APPENDIX F

LANDSET GEOTECHNICAL REPORT

ENGINEERS, INC.
ENGINEERING - LAND PLANNING
SURVEYING - ENVIRONMENTAL CONSULTING

GEOLOGIC HAZARDS REPORT AND SOIL ENGINEERING FEASIBILTY INVESTIGATION FOR RIVER VIEW AT LAS PALMAS (APN 139-211-035) END OF WOODRIDGE COURT MONTEREY COUNTY, CALIFORNIA

PROJECT 1272-01

Prepared for

RIVER VIEW AT LAS PALMAS, LLC
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MARCH 2014

LANDSET ENGINEERS, INC.

ENGINEERING - LAND PLANNING
SURVEYING - ENVIRONMENTAL CONSULTING

March 7, 2014 File No.: 1272-01

Mr. Garrett Shingu
River View at Las Palmas, LLC
c/o Anthony Lombardo & Associates
450 Lincoln Avenue, Suite 101
Salinas, California 93901

Attention: Mr. Dale Ellis

SUBJECT: **GEOLOGIC HAZARDS REPORT & SOILS ENGINEERING FEASIBILITY INVESTIGTION**
River View at Las Palmas (APN 139-211-035)
End of Woodridge Court
Las Palmas Ranch Area, Monterey County

Dear Mr. Shingu:

In accordance with your authorization, Landset Engineers, Inc. has completed a geologic hazards report and soil-engineering feasibility investigation for the proposed River View at Las Palmas senior housing development located at the end of Woodridge Court in the Las Palmas Ranch area, Monterey County, California. This report presents the results of our field investigation, laboratory testing, along with our preliminary conclusions and recommendations for site development.

It is our opinion that the proposed development is feasible from a geologic and soil engineering standpoint. The recommendations included in this report are preliminary and contingent upon further design development. *It is recommended that an additional design level soil engineering investigation should be performed once preliminary development plans have been completed and locations & types of structures, and anticipated loads are known.* The conclusions and recommendations included herein are based upon applicable standards at the time this report was prepared.

It has been a pleasure to be of service to you on this project. If you have any questions regarding the attached report, please contact the undersigned at (831) 443-6970.

Respectfully submitted,
LandSet Engineers, Inc.

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INTRODUCTION

This report summarizes the findings, conclusions, and recommendations for our geologic hazards report and soil engineering feasibility investigation for the River View at Las Palmas senior housing development (hereafter referred to as the site) located at the end of Woodridge Court in the Las Palmas Ranch area, Monterey County, California (see Vicinity Map, Figure 1).

PURPOSE AND SCOPE OF SERVICES

Geologic Hazards Report. This report addresses the feasibility of the development for the site from a geologic viewpoint, with emphasis on the potential for geologic and seismic-related hazards. Our studies included the following:

A. Research, review, and evaluation of data from relevant reports and maps.

B. Examination and interpretation of four sets of stereo aerial photographs of the area.

C. Geological site reconnaissance.

D. Geologic mapping of the subject property based on our site reconnaissance and review of aerial photographs and maps.

E. Development of a site geologic map and geologic cross-sections

F. Analysis of the data generated from by a certified engineering geologist and preparation of a written report in accordance with the California Geological Survey guidelines. The report addresses the following:

- *Site geology*
- *Faulting*
- *Liquefaction Potential*
- *Landsliding & Slope Stability*
- *Ground Shaking*
- *Erosion*
- *Geologically Suitable Building Envelope*

Soils Engineering Feasibility Investigation. The soil engineering feasibility investigation has been prepared to explore surface and subsurface soil and groundwater conditions at the site, and provide preliminary soil-engineering criteria for design and construction of the project.

The conclusions and recommendations of this report are intended to comply with Chapter 18 of the California Building Code (CBC) 2013 edition as modified by standard soil engineering practice in this area. Our scope of services included:

1. A visual site reconnaissance.

2. Exploration, sampling and classification of the surface and subsurface soils by means of excavating 13 exploratory test pits to depths ranging from 3.0 to 12.5 feet below the ground surface.

3. Laboratory testing of selected soil samples collected from the exploratory test pits and to determine their pertinent engineering and index properties.

4. Engineering analysis of the information collected based on the results of the field exploration; laboratory testing program and review of published and unpublished studies in the general area of the site.

5. Preparation of this report summarizing our preliminary findings and soil engineering conclusions and recommendations for site preparations, grading and compaction, foundations, retaining walls, utility trenches, slabs-on-grade, general site drainage, and erosion control.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The center of the site is located at approximate 36.6205° N latitude, 121.6739° W longitude in the northwest quarter of the northeast quadrant of the Spreckels 7.5 minute quadrangle, Monterey County, California. The site is unsectionized and remains part of the Buena Vista Mexican Land Grant. Surrounding land uses are residential and agricultural (Figure 1, Vicinity Map).

The site (APN 139-211-035) consists of an approximate 15.67-acre irregular shaped parcel located at the end of Woodridge Court in the Las Palmas Ranch area of Monterey County, California. Ingress/egress to the site is from and unpaved road located at the end of Woodridge Court (Sheet 1). Topographically, the site is situated on a northeast-southwest trending ridge, bounded by moderate to very steep descending slopes (Figure1). The northeasterly two-thirds of the site is situated on a relatively flat uplifted fluvial terrace with slope gradients ranging from ~5% to 10% (Sheet 1). The terrace area is bounded by steep to very steep northerly, southerly and easterly facing descending slopes with gradients ranging from 50% to 70%. The southwesterly one-third of the site consists of a hilly upland area with 15% to 25% slopes. Overall topographic relief in the proposed development area is about 150 feet (Sheet 1). The site is bounded by agricultural ranch lands to the north, east & west and the Las Palmas Ranch residential development to the south. Vegetative cover consists of a dense eucalyptus grove with scattered cypress trees on the terrace and upland areas. The northerly slopes are covered with a dense oak forest, while the southerly slopes are covered with scattered trees, brush and grasses. Drainage of the site is generally southwest to northeast via sheet flow.

As previously noted, access to the site is from an existing graded dirt driveway off of Woodridge Court at the southeasterly corner of the site (Sheet 1). Other existing site improvements consist of the following: storm drain inlet pipe and electrical vaults at the southeasterly property corner; two cribwall type retaining walls near the westerly end of the access drive; and a reclaimed water irrigation distribution system for the flat northeasterly two-thirds of the site. No records were available for our review such as plans, permits, etc. with respect to the construction of these improvements. Three landslide stabilization walls are located on the steep southeasterly descending slope located near the southwesterly corner of the site. These walls were constructed in the early to mid-1990's in response to the reactivation of an existing landslide (Qls, Sheet 1) located in this area.

During our site reconnaissance and field exploration we also noted that the site has been modified by other past grading activities. Areas of undocumented fill (designated as Af, Sheet 1) were noted to occur on the site. The most notable is a significant trash fill pit located in the south central portion of the site (TP-5, Figure A3 & TP-13, Figure A4). The materials consisted of substantial quantities of decomposing tree branches and limbs extending to a depth of 10.5 feet below the ground surface. It also appears that the most southerly limits of the flat terrace area has been bermed in order to limit the potential of site surface drainage from freely flowing over the steep southeasterly descending slopes adjacent to the Las Palmas ranch residential development.

Proposed future site development will consist of a new senior housing residential development composed of 26 independent living units, 24 assisted living apartments, 48 high level assisted living units and a 24 bed memory care facility. Additional development will consist of new vehicular drives and parking along with associated underground utility infrastructure improvements.

FIELD EXPLORATION

The site was mapped in the field on January 31 and February 25, 2014 on a base topographic map at a scale of 1:600. Additional mapping was done on aerial photographs at an approximate scale of 1:12,000. The field and aerial photograph mapping was then compiled on a topographic base map of 1:600 approximate scale (Site Geologic Map and Cross Sections, Sheets 1 & 2).

As part of our soil engineering feasibility investigation, 13 exploratory test pits were excavated on January 31, 2014 at the approximate locations shown on the Site Geologic Map, Sheet 1. The test pits were excavated with a rubber tired backhoe, equipped with a 24-inch wide bucket. The exploratory test pits were excavated to depths ranging from 3.0 to 12.5-feet below the ground surface. A Certified Engineering Geologist logged the exploratory test pits in the field. Upon completion of trenching, the test pits were backfilled with the previously excavated trench spoils.

Soils encountered in the exploratory test pits were visually classified in the field and logs were recorded. Visual classifications were made in general accordance with the Unified Soil Classification System and ASTM D2487. Logs of the exploratory test pits can be found in Appendix A (Figures A3 & A4). Appendix A also contains a Key to the Unified Soil Classification System and Soil Terminology (Figures A1 and A2).

LABORATORY TESTING

Laboratory tests were performed to determine the physical and engineering characteristics on selected soil samples of the various soil materials encountered in the exploratory test pits considered pertinent to the design of the project. The tests performed were selected on the basis of the probable design requirements as correlated to the site subsurface profile. A summary of the laboratory test results is presented in Appendix B. A brief generalized description of the tests performed is as follows.

- Moisture-Density Determinations: This test was conducted to measure the in-situ moisture contents and dry unit weights. Moisture/density testing was performed in accordance with ASTM D 2922 and ASTM D 3017, Nuclear Test Method. The test results are used to assess the distribution of subsurface pressures and to calculate degrees of in-situ relative compaction.

- Atterberg Limits: This test was performed on a disturbed bulk samples, to determine their liquid limit and plastic limit index values. This test provides water content values for the sample's liquid and plastic phases. This test aids in determining the expansive potential and other engineering characteristics of the soil.

- Grain Size Distribution (Gradation) Analysis: The grain size distribution is used to determine the classification of the site soils. This information is used for foundation design analysis.

- Compaction Curve (ASTM D 1557-91): This test is used to determine the maximum dry density and optimum moisture content based upon a standard compactive effort. When compared to the insitu moistures and densities, degrees of relative compaction can be obtained.

REGIONAL GEOLOGY

The site is situated on the south side of the Salinas River, at the northern terminus of the Sierra de Salinas within the Coast Ranges Geomorphic Province of California (Figure 2, Regional Geologic Map). The Coast Ranges Geomorphic Province consists of a series of mountain ranges paralleling the northwest-southeast structural orientation of the San Andreas fault, San Gregorio-Palo Colorado fault, Rinconada-Reliz fault and other faults within the central coast of California (Figure 5, Regional Fault and Seismicity Map). These faults are characterized by a combination of strike-slip and reverse displacement and show horizontal displacements from tens to hundreds of miles. Several periods of continuous and semi-continuous strike-slip or "transform" movement throughout the late Cenozoic Era has occurred on the San Andreas and related fault systems causing compressional uplift of the mountains of the Coast Ranges Geomorphic Province. The region continues to be characterized by moderate to high rates of seismic and tectonic activity (Figure 5).

The site is located on the southwest side of the San Andreas fault. The San Andreas fault forms the boundary between the North American and Pacific Plates. The southwest side of the San Andreas fault is underlain by Cretaceous age Salinian Block granitic rocks with older Sur Series metamorphic rocks that occur as roof pendants (Allen, 1946 & Dibblee, 1974). These roof pendants predominantly consist of marble and dolomite (Compton, 1966). Overlying the granitic rocks of the Salinian Block is a series of folded and faulted Tertiary age sedimentary and volcanic rocks (Dibblee, 1974).

During early to late Quaternary times, extensive continental, marine terrace, eolian, and fluvial sediments were deposited (Dupre' 1990, Clark Brabb & Rosenberg 2000). These sediments unconformably overlie all older formations with which they are in contact. Holocene activity has consisted of continued tectonic uplift, down cutting and deposition of the local area streams, mass wasting of upland areas by landslides and erosion, and fault creep along the San Andreas and related fault systems. The geology of the site and vicinity is depicted on the Geologic Vicinity Map, Figure 3.

REGIONAL FAULTING AND SEISMICITY

The closest Class A faults that would most likely effect the site are the San Andreas, Rinconda-Reliz, Monterey Bay, Zayante-Vergeles and San Gregorio faults (Figure 5). These faults have shown evidence for late Quaternary movement. Distance from the site, slip rate, and maximum magnitude for these faults is given in Table 1.

Table 1
Local and Regional Faults

Fault Name	Distance From Site	Maximum Magnitude	Slip Rate mm/year
San Andreas	**26.5-km Northeast**	**7.9**	**>5.0**
Rinconada-Reliz	**300 feet Northeast**	**7.3**	**0.2 – 1.0**
Monterey Bay	**14-km Southwest**	**6.8**	**1.0 – 5.0**
Zayante-Vergeles	**23-km Northeast**	**6.8**	**0.2 -1.0**
San Gregorio	**30-km Southwest**	**7.0**	**1.0 – 5.0**

San Andreas Fault

The San Andreas fault is located about 26.5 km. northeast of the site and is the major seismic hazard in northern California. The San Andreas fault is a major right-lateral strike-slip fault that generally delineates the transform plate boundary between the North American and Pacific Plates. Trending to the northwest southeast, the San Andreas fault is nearly vertical as evidenced by the relatively straight outcrop pattern across topography of noticeable relief. Historic earthquakes on the San Andreas fault have caused extensive damage and very strong ground shaking in Monterey County. The 1906 (M_w~8.0) "San Francisco earthquake" ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, causing severe damage in parts of the Monterey-San Francisco Bay area. The earthquake occurred on April 18, 1906 and caused severe ground shaking and structural damage to buildings in Monterey, Santa Cruz and San Benito Counties (Lawson, 1908). The 1989 (M_w 7.1) Loma Prieta earthquake also caused significant damage in the cities of Salinas, Santa Cruz, Watsonville, and Hollister (McCann, 1990).

The San Andreas fault has been divided into several different segments that are characterized by varying slip rates, earthquake intensities, and earthquake recurrence intervals (Bryant and Lundberg, 2002). Located about 26.5 km. northeast of the site, the San Andreas fault can expect a (M6.8) earthquake with an unknown recurrence interval (Petersen et al, 1996). Stronger earthquakes could be experienced at the site similar to the 1906 event with a maximum magnitude of (M7.9) with a recurrence interval of 210 years (Petersen et al, 1996).

Rinconada-Reliz Fault

The Rinconada-Reliz fault is primarily a right lateral strike slip and reverse fault zone (Petersen et al, 1996) with a vertical component having elevated the southwest block to form the Sierra de Salinas uplift (Dibblee, 1976). The Reliz fault zone, Blanco section is located about 300 feet northeast of the site (Rosenberg & Bryant, 286a & 286b, 2003, Rosenberg & Clark, 2009). The Rinconada-Reliz fault is a major structural feature along which granitic rocks of the Sierra de Salinas were uplifted to form the western border of the Salinas Valley (Greene et al, 1973) with movement beginning during early Cenozoic time and has remained active to late Quaternary (Rosenberg & Bryant, 286a & 286b, 2003). Vertical displacement in the Sierra de Salinas may be as much as 10,000 feet (Dibblee, 1976). Slip rate for the Rinconada fault is estimated at 0.2 to 1.0mm/yr. Maximum magnitude is expected to be (M7.3) with a recurrence interval of 1,764 years (Petersen et al, 1996).

Monterey Bay-Tularcitos Fault

Located about 14-km southwest of the site, the Monterey Bay-Tularcitos fault zone is a complex series of northwest trending reverse, right lateral, and oblique faults which include the Tularcitos, Chupines, and Navy faults (Bryant 62b & 62c, 2001). The Monterey Bay-Tularcitos fault zone lies within a fault bounded wedge of granitic basement rocks belonging to the Salinian block and is bounded on the west by the San Gregorio fault and on the east by the San Andreas fault (McKittrick, 1987). The Monterey Bay-Tularcitos fault is 84 km. long (Petersen et al, 1996) and extends from Paloma Creek in upper Carmel Valley (Clark et al, 1997) to the offshore environment within the Monterey Bay. Post Miocene vertical displacement of the Tularcitos fault is about 380 m and 3.2km to as much as 16 km of right lateral displacement (Clark et al, 1997). Offsets of Holocene age colluvial and fluvial terrace deposits indicates that the Tularcitos fault is active (Clark et al, 1997). The Monterey Bay fault is the offshore extension of the Tularcitos fault and comprises a discontinuous series of en echelon faults in the inner Monterey Bay between Monterey and Santa Cruz (Greene et al, 1973). The Monterey Bay fault zone displaces late Tertiary and Pleistocene sediments and in a few locations appears to cut Holocene sediments (Greene et al, 1973). Slip rate for the Monterey Bay-Tularcitos fault is estimated at 0.5mm/yr. Maximum magnitude is expected to be (M7.1) with a recurrence interval of 2,841 years (Petersen et al, 1996).

Zayante-Vergeles Fault

The Zayante-Vergeles fault is located about 23-km northeast of the site (Coppersmith, 1979). The Zayante-Vergeles fault is a right-lateral reverse fault (Petersen et al, 1996) dipping steeply to the south (70°-80°) with a minimum vertical displacement of 3,500 feet (Allen, 1946). No Tertiary sediments are found on the uplifted Salinian Block granite south of the fault, as they have been completely eroded. Two branches of the Vergeles fault break of the main fault trace at low angles to form "splinters", which duplicates portions of the Miocene rock record (Allen, 1946). Initial movement on this fault probably began in the middle Miocene corresponding with the deposition of the Zayante Sandstone. Movement on this fault was probably sporadic through late Pliocene (Allen, 1946). More recent studies suggest that the Zayante fault (the western extension of the Vergeles fault) has at least 10-17 meters of vertical displacement in the last 500,000 years (Coppersmith, 1979). Slip rate for the Zayante-Vergeles fault is estimated at 0.2 to 1.0mm/yr. (Bryant, 2000). Maximum magnitude is expected to be (M6.8) with a recurrence interval of 8,821 years (Petersen et al, 1996).

San Gregorio Fault

Like the San Andreas fault, the San Gregorio fault has been divided into several different segments that are characterized by varying slip rates, earthquake intensities, and earthquake recurrence intervals. Located about 44 km southwest of the site, the San Gregorio (Sur region) is a northwest trending right lateral strike slip fault about 80 km long (Petersen et al, 1996). The San Gregorio fault is part of the San Andreas fault system and is expressed as a complex series of en echelon right lateral strike slip faults (San Gregorio, Palo Colorado, San Simeon, & Hosgri faults) in the offshore and near shore environments. The San Gregorio and related faults are several hundred kilometers long extending from the Santa Barbara Channel in the south, to its juncture with the San Andreas fault near Bolinas Bay in the north. Strong evidence supports that the San Gregorio fault (Sur region) has been active during Holocene time (Greene et al, 1973). Slip rate for the San Gregorio fault (Sur region) is estimated at >5.0mm/yr. (Bryant and Cluett, 1999). Maximum magnitude is expected to be (M7.0) with a recurrence interval of 411 years (Petersen et al, 1996).

SITE GEOLOGY

Previous published & unpublished mapping of the site and its vicinity has been performed by Dibblee, 1974; Dupre', 1990 and Clark, Brabb & Rosenberg, 2000. Dibblee, 1974 mapped the site at a scale of 1:62,500, and as being underlain entirely by Quaternary older alluvium. Mapping performed by Dibblee did not indicate the presence of faults or landslides to occur on the site.

More recent mapping of the site and vicinity was performed by Dupre', 1990 at a scale of 1:24,000. This mapping concentrates on Quaternary geology and liquefaction potential. Dupre' has mapped the northeasterly portion of site as being underlain by Pleistocene age fluvial terrace deposits. The upland southwesterly portion of the site has been mapped as Plio?-Pleistocene age Non-marine continental deposits. Dupre' also maps a landslide in the area of the southwesterly corner of the site. No faults were noted to occur, or were mapped on the site.

Clark, Brabb & Rosenberg, 2000 have performed the most recent and detailed published geologic mapping of the site and vicinity at a scale of 1:24,000 (Figure 3). Clark, Brabb and Rosenberg map the flatter northeasterly portion of the site as being underlain by a veneer of Pleistocene age fluvial terrace deposits, which are underlain by Plio?-Pleistocene age continental deposits. The southwesterly upland portion of the site has been mapped as Plio?-Pleistocene age continental deposits dipping to the northeast. Review of this most recent mapping indicates the presence of a large landslide located in the southwesterly portion of the site that extends offsite to the south. No faults were mapped on the site.

Geology for this report was mapped in the field on January 31 and February 25, 2014. Field mapping was done on aerial photographs at an approximate scale of 1:12,000, and on a base topographic map at a scale of 1:600. The field mapping work was then compiled on a topographic base map of 1:600 scale (Site Geologic Map & Cross Sections, Sheets 1 & 2). As part of our geologic mapping we examined and interpreted of four sets of stereo aerial photographs of the area taken in 1956, 1978 & 1994 of the site and its vicinity. These photographs were scrutinized for site geology, terrain features characteristic of active fault zones, and for landsliding features. Based on the above referenced techniques and our exploratory trenching program, it is our opinion that the geology as mapped by Clark, Brabb & Rosenberg, 2000 is reasonably accurate. Description of the site geology is as follows, refer to Site Geologic Map and Geologic Cross Sections (Sheets 1 & 2) located in the map pocket at the back of this report for the location and distribution of these units.

(Af) Artificial Fill (Holocene): As previously noted in this report, man-made deposits of fill and trash fill are located on the site. The most significant of the features is located in the south central portion of the site in the areas of exploratory test pits TP-5 & TP-13. The materials encountered consist of substantial quantities of decomposing tree branches and limbs extending to a depth of 10.5 feet below the ground surface. *Areas of undocumented fill must be remediated prior to building development and foundation construction.*

(Qc) Colluvium (Holocene): Colluvial deposits have been mapped on the steep southerly slopes of the site adjacent to the Las Palmas Ranch residential development. These Holocene age deposits consist of unconsolidated silt, sand and gravel eroded off of the upland areas deposited by slope wash and mass movement.

(Qls) Landslide deposits (Quaternary): Evidence for landsliding was found to flank both sides of the ridge that trends northeast-southwest through the property. The landslides consist of heterogeneous deposits ranging from block slides to earthflows in weakly to semi-consolidated sand and clay.

(Qt) Terrace deposits (Pleistocene): The northeasterly two-thirds of the site has been mapped as Pleistocene age terrace deposits. These sediments consist of weakly consolidated silt, sand and gravel deposited in a fluvial (river) environment.

(Qtc) Continental deposits (Pleistocene-Pliocene (?)): The surface of the upland areas in the southwesterly one-third of the site have been mapped as Pleistocene-Pliocene (?) age continental deposits which are in conformable contact with the younger overlying terrace deposits. These sediments were also noted to crop out on the steep slopes on the southerly flank of the site. The continental deposits consist of nonmarine consolidated fine to coarse grained sand with pebble and cobble/gravel interbeds.

Site Geologic Structure and Faulting

Bedding inclinations near the site indicate that the Continental deposits are dipping 30 to 35° to the northeast (Clark, Brabb, & Rosenberg, 2000). No structural axis (anticlinal or synclinal) has been mapped underlying site.

The closest Class A fault to the site is the Blanco section of the Reliz fault located approximately 300 feet northeast of the site (Rosenberg & Bryant, 2003, Rosenberg & Clark, 2009). Though the site is not located within an Earthquake Fault Zone as established by the State of California, the Rinconada-Reliz fault has displayed late Pleistocene and probable early Holocene displacement to be classified as significant seismic hazard.

Other faults in proximity to the site are the Las Palmas and Harper faults (Figure 3) which are classified as Class B faults by the U.S. Geological Survey Geologic Survey. Class B faults demonstrate the existence of Quaternary deformation, but either (1) the fault might not extend deeply enough to be a potential source of significant earthquakes, or (2) the currently available geologic evidence is too strong to confidently assign the feature to Class C but not strong enough to assign it to Class A. These faults have not displayed substantial rates of displacement to be classified as significant seismic hazards. However, based on evidence of late Quaternary activity and proximity to the site a discussion of these faults is as follows.

Las Palmas Fault

Located about 2.1-km. southeast of the site, the Las Palmas fault strikes northwest along the foothills south of the Salinas River. Its mapped trace is marked by aligned springs, local offset of Pleistocene-Pliocene(?) continental deposits and a pronounced gravity gradient (Clark, Brabb & Rosenberg, 2000, Rosenberg & Clark, 2009). The parallel orientation and sense of displacement suggest that the Las Palmas fault is probably a branch of the Reliz fault zone (Rosenberg & Clark, 2009). The offset of the continental deposits support the conclusion that a continuous zone of faulting younger than 250-550 ka cannot be demonstrated (Rosenberg & Clark, (2009).

Harper Fault

Located about 1.0-km. west of the site, the Harper fault is a north striking reverse fault that juxtapose Cretaceous age granitic rocks against Pliocene(?)-Pleistocene age continental deposits. Although the Harper fault locally truncates the continental deposits, there is no documented evidence of Holocene activity along this fault.

Landsliding

Evidence for landsliding was found to flank both the northerly and southerly sides of the site. As previously mentioned, Dupre', 1990 and Clark, Brabb & Rosenberg, 2000 (Figure 3) have mapped a large landslide several acres in area on the steep slopes at southwesterly corner of the site extending offsite to the south. During the course of this investigation we confirmed the presence of this large block slide type failure as depicted on Sheet 1. In response to the reactivation of this slide in early to mid-1990's, three landslide stabilization walls were constructed to stabilize the slope and limit the potential impact to the adjacent downslope residences within the Las Palmas Ranch subdivision. On the northerly flank of the site, several smaller earthflow and surficial type of failures have been mapped (Sheet 1).

SUBSURFACE CONDITIONS

As part of the soil engineering feasibility investigation 13 exploratory test pits were excavated in proposed development area. Subsurface constituents were similar to the depths explored in each of the exploratory test pits. The earth materials encountered consisted of Pleistocene age fluvial terrace deposits and Plio(?)-Pleistocene age continental deposits. These materials consist of loose to very dense, silty SAND, well graded SAND and lesser amounts of clayey SAND. One notable exception was the discovery of a significant trash fill pit located in the south central portion of the site (TP-5, Figure A3 & TP-13, Figure A4). The materials consisted of substantial quantities of decomposing tree branches and limbs extending to a depth of 10.5 feet below the ground surface.

GROUNDWATER

Groundwater was not encountered in any of the exploratory test pits. No active springs were noted to occur on the site. Local groundwater levels can fluctuate over time depending on but not limited to factors such as seasonal rainfall, site elevation, groundwater withdrawal, and construction activities at neighboring sites. The influence of these time dependent factors could not be assessed at the time of our investigation.

CONCLUSIONS

Seismic Hazards The site is located in the seismically active Monterey Bay region of the Coast Ranges Geomorphic Province. The site is not located within any Earthquake Fault Zones in accordance with the Alquist-Priolo Earthquake Fault Zoning Act (formerly Alquist-Priolo Special Studies Zone Act) of 1972 (Hart and Bryant, 1997).

Surface Fault Rupture: The Blanco section of the Reliz fault is located approximately 300 feet east of the site (Rosenberg & Bryant, 2003). The Reliz fault has displayed late Quaternary displacement, but it is not located on the subject site, therefore potential for surface rupture to occur on the site is low.

Historical Earthquakes: During recent historic times moderate to large earthquakes have caused significant damage to man-made structures in the greater Monterey Bay area. These include the following:

1857 San Andreas Fault: A large quake occurred on the San Andreas fault, rupturing from Parkfield south to Wrightwood, on January 9, 1857. The quake had an estimated magnitude of 7.8. Very severe shocks were felt in Sacramento and a cabin was knocked down in the Cholame area (Rosenberg, 2001).

1881 Parkfield: On February 2, 1881 a 5.6 magnitude quake occurred in the Parkfield area knocking down several adobe structures and chimneys. Springs and cracks were also noted in the area of the quake (Rosenberg, 2001).

1901 Parkfield: A magnitude 5.8 struck the Parkfield area on March 2, 1901. Again many chimneys were damaged and cracks in the ground were noted. A small tsunami also occurred in the Monterey Bay. (Rosenberg, 2001)

1906 California: The 1906 (M_w~8.0) "San Francisco earthquake", which ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, caused severe damage in parts of the Monterey-San Francisco Bay area and throughout California. The earthquake occurred on April 18, 1906 and caused severe ground shaking, ground settlement, liquefaction, and structural damage to buildings in Monterey, Santa Cruz, and San Benito Counties (Lawson, 1908). The most significant earthquake effects in the area of the site and vicinity were the sinking of the Salinas River bed in the areas of Spreckels, King City and San Ardo. (Rosenberg, 2001). Ground water flow changes were also common. At Paraiso Springs the temperature and flow of water had been decreasing for "some time" before the quake (Lawson, 1908). After the quake the temperature and flow of the springs returned to its previous values (Lawson, 1908).

1922 Parkfield: The March 10, 1922 earthquake that struck the Parkfield area was a magnitude 6.1. It caused ground cracks six to twelve inches in width and a quarter-mile long in the Chalome Valley (Rosenberg, 2001). Chimneys were knocked down and some housed suffered structural damage. An oil pipeline was also damaged in the area.

1926 Monterey Bay Doublet: On October 22, 1926 two magnitude 6.1 earthquakes an hour apart occurred in southern Monterey Bay. Numerous buildings experienced damage and cracking on the Monterey Peninsula and in Salinas. It is postulated that the earthquakes occurred on either the San Gregorio fault or Monterey Bay fault zone (Rosenberg, 2001).

1934 Parkfield: A magnitude 6.1 earthquake again struck the Parkfield area on June 7, 1934. Again this quake caused fracturing of the ground surface and broke the oil pipeline in the area. Chimneys and houses were also damaged in the area (Rosenberg, 2001). This is the closest quake of magnitude 5.0 or greater to the site at approximately 37.4-km away.

1938 Stonewall Canyon: On September 27, 1938 a magnitude 5.0 quake occurred in the Stonewall Canyon area northeast of Soledad. Details of the damage caused by this quake are unknown.

1989 Loma Prieta: The October 17, 1989 (M_w 7.1) Loma Prieta earthquake, which is believed to have occurred on an oblique-slip blind thrust closely associated with the San Andreas fault, also caused significant damage in the San Francisco and Monterey Bay areas. It was the largest earthquake to strike this region of California since the California earthquake of 1906. The effects of this earthquake was felt over an area of 400,000 square miles and resulted in 74 deaths, 3,757 injuries, 12,000 homeless, and over $6 billion in property damage (Plafker & Galloway, 1989). In Monterey County 19 homes were destroyed, 341 homes damaged, two deaths and 14 people injured, and causing approximately $118 million in damages (Rosenberg, 2001). The Carmel Valley area suffered little damage as a result of this quake. The liquefaction experienced in the 1906 quake was absent during this event. The explanation given by Rosenberg, 2001 for the differences in liquefaction occurrence is differences in ground water table at the time of rupture. Groundwater was likely higher in 1906 as they had a wet winter, and the 1989 quake occurred after several years of drought.

As part of our historical earthquake research, we performed a database search of the California Geological Survey (http://redirect.conservation.ca.gov/cgs/rghm/quakes/historical/hiseq.asp) for earthquakes with magnitudes greater than 5.5 within an approximate 50-km radius of the site for the years from 1800 to 2014. The database research indicated a total of 23 events within the search parameters, see Table 1 below.

Table 2
California Historical Earthquakes (M>=5.5)
within 50 km of Latitude 36.6739, Longitude –121.6739

Date	Magnitude	Distance from Site	Location
10/11/1800	5.5	29.2 km	San Juan Bautista
06/10/1836	6.4	34.7 km	Between Monterey & Santa Clara
01/18/1840	6.5	29.2 km	San Juan Bautista
07/03/1841	5.9	27.5 km	San Juan Bautista
07/29/1841	5.8	28.2 km	San Juan Bautista
03/26/1866	5.8	42.3 km	Gilroy
03/30/1883	6.0	25.3 km	San Juan Bautista
04/15/1889	5.5	44.0 km	Hollister
04/24/1890	6.3	31.8 km	Pajaro Gap
11/13/1892	5.9	28.4 km	Hollister
06/20/1897	6.3	45.0 km	Gilroy
04/30/1899	6.0	26.4 km	Watsonville
07/06/1899	5.8	39.5 km	San Juan Bautista
05/18/1906	5.6	27.2 km	San Juan Bautista
03/11/1910	5.8	36.7 km	Watsonville
12/31/1910	5.7	42.9km	Monterey Bay
08/06/1916	5.6	38.2 km	Paicines Area
10/22/1926	6.1	48.0 km	Monterey Bay
10/22/1926	6.1	45.9 km	Monterey Bay
04/25/1954	5.6	31.9 km	Watsonville
04/09/1961	5.5	34.0 km	Hollister
04/09/1961	5.5	34.5 km	Hollister
01/26/1986	5.5	40.3 km	Paicines - Hollister

Our database search within the defined parameters revealed that the largest earthquake to occur was a M6.5 on January 18, 1840 located in the San Juan Bautista area about 29.2 km northeast of the site. The closest earthquake with a M>=5.5 was a M6.0 event that occurred 25.3 km northeast on the site in the San Juan Bautista area on March 30, 1883.

Ground Shaking: The 1906 (M_w~8.0) "San Francisco earthquake", which ruptured a portion of the active San Andreas fault from approximately San Juan Bautista to Cape Mendocino, caused severe damage in parts of the Monterey-San Francisco Bay area. The earthquake occurred on April 18, 1906 and caused severe ground shaking and structural damage to buildings in Monterey, Santa Cruz and San Benito Counties (Lawson, 1908). The 1989 (M_w 7.1) Loma Prieta earthquake also caused significant damage in San Benito, Santa Cruz and Monterey Counties (McCann, 1990). Strong ground shaking associated with major earthquakes along the San Andreas and other nearby faults will undoubtedly occur at the site in the future. The peak ground acceleration with a 10 percent probability of being exceeded in a 50-year period in the vicinity of the site is 0.348g (USGS Ground Motion Parameters computer program, Version 5.1.0). The site modified calculated peak ground acceleration with a 2 percent probability of being exceeded in a 50-year period is 0.522g (USGS Ground Motion Parameters computer program, Version 5.1.0).

Seismic Design Parameters: For seismic design using the 2013 CBC, we recommend the following design values be used. The parameters were calculated using the U.S. Geological Survey Ground Motion Parameters computer program (Version 5.1.0) and were based on the approximate center of the site located at 36.6205° N. latitude and –121.6739° W. longitude.

2013 CBC Seismic Design Parameters

Design Parameter	***Site Design Value***
Site Class	**D – Stiff Soil**
Spectral Acceleration Short Period	**(S_s) = 1.304g**
Spectral Acceleration 1 Second Period	**(S_1) = 0.590g**
Short Period Site Coefficient	**(F_a) = 1.00**
1 Second Period Site Coefficient	**(F_v) = 1.50**
MCE Spectral Response Acceleration Short Period	**(S_{MS}) = 1.304g**
MCE Spectral Response Acceleration 1-Second Period	**(S_{M1}) = 0.884g**
5% Damped Spectral Response Acceleration Short Period	**(S_{DS}) = 0.870g**
5% Damped Spectral Response Acceleration 1-Second Period	**(S_{D1}) = 0.590g**

Liquefaction, Lateral Spreading, and Dynamic Compaction: Liquefaction is the transformation of soil from a solid to a liquid state as a consequence of increased pore-water pressures in response to strong ground shaking during an earthquake. Liquefaction most often occurs in loose saturated silts, and saturated poorly graded fine-grained sands. Liquefaction potential maps prepared by Dupre' (1990) show that the site is in an area of very low potential for liquefaction. Based on our field investigation and research, it is our opinion that the potential for liquefaction at the site is very low.

Lateral spreading can occur when soils liquefy beneath a slope, or even beneath level ground if an open topographic face is nearby. Since the potential for liquefaction at the site is judged to be very low, the potential for lateral spreading is likewise estimated to be very low. Dynamic compaction occurs when loose, unsaturated soils densify in response to ground shaking during a seismic event. Because no such materials were encountered on the site, it is our opinion that the potential for dynamic compaction is very low.

Ridge-Top Shattering: Ridge-top shattering was well documented after the 1971 San Fernando earthquake and also occurred during the 1989 Loma Prieta earthquake in the Santa Cruz Mountains. The phenomenon occurs most commonly on the crests of sharp ridges, where seismic shaking energy is concentrated as in the chimney of a building. Spreading can effect both soil and the underlying bedrock and gives the appearance of plowed ground (Barrows, 1975; Kahle, 1975). Most of the site is situated on gentle to steep slopes with rounded topography, as opposed to sharp ridge crests. Topographic features associated with ridge top spreading are also not present at the site or its vicinity. Therefore, the potential for ridge-top shattering is considered to be low.

Landsliding and Slope Stability: The steep slopes on the northerly and southerly flanks of the site are prone to landsliding and slope failure. Block landslides, surficial failures, earth flows and colluvial deposits of varying ages are present on these slopes, which are considered the most significant geologic hazards on the site. In order to mitigate the potential hazards from landsliding and slope instability, future building foundations should be located within the Geologically Suitable Building Envelope as depicted on Sheet 1 of this report. *A supplemental site specific numerical slope stability analysis must be performed if structures designed for human occupancy extend beyond the limits of the of the identified Geologically Suitable Building Envelope.*

Grading: As the soil materials that will be supporting future foundations at depth consist of Pleistocene age semi-consolidated sediments, deep remedial grading is not considered necessary to improve the soils for foundation support. As previously noted, man-made deposits of fill and trash fill over 10 feet in depth are located on the site. The areas of undocumented fill must be completed removed down to firm and dense native earth materials and be backfilled with engineered fill prior to building development and foundation construction.

Soil Expansion: Based on visual observations and laboratory testing the near surface site soils are classified as silty SAND and well graded SAND, and are considered to be non-plastic. No special measures are required to mitigate the effect of soil expansion on foundations, and interior or exterior concrete slabs-on-grade.

Erosion: The site soils and earth materials are highly erodible. Stringent erosion control measures should be implemented to provide surficial stability of area that will be disturbed by proposed grading.

RECOMMENDATIONS

In our opinion, the site is suitable from a geologic and soil-engineering standpoint for the proposed development provided that the preliminary recommendations contained herein are implemented in the design and construction. The following preliminary recommendations are presented as guidelines to be used by project planners and designers for the geologic and soil engineering aspects of the project design and construction. *These recommendations have been prepared assuming that Landset Engineers, Inc. will be commissioned to perform additional design level investigation(s),* review proposed grading & drainage and foundation plans before construction, and to observe, test and advise during earthwork and foundation construction. Soil and groundwater conditions can deviate from the conditions encountered at the test pit locations. If significant variations in the subsurface conditions are encountered during construction, it may be necessary for Landset Engineers, Inc. to review the recommendations presented herein, and recommend adjustments as necessary.

Geologic

The following recommendations are drawn from the data acquired and evaluated during this investigation for the proposed project.

1. Prior to construction, the project geologist should review the site grading and improvement plans and their potential impacts on identified geologic hazards.
2. In order to mitigate the potential hazards from landsliding and slope instability, future building foundations should be located within the Geologically Suitable Building Envelope (Sheet 1). Structures designed for human occupancy should be located within this envelope.
3. Structures designed for human occupancy shall be designed according to the current edition of the CBC. Structures should be designed for peak horizontal ground acceleration of 0.522g.
4. The project geologist should review the site grading during earthwork. The purpose of this review is to examine the site for overall stability and to provide additional recommendations if site conditions differ those identified during the course of this investigation.

Soil Engineering

Site Preparation and Grading

1. The soil engineer should be notified **at least ten (10) working days prior to any site clearing or grading** so that the work in the field can be coordinated with the grading contractor, and arrangements for testing and observation services can be made. The recommendations contained in this report are based on the assumption that Landset Engineers, Inc. will perform the required testing and observation services during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

2. Prior to grading, construction areas should be cleared of obstructions, buried structures & utilities, tree roots, undocumented fill and other deleterious materials. Site clearing should be observed by a field representative of Landset Engineers, Inc. Voids created by removal of material as described above should be called to the attention of the soil engineer. No fill should be placed unless a representative of this firm has observed the underlying soil.

3. Following site clearing, the upper 1.5 to 3-feet of native soil should be overexcavated from the building areas. The actual depth of subexcavation should be determined by additional design level soil engineering investigation(s). Building areas are defined as the soils within and extending a minimum of 5 feet beyond the foundation perimeters and structural fill areas.

4. The soils exposed by overexcavation should be scarified 12 inches; moisture conditioned to above optimum moisture content, and compacted to at least 90% of maximum dry density. Where referenced in this report, percent relative compaction and optimum moisture content shall be based on ASTM test D1557. Areas to receive structural fill outside the building pad should be scarified and recompacted in a similar manner.

5. In order to limit the potential for differential settlement of conventional footings, foundations should not be supported on both fill and cut. Therefore, we recommend that the cut side of the building area should be overexcavated (undercut). The proposed grading within the building area should be designed so that no more than 5 feet of differential fill thickness exists below foundations. The portion of the building foundations bearing on cut should be undercut at least 3 feet below the proposed building pad so that the entire foundation is bearing on a uniform layer of compacted fill. Deeper overexcavation may be necessary in order to satisfy the differential fill thickness recommendations.

6. If structural fill is to be placed on slopes steeper than 6:1 (horizontal to vertical), keyways should be established at the toe of the proposed fill slopes. The keyways should have minimum widths of 12-feet and should be sloped approximately 2% back into the hillsides. The keyways and subsequent upslope benches should penetrate into sufficiently stable material at determined by the soil engineer at the time of grading.

7. If structural fill is to be placed on slopes steeper than 10:1, the slopes should be benched. The benches should have a minimum width of 12-feet and should be sloped approximately 2% back into the hillsides. The soil engineer will determine the depth, scarification, and recompaction of the bench bottoms at the time of grading.

8. If fill over cut slopes are to be constructed, keyways should be established at the cut/fill daylight lines. The keyways should have minimum widths of 12-feet and should be sloped approximately 2% back into the hillsides. The keyways and subsequent upslope benches should penetrate into sufficiently stable material as determined by the soil engineer at the time of grading.

9. The soil engineer should also observe keyways and benches to assess the need for subsurface drains (subdrains). Subdrains in other areas may also be recommended depending on the grading plan and site conditions observed at the time of grading.

10. Fill slopes should be constructed at a maximum finished slope inclination of 2:1 (horizontal to vertical). Fill slopes should be overfilled and trimmed back to competent material. Further compaction of exposed fill slope faces using sheepsfoot rollers or tracked equipment may be recommended by the soil engineer. Cut slopes should be constructed at an inclination of 2:1.

11. Fill, material should be placed in thin lifts, moisture conditioned to a level above optimum moisture content, and compacted to a minimum of 90 percent of maximum dry density. Prior to compaction, the soil should be cleaned of any rock, debris, and irreducible material larger than 3-inches in diameter.

12. Fill material should consist of non-expansive Select Structural Fill. Select Structural Fill is defined herein as a native or import fill material which, when properly compacted, will support foundations, pavements, and other fills without detrimental settlement or expansion. Select Structural Fill is specified as follows:

Select Structural Fill

- Clean native soil may be utilized, but import fill shall have a Plasticity Index of less than 12
- Be free of debris, vegetation, and other deleterious material
- Have a maximum particle size of 3-inches in diameter
- Contain no more than 15% by weight of rocks larger than 21/2-inches in diameter
- Have sufficient binder to allow foundation and unshored excavation stand without caving
- Prior to delivery to the site, a representative sample of proposed import should be provided to Landset Engineers, Inc. for laboratory evaluation

13. In areas to be paved, the upper 12-inches of subgrade soils and all aggregate base should be compacted to a minimum of 95 percent of maximum dry density. Aggregate base and subgrade should be firm and unyielding when proof rolled by heavy rubber-tired equipment prior to paving.

Foundations

14. Structures may be supported by conventional continuous and spread (pad) footings or drilled pier & grade beam foundations.

Conventional Footings

15. Conventional footings may be supported ***entirely on recompacted engineered fill or entirely on firm native soil, but not a combination of both***. Footings should have minimum depths of 12-inches below lowest adjacent grade for single story structures, and 18-inches below lowest adjacent grade for two story structures, and 24-inches below lowest adjacent grade for three story structures. For the above conditions, the footings for a proposed structure may be designed for an allowable bearing pressure range of 1,000 to 3,000 psf for dead plus live loads. Footings should be reinforced as directed by the architect/structural engineer.

16. Footing excavations should be observed by a representative of this firm prior to placement of formwork or reinforcement. Concrete should be placed only in foundation excavations that have been kept moist, and contain no loose or soft soil debris.

17. Footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches.

Pier & Grade Beam Foundations

18. Drilled friction and/or end bearing pier and grade beam foundations should penetrate through any engineered fill and/or topsoil and bear entirely into the dense native earth materials as verified by a representative of this firm at the time of drilling.

19. Foundation piers should be 12 to 24-inches in diameter and should be spaced apart at least 3 pier diameters, center to center. These cast-in-place concrete piers should be reinforced as directed by the project architect/structural engineer.

20. For the above conditions, the piers for a proposed structure may be designed for an allowable skin-friction range of 200 to 350 psf. for pier lengths in native earth materials for dead plus live loading. This value may be increased by one-third when considering temporary additional short-term wind or seismic loading. The support from end bearing of the piers should be neglected. Due to possible disturbance during drilling, skin friction on the upper 2-feet of the piers should be discounted in the calculations. Piers should be structurally connected to grade beams designed to transfer imposed loads to the foundation piers.

21. For calculating resistance to lateral loading, a passive resistance equal to an equivalent fluid weight range of 200 to 350 pcf. can be used (ultimate value). For pier foundations, this lateral resistance can be used over two times the cross sectional area of the pier. Only competent native earth material and engineered structural fill may be utilized in calculating lateral passive resistance. Additionally, the upper 2-feet of the pier should be ignored in providing lateral passive resistance.

22. Perimeter foundation piers and piers adjacent to structural concrete slabs-on-grade should be laterally restrained by concrete grade beams penetrating a minimum of 12-inches below lowest adjacent grade. Grade beams between interior piers are not considered necessary. Grade beams should be reinforced as directed by the project architect/structural engineer.

Slabs-on-Grade and Exterior Flatwork

23. For buildings utilizing conventional footings, interior slabs-on-grade should have a thickness of 4 to 6-inches. It should be noted that the project structural engineer might require thicker slab sections to provide the necessary support for the anticipated structural loads. Conventional concrete slabs-on-grade should be reinforced with steel as specified by the structural engineer.

24. To minimize floor dampness, such as where moisture sensitive floorings will be present, a section of capillary break material at least 4-inches thick covered with a membrane vapor barrier should be placed between the floor slab and the compacted soil subgrade. The capillary break should consist of a clean, free draining material such as ½ to ¾-inch drain rock with not more than 10 percent of the material passing a No. 4 sieve. The drain rock should be free of sharp edges that might damage the membrane vapor barrier. The membrane vapor barrier should be a minimum 10 mil in thickness, and care should be taken to properly lap and seal the vapor barrier, particularly around utilities. The sand cushion should be lightly moistened immediately prior to concrete placement.

25. Exterior concrete flatwork such as driveways, patios and sidewalks should be designed to act independently of building foundations. Exterior flatwork should be constructed on compacted soil subgrade moisture conditioned to over optimum moisture content. Reinforcement and joint spacing should be at the direction of the architect/structural engineer.

Retaining Walls

26. Retaining walls for the site may be designed using the following general design parameters, which assume fully drained wall backfill conditions. The average bulk density of material placed on the backfill sides of walls considers a design range of 120 to 130 pounds per cubic foot (pcf).

27. The vertical plane extending down from the ground surface to the bottom of the heel of the vertical wall will be subject to lateral soil pressures (plus surcharge loads). An Active Soil Pressure of 35 to 50pcf (equivalent fluid weight) should be used in design of site walls that are free to move laterally and resultant settlement of backfill is tolerable. An At-Rest Soil Pressure of 50 to 70pcf should be used in design for walls, which are restricted from movement at the top (such as foundation walls). The above pressures are applicable to a horizontal retained surface behind the wall. Walls having a retained surface that slopes upward from the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at rest case, for every two degrees of slope inclination.

28. The additional effects of earthquakes on the walls may be simulated by applying a horizontal line force of $10H^2$ pounds per foot length of wall. This force should be applied at a height of 0.6H above the wall heel. The additional effects of vertical live loads on the backfill side of walls may be simulated by applying 50 percent of the live loads as a horizontal surcharge force on the walls. The point of application of the live load surcharge may be estimated by assuming a 45-degree line of action down from the live load to the design plane or wall stem.

29. Retaining walls should be supported on foundations bearing uniform soil conditions as described in the preceding foundation section of this report. The range for ultimate coefficient of friction below the base of the wall = 0.25 to 0.35. Passive soil resistance against the portion of the wall base and key is estimated to range from 200 to 350psf/ft.

for level ground in front of the wall. Lateral support from the soil that may be excavated or used in landscaping near the wall footing should be neglected. Typically this would include the top 12-inches of soil around the wall.

30. The earth pressures are based on fully drained conditions. We recommend that a zone of drainage material at least 12-inches wide should be placed on the backfill side of the walls. Drainage materials should consist of Class 2 permeable material complying with Section 68 of the Caltrans Standard Specifications, latest edition, or ¾-inch permeable drain rock wrapped in Mirafi 140N or equivalent. Manufactured drains such as Miradrain or Enkadrain are acceptable alternatives to the use of permeable or gravel material, provided that they are installed in accordance with the recommendations of the manufacturer. The drains should extend from the base of the walls to within 12-inches of the top of the wall backfill. The upper 12-inches of wall backfill should consist of compacted structural fill. A perforated pipe should be placed (holes down) about 4-inches above the bottom of the wall or below lowest adjacent grades in front of the wall. The perforations should be no larger than ¼-inch diameter, and the perforated pipe should be connected via a solid collector pipe to an approved point appropriate discharge facility.

31. Wall backfill should be moisture conditioned and compacted to a minimum of 90% of maximum dry density. If heavy compaction equipment will be used for compaction of the wall backfill, the wall design should include a compaction surcharge in addition to the soil pressures given above. Landset Engineers, Inc. should be consulted for proper compaction surcharge pressures. To avoid surcharging the walls, backfill within 3-feet of the wall should be compacted by hand operated equipment.

Utility Trenches

32. On-site soils should be properly shored and braced during construction to prevent sloughing and caving of trench sidewalls. The contractor should comply with the Cal/OSHA and local safety requirements and codes dealing with excavations and trenches.

33. A select non-corrosive, granular, material should be used as bedding and shading immediately around underground utility pipes and conduits. Native soils may be used for trench backfill above the select material.

34. Trench backfill in landscaped or unimproved areas should be compacted to a minimum of 85 percent of maximum dry density. Trench backfill beneath asphalt and concrete pavements should be compacted to a minimum of 95 percent of maximum dry density. Trench backfill in other areas should be compacted to a minimum of 90 percent of maximum dry density.

35. The bottoms of utility trenches that are parallel to foundations should not extend below an imaginary plane sloping downward at a 1:1 (horizontal to vertical) angle from the bottom outside edges of foundations.

Site Drainage

36. The site soils are highly erodible and a drainage & erosion control plan is essential to the project. Fluctuations of moisture contents are a major consideration, both before and after construction. Site runoff will be increased due to the new paved and roofed surfaced areas. A comprehensive drainage & erosion control plan designed by a Registered Civil Engineers essential to the long-term sustainability of the project.

37. Surface drainage should provide for positive drainage so that runoff is not permitted to pond adjacent to foundations, concrete slabs-on-grade, and pavements. Pervious ground surfaces should be finish graded to direct surface runoff away from site improvements at

a minimum 5 percent grade for a minimum distance of 10-feet. If this is not practicable due to the terrain or other site features, swales with improved surfaces should be provided to divert drainage away from improvements. Surface runoff collected in this swale should be controlled and flow in a non-erosive manner to an approved point of discharge.

38. Roof gutters should be utilized around the building eaves. Roof gutters should be connected to downspouts, which in turn should be connected to pipes leading to the site storm drain system. Runoff from downspouts, planter drains and other improvements should discharge in a non-erosive manner away from site improvements in accordance with the requirements of the governing agencies.

39. The migration of water or spread of root systems below foundations, slabs, or pavements may cause differential movement and subsequent damage. Landscaping runoff collection facilities should be incorporated in the project design.

40. Cut-off drainage swales should be constructed at the top of all cut and fill slopes. These drainage swales should be of adequate size to collect surface runoff and flow to an approved point of discharge in a non-erosive manner. Proper drainage and re-vegetation of graded slopes is essential to ensure stability.

QUALITY CONTROL

The conclusions and recommendations contained in this geologic hazards report and soil engineering feasibility investigation are preliminary in nature. We recommend that Landset Engineers, Inc. be retained to review preliminary plans once they are available. Additionally, we should provide final engineered grading, foundation, and retaining wall design criteria based on a site specific design level investigation(s) once the proposed site improvements, construction type, locations and anticipated loads are known. These services are beyond the scope of this investigation.

The following items should be performed, reviewed, tested, or observed by this firm:

- ***Design level soil engineering investigation(s)***
- ***Final grading & drainage and foundation plans***
- ***Site stripping and clearing***
- ***Overexcavation***
- ***Scarification and recompaction***
- ***Fill placement and compaction***
- ***Foundation excavations***
- ***Surface & subsurface drainage improvements.***
- ***Underground utility backfill and retaining wall construction.***
- ***Compaction of subgrade and Class 2 A.B. in areas to be paved.***

If Landset Engineers, Inc. is not retained to provide design level engineering geologic services, design level soil engineering services, or construction observation and compaction testing, we shall not be responsible for the interpretation of the information by others or any consequences arising therefrom.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The preliminary recommendations contained in this report are based, in part, on certain plans, information, and data that has been provided to us. Any changes in those plans, information, and data will render our recommendations invalid unless we are commissioned to review the changes and to make any necessary modifications and/or additions to our recommendations. The criteria in this report are considered preliminary until such time as they are modified or verified by the soil engineer in the field during construction. No representation, warranty, or guarantee is either expressed or implied. This report is intended for the exclusive use by the client and the client's architect/engineer. Application beyond the stated intent is strictly at the user's risk.

The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, Landset Engineers, Inc. should be notified so that supplemental recommendations can be given.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractor and Subcontractors carry out such recommendations. The conclusions and recommendations contained herein are professional opinions derived in accordance with current and local standards of professional practice.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or in part, by changes outside of our control. Therefore, this report should not be relied upon after a period of three years, without being reviewed by Landset Engineers, Inc. from the date of issuance of this report.

This report does not address issues in the domain of the contractor such as, but not limited to, loss of volume due to stripping of the site, shrinkage of fill soils during compaction, excavatability, and construction methods. The scope of our services did not include any determination or evaluation of soil corrosion potential, environmental assessment of wetlands, radioisotopes, hydrocarbons, hazardous or toxic materials, or other chemical properties in the soil, surface water, groundwater or air, on or below or around the site.

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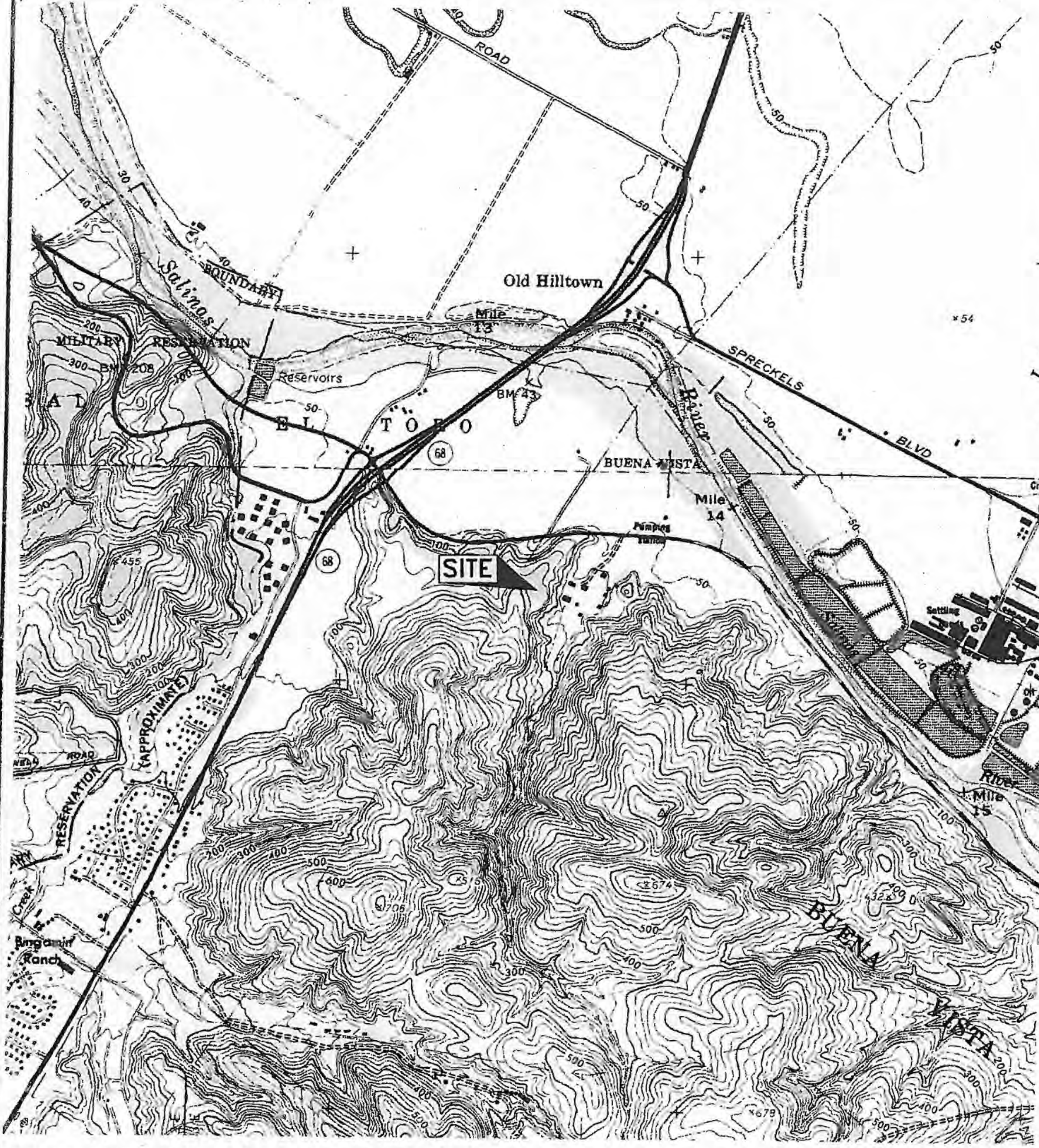
______June 16, 1978, 06053-178-122 and 06053-178-123, vertical false color infrared, approximate scale 1:24,000.

______1994, NAPP-6356-191L, vertical false black and white, approximate scale 1:7,920.

FIGURES

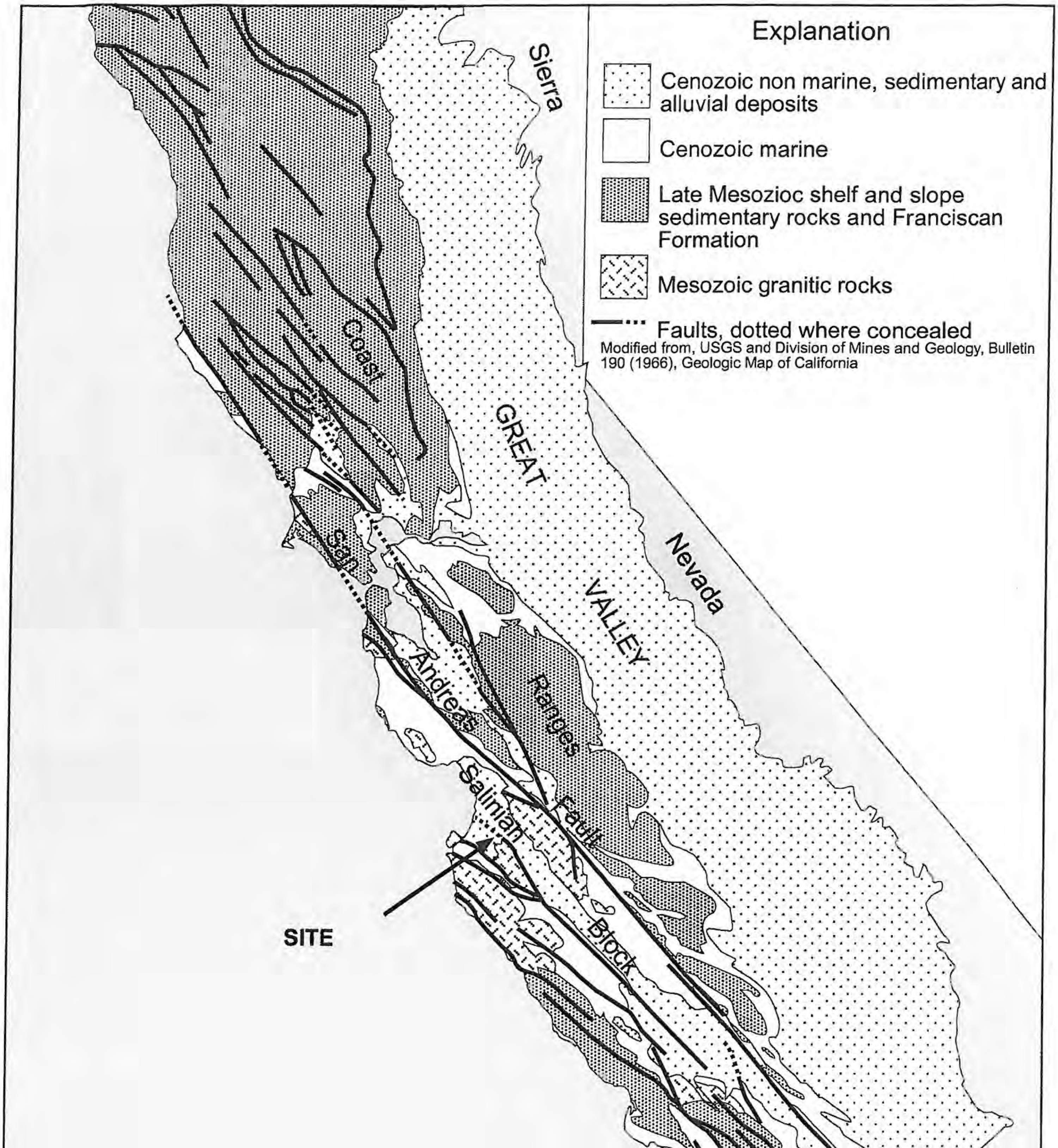


BASE MAPS: Salinas & Spreckels Point, California
U.S.G.S. 7.5' Topographic
Quadrangle Maps

Scale: 1"=2000'

N

LANDSET ENGINEERS, INC. 520B CRAZY HORSE CANYON ROAD, SALINAS, CA 93907 (831) 443-6970 FAX(831) 443-3801	**Vicinity Map** Riverview at Las Palmas (APN 139-211-035) Woodridge Court Monterey County, California	FIGURE 1 PROJECT 1272-01



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Regional Geologic Map

Riverview at Las Palmas
(APN 139-211-035)
Woodbridge Court
Monterey County, California

FIGURE
2

PROJECT
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After Clark, Brabb & Rosenberg, 2000

Approximate Scale: 1" = 2,000'

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Geologic Vicinity Map
Riverview at Las Palmas (APN 139-211-035)
Woodridge Court
Monterey County, California

FIGURE
3

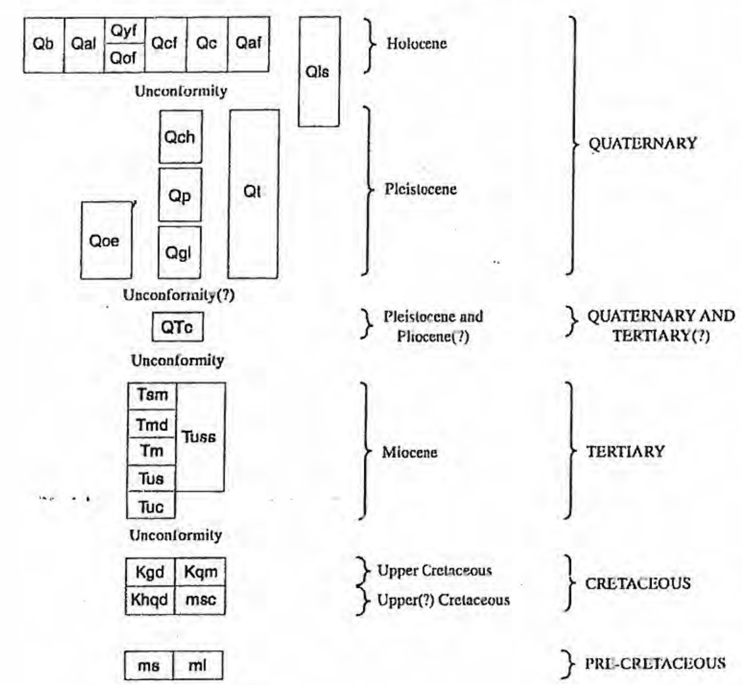
PROJECT
1272-01

LIST OF MAP UNITS

[See accompanying pamphlet for more detailed Description of Map Units]

- **Qb** **Basin deposits (Holocene)**—Unconsolidated, plastic clay and silty clay containing much organic material; locally contains interbedded thin layers of silt and silty sand
- **Qal** **Alluvial deposits, undivided (Holocene)**—Unconsolidated, heterogeneous, moderately sorted silt and sand with discontinuous lenses of clay and silty clay
- **Qyf** **Younger flood-plain deposits (Holocene)**—Unconsolidated, relatively fine-grained, heterogeneous deposits of sand and silt; commonly includes relatively thin, discontinuous layers of clay
- **Qcf** **Abandoned channel fill deposits (Holocene)**—Unconsolidated, plastic, poorly sorted clay, silty clay, and silt. Deposited within channels on younger and older flood-plain deposits. Thickness generally less than 3 m
- **Qc** **Colluvium (Holocene)**—Unconsolidated, heterogeneous deposits of moderately to poorly sorted silt, sand, and gravel deposited by slope wash and mass movement
- **Qaf** **Artificial fill (Holocene)**—Heterogeneous mixture of artificially deposited material ranging from well-compacted sand and silt to poorly compacted sediment high in organic content; only locally delineated
- **Qls** **Landslide deposits (Quaternary)**—Heterogeneous mixture of deposits ranging from large block slides of indurated bedrock to debris flows in semiconsolidated sand and clay
- **Qof** **Older flood-plain deposits (Holocene)**—Unconsolidated, relatively fine-grained, heterogeneous deposits of sand and silt, commonly includes relatively thin layers of clay
- **Qch** **Alluvial fan deposits of Chualar (Pleistocene)**—Weakly consolidated, moderately to poorly sorted sand, silt, and gravel deposited as a series of alluvial fans flanking the Salinas Valley, south of Spreckels. Unit age is late Pleistocene
- **Qt** **Terrace deposits, undivided (Pleistocene)**—Weakly consolidated to semiconsolidated, moderately to poorly sorted silt, silty clay, sand, and gravel deposited in a fluvial environment
- **Qp** **Alluvial fan deposits of Placentia (Pleistocene)**—Semiconsolidated, moderately to poorly sorted sand, silt, and gravel; gravel content increases toward head of the fan. Similar to the alluvial fan deposits of Chualar (Qch), except capped by well-developed soils. Unit age is middle(?) Pleistocene
- **Qoe** **Older eolian deposits (Pleistocene)**—Moderately well-sorted sand as much as 60 m thick that contains no intervening fluvial deposits
- **Qgl** **Alluvial fan deposits of Gloria (Pleistocene)**—Moderately consolidated, deeply weathered, moderately to poorly sorted sand, silt, and gravel, capped with moderately well drained, maximally developed soils with duripans. Unit age is middle to early(?) Pleistocene
- **QTc** **Continental deposits, undivided (Pleistocene-Pliocene(?))**—Nonmarine, semiconsolidated, poorly sorted, fine- to coarse-grained sand with pebble and cobble gravel interbeds
- **Tsm** **Santa Margarita Sandstone (Miocene)**—Marine and brackish-marine, white, friable, fine- to coarse-grained, arkosic sandstone. Age of unit is late Miocene
- **Tuss** **Unnamed sandstone, undifferentiated (Miocene)**—Marine; buff to light-gray, poorly to well-sorted arkosic sandstone, lithologically similar to unnamed sandstone (Tus). Rests with apparent conformity on the unnamed conglomerate (Tuc). Where Monterey Formation is absent, the unnamed sandstone is not differentiated from the younger Santa Margarita Sandstone, and both units are mapped as Tuss. Age of unit is middle to late Miocene
- **Tmd** **Monterey Formation, diatomite (Miocene)**—Very pale orange to white, soft, punky, commonly silty; Mohnian Stage
- **Tm** **Monterey Formation, porcelanite (Miocene)**—Light-brown to white, hard, brittle, platy; Mohnian Stage
- **Tus** **Unnamed sandstone (Miocene)**—Marine; buff to light-gray, poorly to well-sorted arkosic sandstone, locally friable, locally conglomeratic. Age of unit is middle Miocene
- **Tuc** **Unnamed conglomerate (Miocene)**—Nonmarine; buff to light-gray, poorly sorted sandy cobble conglomerate, well-indurated. Age of
- **Kgd** **Granodiorite of Cachagua of Ross (1976a) (Late Cretaceous)**
- **Kqm** **Garnetiferous quartz monzonite of Pine Canyon of Ross (1976a) (Late Cretaceous)**
- **Khqd** **Hornblende-biotite quartz diorite and diorite of Corral de Tierra (Late? Cretaceous)**
- **msc** **Schist of the Sierra de Salinas of Ross (1976a) (Late? Cretaceous)**
- **ms** **Quartzofeldspathic rocks (pre-Cretaceous)**
- **ml** **Marble (pre-Cretaceous)**

CORRELATION OF MAP UNITS



EXPLANATION

Contact—Dashed where approximately located or gradational, dotted where concealed, queried where questionably located

Fault—Dashed where inferred, dotted where concealed, queried where doubtful. U, relatively upthrown side; D, relatively downthrown side

Fold—Dashed where approximately located, dotted where concealed. Showing direction of plunge

Anticline

Syncline

Strike and dip of beds

Horizontal

Inclined

Overturned

Vertical

Ash Bed

Foliation

Water well

Dry hole drilled for hydrocarbon exploration

Landslide– Arrows show general direction of movement. Query indicates that identification of feature as a landslide is in doubt

Spring

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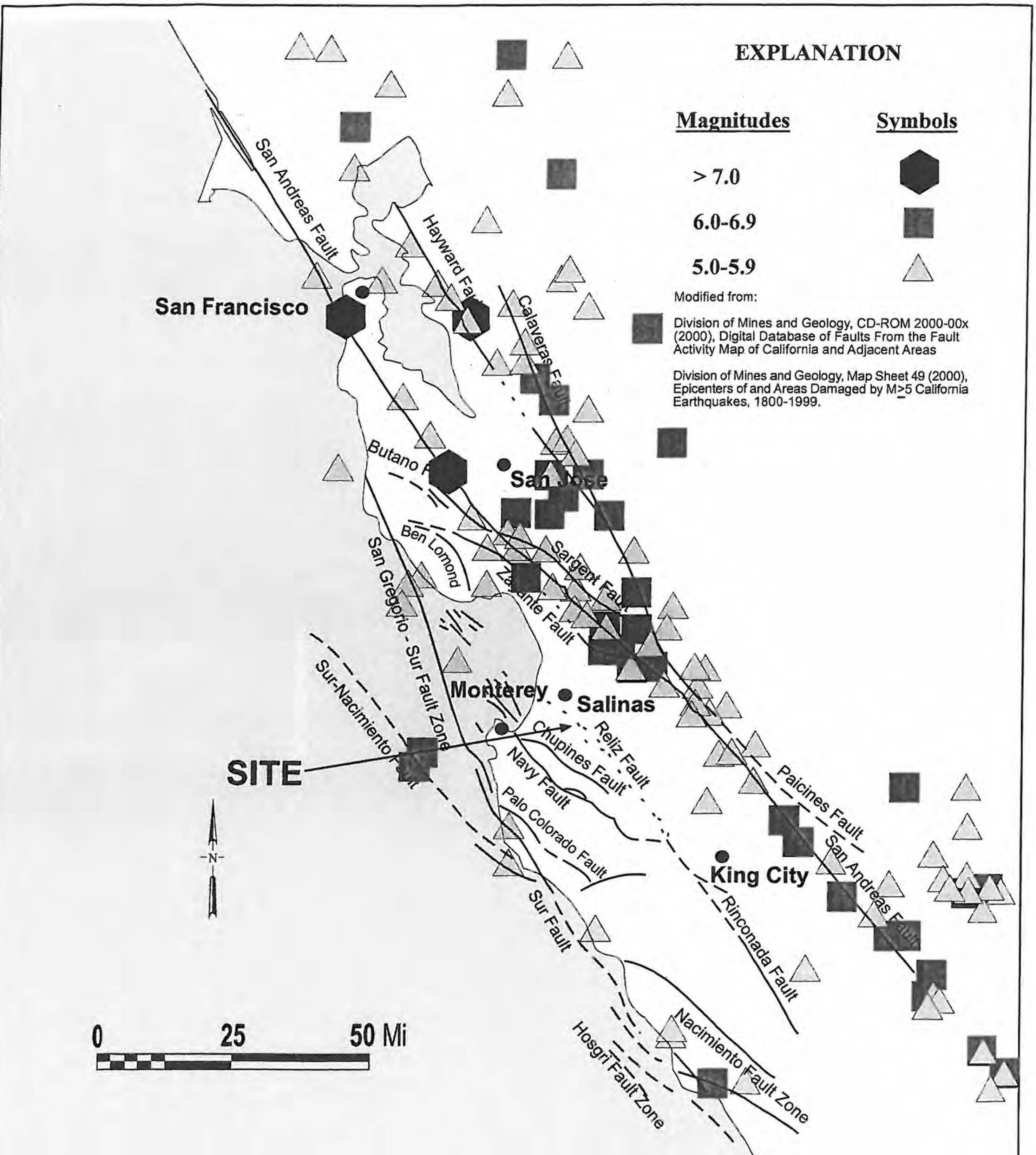
Explanation to Geologic Vicinity Map

Riverview at Las Palmas (APN 139-211-035)
Woodridge Court
Monterey County, California

FIGURE 4

PROJECT 1272-01



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Regional Fault and Seismicity Map
Riverview at Las Palmas
(APN 139-211-035)
Woodbridge Court
Monterey County, California

FIGURE
5

PROJECT
1272-01

APPENDIX A

Unified Soil Classification Systems
Soil Terminology
Logs of Exploratory Test Pits TP-1 through TP-13

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
				GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
	More than 50% of coarse fraction retained on No. 4 sieve.	GRAVELS WITH FINES		GM	Silty gravel, gravel-sand-silt mixtures.
				GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	CLEAN SAND		SW	Well-graded sands, gravelly sands, little or no fines.
More than 50% of material is larger than No. 200 sieve size.		(Little or no fines)		SP	Poorly-graded sands, gravelly sands, little or no fines.
	More than 50% of coarse fraction passing No. 4 sieve.	SAND WITH FINES		SM	Silty sands, sand-silt mixtures.
		(Appreciable amount of fines)		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
				OL	Organic silts and organic silty clay of low plasticity.
More than 50% of material is smaller than No. 200 sieve size.		LIQUID LIMIT GREATER THAN 50		MH	Inorganic silty, micaceous or diatomaceous fine sand or silty soils.
				CH	Inorganic clays of high plasticity, fat clays.
				OH	Organic clays or medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents.
VARIOUS SOILS AND MAN MADE MATERIALS					Fill materials.
MAN MADE MATERIALS					Asphalt and concrete.

SOIL TERMINOLOGY

SOIL TYPES (Ref. 1)

Boulders:	Particles of rock that will not pass a 12 inch screen.
Cobbles:	Particles of rock that will pass a 12 inch screen, but not a 3 inch sieve.
Gravel:	Particles of rock that will pass a 3 inch sieve, but not a No.4 sieve.
Sand:	Particles that will pass a No. 4 sieve, but not a No. 200 sieve.
Silt:	Soil that will pass a No. 200 sieve, that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry.
Clay:	Soil that will pass a No. 200 sieve, that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength when dry.

MOISTURE AND DENSITY

Moisture Condition:	An observational term; dry, slightly moist, moist, very moist, saturated.
Moisture Content:	The weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage.
Dry Density:	The pounds of dry soil in a cubic foot of soil.

DESCRIPTORS OF CONSISTENCY (Ref. 3)

Liquid Limit:	The water content at which a No. 40 soil is on the boundary between exhibiting liquid and plastic characteristics. The consistency feels like soft butter.
Plastic Limit:	The water content at which a No. 40 soil is on the boundary between exhibiting plastic and semi-solid characteristics. The consistency feels like stiff putty.
Plasticity Index:	The difference between the liquid limit and the plastic limit, i.e. the range in water contents over which the soil is in a plastic state.

MEASURES OF CONSISTENCY OF COHESIVE SOILS (CLAYS) (Ref's. 2 & 3)

Very soft	N=0-1 *	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000 psf	Dented slightly by a pencil point

* N = Blows per foot in the Standard Penetration Test. In cohesive soils, with the 3" diameter sampler, 140 pound weight, divide the blow count by 1.2 to get N (Ref. 4).

MEASURES OF RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS AND SILTS) (Ref's. 2 & 3)

Very Loose	N=0-4 **	RD=0-30	Easily push a 1/2" reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2" reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2" reinforcing rod
Dense	N=31-50	RD=70-90	Drive a 1/2" reinforcing rod 1 foot
Very Dense	N>50	RD=90-100	Drive a 1/2" reinforcing rod a few inches

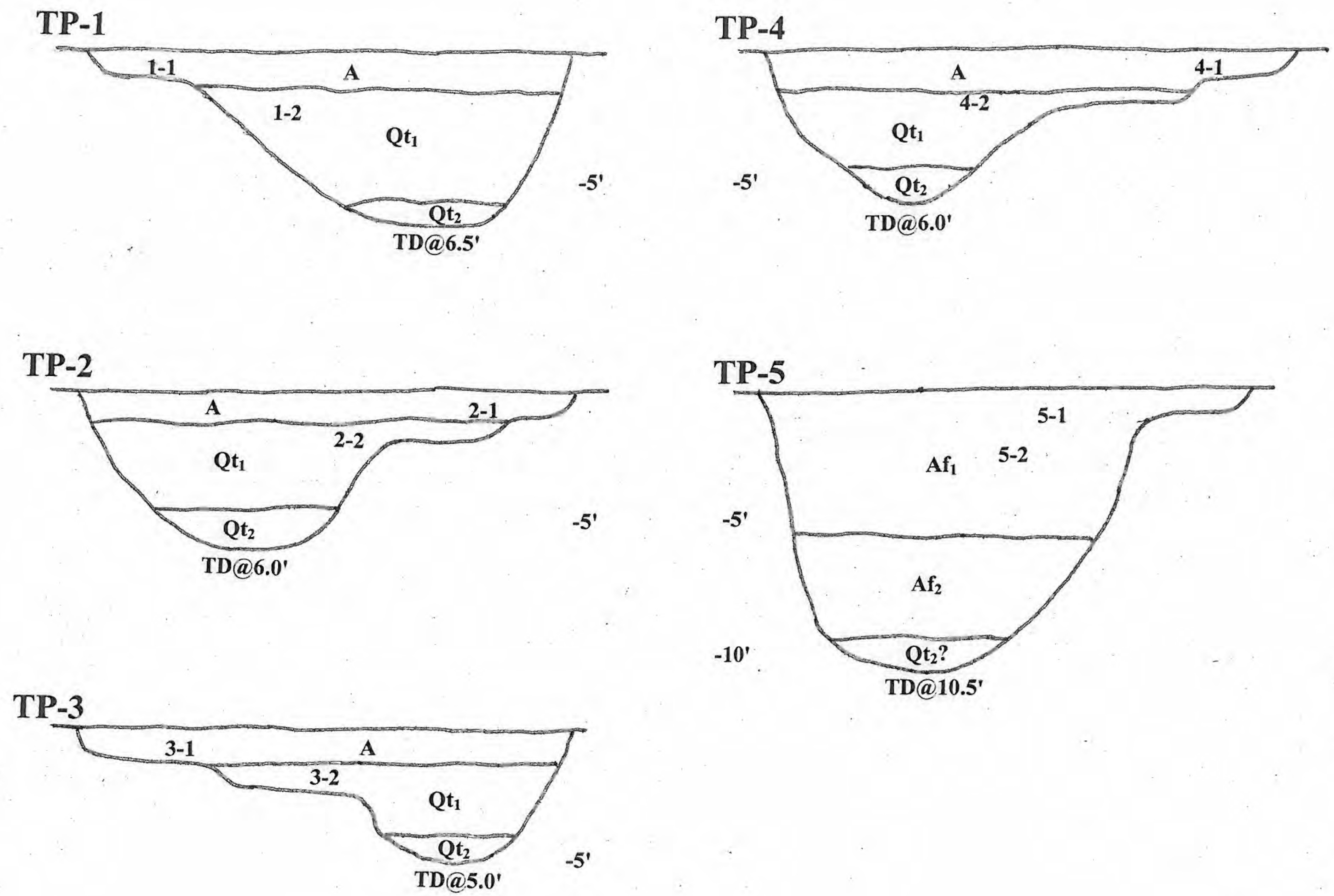
** N = Blows per foot in the Standard Penetration Test. In granular soils, with the 3" diameter sampler, 140 pound weight, divide the blow count by 2 to get N (Ref. 4). RD = Relative Density

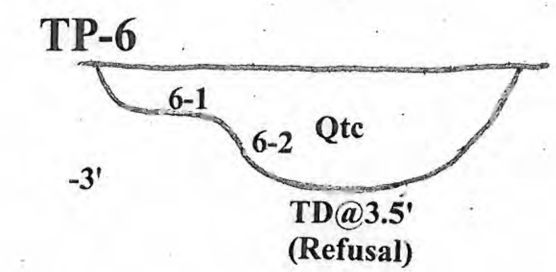
Ref. 1: ASTM Designation: D 2487-93, Standard Classification of Soils for Engineering Purposes (Unified Soils Classification System).

Ref. 2: Terzaghi, Karl, and Peck, Ralph B., Soil Mechanics in Engineering Practice, John Wiley & Sons, New York, 2nd Ed., 1967, pp. 30, 341, 347.

Ref. 3: Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, Macmillan Publishing Company, New York, 4th Ed., 1979, pp. 80,81 and 312.

Ref. 4: Lowe, John III, and Zaccheo, Phillip F., Subsurface Explorations and Sampling Chapter 1 in "Foundation Engineering Handbook," Hsai-Yang Fang, Editor, Van Nostrand Reinhold Company, New York, 2nd Ed., 1991, p. 39.





Explanation

Af_1: Artificial fill: Man made undocumented fill: Moderate gray clayey sand, loose to medium dense, moist, well graded, 35-40% fines.

Af_2: Artificial trash fill: Decomposing tree limbs & branches supported in dark gray clayey sand matrix.

A: Topsoil: Moderate brown silty SAND, loose, dry, 25-35% fines, rooted.

Qt_1: Terrace deposits (Pleistocene) – Weakly consolidated fluvial sediments composed of moderate brown silty SAND, medium dense to dense, slightly moist, well graded, 25-40% fines, trace gravel.

Qt_2: Terrace deposits (Pleistocene) – Semi-consolidated fluvial sediments composed of very pale orange silty SAND with gravel, dense to very dense, slightly moist, 15-25% fines.

Qtc: Continental deposits (Pleistocene-Pliocene (?)) – Semi-consolidated nonmarine sediments composed of very pale orange well graded SAND with gravel and cobbles, dense to very dense, slightly moist, 15-25% fines, locally cemented.

6-2: Approximate location of moisture/density test.

Scale: 1" = 5'

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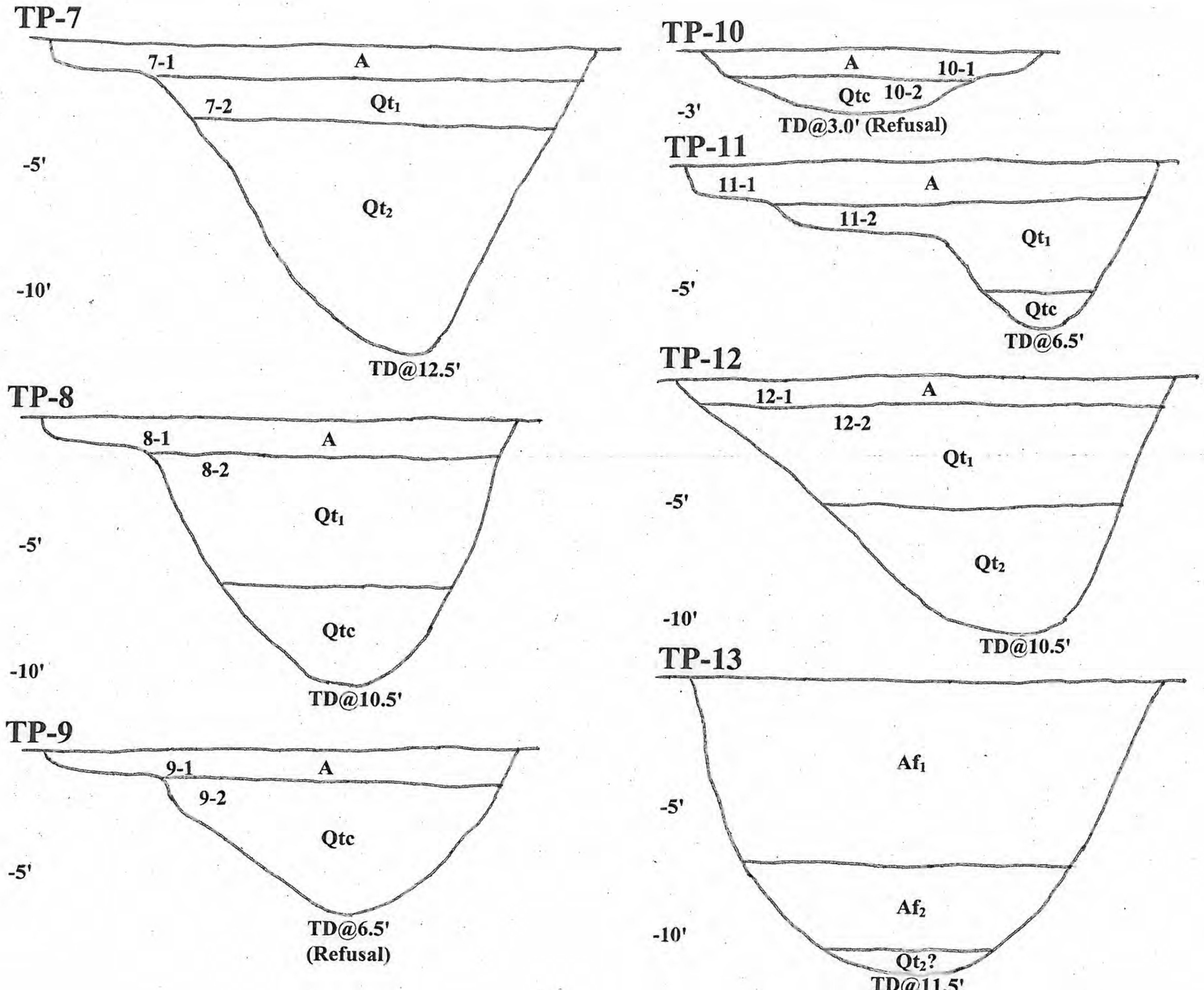
Logs of Exploratory Test Pits
TP-1 to TP-6
Riverview at Las Palmas (APN 139-211-035)
Woodridge Court
Monterey County, California

FIGURE
A3

PROJECT
1272-01



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Qtc: Continental deposits (Pleistocene-Pliocene (?)) – Semi-consolidated nonmarine sediments composed of very pale orange well graded SAND with gravel and cobbles, dense to very dense, slightly moist, 15-25% fines, locally cemented.

12-2: Approximate location of moisture/density test.

Scale: 1" = 5'

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Logs of Exploratory Test Pits
TP-7 to TP-13
Riverview at Las Palmas (APN 139-211-035)
Woodridge Court
Monterey County, California

FIGURE
A4

PROJECT
1272-01

APPENDIX B

Laboratory Test Results

Table B-1
Summary of Laboratory Test Results

Sample No.	Depth (ft.)	Dry Density (pcf)	Water Content (%)
1-1	1.0-1.5	90.7	5.6
1-2	2.0-2.5	94.1	7.3
2-1	1.0-1.5	91.1	4.9
2-2	2.0-2.5	95.2	8.1
3-1	1.0-1.5	107.7	8.1
3-2	2.0-2.5	105.2	12.3
4-1	1.0-1.5	87.9	4.4
4-2	2.0-2.5	100.3	9.7
5-1	1.0-1.5	110.4	6.4
5-2	2.0-2.5	106.5	12.5
6-1	1.0-1.5	83.6	9.9
6-2	3.0-3.5	91.7	11.8
7-1	1.0-1.5	82.6	9.1
7-2	2.5-3.0	98.7	9.9
8-1	1.0-1.5	83.2	5.4
8-2	2.0-2.5	92.1	11.1
9-1	1.0-1.5	88.1	5.4
9-2	2.0-2.5	91.8	8.8
10-1	1.0-1.5	92.2	7.9
10-2	2.0-2.5	83.5	12.0
11-1	1.0-1.5	86.8	5.1
11-2	2.0-2.5	92.8	9.8
12-1	1.0-1.5	86.2	5.8
12-2	2.0-2.5	96.4	7.2

Summary of
Compaction Curves
(ASTM D 1557)

Sample No.	Description & (Source)	USCS	Density p.c.f.	Optimum Moisture % dry wt.
A	Brown silty SAND (TP-1 @ 0.0-2.0)	SM	129.0	8.0
D	Light brown silty SAND w/gravel (TP-9 @ 0.0-5.0)	SM	129.0	7.0

Summary of Atterberg Limits Test Results

Sample No.	Depth (ft.)	Liquid Limit	Plastic Limit	Plasticity Index
Bulk B, TP-5	0.0-5.0	17	14	3
Bulk C, TP-6	0.0-3.0	16	14	2

Summary of Sieve Analysis Test Results, Sample B (TP-5 – 0.0'-5.0')

Sieve No.	Cumulative % Retained	Cumulative % Passing
#4	0.2	99.8
#8	2.9	97.1
#16	10.3	89.7
#30	20.7	79.3
#50	35.3	64.7
#100	48.4	51.6
#200	59.9	40.1

Summary of Sieve Analysis Test Results, Sample B (TP-6 – 0.0'-3.0')

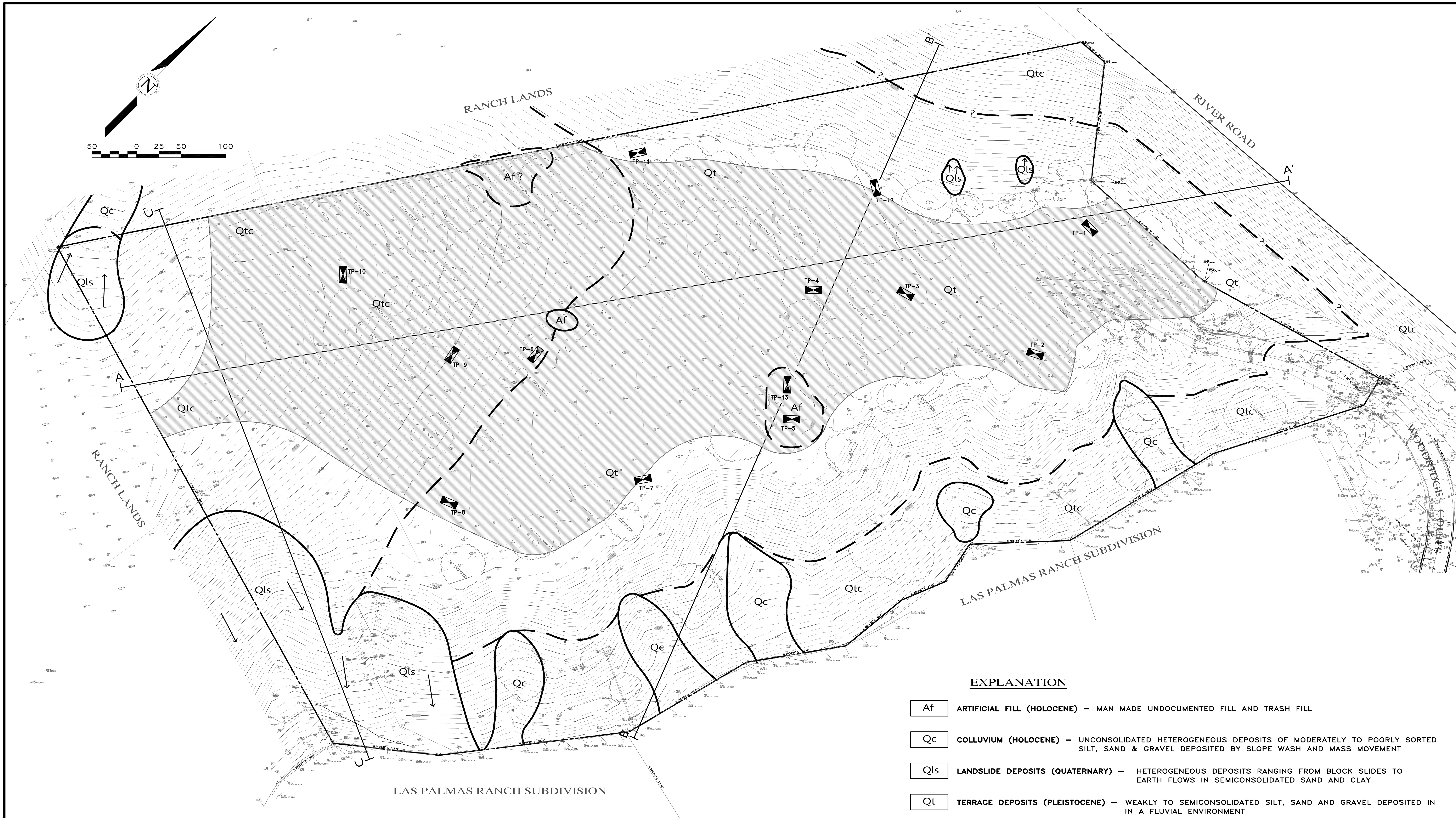
Sieve No.	Cumulative % Retained	Cumulative % Passing
#4	0.1	99.9
#8	2.6	97.4
#16	8.6	91.4
#30	18.7	81.3
#50	32.5	67.5
#100	43.1	56.9
#200	54.3	45.7

Summary of Sieve Analysis Test Results, Sample D (TP-9 – 0.0'-5.0')

Sieve No.	Cumulative % Retained	Cumulative % Passing
#4	3.4	96.6
#8	11.0	89.0
#16	23.2	76.8
#30	36.4	63.6
#50	51.6	48.4
#100	62.0	38.0
#200	73.9	26.1

Summary of Sieve Analysis Test Results, Sample E (TP-7 – 0.0'-5.0')

Sieve No.	Cumulative % Retained	Cumulative % Passing
#4	0.6	99.4
#8	4.7	95.3
#16	14.4	85.6
#30	26.3	73.7
#50	42.7	57.3
#100	57.5	42.5
#200	71.9	28.1



EXPLANATION

Af — **ARTIFICIAL FILL (HOLOCENE)** — MAN MADE UNDOCUMENTED FILL AND TRASH FILL

Qc — **COLLUVIUM (HOLOCENE)** — UNCONSOLIDATED HETEROGENEOUS DEPOSITS OF MODERATELY TO POORLY SORTED SILT, SAND & GRAVEL DEPOSITED BY SLOPE WASH AND MASS MOVEMENT

Qls — **LANDSLIDE DEPOSITS (QUATERNARY)** — HETEROGENEOUS DEPOSITS RANGING FROM BLOCK SLIDES TO EARTH FLOWS IN SEMICONSOLIDATED SAND AND CLAY

Qt — **TERRACE DEPOSITS (PLEISTOCENE)** — WEAKLY TO SEMICONSOLIDATED SILT, SAND AND GRAVEL DEPOSITED IN IN A FLUVIAL ENVIRONMENT

Qtc — **CONTINENTAL DEPOSITS (PLEISTOCENE–PLIOCENE(?))** — NONMARINE, CONSOLIDATED FINE TO COARSE GRAINED SAND WITH PEBBLE AND COBBLE GRAVEL INTERBEDS

? **CONTACT** — DASHED WHERE APPROXIMATELY LOCATED, QUERIED WHERE QUESTIONABLY LOCATED

LANDSLIDE — ARROWS SHOW GENERAL DIRECTION OF MOVEMENT

TP-13 **TEST PIT** — APPROXIMATE LOCATION OF EXPLORATORY TEST PIT

C C' **CROSS SECTION** — LOCATION OF GEOLOGIC CROSS SECTION (SEE SHEET 2)

GEOLOGICALLY SUITABLE BUILDING ENVELOPE

PROFESSIONAL GEOLOGIST BRIAN E. PAPURELLO No. CEG 2226 CERTIFIED ENGINEERING GEOLOGIST Exp. 02-28-16 STATE OF CALIFORNIA

APPROVED BY:

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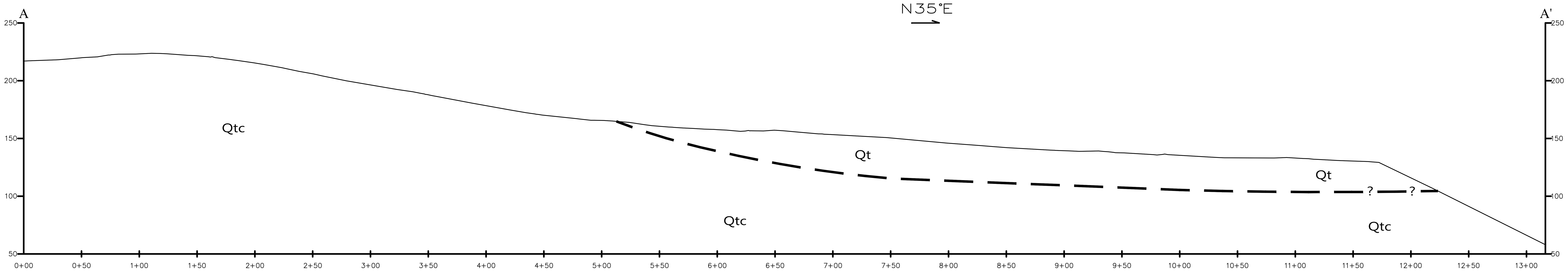
ENGINEERING – LAND PLANNING
SURVEYING – ENVIRONMENTAL CONSULTING
520-B CRAZY HORSE CANYON ROAD, SALINAS, CA 93907

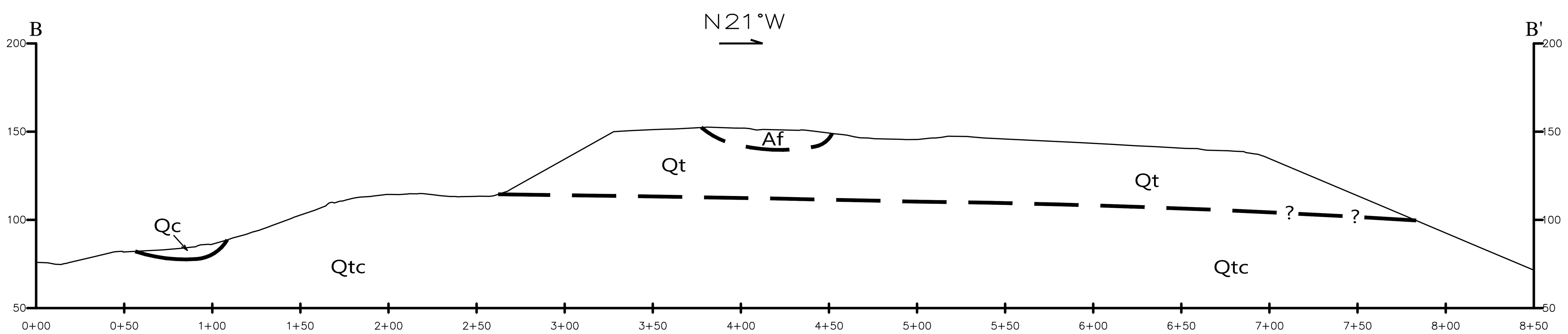
SITE GEOLOGIC MAP OF RIVERVIEW AT LAS PALMAS

APN: 139-211-035
MONTEREY COUNTY, CALIFORNIA
FOR
RIVER VIEW AT LAS PALMAS, LLC

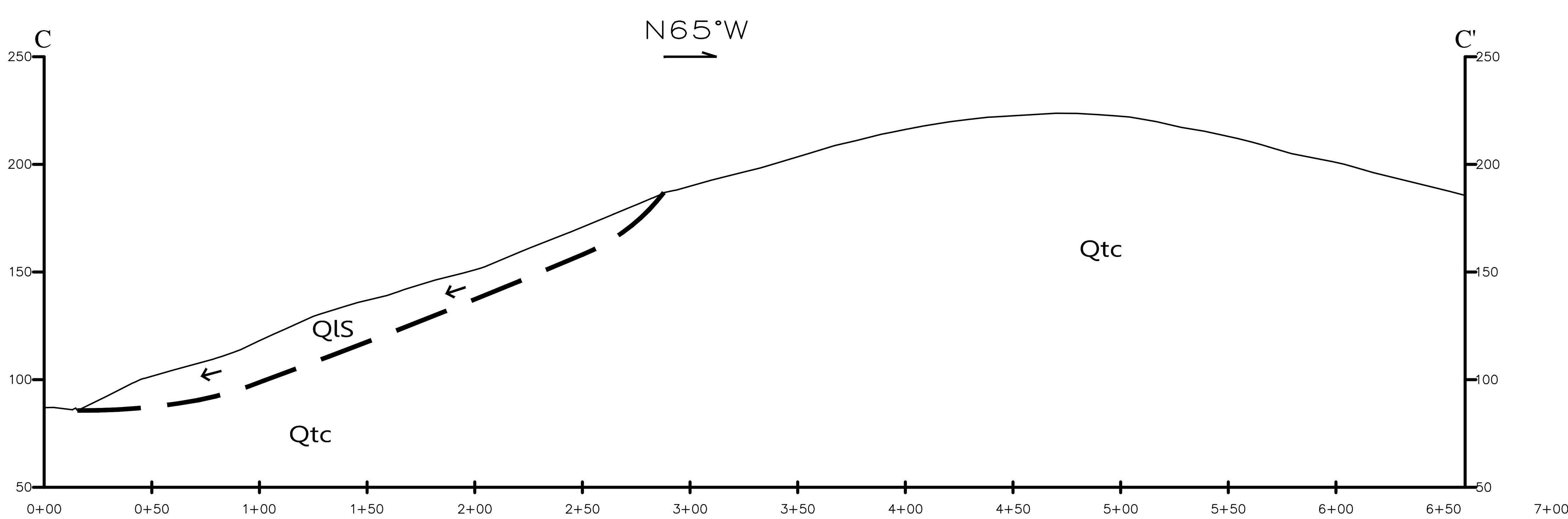
SCALE: 1"=50'
DATE: FEBRUARY 2014
JOB NO. 1272-01
SHEET 1
OF 2 SHEETS



SECTION A - A'
SCALE: 1"=50' H&V



SECTION B - B'
SCALE: 1"=50' H&V



SECTION C - C'
SCALE: 1"=50' H&V

EXPLANATION

Af	**ARTIFICIAL FILL (HOLOCENE)** – MAN MADE UNDOCUMENTED FILL AND TRASH FILL
Qc	**COLLUVIUM (HOLOCENE)** – UNCONSOLIDATED HETEROGENEOUS DEPOSITS OF MODERATELY TO POORLY SORTED SILT, SAND & GRAVEL DEPOSITED BY SLOPE WASH AND MASS MOVEMENT
Qls	**LANDSLIDE DEPOSITS (QUATERNARY)** – HETEROGENEOUS DEPOSITS RANGING FROM BLOCK SLIDES TO EARTH FLOWS IN SEMICONSOLIDATED SAND AND CLAY
Qt	**TERRACE DEPOSITS (PLEISTOCENE)** – WEAKLY TO SEMICONSOLIDATED SILT, SAND AND GRAVEL DEPOSITED IN IN A FLUVIAL ENVIRONMENT
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?	**CONTACT** – DASHED WHERE APPROXIMATELY LOCATED, QUERIED WHERE QUESTIONABLY LOCATED
	LANDSLIDE – ARROWS SHOW GENERAL DIRECTION OF MOVEMENT

PROFESSIONAL GEOLOGIST ★ BRIAN E. PAPURELLO No. CEG 2226 CERTIFIED ENGINEERING GEOLOGIST Exp. 02-29-16 ★ STATE OF CALIFORNIA

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520-B CRAZY HORSE CANYON ROAD, SALINAS, CA 93907

GEOLOGIC CROSS SECTIONS " A-A', B-B' & C-C' "
OF
RIVERVIEW AT LAS PALMAS
APN: 139-211-035
MONTEREY COUNTY, CALIFORNIA
FOR
RIVER VIEW AT LAS PALMAS, LLC

SCALE: 1"=50' H&V
DATE: FEBRUARY 2014
JOB NO. 1272-01
SHEET 2
OF 2 SHEETS

1272-RIVER VIEW LAS PALMAS\DWG\1272-SOILS\SHT2