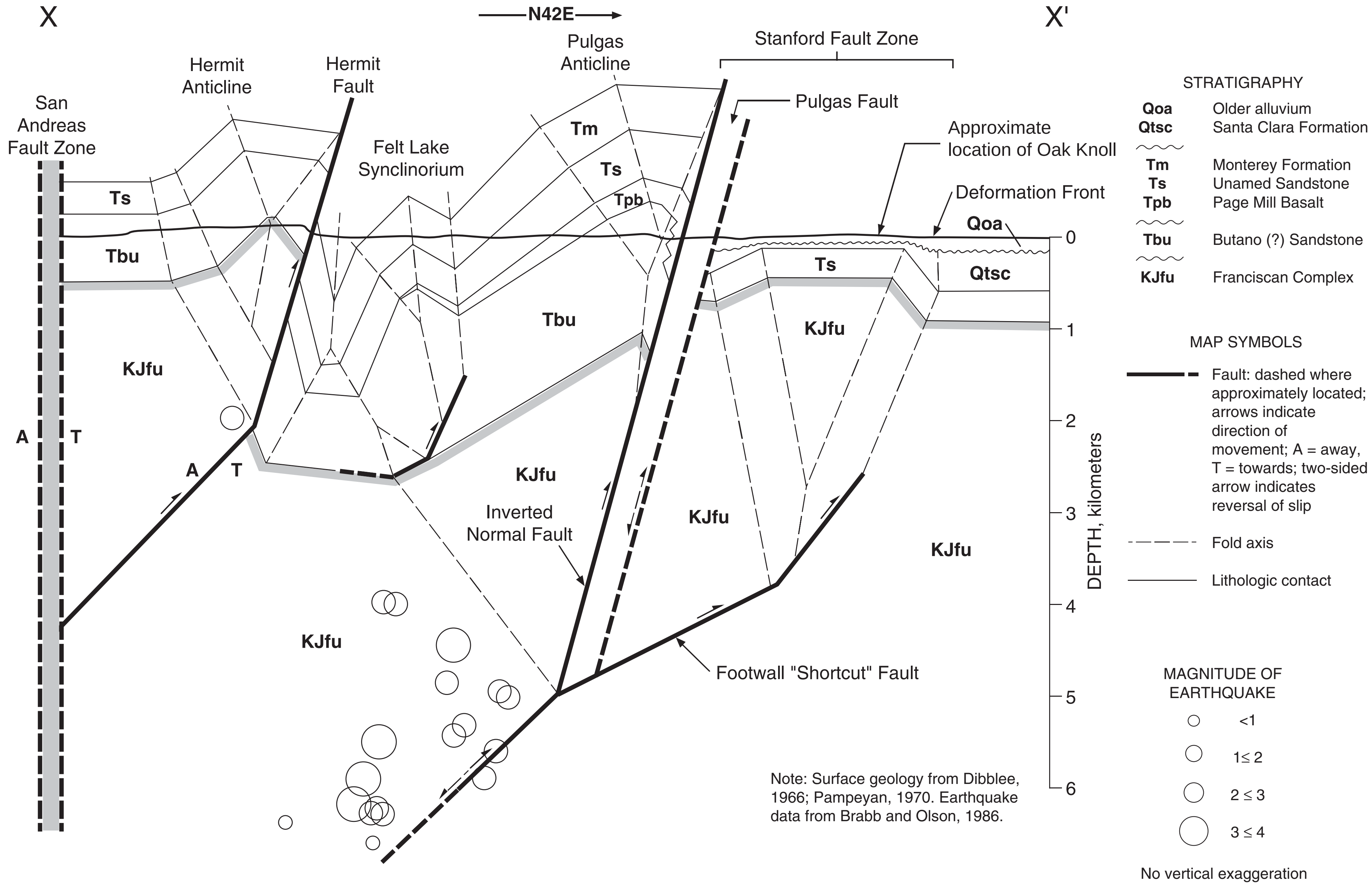


Figure 1. Simplified geologic map, San Francisco Bay Region (modifed from Aydin and Page, 1984). BFZ - Berrocal Fault zone, CFZ-Calaveras Fault zone, CF-Carnadero Fault, FTS-Foothills Thrust System, GFZ-Green Valley Fault zone, HFZ-Hayward Fault zone, MFZ-Monte Vista Fault zone, SFZ-Sargent Fault zone, SAFZ-San Andreas Fault zone, SeFZ-Serra Fault zone, SG-HFZ-San Gregorio-Hosgri Fault zone, ShFZ-Shannon Fault zone, StFZ-Stanford Fault Zone,

Base: Characterization of Quaternary contractional deformation adjacent to the San Andreas fault, Palo Alto, California, Figure 1, Simplified geologic map, San Francisco Bay Region, Angell and others, 1997 (Modified from Aydin and Page, 1984) Scale: As shown

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA		BAY AREA FAULT MAP
	PROJECT NO. 589-3L1	JUNE 2007	FIGURE A-5

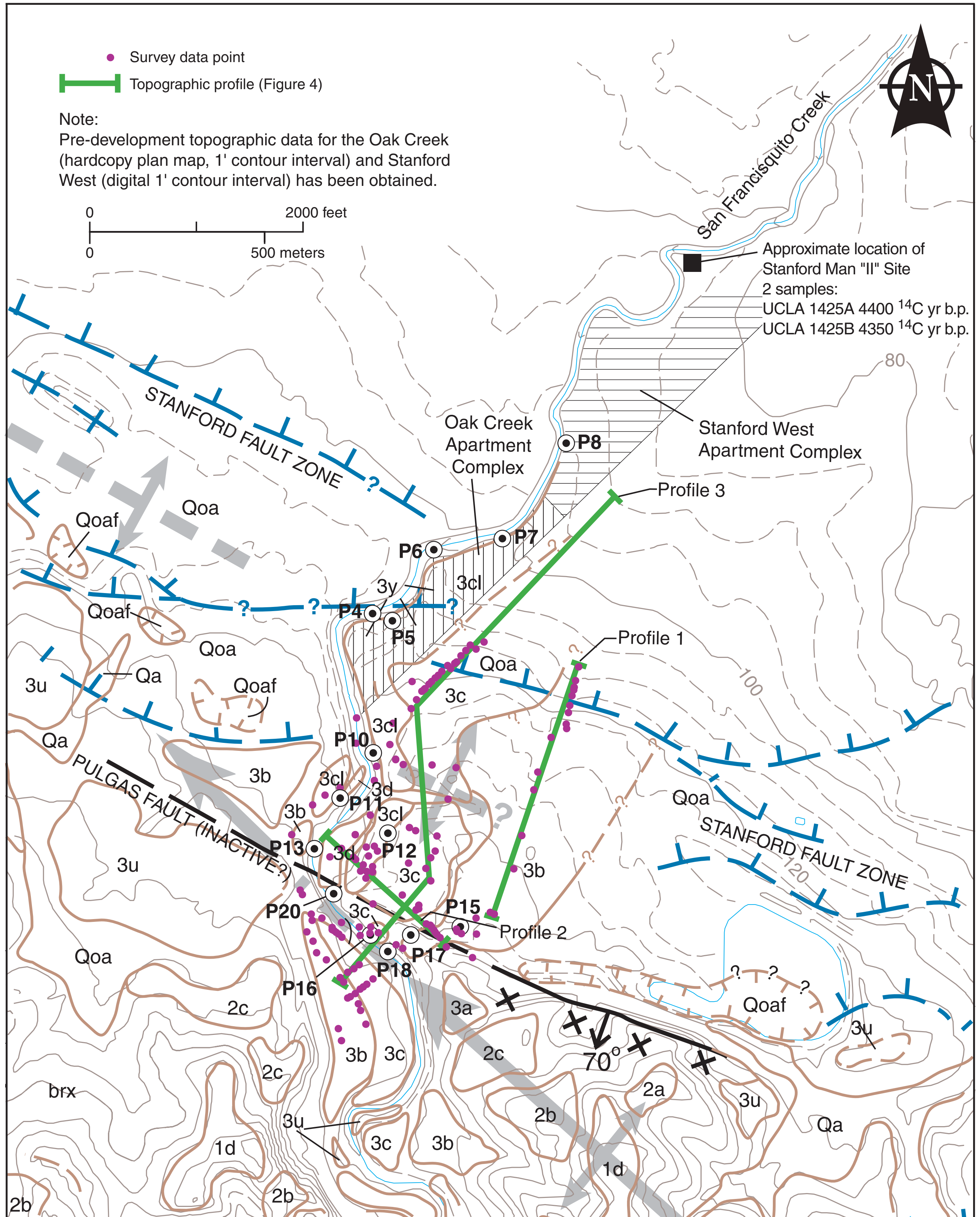


Scale: As shown

Base: Quaternary investigations to evaluate seismic source characteristics of the frontal thrust belt, Palo Alto region, California, Figure 3, Regional cross-section X-X' from Woodside to Palo Alto (from Angell and others, 1997) Bullard and others, 2004

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	REGIONAL GEOLOGIC CROSS-SECTION (ANGELL AND OTHERS, 1997)
OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA	

PROJECT NO. 598-3R2	JUNE 2007	FIGURE A-6



MAP SYMBOLS

Fault Arrow indicates dip direction of high-angle reverse fault; teeth on upper plate of thrust fault; half arrows indicate sense of lateral slip; dashed where approximately located; dip in degrees

Scarp Ticks indicate slope direction of scarp face; x's indicate bedrock scarp along range front; deformation front defined by northeastern-most scarps in alluvium.

Ridge Topographic feature within alluvial deposits northeast of range front. Locally coincident with anticlinal structures and reverse faults.

Basin Circular to elongate localized depression within alluvial deposits northeast of range front and southwest of deformation front; contains fine-grained, "ponded" alluvial deposits.

Terrace Fluvial terrace geomorphic landform; circles indicate presence of gravel on Qt2 and Qt1 surfaces; dashed where uncertain.

Uplift Axis Approximate axis of Quaternary uplift defined by distribution of Quaternary landforms, deposits and structural features; dashed where uncertain.

Anticline Location of Quaternary anticline defined by structural data within Quaternary alluvium.

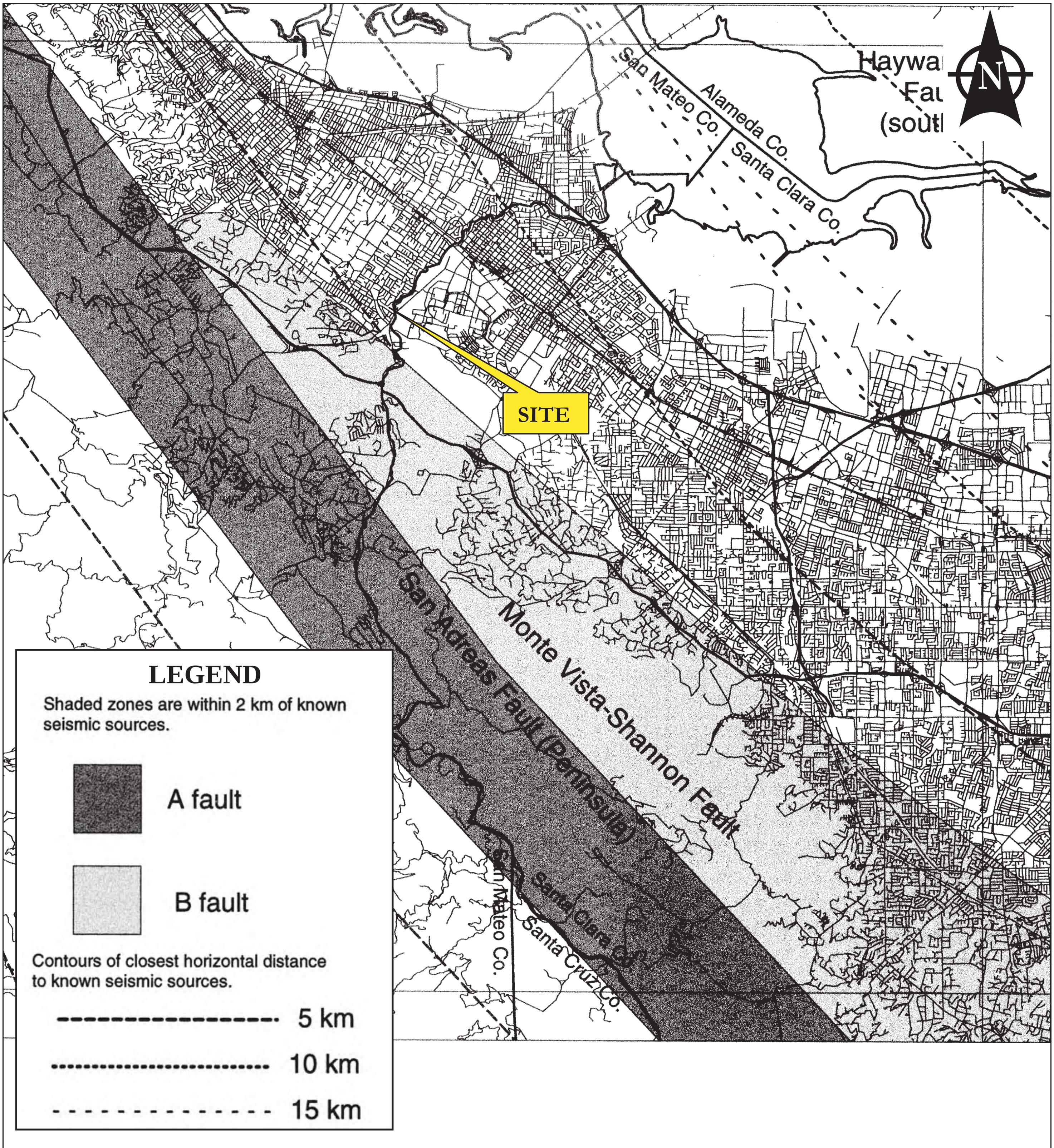
Syncline Location of Quaternary syncline defined by structural data within Quaternary alluvium.

Generalized contours of San Francisquito Creek alluvial fan; represents late Pleistocene fan surface.

Age-date sample locality

Base: Quaternary investigations to evaluate seismic source characteristics of the frontal thrust belt, Palo Alto region, California, Figure 2, Quaternary geology of frontal thrust fault system at San Francisquito Creek, Bullard and others 2004 Scale: As shown

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	LOCAL GEOLOGIC MAP (BULLARD & OTHERS, 2004)	
OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA		
PROJECT NO. 598-3R2	JUNE 2007	FIGURE A-7



Base: Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, Map E-19, California Division of Mines and Geology, 1998
Scale: 1 inch = 4 kilometers.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE SAN MATEO COUNTY, CALIFORNIA		ACTIVE FAULT NEAR-SOURCE ZONES
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE A-8

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type	Total Depth of Borehole **5 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **118 feet**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Parking Lot**	

Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0		16 21	Stiff	CL	SILTY CLAY, dark brown grading to mottled olive-brown and yellowish-brown, homogeneous, moderate plasticity, very moist (Alluvial Topsoil)	18	87	70	31	16
		15	Stiff	CL		14	115			
5					SILTY CLAY with sand, yellowish-brown, trace fine gravels, homogeneous, low to moderate plasticity, very moist (Alluvium)					
					Bottom of Boring at 5 feet bgs					
10										
15										
20										
25										
30										
35										
40										
45										
50										

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY
1895 OAK KNOLL LANE
MENLO PARK, CALIFORNIA

LOG OF BORING B-1

PROJECT NO. 598-3R1 | **JUNE 2007** | **FIGURE B-1**

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch O.D. soil bit**	Total Depth of Borehole **5 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **118 feet**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Parking Lot**	

Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0		20 27	Stiff to Very Stiff	ML	CLAYEY SILT, dark brown, homogeneous, low to moderate plasticity, very moist (Alluvial Topsoil)	15	100			
		17	Very Stiff	ML	CLAYEY SILT, olive-gray to dark brown, homogeneous, low to moderate plasticity, slightly moist (Alluvium)	11	104			
5					Bottom of Boring at 5 feet bgs					
10										
15										
20										
25										
30										
35										
40										
45										
50										

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA		LOG OF BORING B-2
PROJECT NO. 598-3R1	JUNE 2007	FIGURE B-2

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch O.D. soil bit**	Total Depth of Borehole **30 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **119 feet**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Building**	

Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0		11	Stiff	Asphalt	~2" AC over ~6" AB	14	86			
		14		ML	CLAYEY SILT, very dark brown, trace fine sands, gravels, and sandstone fragments, moderate plasticity, very moist (Alluvium)	12	113			
5		14	Stiff	CL		10	114			
					SANDY CLAY, mottled light yellowish-brown and dark brown, minor fine subangular gravels up to ~1.5" diameter, homogeneous, low plasticity, very moist (Alluvium)					
10		12	Stiff	CL		20	116			
15		27	Medium Dense	SC	CLAYEY SAND, dark yellowish-brown, trace subangular gravels up to ~0.5" diameter, homogeneous, low plasticity, very moist (Alluvium)	14	117	33		
20		17	Soft*	BR	CLAYEY SAND with fine subangular to subrounded gravels up to ~0.5" diameter, dark yellowish-brown, homogeneous, very moist (Alluvium)	14 / 32	96 / 85			
25		32			SILTSTONE, gray with pale yellow mottling to dark gray, moderately expansive, severely weathered, homogeneous, very moist (Bedrock)	20	99			
30		61			*denotes hardness of bedrock (see Figure B-9)	20	94			
					Bottom of Boring at 30 feet bgs					
35										
40										
45										
50										

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY
1895 OAK KNOLL LANE
MENLO PARK, CALIFORNIA

LOG OF BORING B-3

PROJECT NO. 598-3R1 | **JUNE 2007** | **FIGURE B-3**

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch O.D. soil bit**	Total Depth of Borehole **30 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **119 feet**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Building**	

Depth, feet	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0	19	Stiff	CL	FILL: SANDY CLAY, abundant wood fragments & rootlets, heterogeneous, very moist	19	100			
	21	Stiff to Very Stiff	CL	SANDY CLAY, yellowish-brown, abundant subrounded pebbles and fine subangular gravels, homogeneous, low plasticity, very moist (Alluvium)	16				
5	10				16	111			
10	24				10	115			
15	22				14	118			
		Hard	CL	GRAVELLY CLAY, dark brown, gravels are subangular and up to ~1" diameter, trace yellowish brown silty sand lenses, very moist (Alluvium)					
20	59				11	124			
25	35	Dense	GW	SANDY GRAVEL, dark brown, gravels are up to ~1" diameter, minor silts and clays, well graded, saturated (Alluvium)	14	139	14		
30	41	Soft*	BR	FINE-GRAINED SILTY SANDSTONE, light yellowish brown, severely weathered, very moist (Bedrock) *denotes hardness of bedrock (see Figure 9)	19	100			
35				Bottom of Boring at 30 feet bgs					
40									
45									
50									

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA		LOG OF BORING B-4
PROJECT NO. 598-3R1	JUNE 2007	FIGURE B-4

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch O.D. soil bit**	Total Depth of Borehole **50 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **118 feet**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Building**	

Depth, feet	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0	16	Stiff	CL	SILTY CLAY, dark brown, homogeneous, trace rootlets, very moist (Alluvial Topsoil)	17	106			
	33	Very Stiff	CL	SANDY CLAY with fine sand and subrounded gravels up to ~1.5" diameter, dark yellowish-brown, trace shale clasts up to 1.5" diameter, homogeneous, low to moderate plasticity, very moist (Alluvium)	11	115			
	23				12	116			
5	25				14	110			
10	21				17	106			
15	81/8"				14	111	20		
20	28	Medium Dense	SC	CLAYEY SAND, yellowish-brown to olive-brown, trace fine gravels, homogeneous, very moist (Alluvium)	17	113			
25	23	Soft*	BR	CLAYEY SILTSTONE, olive-gray to yellowish-brown, trace sandstone clasts, severely weathered, highly expansive, very moist (Bedrock) *denotes hardness of bedrock (see Figure 9)					
30	29				23	95			
35	26	Soft*	BR	CLAYEY SILTSTONE, bluish-gray to dark bluish-gray, highly expansive, severely weathered, very moist (Bedrock) *denotes hardness of bedrock (see Figure 9)	26	91			
40	35				18	104			
45	93/11"				17	122			
50				Bottom of Boring at 50 feet bgs					

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MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY
1895 OAK KNOLL LANE
MENLO PARK, CALIFORNIA

LOG OF BORING B-5

PROJECT NO. 598-3R1 | **JUNE 2007** | **FIGURE B-5**

Date(s) Drilled **February 22, 2007**	Logged By **WPC**	Checked By **JAS**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch O.D. soil bit & rock bit**	Total Depth of Borehole **50 feet bgs**
Drill Rig Type **Truck-Mounted Mobile B-40**	Drilling Contractor **Exploration Geoservices, Inc.**	Approximate Surface Elevation **119 feet**
Groundwater Level and Date Measured **27.5 feet ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, wireline**
Borehole Backfill **Cement Slurry**	Location **Area of Proposed New Building**	

Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
0		44	Very Stiff	AC	Pavement Section: ~2" Ashpaltic Concrete (AC) o/~4" Aggregate Baserock	8	99			
		27		CL		8	120			
5		17			SANDY SILT with clay, mottled dark brown and pale olive-brown grading to yellowish-brown, minor fine gravels, low to moderate plasticity, very moist (Alluvium).	7				
		20				7	124	54		
10		61	Very Dense	GP	SANDY GRAVEL, yellowish to reddish-brown, minor silts and clays, poorly graded, gravels are up to ~2" diameter, homogeneous, slightly moist (Alluvium)	16				
15		53				12	125	12		
20						9				
25		51	Very Dense	GW	GRAVELLY CLAY with SAND, yellowish to reddish-brown, homogeneous, very moist (Alluvium) (ATD)	12	123	42		
30		39	Soft*	BR	CONGLOMERATE, bluish gray, severely weathered, very moist (Bedrock)	16	127			
35		90/11"			*denotes hardness of bedrock (see Figure 9)	16	123			
40		95/8"				16				
45		50/6"				26				
50					Bottom of Boring at 50 feet bgs					

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA		LOG OF BORING B-6
PROJECT NO. 598-3R1	JUNE 2007	FIGURE B-6

Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (pcf)	Percent Fines	LL, %	PI, %
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 **Depth, feet:** Depth in feet below the ground surface.

2 **Sample Type:** Type of soil sample collected at the depth interval shown.

3 **Sampling Resistance, blows/foot:** Number of blows to advance driven sampler foot (or distance shown) beyond seating interval using the hammer identified on the boring log.

4 **Relative Consistency:** Relative consistency of the subsurface material.

5 **USCS Symbol:** USCS symbol of the subsurface material.

6 **MATERIAL DESCRIPTION:** Description of material encountered. May include consistency, moisture, color, and other descriptive text.

7 **Water Content, %:** Water content of the soil sample, expressed as percentage of dry weight of sample.

8 **Dry Density (pcf):** In-place Dry Density of Soil

9 **Percent Fines:** The percent fines (soil passing the No. 200 Sieve) in the sample. WA indicates a Wash Sieve, SA indicates a Sieve Analysis.

10 **LL, %:** Liquid Limit, expressed as a water content

11 **PI, %:** Plasticity Index, expressed as a water content

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

TYPICAL MATERIAL GRAPHIC SYMBOLS

Asphaltic Concrete
Sandstone
Well graded GRAVEL (GW)
Poorly graded GRAVEL (GP)
Well graded GRAVEL with Silt (GW-GM)
Well graded GRAVEL with Clay (GW-GC)
Poorly graded GRAVEL with Silt (GP-GM)
Poorly graded GRAVEL with Clay (GP-GC)
Silty GRAVEL (GM)
Clayey GRAVEL (GC)
Well graded SAND (SW)
Poorly graded SAND (SP)
Well graded SAND with Silt (SW-SM)
Well graded SAND with Clay (SW-SC)
Poorly graded SAND with Silt (SP-SM)
Poorly graded SAND with Clay (SP-SC)
Silty SAND (SM)
Clayey SAND (SC)
SILT, SILT w/SAND, SANDY SILT (ML)
Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
SILT, SILT w/SAND, SANDY SILT (MH)
Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
SILT, SILT with SAND, SANDY SILT (ML-MH)
Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL-CH)
SILTY CLAY (CL-ML)
Lean CLAY/PEAT (CL-OL)
Fat CLAY/SILT (CH-MH)
Fat CLAY/PEAT (CH-OH)
Silty SAND to Sandy SILT (SM-ML)
Silty SAND to Sandy SILT (SM-MH)
Clayey SAND to Sandy CLAY (SC-CL)
Clayey SAND to Sandy CLAY (SC-CH)
SILT to CLAY (CL/ML)
CLAYEY SAND to POORLY GRADED SAND (SC/SP)
Silty to Clayey SAND (SC/SM)

TYPICAL SAMPLER GRAPHIC SYMBOLS

2 inch-OD Unlined Split Spoon (SPT)
2.5 inch-OD Unlined Split Spoon
3 inch-OD Unlined Split Spoon
Shelby Tube (thin-walled, fixed head)
Grab Sample
Bulk Sample
Pitcher Sample
Other Sampler

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting a given time)
Minor change in material properties within a stratum
Inferred or gradational contact between strata
? Queried contact between strata

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY 1895 OAK KNOLL LANE MENLO PARK, CALIFORNIA		KEY TO BORING LOGS
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE B-7

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or 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.
	SAND	CLEAN SAND (< 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		MH	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

RELATIVE DENSITY

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

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

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

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Shear strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

WEATHERING

Fresh
Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.

Very Slight
Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.

Slight
Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.

Moderate
Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.

Moderately Severe
All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.

Severe
All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.

Very Severe
All rock except quartz discolored and stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.

Complete
Rock reduced to "soil". Rock fabric not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

HARDNESS

Very hard
Cannot be scratched with knife or sharp pick. Hand specimens requires several hard blows of geologist's hammer.

Hard
Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.

Moderately Hard
Can be scratched with knife or pick. Gouges or grooves to 1/4 inch deep can be excavated by hard blow of point of a geologist's pick. Hard specimen can be detached by moderate blow.

Medium
Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.

Soft
Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be brocken by finger pressure.

Very Soft
Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

JOINT BEDDING AND FOLIATION SPACING

Spacing	Joints	Bedding and Foliation
Less than 2 in.	Very Close	Very Thin
2 in. to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately Close	Medium
3 ft. to 10 ft.	Wide	Thick
More than 10 ft.	Very Wide	Very Thick

ROCK QUALITY DESIGNATOR (RQD)

RQD, as a percentage	Descriptor
Exceeding 90	Excellent
90 to 75	Good
75 to 50	Fair
50 to 25	Poor
Less than 25	Very Poor

APPENDIX C

LABORATORY TESTS

Samples from the subsurface exploration were selected for tests to establish the physical and engineering properties of the soils. The tests performed are briefly described below.

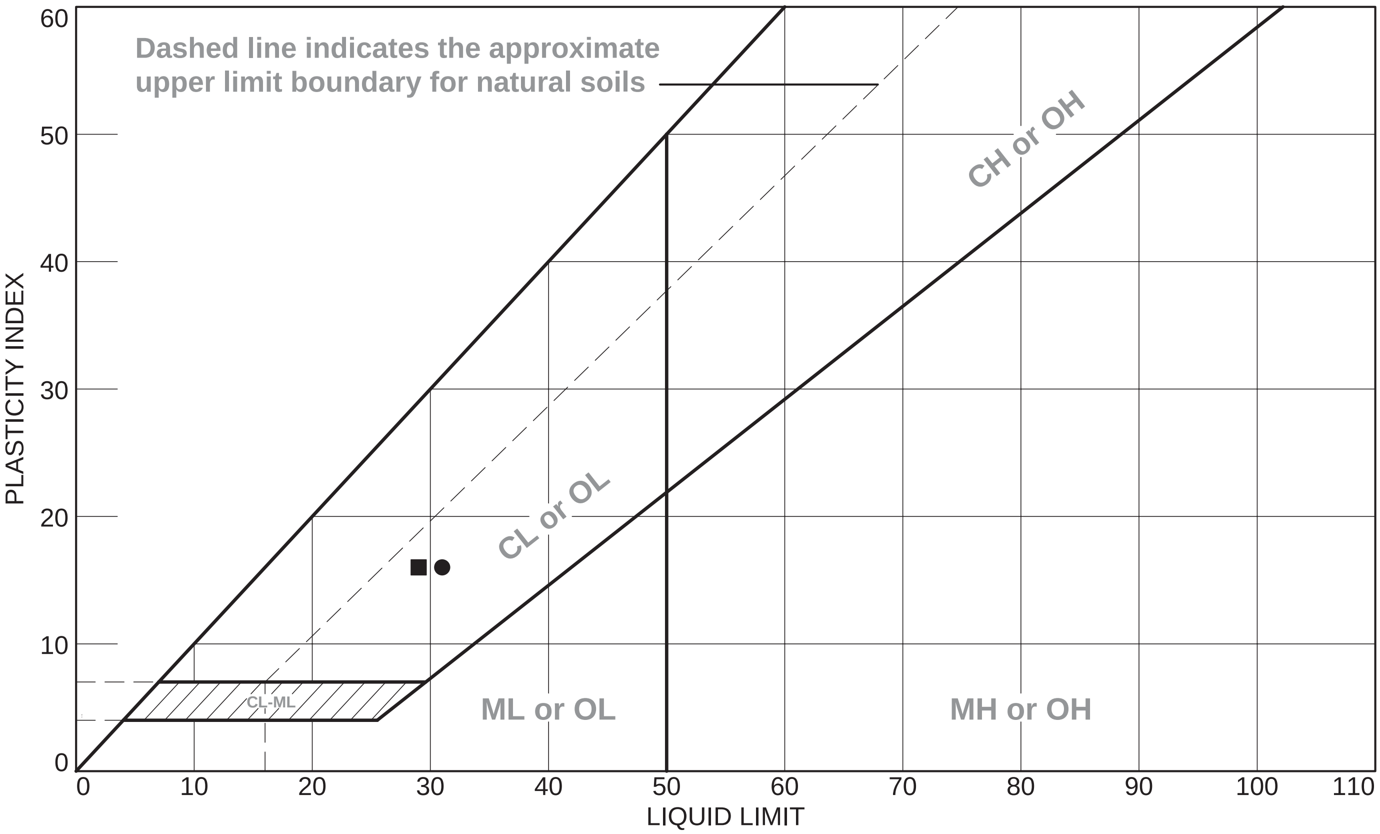
Natural moisture content and dry density was determined on most samples recovered from the boring. The samples were initially trimmed to obtain volume and wet weight measurements and subsequently dried in accordance with ASTM D2216. After drying, the weight of each sample was obtained to determine the moisture content and dry density representative of field conditions and time the samples were collected. The results are presented on the boring logs at the appropriate sample depths.

The Atterberg Limits were determined on two samples in accordance with ASTM D 4318. The Atterberg limits are the moisture content within which the soil is workable or plastic. The results of these tests are presented in Figure C-1 and on the boring logs, at the appropriate sample depths.

The amount of silt and clay sized material present was determined on seven samples in accordance with ASTM D1140. The results are presented on the boring logs at the appropriate sample depths.

A direct shear test was performed by Cooper Testing Laboratory and the results of this test are presented as Figure C-2 and on the boring logs, at the appropriate sample depth. An R-Value test was performed by Cooper Testing Laboratory and the results of this test are presented as Figure C-3. Corrosivity testing was performed by Cooper Testing Laboratory and the results of this test are presented as Figure C-4.

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring 1	1	1' to 1.5'	18.2	15	31	16	CL
■	Boring 5	2	1.5' to 2'	17.3	13	29	16	CL

MURRAY ENGINEERS INC
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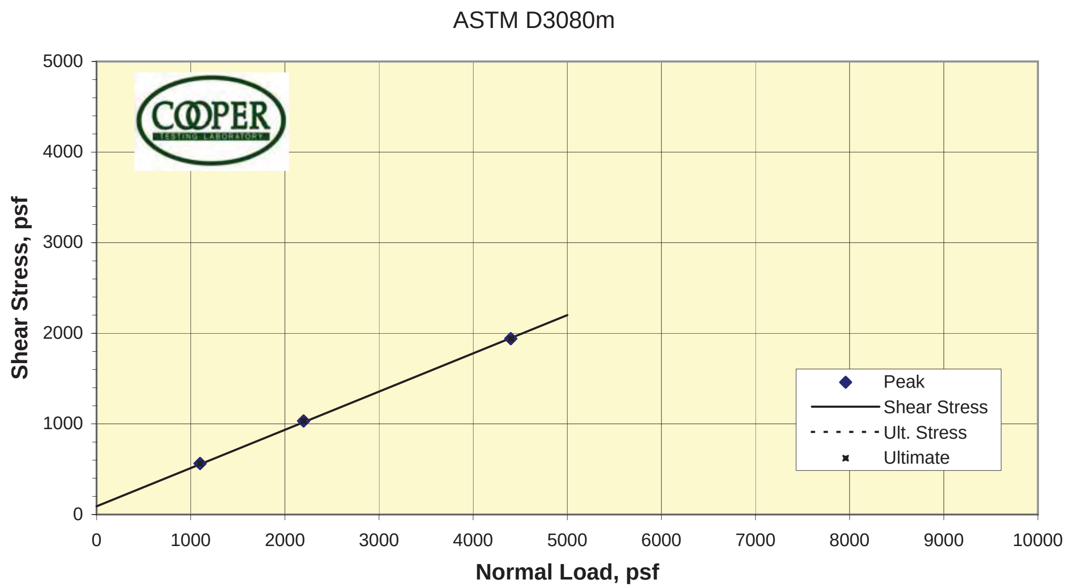
MENLO PARK CITY SCHOOL DISTRICT
OAK KNOLL ELEMENTARY SCHOOL
MENLO PARK, CALIFORNIA

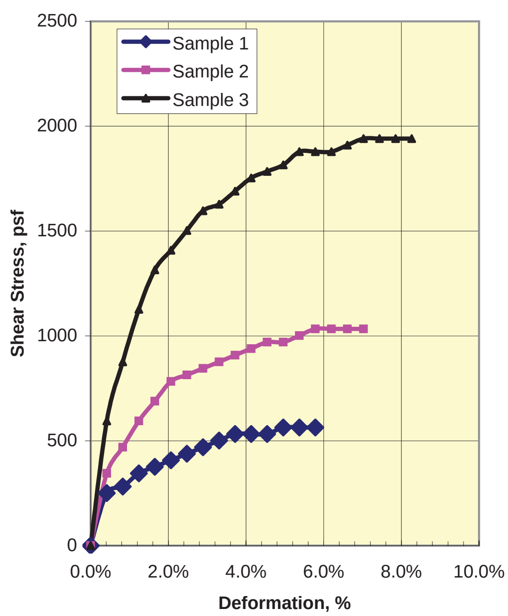
PLASTICITY CHART

PROJECT NO. 598-3R1 | JUNE 2007 | FIGURE C-1

Direct Shear



P. Phi (degrees)	22.9	Ult. Phi (degrees)	
P. Cohesion(psf)	90	Ult. Cohesion (psf)	



	Sample Data: Initial			
	1	2	3	4
Moisture %	14.4	14.6	15.7	
Dry Dens., pcf	89.2	92.3	93.2	
Void Ratio	0.890	0.827	0.809	
Saturation %	43.7	47.8	52.5	
Diameter	2.42	2.42	2.42	
Height	1.00	1.00	1.00	
	Sample Data: At Test			
Moisture %	23.8	21.8	21.6	
Dry Dens., pcf	94.5	97.8	100.9	
Void Ratio	0.783	0.723	0.671	
Saturation %	82.0	81.5	86.9	
Diameter	2.42	2.42	2.42	
Height	0.94	0.94	0.92	
Normal Stress, psf	1100	2200	4400	
Shear Stress, psf	564	1033	1941	
Strengths picked at	5.8%	7.0%	8.3%	
Ult. Stress, psf	564	1033	1941	
Strain Rate, in/min	0.020	0.020	0.020	

CTL #	560-035
Client:	Murray Engineers
Project	MPCSD Oak Knoll - 589-3
Tested By:	MD
Reduced By:	RU/MD
Date:	3/22/2007

Specimen #	Boring:	Sample:	Depth, ft:	Visual Soil Classification
1	B-2	1	1-1.5	Dark Brown Sandy CLAY w/ organics
2	B-2	1	1-1.5	Dark Brown Sandy CLAY w/ organics
3	B-2	1	1-1.5	Dark Brown Sandy CLAY w/ organics
4				

Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA		DIRECT SHEAR TEST CHART
PROJECT NO. 598-3R1	JUNE 2007	FIGURE C-2

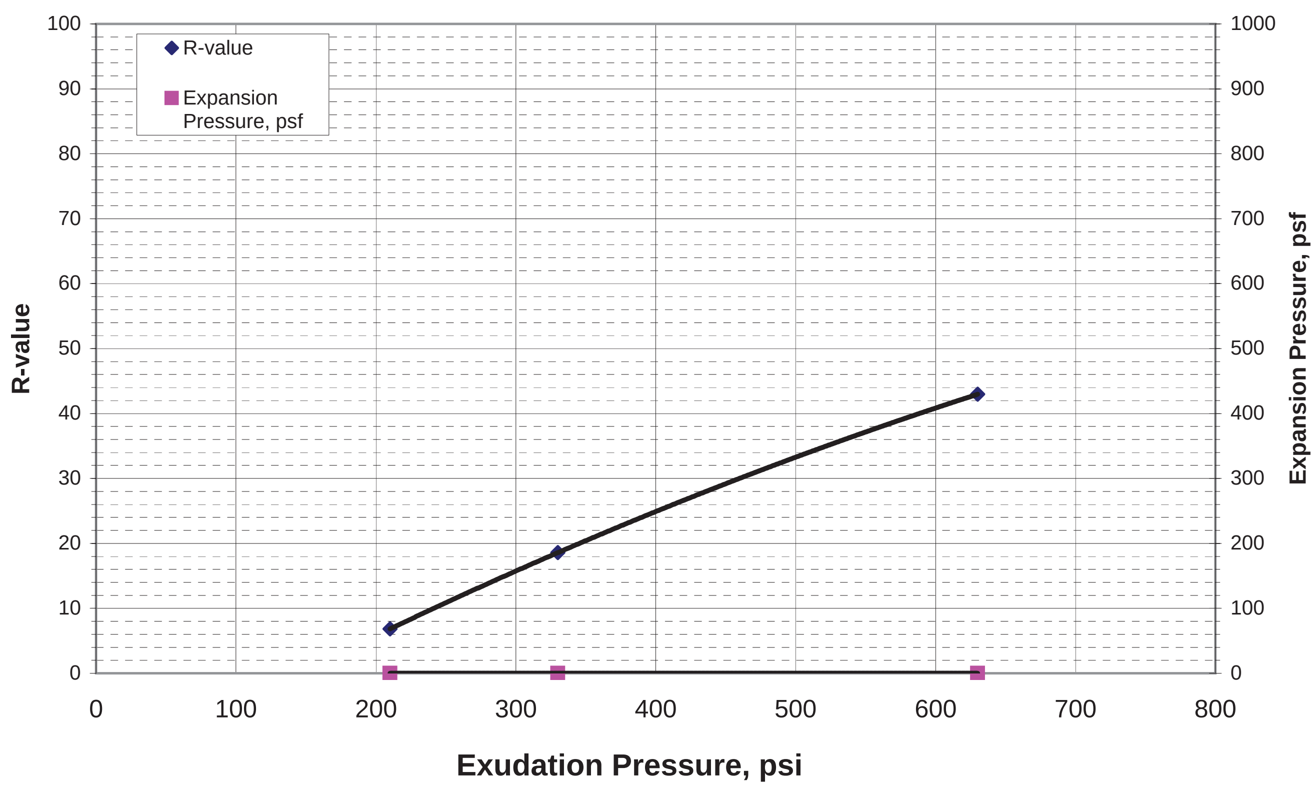
R-value Test Report (Caltrans 301)

COOPER TESTING LABORATORY

Job No.:	560-035	**Date:**	03/20/07	**Initial Moisture,**	12.7%
Client:	Murray Engineers	**Tested**	MD	**R-value by Stabilometer**	16
Project:	MPCSD Oak Knoll - 589-3	**Reduced**	RU		
Sample	B-1 @ 0-5'	**Checked**	DC	**Expansion Pressure**	0 psf
Soil Type:	Brown Sandy CLAY				

Specimen Number	A	B	C	D
Exudation Pressure, psi	630	330	210	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	25	62	85	
Weight of Soil & Mold, grams	3259	3240	3264	
Weight of Mold, grams	2108	2101	2098	
Height After Compaction, in.	2.65	2.73	2.76	
Moisture Content, %	15.0	18.5	20.6	
Dry Density, pcf	114.4	106.6	106.0	
Expansion Pressure, psf	0.0	0.0	0.0	
Stabilometer @ 1000				
Stabilometer @ 2000	90	126	144	
Turns Displacement	3	3.46	4.2	
R-value	43	19	7	

Remarks:



MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA		R-VALUE TEST REPORT
PROJECT NO. 598-3R1	JUNE 2007	FIGURE C-3

Corrosivity Test Summary

CTL # 560-035 **Date:** 3/13/2007 **Tested By:** PJ **Checked:** PJ

Client: Murray Engineers **Project:** MPCSD Oak Knoll **Proj. No:** 589-3

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
B-4	2	1-1.5	5,143	-	-	8	528	0.0528	5.5	229	-	22.2	Very Dark Grayish Brown Sandy CLAY

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

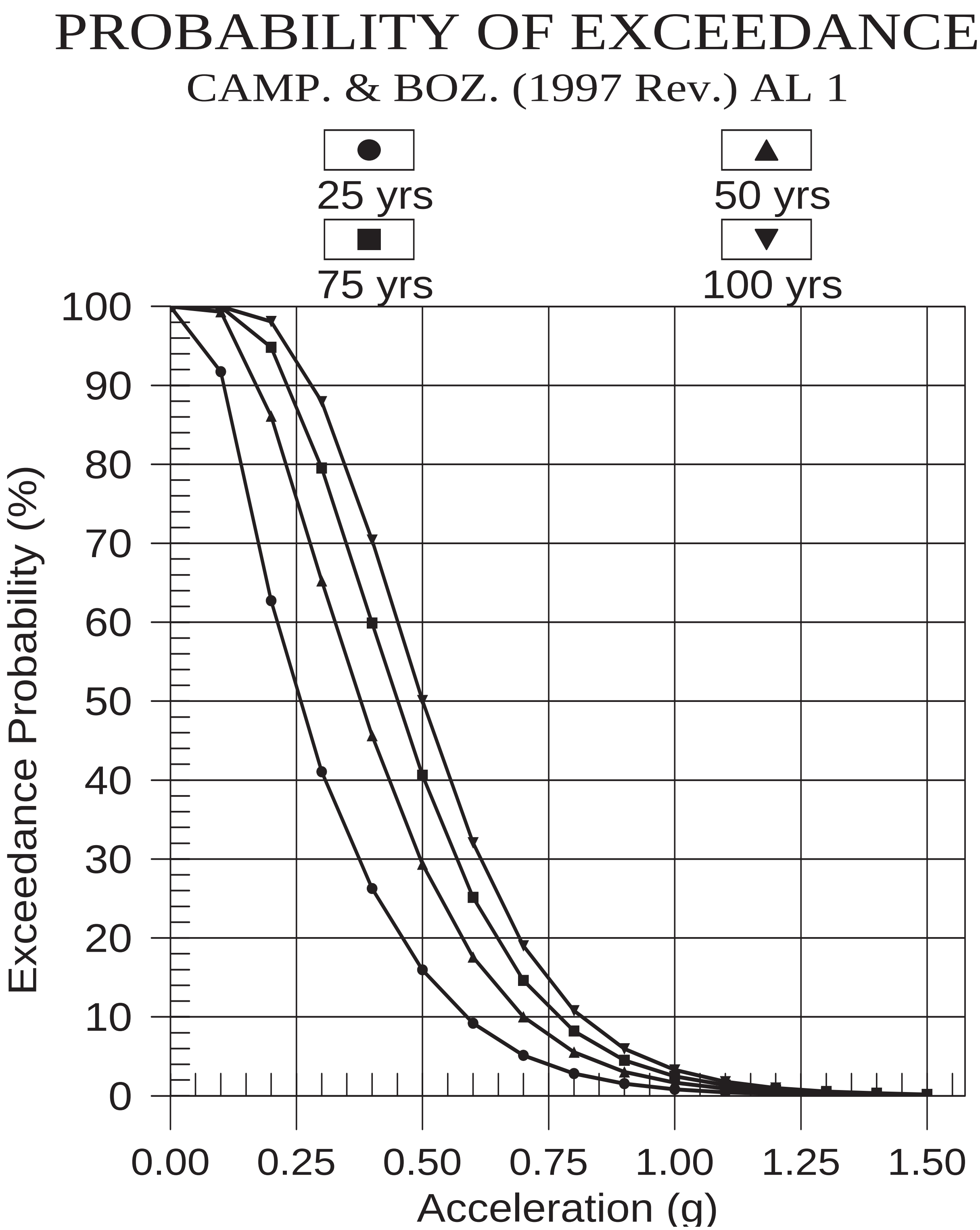
OAK KNOLL ELEMENTARY SCHOOL
MENLO PARK CITY SCHOOL DISTRICT
MENLO PARK, CALIFORNIA

CORROSIVITY TEST SUMMARY

PROJECT NO. 598-3R1 | JUNE 2007 | FIGURE C-4

APPENDIX D

SUMMARY OF LIQUEFACTION ANALYSES

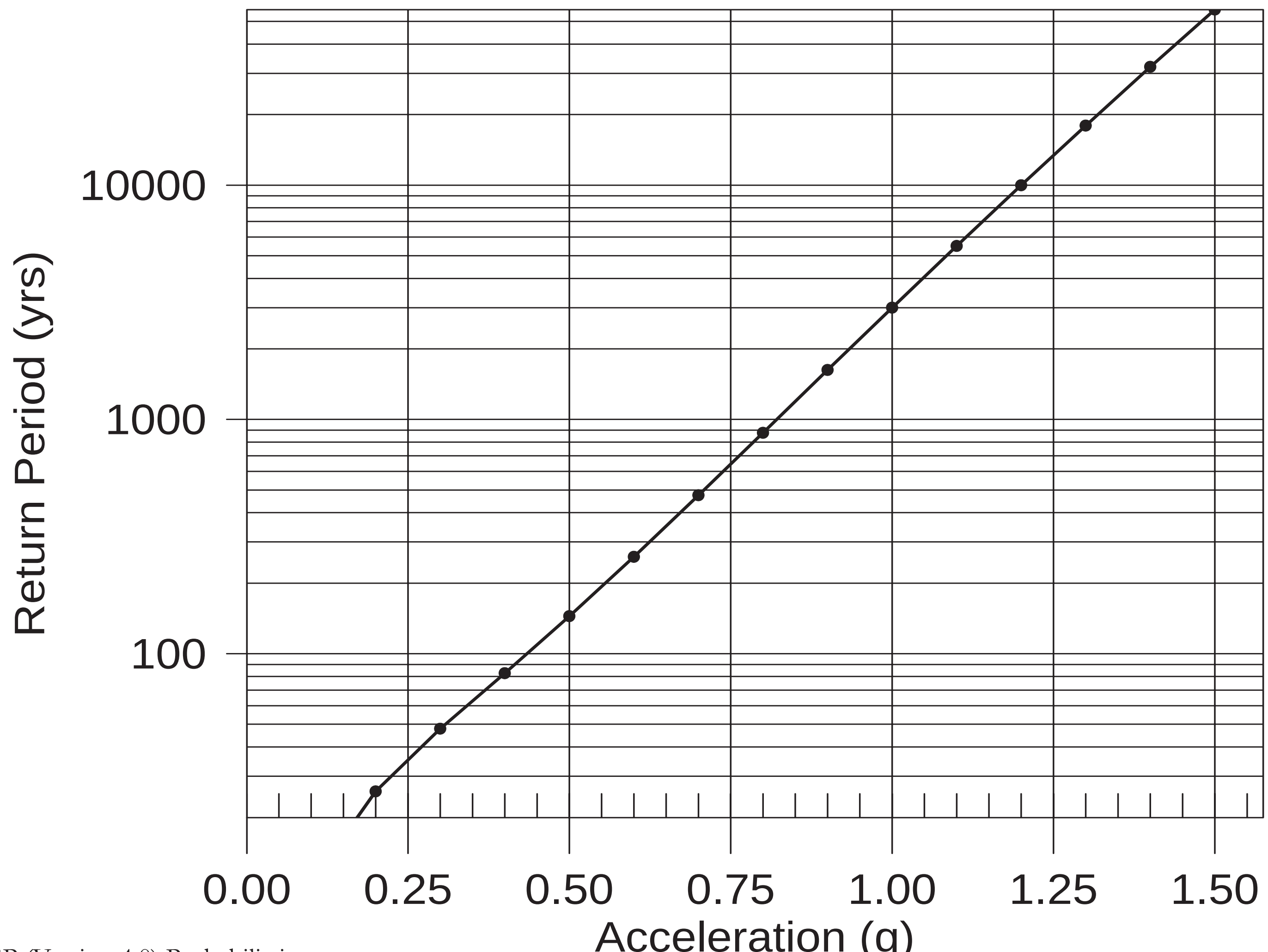


Source: FRISKSP (Version 4.0) Probabilistic Seismic Hazard Analysis Software

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA		PROBABILITY OF EXCEEDENCE DIAGRAM
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE D-1

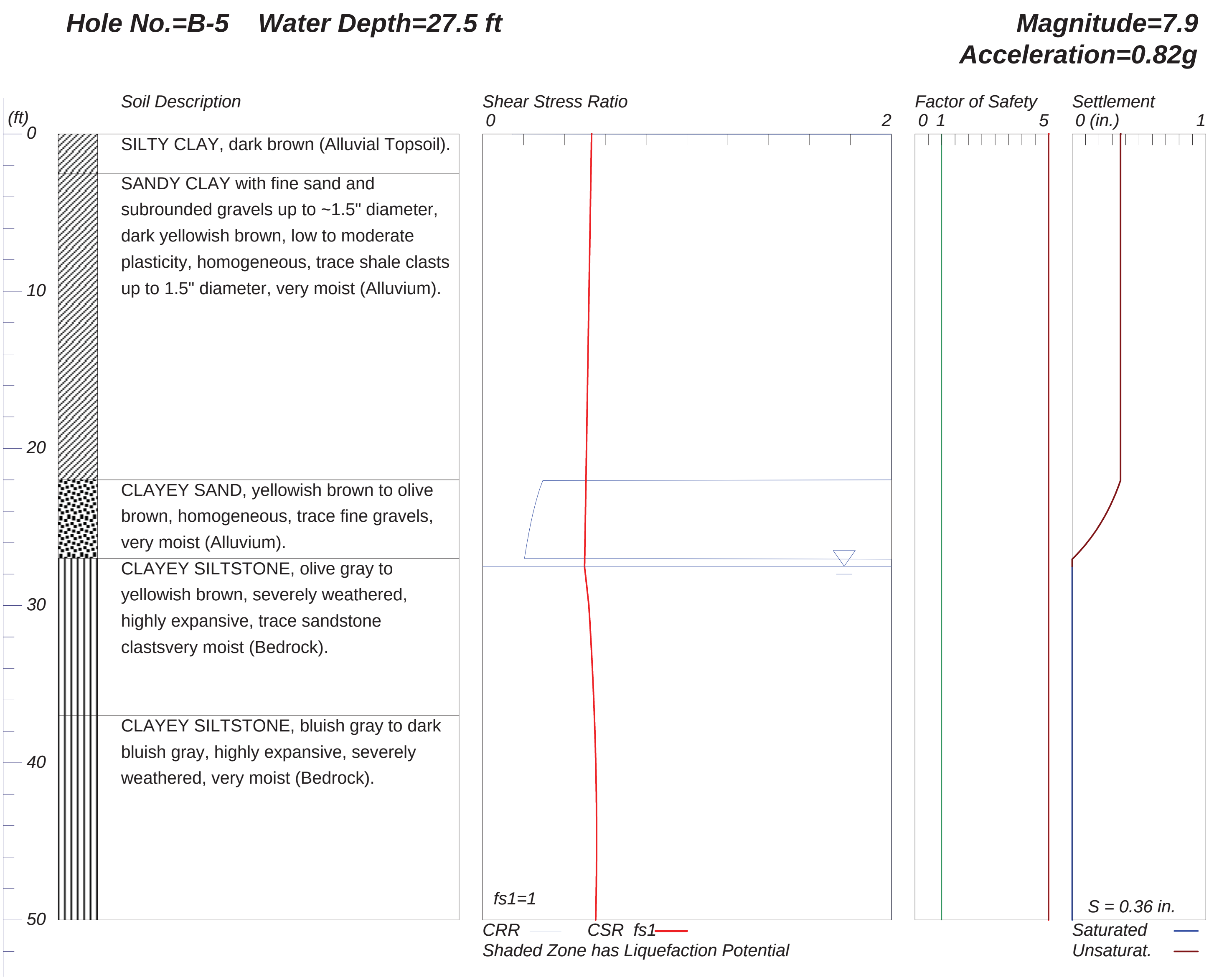
RETURN PERIOD vs. ACCELERATION

CAMP. & BOZ. (1997 Rev.) AL 1



Source: FRISKSP (Version 4.0) Probabilistic
Seismic Hazard Analysis Software

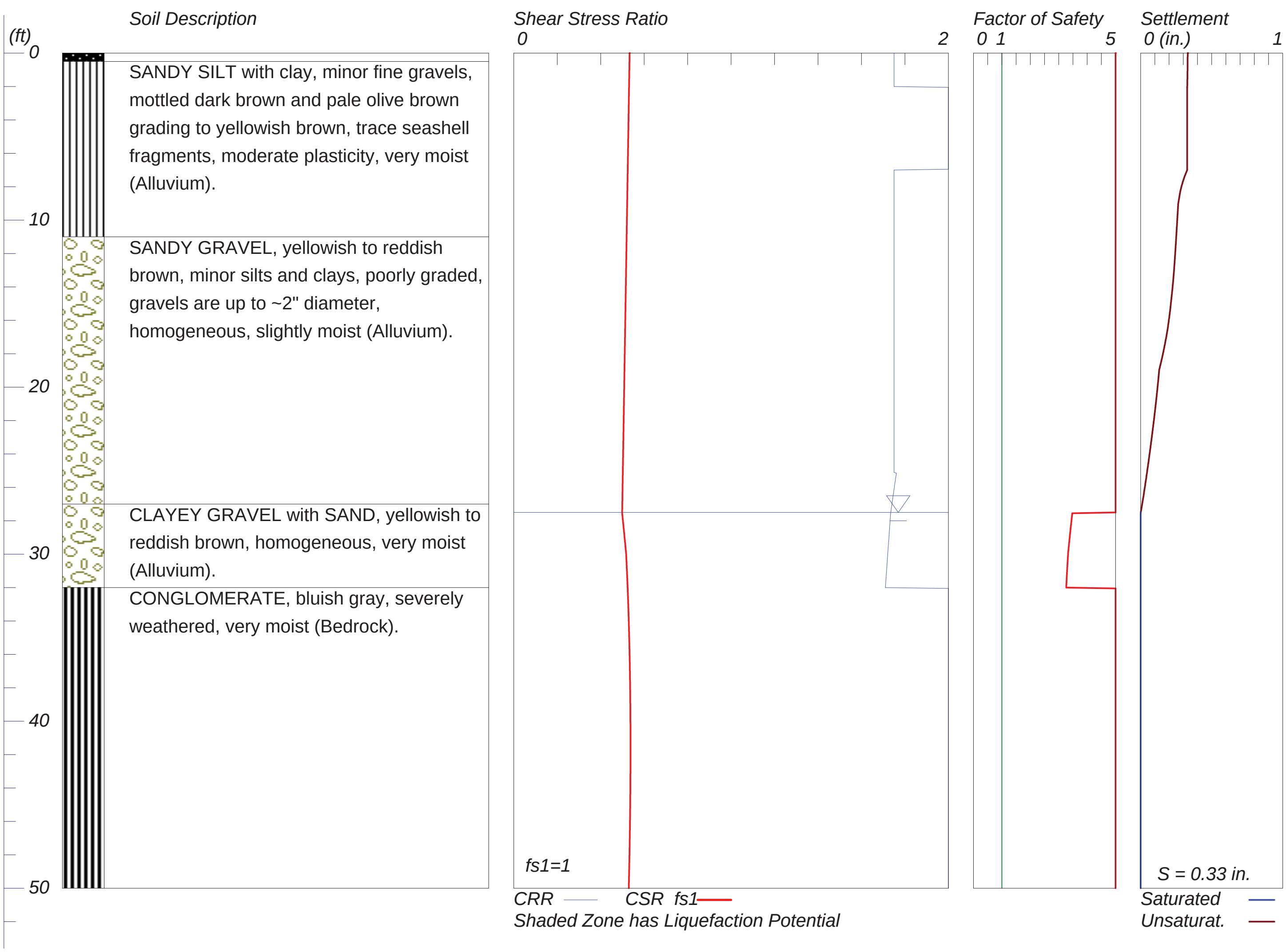
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA		RETURN PERIOD VS. ACCELERATION
	PROJECT NO. 598-3R1	JUNE 2007	FIGURE D-2



MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA		LIQUEFACTION HAZARD ANALYSIS BORING B-5
PROJECT NO. 598-3R1	JUNE 2007	FIGURE D-3

Hole No.=B-6 Water Depth=27.5 ft **Magnitude=7.9**
Acceleration=0.82g



MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	OAK KNOLL ELEMENTARY SCHOOL MENLO PARK CITY SCHOOL DISTRICT MENLO PARK, CALIFORNIA	LIQUEFACTION HAZARD ANALYSIS BORING B-6
PROJECT NO. 598-3R1	JUNE 2007	FIGURE D-4