Attachment H
Preliminary Geologic Investigation
Prepared by
Moore Twining
(LIB080087)

PLN060603

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MOORE TWINING ASSOCIATES, INC.
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GEOTECHNICAL ENGINEERING • DRILLING SERVICES
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LIB080087

PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED RANCHO EL POTRERO TEN (10) LOT RESIDENTIAL SUBDIVISION, PLN060603 APN 157-181-006, 007, AND 008 CARMEL, MONTEREY COUNTY, CALIFORNIA

Project Number: D69201.01-01

For:

Mr. Jeff Taylor
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January 16, 2008

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January 16, 2008 D69201.01-01

Mr. Jeff Taylor
Heritage Development
280 Corral de Tierra
Salinas, CA 93908

Subject: Preliminary Geotechnical Engineering and Geologic Investigation
Proposed Rancho El Potrero
Ten (10) Lot Residential Subdivision, PLN060603
APN 157-181-006, 007, and 008
Carmel, Monterey County, California

Dear Mr. Taylor:

We are pleased to submit this Preliminary Geotechnical Engineering and Geologic Investigation report prepared for the proposed ten (10) lot residential subdivision, Carmel, Monterey County, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

This investigation was conducted to summarize anticipated geotechnical and geologic feasibility issues which could impact the proposed development and to provide preliminary recommendations for site grading and preliminary geotechnical design parameters for planning purposes. The report is also intended to provide recommendations for rough grading of the project roadways and driveways, and rough grading of the proposed Lot 1 equestrian facilities. Prior to final grading and paving of the roadways for the development, additional sampling and testing of the subgrade soils should be conducted to provide recommendations for final pavement section design. Also, individual design level geotechnical investigations should be conducted prior to development of final grading plans and construction on all of the lots.

We appreciate the opportunity to be of service to Heritage Development. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
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Kenneth J. Clark, CEG
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KC/pc

EXECUTIVE SUMMARY

The intent of this report is to investigate the proposed new home site areas, provide a preliminary geotechnical and geologic characterization of the subsurface conditions, and provide a preliminary assessment of the geologic and geotechnical feasibility of the proposed development for planning and preliminary design purposes. In addition, the report is also intended to provide recommendations for rough grading the proposed roadways and driveways, and rough grading the Lot 1 equestrian facilities.

It should be noted that this investigation did not include review or assessment of conditions for existing structures. In addition, areas outside the proposed home sites were not investigated except as described in the report to assess the geologic profile pertinent to the proposed home site areas. The valley area and northwest facing slopes, outside the proposed home site area on Lot 10 were not investigated.

The proposed subdivision development is located within an approximate 103.2 acre area west of Rancho San Carlos Road and south of the Carmel River and Quail Lodge Resort golf course, Carmel, Monterey County, California (see the Vicinity Map, Drawing No. 1, Appendix A). The majority of the proposed development is located on a heavily vegetated north facing hillside. Lot 10 encompasses 82.1 acres and extends from the golf course to the about ½ mile south, and includes the lower alluvial plain areas of the site and a broad heavily vegetated valley area.

The proposed lot sizes for Lots 1 through 9 range from about 1.2 to 4.2 acres. Lot 10 comprises the remaining 82.1 acre portion of the site and wraps around the west, south, and east sides of Lots 1 through 9. Slope gradients across Lots 1 through 9 range from as steep as about 3 horizontal (H) to 1 vertical (V) at the higher (south) lots to about 4H to 1V to 8H to 1V at the lower lots. Lot 10 encompasses 82.1 acres and extends from the golf course to as far as about ½ mile south. Lot 10 includes the lower alluvial plain areas of the site, a broad heavily vegetated valley area in the central portion of the site, and the relatively steep slopes in the higher elevation (southern) portions of the site. Slope gradients within Lot 10 range from nearly flat at the alluvial plain (northern) portions to as steep as about 1½H to 1V along the uppermost (southern) portions of the site.

An incised drainage ravine is located in the east portion of the development site, between lots 4 and 9 on the west and lots 10 and 3 on the east. Ground slope gradients near the ravine are as steep as nearly vertical. In some areas, the near vertical slopes bordering the ravine are 20 feet or more in height.

The proposed home site areas (about ¼ acre to 2 acres areas) had generally been cleared of heavy brush at the time of our field investigations in May and November 2007. Based on our observations and discussions with Mr. Taylor, it is our understanding that grading has not be conducted on the cleared home site prior to our field observations. The ground surface at the cleared lots was generally covered with green native grasses, about 1 to 2 feet high. Native vegetation, including mature trees and generally thick brush with grasses was pervasive outside the cleared home site areas.

EXECUTIVE SUMMARY

At the time of the field investigation, an existing residence was noted in the north central portion of the site, downslope (north) of the Lot 10 home site. Out buildings were also noted in the central portion of Lot 10. It is our understanding that the residence and buildings will remain after development of the site. Other existing improvements noted on the site included three (3) water wells located just east of Lot 8, just north of Lot 5, and in the north-central portion of Lot 10. Based on our site review, it appears that underground irrigation/electrical utilities have been installed at some of the lots including in the vicinity of the wells. It is our understanding that the existing water wells will remain after development of the site.

Unimproved access roads transect the north portion of the site, and transect the project site from the area of the southwestern most lots, to the two northernmost lots. The roads appear to have been graded by cut and fill type grading.

It is our understanding that the proposed new residential structures will consist of one or two-story buildings with predominantly wood-frame construction and concrete slabs-on-grade or raised wood floor systems supported on shallow foundation systems.

Considering the existing ground slope gradients on some of the lots, basements or partial basements and building and landscape retaining walls would be anticipated to support grade changes. It is anticipated that cut and fill thicknesses of as much as 10 feet would be required to create relatively level pads for the residential structures. For the purpose of this report, a maximum cut and fill slope depth of 10 feet is assumed.

Appurtenant construction, including asphaltic concrete paved roadways and driveways, and underground utilities, is anticipated.

At the time of preparation of this report, the details of the proposed construction type and magnitude of foundation loads for the proposed structures were not known. For the purpose of this feasibility evaluation, anticipated foundation loads for the typical residential structures were assumed to be 25 kips or less for interior column loads and 2.5 kips / foot or less for continuous footings. Design level geotechnical investigations should consider the actual design building loads.

It is our understanding that on-site sewage disposal systems will not be used for the project. On-site disposal systems, as well as unlined storage ponds, are not recommended due to the infiltration of water and potential for slope instability.

Literature research included review of the conceptual plans provided, review of aerial photographs, and review of geologic, fault, and seismicity maps, and reports. The results of the literature research are presented under the Geologic Hazards and Geologic and Geotechnical Feasibility sections of this report.

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The site is located about 29 miles (47 km) southwest of the seismically active San Andreas Fault zone and 5 miles (8 km) east of the San Gregorio fault zone. These two fault zones mark the northeastern and southwestern boundaries, respectively, of the Salinian block with its crystalline basement of granitic and regionally metamorphosed rocks. A series of high-angle faults comprising the Monterey Bay - Tularcitos fault zone trend north-westward across the site region. This zone is a complex, generally northwest-striking fault zone up to 15 km wide, which includes several active (Holocene) and potentially active (Quaternary) faults.

The Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997), indicates three primary geologic materials in the area of the project site; Tertiary Monterey Formation - porcelanite; Older Holocene flood plain deposits, and Quaternary landslide deposits. Surface exposures of Miocene marine sandstone are mapped about 1 mile west of the site. Surface exposures of Cretaceous porphyritic granodiorite are mapped about 1¼ mile southwest of the site.

Old landslide deposits are pervasive in the Carmel/Monterey area and have been extensively documented in the Santa Lucia Preserve area, east of the site. Numerous developments have been constructed on old landslide materials that have been determined to be stable under current conditions. These developments include the Santa Lucia Preserve and Quail Meadows residential developments, and Carmel Valley Ranch Golf Club.

Grading requirements for developments typically stipulate provisions to effectively remove surface water from slopes and reduce shallow groundwater seepage, which decreases the potential for erosion and landslides. Recommendations are provided in this report to reduce the potential for slope instability on the subject site (see Recommendations section of this report), including surface and subsurface drainage measures, slope grading limitations, etc.

Based on review of published geologic maps and our site investigations, it appears that the majority of the project site area is covered with Quaternary landslide deposits. Clark, Dupre, and Rosenberg (1997) describe the Quaternary landslide deposits as "Heterogenous mixture of deposits ranging from large block slides in indurated bedrock to debris flows in semi-consolidated sand and clay." Surface soils in the lower (northernmost) portions of the site (at the toe of the site hillside slope) comprise Older Holocene flood plain deposits.

The geological and geotechnical investigation reports (Cleary Consultants, dated February 15, 1994 and August 22, 2000), for the Santa Lucia Preserve area located east of the site (as close as about 1 mile east of the project site) describe three types of landslides present in that area as: active landslides, dormant landslides, and old landslides, differentiated based on geomorphologic conditions. This report utilizes a similar approach to characterizing and describing the recency of movement of landslide deposits at the site (see subsection 8.1.1 of this report).

EXECUTIVE SUMMARY

Field exploration was conducted on May 11, 12, 24, and 25 and November 27, 2007, and consisted of site reconnaissance, excavating and logging backhoe test pits, outcrop mapping, drilling test borings, and collecting and logging soil samples, and standard penetration tests.

The results of site observations and subsurface exploration (test pits and borings) suggest that the typical near surface native soil profile at the site consists of an upper organic rich brown to dark grey, silt, "A horizon" soil (topsoil) extending from the ground surface to depths of about 1½ to 5 feet BSG. Scattered, gravel-size and smaller fragments of rock (siltstone, etc.) were noted in these soils. The organic rich topsoils were not encountered in the test borings drilled in roadway cut areas where the upper soils had been previously been removed. The topsoils were generally underlain by an "E horizon" soil comprising a greyish tan silt matrix with significant humus and abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, and a porous and loose appearance. The texture of these soils suggests that the material was deposited as part of an old landslide - earthflow type movement. The E horizon soils, where encountered, were typically about 1 foot thick.

The relatively loose A and E horizon soils were typically underlain by silt soils with a similar composition (i.e. silt matrix with abundant siltstone and porcelanite fragments), also interpreted to be of earthflow origin. Earthflow soils consisting of a lean clay matrix with abundant siltstone and porcelanite fragments were typically noted in test pits underlying the more silty earthflow type soils. Siltstone and granodiorite were encountered below the earthflow soils.

A review of geologic maps, test boring logs and test pit logs suggest that the uppermost silty and lean clay soils across the site area investigated are predominantly earthflow type landslide materials. These materials correlate with the Quaternary landslide deposits noted on the Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997). The silty and lean clay soils noted below the loose near surface soils appeared stiff to hard in the test pit exposures, and were very stiff to hard as indicated by standard penetration resistance, N values, ranging from 31 to greater than 50 blows per foot (test borings).

Site soils predominantly comprise lean clay and silt soils, with some silty sands. The predominant lean clays and silt soils are expected to exhibit very low to low expansion potential and moderate to high compressibility. It should be anticipated that engineered fill will be required below foundations to reduce settlement, and non-expansive soils will be required below slabs-on-grade to reduce the impacts of shrink and swell. The future design level geotechnical investigations for the individual lots should include analyses of the expansiveness and compressibility of the soils and provide recommendations for earthwork (over-excavation and re-compaction), grading, and site preparation to reduce the impact of adverse soil conditions on the residences.

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Groundwater was not encountered in near surface soils in test pits or borings conducted for this investigation. It is anticipated that shallow groundwater will not be pervasive across the site. However, it should be anticipated that shallow perched groundwater will occur in some areas and subdrains will be required to cut-off and redirect shallow subflow away from the residences and roadways. Preliminary recommendations for subdrainage are included in the Recommendations section of this report.

One relatively small landslide was identified based on observations of the home site areas cleared of vegetation. The landslide feature on Lot 3 exhibited a slight increase in downslope gradient at the scarp of the slide and a slight bulge at the toe of the slide. The top of the scarp area was rounded and not angular. The eroded appearance of the headscarp and established vegetation on the slide area suggests that the last significant episode of movement did not likely occur within roughly the last 10 or more years, and this slide is classified as a dormant landslide.

Shallow rotational/translational slides are triggered by saturation of upper poorly drained clayey soils, which causes the soil unit weight to increase and the effective shear strength to decrease. Past observation of numerous similar slides in the Santa Lucia Preserve area indicate that these types of slides occur most prevalently in the spring after long periods of rain. These types of landslides typically do not occur rapidly and slide movement can range from hours to months, or years.

The presence of the relatively shallow dormant landslide feature noted on Lot 3 underscores the potential that other such features may be present on the development site, or may develop in the future. Based on the limited scope of this feasibility level investigation, and the existing ground cover which obscures, additional relatively shallow dormant or active slides may be present that were not identified. Future design level geotechnical/geologic investigations (conducted for each of the proposed home sites/residences) should assess the potential presence of slides and the potential for reactivation of these slides. Mitigation of landslide risk typically includes such measures as removal and recompaction of, or buttressing of the slide and installation of drainage controls. General recommendations for shallow slide repair are provided in subsection 10.7 of this report.

It should be noted that the potential is moderate for shallow landslides to occur outside of the areas to be developed, where mitigative measures are not implemented. However, recommendations provided in this report for mitigative measures such as setbacks, benching and keying fills, surface drainage and subdrainage facilities, etc. to be implemented as a part of the site development, would be expected to significantly decrease the potential for shallow slides occurring in areas to be developed.

Design level investigations should include slope stability analyses based on subsurface investigations at each lot, laboratory soil strength testing results, and the proposed grading configuration.

EXECUTIVE SUMMARY

Preliminary slope stability analyses were conducted to provide a preliminary evaluation of the general stability of the existing native slopes, as well as the stability of future cut and fill slopes. The stability analyses were based in part on approximating soil conditions at the site from direct shear laboratory testing of in-situ soil samples obtained from soil borings, and testing of samples remolded from bulk samples of soils excavated from test pits. The analyses incorporated a generalized topographic and geologic profile based on the results of our field investigation, and included consideration of both static and psuedo-static (seismic) shaking conditions.

Based on the cited literature, results of our field observations, results of our slope stability analyses, and contingent on implementations of the recommendations in this report (including grading, drainage and subdrainage), it is our opinion that the potential would be low for relatively deep seated landslides to occur on the native slopes, or on slopes graded in accordance with the recommendations in this report.

Based on the soil and rock conditions encountered at the proposed home site areas, the potential for liquefaction to occur at the home site locations is also considered low.

The project site is located in Seismic Zone 4. The site is not located in an Alquist-Priolo Earthquake Fault Zone and the potential for fault rupture to occur on the site is estimated to be low.

Based on the evaluation of the field and laboratory data, site reconnaissance, document review and our geotechnical experience in the vicinity of the project, preliminary recommendations are provided in this report for stripping top soils, maximum cut and fill gradients, slope setbacks, surface drainage, subdrainage, over-excavation and support of foundations on engineered fill, non-expansive soils below slabs-on-grade, soil corrosivity, retaining walls, and asphaltic concrete pavement design. Design level geotechnical investigations should be conducted for the individual lot home sites being developed prior to development of residential grading plans and construction of the residences.

Preliminary recommendations are provided herein are to be used for preliminary design and planning purposes except that recommendations in this report may be used for rough grading of the project roadways and driveways, and Lot 1. This report contains specific recommendations for maximum cut and fill slope gradients for roadway construction which differ from the recommendations for pad grading. Prior to final grading and paving of the roadways for the development, additional sampling and testing of the subgrade soils should be conducted to provide recommendations for final pavement section design. Also, individual design level geotechnical investigations should be conducted prior to development of final grading plans and construction on all of the lots.

The results of this preliminary investigation indicate the site is suitable for the proposed residential construction with regard to slope stability, support of the anticipated fill soils, foundations, concrete slabs-on-grade, and pavements provided the recommendations contained in this report and future design level geotechnical reports are followed.

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GEOTECHNICAL ENGINEERING AND
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PROPOSED RANCHO EL POTRERO
TEN (10) LOT RESIDENTIAL SUBDIVISION, PLN060603
APN 157-181-006, 007, AND 008
CARMEL, MONTEREY COUNTY, CALIFORNIA

Project Number: D69201.01-01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering and geologic investigation for the proposed ten (10) lot residential subdivision, Carmel, Monterey County, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by a written agreement by Mr. Jeff Taylor with Heritage Development to conduct this investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations for future studies. The four report appendices contain the drawings (Appendix A), the logs of borings (Appendix B), the results of laboratory tests (Appendix C), and site photographs (Appendix D).

The Geotechnical Engineering Division of Moore Twining, headquartered in Fresno, California, performed the investigation. This report is provided specifically for the proposed 10 lot residential development referenced in the Site Description and Anticipated Construction sections of this report.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The intent of this report is to provide a preliminary geotechnical and geologic characterization of the subsurface conditions, and provide a preliminary assessment of the geologic and geotechnical feasibility of the proposed development for planning and preliminary design purposes. In addition, the report is also intended to provide recommendations for rough grading of the proposed roadways and driveways, and rough grading of the Lot 1 equestrian facilities.

The purpose of the investigation is to provide:

2.1.1 Description of general subsurface soil/rock conditions and groundwater conditions at the site;

2.1.2 Evaluation of potential geologic hazards and feasibility issues including landslides, faults, liquefaction;

2.1.3 Preliminary slope stability analysis and assessment of the stability of native and graded slopes;

2.1.4 Evaluation of potential geotechnical hazards including the potential for expansive soils and foundation settlement;

2.1.5 Preliminary recommendations for grading, design, and construction of building pads and building setbacks from slopes for project planning purposes;

2.1.6 Recommendations for rough grading the proposed subdivision roads and driveways, and preliminary rough grading of the equestrian facility;

2.1.7 Preliminary asphaltic concrete pavement section design recommendations for planning purposes;

2.1.8 Preliminary evaluation of soil corrosivity; and

2.1.9 Recommendations regarding tasks recommended for future evaluations and design level investigations.

It should be noted that this investigation did not include review or assessment of conditions for any existing structures. In addition, areas outside the proposed home sites referenced in this report were not investigated except as necessary to assess the geologic profile pertinent to the proposed home site areas. The valley area and northwest facing slopes on Lot 10, outside the proposed home site area, were not investigated.

Prior to final grading and paving of the roadways, additional sampling and testing of the subgrade soils should be conducted to provide the final pavement design recommendations. Also, individual design level geotechnical investigations should be conducted for the proposed residential lots and the proposed equestrian facilities prior to development of grading plans and design and construction of those residences.

2.2 **Scope**: Our proposal (reference number TLP 4607-0446), dated April 16, 2007, outlined the scope of our services. The scope of the investigation included a site reconnaissance, literature review, field exploration and laboratory testing program, and evaluation of the data collected during the field and laboratory portions of the investigation. In addition, the following were conducted:

2.2.1 A conceptual site grading plan of the proposed ten (10) lot subdivision, prepared by Whitson Engineers (undated),was reviewed.

2.2.2 A grading plan (Sheet C2.0) for the Horse Rehabilitation Facility, prepared by Whitson Engineers, dated June 14, 2007, was reviewed.

2.2.3 The Rancho El Potrero Vesting Tentative Map, PLN060603, prepared by Whitson Engineers, dated October 26, 2007, was reviewed.

2.2.4 A preliminary review of the foundation plans and details for the shedrow, single slope freespan building, and dual breezeway barn, prepared by ZJS Engineering Services, Inc, dated July, 2007, was conducted.

2.2.5 A map entitled "Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles, California" (Clark, Dupre, and Rosenberg, 1997), was reviewed.

2.2.6 A map entitled "Geologic Map of the Monterey Peninsula and Vicinity, Monterey, Salinas, Point Sur, and Jamesburg 15-Minute Quadrangles, Monterey County, California" (Dibblee, 1999), was reviewed.

2.2.7 The U.S.G.S. topographic map of the Monterey Quadrangle (7.5 minute series, 1:24,000) was reviewed.

2.2.8 A report entitled "Geological and Geotechnical Investigation for Vesting Tentative Map Submittal, Rancho San Carlos Project, Monterey County, California," prepared by Cleary Consultants, Inc., dated February 15, 1994, was reviewed.

2.2.9 A report entitled "Geological and Geotechnical Investigation Report, The Potrero Area Subdivision of the Santa Lucia Preserve, Monterey County, California," prepared by Cleary Consultants, Inc., dated August 22, 2000, was reviewed.

2.2.10 The Santa Lucia Preserve, Fault Lines," prepared by Hart/Howerton, dated August 10, 1999 (maps showing landslides and faults, Santa Lucia Preserve, after Cleary Consultants, Inc., February 15, 1994), was reviewed.

2.2.11 Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazard in California, prepared by American Society of Civil Engineers, Southern California Earthquake Center, dated June 2002, was reviewed.

2.2.12 Additional geologic maps and literature listed in the References section of this report were reviewed.

2.2.13 The Monterey County Grading Code, with updates through October 3, 2006, was reviewed.

2.2.14 The Monterey County Draft Environmental Impact Report (section 5.3 Geological Resources and Constraints), dated March 27, 2002, was reviewed.

2.2.15 A visual site reconnaissance and aerial photograph review were conducted.

2.2.16 Subsurface exploration was conducted consisting of excavation of test pits, and drilling test borings. The exploration was conducted to document soil and rock conditions and obtain soil and rock samples for testing.

2.2.17 Laboratory tests were conducted on soil/rock samples to determine geotechnical design index properties.

2.2.18 Mr. Jeff Taylor (Heritage Development), Ms. Asliegh Trujillo (Whitson Engineers), and Mr. Joel Panzer (Maureen Wruck Planning Consultants) were consulted during the investigation.

2.2.19 The data obtained from the investigation were evaluated to develop an understanding of the subsurface conditions and engineering properties of the subsurface soils.

2.2.20 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions regarding the geologic and geotechnical feasibility of the project, and recommendations for preliminary design and construction for project planning purposes.

3.0 BACKGROUND INFORMATION

A general site description, the anticipated construction, and the anticipated construction grading are summarized in the following subsections.

3.1 General Site Description: The proposed subdivision development is located within an approximate 103.2 acre area, consisting of three legal lots of record, west of Rancho San Carlos Road and south of the Carmel River and Quail Lodge Resort golf course, Carmel, Monterey County, California (see the Vicinity Map, Drawing No. 1, Appendix A). The majority of the proposed development is located on a heavily vegetated north facing hillside. The large lot (Lot 10) includes a heavily vegetated broad valley area.

The proposed lot sizes for Lots 1 through 9 range from about 1.2 to 4.2 acres. Lot 10 comprises the remaining 82.1 acre portion of the site and wraps around the west, south, and east sides of Lots 1 through 9. Slope gradients across Lots 1 through 9 range from as steep as about 3 horizontal (H) to 1 vertical (V) at the higher (south) lots to about 4H to 1V to 8H to 1V at the lower lots. Lot 10 encompasses 82.1 acres and extends from the golf course to as far as about ½ mile south. Lot 10 includes the lower alluvial plain areas of the site, a broad heavily vegetated valley area in the central portion of the site, and the relatively steep slopes in the higher elevation (southern) portions of the site. Slope gradients within Lot 10 range from nearly flat at the alluvial plain (northern) portions to as steep as about 1½H to 1V along the uppermost (southern) portions of the site.

An incised drainage ravine is located in the east portion of the development site, between lots 4 and 9 on the west and lots 10 and 3 on the east. Ground slope gradients near the ravine are as steep as nearly vertical. In some areas, the near vertical slopes bordering the ravine are 20 feet or more in height.

Site elevations range from about 34 to 50 feet above mean sea level (AMSL) on the north sides of the lower lots adjoining the golf course, to about 530 feet AMSL at the south (uphill) side of Lot 10. Lot numbers 1, 2, 3, and 4 are located on the north portion of the site between elevations of about 34 and 130 feet AMSL. Lot numbers 5, 8 and 9 are located mid-slope between elevations 90 and 200 feet AMSL. Lot numbers 6 and 7 are located in the south portion of the development site, roughly between elevations 180 and 330 feet AMSL.

The proposed building areas within each of the lots (delineated on the development site plan) range from about ¼ acre to 2 acres. The approximate locations of the proposed building areas are shown on Drawing No. 2, Appendix A. Slope gradients across the proposed home site areas (Lots 1 through 10) range from as steep as about 3 horizontal (H) to 1 vertical (V) at Lots 5, 6 and 7, to about 10H to 1V at the lower lots.

The proposed home site areas (about ¼ acre to 2 acres areas) had generally been cleared of heavy brush at the time of our field investigations in May 2007. Based on our observations and discussions with Mr. Taylor, it is our understanding that grading has not be conducted on the cleared home sites prior to our field observations. The ground surface at the cleared lots was generally covered with green native grasses, about 1 to 2 feet high. Native vegetation, including mature trees and generally thick brush with grasses was pervasive outside the cleared home site areas (see Photograph Nos. 16, 17, and 18, Appendix D).

At the time of the field investigation, an existing residence was noted in the north central portion of the site, downslope (north) of the Lot 10 home site (see Photograph No. 1, Appendix D). Existing out buildings were also noted in the central portion of Lot 10. It is our understanding that the existing residence and buildings will remain after development of the site. Other existing improvements noted on the site included a total of three (3) water wells located just east of Lot 8, just north of Lot 5, and in the north-central portion of Lot 10, respectively (see Drawing No. 2 for approximate well locations).

Based on our initial site review (April 12, 2007), it appears that underground irrigation/electrical utilities have been installed at some of the lots including in the vicinity of the wells. It is our understanding that the existing water wells will remain after development of the site.

Unimproved access roads transect the north portion of the site. Unimproved roads also transect the project site from the area of the southwestern most lots, to the two northernmost lots. The roads appear to have been constructed by cut and fill type grading.

3.2 **Anticipated Construction**: It is our understanding that the proposed residential structures will consist of one or two-story buildings with predominantly wood-frame construction and concrete slabs-on-grade or raised wood floor systems supported on shallow foundation systems.

It is also understood that the barn structures proposed for Lot 1 will be comprised of wood and steel construction with shallow spread foundations and slabs-on-grade.

Considering the existing ground slope gradients on some of the lots, basements or partial basements and building and /or landscape retaining walls and graded slopes would be anticipated to support grade changes.

Appurtenant construction, including asphaltic concrete paved roadways and driveways, and underground utilities, is anticipated.

At the time of preparation of this report, the details of the proposed construction type and magnitude of foundation loads for the proposed structures were not known. For the purpose of this feasibility evaluation, anticipated foundation loads for the typical residential structures were assumed to be 25 kips or less for interior column loads and 2.5 kips / foot or less for continuous footings. Design level geotechnical investigations should consider the actual design building loads.

It is our understanding that on-site sewage disposal systems will not be used for the project. On-site disposal systems, as well as unlined storage ponds, are not recommended to be used in the sloped portion of the site due to the potential for infiltration of water and slope instability as discussed in this report. Based on discussions with the property owner and our site observations, it is our understanding that no existing disposal systems or ponds are located on the sloped portions of the site.

3.3 **Anticipated Construction Grading**: Moore Twining was provided conceptual grading plans for the proposed access roads, driveways and for the equestrian center. Grading plans and pad elevations had not been determined for any of the residential lots at the time of this report. Based on the site topography, and assuming that the residences range in plan dimension from about 3,000 to 8,000 square feet, it is anticipated that cuts as deep as 15 feet below native ground surface and fills as thick as 15 feet above native ground surface would be required for lots on the steeper portion of the development to create relatively level pads for the residential structures, although smaller cuts and fills would be more prevalent. For the purpose of this report, each lot would be anticipated to have one building pad, with cuts as deep as 15 feet below native ground surface and fills as thick as 15 feet above native ground surface are assumed. If thicker cuts and fills are contemplated in the future, recommendations for these cuts and fills should be based on additional design level geotechnical investigations and global slope stability analyses.

3.4 **Previous Studies:** Moore Twining and others have conducted numerous previous geotechnical and geologic investigations for specific projects within the Santa Lucia Preserve development, an approximate 20,000 acre development located roughly one mile east of the site. It is our understanding that the subject site is a portion of the Santa Lucia Preserve. Previous studies of the Santa Lucia Preserve area reviewed by Moore Twining include a geological and geotechnical investigation for the Santa Lucia Preserve vesting tentative map (report dated February 15, 1994), and a geological and geotechnical investigation for the Potrero Area Subdivision of the Santa Lucia Preserve (report dated August 22, 2000), both prepared by Cleary Consultants, Inc. The Cleary investigations were not conducted for the subject site area; however, they included areas with landslides and geology similar to the subject site, as close as about 1 mile east of the proposed subdivision site. Moore Twining also reviewed maps entitled "The Santa Lucia Preserve, Fault Lines," prepared by Hart/Howerton, dated August 10, 1999 (compiled from data from the Cleary reports). We understand that previous site specific geotechnical and/or geologic investigations have not been performed for the subject development site.

4.0 INVESTIGATIVE PROCEDURES

The research, field exploration, and laboratory testing program conducted for this investigation are summarized in the following subsections.

4.1 **Literature Research:** Literature research included review of the conceptual plans provided, review of aerial photographs, and review of geologic, fault, and seismicity maps, and reports. The results of the literature research are presented under the Geologic Hazards and Geologic and Geotechnical Feasibility sections of this report.

4.2 **Field Exploration:** The field exploration consisted of a site reconnaissance, excavating and logging backhoe test pits, outcrop mapping, drilling test borings, and collecting and logging soil samples, and standard penetration tests.

4.2.1 **Site Reconnaissance:** The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Kenneth Clark, Certified Engineering Geologist (Moore Twining), on May 11, 12, 24, and 25, and November 27, 2007. The site features noted during the reconnaissance are described in the Background Information section of this report.

4.2.2 **Aerial Photograph Review:** Aerial photographs obtained from online sources and Natural Resources Conservation Services were reviewed. Land features, such as scarps, hummocky or lobate topography, indicative of recent landslide earth movement which could impact the home site areas, were not noted on the photographs within the home site areas, or upslope of the home sites. A description of the old landslide deposit that comprises a large portion of the project site area is provided under subsection 5.2 of this report. A relatively small shallow slide, delineated based on field investigation, is located on Lot 3 (see subsection 8.1.2).

4.2.3 **Outcrop Mapping:** Outcrop mapping was conducted to document accessible exposures of rock at the site by visual observation. Due to the relatively dense vegetation present over most of the site and the paucity of rock outcrops, relatively few outcrops were documented.

The rock outcrops documented are provided on Drawing No. 2. The results of the outcrop mapping were used with the test boring and test pit data to estimate the geologic profile for slope stability analyses (see subsection 8.1.3 of this report). Drawing No. 3, Appendix A, includes a copy of a regional geologic map indicating the mapped geologic units at and within the vicinity of the site. (Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles, California," Clark, Dupre, and Rosenberg, 1997). Geologic conditions of the project site are described under section 5.0 of this report.

4.2.4 <u>Test Borings and Test Pits</u>: The depths and locations of test borings and exploratory backhoe pits were selected based on the locations of the proposed lots, accessibility to the drill rig and backhoe, site geologic conditions, the anticipated foundation loads, and the anticipated depths of cut and fills.

On May 10 and 11, 2007, fifteen (15) backhoe test pits were excavated across the site to depths ranging from 6 to 12 feet below site grade (BSG). At least one test pit was excavated on each proposed lot and a potential future lot. The test pits were excavated under the direction of a Moore Twining project geologist using a 214 - CASE 580 backhoe equipped with a 24-inch wide bucket.

On May 24 and 25, 2007, hollow-stem auger test borings were drilled at five (5) locations accessible to the drill rig (see Drawing No. 2, Appendix A). The borings were drilled to depths ranging from 16 to 50 feet BSG. Sample refusal due to granitic rock was encountered at termination of all of the test borings. Sample refusal was defined as the depth of the deepest sample requiring more than 50 blows to drive the sampler 12 inches. Under the direction of a Moore Twining project geologist, the test borings were drilled using a CME-75 drill rig equipped with 6-5/8 inch outside diameter (O.D.) hollow stem augers. Photograph Nos. 2 and 3 (Appendix D) show the drill rig on the site.

During excavation of the backhoe pits and the test borings, bulk samples of soil were obtained for resistance value (R-value) testing, moisture-density relationship, expansion index, direct shear, remolded direct shear, Atterberg Limits, Loss-on-Ignition, and corrosion testing.

The soil and rock encountered in the test borings and test pits were visually logged in the field by the geologist. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil. The field rock classification was in general accordance with NAVFAC DM-7.1 and consisted of degree of weathering, discontinuity, color, grain size, hardness, geological classification, and other distinguishing features of the rock. The test pit logs and boring logs are included in Appendix B of this report.

The presence and elevation of free water, if any, in the borings and test pits were noted and recorded during drilling and immediately following completion of the borings and test pits.

Test boring and test pit locations were determined by measurement with reference to existing site features shown on the site plan prepared by Whitson Engineers. The approximate locations of the test borings and test pits are shown on Drawing No. 2, Appendix A. The locations should be considered accurate to within about 15 feet.

In accordance with our proposal, the test pits were loosely backfilled with the excavated materials. Therefore, some settlement should be anticipated at the test pit locations. The test pit locations should be surveyed and the test pit areas should be delineated on the final grading plans for bidding purposes. At the time of construction for the project, the loose soils at the test pit locations should be completely removed, moisture conditioned and compacted as engineered fill according to the recommendations provided in the Recommendations section of this report.

4.2.5 Soil Sampling: Standard penetration tests were conducted during the hollow-stem auger drilling program and both disturbed and relatively undisturbed soil samples were obtained. The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1-3/8 inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1 inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.3 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples representative of the subsurface material.

The results of laboratory tests are summarized on Figure Numbers 1 through 16 in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 GEOLOGIC CONDITIONS

The geologic conditions in the site area are described below.

5.1 General Geologic Setting: The site is located about 29 miles (47 km) southwest of the seismically active San Andreas Fault zone and 5 miles (8 km) east of the San Gregorio fault zone. These two fault zones mark the northeastern and southwestern boundaries, respectively, of the Salinian block with its crystalline basement of granitic and regionally metamorphosed rocks. A series of high-angle faults comprising the Monterey Bay - Tularcitos fault zone trend north-westward across the site region. This zone is a complex, generally northwest-striking fault zone up to 15 km wide, which includes several active (Holocene) and potentially active (Quaternary) faults.

The Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997), indicates three primary geologic materials in the area of the project site; Tertiary Monterey Formation - porcelanite; Older Holocene flood plain deposits, and Quaternary landslide deposits. Surface exposures of Miocene marine sandstone are mapped about 1 mile west of the site. Surface exposures of Cretaceous porphyritic granodiorite are mapped about 1¼ mile southwest of the site.

Old landslide deposits are pervasive in the Carmel/Monterey area and have been extensively documented in the Santa Lucia Preserve area, east of the site. Numerous developments have been constructed on old landslide materials that have been determined to be stable under current conditions. These developments include the Santa Lucia Preserve and Quail Meadows residential developments, and Carmel Valley Ranch Golf Club. The approximate locations of numerous residential structures constructed on landslide deposits within about 1 mile east of the site are shown on Drawing No. 4, Appendix A. Further discussion of the age and activity of landslides in the site region is provided in subsection 8.1.1 of this report.

Grading requirements for developments typically stipulate provisions to effectively remove surface water from slopes and reduce shallow groundwater seepage in order to reduce the potential for erosion, soil movements and landslides. Recommendations are provided in this report to reduce the potential for slope instability on the subject site (see Recommendations section of this report), including surface and subsurface drainage measures, slope grading limitations, etc. Old landslide deposits are defined in the Evaluations section of this report.

5.2 <u>Mapped Site Geologic Conditions:</u> Based on review of published geologic maps and our site investigations, it appears that the proposed home site areas and a large portion of the project site area is covered with Quaternary landslide deposits. Clark, Dupre, and Rosenberg (1997) describe the Quaternary landslide deposits as "Heterogenous mixture of deposits ranging from large block slides in indurated bedrock to debris flows in semi-consolidated sand and clay." Surface soils in the lower (northernmost) portions of the site comprise Older Holocene flood plain deposits.

The geological and geotechnical investigation reports (Cleary Consultants, dated February 15, 1994 and August 22, 2000), for the Santa Lucia Preserve area located east of the site (as close as about 1 mile east of the project site) describe three types of landslides present in that area as: active landslides, dormant landslides, and old landslides, differentiated based on geomorphologic conditions. This report utilizes a similar approach to characterizing and describing the recency of movement of landslide deposits at the site (see subsection 8.1.1 of this report).

The geologic map prepared by Clark, Dupre, and Rosenberg (1997) indicate Monterey Formation porcelanite mapped in the upslope (southern) portions of the project site (upslope of the Quaternary landslide deposits). Natural, near vertical cliffs of Monterey Formation porcelanite rock are also mapped near the east and west portions of the project site. The geologic map indicates that the Monterey Formation is located upslope of Lots 5 and 6, above an elevation of about 500 feet AMSL.

Based on outcrop mapping conducted for this investigation, the actual contact between the Monterey Formation and the landslide deposits on the development site appears to correspond roughly to the change in slope gradient to steeper slopes upslope of Lots 5 and 6. Moore Twining interprets this contact to be the eroded headscarp of the landslide mapped on the project site.

Clark, Dupre, and Rosenberg (1997) also indicate a terrace and scarp corresponding to the downslope (northern) edge of the mapped landslide deposit. Our site reconnaissance indicates the erosional scarp is about several feet high at the north end of Lots 3 and 4. The erosional scarp cut into the mapped landslide soils appears to have been created by the historic meandering of the Carmel River.

Detailed descriptions of the site soil and rock conditions revealed by our investigation are presented under subsections 6.1 and 6.2 of this report.

6.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

6.1 Site Soil/Rock Conditions: The following soil and rock descriptions are based on the outcrop mapping, test borings drilled and backhoe test pits excavated for this investigation. Detailed descriptions of the soils encountered at each test boring and test pit location are presented on the logs of borings and test pits in Appendix B. The stratification lines shown on the boring logs represent the approximate boundaries between soil/rock types; the actual in-situ transition may be gradual.

The results of site observations and subsurface exploration (test pits and borings) suggest that the typical near surface native soil profile at the site consists of an upper organic rich brown to dark grey, silt, "A horizon" soil (topsoil) extending from the ground surface to depths of about 1½ to 5 feet BSG. Scattered, gravel-size and smaller fragments of rock (siltstone, etc.) were noted in these soils. The organic rich topsoils were not encountered in the test borings drilled in roadway cut areas where the upper soils had been previously removed. The topsoils were generally underlain by an "E horizon" soil comprising a greyish tan silt matrix with significant humus and abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, and a porous and loose appearance. The texture of these soils suggests that the material was deposited as part of an old landslide - earthflow type movement. The E horizon soils, where encountered, were typically about 1 foot thick.

The relatively loose A and E horizon soils were typically underlain by silt soils with a similar composition (i.e. silt matrix with abundant siltstone and porcelanite fragments), also interpreted to be of earthflow origin. Earthflow soils consisting of a lean clay matrix with abundant siltstone and porcelanite fragments were typically noted in test pits underlying the more silty earthflow type soils. Photograph 4 (Appendix D) illustrates a typical A horizon soils underlain by E horizon soils, underlain by a lean clay with siltstone and porcelanite fragments. Slickensides indicative of downslope creep were noted in the lean clays at some of the test pit locations (see Photograph No.

5, Appendix D). Slope creep is discussed in subsection 8.2 of this report. A thin clay shear surface was noted in test pit TP-13, Lot 9 (see Photographs Nos. 6, 7, and 8, Appendix D).

A review of geologic maps, test boring logs and test pit logs suggest that the uppermost silty and lean clay soils across the site area investigated are predominantly earthflow type landslide materials. These materials correlate with the Quaternary landslide deposits noted on the Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997). The silty and lean clay soils noted below the loose near surface soils appeared stiff to hard in the test pit exposures, and were very stiff to hard as indicated by standard penetration resistance, N values, ranging from 31 to greater than 50 blows per foot (test borings).

An outcrop of Monterey Formation porcelanite and siltstone was noted on the slope above the proposed Lot 6 home site. Outcrops of Monterey Formation were also observed along a near vertical cut slope for an unimproved (old) road located in the southern portion of the property and along the south side of the site access road, east of the area of investigation. Bedding plane orientations typically were noted to strike approximately northwest-southeast and dip about 10 to 15 degrees to the east and the rock was intensely fractured with the fractures oriented near vertical. The Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles indicates numerous bedding plane strike and dip orientations for the Monterey Formation within about 2,500 feet west, south, and east of the project site. The majority of the bedding plane measurements indicate a northwest-southeast strike, with dips ranging from 3 to 15 degrees northeast.

The slopes underlain by Monterey Formation upslope of the project site are characteristically heavily wooded and steep, with gradients of 2H to 1V, or steeper, and some near vertical exposures of Monterey Formation rock. Natural, near vertical cliffs of Monterey Formation rock 20 to 40 feet high (or more) are also mapped, roughly along geologic strike, near the east and west portions of the project site.

Fractured siltstone was encountered below the earthflow soils in some of the pits and test borings (see Photograph Nos. 9 and 10, Appendix D). The siltstone exposed in test pits appeared to be dense. Based on the high degree of fracturing and shearing in the siltstone material (encountered below the silt and lean clay landslide deposits), it is likely that this siltstone is either a portion of the lower part of the Monterey Formation, or the upper portion of a geologic unit described locally as "unnamed sandstone" (see the Regional Geologic Map, Drawing No. 3, Appendix A). Clark, Dupre, and Rosenberg (1997) describe the unnamed sandstone as a coarse to fine-grained, poorly to well sorted arkosic sandstone....with rare siltstone beds in the upper part. The "unnamed sandstone" unit is mapped by Clark, Dupre, and Rosenberg (1997) about 1 mile west of the site.

Sandstone was encountered in test borings B-1 and B-2 between the upper lean clay landslide material and weathered granodiorite encountered at depths of about 7 and 13 feet BSG. The sandstone was medium dense to dense as indicated by standard penetration resistance, N values, ranging from 28 to 55 blows per foot. It is likely that this siltstone is either a portion of the lower part of the Monterey Formation, or the upper portion of the "unnamed sandstone" referenced above.

Granitic rock was encountered in all five (5) of the test borings and test pit TP-12, Lot 1 (see Photograph No. 11, Appendix D). The degree of weathering of the granitic rock was highly variable from one boring location to another, and varied with depth. Standard penetration blow counts measured in the granitic rock were greater than 50 blows per foot at all sample locations (very dense if considered as a soil), except at a depth of 45 feet in test boring B-3, where a blow count of 25 was measured and moist rock material was noted. The granitic rock encountered in all five (5) of the test borings is likely Mesozoic granodiorite basement rock material of Salinian Block. Outcrops of the granodiorite basement rock are mapped less than 1 mile west of the site (Clark, Dupre, and Rosenberg, 1997).

6.2 **Site Soil and Rock Engineering Properties:** The physical characteristics of the soil and rock types are summarized below. Unit designations correlate with those designations on the test pit logs (Appendix A). Pavement support characteristics (R-value test results), and soil corrosivity characteristics are also discussed.

Organic Rich Top Soil - Unit A: This A horizon soil is a dark brown to black, organic rich, sandy silt topsoil with abundant fine grass rootlets and occasional roots about ¼ inch in diameter. Photograph No. 11, Appendix D illustrates a typical A horizon top soil. Scattered rounded granitic cobbles, pebbles and boulders to 1 foot in diameter, and angular siltstone fragments were noted at some locations. Undulatory bottom surface typically grades down to a porous silt. An O horizon organic mat typically was noted overlying the topsoil, ranging in thickness from less than 1 inch to about 4 inches.

Organic rich, sandy silt top soils typically extended from the ground surface to depths of 24 to 36 inches BSG (deeper organic rich soils were noted in TP-10 located on a potential future lot). These soils appeared loose/soft, and the results of a Loss-on-Ignition test conducted on a sample of the material indicated 4.8 percent organic content. On a preliminary basis, due to the loose/soft condition and the potential for decomposition of organic matter and settlement, the top soils (typically encountered to depths of about 3 feet BSG and deeper in some areas) are not considered suitable to support fills or structures and should be removed from building areas. Additional organic content testing should be conducted during design level geotechnical investigations. However, on a preliminary basis, the organic rich soils should not be used as engineered fill within pavement, building, overbuild zones, unless blended with deeper soil containing a lower organic content under a controlled method. Blending of soils is typically done during site grading. Details regrading blending are provided in the Evaluations section of this report (subsection 8.7).

Porous Silt - Unit 1A: The undulatory bottom surface of Unit A typically grades down to a tan to light grey silt (E horizon soil). This unit was identified at most of the test pit locations and appears porous and soft to medium stiff, with abundant angular siltstone and porcelanite fragments, and contains some rootlets.

Silt - Unit 2: This unit was identified at most of the test pit locations. The material is yellowish brown and greyish tan, appears stiff to hard, and contains abundant angular siltstone and porcelanite fragments (typically less than 1 inch diameter) with trace of clay in some areas. Photograph No.12, Appendix D illustrates a typical Unit 2 silt soil. Occasional lenses of silty sand with scattered rounded granitic pebble and cobbles were also present.

Remolded direct shear testing was conducted to simulate an engineered fill condition for the slope stability analyses (see subsection 8.1.3). Bulk samples of the silty earthflow material with angular fragments of porcelanite (collected from test pit TP-6, Lot 6 and TP-12, Lot 1) were remolded into ring samples at about 90% of the maximum dry density. Prior to remolding, the material was sieved over a #8 screen to remove the larger angular fragments of porcelanite and siltstone. Particles larger than #8 (about 1 millimeter) were removed to provide a conservative estimate of shear strength and roughly approximate potential fragment size reduction during the compaction process.

The remolded samples were subjected to direct shear testing (undrained/consolidated). The test results indicated a peak angle of internal friction of 24 degrees with a cohesion value of 680 pounds per square foot.

An expansion index (E.I.) test conducted on the bulk sample of silt indicated a very low potential for expansion (E.I.=0).

Lean Clay - Unit 3: This unit was identified at some of the test pit locations, although not as frequently noted as Unit 2. The material is dark brown or grey, and contains abundant angular siltstone and porcelanite fragments (typically less than 1 inch diameter).

The lean clay earthflow material appears stiff to hard in the test pit exposures and was very stiff to hard as determined by standard penetration resistance, N-values, ranging from 31 to greater than 50 blows per foot. The measured moisture contents of the lean clay soils ranged from about 28 to 33 percent. In-place density tests performed on the lean clay soils indicated dry densities of about 77 and 107 pounds per cubic foot.

A relatively undisturbed ring sample of the clayey earthflow material with angular fragments of porcelanite (collected from test boring B-1) was subjected to direct shear testing (undrained/consolidated). The test result indicated a peak angle of internal friction of 26 degrees with a cohesion value of 790 pounds per square foot.

A direct shear test was also conducted on a relatively undisturbed sample of stiff to hard lean clay earthflow material collected from test boring B-2 (near the Lot 2 home site). The sample had scattered angular fragments of sandstone and porcelanite slightly less than ¼ inch in diameter. Residual shear strength testing was conducted by shearing the same sample multiple times at a rate of 0.09 mm/minute to approximate residual shear strength under drained conditions. Normal (overburden) pressures of 1,000, 2,000, and 3,000 pounds per square foot were applied to each of three ring samples (from the same 6 inch long sleeve) during the multiple cycles of shearing. The results indicated that the clay material had a residual angle of internal friction of 28 degrees with a residual cohesion of 220 pounds per square foot.

An expansion index (E.I.) test conducted on a sample of native lean clay with silt indicated a very low potential for expansion (E.I.=12).

<u>Siltstone - Unit 4</u>: The siltstone is pale olive and yellowish brown (mottled), weak to moderately strong, moderately weathered rock, and intensely fractured and sheared. Siltstone with narrow to moderately wide fracture and shear surfaces was encountered in the bottom of test pits TP-1 (Lot 5) and TP-7 (Lot 6). Siltstone with moderately wide clay filled shears was encountered below a depth of 30 inches BSG in TP-11 (Lot 7), extending to the maximum depth explored of 6 feet BSG.

<u>Sandstone:</u> A yellowish brown, dense sandstone was encountered below the lean clay earthflow materials and above a highly weathered granitic rock in test borings B-1 and B-2 (near Lots 2, 3, 4, and 5). The results of Atterberg Limits tests conducted on samples collected from about 5 and 8 feet BSG in B-2 indicated liquid limits of 21 and 29 percent, and plasticity indices of 2 and 11 percent, respectively (slightly plastic to low plasticity).

<u>Granitic Rock - Unit 5</u>: The granitic rock was field classified as granodiorite and was encountered at depths ranging from 5 to 13 feet BSG, extending to the bottom of test pits TP-1, TP-2, TP-3, and TP-5 (16 to 50 feet BSG). The granitic rock was also encountered in test pit TP-4 at a depth of 36 feet BSG, and extended to the bottom of that test pit (42 feet BSG).

The granitic rock, encountered in all of the test borings at depths ranging from 5 to 36 feet BSG, was visually classified as a granodiorite. The samples collected were typically highly weathered and friable. The results of an in-place density test performed on a highly weathered sample of the granitic rock indicated a dry density of 119 pounds per cubic foot.

<u>Pavement Support and R-value Testing:</u> R-value tests (CTM 301) were performed to provide a preliminary indication of the pavement support characteristics of the site soils. The test were conducted on two (2) bulk samples of the lean clay soils collected from test borings drilled on exiting unimproved site access roads, at depths between the ground surface and 3 feet BSG. The results of the testing indicated poor to fair pavement support as indicated by R values of 9 and 35.

<u>Corrosivity Testing</u>: Chemical tests performed to assess the corrosivity of two (2) bulk soil samples indicated pH values of 6.8 and 6.1, and minimum resistivity values of 690 and 370 ohms-centimeter, respectively. Test results also indicated 0.073 and 0.54 percent by weight concentrations of sulfate, and 0.064 and 0.018 percent by weight concentrations of chloride.

6.3 <u>Site Groundwater Conditions</u>: Groundwater was encountered in test boring B-3 within a weathered zone of granodiorite (likely fractured) at about 45 feet BSG (about 35 feet below the top of the granodiorite). Groundwater was not encountered in the other four (4) test borings to the maximum depth explored of about 42 feet BSG. In addition, groundwater was not encountered in any of the fifteen (15) test pits excavated on May 24 and 25, 2007, to maximum depths of about 6 to 12 feet BSG. However, test pit TP-7 (Lot 6) encountered moist soils which appeared to represent a potential for perched groundwater at about 5 feet BSG. This test pit was located in a subtle swale which likely concentrated surface water. The moist zone appeared to be located above a lean clay soil layer.

Three (3) water wells are known to be located just east of Lot 8, just north of Lot 5, and in the north-central portion of Lot 10 (see Drawing No. 2 for approximate well locations). The depths of these wells and pumping levels are not known.

It should be recognized that water table elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. In addition, subsurface seepage in rock is largely controlled by fractures and weathering. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

7.0 GEOLOGIC HAZARDS

The potential for geologic hazards to affect the project site including flooding due to dam breach, seiches, tsunamis, volcanic activity, asbestos containing soil/rock materials, fault rupture, and seismicity and ground shaking are discussed in this section. Landsliding, liquefaction and seismic settlement are included in the Evaluations section of this report.

7.1 **Dam Breach:** The site is not located in an area that would be subjected to flooding due to a seismically induced dam breach.

7.2 **Tsunamis and Seiches:** Tsunamis and seiches are waves generated in oceans and lakes from seismic activity. Due to the location (approximately 1¼ miles from the Pacific Ocean) and elevation of the site (greater than 30 feet) above sea level, and the absence of nearby lakes, tsunamis and seiches are not considered potential hazards.

7.3 **Volcanic Activity:** According to the Fault Activity Map of California and Adjacent Areas, with Locations and Ages of Recent Volcanic Eruptions (Jennings, 1994), the nearest recent volcanic sources are located in the Sierra Nevada Range, over 140 miles east of the site. The potential for volcanic activity to affect the project site is low.

7.4 **Naturally Occurring Asbestos (NOA):** Asbestos occurs in soil and rock naturally in certain geologic settings in California. It has been documented by others that inhalation of asbestos fibers may cause cancer and other negative health effects. Most commonly, asbestos is associated with serpentinite and partially serpentized ultramafic rocks. Ultra mafic rocks are scattered throughout much of the Sierra Nevada mountain and Coast Ranges regions. Review of the referenced Open-File Report 2000-19 entitled *A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos,* prepared by State of California Department of Conservation, Division of Mines and Geology, dated August, 2000, indicates that ultramafic rock are not pervasive in the Monterey Bay/Carmel region and does not indicate ultramafic rocks at or near the area of the site.

Ultramafic rocks were not observed on the site, in test pits, or in samples collected from the test borings and are not common to the geologic environment of the site. The Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997), does not indicate ultramafic rock exposures on or near the subject site.

Based on the cited literature and our site observations, it is our opinion that the potential to encounter naturally occurring asbestos containing rock in the near surface soil and rock materials at the site is low.

7.5 **Fault Rupture:** Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it is manifested by sudden and abrupt relative ground displacement. Damage resulting from fault rupture occurs only where structures are located astride the fault traces that move.

The Alquist-Priolo Earthquake Fault Zoning Act was enacted to prevent the construction of buildings used for human occupancy astride the surface trace of active faults. The Alquist-Priolo Act requires that before a project is permitted, cities and counties must require specific geologic investigations for projects within mapped Alquist-Priolo Earthquake Fault Zones to demonstrate that proposed buildings will not be constructed across active faults. The project site is not located within an Alquist-Priolo Earthquake Fault Zone.

An "active fault" is defined, for the purpose of this evaluation, as a fault that has had displacement within Holocene time (about the last 11,000 years).

A widely accepted definition of potentially active is a fault showing evidence of displacement older than 11,000 years and younger than 1.6 million years (Pleistocene). Faults showing evidence of displacement older than 1.6 million years are usually classified as "inactive."

In the project site region, several active (Holocene) and potentially active (Quaternary) faults comprise the Monterey Bay - Tularcitos fault zone, a complex, generally northwest-striking fault zone up to 15 kilometers wide. The fault zone includes the Hatton Canyon and Sylvan Thrust. The site is located about 1½ miles (2½ km) southwest of the Hatton Canyon fault, the nearest fault to the site.

According to the Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997), and the Monterey Fault Map, prepared by Staal, Gardner & Dunne, Inc. 1994, no faults cross the project site. The Cleary Geological and Geotechnical investigation report does not identify any active faults within the Santa Lucia Preserve vesting tentative map (located about 2 miles east of the subject site).

Based on our site review and review of the above referenced literature, the potential for surface fault rupture at the site is considered low.

7.6 **Regional Seismicity:** The general area of the site has experienced recurring seismic activity. Based on historical earthquake catalogs published by the California Division of Mines and Geology, supplemental data from Townley and Allen (1939), and the U.S. Geological Survey's earthquake data base system (data base updated through June 2005), approximately 736 historical earthquakes with magnitude 4.0 or greater were recorded from 1800 through 2005 within a 100-mile search radius of the site.

The peak horizontal ground acceleration predicted to have occurred at the subject site from each of the historical earthquakes within the 100-mile search radius was estimated using a seismic ground motion attenuation relationship developed by Boore et al. (1997). The source data presented includes: latitude, longitude, date, time, depth, Richter Magnitude, estimated horizontal ground acceleration, computed site Modified Mercalli intensity, and the approximate earthquake-to-site distance in miles and kilometers. This analysis was performed by a computer program titled EQSEARCH 3.00b (Blake, 2000, with updates through June 2005).

The seismic event with the nearest epicenter found during the search occurred in 1918 approximately 3½ miles northwest of the project site (Magnitude 4.3; peak horizontal ground acceleration at the site = 0.138g). The earthquake event that produced the highest estimated horizontal ground acceleration since 1800 occurred in 1800, about 27 miles southeast of the site (on the San Andreas Fault), and produced an estimated peak horizontal ground acceleration of about 0.15g at the subject site. The 1800 event was also the largest magnitude event to have occurred within the search radius since 1800, magnitude 7.0.

7.7 **Seismic Ground Motion and Seismic Design Factors and Coefficients:** An evaluation of potential seismic ground shaking was conducted using a probabilistic analyses. The Design Basis earthquake ground motion is the magnitude of earthquake ground shaking that should be considered for geotechnical/geologic design, structural design, liquefaction and seismic settlement analysis, and slope stability analyses. The Design Basis Ground Motion as the seismic ground motion having a 10 percent probability of being exceeded in a 50-year period.

Probabilistic ground motion evaluation requires use of a seismicity model and ground motion attenuation functions to approximate the modification of seismic waves between the earthquake hypocenter (source) and the site. The seismicity model, including the location and fault parameters (such as slip rate, fault length, magnitude and rupture area) of faults capable of impacting the site (active and potentially active faults), was based on published geologic papers and corresponds with those listed in the California Geological Survey (CGS) database entitled "California Fault Parameters" (Cao et. al., 2003). Probabilistic evaluations were conducted using the FRISKSP computer program (version 4.0, Blake, 2002, updated based on Cao et. al., 2003) and the faults indicated as those active and potentially active faults listed in the "California Fault Parameters" database. The computation of attenuated ground motion is based on the closest distance between the site and various measures of potential fault-plane ruptures along selected faults.

The analysis was conducted using a "very dense soils and soft rock" class C (average N-value of greater than 50 blows per foot in the upper 100 feet BSG). Our evaluation considered the average of the design basis ground motions calculated using ground motion attenuation functions based on Boore et. al. (1997), Sadigh et. al. (1997), and Idriss (1994). The analyses incorporated the twenty-seven (27) active and potentially active faults within 100 kilometers of the site. The average of the design basis ground surface accelerations calculated based on the above attenuation relationships was determined to be 0.34g.

Hazard deaggregation was conducted using the FRISKSP computer program. The results indicate that an earthquake magnitude of 6.8 represents the predominant earthquake magnitude for the site. The predominant earthquake distance is estimated to be about 7 kilometers. The earthquake magnitude and the above ground motion estimate, was considered in assessment of potential seismic settlement (subsection 8.5 of this report).

It is expected that the 2007 CBC will be used for structural design, and that seismic site coefficients are needed for design. The 2007 CBC site coefficients are included in the Recommendations section of this report. Site coefficients for 2001 CBC are also provided for reference. Based on the CBC, the site classification is estimated to be a very dense soil and soft rock ©) site with standard penetration resistance N-values averaging greater than 50 blows per foot in the upper 100 feet below site grade.

The site coefficients for acceleration and velocity are based on the distance and activity of the local active faults (faults showing Holocene age displacement in the past 11,000 years). Digitized seismic models published by the CGS indicate that the Monterey Bay-Tularcitos and San Gregorio fault are located about 3½ miles (5½ kilometers) northeast of the site, and 4¾ miles (7½ kilometers) southwest of the site, respectively. The Monterey Bay-Tularcitos fault zone is characterized as a Type B fault based on a maximum earthquake magnitude of 7.3 with a slip rate 0.5 millimeters/year. The San Gregorio fault zone is characterized as a Type A fault based on a maximum earthquake magnitude of 7.44 with a slip rate 6 millimeters/year. No other known active faults are located within 15 kilometers of the site.

Tables providing the recommended seismic coefficients and near source factors for the project site are included under the Foundation Systems subsection of this report (under subsection 10.9).

8.0 EVALUATION OF GEOLOGIC AND GEOTECHNICAL FEASIBILITY

The data and methodology used to develop conclusions and general recommendations concerning the geotechnical and geologic constraints of the project are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the investigation and our understanding of the proposed construction.

The primary geologic and geotechnical concerns for design and construction of the proposed project are: 1) the potential for landsliding and stability of slopes; 2) potential for slope creep; 3) design and construction of manufactured slopes; 4) building to slope setbacks; 5) surface soils with excessive organics for use as engineered fill; 6) relatively high organic content in soils; 7) the potential for differential static settlement of shallow foundations due to cut to fill transitions and differential fill thickness across the building pads; and 8) shallow groundwater and subsurface seepage.

8.1 Evaluation of the Potential for Future Landslides to Impact the Site: For a landslide to occur, zones of weakness must be developed (shear zones) in the soil or rock. Groundwater and the soil/rock weathering processes often cause zones of weakness in soils and rock with relatively low shear strength. Creep or abrupt failure (landsliding) may occur on a slope along zones of weakness. Inertial triggering mechanisms typically include seismic ground motion, an increase of soil unit weight as a result of rainfall, surcharging slopes by placement of fill, or by removal of support (i.e., by cut type grading activities, erosion, etc.) at the toe of slopes. In addition, surface water infiltration or shallow groundwater can reduce the shear strength of the in-place soils / rock and initiate slope movements. The process of earth movement during a landslide typically causes shear zones to develop. Shear zones can be identified in samples or test pits by the presence of shiny, striated surfaces termed slickensides, clay shears zones and broken clastic fragments. The original soil/rock material shear strength parameters (cohesion and angle of internal friction) are typically reduced by movement along the shear zones.

The scope of our evaluation of the potential for future landslides to occur at the subject site is summarized below in four sections: 1) describing known landslides in the site region including age of landslide movement; 2) describing existing landslides and landslide materials on the site; 3) conducting preliminary slope stability evaluations of current and estimated future slope configurations, and 4) discussion of the potential for future landslides to occur on the site.

8.1.1 Landsliding in the Site Region: According to the Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997) and the Geologic Map of the Monterey Peninsula and Vicinity, Monterey, Salinas, Point Sur, and Jamesburg 15-Minute Quadrangles, Monterey County, California" (Dibblee, 1999), the proposed home site areas and most of the project site area are underlain by Quaternary landslide materials. The maps also indicate numerous Quaternary landslides overlying Monterey Formation rock in the Carmel and Monterey areas including a landslide covering about 575 acres located just east of the subject site (in the area of the existing Quail Meadows residential development), and the Potrero Area Subdivision portion of the Santa Lucia Preserve, about 1 to 3 miles east of the site (see Drawing No. 4, Appendix A).

Clark, Dupre, and Rosenberg (1997) indicate:

> *Landslides form in all the geologic units, but are most common in the Monterey Formation. Younger landslides have fresh scarps, disrupted drainages, closed depressions, and disturbed vegetation. Older landslides are modified by erosion, resulting in subdued scarps, reestablished vegetation, and new drainage paths. Soils have formed on some of the older landslide deposits, however, most soils are poorly developed or absent because of high erosion rates and steep slopes.*
>
> *Many of the younger slides are probably early Holocene as indicated by poorly to moderately defined scarps, hummocky topography, and well developed drainages.*

Clark, Dupre, and Rosenberg (1997) cite several radiocarbon age dates for the slides in the region ranging from 9,600 +/- 160 years before present to 31,000 years before present.

Cleary Consultants conducted comprehensive geological and geotechnical investigations of the Santa Lucia Preserve area, located as close as about 1 mile east of the subject proposed development area.

The Cleary Consultants report (1994) indicates:

> *"The hillsides of Rancho San Carlos (more recently referred to as "Santa Lucia Preserve"), like most hillside areas in coastal California, are subject to landslide activity - past and present. Several massive old landslides are located along east-facing slopes near the northern end of Potrero Canyon, and east of Robinson Canyon on generally west facing slopes. These landslides are generally believed to be old features formed in late Pleistocene to early Holocene time (10,000 to 20,000 years before present).....Studies north of the subject property, of an apparently similarly formed old landslide, indicate late Pleistocene to early Holocene age of landsliding (Johnson, 1986) during a period when the climate is believed to have been much wetter. Old landslides are generally suitable for development based upon the confirmation of geological and geotechnical studies."*

For the purpose of this investigation, three categories (active, dormant, and old) are used to define the recency of landslide activity (after Cleary Consultants, 1994 and 2000). Active and dormant landslides typically present a generally higher risk of future movement than old landslides.

> **Active Landslides:** Active landslides are those which recently moved or are creeping downslope. These landslides are characterized by well defined geomorphic features such as steeply inclined angular head and flank scarp regions. Poorly established internal drainage, with areas of ponded water, are common, and adjacent drainage paths are disrupted. Vegetation is disturbed and includes downed trees and breaks in the continuity of grasses and brush growing in within the landslide. A hummocky ground surface topography is commonly seen in the toe (bottom) region of a slide.
>
> Moore Twining has mapped several relatively small active landslides in the Santa Lucia Preserve area as part of other investigations ranging from several hundred to several thousand square feet in areal dimension.

Dormant Landslides: Dormant landslides are not currently moving, but are potentially unstable and could re-activate. Dormant landslides are characterized by moderately well defined geomorphic features such as moderately steep to steeply inclined, subangular head and flank scarp regions. Internal drainage patterns are being established, with diminishing areas of ponded water. Vegetation is being established on the scarps and interior portions of the landslide.

Old Landslides: Old landslides are not currently moving and may be described as moderately stable to stable. These landslides are characterized by weathered and eroded geomorphic features. Scarp areas are rounded, drainage is well established and entrenched, and vegetation, including large trees, commonly cover the landslide masses. Old landslides mapped in the Monterey/Carmel area are commonly very large covering several hundred acres or more. The results of slope stability analyses conducted for five old large landslides identified on the Santa Lucia Preserve site, and presented in the Cleary Consultants report, indicate that: "Old landslides are generally suitable for development based upon the confirmation of geological and geotechnical studies."

Reactivation of Slides: Active, dormant or old landslides can reactivate by processes such as those described in subsection 8.1. Grading, surface and subsurface drainage designs and construction can be used to reduce the potential for reactivation of slides. The slope stability assessment presented in this report evaluates the potential for slide reactivation by incorporating soil strength parameters representative of pre-existing slide planes (weak layers).

The results of landslide stability analyses conducted by Cleary Consultants (1994) on five (5) old landslide masses located on the Santa Lucia Preserve (east of the subject site) indicated that the gross stability of the old landslides was satisfactory. Cleary Consultants (2000) also investigated five old landslides in the Potrero Area Subdivision of the Santa Lucia Preserve, all located within about 1 to 3 miles east of the subject development site. The results of stability analyses conducted for the old landslides, and presented in the Cleary Consultant report (2000) indicated that;

"Based on the findings of our slope stability analyses as presented in Table 2, it is our opinion that the old landslides are unlikely to reactivate under existing and post-development conditions as long as care is taken to avoid adversely impacting them during the planned development work.....Factors which could lower the slope stability of the landslide deposits include removal of landslide toe support by grading or rapid erosion, or major changes in the groundwater regime which would permanently raise the water table."

8.1.2 Existing Landslides and Landslide Materials on the Site: This subsection presents discussion of the landslide materials on the proposed development site, and our assessment of the recency of landslide movement.

Earthflows: Soils indicative of earthflow type landsliding were encountered in all of the test pits and soil borings. These soils exhibited a chaotic texture comprising a silt or lean clay matrix with abundant angular siltstone, claystone, and porcelanite fragments. It is our interpretation that the chaotic soil texture formed as a result of earthflow movement and deposition. Soils of similar description are termed "Landslide Debris" on test boring logs presented in the Cleary Consultants report for the Potrero Area Subdivision," (August 22, 2000). Earthflows are downslope, viscous flows of saturated, fine-grained materials. The velocity of the earthflow is dependent on the water content of the flow materials and the steepness of the ground where the earthflow occurs. Higher water contents and steeper slopes produces higher velocities. Clay, sand and silt soils are all susceptible to earthflows. Earthflows occur typically during or after periods of high precipitation, which saturates the ground. Earthflows initiate when pore pressures in the soils increase, thus decreasing the internal shear strength of the material.

The Geologic Map of the Monterey and Seaside 7.5-Minute Quadrangles (Clark, Dupre, and Rosenberg, 1997), indicates the majority of the site area is covered by Quaternary landslide deposits (Drawing No. 3, Appendix A). Visual observations were made near the upslope extent of the mapped landslide material, in a cleared area near the upslope (south) boundary of the proposed Lot 6 home site and upslope of the Lot 6 homes site, to the to the property boundary. The ground slope gradient increases from about 4 H to 1V to about 2 to 2½ H to 1V upslope of Lot 6 near the upslope (south) boundary of the proposed Lot 6 home site. The geologic map prepared by Clark, Dupre, and Rosenberg (1997) indicates that the Monterey Formation is located upslope of Lots 5 and 6. Moore Twining interprets this contact to be the eroded headscarp of the landslide mapped on the project site. Observations in this area did not reveal fissures, disturbed vegetation, or irregular surface drainage, which would be anticipated near the headscarp of an active landslide.

The downslope extent of the mapped landslide material is located near the break in slope gradient at the base of the hillside, at the contact with Quaternary alluvial soils. Clark, Dupre, and Rosenberg (1997) also indicate a terrace corresponding to the downslope (northern) edge of the mapped landslide deposit. Our site reconnaissance indicates this feature is an erosional terrace noted to be several feet high at the north end of Lots 3 and 4. The terrace cut into the mapped landslide soils appears to have been created by the historic meandering of the Carmel River.

Observations of the ground surface and from test pit excavations, predominantly in the cleared building areas located on the central and downslope portions of the mapped landslide material, did not reveal geomorphic landslide features such as cracks, scarps and ridges. The lack of notable surface features indicative of slope movement suggests that these features associated with the mapped deposit were likely eroded and obliterated over time. It is also conceivable that minor features could have been disturbed by site grubbing and clearing operations.

The large ravine located between lots 6 and 7 and between lots 8 and 9 is deeply incised into the landslide material and well vegetated, with brush and large trees.

Based on the lack of notable landslide geomorphic features, lack of disrupted vegetation, and the deeply incised ravine cutting through the landslide body, the earthflow landslide material noted on the site is believed to represent old landsliding, likely associated with the wetter climate of the late Pleistocene to early Holocene.

<u>Shallow Rotational/Translational Landslides on the Site</u>: The slope surface conditions in the home site areas where vegetation had been cleared were observed by Kenneth Clark, engineering geologist, in May 2007. The general surface conditions were fairly observable due to the home sites having been cleared or partially cleared of brush. However, native grasses on the order of one foot high were present on the home sites at Lots 1, 2, 3, 4, 8, 9, and 10, and one to two feet high on Lots 5, 6, and 7. Accordingly some small scale, shallow landslide features may have been obscured from view. Much of the site, outside the proposed home site areas, was covered with dense brush and trees and could not be observed during the site reconnaissance. Based on our observations, it is estimated that most recent landslide surface features (slides affecting areas on the order of 200 square feet in plan area, or larger), if present on the cleared home site areas, could have been identified by our site observations.

One relatively small landslide was identified at a proposed home site during the aforementioned observations (the approximate location of the slide on Lot 3 is shown on Drawing No. 2, Appendix A). This landslide feature exhibited a slight increase in downslope gradient at the scarp of the slide and a slight bulge at the toe of the slide. The top of the scarp area was rounded and not angular. The rounded appearance of the headscarp suggests that erosion has occurred since the last episode of landslide movement. The eroded appearance of the headscarp and established vegetation on the slide area suggests that the last significant episode of movement did not likely occur within roughly the last 10 or more years. Accordingly, this slide is classified as a dormant landslide. Photograph No. 13 (Appendix D) illustrates the geomorphologic conditions at the slide site located on Lot 3. Exploratory trench TP-5 was excavated into the main body of this landslide, near the head scarp. Photograph Nos. 14 and 15 illustrate clay shears exposed in the sidewalls of the trench. The clay shear surfaces represent zones along which sliding has occurred in the past. Based on the orientation of the clay shear zones, both translational and rotational slide movements are suggested. It is estimated that the slide body is about 4 feet deep at the location of test pit TP-5.

Shallow rotational/translational slides are triggered by saturation of upper poorly drained clayey soils, which causes the soil unit weight to increase and the shear strength to decrease. Past observation of numerous similar slides in the Santa Lucia Preserve area indicate that these types of slides occur most prevalently in the spring after long periods of rain. These types of landslides typically do not occur rapidly and slide movement can range from hours to months, or years.

The presence of the relatively shallow dormant landslide feature noted on Lot 3 underscores the potential that other such features may be present on the development site, or may develop in the future. Based on the scope of this feasibility level investigation, and the existing ground cover over much of the site, additional relatively shallow dormant or active slides may be present that were not identified. It is anticipated that the proposed development grading and drainage design facilities will generally reduce infiltration of water into soils, and reduce the potential for future similar slides. Future design level geotechnical/geologic investigations and construction/grading monitoring services conducted by engineers and geologists (conducted for each of the proposed home sites/residences) should assess the potential presence of slides, potential for reactivation of slides, mitigation of slide risk, and the risk factors for future slides in general (such as finished slope grades, weak layers, adverse bedding conditions, etc.).

Measures to mitigate the potential for damage from new shallow slides are described in the recommendations section of this report (e.g. over-excavation of upper loose soils, surface and subsurface drainage controls, etc.).

Repair of the existing landslide typically includes such measures as removal and recompaction of, or buttressing of the slide and installation of drainage controls. Mitigation of the Lot 3 slide and other slides (if identified) may be required depending on the locations and nature of the proposed residential development. For project planning purposes and to assess the feasibility of mitigating the potential for shallow landslides at the project site, general recommendations for slide repair are provided in subsection 10.7 of this report.

Potential Debris Flow Type Landslide: A relatively deep ravine (20 feet deep or more in some areas) crosses the site from south to north. The ravine is located between the proposed home sites for Lots 6 and 7, and lots 8 and 9. Drainage in the ravine enters a culvert located on Lot 10 near the north end of lots 8 and 9. Due to heavy vegetation, most of the ravine area could not be observed. However, based on our experience on similar nearby sites, it is anticipated that earth slump soils and loose colluvial soils are present in some areas within the ravine. These types of deposits, if large enough and in conjunction with heavy rainfall, can result in damaging debris flows impacting the lower (mouth) areas of the ravine. An assessment of the potential for debris flows to impact the culvert structure and the Lot 10 home site area should be conducted in conjunction with the design level geotechnical investigation for the Lot 10 home site. General recommendations are provided in this report to conduct additional studies to assess the presence of debris in the ravine that could result in damaging debris flows, and to evaluate mitigation measures.

8.1.3 Preliminary Slope Stability Analyses of Site Slopes: Computer aided slope stability analyses were conducted to provide a preliminary evaluation of the general stability of the existing native slopes, as well as the stability of future cut and fill slopes.

The stability analyses were based in part on approximating soil conditions at the site from direct shear laboratory testing of in-situ soil samples obtained from soil borings, and testing of samples remolded from bulk samples of soils excavated from test pits. Explanations of the shear testing procedures and test results are provided in subsection 6.2 of this report.

Shear Strength Values Used in The Analyses: Residual shear strength values were used to model the stability of the existing earthflow landslide material on the site slopes. The residual shear strength value used was selected based on the results of our direct shear laboratory testing conducted by repeatedly re-shearing the same sample (i.e., multiple reverse direct shear), and consideration of the residual soil strength values indicated in the Cleary Consultants investigation reports for landslide debris at the Santa Lucia Preserve. The shear strength value selected for the analyses included the measured angle of internal friction and one-half of the cohesion value measured (residual).

Some small clay shears were noted associated with the relatively small, shallow slide area identified on Lot 3. However, shear zones of a size, continuity, and orientation which would predict a significant slide were not identified at the site within the scope of the field investigation. If significant shear zones are encountered as a result of the design level investigations, these zones should be considered in slope stability assessments conducted for the individual lots. Although continuous weak layers representing potential failure surfaces were not identified during our investigation, the possible presence of weak layers (such as clay layers) was modeled by analyzing translational failure surfaces within the old landslide mass. The residual shear strength value used for these possible weak layers was selected based on the results of our direct shear laboratory testing, and consideration of the residual soil strength values indicated in the Cleary Consultants investigation reports for landslide debris at the Santa Lucia Preserve. The shear strength value selected for the analyses included an angle of internal friction of 2 degrees less than the measured value and one-quarter of the cohesion value measured (residual).

The stability of proposed cut slopes was modeled using a residual shear strength value based on the results of our direct shear laboratory testing, and consideration of the residual soil strength values indicated in the Cleary Consultants investigation reports for landslide debris at the Santa Lucia Preserve. The shear strength value selected for the analyses included the measured angle of internal friction and one-half of the cohesion value measured (residual).

The stability of proposed fill soil slopes was modeled using the peak shear strength results of three (3) direct shear laboratory tests conducted on bulk samples screened over a No. 8 sieve, and remolded to approximate the compacted condition. The samples were screened to simulate potential breakdown of the rock fragments during excavation and recompaction activity. The shear strength value used in the analyses included the lowest angle of internal friction of a sample screened over a No. 8 sieve measured and one-half of the lowest cohesion value measured.

The strength parameters for the granitic rock were estimated based on published values and our experience with similar rock materials in the site region.

Tables No. 1 and 2 provide summaries of the shear strength properties and unit weight values used for the slope stability analyses. Discussion of shear testing and derivation of the strength values is provided in subsection 6.2 of this report.

TABLE NO. 1
SUMMARY OF SOIL STRENGTH AND UNIT WEIGHT PROPERTIES USED FOR NATIVE SLOPE AND CUT SLOPE STABILITY ANALYSES

Soil/Rock Type	Saturated Unit Weight Values Used in Analyses (pcf)	Estimated* or Measured Residual Angle of Internal Friction (ϕ), degrees	Estimated* or Measured Residual Cohesion, psf	Residual Angle of Internal Friction Used in Analyses, psf (ϕ), degrees	Residual Cohesion Used in Analyses, psf
Old Landslide Material	125	28	220	28	110
Weathered Granitic Rock (2)	130	34*	50*	32	0
Potential weak layer/translational failure (1)	125	NM	NM	26	55

pcf - Pounds per cubic foot
psf - Pounds per square foot
1) Unit weight and shear values based on Moore Twining laboratory test results and consideration of data contained in the Cleary Consultant reports (1994 and 2000).
2) Unit weight and shear values based on published ranges for granitic rock at other locations in California.

The results of eleven (11) residual shear tests conducted for the Potrero Subdivision project (Cleary Consultants, Inc., 2000) on samples collected from five (5) old landslides, and generally referred to as landslide debris - angular shale fragments in sandy clay matrix, were reviewed. This material appears to be similar to the landslide debris noted on the subject site. The results indicate angles of internal friction ranging from 23 to 40 degrees (average 33 degrees) and cohesion values ranging from 0 to 390 pounds per square foot (average of 75 pounds per square foot). These soil strength values compare reasonably well with the strength values measured for this investigation, and the average angle of internal friction and cohesion values represent a significantly higher soil strength than was used for modeling landslide stability for this investigation.

TABLE NO. 2
SUMMARY OF SOIL STRENGTH AND UNIT WEIGHT PROPERTIES USED FOR FILL SLOPE STABILITY ANALYSES

Soil/Rock Type	Saturated Unit Weight Values Used in Analyses (pcf)	Measured Residual Angle of Internal Friction (ϕ), degrees	Measured Residual Cohesion, psf	Residual Angle of Internal Friction Used in Analyses, psf (ϕ), degrees	Residual Cohesion Used in Analyses, psf
Potential Engineered Fill (1)	125	24	680	22	105

pcf - Pounds per cubic foot
psf - Pounds per square foot
1) Unit weight and shear values based on Moore Twining laboratory test results. Bulk sample was screened over #8 screen to remove fragments larger than about 2.36 mm in diameter.

Analysis Type, Acceptable Factors of Safety, and Seismic Coefficient: The computer program "Slide," developed by Rocscience, was used to model the slope stability using the Spencer's Method for rotational and translational failure. The analyses were conducted to determine if the existing and planned slopes possess acceptable factors of safety for static and pseudo-static (seismic) stability. A minimum acceptable factor of safety of 1.5 was used for the static case.

To model the impact of seismic ground shaking on site slopes, a seismic coefficient of 0.18 was estimated based on the 5 centimeter Newmark displacement threshold for screening, derived in accordance with the document entitled "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, dated June 2002. The aforementioned document indicates the 5 centimeter displacement value likely distinguishes conditions in which very little slope displacement is likely as a result of the seismic ground shaking. A minimum acceptable factor of safety of 1.0 was used for the pseudo-static (seismic) case in accordance with the Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California.

Site Slope Topography Modeled: Static and pseudo-static slope stability analyses were conducted based on a topographic cross-section through the site, extending downslope across Lots 6, 9, and 10. The plan location of the cross- section is indicated on Drawing 2, Appendix A. The topographic profile is shown on Drawing Nos. 6 through 11, Appendix A. The upper extent of the cross-section extends to the top of the hillside located about 400 feet southwest of the development site, at an elevation of about 730 feet AMSL (the wooded hillside - upper portion of the cross-section is shown on Photograph Nos. 16 and 17, Appendix D). The downslope extent of the profile was located about 350 feet northwest of the Lot 10 home site. The location of the cross section was selected to include areas on the site with the steepest native slopes, and the slightly steeper natural grades upslope of the site. With the exception of the ravine area, the steepest native slopes at the project site, are generally about 2½ H to 1V. Slopes slightly steeper than 2H to 1 V are located in the Monterey Formation material located upslope of the home site areas proposed for Lots 5, 6, and 7.

To model cut and fill slopes, a maximum cut and fill height of 10 feet and a maximum cut and fill slope gradient of 2½H to 1V were assumed.

Generalized Soil Rock Profile and Failure Surface Geometries: A generalized soil/rock profile was developed for use in the stability analyses for the native slopes (see Drawing Nos. 5 through 11, Appendix A). The soil/rock profile was based on the results of our field investigations and interpretation of subsurface conditions based on the mapped geology of the site region. The generalized soil/rock profile includes Monterey Formation porcelanite outcropping on the hillside and hilltop, outside the landslide area at the south end, and south of the project site. As previously indicated, it is estimated that the potential for deep seated slope failure to occur in Monterey Formation material upslope of the project site is low. The Monterey Formation rock is underlain by granodiorite (encountered in all five of the test borings). Moving downslope, the profile crosses the eroded headscarp of the landslide. The soil/rock profile for the remainder of the downslope portions of the site includes upper silty clay soils (earthflow landslide deposit) overlying the granodiorite. Based on the test boring results, the thickness of the earthflow deposit (depth to granodiorite below the ground surface) was varied in the analyses between about 10 and 45 feet BSG near the mid-slope and upper portions of the profile, and a depth of 5 feet BSG was used near the toe of the existing site slope.

The geometry of the failure surface was considered for the analyses. Based on literature review, it is our understanding that the larger landslides in the site region were the result of both rotational and translation failures. Cleary Consultants (2000) reported that four of the five landslides studied in the Potrero Canyon area were rotational failures or had a rotation component. Relatively thin clay shear failure surfaces were identified in some of the test pits excavated for this investigation and were interpreted as clay shear failure surfaces along which movement occurred within the old landslide material. The basal failure surface of the old landslide was not delineated in the test pits or test borings. This may be due to the earthflow nature of the landslide. Cleary Consultants (2000) reported for Landslide #1 in Potrero Canyon: *"A well defined landslide failure plane was not encountered in the exploratory borings drilled through the landslide mass. This may be due to the highly brecciated condition of the landslide materials which suggests rapid disintegration and flow of the landslide block during its descent into Potrero Canyon."*

Considering that a clear failure surface was not identified below the landslide materials on the site, both rotational and translational failure surfaces were modeled. For rotational failure analyses, the program "Slide" automatically analyzes numerous possible failure surfaces based on a "grid of slip centers" designated by the user and calculates the failure surfaces with the lowest factors of safety. Thus, numerous possible arcuate failure surfaces are considered. The grid of slip centers was oriented so that possible circular failure surfaces extending to about the base of the old landslide material (as deep as about 50 feet BSG) were analyzed.

Translational failure was modeled by designating potential weak soil layers (planes) within the old landslide mass material. Based on the designated weak soil layers (planes), the program Slide automatically analyzes numerous potential translational failures along the plane. Planes representing possible shear failure surfaces were oriented both roughly parallel with, and adverse to, the ground slope surface. Shallow and deep translational failure surface depths were also modeled.

Groundwater: A groundwater surface approximately at the ground surface was used for the analyses and estimates of saturated soil unit weight were used in accordance with the Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazard in California, prepared by American Society of Civil Engineers, Southern California Earthquake Center, dated June 2002.

Analyses Results - Native Existing Slopes - Circular Slip Failure: Based on the preliminary slope stability analyses, the native existing site slopes modeled exhibited minimum factors of safety of 1.7 under static conditions and 1.06 under pseudo-static conditions. Accordingly, the minimum factors of safety indicate that the native existing site slopes modeled are theoretically stable under static and pseudo-static (seismic) conditions when considering potential circular slip failure surface (see Drawing Nos. 5A and 5B, Appendix A). In addition to the shear strengths noted in Table No. X above, a preliminary static slope stability analysis was conducted using a lower internal angle of friction of 25 degrees for the older landslide material. The analysis indicated acceptable safety factors (1.5 minimum) for the lower strength values.

Analyses Results - Native Existing Slopes - Translational Failure (Presumed Weak Layers): Based on the preliminary slope stability analyses, the native existing site slopes modeled exhibited minimum factors of safety ranging from 1.0 to about 1.3 for the failure scenarios modeled under pseudo-static conditions (Drawing Nos. 6B, 7B, 8B and 9B, Appendix A). The native existing site slopes modeled exhibited a minimum factors of safety of from about 1.8 to 2.1 under static conditions (Drawing Nos. 6A, 7A, 8A, and 9A, Appendix A). Accordingly, the minimum factors of safety indicate that the native existing site slopes modeled are theoretically stable under static and pseudo-static (seismic) conditions when considering translational failure surfaces.

Cut Slopes Resulting from Excavation of Native Slopes: A maximum cut height of 10 feet and a maximum cut slope gradient of 2½H to 1V were assumed. Based on the preliminary slope stability analyses, the cut slopes modeled using circular slip failure exhibited minimum factors of safety of about1.6 under static conditions and 1.039 under pseudo-static conditions. Higher factors of safety were calculated using translational failure analyses (about 1.8 under static and 1.3 under pseudo-static conditions, see Drawing Nos. 10C and 10D). Accordingly, the minimum factors of safety indicate that the proposed steepest cut slopes modeled are theoretically stable under static and pseudo-static conditions for both circular slip and translational failure surface.

Engineered Fill Slopes: A maximum fill height of 10 feet and a maximum fill slope gradient of 2½H to 1V were assumed. Based on the preliminary slope stability analyses, the fill slopes modeled using circular slip failure exhibited minimum factors of safety of 1.9 under static conditions and 1.17 under pseudo-static conditions (see Drawing Nos. 11A, and 11B, Appendix A). Accordingly, the minimum factors of safety indicate that the proposed steepest fill slopes modeled are theoretically stable under static and pseudo-static (seismic) conditions for both circular slip.

8.1.4 Potential for Future Landslides to Occur on the Site: Based on the cited literature, results of our field observations, results of the slope stability analyses, and contingent on implementation of the recommendations in this report (including grading, drainage and subdrainage), it is our opinion that the potential would be low for relatively deep seated landslides to occur on the native slopes and impact the proposed home site areas, or on slopes graded in accordance with the recommendations in this report. This conclusion also considers our estimate that no natural forces will cause significant changes to the topography upslope or downslope of the project site. Erosion of the toe of the site hillside slope would be unlikely due to the distance of the Carmel River north of the site (300 to 900 feet) and the presence of the golf course between the site and the river. In addition, any significant grading conducted upslope or downslope of the project site should occur under the jurisdiction of Monterey County codes, and appropriate geologic/geotechnical studies should be conducted to demonstrate that the proposed off-site projects would not adversely impact the subject site.

It should be noted that the potential is moderate for shallow landslides to occur outside of the areas to be developed, where mitigative measures are not implemented. However, recommendations provided in this report for mitigative measures such as setbacks, benching and keying fills, surface drainage and subdrainage facilities, etc. to be implemented as a part of the site development, would be expected to significantly decrease the potential for shallow slides occurring in areas to be developed.

Design level investigations should include slope stability analyses based on subsurface investigations at each lot, laboratory soil strength testing results, and the proposed grading configuration.

8.2 Potential for Slope Creep: Slope creep is an imperceptibly slow, generally continuous downward movement of slope forming soils cause by gravity. Based on the test pit observations and site reconnaissance, evidence of some slope creep was noted on native slopes. It should be noted that slope creep would be anticipated on native slopes after development of the site. Accordingly, structures, flatwork, etc. constructed on native slopes without implementing the grading and drainage recommendations of this report, could be damaged. Remedial grading and drainage measures, such as recommended for site development (see section 10.5), should be implemented to decrease the potential for slope creep from occurring in all areas to be developed.

8.3 Design and Construction of Manufactured Slopes: Design and construction of manufactured slopes should consider maximum cut and fill slope gradients, fill placement, and surface and subsurface drainage to enhance the long term stability of graded slopes. Preliminary slope design and construction recommendations are provided in this report based on the existing slope conditions, and the soil conditions revealed during our preliminary investigation. It should be noted that the maximum slope grades are provided on a preliminary basis only. Design level investigations should provide recommendations for maximum cut slopes gradients based on the grading proposed and the slope, soil and rock conditions specific to the building sites.

Recommendations for stripping and removal of top soils, construction of fill slope keyways, and benching of fills horizontally into firm existing soils (as slopes are constructed), are described in the Recommendations section of this report and Drawing No. 5, Appendix A.

8.4 Building Setbacks from Slopes: Preliminary building setbacks from slopes for structures are provided in this report for cut, fill, and native slopes to provide adequate foundation support and protection for structures against erosion. It should be noted that many of the existing slopes near the ravine located in the central portion of the site are as steep as near vertical and are not considered to be stable. Buildings should not be constructed near these steep slopes. In addition, setbacks are recommended from the toe of the steep slope (headscarp) bordering the south side of the Lot 6 home site area.

Design level geotechnical/geologic investigations for the individual residences should assess the stability of existing and proposed slopes based on the proposed grading plan configuration. Setbacks should be provided to protect structures from slope movement, erosion, etc., including the slopes in the area of the ravine. Preliminary recommendations for building setbacks from the ravine and headscarp are presented in this report for preliminary planning purposes.

8.5 Liquefaction and Seismic Settlement: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing usually results. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction. One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments.

The "Geologic Maps Showing Geology and Liquefaction Susceptibility of Quaternary Deposits in the Monterey, Seaside, Spreckles, and Carmel Valley Quadrangles" (Miscellaneous Field Studies Map MF-2096), prepared by William Dupree, 1990, indicate that the project development site is located within two (2) zones of varying liquefaction susceptibility. The proposed home site areas for Lots 1, 2, 3, 4, and 10 are indicated to have "moderate susceptibility for liquefaction," defined as sediments:

> *"which may liquefy in the event of a nearby major earthquake. They include sediments for which moderate susceptibilities were calculated but historical evidence of liquefaction is absent, as well as sediments with high susceptibilities but where the water table is between 10 and 30 feet below the surface."*

Lots 5, 6, 7, 8 and 9 are indicated to have "low susceptibility for liquefaction," defined as sediments:

> *"unlikely to liquefy, even in the event of a major earthquake."*

Loose or medium dense sandy soils were generally not encountered on any of the proposed lots. Dense and hard soils or rock were generally encountered within 5 to 10 feet BSG. Considering the soil and bedrock conditions revealed at the home site locations (including Lots 1, 2, 3, 4, and 10 located in the "moderate susceptibility" area, the potential for liquefaction and lateral spreading (liquefaction occurring on slopes) at the proposed home sites is considered low. It is likely that the aforementioned maps showing geology and liquefaction susceptibility do not accurately delineate the boundary between moderate and low susceptibility areas located near the north portion of the site. It is anticipated that conditions favoring liquefaction (i.e., shallow groundwater, granular soils) persist to the north of (beyond) the lower home sites investigated, closer to the Carmel River.

The Map of Relative Liquefaction Susceptibility contained in the Monterey County Draft General Plan indicates that the proposed home sites (home sites shown on Drawing No. 2) are located in areas of "Low" relative liquefaction susceptibility. The lower elevation portion of Lot 10 (relatively flat area) is located within zones of "High" and "Moderate" susceptibility to liquefaction. These areas of "High" and "Moderate" susceptibility to liquefaction do not include proposed home site areas. If future development is planned in these areas, a site specific liquefaction assessment should be conducted to evaluate the potential for liquefaction and seismic settlement.

Based on the rock and native soil conditions encountered during the field investigation, and considering the fill soil recommendations in this report, the potential for liquefaction to occur at the proposed home site locations is low. A total seismic settlement of ¼ inch is preliminarily estimated for the proposed home site areas as a result of shaking from a design basis earthquake (peak horizontal ground acceleration 0.34g and a predominant maximum magnitude of 6.8). This settlement may not occur uniformly over the site due to variations in the thicknesses of different soil layers; therefore, a differential settlement of about ¼ inch in 40 feet should be anticipated for preliminary design purposes. Analyses for liquefaction and estimates of seismic settlement for structural design should be provided in the design level geotechnical investigation reports.

8.6 **Soil/Rock Expansion Potential**: The surface soils were evaluated for expansion potential. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade and foundations supported on expansive soils can be reduced by placing non-expansive sections underlying foundations and slabs-on-grade.

In consideration of the expansive soils at the site, expansion testing was performed on representative composite samples of the near surface soils. Expansion testing was performed in accordance with UBC Standard 18-2 on two pervasive clayey soil types. The samples tested included a sample of the lean clay earthflow deposit material with abundant siltstone and porcelanite fragments, and a sample of the silt earthflow deposit material with abundant siltstone and porcelanite fragments. The soils tested were classified by expansion potential in accordance with UBC Table 18-1-B. The results of the tests indicate that the soils exhibited "very low" expansive potential (expansion index results of 0 and 12). For planning purposes, and to account for potential variations in the expansive clay content of soils, it is recommended that slabs on grade be supported on a non-expansive soil and aggregate base to over moisture conditioned engineered fill to reduce the potential for shrink/swell damage. For planning purposes, interior slabs should be underlain with 6 inches of imported Caltrans Class 2 aggregate base, and exterior slabs should be underlain by 4 inches of Caltrans Class 2 aggregate base. The aggregate base will serve to reduce the potential impacts of swell and as a capillary break to reduce moisture intrusion. Recommendations are also provided in this report for minimum foundation depths in consideration of the expansion potential of site soils. The foundations should extend to this minimum depth around the entire perimeters of the buildings, including doorways, so that the perimeter foundations can act as a lateral cutoff for migration of moisture.

8.7 **Soil Organic Content**: Based on observations and the results of a Loss-on-Ignition test, it is anticipated that much of the near surface top soils have relatively high organic contents exceeding 3 percent by weight. Soils with relatively high organic contents are susceptible to excessive settlement resulting from decay of the organic matter. Accordingly, organic content testing of the near surface soils (typically dark brown, gray, or black soils in the upper 5 feet BSG) should be conducted during design level geotechnical investigations. However, on a preliminary basis, the organic rich soils should not be used as engineered fill within pavement, building or overbuild zones, unless blended with deeper soil containing a lower organic content under a controlled method approved by the geotechnical engineer. The blending of the upper soils with excessive organic contents would need to be further assessed by additional organic content testing. Blending of soils is typically done during site grading using a pugmill type mixer. Quality control measures should be implemented to assure proper mixing of soils to achieve 3 percent by weight or lower organic content. Testing of the mixed soils should be conducted to confirm suitable blending.

8.8 **Difficulty of Excavation and General Suitability of Site Soils for Use as Engineered Fill:** Considering the results of our field investigations, and the anticipated maximum cuts of 10 feet, it is not anticipated that blasting would be required for excavations planned for the project. However, it should be noted that granitic rock was encountered at depths of about 5, 8, and 13 feet BSG in test borings drilled near lots 1, 2, 5, 6, and 7, and granitic noted rock was noted outcropping east of Lot 5. Deeper excavations into the granitic rock (more than about 3 to 5 feet will likely require pre-ripping with a dozer equipped with rippers. Design level geotechnical investigations should evaluate rippability of the soil and rock materials planned for excavation (based on the individual grading plans).

8.9 **Soil Structural Support and Differential Settlement Across Building Pads:** Our observations and testing indicate that the upper soils are loose/soft, are not suitable to support foundations, fill soils, pavement sections, etc., and could experience excessive settlements if structurally loaded. These soils include the upper brown to dark brown sandy silt and underlying greyish tan and porous silt soils extending to depths of about 1 to 5 feet BSG (soil horizons A and E). During grading of the lots, the loose/soft soils should be removed (over-excavated) prior to placement of fills, pavements or structures. Soils underlying the loose/soft soils are stiff to hard silts and lean clays and appear to be generally suitable to provide support for fill soils, pavement sections, etc. However, the static settlement of these soils should be evaluated as part the design level geotechnical investigations, based on the proposed grading for the individual structures. If excessive static settlement is estimated, additional over-excavation would be required to provide engineered fill below fills and reduce the predicted settlement.

Considering the steepness of the native terrain, it is anticipated that many of the building pads will be constructed by cut and fill grading (i.e., cut/fill transitions). The potential exists for differential settlement to occur laterally across the building pads which are underlain by variable soil and rock conditions, cut-fill transitions and differential fill thicknesses. These conditions present the potential for variable foundation support characteristics and differential settlements across the building pad. For example, foundations supported on very dense/stiff soils or rock would not be anticipated to settle the same as footings supported on the anticipated thicker engineered fill soil sections. Over-excavation of the cut areas of building pads and placement of a relatively uniform layer of engineered fill under foundations would be recommended to reduce differential static settlement occurring over cut and fill transitions (i.e., support footings entirely on a minimum thickness of engineered fill). The required depth of over-excavation to reduce differential settlement should be considered in the design level investigations and will depend on the degree of lateral variation in fill thickness, and the actual structural loads. For planning purposes, it should be expected that over-excavation will be required to provide at least two (2) feet of engineered fill below building foundations. In addition, grading should be conducted to limit the differential fill thickness of the engineered fill to not more than 1 foot of fill thickness variation vertically in 5 horizontal feet of fill.

Contingent on the preliminary recommendations for site preparation in this report (including removal of all loose soils), and over-excavation to reduce differential settlement across building pads, it is estimated that a maximum net allowable soil bearing capacity of 2,000 pounds per square foot may be used for preliminary planning purposes. Based on the above conditions, post construction total and differential static settlements of 1 inch and ½ inch in 40 feet may be used for preliminary planning purposes. Foundation design should also consider seismic settlements of ¼ inch total and ¼ inch differential in 40 feet, in addition to the static settlements.

Net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and concrete foundations may be neglected when calculating the actual bearing pressure imposed by the structure. The net allowable soil bearing pressure presented was selected to satisfy both the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A factor of safety of 3 was used to determine the allowable bearing capacity based on Terzaghi equations.

Design level geotechnical investigations should explore the soil conditions in the individual proposed building areas and provide specific recommendations for the over-excavation depths and allowable bearing capacities for design.

8.10 **<u>Shallow Perched Groundwater and Subdrainage</u>:** Observations of test pits did not reveal pervasive conditions favoring abundant perched groundwater. A potential for a perched groundwater condition was noted in one test pit excavated on Lot 6. It should be anticipated that shallow perched groundwater will occur in some areas and subdrains will be required to cut-off and redirect shallow subflow away from the residences and roadways. Subdrains should also be incorporated into keyways and slope bench fills (see subsection 10.5 of this report and Drawing No. 5, Appendix A). Recommendations for the locations of the subdrains will depend on the details of the proposed site grading. The locations and design of subdrains for residential development should be determined during the design level investigations, based on grading and storm drain plans. However, it should be anticipated that it will be necessary to modify the design and location of subdrains (and perhaps add additional drains) based on the soil/rock and groundwater conditions encountered during rough grading. The engineering geologist should observe the site during rough grading and determine the final design and location details for subdrains.

Depending on the locations of improvements, and the proposed grading, some subsurface water should be anticipated during earthwork and underground utility installation. Dewatering of excavations may be required to facilitate construction and permanent dewatering trenches, or subdrains may be necessary to reduce the potential for saturation of proposed fills and groundwater seepage.

8.11 **<u>Typical Subdrain - General Details</u>:** General details are provided below for typical perimeter subdrains. Specific details will need to be developed during the design level geotechnical investigations for subdrains depending on the application and site conditions (e.g. keyway and slope fill drains). Typical perimeter subdrains include an 18 to 24-inch wide trench extending through permeable soils into dense soils or rock. The trench should be backfilled with a granular material such as a Caltrans Class 2 (non-open graded) Permeable material. If an open graded material, such as crushed rock, is used as backfill for the subdrain trench, it should be fully encapsulated in a geotextile fabric such as Mirafi 140N to reduce the potential for the fine grained soils infiltrating the porous gravel. A perforated drain pipe (4-inch diameter minimum) should be placed (holes down) within the backfill, about 2 inches from the bottom of the trench. All of the drain pipe should be sloped a minimum of 1 percent to drain to the downslope end of the drain. At the downslope end of the drain, the perforated pipe should transition to solid pipe, and the end of the rock filled trench should be filled with a two-sack sand slurry to form a cutoff wall and reduce the potential for water migration within the portion of the trench incorporating the solid piping. The solid drain pipe should be constructed to flow into a natural drainage path, or to a storm drain drop inlet, or other approved drainage outlet. The discharge structure for the subdrains should be designed to prevent damage to the discharge pipe due to future activities such as landscaping, etc., and allow the pipe to be located and cleaned in the future.

If the subdrains are directed to natural drainages, an energy dissipater (3 inch to 6 inch diameter rip-rap) should be placed at the end of the pipe for erosion control as determined by the project civil engineer. It is imperative that the outlet end of the drain pipe be periodically inspected and sediments and/or debris be removed from the end of the drain pipe. Screens should be installed to prevent animals from entering the drain pipes. Subdrains should be designed with adequate clean-outs and inspection ports. The drains should be inspected, i.e., video taped, etc., prior to completion of construction to ensure proper construction.

8.12 **<u>Corrosivity:</u>** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report.

Two (2) samples were tested to assess the corrosivity of soils material which could be encountered during development of the project. The results indicated pH values of 6.1 and 6.8, and minimum resistivity values of 370 and 690 ohm-centimeters, respectively. Based on the resistivity values, the soils exhibited a "very corrosive" corrosion potential. Corrosion soils are typically mitigated by using corrosion resistant materials, coatings, and cathodic protection for buried steel.

If piping or concrete are placed in contact with deeper soils or imported engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

8.13 Sulfate Attack of Concrete: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfate concentrations present in the soils. The test results are then compared with the categories of ACI 318 to provide guidelines to reduce the impact of sulfates on concrete exposed to sulfate-containing soils. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios.

The results of analytical testing of two (2) samples indicated 0.54 and 0.073 percent by weight concentrations of sulfate The results of soil sample analyses indicate negligible and severe sulfate concentrations. Therefore, a "severe" potential for sulfate attack on concrete placed in contact with soils should be anticipated for preliminary design and planning purposes. It is recommended the concrete mix design for concrete in contact with soils be prepared to meet the minimum requirements of the "severe" sulfate exposure.

8.14 Pavement Support: Recommendations are provided in this report for rough grading and pavement support for the development roadways. Recommendations are not provided in this report for private driveways, and sampling, R-value testing should be conducted during design level investigations, and recommendations provided for the private driveways.

Asphaltic concrete (AC) pavements will be required for the proposed development roadways. The anticipated subgrade soil and rock conditions vary widely across the site. The subgrade support characteristics of the native soils were approximated by Caltrans Test Method 301, Resistance (R)-value tests. The results of the tests conducted on two samples of the anticipated subgrade material collected from the upper 3 feet below site grade indicated R-values of 9 and 35. It is recommended that an R-value of 9 be used to estimate preliminary roadway pavement sections for the purpose of preliminary design and planning. The preliminary pavement sections are included in the Recommendations section of this report.

It is customary to collect additional soil samples during rough grading of this type of project, and conduct additional R-value tests to determine appropriate pavement sections for each of the proposed roads.

9.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, documents reviewed, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, we present the following conclusions.

9.1 The results of this preliminary investigation indicate the proposed residential construction is feasible with regard to potential geologic and geotechnical hazards. Supplemental, geotechnical and geologic investigations are recommended as described in subsection 10.2 of this report for preparation of design-level recommedations.

9.2 The soils encountered at the site generally consisted of soft, organic rich, silt top soils underlain by very stiff to hard silts and lean clays with abundant siltstone, claystone, and porcelanite fragments. The underlying silts and lean clays exhibited a chaotic texture comprising a silt or lean clay matrix with abundant angular siltstone, claystone, and porcelanite fragments (⅛ to 1 inch in diameter). The chaotic soil texture appears to represent earthflow type landslide deposit material. Siltstone and granodiorite were encountered below the earthflow soils.

9.3 Groundwater was not encountered in near surface soils in test pits or borings conducted for this investigation. It is anticipated that shallow groundwater will not be pervasive across the site. However, it should be anticipated that shallow perched groundwater will occur in some areas and subdrains will be required to cut-off and redirect shallow subflow away from the residences and roadways. Preliminary recommendations for subdrainage are included in the Recommendations section of this report.

9.4 Based on the cited literature, results of our field observations, results of our slope stability analyses, and contingent on implementations of the recommendations in this report (including grading, drainage and subdrainage), it is our opinion that the potential would be low for relatively deep seated landslides to occur on the native slopes, or on slopes graded in accordance with the recommendations in this report.

9.5 The potential for future shallow rotational or translational slides in native soils, such as noted on Lot 3, is moderate. Design level investigations should include stability analyses based on subsurface investigations at each lot, laboratory soil strength testing results, and consider the proposed grading configuration.

9.6 Test results indicate that the near surface soils predominantly possess low to medium plasticity and very low expansion potential. However, for planning purposes and to account for potential variations in the expansive clay content of soils, it is recommended that a non-expansive engineered fill be placed under slabs-on-grade to reduce the potential for shrink/swell damage as indicated in the Recommendations section of this report.

9.7 Total and differential static settlements for foundations of 1 inch and ½ inch in 40 feet may be used for planning purposes contingent on the site preparation recommendations presented in this report.

9.8 A total seismic settlement for foundations of ¼ inch and a differential seismic settlement of ¼ inch in 40 feet may be used planning purposes contingent on the site preparation recommendations presented in this report.

9.9 For planning purposes, interior slabs should be underlain with 6 inches of imported Caltrans Class 2 aggregate base over moisture conditioned engineered fill, and exterior slabs should be underlain by 4 inches of Caltrans Class 2 aggregate base over moisture conditioned engineered fill. Depending on the expansion potential of the slab subgrade soils, as determined by the design level geotechnical investigations, additional non-expansive fill soils may be required under the aggregate base to reduce the potential for shrink/swell related damage to slabs.

9.10 Based on the resistivity test results, the site soils exhibit a "very corrosive" corrosion potential.

9.11 Moore Twining's analyses of soil samples also indicated a negligible to severe potential for sulfate attack concrete placed in the near-surface soils.

9.12 Based on the soil and rock conditions encountered at the proposed home site areas, the potential for liquefaction to occur at the home site locations is considered low.

9.13 The project site is located in Seismic Zone 4.

9.14 The site is not located in an Alquist-Priolo Earthquake Fault Zone and the potential for fault rupture to occur on the site is estimated to be low.

9.15 The anticipated subgrade soil and rock conditions vary widely across the site. It is recommended that an R-value of 9 be used to estimate preliminary pavement sections for the purpose of preliminary design and planning. Additional sampling and R-value testing should be conducted after rough grading to establish pavement design sections for each roadway. Additional sampling and R-value testing should also be conducted for private driveways during design level investigations .

10.0 PRELIMINARY RECOMMENDATIONS

Based on the evaluation of the field and laboratory data, site reconnaissance, document review and our geotechnical experience in the vicinity of the project, the following preliminary recommendations are provided for site grading, site preparation, and geotechnical design to be used for preliminary design and planning purposes. The report is also intended to provide recommendations for rough grading the proposed roadways.

10.1 General Recommendations

10.1.1 This report should be considered in its entirety. When applying the recommendations for preliminary design, estimating and planning, the background information, procedures used, findings, evaluation, and conclusions should be considered. This is a preliminary geotechnical and geologic investigation only. Supplemental geotechnical and geologic investigations are recommended as described in subsection 10.2 of this report. The recommended design consultation and observation of construction activities by Moore Twining are integral to the proper application of the recommendations.

10.1.2 Preconstruction meetings, including, as a minimum, the owner, general contractor, land surveyor, earthwork subcontractor, foundation and paving subcontractors, civil engineer, a qualified geotechnical engineer and engineering geologist should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing for all phases of the development. The purpose of the meetings should be to discuss critical project issues, concerns and scheduling.

10.1.3 This report provides recommendations for observation of rock and soil conditions on cut slopes during grading, observation of foundation excavations and general excavations. These observations should be conducted directly by a trained geologist or engineer and not a field technician under the direction of a professional. The firm which is retained to provide construction observation services should have a qualified geotechnical engineer and engineering geologist that can perform the required observations.

10.1.4 It is our understanding that on-site sewage disposal systems will not be used for the project. On-site sewage or stormwater disposal systems, as well as unlined storage ponds, are not recommended due to the infiltration of water and potential for slope instability.

10.1.5 If any city, county, and/or state standards are cited on the plans or specifications, these standards should be in addition to the recommendations in this report.

10.1.6 Prior to final grading and paving of the roadways for the development, additional sampling of the subgrade soils and soil testing should be conducted to provide the design pavement sections.

10.1.7 Prior to final bidding and grading. Moore Twining should be provided with detailed improvement plans for review which show the proposed road and driveway construction.

10.1.8 The contractor is responsible to conduct grading in compliance with the applicable building code, the project geotechnical report, the project plans, the project specifications, and Monterey County requirements, whichever is more stringent.

10.1.9 The contractor should comply with SWPPP guidelines, the project plans, the project specifications, whichever is most stringent.

10.2 Future Design Level Investigations

10.2.1 Recommendations for rough grading of the project roadways and driveways, and Lot 1 are provided in this report. During development of the civil and building plans for the roadways and driveways, and equestrian facility (Lot 1), plans should be reviewed by the geotechnical engineer. Also, individual design level geotechnical investigations should be conducted for all of the lots prior to development of final grading plans and construction of the residences.

10.2.2 Design level investigations should evaluate the allowable differential fill thickness when the planned grading is known for individual lots.

10.2.3 Subsurface data from the design level investigations for the proposed residence locations should be evaluated and additional slope stability analyses conducted for the lots and residences based on data from the design level investigations.

10.2.4 Site soils predominantly comprise silts and lean clays, with some silty sands. These soils are expected to exhibit very low to low expansion potential and moderate to high compressibility. The future design level geotechnical investigations for the individual lots should include analyses of the expansiveness and compressibility of the soils and provide recommendations for earthwork (over-excavation and re-compaction), grading, and site preparation to reduce the impact of adverse soil conditions on the residences.

10.2.5 Foundation design should be based on the results of future design level geotechnical investigations and the building plans.

10.2.6 The future design level geotechnical investigation reports should include specific recommendations for grading to improve drainage and reduce the potential for saturation of near surface soils (and potential for slope movements) on the lots. The reports should also include recommendations for subdrains to reduce subsurface water and the potential for instability on the lots, including subdrain construction details, depths, and locations.

10.2.7 The future design level geotechnical investigation report for Lot 10 and any drainage piping/headworks facility (if constructed near the bottom of the ravine) should include upslope investigation near the axis of the ravine to assess the possible presence of debris that could be mobilized as a debris flow and damage the downslope lot and facilities.

10.2.8 In conjunction with the future design level geotechnical investigation reports for Lots 6, 7, 8 and 9 (located in the vicinity of the ravine), surveys should be conducted to establish profiles between the proposed locations of the residences and the axis of the ravine. These profiles should be used to establish structure to ravine setback requirements (see subsection 10.4.6 of this report). In addition, future design level geotechnical investigations for these lots should include assessment of the potential for debris flows to impact the lots.

10.3 Site Drainage on Building Pads

10.3.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two (2) percent for a distance of at least ten (5) feet away from structures, or as necessary to establish positive drainage and to preclude ponding of water adjacent to foundations.

10.3.2 Surface water must not be allowed to pond adjacent to building foundations. To reduce this potential, it is recommended to provide rain gutters and direct all water from roof drains into closed conduits that are connected to an acceptable discharge area away from the building foundations, or upon an impervious surface that will direct water away into a storm drain, or directly into the site storm drain system.

10.3.3 It is not recommended to place landscape or planted areas adjacent to building foundations and/or interior slabs-on-grade. Trees should be setback from proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

10.3.4 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with minimal water requirements are recommended.

10.3.5 Perimeter curbs should be extended to the bottom of the aggregate base section, where irrigated landscape areas meet pavements, exterior slabs on grade, curbs, curbs and gutters, curbs at planters, etc.

10.4 Permanent Site Slope Gradients, Setbacks, Slope Drainage, and Protection

The following preliminary recommendations are provided for planning purposes. Slope grading should be in accordance with the Monterey County Grading Code and recommendations for slope grades and setbacks should be included in the geotechnical design level investigation reports. Subsection 10.6 provides a recommended permanent maximum cut slope gradient for grading roadways (not for driveways).

10.4.1 Cut slopes in the upper topsoil materials should be graded at a repose of 3 H to 1V, or flatter, for stability, and to reduce erosion potential. These soils are most typically present in approximately the upper 3 feet BSG.

10.4.2 Engineered fill slopes and cut slopes below the upper top soil material, to a maximum height of 15 feet, may be graded at a repose of 2 H to 1V, or flatter for stability.

10.4.3 The recommended cut and fill slope grades are provided for project planning and preliminary design purposes. It is anticipated that flatter slopes may be necessary in some areas, depending on the results of design level investigations.

10.4.4 An engineering geologist should observe the cut slopes periodically during grading. Based on observations of test pits, it is anticipated that weak clay layers (shear zones) oriented adversely to cut slopes may be encountered, associated with relatively small dormant and shallow landslides. If these conditions are encountered during grading, supplemental recommendations for grading and design (e.g. flatter grades and higher earth pressures for retaining walls) would be required.

10.4.5 Cut slopes above retaining walls, or cut slopes ascending from backyard areas should be designed with a flat bench at the toe of the slope. These benches should be wide enough to accommodate a surface drain (minimum 10 feet wide), and to permit periodic clearing of rock fall and erosional debris. Cut slopes should also be oriented to achieve at least the minimum horizontal setbacks to permanent structures. A higher frequency of slope maintenance should be expected for the first few seasons after slope grading.

10.4.6 Structures should be setback from cut, fill, and native slopes to provide adequate foundation support and protection for the structure against erosion. The minimum structural setback from ascending cut slopes greater than 10 feet high (foundations at base of slope) and steeper than 3½ H to 1V, is 10 feet or ½ the slope height, whichever is greater. Setbacks should be designed anticipating that some slope erosion will occur and that sediments will have to be removed periodically from the base of the slope. Structures should be setback a minimum of 10 feet or ⅓ the slope height, whichever is greater, from the top of descending slopes (foundations at the top of the cut, fill, or native slopes) which exceed 5 feet in height and are steeper than 3½H to 1V. Where native slopes exceed 2H to 1V and 5 feet in height, structures should be setback from the top of ravines and descending slopes a minimum distance equal to the slope height. This would apply to lots located near the ravine. Additional setback distance may be warranted if design level geotechnical investigations reveal a potential for erosion at the toes of slopes to increase the composite slope gradients.

10.4.7 The home on Lot 6 should be setback at least 75 feet from the toe of the steep slope (headscarp) bordering the south site of the Lot 6 home site area. This is recommended to reduce the potential for damage resulting from slope run-off.

10.4.8 In accordance with the Monterey County Grading Code, the toe of fill slopes should not be closer than twelve feet horizontally from the top of any existing or planned cut slope.

10.4.9 To maintain the stability of site slopes, it will be critical that surface drains and subdrains will be required near the top of cut slopes. It is expected that subdrain trenches will be required to extend into weathered rock or dense soils and aligned perpendicular to the drainage flow. Typical details of the subdrains are provided in the Evaluations section of this report. At a minimum, subdrains should be designed and constructed in accordance with the typical design provided in the Evaluations and Recommendations sections. Keyways and keyway drains are described under subsection 10.5 of this report (also see Drawing No. 5, Appendix A). Lined (concrete or asphalt) brow ditches, "J-gutters," swales, interceptor drains, etc. should also be provided above cut and fill slopes to reduce the potential for surface runoff above the slopes to accelerate erosion and/or instability. At a minimum, the lining should be in accordance with section16.08.330 of the Monterey County Grading Code.

10.4.10 Surface drains and subdrains should be installed upslope of residences to capture and redirect surface and shallow subsurface water away from slopes to closed pipes of the site storm drain system, or to approved swales or drainages. At a minimum, it should be anticipated that perimeter J-gutters and subdrains will be installed upslope of the lots 5, 6, and 7 in accordance with the typical subdrain detail described in the Evaluations section of this report. For planning purposes, it should be assumed that the total lengths of the subdrains above lots 5, 6, and 7 would be approximately 750 feet, with a trench depth of 5 feet.

10.4.11 Lined (concrete or asphalt) gutters, "U-gutters," swales, etc. should be provided at the bottom of slopes, including at the tops of retaining walls where drainage trends toward the walls.

10.4.12 Drainage water from gutter and subdrains should be directed into an approved discharge area such as a natural swale in a non-erosive manner. Energy dissipaters such as rip-rap should be designed to control erosion.

10.4.13 To reduce the potential for rock fall hazards and sediment transport, both during and after fine grading of the cut slopes, all loose materials (soil, rocks, boulders, etc.) should be removed from the slopes to the satisfaction of the project geotechnical engineer. It should be anticipated that periodic, regular maintenance will be required to clean swales and gutters, and repair erosion damage on the slopes until the vegetative cover is well established.

10.4.14 The existing trees, bushes, native grasses, and weeds should remain on the slopes where possible. If the existing vegetation is disturbed, shallow rooted ground cover, as well as deeper rooted trees or bushes, should be planted on the disturbed portions of the slopes to reduce the potential for erosion and surficial slope instability. Cut slopes should be vegetated using effective erosion control procedures such as reinforced, vegetative mats, etc. The manufacturer should be required to provide, in writing, that the material selected is suitable for the intended erosion control measures.

10.4.15 If future erosion or instability in the form of slides, debris or earth flow, accelerated erosion, or other forms of slope instability occur on native or graded slopes, Moore Twining should be contacted to provide recommendations for repair, and the distressed areas should be repaired as soon as possible under the direction of Moore Twining. If instability is allowed to continue, these types of conditions could be an impact to the improvements.

10.5 General Site Preparation

The following preliminary site preparation recommendations are provided for planning purposes only. Supplemental, design level investigations will be required to prepare design level recommendations and geotechnical-related documents for grading and foundation design and for bidding purposes.

10.5.1 The contractor should locate all on-site water wells. All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner upon completion.

10.5.2 All topsoil, vegetation, organics, and trees should be removed from the proposed building, exterior slab, pavement areas, and areas to receive fills. The general depth of stripping should be sufficiently deep to remove the root systems and organic top soils. On a preliminary basis, due to the loose/soft condition and the potential for decomposition of organic matter and settlement, the top soils (typically encountered to depths of about 3 feet BSG and deeper in some areas) are not considered suitable to support fills or structures and should be removed from building areas. The actual depth of stripping should be reviewed by the geotechnical engineer at the time of construction and will need to be deeper in some areas. It is anticipated that deeper stripping than 3 feet will be required in some areas to removed dark brown, organic rich soils. Deeper stripping may also be required if any roots larger than ¼-inch are encountered during grading and in localized areas, such as low areas where water may pond. Stripping should extend laterally a minimum of 10 feet outside the limits of site improvements/grading. Additional organic content testing should be conducted during design level geotechnical investigations. However, on a preliminary basis, the organic rich soils should not be used as engineered fill within building and overbuild zones, unless blended with deeper soil containing a lower organic content. Blending of soils is typically done during site grading and should be conducted under a controlled method (such as the using a pugmill type mixer). Quality control measures should be implemented to assure proper mixing of soils to achieve 3 percent by weight or lower organic content. Testing of the mixed soils should be conducted to confirm suitable blending.

10.5.3 Soils containing organic matter such as root clumps, roots exceeding ¼ inch in diameter, with organic contents above 3 percent should be removed. These materials should be raked and hand-picked, as necessary, to remove tree roots larger than ¼ inch in diameter and concentrated root masses. All roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent should be removed and not used as engineered fill. Limbs, tree branches, roots, etc. should not be disced into the near-surface soils. These materials should be raked and hand-picked, as necessary, to ensure proper removal. It should be anticipated that tree roots exceeding ¼ inch in diameter will extend below the minimum stripping depth. The proper removal of trees and their associated root structures is an important aspect of this project and should be properly planned and monitored. It is anticipated that, based on the size of the trees on site, most tree roots requiring removal may extend to depths of 3 to 6 feet BSG. Tree removal operations and site preparation related to tree removal in building areas should be observed by a qualified geotechnical engineer. If grinding operations are conducted on the site, these areas need to be cleaned of all organic matter. The organic matter should not be used as engineered fill.

10.5.4 After stripping, existing fill soils in areas to be developed (such as fill associated with existing access roads on the site) should be removed and compacted as engineered fill under the observation of Moore Twining prior to placement of fill or structures.

10.5.5 On a preliminary basis, after stripping and removal of vegetation, organic rich soils, and undocumented fill soils, building areas should be over-excavated to: 1) remove all loose or soft native soils (A and E Horizon soils typically extending to roughly 3 to 4 feet BSG), 2) to a depth of at least 2 feet below the bottom of proposed foundations, and 3) to the depth necessary to limit the differential fill thickness to not more than 1 foot vertically in 5 feet horizontally, whichever is deeper. For planning purposes, it should be anticipated that loose and soft soils extend to depths of about 3 to 4 feet BSG across the site. It is anticipated that deeper excavation will be required to remove loose soils in some areas of the site.

10.5.6 Deeper excavations into the granitic rock (more than about 3 to 5 feet into rock will likely require pre-ripping with a dozer equipped with rippers. Design level geotechnical investigations should evaluate rippability of the soil and rock materials planned for excavation (based on the individual grading plans).

10.5.7 Building pad over-excavation should include the building areas and a minimum of five (5) feet beyond the building perimeters, or a horizontal distance equal to the depth of fill below structures, whichever is greater. The building pad over-excavation should also include areas to be occupied by adjacent concrete slabs. The horizontal limits of over-excavation should be shown on the grading plan. It is recommended that extra care be taken by the Contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction for the building pad conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. This is the sole responsibility of the Contractor. The Contractor shall verify in writing to the owner and

Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). This verification shall be performed by a licensed surveyor. The licensed surveyor shall provide a plan and cross-sections that demonstrate that the horizontal and vertical extent of the over-excavation required by this report were achieved. The surveyor shall also provide a written report that states the over-excavation was performed in accordance with the project geotechnical engineering report. This verification should be provide prior to requesting pad certification from Moore Twining or excavating for foundations.

10.5.8 If pools are proposed, it should be anticipated that, at a minimum, over-excavation will be required in the area of the proposed swimming pool and 10 feet beyond the edge of the pool, to provide a minimum of 1 foot of engineered fill below the entire pool bottom and support flatwork. Provisions for preventing hydrostatic pressure below pool shells should also be provided if the geotechnical investigation identifies a potential for groundwater to impact the pool (such as blanket drains and/or purge valves).

10.5.9 After the over-excavation, and upon approval of the bottom of the over-excavation by Moore Twining, the exposed surface should be scarified to a depth of 8 inches, moisture conditioned to at least optimum to three (3) percent above optimum moisture content and compacted as engineered fill. The depth of scarification and compaction should not be included in the depth of engineered fill. All fill required to bring the site to final grades should be placed as engineered fill. In addition, all soils over-excavated should be compacted as engineered fill.

10.5.10 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for over-excavation and native soil moisture conditioning recommended for building pad preparation and the interior slab-on-grade.

10.5.11 Fills should not be placed on slopes steeper than 3 H to 1V unless the fill is buttressed (keyed) in a manner approved by the geotechnical engineer and the final slopes gradients should meet the recommendations of this report.

10.5.12 All fill placed on native slopes steeper than 5H to 1V should be benched horizontally into firm soils or rock at minimum intervals of six (6) feet horizontally prior to receiving additional engineered fill. This includes fills in structural and non-structural areas. A minimum 12 foot wide keyway should be constructed at the toe of the slope. The bottoms of keyway excavations should be sloped a minimum of 1% in the upslope direction.

For bidding and planning estimates, the depth of keyways should extend a minimum of 3 feet into firm old landslide material or rock. Actual keyway depths should be specified by the design level geotechnical report and may be greater based on soil and rock conditions. For purposes of planning, keyway drainage should be included for all fill slopes exceeding 6 feet in height. Backdrains should be installed at the heel of keyways and at every other bench. Backdrains shall consist of an 18 inch wide by 24 inch high section of ¾ inch crushed rock fully encapsulated in a geotextile fabric such as Mirafi 140N or equivalent. Drawing No. 5 should be consulted for details of the recommended keyway and benching. A Moore Twining geologist should be contacted to observe the base of the keyway excavation, benching and fill placement procedures, and keyway/fill drains prior to backfilling.

10.5.13 After excavating cut slopes to design grades, additional removal of material from the slopes may be necessary if loose soils or rock materials remain on the slopes. If left on the slope, loose soils may exhibit instability or rapidly erode, and could present a significant maintenance issue. Loose soils, thicker than 4 inches, should not be left on fill slopes. Fill slopes should be over-built and trimmed, or track-walked to compact and provide a firm soil surface.

10.5.14 After stripping and removal of vegetation and organics, areas proposed for fills which do not support structures, areas of pavements, or areas with exterior slabs (outside the building and overbuild zones) should be over-excavated to: 1) remove loose or soft native soils and organic rich soils (soil horizons A and E), and 2) to a depth of at least 12 inches below the bottom of the aggregate base section, whichever is deeper. For planning purposes, it should be anticipated that loose/soft and organic rich unsuitable soils requiring removal extend to depths of 3 to 4 feet BSG.

10.5.15 Interior floor slabs should be supported on a minimum of 6 inches of aggregate base, over the depth of engineered fill which extends to the depth recommended below the foundations.

10.5.16 Areas proposed for exterior slabs (outside the building and overbuild zones) should be over-excavated to provide a minimum of 4 inches of Class 2 aggregate base, over engineered fill that extends to firm soils or rock (below soil horizons A and E).

10.5.17 It is recommended that prior to placement of asphaltic concrete adjacent to slabs-on-grade, curbs, and gutters, that the areas immediately adjacent to these features be compacted with equipment that can provide adequate compactive effort to the aggregate base adjacent to the vertical face of the concrete to achieve a dense, non-yielding condition. This compaction operation should be observed by Moore Twining.

10.5.18 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill, including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If used, open graded materials should be vibrated, and mechanically compacted to a dense, non-yielding condition. Maximum lift thickness of 12 inches is recommended. Each lift must be approved prior by Moore Twining prior to placing the next lift. The fabric manufacturer should provide written confirmation that the fabric is suitable for the intended use. Contractors should assume for the purpose of bid that no rock or gravel can be used for backfill on the project including utility trenches of any kind.

10.6 Maximum Cut Slope Gradients Adjacent to Roadways

Recommendations are provided in this report for rough grading for the development roadways. Recommendations are not provided in this report for private driveways, and should be included in the design level investigation reports.

10.6.1 Exposed cut slope gradients associated with roadway grading of between 1.5 horizontal (H) to 1 vertical (V) and 2 horizontal to 1 vertical can be excavated up to 4 feet high during rough grading. Steeper cut slopes may be used to preserve trees on a case-by-case basis, based on site review and approval by Moore Twining. It should be noted that slopes flatter than 3H to 1V will be less susceptible to surficial instability and will require considerably less maintenance that steeper slopes. Where building or other roads are anticipated to be near the top of slopes, flatter slope gradients or retaining walls may be required. A maximum slope gradient of 2 H to 1 V is recommended in this report for lot grading. Higher grade changes could be accommodated by retaining walls placed at the fill or cut sides of the roads. Retaining walls at the cut sides of the roads should have an 18 to 24 inch wide bench with a drainage swale and freeboard at the top of the wall to reduce the potential for soils overtopping the walls. If slopes can be kept to 2H to 1 V or flatter, maintenance and erosion/stability issues will be reduced over the 1.5 H to 1 V slopes. Surface drainage should be directed away from slopes (not onto slopes) as indicated in subsection 10.5 of this report. Recommendations for retaining walls are provided in subsection 10.11 of this report.

10.7 Slide Repair

General recommendations are provided for project planning purposes for repair of relatively shallow slides such as the shallow slide identified on Lot 3. Appropriate details for slide repairs will be dependent on the individual lot grading plans and proposed building locations. Slide repair recommendations should be provided for dormant and active slides, if revealed during the design level investigations for the individual lots.

10.7.1 Slide repairs should address the causes of the slides specifically. Shallow slides in the site region are typically caused by seepage of surface and subsurface water into near surface clayey soils, and a reduction of shear strength in low strength clayey soils. Recommendations for repair should be based on a field investigation including trenches and characterization of the nature and extent of the slide by a qualified certified engineering geologist. In some cases, the addition of surface and subsurface drainage and buttressing at the toe of the slope may mitigate the potential for re-activation of the slide. However, removal and replacement of the slide mass, in conjunction with drainage enhancements, generally produces a more stable slope configuration because over-steepened slopes in the slide headscarp area are buttressed with engineered fill.

10.7.2 Design level geotechnical recommendations should be prepared for the repair of the shallow slide noted on Lot 3 (design level geotechnical report). However, for planning purposes, the following recommendations for removal and replacement of the slide mass, in conjunction with drainage enhancements, may be used for planning purposes. The slide mass on Lot 3 (roughly estimated to be about 60 cubic yards). The excavation should extend laterally beyond all scarps and headscarps, and vertically to remove low shear strength clayey soils to below the surface of the rupture. The actual extent of the landslide and volume of the excavation should be determined by a qualified certified engineering geologist based on field observations in conjunction with the design level geotechnical investigation or during the grading repairs.

10.7.3 A keyway should be excavated at the downslope edge of the repair area and a keyway drain should be installed (see subsection 10.5 of this report for keyway recommendations). The depth of keyways should be a minimum of 3 feet into firm old landslide material or rock. The keyway and excavated landslide area should be backfilled with engineered fill benched into firm native soils as recommended in this report. Subdrains may be required at intermediate benches depending on the size of the slide repair and groundwater conditions. Observations and approval of grading and fill placement procedures by a geologist should be in accordance with subsection 10.5 of this report.

10.7.4 A perimeter subdrain should be installed upslope of the slide repair area (see subsection 10.5 of this report for subdrain recommendations).

10.7.5 Surface water should be directed away from the slide repair area by the use of gutters, swales, etc., placed upslope of the slide area.

10.8 Engineered Fill

10.8.1 For planning purposes, it should be anticipated that native organic rich top soils (greater that 3 percent by weight organic material as determined by loss-on-ignition test) will extend to an average depth of about 36 inches BSG. Additional organic content testing should be conducted during design level geotechnical investigations. However, on a preliminary basis, the organic rich soils should not be used as engineered fill within building and overbuild zones, unless blended with deeper soil containing a lower organic content based on methods acceptable to Moore Twining (see subsection 10.5 of this report). Blending of soils is typically done during site grading. Soils with greater than 3 percent organic content may be suitable for use as non-structural engineered fill outside of building and pavement areas.

10.8.2 For planning purposes, it should be anticipated that on-site native soils below about 36 inches BSG, will be generally suitable for use as engineered fill material within structural/building and pavement areas (except where imported, non-expansive soils or granular free draining soils are recommended) provided they are free of organics (roots less than ¼ inch in diameter and less than 3 percent by weight as determined by loss-on-ignition test), debris, the moisture content of the soil is within optimum to three (3) percent above optimum moisture content at the time of placement, and the maximum particle sizes comply with the recommendations below. Clayey soils should be placed and compacted in accordance with subsection 10.8.10 of this report.

10.8.3 Rock materials smaller than 6 inches in diameter may be used as fill (building areas plus 5 feet beyond the perimeter of the buildings, sidewalks, canopy foundations, etc., whichever is further), at depths below 3 feet below the bottoms of foundations and utility trenches. Rocks larger than 6 inches in diameter should be encapsulated by soil and nesting of rock fragments will not be permitted. Voids between rock fragments will not be permitted. Rock material larger than 3 inches should not be used as fill in the building zone within the upper foot below bottom of foundations and utility trenches.

10.8.4 If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

10.8.5 On-site soil materials should not be placed in the areas behind any retaining wall that is defined by a line which extends from one (1) foot behind the wall at the base and, to the surface at an inclination of 1H to 1V, or flatter (granular soils required).

10.8.6 It is recommended that imported granular fill be placed behind retaining walls to enhance subdrainage behind the walls and the reduce the potential for swell related damage to walls. At a minimum, retaining walls require granular fill placed within the zone defined by a line which extends from one (1) foot behind the wall at the base and, to the surface at an inclination of 1H to 1V, or flatter. Retaining walls should be constructed with imported fill meeting the requirements of this section for import fills. Additional minimum requirements for retaining walls are provided in subsection 10.11.

10.8.7 The compactability of the soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, they should be evaluated by the contractor during preparation of bids and construction of the project.

10.8.8 Imported fill should be non-contaminated, non-corrosive, non-expansive, granular in nature and contain enough fine grained material (binder) to allow cutting "neat" footing trenches with the following acceptance criteria recommended.

Table No. 3
Acceptance Criteria for Import Fills

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	15 - 40
Plasticity Index	Less than 15
Expansion Index (UBC 18-2)	Less than 10
R-Value	Minimum 40
Organics	< 3% by weight
Sulfates	< 0.05 % by weight
Min. Resistivity	>5,000 ohm-cm

Prior to importing fill, the Contractor shall submit test data to Moore Twining that demonstrates that the proposed import material complies with the recommended geotechnical criteria. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of Moore Twining) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, recent analytical data, including appropriate chain-of-custody documentation, specific to the source of the import material. After receipt and approval by Moore Twining, of the data for geotechnical and environmental compliance of the proposal import material, Moore Twining will sample and test the proposed import material. Prior to being transported to the site, the import fill material should be tested and approved by Moore Twining. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested.

10.8.9 On-site and imported fill soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to three (3) percent over optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

10.8.10 Significant quantities of medium to high plastic clay soils are not anticipated. However, if clay soils are used as engineered fill, the soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within one (1) to four (4) percent above optimum moisture content, and compacted to a dry density of at least 90 but not more than 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Clay soils are potentially expansive and should not be used as fill within the upper 24 inches below building slabs-on-grade.

10.8.11 For all fills placed which will be deeper than 5 feet below finished grades, soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to three (3) percent above the optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

10.8.12 All fill required to bring the site to final grade should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

10.8.13 Aggregate base should be moisture-conditioned to within optimum to two (2) percent above optimum moisture content and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

10.8.14 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill, including trench backfill (except that open graded gravel may be used as drain material if fully encapsulated in a filter fabric). In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If used, open graded materials should be vibrated, and mechanically compacted to a dense, non-yielding condition. Maximum lift thickness of 12 inches is recommended. Each lift must be approved prior by Moore Twining prior to placing the next lift. The fabric manufacturer should provide written confirmation that the fabric is suitable for the intended use.

10.8.15 Aggregate base should meet the requirements of Caltrans Class 2 aggregate base. The contractor shall test the aggregate base and provide the results to the Owner, Architect and Moore Twining for approval prior to delivery of the aggregate base to the site. The Contractor shall provide a certification that the aggregate base is clean, i.e., does not contain contaminates that are regulated by the local, state and federal government.

10.8.16 Recycled materials (AC materials, construction materials, etc.) should not be used within 10 feet of any improvement without approval by the owner, and/or the qualified geotechnical engineer. Contractors should not assume that recycled materials (AC construction materials, etc.) can be used in preparing bids for the project without approval by the owner, and/or architect.

10.9 Foundation Design

This section provides preliminary foundation design criteria for planning purposes. Final settlement design criteria should be based on the project grading and building plans and the results of the additional design level investigations.

10.9.1 For planning purposes, it should be expected that over-excavation will be required to provide at least two (2) feet of engineered fill below building foundations. In addition, grading should be conducted to limit the differential fill thickness of the engineered fill to not more than 1 foot of fill thickness variation vertically in 5 horizontal feet of fill.

10.9.2 Contingent on the preliminary recommendations for site preparation and over-excavation to reduce differential settlement across building pads, it is estimated that a range of maximum net allowable soil bearing capacities of 2,000 to 2,500 pounds per square foot may be used for preliminary planning purposes. These values may be increased by one-third for short duration wind or seismic loads. Preparation of fill below the bottom of foundations should be in accordance with the recommendations in the Site Preparation section of this report.

10.9.3 The foundations should be designed and reinforced for the anticipated settlements. As a minimum, all continuous footing should be reinforced with one (1) No. 4 reinforcing bar top and bottom in the foundation. However, a structural engineer experienced in foundation design should recommend the thickness, design details, concrete and reinforcing specifications for the foundations based on: 1) a combined total static and seismic settlement of 1¼ inch, 2) a combined differential static plus seismic settlement of ⅝ inch in 40 linear feet of continuous footings; 3) a swell of ½ inch in 40 feet.

10.9.4 Exterior foundations for one-story should be supported at a minimum depth of 18 inches below finish pad grades or adjacent finished grades, whichever is lower. Interior footings should be supported at a minimum depth of 12 inches (18 inches for 2-story portions) measured from the top of the interior slab-on-grade. Footings should have a minimum width of 12 inches and 15 inches for one and two story construction, respectively, regardless of load.

10.9.5 The perimeter foundations should be continuous around the entire perimeters of the structures to reduce the potential for moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

10.9.6 Foundation excavations or exposed soils should not be left uncovered and allowed to dry such that the moisture content of the soils is less than optimum moisture content or drying produces cracks in the soils. The exposed soils, such as sidewalls, excavation bottoms, etc. should be continuously moisture conditioned to maintain the moisture content at least one percent above optimum until concrete is placed. It should be noted that the contractor should take precautions not to allow the exposed soils to dry, including on weekends and holidays. The geotechnical engineer should observe the bottoms and sides of the foundations excavations, and exposed soils to verify that the excavations and exposed soils are properly moisture conditioned, and comply with the requirements of the geotechnical engineering investigation report prior to placement of concrete. If dry soils are noted, the contractor should request written recommendations from our firm to properly moisture condition the foundation excavations.

10.9.7 For preliminary planning purposes, structural loads for miscellaneous foundations (such as retaining walls), may be supported on spread or continuous footings placed entirely on at least 2 feet of engineered fill which extends to undisturbed firm native soils or rock. The zone of over-excavation and compacted engineered fill should extend a minimum of 5 feet outside the edges of foundations. Spread and continuous footings may be designed for maximum net allowable soil bearing pressures of 2,000 to 2,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

10.9.8 The following factors were developed based on the tables in Chapter 16 of the 2001 CBC and the digitized active fault locations published by CGS.

Table No. 4
Seismic Design Factors/Coefficients

Design Factor/Coefficients	2001 CBC Value
Soil Type	S_c
Seismic Source Type	A*
CBC Seismic Zone	Z = 0.4
Near Source Acceleration Factor	1.1
Near Source Velocity Factor	1.4
Seismic Acceleration Coefficient, **Ca**	0.44
Seismic Velocity Coefficient, **Cv**	0.74

* Although not the closest fault to the site, the San Gregorio fault *(Type A source) governs design.

10.9.8.1 The following factors were developed based on the tables in Chapter 16 of the 2007 CBC and the digitized active fault locations published by USGS.

Item	2007 CBC Value
Site Class	C
Spectral Response At Short Period (0.2 Second), Ss	1.519
Spectral Response At (1-Second) Period, S_1	0.631
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.3
Maximum considered earthquake spectral response acceleration for short period (0.2 second), S_{MS}	1.519
Maximum considered earthquake spectral response acceleration for 1 second, S_{M1}	0.820
Five percent damped design spectral response acceleration for short period, S_{DS}	1.012
Five percent damped design spectral response acceleration at 1-second period, S_{D1}	0.547

10.9.9 Foundation excavations should be observed by the geotechnical engineer prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement.

10.10 Retaining Walls

This section provides preliminary design criteria for retaining walls for planning purposes. Final design criteria should be based on the project grading and building plans and the results of the additional design investigations.

10.10.1 Retaining wall plans, when available, should be provided to the geotechnical engineer of record for review of actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters.

10.10.2 For preliminary design, retaining wall foundations should be supported on at least 2 feet of engineered fill which extends to undisturbed and firm native soils. The zone of over-excavation and compacted engineered fill should extend a minimum of 5 feet outside the edges of foundations. Footings should have a minimum width of 12 inches, regardless of load, and a minimum depth of 24 inches below rough pad grades or adjacent exterior grades, whichever is lower.

10.10.3 It is anticipated that retaining wall footings may be designed for maximum net allowable soil bearing pressures of 2,000 to 2,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

10.10.4 An engineering geologist should observe the cut slopes periodically during excavation for retaining walls supporting cuts. Based on observations of test pits, it is anticipated that bedding and weak clay layers oriented adversely to cut slopes may be encountered. If these conditions are encountered during grading, supplemental recommendations for grading and design (e.g. flatter grades and higher earth pressures for retaining walls) would be required.

10.10.5 Retaining walls should be constructed/backfilled with imported fill meeting the requirements below. The select fill requirements apply to all retaining wall fill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular backfill will reduce the effects of swell pressures on the wall. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	10 - 30
Plasticity Index	Less than 10
Expansion Index (UBC 18-2)	Less than 10
Organics	< 3% by weight

10.10.6 Retaining wall backfill material should be tested and approved as indicated under the Engineered Fill recommendation section of this report.

10.10.7 Segmented wall design (mechanically stabilized walls) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for segmental wall design. A design level geotechnical report should be conducted to provide segmental wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. The design engineer shall provide sufficient site observations during construction to certify that the wall(s) were constructed in accordance with the approved plans and specifications.

10.10.8 Retaining walls may be subject to lateral loading from pressures exerted from the soils, groundwater, slabs-on-grade, and pavement traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

10.10.9 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

10.10.10 Retaining walls should be designed with a drain system including permeable backfill and drain pipes behind the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The wall designer should provide the design details for the drainage system. The Contractor shall have the connection points to the storm drain system for the wall drainage system surveyed and document this survey in writing. This documentation shall be provided to the Owner and project architect.

10.10.11 If open graded materials such as crushed rock are used as drain material, these materials should be fully encased in a filter fabric and compacted to a non-yielding condition under the observation of the geotechnical engineer. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric. Class 2 permeable material should be compacted to 95 percent relative compaction (CAL Test 216) using a vibratory plate.

10.10.12 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

10.10.13 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., waterproofing measures should be applied to moisture proof the exterior of the walls. Waterproofing should also be used if effervescence (discoloration of wall face) is not acceptable. Waterproofing should be designed by a qualified professional.

10.10.14 It is recommended that the designers for retaining walls be required to observe the construction of the walls as required to certify that the walls were constructed in accordance with the approved designs.

10.11 <u>Frictional Coefficient and Earth Pressures</u>

Ranges of preliminary earth pressures provided below for planning purposes and are based on level backfill conditions above retaining walls and do not include the effects of surcharges, such as foundation or traffic loads. Retaining walls should be designed based geotechnical recommendations from design level investigations.

10.11.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.25, can be used for design. In areas where slabs are underlain by a synthetic moisture retarding membrane, an allowable coefficient of friction of 0.10, reduced by an appropriate factor of safety, can be used for design.

10.11.2 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 220 pounds per cubic foot.

10.11.3 A combination of passive earth resistance and friction may be utilized provided that the friction component of the total is further reduced by a factor of 1.5 (an additional reduction). The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

10.11.4 The passive pressure was calculated based on a minimum soil unit weight of 100 pounds per cubic foot. The soils within the passive zone at the foot of retaining walls (one footing width in front of the wall to a depth equal to the footing depth) should be tested to verify that the soils have the minimum unit weight of 100 pounds per cubic foot (with moisture). If the soils have a unit weight of less than 100 pounds per cubic foot, the soils within this zone should be over-excavated and replaced as engineered fill. These soils should be tested prior to backfilling behind the wall.

10.11.5 Active pressure of the native soil and engineered fill may preliminarily be assumed to be equal to the pressures developed by a fluid with a density of 59 pounds per cubic foot. The at-rest pressures of the native soils and engineered fill may preliminarily be assumed to be equal to the pressures developed by a fluid with a density of 81 pounds per cubic foot. These ranges of earth pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

10.11.6 The active and at-rest pressures were calculated based on a maximum soil unit weight of 130 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 130 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 130 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of over 130 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind retaining walls.

10.11.7 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

10.11.8 The wall designer should determine if seismic increments are required. If seismic increments are required, Moore Twining should be contacted to provide the loads.

10.11.9 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a drain, a filter fabric encased gravel section and a geo-composite system, to prevent hydrostatic pressures from acting on the walls. Drainage should be directed to perforated pipes running parallel to the walls which can carry drainage from behind the walls to the on-site drainage system. Clean-outs should be incorporated into the design.

10.12 Interior Concrete Slabs-on-Grade

The following preliminary slabs-on-grade recommendations may be used for preliminary design and planning.

10.12.1 The recommendations provided herein are intended only for the preliminary design of interior concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, concrete trucks, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

10.12.2 As a minimum, all slabs on grade should be reinforced with No. 4 reinforcing bar at 18 inches on center each way. However, a structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade based on the anticipated foundation settlements. It is recommended that the slabs-on-grade be reinforced. As noted previously, a static settlement of 1 inch total and ½ inch differential settlement over 40 feet should be anticipated for design. A total and differential seismic settlement of ⅛ inch should also be considered.

10.12.3 Interior floor slabs should be supported on a minimum of 6 inches of aggregate base compacted to a minimum of 95 percent relative compaction over engineered fill which extends to the depth of engineered fill recommended below the bottom of foundations.

10.12.4 The moisture content of the subgrade or engineered fill below the base rock should be verified to be optimum to three (3) percent above optimum moisture content prior to placing non-expansive fill, and also within 48 hours of placement of the vapor retarding membrane or the concrete for the slab-on-grade if a vapor retarding membrane is not used. The moisture content of the subgrade beneath the base rock section, to a depth of at least 12 inches, should be tested and confirmed prior to placement of the base rock section, vapor retarding membrane or slab-on-grade. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

10.12.5 In the event that the earthwork operations for this project are conducted prior to the construction of the individual structures such that the construction sequence is not continuous (or if construction operations disturb the surface soils), it is recommended that the exposed subgrade to receive floor slabs be tested to verify adequate moisture content and compaction. If the moisture content just prior to placement of the floor slab is not at least above optimum to three percent above optimum, the soils should be moisture conditioned to at least optimum prior to placing a vapor retarder or concrete. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 92 percent of the maximum dry density as determined by ASTM Test Method D1557. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

10.12.6 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

10.12.7 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarding membrane when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 or equivalent should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The layer of Stegowrap 15 should overlay a minimum of 6 inches of compacted Class 2 AB. It should be noted that placing the PCC slab directly on the vapor retarding membrane will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Mr. Eric Gerst with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the Class 2 AB and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with ASTM C 755-02, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to ASTM E 154-99 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor retarding membrane selection and installation conform to the ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R-96), Addendum, Vapor Retarder Location and ASTM E 1643-98, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements.

10.12.8 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

10.12.9 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.

10.12.10 The vapor retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusions into the structure are permissible for the design life of the structure.

10.12.11 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.45 lb./lb. or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

10.12.12 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity, etc., at a frequency and method as specified by the flooring manufacturer or as required by the plans and specifications, whichever is most stringent. The results of vapor transmission tests, pH tests, internal relative humidity tests, ambient building conditions, etc. should be within floor manufacturer's and adhesive manufacturer's specifications at the time the floor is placed. It is recommended that the floor manufacturer and subcontractor review and approve the test data prior to floor covering installation.

10.12.13 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to interior columns; and 2) provide at least 6 inches of aggregate base below the slabs. The loaded track and/or pad pressure of any crane which may operate on slabs or pavements should be considered in the design of the slabs and evaluated by the contractor prior to loading the slab. If cranes are to be used, the contractor should provide slab loading information to the slab design engineer to determine if the slab is adequate.

10.13 Exterior Slabs-On-Grade

The preliminary recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc. The slabs on the project to be prepared as exterior flatwork include: all sidewalks not including the store front, sidewalks adjacent to the residences and other slabs adjacent to the residences.

10.13.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for foundations and floor slabs. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

10.13.2 The exterior slabs-on-grade (slabs, sidewalks, etc.) should be supported on a minimum of 4 inches of aggregate base over engineered fill which extends to firm native soils. The overbuild limits for these exterior slabs should be a minimum of 5 feet beyond the outside edges of the slabs. If any city, county, and/or state standards are cited on the plans or specifications, these standards should be in addition to the recommendations in this report.

10.13.3 The exterior slabs-on-grade should be designed with thickened edges which extend a minimum of 4 inches into subgrade below the bottom of the aggregate base section and/or non-expansive engineered fill. This should reduce the potential for infiltration of water into the non-expansive engineered fill or aggregate base section below exterior slabs and to reduce a potential for swell/expansion related damage.

10.13.4 The moisture content of the subgrade below the non-expansive section should be verified to be optimum to three (3) percent above optimum prior to placing non-expansive fill, and also within 48 hours of placement of the concrete for the slab-on-grade. The moisture content of the subgrade beneath the non-expansive section to a depth of at least 12 inches should be tested and confirmed prior to placement of the non-expansive fill section, vapor retarding membrane or slab-on-grade. If necessary to achieve the recommended moisture content, the clayey subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

10.13.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify the geotechnical engineer to conduct in-place moisture and density tests prior to placing non-expansive fill and concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

10.14 Asphaltic Concrete (AC) Pavements

Preliminary recommendations are provided in this report for pavement support for the development roadways. Recommendations are not provided in this report for private driveways, and sampling, R-value testing should be conducted during design level investigations, and recommendations should be provided for the private driveways.

10.14.1 Grading for the roadways should be conducted in accordance with the recommendations of this report. Maximum slope grades should be designed and constructed in accordance with subsection 10.6 of this report.

10.14.2 After stripping, all existing fill soils as associated with the downslope portions of the existing access roads should be removed and compacted as engineered fill under the observation of Moore Twining.

10.14.3 Additional R-value sampling and testing should be conducted after rough grading to determine appropriate pavement sections for each of the proposed roads.

10.14.4 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for over-excavation, non-expansive soils, and native soil moisture conditioning recommended in the interior slab-on-grade section of this report.

10.14.5 Areas proposed for pavements (outside the building and overbuild zones) should be excavated to remove the dark, organic rich top soils (generally encountered to a depth of about 3 feet BSG) and a minimum of 12 inches below the structural pavement sections, whichever is deeper. The excavation should expose undisturbed firm soils below the top soils (estimated at a depth of at least 3 feet below preconstruction site grades).

After the over-excavation, and upon approval of the bottom of the over-excavation by the geotechnical engineer, the exposed surface should be scarified to a depth of 8 inches, moisture conditioned to at least optimum to three (3) percent above optimum moisture content and compacted as engineered fill. The pavement subgrade (to 12 inches beneath aggregate base or subbase) should be conditioned, i.e., wetted or aerated, as necessary to achieve the required moisture content and compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

10.14.6 The following preliminary pavement sections are based on an R-value of 9 and traffic index values of 4.0 to 8.0. The traffic indices and sections used for the project should be based on projected traffic loads and Monterey County requirements. The project civil engineer should select the appropriate traffic indices for each roadway. Construction traffic should be considered in the traffic loading design if pavements are installed prior to construction. If traffic indices higher than 8.0 are considered for the project, Moore Twining should be contacted to provided additional pavement sections.

Table No. 5
Recommended Asphaltic Concrete (AC)
Pavement Sections for New Roads

Traffic Index	AC thickness (inches)	AB thickness, (inches)	ASB (inches)	Compacted Subgrade (inches)
4.0	2.0	8.0	--	12
4.0	2.0	4.0	4.5	12
4.5	2.5	10.0	--	12
4.5	2.5	5.0	5.0	12
5.0	2.5	11.0	--	12
5.0	2.5	5.0	5.5	12
5.5	3.0	12.5	--	12
5.5	3.0	6.0	7.0	12
6.0	3.0	14.0	--	12
6.0	3.0	6.5	7.5	12
6.5	3.5	15.5	--	12

Traffic Index	AC thickness (inches)	AB thickness, (inches)	ASB (inches)	Compacted Subgrade (inches)
6.5	3.5	7.0	8.0	12
7.0	3.5	16.5	--	12
7.0	3.5	7.5	9.5	12
7.5	4.0	7.5	--	12
7.5	4.0	17.0	10.0	12
8.0	4.5	17.0	--	12
8.0	4.5	7.5	10.5	12

AC - Asphaltic Concrete compacted to a minimum of 95 percent relative compaction.
AB - Caltrans Class 2 Aggregate Base compacted to at least 95 percent relative compaction (ASTM D-1557) or Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) or Processed Miscellaneous Base (PMB) per Greenbook Standard Specifications.
ASB - Caltrans Class 2 aggregate subbase or Greenbook select subbase (R-value =50 min.) compacted to at least 95 percent relative compaction (ASTM D-1557).
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557).

10.14.7 Aggregate base shall comply with Class 2 aggregate base (AB) per State of California Standard Specifications, or comply with Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) or Processed Miscellaneous Base (PMB) per Greenbook Standard Specifications, where specified. Aggregate base below the interior slab-on-grade should consist of either Class 2 AB or CAB. Aggregate base shall be compacted to a minimum relative compaction of 95 percent in accordance with ASTM D1557 standards. Documentation should be provided to the Owner, Architect and Moore Twining prior to delivery of the aggregate base to the site, or prior to placement and compaction of recycled base materials if recycled materials are considered acceptable by the project owner.

10.14.8 Aggregate subbase (ASB) shall comply with the State of California Department of Transportation requirements for Class 2 aggregate subbase, or comply with the requirements for Select Subbase (Greenbook, Section 200-2.6) and be compacted to at least 95 percent relative compaction (ASTM D1557). Documentation should be provided to the Owner, Architect and Moore Twining prior to delivery of the aggregate subbase to the site, or prior to placement and compaction of recycled subbase materials if recycled materials are considered acceptable by the owner.

10.14.9 The project owner and design team should consider placing a geotextile fabric of Mirafi 600X, or equivalent, below the AB section and on a compacted subgrade to extend the life of the pavements. This is an alternative for the owner to consider and is not intended to become a project requirement unless elected by owner. A geotextile fabric would help prolong the life of the pavements by preventing fine grained subgrade soils from migrating into the AB section.

10.14.10 The contractor shall proof roll the subgrade of the areas to receive pavements prior to placement and compaction of the aggregate base (AB). All unstable areas should be removed, stabilized, and replaced with engineered fill under the observation of Moore Twining.

10.14.11 Prior to placement of asphaltic concrete adjacent to slabs-on-grade, curbs, and gutters, the contractor shall compact the area immediately adjacent (2 feet minimum) to these features with equipment that can provide adequate compactive effort to the aggregate base adjacent to the vertical face of the concrete to achieve a dense, non-yielding condition and a minimum of 95 percent relative compaction. These compaction operations should be observed by Moore Twining.

10.14.12 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

10.14.13 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

10.14.14If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

10.14.15 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

10.14.16 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the State of California Standard Specification Requirements, or Monterey County Standards, whichever is the most stringent.

10.14.17 The asphaltic-concrete should be compacted to a minimum relative compaction of 95 percent based State of California Test Methods maximum density and a minimum joint density of 96 percent.

10.14.18 The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements, or as required by Monterey County Standards, whichever is more stringent.

10.15 Temporary Excavations

10.15.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

10.15.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. For planning purposes, temporary cut slopes in soil should not be steeper than 1½ to 1, horizontal to vertical, and flatter if possible. Temporary cut slopes into rock should not be steeper than 1 to 1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be shored.

10.15.3 Shoring systems, if used, should be designed by an engineer with experience in designing shoring systems and registered in the State of California.

10.15.4 In no case should excavations extend below a 2H to 1V zone below existing utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

10.15.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. Excavation safety is the responsibility of the contractor. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the contractor should take appropriate actions to prevent further damage or injury. It is anticipated that bedding and weak clay layers oriented adversely to cut slopes may be encountered. At a minimum, an engineering geologist should periodically observe cut slopes higher than 5 feet during and after excavating.

10.16 Utility Trenches and Design

10.16.1 Excavations for utilities could encounter rock that will require heavy equipment. The Contractor shall, as part, of the project bidding assess the locations and depths of planned utilities to determine the methods required to excavate, process the excavated materials, and backfill the utilities. No change orders will be allowed for excavation, material processing or backfill of utilities due to rock and subsurface conditions identified or referenced in this report.

10.16.2 Depending on the planned depth of excavation and the weather and groundwater conditions at the time of excavation, construction of underground utilities may encounter groundwater and require dewatering. In general, it is not anticipated that shallow groundwater will be pervasive at the site. However, it is anticipated that shallow groundwater will be encountered during excavations in the areas of natural drainage swales. These conditions, and measures to remove and dispose of water, and stabilize trenches, and should be considered for bidding purposes by the contractor. No change orders will be allowed for dewatering, management of seepage, subsurface flow due to existing conditions or weather conditions.

10.16.3 The design engineers and the contractor should consider buoyant conditions for design and construction of subsurface utilities which are placed in topographically low areas, such as natural drainage swales. Areas susceptible to shallow groundwater should be further delineated during the design level investigations.

10.16.4 Slurry cut-off walls should be incorporated into the design of the utilities to reduce the potential for excessive flow and gradients in the utility trenches. It is recommended the spacing and location of the cut-off walls be determined by the civil engineer or appropriate designer of each utility, as applicable.

10.16.5 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable the contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 92 percent relative compaction prior to placement of bedding material. The contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of pipe and backfill of the trench.

10.16.6 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The Contractor is responsible for contacting the governing agency and pipe manufacturer to determine the requirements for pipe bedding, pipe zone and final backfill. The Contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency, manufacturer and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be approved non-expansive or on-site materials compacted to a minimum of 92 percent relative compaction. All materials should be placed within optimum to three (3) percent above optimum moisture content.

10.16.7 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321-00 listed in Table No. 2 (minimum manufacturer requirements). As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 6
Minimum Trench Widths for HDPE Pipe with Sand Bedding Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321-00
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80
60	67.3	96

10.16.8 Open graded, crushed gravel is not allowed for use as backfill in trenches. Contractors should assume for the purpose of bid that no rock or gravel can be used for backfill on the project including utility trenches of any kind. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to reduce the potential for migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining.

10.16.9 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to within optimum to 3 percent above the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

10.16.10 Trench backfill should be placed in 8 inch lifts, moisture conditioned to within optimum to 3 percent above optimum and compacted to achieve the minimum relative compaction.

10.16.11 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

10.16.12 Jetting of trench backfill is not recommended to compact the backfill soils.

10.16.13 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

10.16.14 Storm drains and/or utility lines should be designed to be watertight. If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines be inspected by video prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are watertight.

10.16.15 The utility trenches for electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 92 percent per ASTM D-1557. This requirement should be noted on the plans.

10.16.16 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

10.16.17 Utility trenches should be designed with cutoff collars to reduce the potential for utility trenches to act as conduit for subsurface flow. The locations of collars should be recommended by Moore Twining after review of the plans.

10.17 Corrosion Protection

This section provides preliminary corrosion recommendations for planning purposes. Final corrosion design should be based on the project grading and building plans and the results of the additional individual design level investigations.

10.17.1 Based on the ASTM Special Technical Publication 741 and the results of laboratory analyses, the soils exhibit a "very corrosive" corrosion potential. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "very corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

10.17.2 A high potential for corrosion of concrete due to sulfate attack is anticipated based on the "severe"concentrations (by dry weight) of sulfates determined for one of the near-surface soil samples. This concentration of sulfates falls in the severe classification (0.20 to 2.00 percent by weight) for concrete. The California Building Code (based on ACI criteria), recommends using Type V cement with a minimum compressive strength of at least 4,500 pounds per square inch for concrete in contact with these soils and that a maximum 0.45 water-cement ratio and pozzolan be used.

10.17.3 The Contractor should provide these soil corrosion data to the manufacturer's or supplier's of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining does not provide corrosion consulting.

11.0 DESIGN CONSULTATION

11.1 Moore Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not a part of this current contractual agreement.

11.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

11.3 If Moore Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review should be documented by a formal plan/specification review report provided by Moore Twining.

12.0 CONSTRUCTION MONITORING

12.1 Moore Twining should be retained to conduct the necessary observation, field-testing services and provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, the geotechnical engineer should provide a written summary of the observations, field-testing and conclusions regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

12.2 In the event that the earthwork operations for this project are conducted prior to the construction of the individual structures such that the construction sequence is not continuous (or if construction operations disturb the surface soils), it is recommended that the exposed subgrade to receive floor slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted as recommended in this report. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

12.3 Compaction tests should be conducted at a frequency of at least:

Area	Minimum Test Frequency
Mass Fills or Subgrade	1 test per 2,500 square feet per compacted lift
Pavement Subgrade	1 test per 5,000 square feet per compacted lift
Utility Lines	1 test per 100 feet per lift

The above testing frequencies are suggested rates for tests. Testing frequency should be adjusted by the field technician and the engineer as needed based on continuous earthwork observation considering the methods used for compaction and the soil conditions.

12.4 The construction monitoring is an integral part of this investigation. This phase of the work provides the geotechnical engineer the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

12.5 If the Moore Twining is not afforded the opportunity to provide engineering observation and field testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. We recommend that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least $3,000,000 and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

12.6 Upon the completion of work, a final report should be prepared by Moore Twining per the requirements of the California Building Code, Chapter 33, "Excavation and Grading," Section 3318.1, "Final Reports." This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify the geotechnical engineer upon the completion of work to provide this report. This service is not, however, part of this current contractual agreement.

13.0 NOTIFICATION AND LIMITATIONS

13.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations.

13.2 The nature and extent of subsurface variations between borings may not become evident until construction.

13.3 This report does not include assessment of liquefaction outside of the proposed home site areas. Some liquefaction would be anticipated in the low alluvial areas of the site (outside the home site areas) as a result of a large seismic event. Liquefaction analyses should be conducted in conjunction with goetechnical/geologic investigation for future development.

13.4 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and the recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

13.5 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

13.6 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

13.7 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in the Site Description portion of this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

13.8 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

13.9 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

13.10 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

13.11 This investigation report should not be used in the preparation of a Storm Water Pollution Prevention Plan (SWPPP). Use of this report or any data included in the report in preparation of a SWPPP would be at the owner's sole risk.

13.12 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Heritage Development. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.

Kenneth J. Clark, CEG
Engnineering Geologist
Geotechnical Engineering Division

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KENNETH JAMES CLARK
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KJC/pc

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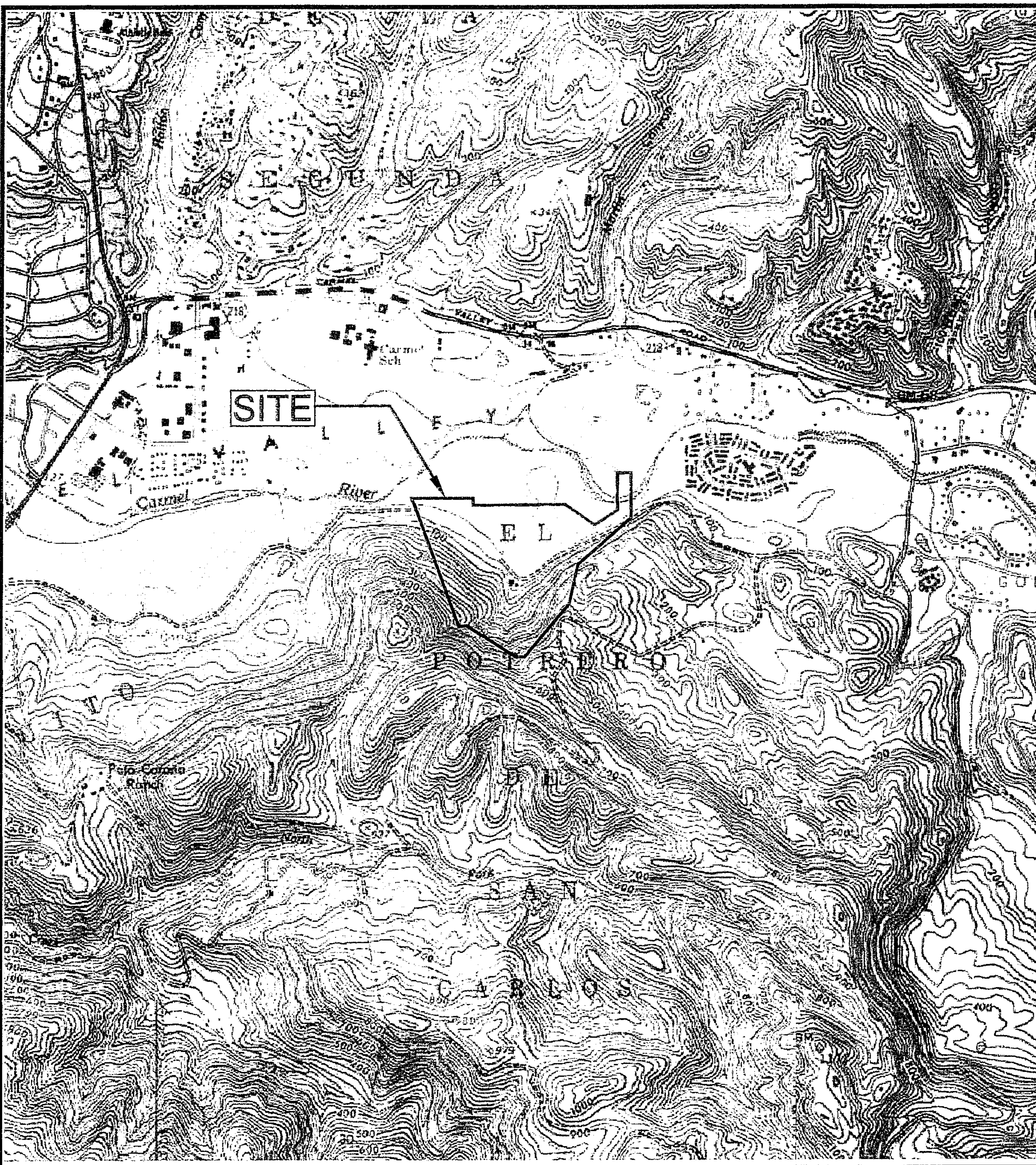
APPENDIX A

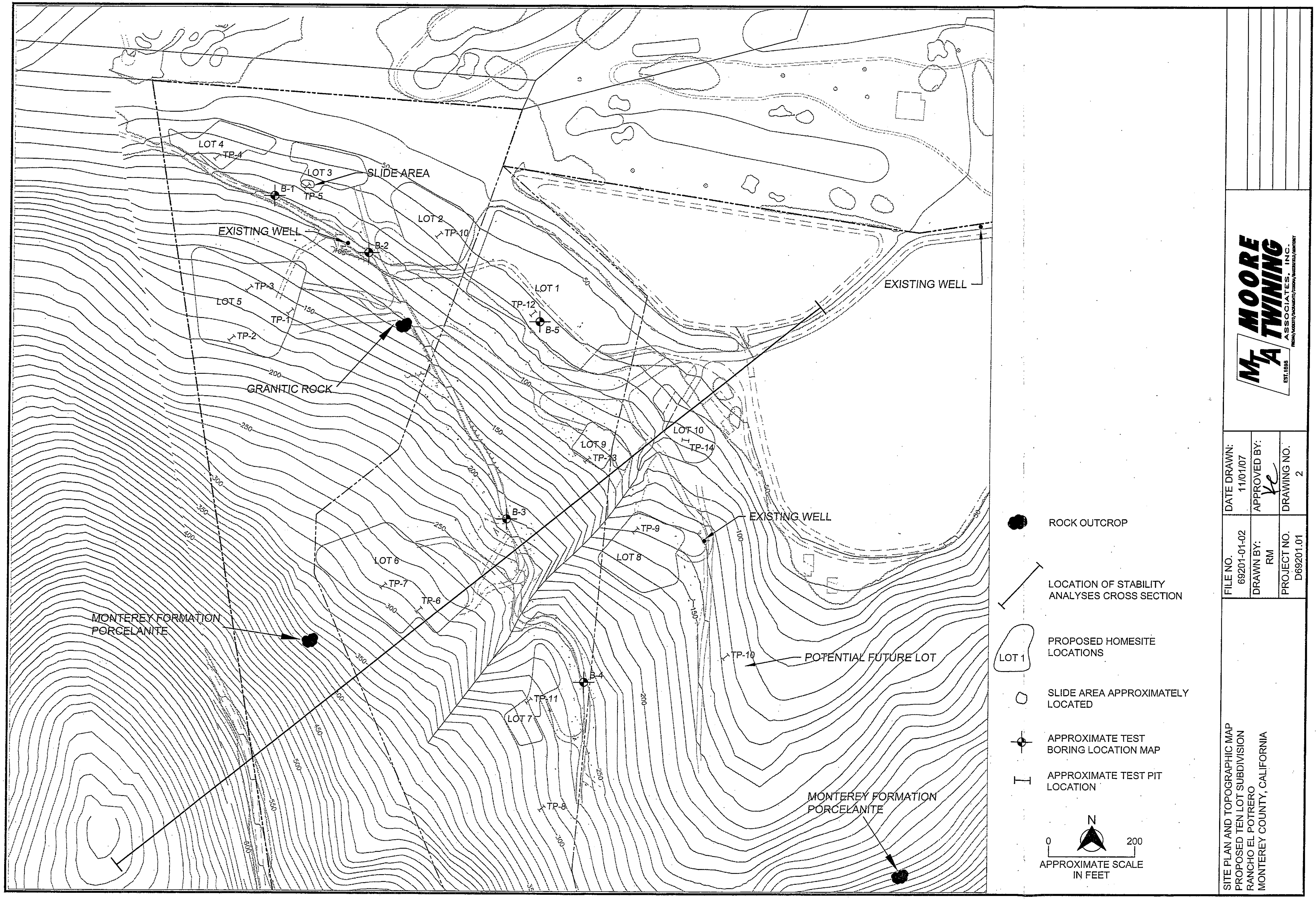
APPENDIX A - DRAWINGS

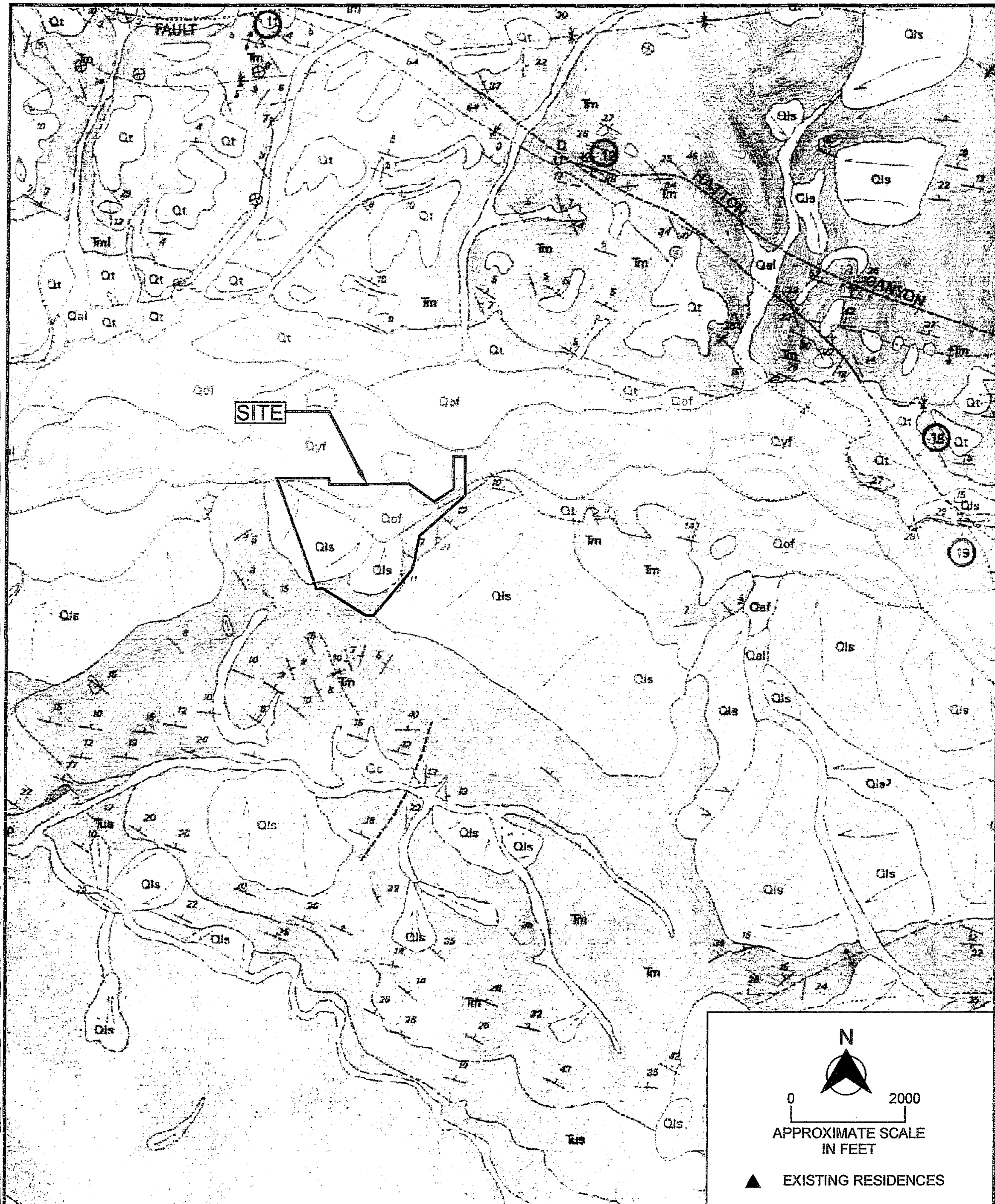


SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
MONTEREY, CALIFORNIA QUADRANGLE, 1971 PHOTOREVISED 1983

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED 10 LOT SUBDIVISION RANCHO EL POTRERO MONTEREY COUNTY, CALIFORNIA	FILE NO.: 69201-01-01	DATE: 11/07/07	MTA MOORE TWINING ASSOCIATES, INC. EST. 1898
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO. 1	


LOT 1
LOT 2
LOT 3
LOT 4
LOT 5
LOT 6
LOT 7
LOT 8
LOT 9
LOT 10
SLIDE AREA
EXISTING WELL
GRANITIC ROCK
MONTEREY FORMATION PORCELANITE
POTENTIAL FUTURE LOT
B-1
B-2
B-3
B-4
B-5
TP-1
TP-2
TP-3
TP-4
TP-5
TP-6
TP-7
TP-8
TP-9
TP-10
TP-11
TP-12
TP-13
TP-14
ROCK OUTCROP
LOCATION OF STABILITY ANALYSES CROSS SECTION
PROPOSED HOMESITE LOCATIONS
SLIDE AREA APPROXIMATELY LOCATED
APPROXIMATE TEST BORING LOCATION MAP
APPROXIMATE TEST PIT LOCATION
N
0
200
APPROXIMATE SCALE IN FEET
MOORE TWINING ASSOCIATES, INC.
EST. 1898
FILE NO. 69201-01-02
DATE DRAWN: 11/01/07
DRAWN BY: RM
APPROVED BY:
PROJECT NO. D69201.01
DRAWING NO. 2
SITE PLAN AND TOPOGRAPHIC MAP
PROPOSED TEN LOT SUBDIVISION
RANCHO EL POTRERO
MONTEREY COUNTY, CALIFORNIA



REGIONAL GEOLOGIC MAP
PROPOSED 10 LOT SUBDIVISION
RANCHO EL POTRERO
MONTEREY COUNTY, CALIFORNIA

REFERENCE BASE MAP: GEOLOGIC MAP OF THE MONTEREY AND SEASIDE 7.5-MINUTE QUADRANGLES, CALIFORNIA" (CLARK, DUPRE, AND ROSENBERG, 1997)

FILE NO. 69201-01-03	DATE DRAWN: 11/20/07
DRAWN BY: RM	APPROVED BY: ILe
PROJECT NO. D69201.01	DRAWING NO. 3

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FIGURE 3 - REGIONAL GEOLOGIC MAP (CONTINUED)

DESCRIPTION OF MAP UNITS

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

Qbs **Beach sand deposits (Holocene)**—Unconsolidated, well-sorted, medium- to coarse-grained sand; local layers of pebbles and cobbles

Qs **Marine sand deposits (Holocene)**—Unconsolidated, gray to buff, fine- to coarse-grained sand on sea floor

Qd **Dune sand deposits (Holocene)**—Unconsolidated, well-sorted, fine-to medium-grained sand; deposited as linear strip of coastal dunes

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. Near mouth of Carmel River, these occur as a veneer of levee deposits over older flood-plain deposits, indicated by a subscript (a) following symbol.

Qof **Older flood-plain deposits (Holocene)**—Unconsolidated, relatively fine-grained, heterogeneous deposits of sand and silt, commonly includes relatively thin layers of clay

Qc **Colluvium (Holocene)**—Unconsolidated, heterogeneous deposits of moderately to poorly sorted silt, sand, and gravel deposited by slope wash and mass movement

Qfd **Flandrian dune deposits of Cooper (1967) (Holocene)**—Unconsolidated, well-sorted sand deposited in a belt of parabolic dunes

Qls **Landslide deposits (Quaternary)**—Heterogeneous mixture of deposits ranging from large block slides in indurated bedrock to debris flows in semiconsolidated sand and clay

Qod **Older coastal dunes (Pleistocene)**—Weakly consolidated, well-sorted, fine-to medium-grained sand. Some geologic deposits are covered with a thin veneer of eolian deposits. In some areas, this is indicated by a subscript (e) following the symbol for the geologic unit overlain by the eolian deposits. Locally divided into:

- **Qod_1** **Younger dune deposits (Pleistocene)**—Weakly consolidated, well-sorted, fine- to medium-grained sand deposited in an extensive coastal dune field. Age of unit is middle(?) Wisconsinan

- **Qod_2** **Older dune deposits (Pleistocene)**—Weakly to moderately consolidated, moderately well-sorted silt and sand deposited in extensive coastal dune fields. Age of unit is early(?) Wisconsinan

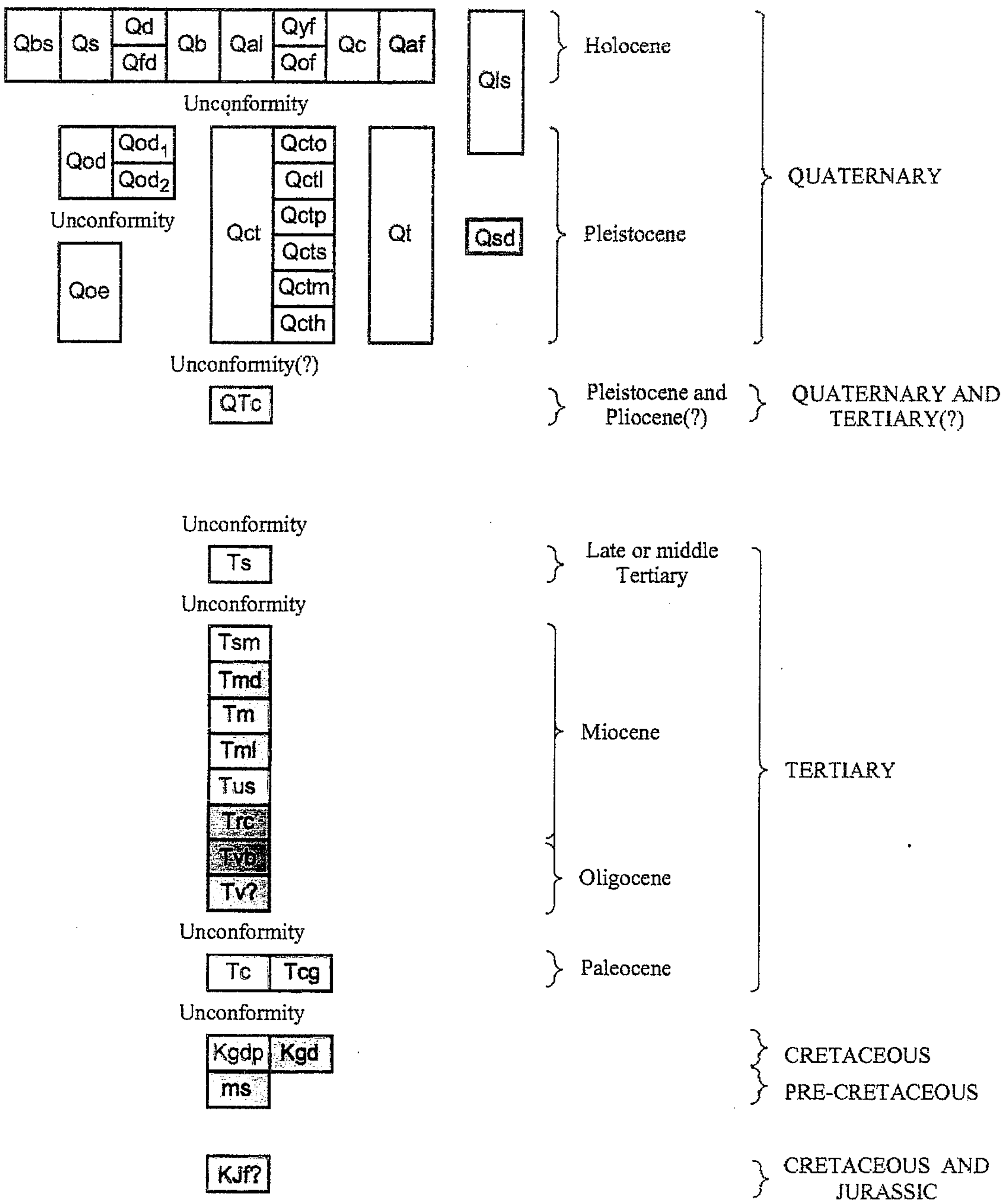
Qct **Coastal terrace deposits, undivided (Pleistocene)**—Semiconsolidated, moderately well-sorted marine sand containing thin, discontinuous gravel-rich layers. Locally divided into:

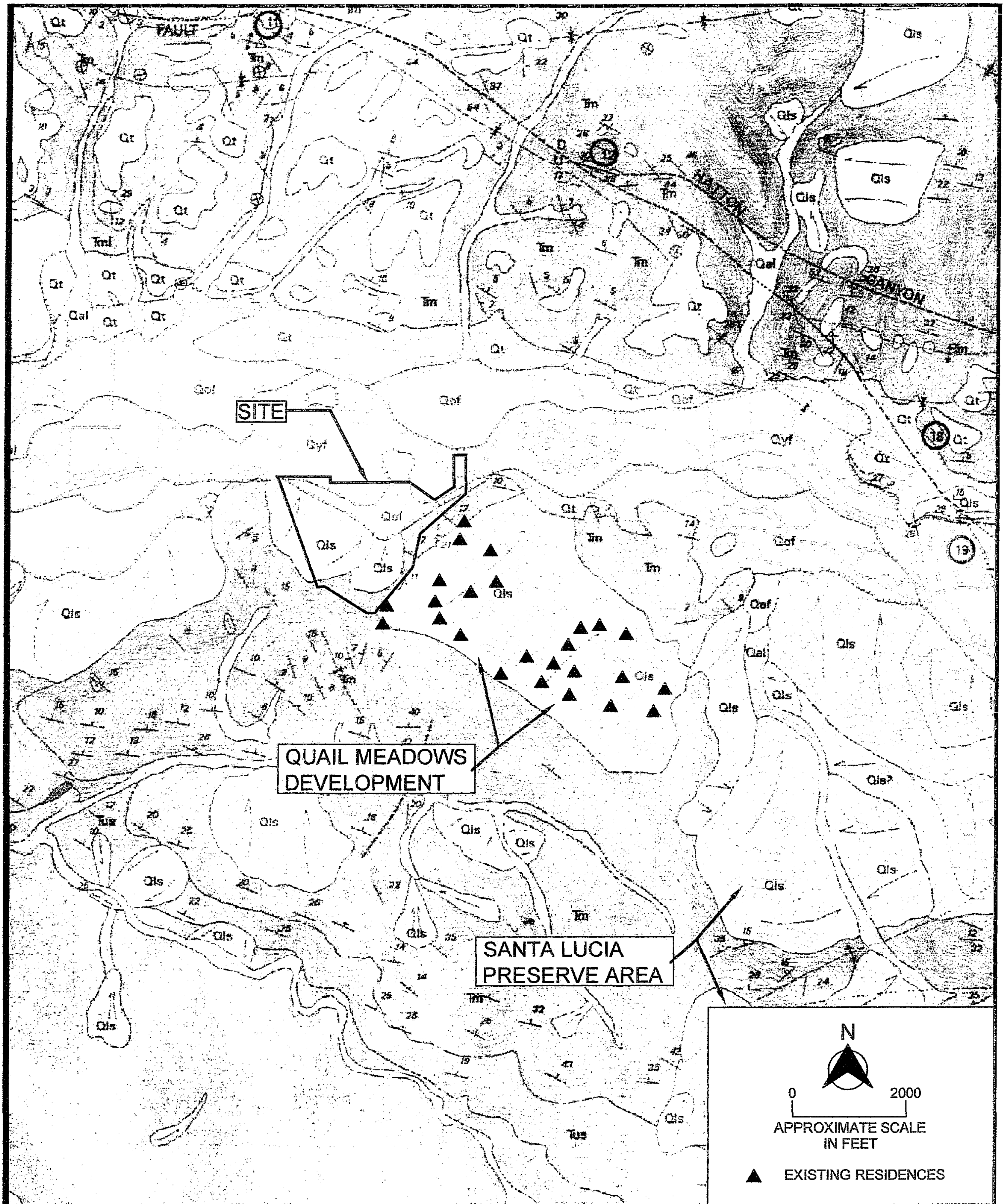
- **Qcto** **Ocean View coastal terrace (Pleistocene)**
- **Qctl** **Lighthouse coastal terrace (Pleistocene)**
- **Qctp** **Peninsula College coastal terrace (Pleistocene)**
- **Qcts** **Sylvan coastal terrace (Pleistocene)**

Qctm	**Monte Vista coastal terrace (Pleistocene)**
Qcth	**Huckleberry coastal terrace (Pleistocene)**
Qt	**Terrace deposits, undivided (Pleistocene)**—Weakly consolidated to semiconsolidated, moderately to poorly sorted silt, silty clay, sand, and gravel mostly deposited in a fluvial environment
Qoe	**Older eolian deposits (Pleistocene)**—Moderately well-sorted sand as much as 60 m thick that contains no intervening fluvial deposits
Qsd	**Sedimentary deposits (Quaternary)**—Seismic characteristics suggest poorly bedded sand and gravel; stratigraphic position unknown. Unit crops out on sea floor
QTc	**Continental deposits, undivided (Pleistocene-Pliocene?)**—Semiconsolidated, relatively fine-grained, oxidized sand and silt; includes some deposits of marine origin (locally mapped as QTm)
Ts	**Sedimentary rocks (Tertiary)**—Marine; mudstone and coarse-grained, arkosic sandstone. Unit crops out on sea floor
Tsm	**Santa Margarita Sandstone (Miocene)**—Marine and brackish-marine, white, friable, fine- to coarse-grained, arkosic sandstone. Age of unit is 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
Tml	**Monterey Formation, semi-siliceous mudstone (Miocene)**—Thin-bedded, yellowish-brown, foraminiferal; includes interbedded siltstone; Luisian 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
Trc	**Red Beds Of Robinson Canyon (Miocene)**—Terrestrial; red to gray, poorly sorted arkosic sandstone, cobble conglomerate, and siltstone. Age of unit is probably middle Miocene
Tvb	**Volcanic rocks (Oligocene)**—Flows and flow-breccias of basaltic andesite
Tv?	**Vaqueros(?) Sandstone (Oligocene)**—Marine; yellowish-gray, thick-bedded arkosic sandstone
Tc	**Carmelo Formation of Bowen (1965) (Paleocene)**—Marine; thin- to thick-bedded and graded arkosic sandstone with interbedded siltstone and pebble and cobble conglomerate. Locally divided into:
Tcg	**Cobble and boulder conglomerate (Paleocene)**—Consists mostly of porphyritic granodiorite clasts
Kgdp	**Porphyritic granodiorite of Monterey of Ross (1976) (Cretaceous)**
Kgd	**Granodiorite of Cachagua of Ross (1976) (Cretaceous)**
ms	**Schist of the Sierra de Salinas of Ross (1976) (pre-Cretaceous)**—Quartzofeldspathic schist
KJf?	**Franciscan Complex, undifferentiated**—Unit crops out on sea floor

FIGURE 3 - REGIONAL GEOLOGIC MAP (CONTINUED)

CORRELATION OF MAP UNITS

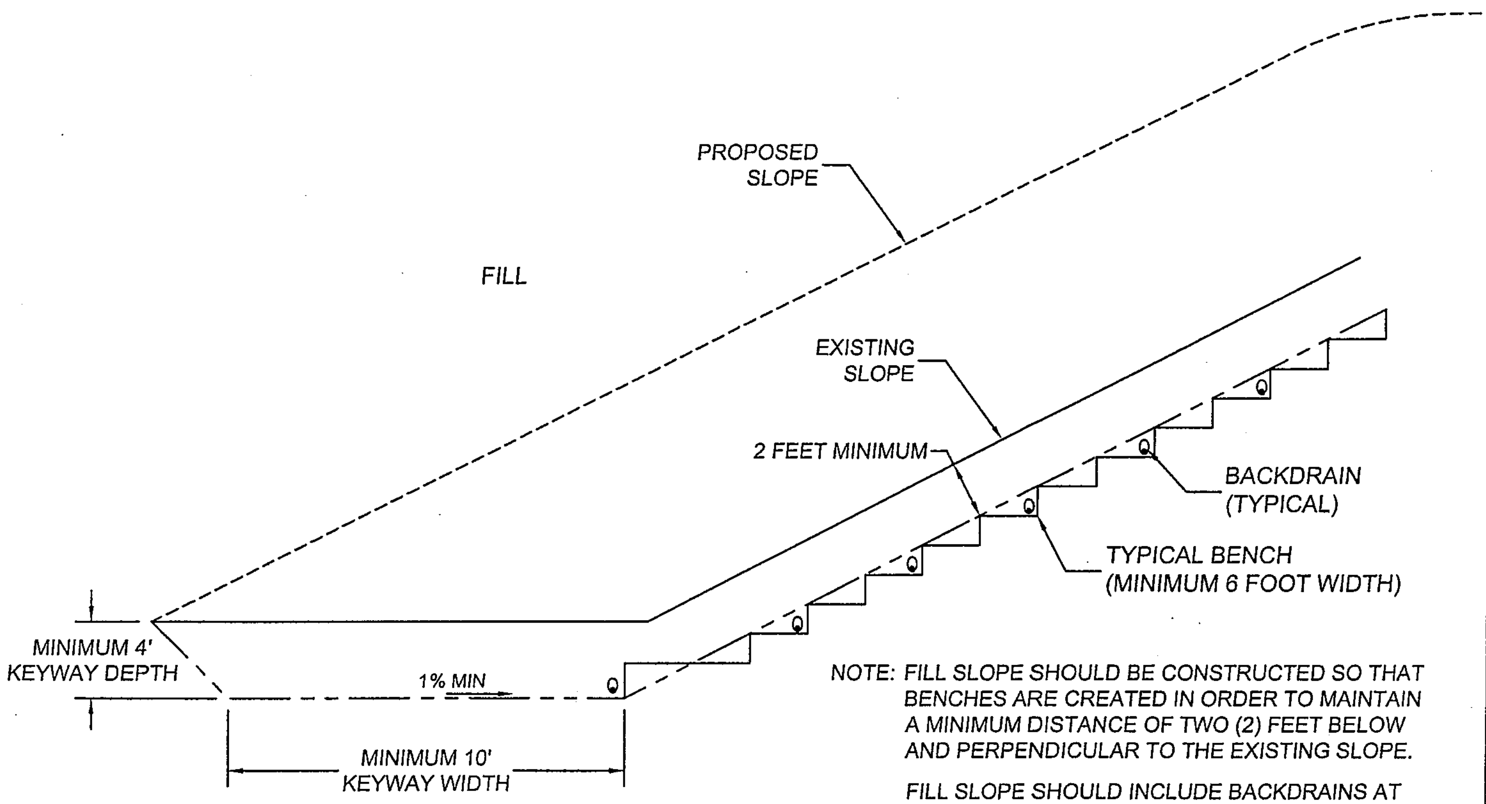




RESIDENCES LOCATED ON MAPPED LANDSLIDE DEPOSIT WITHIN APPROXIMATELY 1 MILE EAST OF PROPOSED RANCHO EL POTRERO SUBDIVISION	FILE NO. 69201-01-04	DATE DRAWN: 11/07/07	MOORE TWINING ASSOCIATES, INC. EST. 1998
	DRAWN BY: RM	APPROVED BY: KC	
REFERENCE BASE MAP: GEOLOGIC MAP OF THE MONTEREY AND SEASIDE 7.5-MINUTE QUADRANGLES, CALIFORNIA" (CLARK, DUPRE, AND ROSENBERG, 1997)	PROJECT NO. D69201.01	DRAWING NO. 4	

TYPICAL KEYWAY AND FILL SLOPE BENCHING AND SUBDRAINAGE

SCALE: 1' = 10'

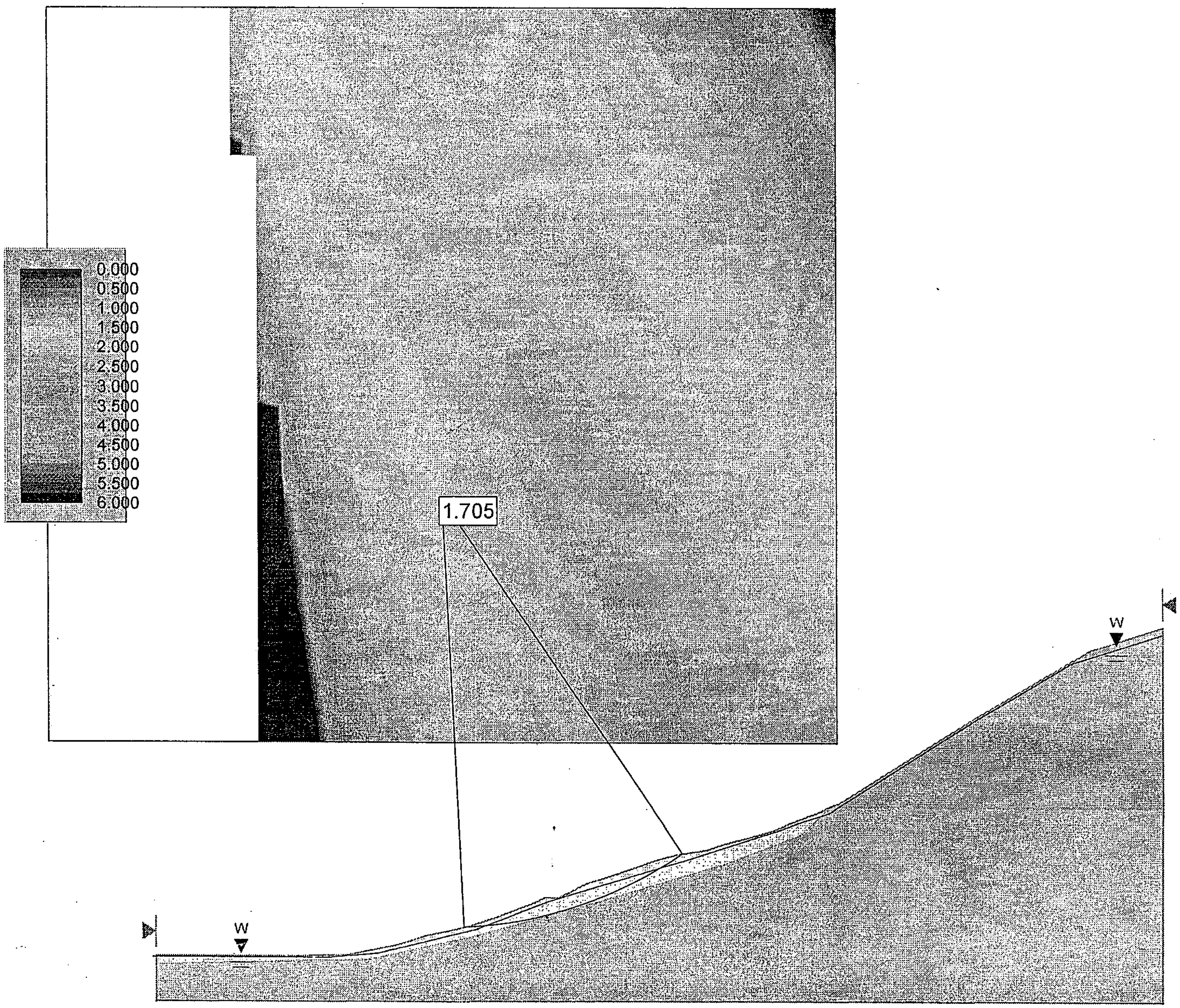


NOTE: FILL SLOPE SHOULD BE CONSTRUCTED SO THAT BENCHES ARE CREATED IN ORDER TO MAINTAIN A MINIMUM DISTANCE OF TWO (2) FEET BELOW AND PERPENDICULAR TO THE EXISTING SLOPE.

FILL SLOPE SHOULD INCLUDE BACKDRAINS AT THE BACK OF THE KEYWAY, AND AT EVERY OTHER BENCH. KEYWAY EXCAVATION SHALL BE EXCAVATED TO SLOPE INTO THE UPSLOPE DIRECTION AT A MINIMUM OF 1%.

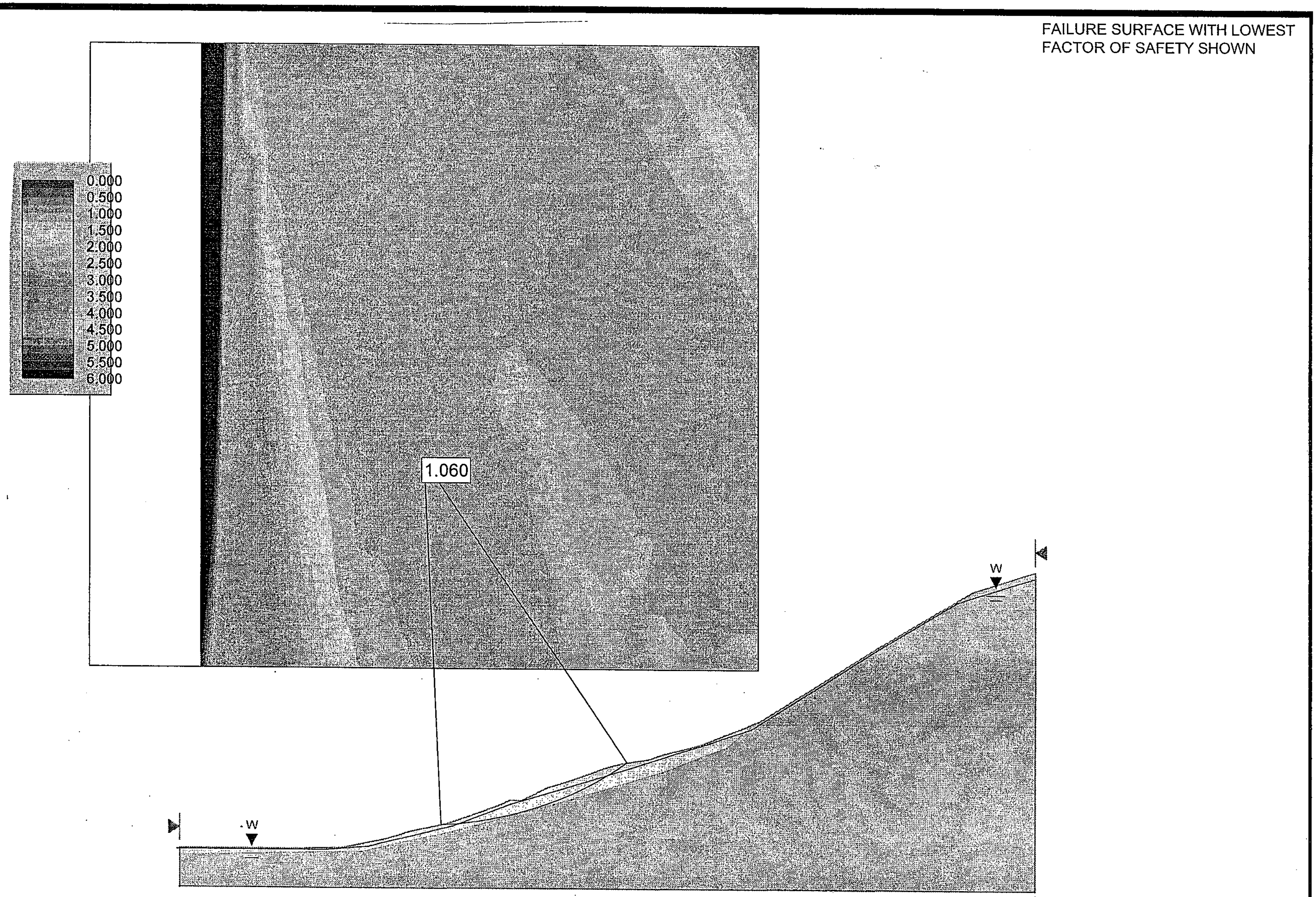
TYPICAL KEYWAY AND FILL SLOPE BENCHING AND SUBDRAINAGE PROPOSED RANCHO EL POTRERO MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-05	DATE DRAWN: 11/20/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: [signature]	
	PROJECT NO. D69201.01	DRAWING NO. 5	

FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - CIRCULAR SLIP FAILURE SURFACES - STATIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 5A

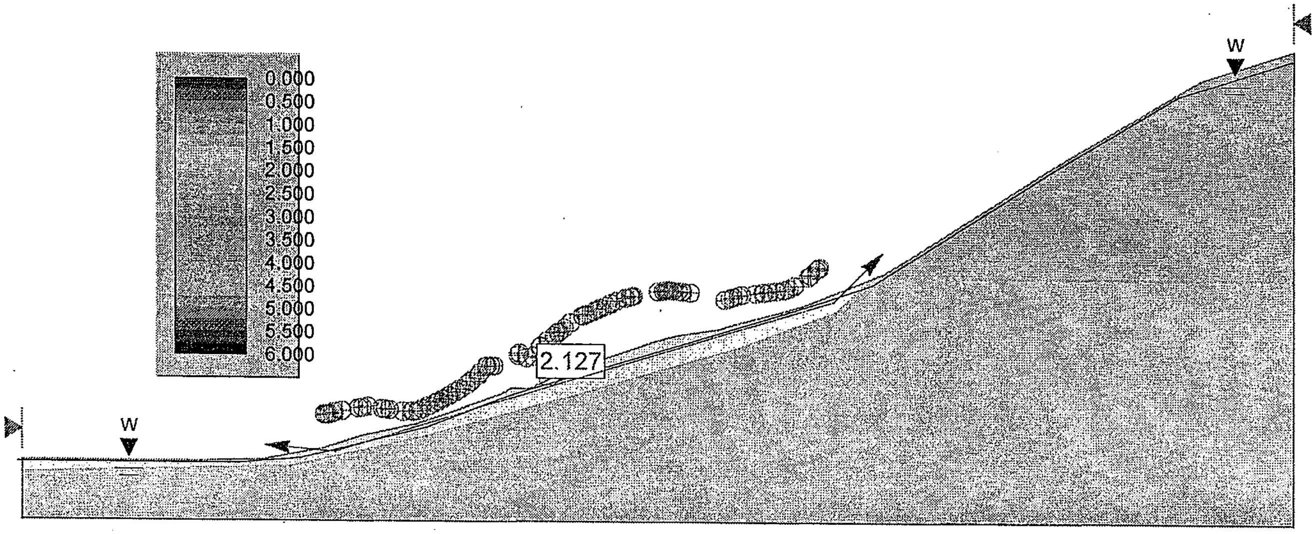
MOORE TWINING ASSOCIATES, INC.
EST. 1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - CIRCULAR SLIP FAILURE SURFACES - SEISMIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 5B

MOORE TWINING ASSOCIATES, INC.
EST.1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

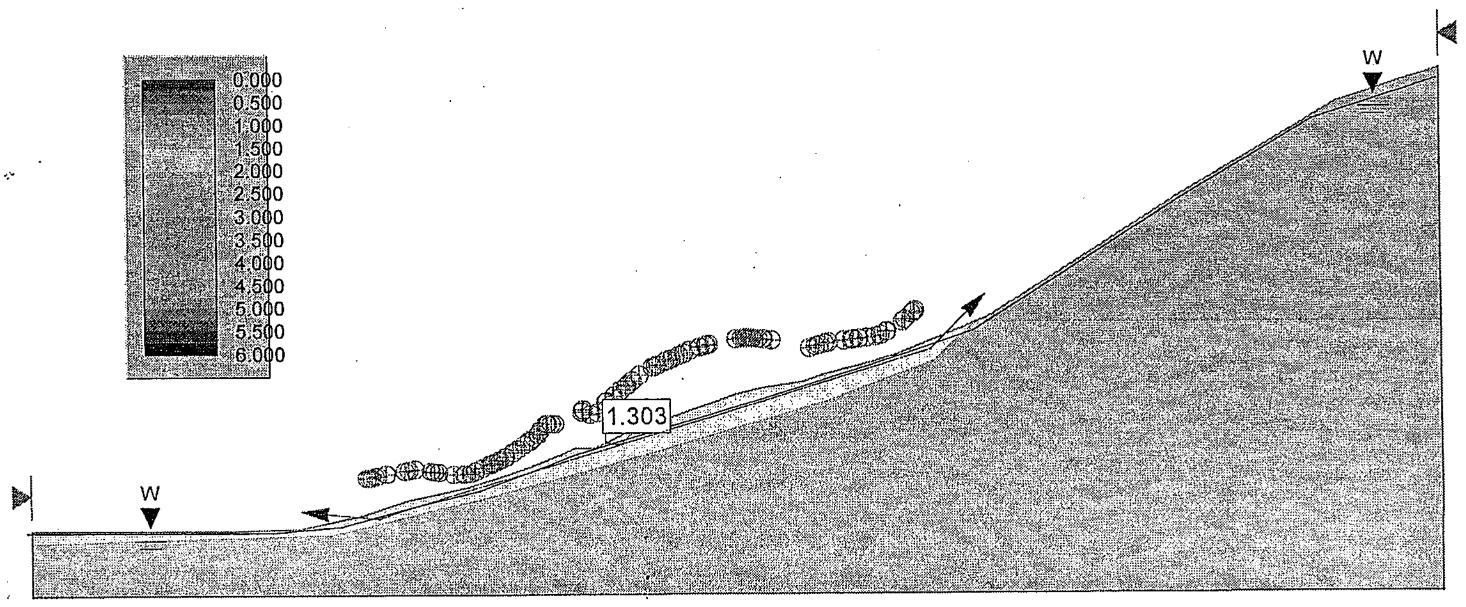
FAILURE SURFACE WITH LOWEST FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - SHALLOW TRANSLATIONAL SLIP FAILURE SURFACES - STATIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 6A

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FAILURE SURFACE WITH LOWEST FACTOR OF SAFETY SHOWN

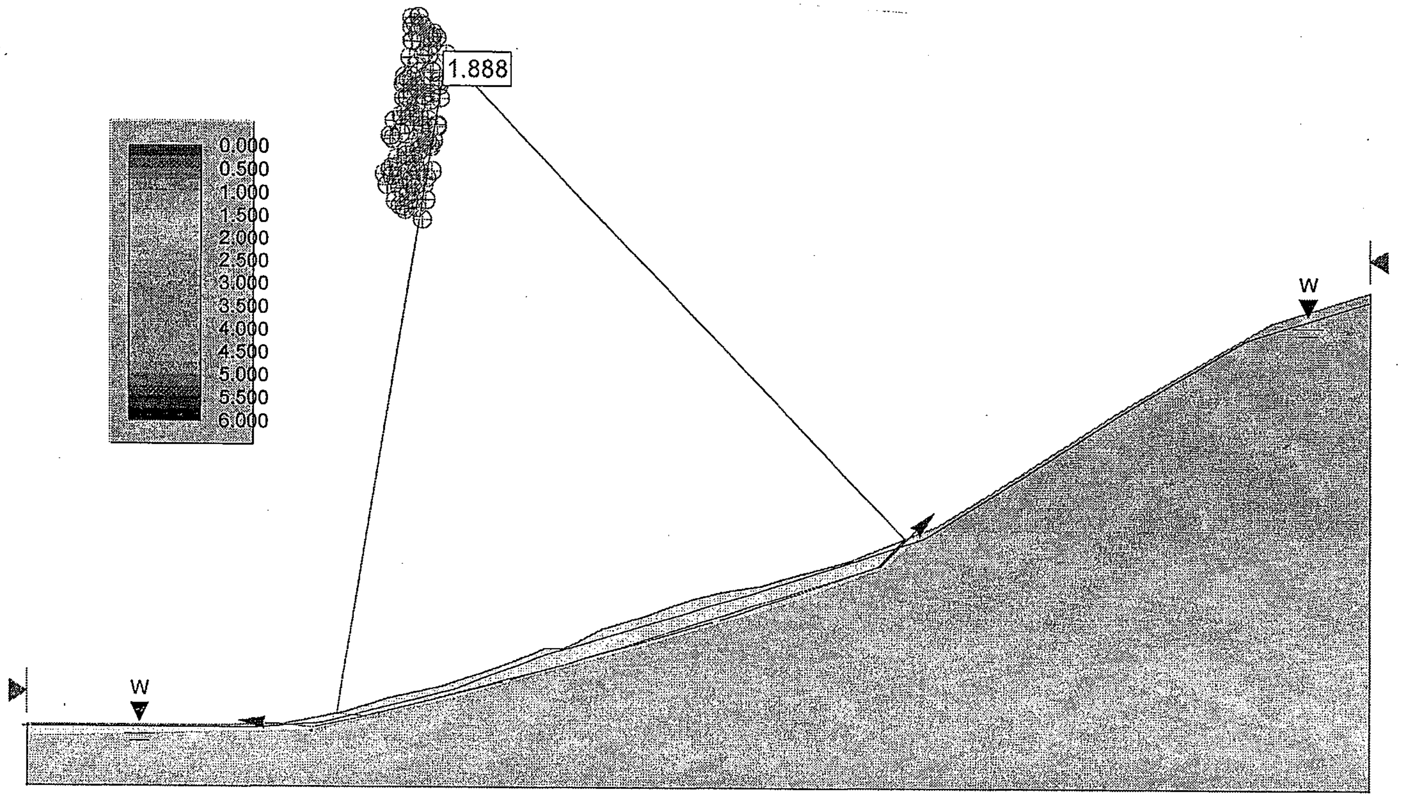


RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - SHALLOW TRANSLATIONAL SLIP FAILURE SURFACES - SEISMIC CONDITION

FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
DRAWN BY: RM	APPROVED BY: KC
PROJECT NO. D69201.01	DRAWING NO. 6B

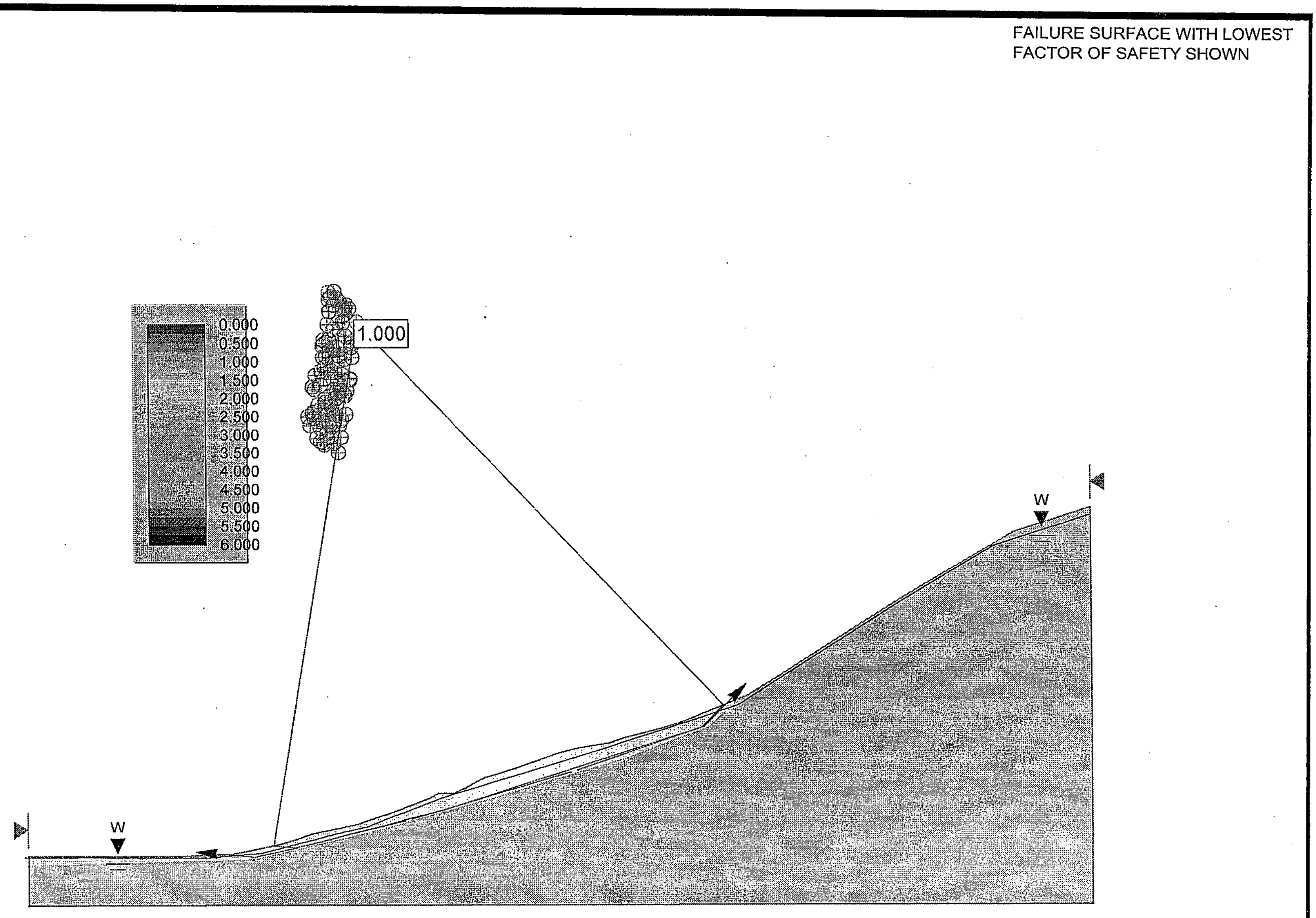
MOORE TWINING ASSOCIATES, INC.
EST. 1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - DEEP TRANSLATIONAL SLIP FAILURE SURFACES - STATIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08	
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO. 7A	

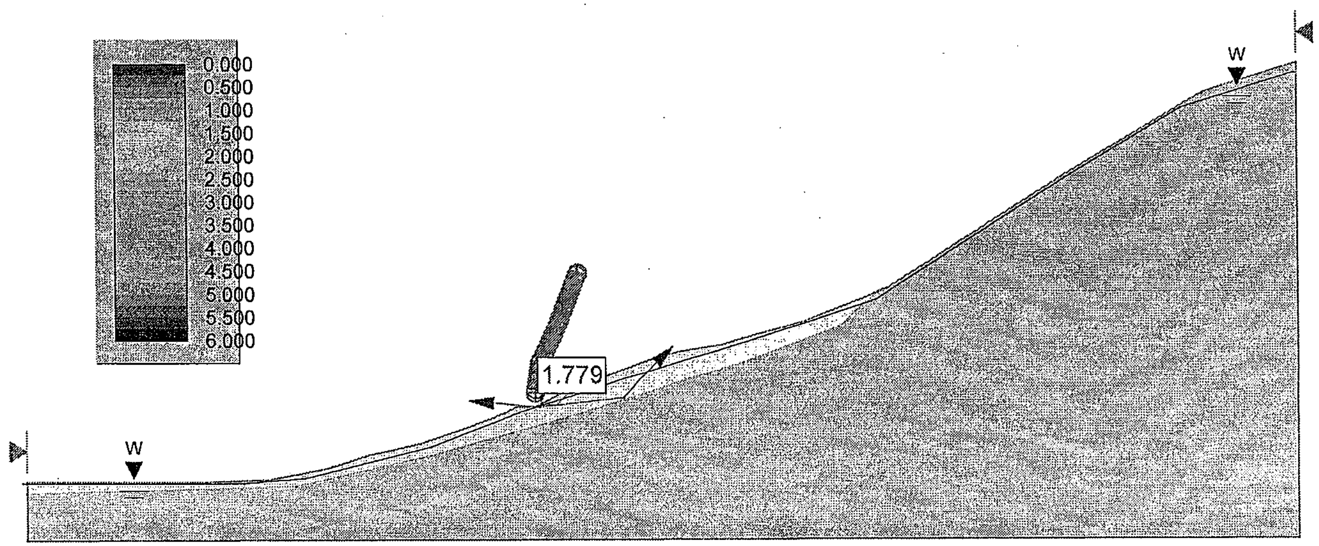
MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - DEEP TRANSLATIONAL SLIP FAILURE SURFACES - SEISMIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. D69201.01	DRAWING NO. 7B

MOORE TWINING ASSOCIATES, INC.
EST.1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN

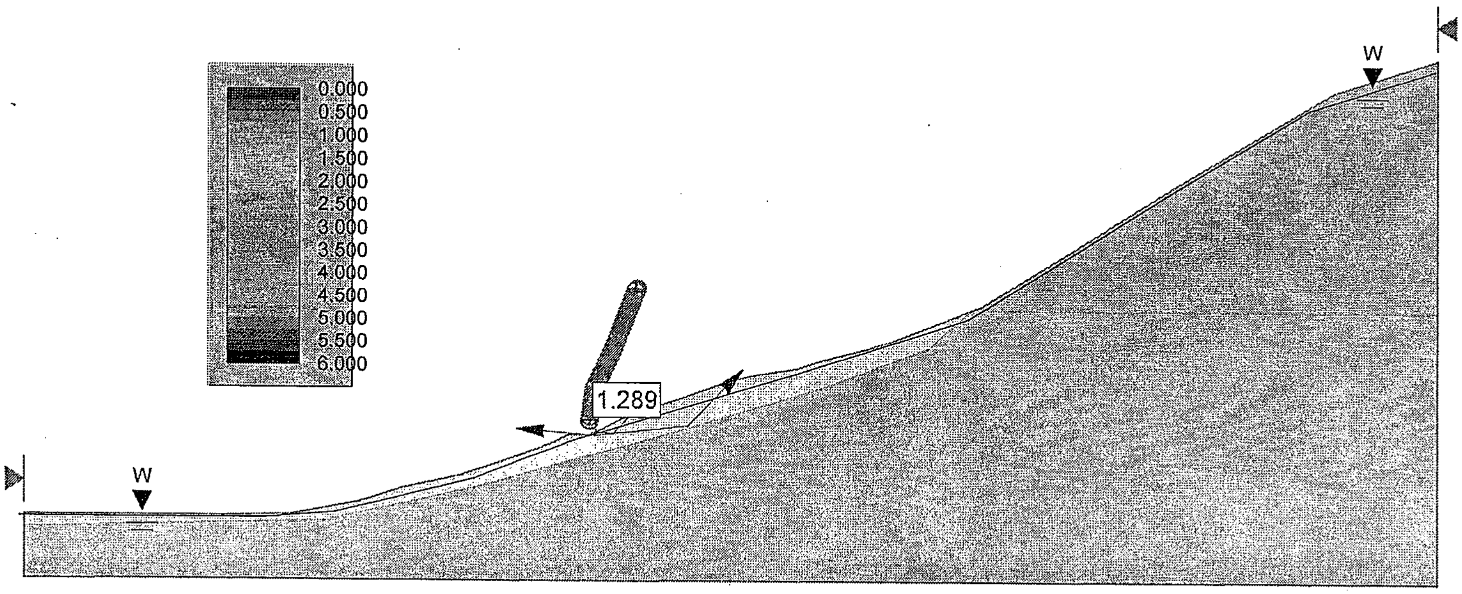


RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES -
ADVERSE SLIP PLAN ORIENTATION 1 - STATIC CONDITION

FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
DRAWN BY: RM	APPROVED BY:
PROJECT NO. D69201.01	DRAWING NO. 8A

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

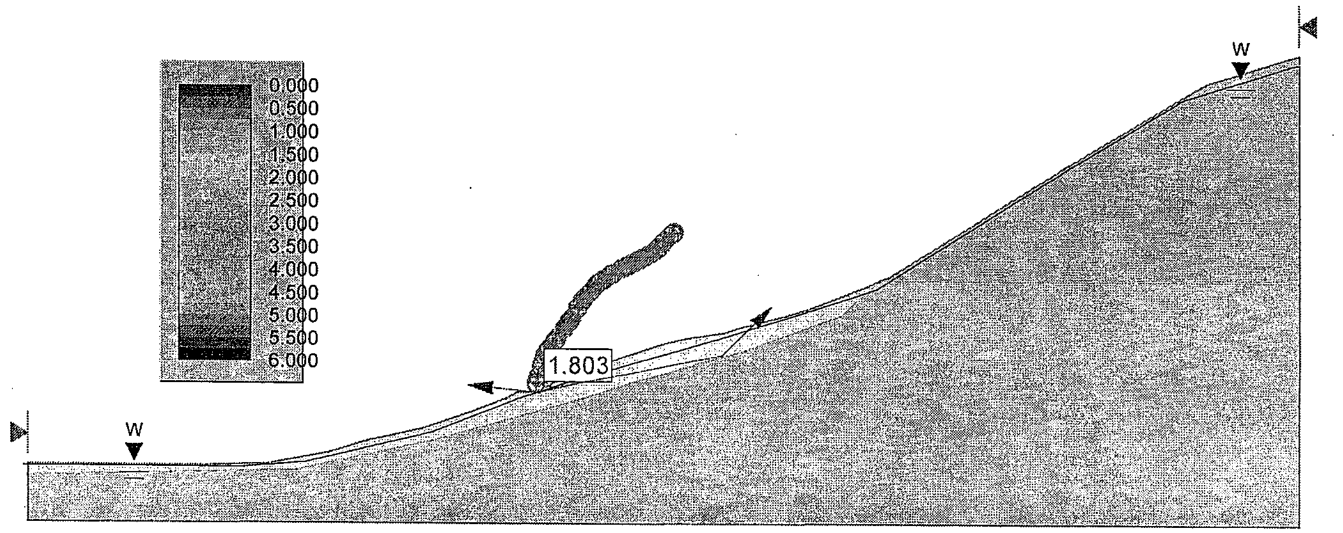
FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - ADVERSE SLIP PLANE ORIENTATION 1 - SEISMIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 8B

MOORE TWINING ASSOCIATES, INC.
EST.1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FAILURE SURFACE WITH LOWEST FACTOR OF SAFETY SHOWN

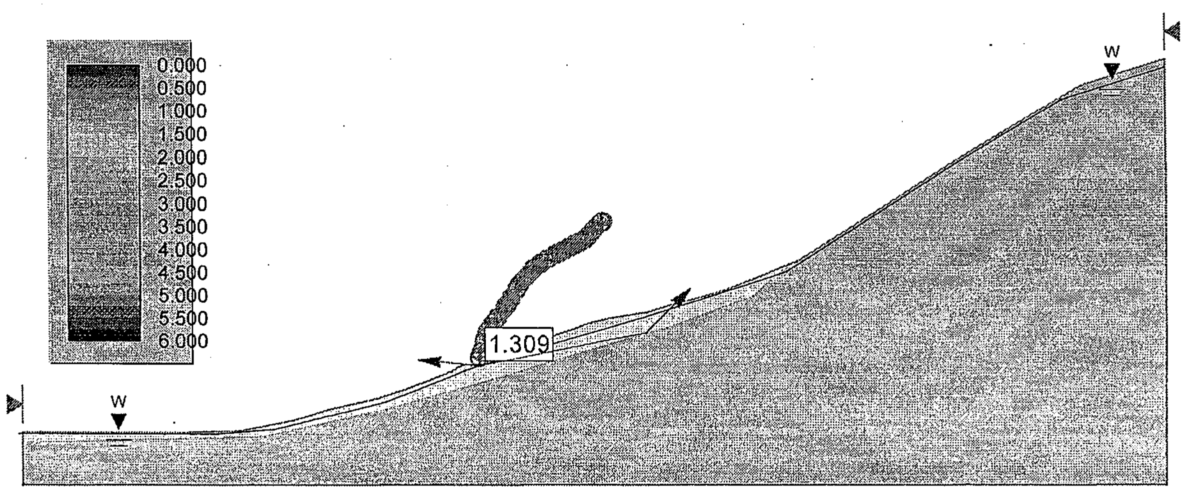


RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - ADVERSE SLIP PLANE ORIENTATION 2 - STATIC CONDITION

FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
DRAWN BY: RM	APPROVED BY: KE
PROJECT NO. D69201.01	DRAWING NO. 9A

MTA EST. 1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

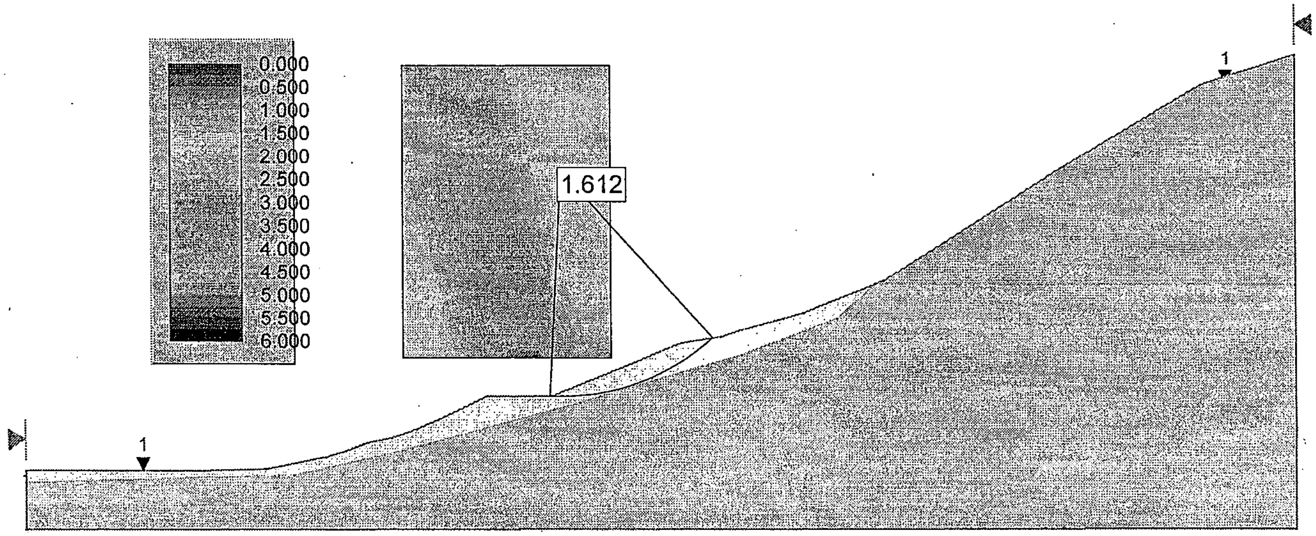
FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES - NATIVE SITE SLOPES - ADVERSE SLIP PLANE ORIENTATION 2 - SEISMIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 9B

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

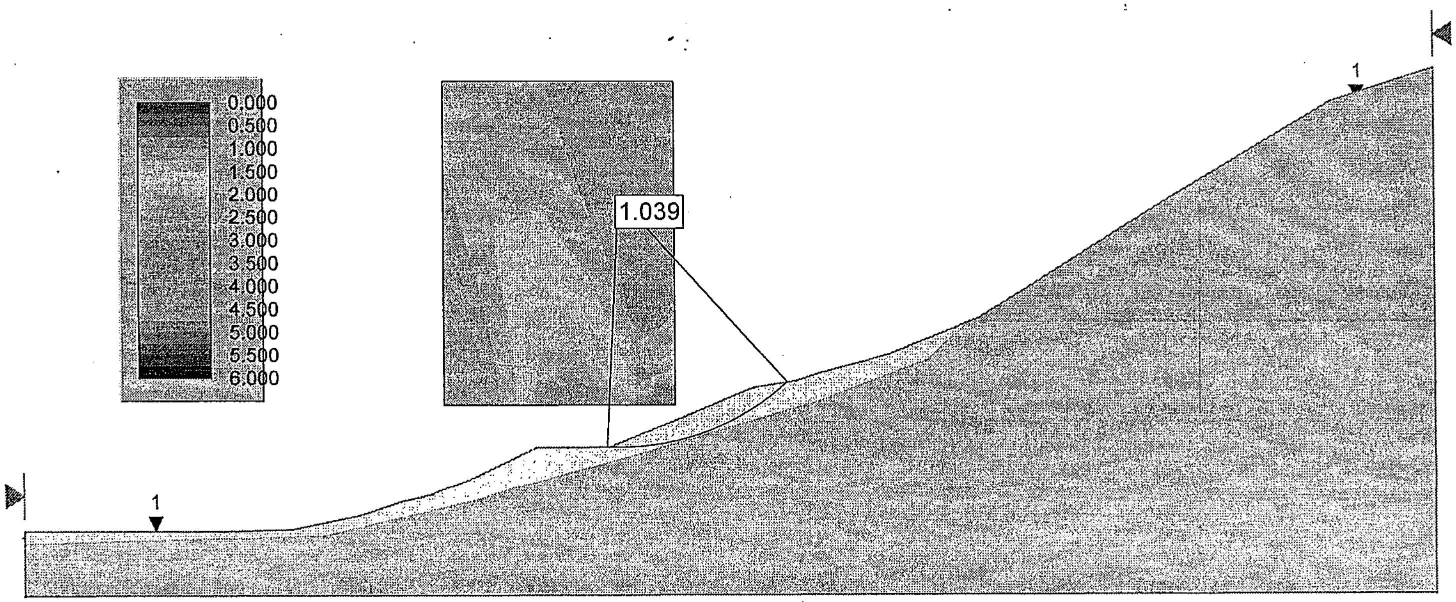
FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES FOR PROPOSED 2½H TO 1V CUT SLOPES - STATIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KE
	PROJECT NO. D69201.01	DRAWING NO. 10A

MTA MOORE TWINING ASSOCIATES, INC.
EST.1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

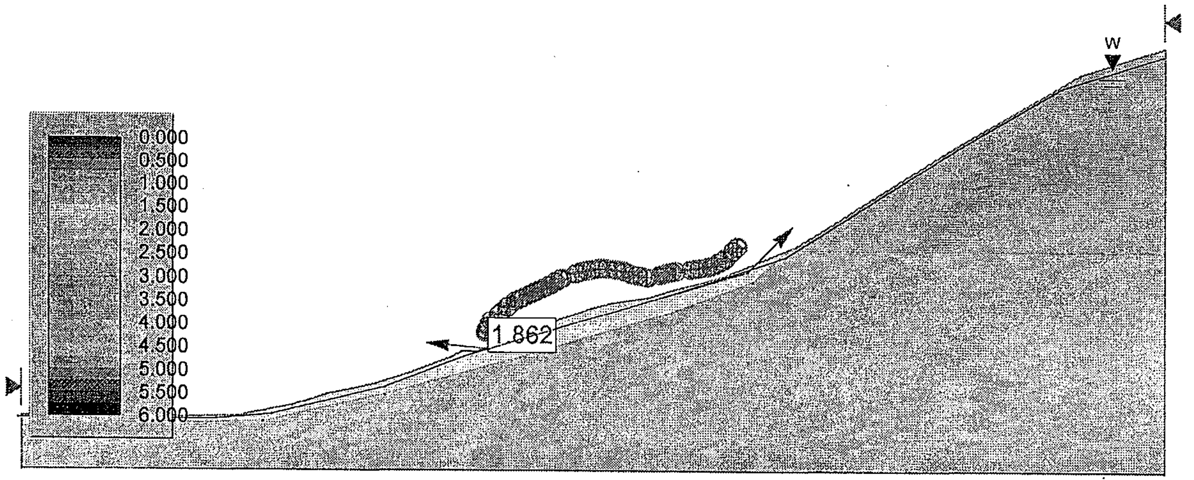
FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES FOR PROPOSED 2½H TO 1V CUT SLOPES - SEISMIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. D69201.01	DRAWING NO. 10B

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

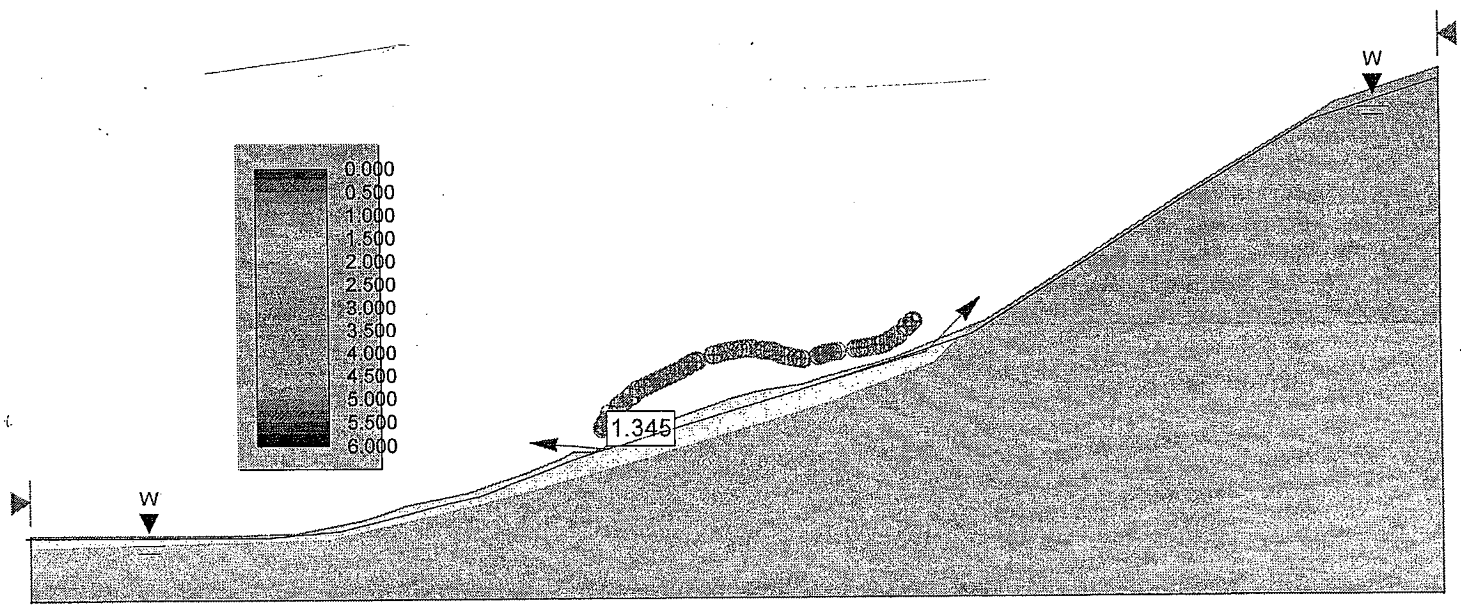
FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES FOR PROPOSED 2½H TO 1V CUT SLOPES - TRANSLATIONAL SLIP FAILURE SURFACES - STATIC CONDITION	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: [signature]
	PROJECT NO. D69201.01	DRAWING NO. 10C

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

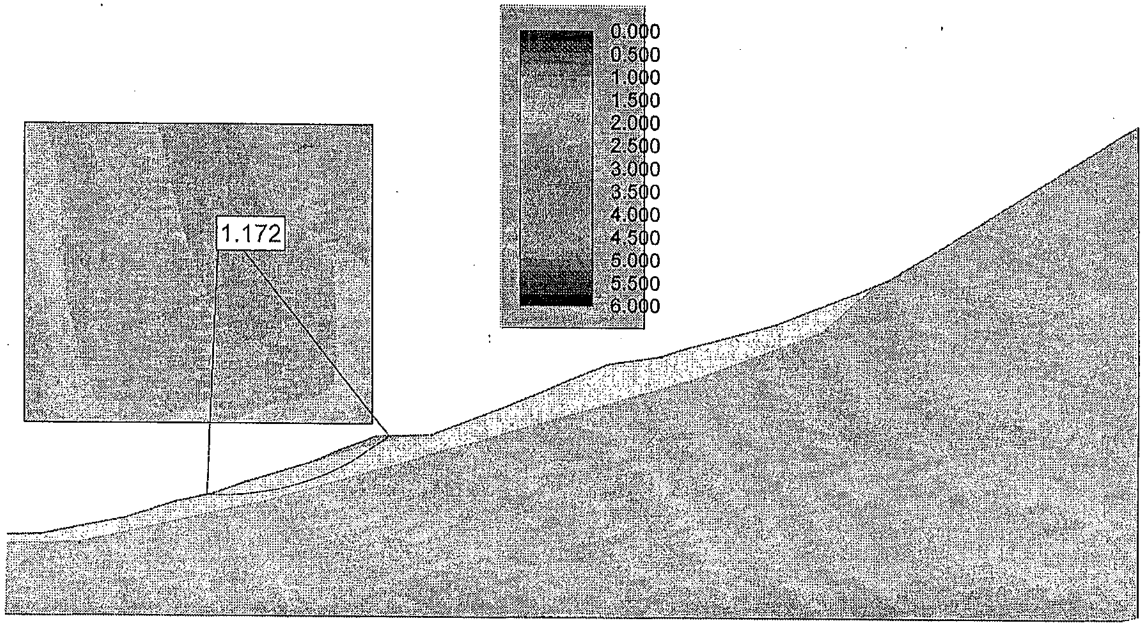
FAILURE SURFACE WITH LOWEST FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES FOR PROPOSED 2½H TO 1V CUT SLOPES - TRANSLATIONAL SLIP FAILURE SURFACES - SEISMIC CONDITIONS	FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
	DRAWN BY: RM	APPROVED BY: KC
	PROJECT NO. D69201.01	DRAWING NO. 10D

MOORE TWINING ASSOCIATES, INC.
EST.1898
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

FAILURE SURFACE WITH LOWEST
FACTOR OF SAFETY SHOWN



RESULTS OF SLOPE STABILITY ANALYSES FOR PROPOSED 2½H TO 1V FILL SLOPES - SEISMIC CONDITION

FILE NO. 69201-01-X	DATE DRAWN: 01/24/08
DRAWN BY: RM	APPROVED BY: Ke
PROJECT NO. D69201.01	DRAWING NO. 11B

MTA EST.1898 MOORE TWINING ASSOCIATES, INC.
FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY

APPENDIX B

APPENDIX B

LOG OF TEST BORINGS AND TEST PITS

This appendix contains the final logs of borings and test pits. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Jeff Taylor Property

Location: Carmel, CA

Logged By: K.C.

Drilled By: J.R.

Drill Type: CME 75

Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01

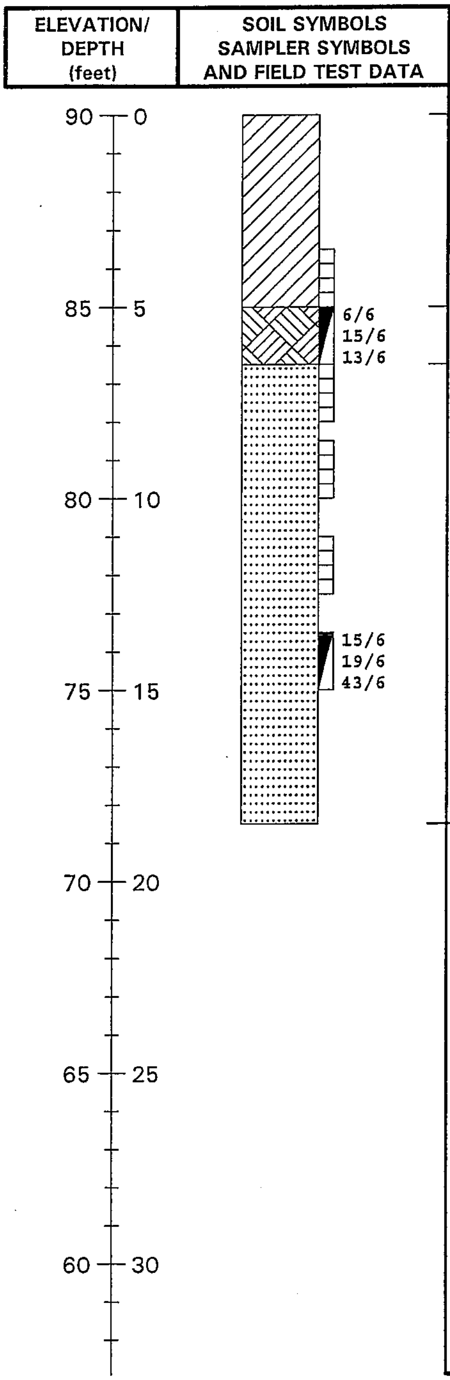
Date: 5/24/07

Elevation: 90 feet AMSL

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: TRIP



USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
CL	LEAN CLAY, Silty; Organics, light gray			
		DD = 106.7 pcf	--	17
ROCK	SANDSTONE; Yellow, medium dense, damp		28	16
			--	10
	Rock Granodiorite, moderately weathered		--	
			--	10
			62	10
	Sample refusal at 18.5 feet			

Notes: *Drilled on existing unimproved (dirt) road.*

Figure Number B-1

SOIL TEST BORING SYMBOLIC LOG

MOORE TWINING ASSOCIATES, INC. EST. 1898

BORING B-2

Project: Jeff Taylor Property

Location: Carmel, CA

Logged By: K.C.

Drilled By: S.R.

Drill Type: CME 75

Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01

Date: 5/24/07

Elevation: Approx. 98 feet AMSL

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
0		CL	LEAN CLAY, Silty; Chaotic landslide material with angular sandstone pebbles			
				DD = 100 pcf	--	11
5	15/6 25/6 30/6	ROCK	Sandstone; Dense, yellow to brown, damp	PI = 2 LL = 21	55	
			Sandstone		--	17
10	12/6 14/6 17/6				31	
	24/6 27/6 45/6	ROCK	Granodiorite, highly weathered		72	
15						
			Auger Refusal at 16 feet			
20						
25						
30						

Notes: *Drilled on existing unimproved (dirt) road.*

Figure Number B-2

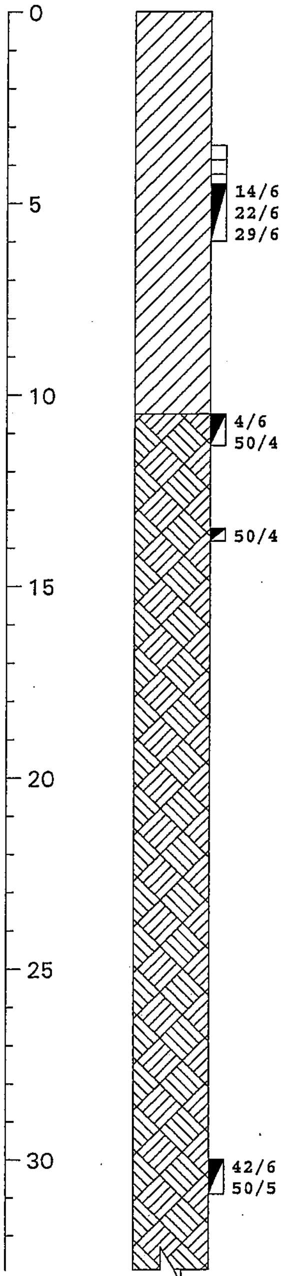
SOIL TEST BORING SYMBOLIC LOG

MOORE TWINING ASSOCIATES, INC. EST. 1898

BORING B-3

Project: Jeff Taylor Property
Location: Carmel, CA
Logged By: K.C.
Drilled By: S.R.
Drill Type: CME 75
Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01
Date: 5/24/07
Elevation: Approx. 212 feet AMSL
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: TRIP



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
0		CL	LEAN CLAY; Chaotic landslide deposit			
				DD = 77 pcf	--	28
5	14/6 22/6 29/6				51	
			Increase in drilling resistance			
10	4/6 50/4	ROCK	Granodiorite, moderately weathered		>50	33
	50/4				>50	6
30	42/6 50/5				>50	5

Notes: *Drilled on existing unimproved (dirt) road.*

Figure Number B-3

MOORE TWINING ASSOCIATES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-3

Project: Jeff Taylor Property

Location: Carmel, CA

Logged By: K.C.

Drilled By: S.R.

Drill Type: CME 75

Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01

Date: 5/24/07

Elevation: Approx. 212 feet AMSL

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
35						
40			Moist, friable, hard drilling at 45-50 feet			
45	4/6 10/6 15/6		Fresh granodiorite in sample shoe- partial recovery		25	16
50			Bottom of boring at 50 feet			
55						
60						
65						

Notes: *Drilled on existing unimproved (dirt) road.*

Figure Number B-3

SOIL TEST BORING SYMBOLIC LOG

MOORE TWINING ASSOCIATES, INC. EST. 1898

BORING B-4

Project: Jeff Taylor Property | **Project Number:** D69201.01

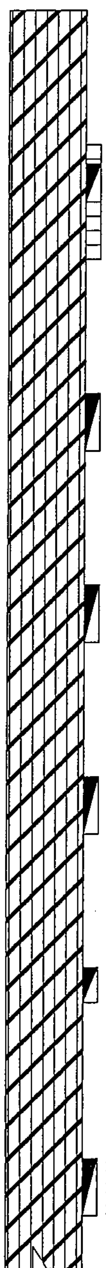
Location: Carmel, CA | **Date:** 5/25/07

Logged By: K.C. | **Elevation:** Approx. 254 feet AMSL

Drilled By: S.R. | **Depth to Groundwater:** N/E

Drill Type: CME 75 | **Cased to Depth:** N/A

Auger Type: HSA 6 5/8' O.D. | **Hammer Type:** TRIP



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
0		CL-ML	LEAN CLAY and Silt; Hard, yellowish brown matrix with fragments of porcelenite, chaotic landslide deposit	RI = 12		
5	17/6 50/6				-- >50 --	
10	18/6 32/6 32/6				64	
15	10/6 15/6 16/6				31	
20	11/6 15/6 24/6				39	
25	25/6 50/5				>50	
30	11/6 15/6 21/6				36	

Notes: *Drilled on existing unimproved (dirt) road.*

Figure Number B-4

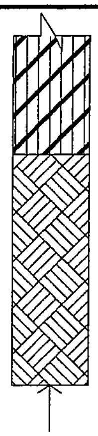
SOIL TEST BORING SYMBOLIC LOG

MOORE TWINING ASSOCIATES, INC. EST. 1898

BORING B-4

Project: Jeff Taylor Property
Location: Carmel, CA
Logged By: K.C.
Drilled By: S.R.
Drill Type: CME 75
Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01
Date: 5/25/07
Elevation: Approx. 254 feet AMSL
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: TRIP



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
35						
		ROCK	Granodiorite; Highly weathered	Partial sample Recovery: Fragments of rock in sample shoe at 36 and 42 feet		
40						
			Drilling Refusal at 42 feet. No sampler advance at 35, 40, or 42 feet BSG			
45						
50						
55						
60						
65						

Notes: *Drilled on existing unimproved (dirt) road.*

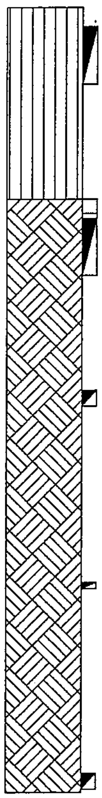
MOORE TWINING ASSOCIATES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-5

Project: Jeff Taylor Property
Location: Carmel, CA
Logged By: K.C.
Drilled By: S.R.
Drill Type: CME 75
Auger Type: HSA 6 5/8' O.D.

Project Number: D69201.01
Date: 5/25/07
Elevation: 65 feet AMSL
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: TRIP



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-values blows/ft.	Moisture Content %
65 — 0	7/6 6/6 4/6	ML	SILT, Sandy; Loose, gray, chaotic landslide material		10	17
60 — 5	35/6 50/	ROCK	Granodiorite rock, highly weathered	DD = 118.5pcf	>50	4
55 — 10	50/5				>50	2
50 — 15	50/2				>50	
45 — 20	50/5				>50	3
			Bottom of boring at 20.5 feet			
40 — 25						
35 — 30						

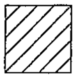
Notes: *Drilled on Lot 1*

Figure Number B-5

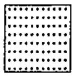
KEY TO SYMBOLS

Symbol Description

<u>Strata symbols</u>

 Low plasticity clay

 Basalt (or generic rock)

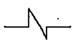
 Well graded sand

 Silty low plasticity clay

 Silt

<u>Misc. Symbols</u>

 Drill rejection

 Boring continues

<u>Soil Samplers</u>

 California Modified split barrel ring sampler

 Standard penetration test

<u>Notes:</u>

1. Exploratory borings were drilled on 5/24/07 and 5/25/07 using a 6 5/8 hollow stem auger.

2. No free water was encountered at the time of drilling.

3. Boring locations were taped from existing features and elevations extrapolated from the vesting tentative map.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.

KEY TO DESCRIPTIVE TERMS USED ON CORE LOGS

ROCK WEATHERING

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Extremely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 2-inch-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 2-inch-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

ROCK STRENGTH

Description	Recognition	Approximate Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Can be indented by thumbnail	35 - 150
Very Weak Rock	Can be peeled by pocket knife	150 - 700
Weak Rock	Can be peeled with difficulty by pocket knife	700 - 3,600
Moderately Strong Rock	Can be fractured with single firm hammer blow	3,600 - 7,200
Strong Rock	Requires more than one hammer blow to fracture	7,200 - 14,500
Very Strong Rock	Requires many hammer blows to fracture	14,500 - 36,000
Extremely Strong Rock	Can only be chipped with hammer blows	36,000

ROCK FRACTURING

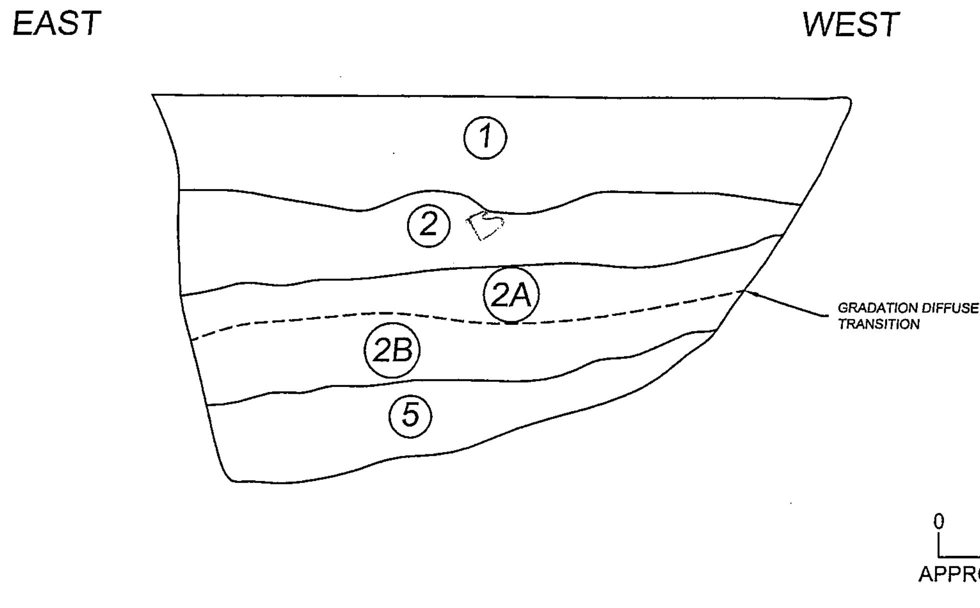
Description	Recognition
Intensely Fractured	Spacing less than 2 inches
Highly Fractured	Spacing from 2 inches to 1 foot
Moderately Fractured	Spacing from 1 foot to 3 feet
Slightly Fractured	Spacing from 3 feet to 10 feet
Massive	Spacing greater than 10 feet

RQD, ROCK QUALITY DESIGNATION

RQD	Classification
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

LOT 1

TP-12



Unit 1: Organic rich top soil: Sandy silt, brown to dark brown, with abundant fine grass rootlets and abundant roots about $\frac{1}{4}$ inch diameter to about $1\frac{1}{2}$ feet below site grade.

Unit 2: Silt: tan to yellowish brown, abundant siltstone, claystone, and porcelanite fragments $\frac{1}{8}$ to 1 inch in diameter. Increase in clay content below about 5 feet below site grade. Granitic boulder note, rounded granitic and fine sandstone cobbles.

Unit 2A: Clay: pale olive to brown , blocky with some slickensides (slip) surfaces (see photograph).

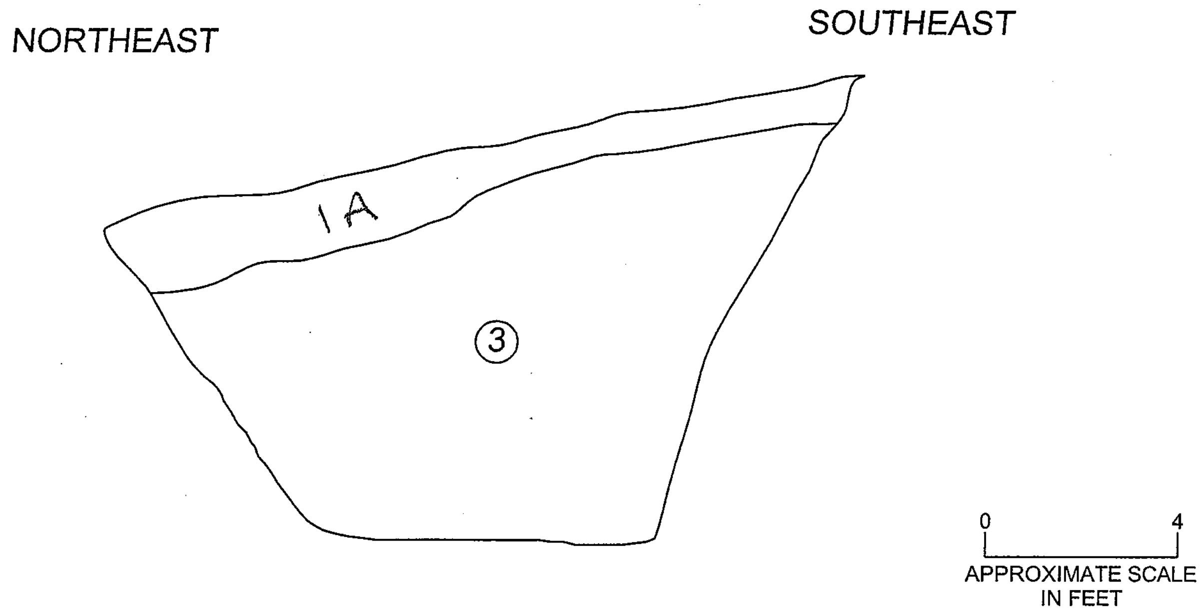
Unit 2B: Silty clay: yellowish brown to pale olive.

Unit 5: Granitic rock, highly weathered, difficult to dig with backhoe.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 2

TP-15



Unit 1A: Silt: greyish tan, some rootlets, abundant siltstone and porcelanite fragments, porous, appears loose.

Unit 3: Lean Clay: Grey, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter.

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	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO.	

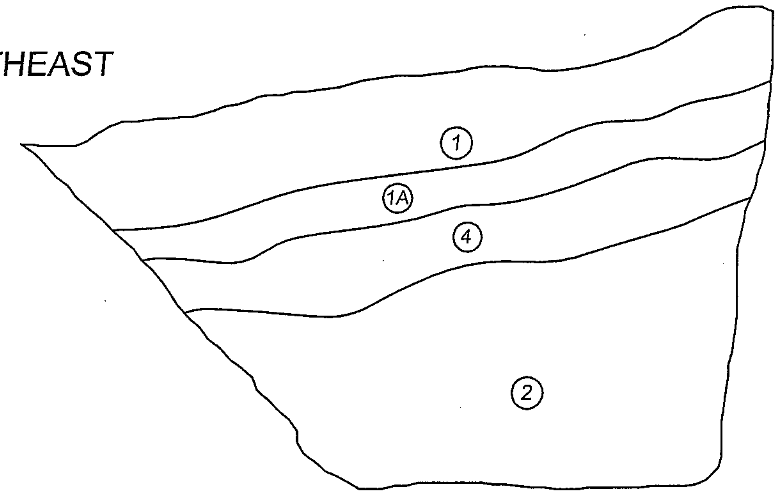
LOT 3

TP-5

NORTHEAST

SOUTHWEST

0 4
APPROXIMATE SCALE
IN FEET



TP-5

Unit 1: Organic rich top soil: Sandy silt, dark brown to black, organic rich with scattered fine grass rootlets to about 18 inches below site grade. Scattered rounded granitic cobbles and pebbles up to 1 foot in diameter. Undulatory bottom surface grading to light grey silt (Unit 1A).

Unit 1A: Silt, light grey, with fragments of siltstone.

Unit 4: Lean Clay, sandy, blocky, greyish brown, several shear surfaces dipping downslope (see photographs below).

Unit 2: Silt with some layers of clay and scattered rounded granitic boulders, cobbles and pebbles, yellowish brown.

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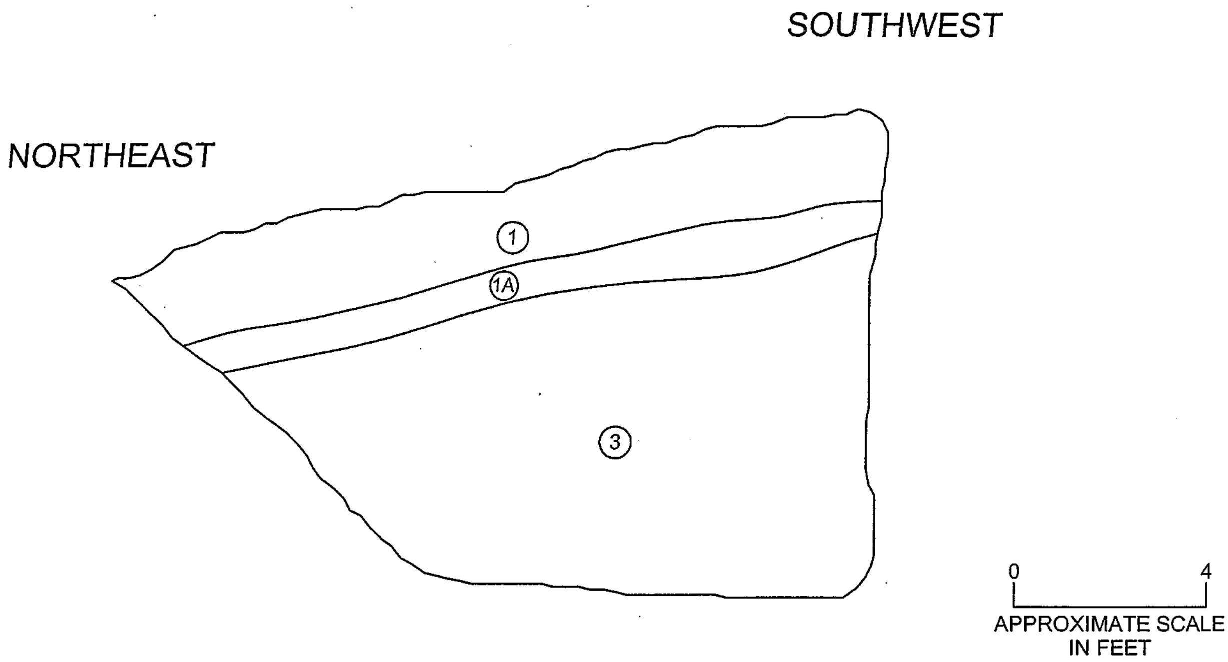
	DRAWN BY: RM	APPROVED BY: YC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 4

TP-4



TP-4

Unit 1: Organic rich top soil: Sandy silt, dark grey, organic rich with scattered fine grass rootlets to about 18 inches below site grade. Scattered rounded granitic boulders, cobbles and pebbles. Undulatory bottom surface grading to light grey silt (Unit 1A).

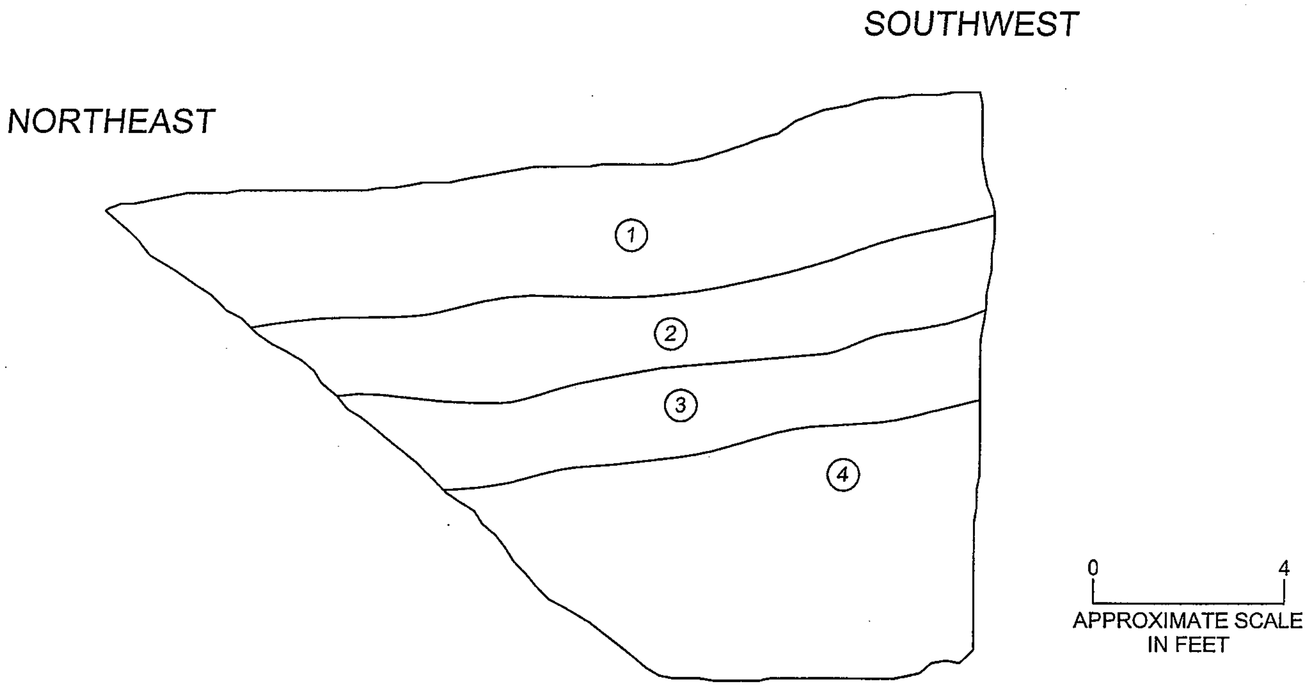
Unit 1A: Light grey silt with fragments of siltstone.

Unit 3: Lean Clay, dark brown, with abundant siltstone and porcelanite fragments generally less than 1 inch in diameter.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 5

TP-1



TP-1

Unit 1: Organic rich top soil: Sandy silt, dark brown, organic rich with abundant fine grass rootlets and occasional roots about 1/4 inch diameter. Undulatory bottom surface grading to greyish tan silt (Unit 1A).

Unit 1A: Silt: Greyish tan, blocky, some rootlets, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, porous, appears loose.

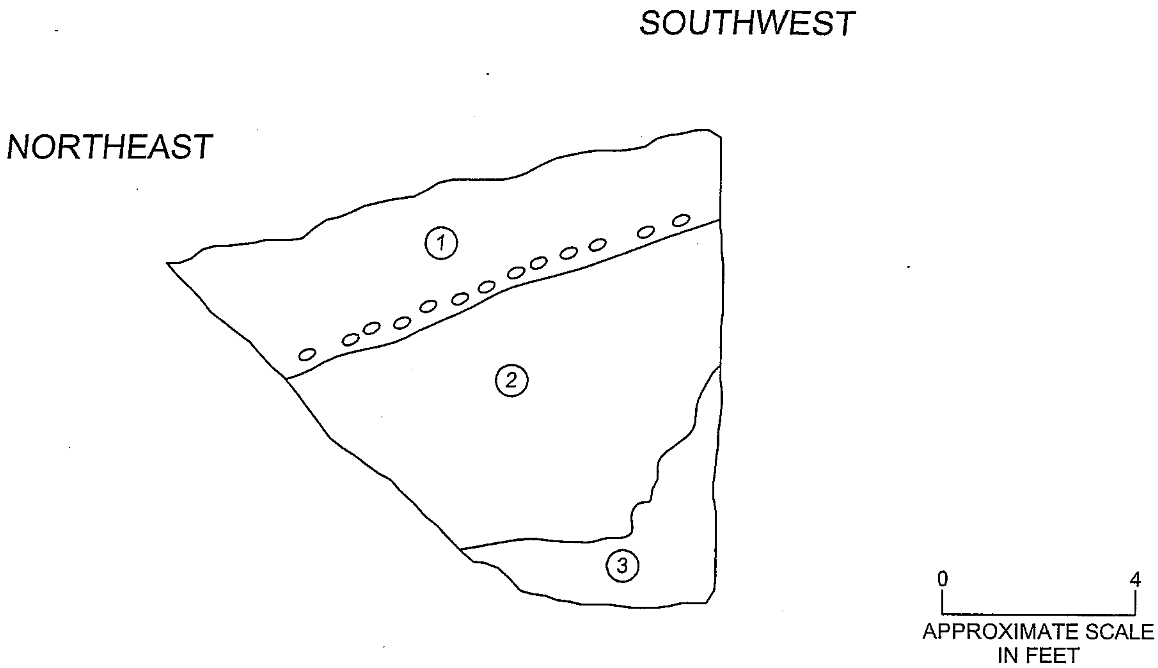
Unit 3: Lean Clay: Grey, blocky with abundant siltstone and porcelanite fragments generally less than 1 inch in diameter.

Unit 4: Siltstone, pale olive and yellowish brown (mottled), intensely fractured and sheared, weak to moderately strong rock, moderately weathered.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: CC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 5

TP-2



TP-2

Unit 1: Organic rich top soil: Sandy Silt, dark brown to black, organic rich with abundant fine grass rootlets and occasional roots about 1/4 inch diameter. Undulatory bottom surface grading to greyish tan silt (Unit 2).

Unit 2: Silt: grayish tan, blocky, some rootlets, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter.

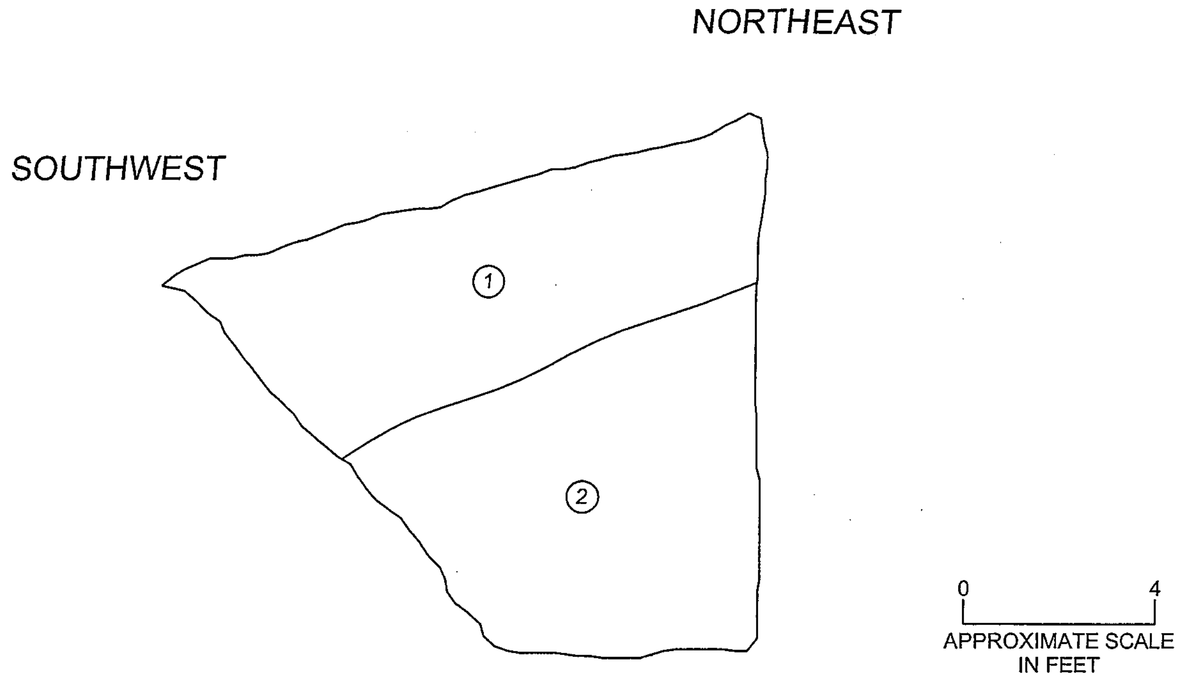
Unit 3: Lean Clay: dark brown, with abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, no rootlets.

* Angular fragments of siltstone from 1 inch to greater than 6 inches in diameter.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA EST.1898 MOORE TWINING ASSOCIATES, INC. FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: XC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 6

TP-6



TP-6

Unit 1: Organic rich top soil: Sandy silt, brown to dark brown, organic rich with scattered fine grass rootlets to about 36 inches below site grade and occasional roots to 1/4 inch diameter.

Unit 2: Silt: greyish tan, minor clayey silt, abundant siltstone, claystone, and porcellanite fragments 1/8 to 1 inch in diameter. No granitic pebbles/cobbles.

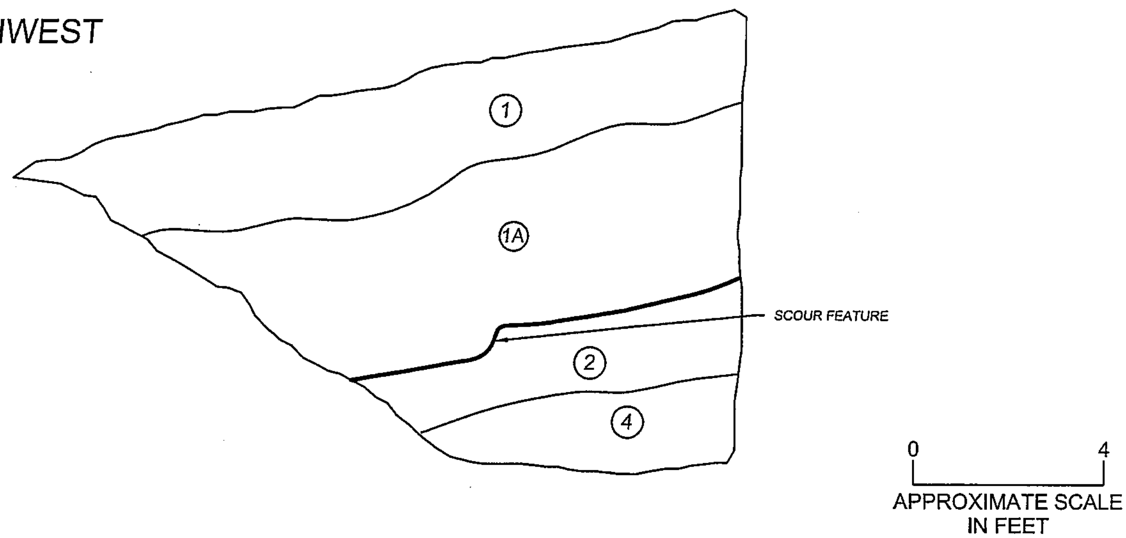
TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: VC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 6

TP-7

NORTHEAST

SOUTHWEST



TP-7

Unit 1: Organic rich top soil: Sandy silt, brown to dark brown, organic rich with scattered fine grass rootlets to about 24 to 54 inches below site grade and occasional roots to 1/4 inch diameter.

Unit 1A: Silt: greyish tan, blocky, some rootlets, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, porous, appears loose. Lower 1 to 2 feet of unit is moist soil zone perched on underlying blocky siltstone.

Unit 2: Silt: Greyish tan, minor clayey silt, abundant siltstone, claystone, and porcelanite fragments 1/8 to 1 inch in diameter. No granitic pebbles/cobbles.

Unit 4: Siltstone, pale olive and yellowish brown (mottled), intensely fractured - near vertical fissures spaced at several inches, weak to moderately strong rock, moderately weathered.

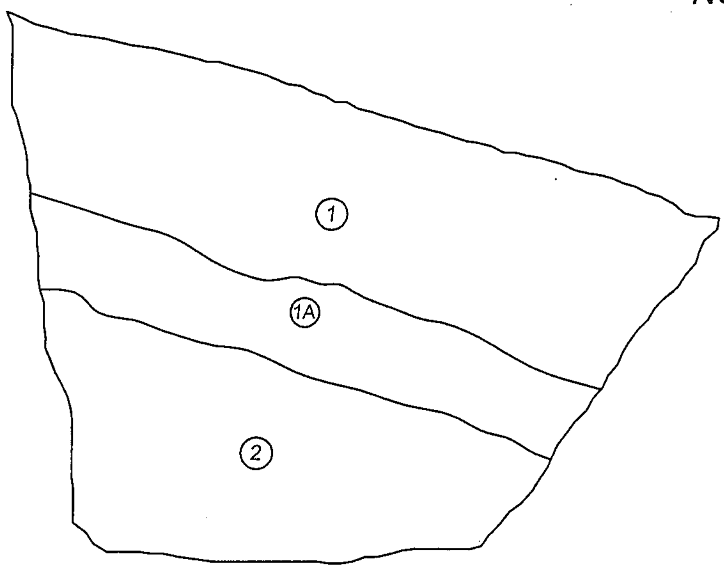
TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 7

TP-8

SOUTHWEST

NORTHEAST



0 4

APPROXIMATE SCALE
IN FEET

TP-8

Unit 1: Organic rich top soil: Sandy silt, dark brown, organic rich with abundant fine grass rootlets and occasional roots about 1/4 inch diameter to depths of 3 to 3½ feet below site grade. Undulatory bottom surface grading to greyish tan silt (Unit 1A).

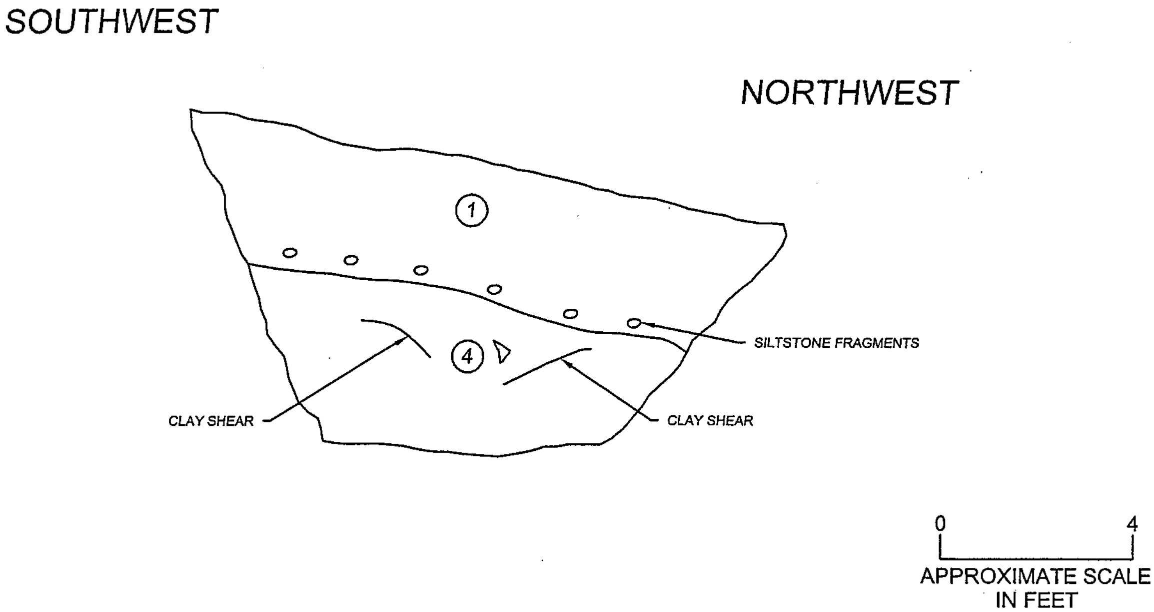
Unit 1A: Silt: Greyish tan, blocky, some rootlets, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, porous, appears loose.

Unit 2: Silt: Greyish tan, abundant siltstone, claystone, and porcellanite fragments 1/8 to 1 inch in diameter. No granitic pebbles/cobbles.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST. 1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: JC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 7

TP-11



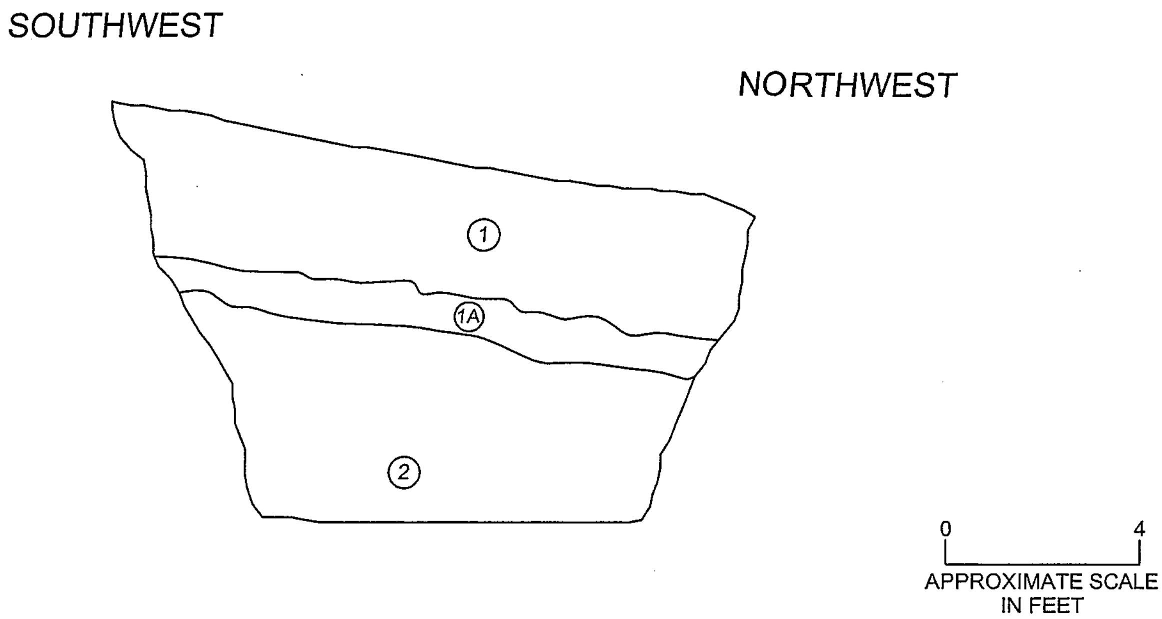
Unit 1: Organic rich top soil: Sandy silt, dark brown, organic rich with abundant fine grass rootlets and occasional roots about 1/4 inch diameter to depth of 2½ feet below site grade. Undulatory bottom surface with angular siltstone fragments notable near bottom of unit.

Unit 4: Massive highly fractured siltstone, light tan, several clay in-filled joints ¼ to 1 inch wide, no clear preferred fracture orientation noted.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 8

TP-9



TP-9

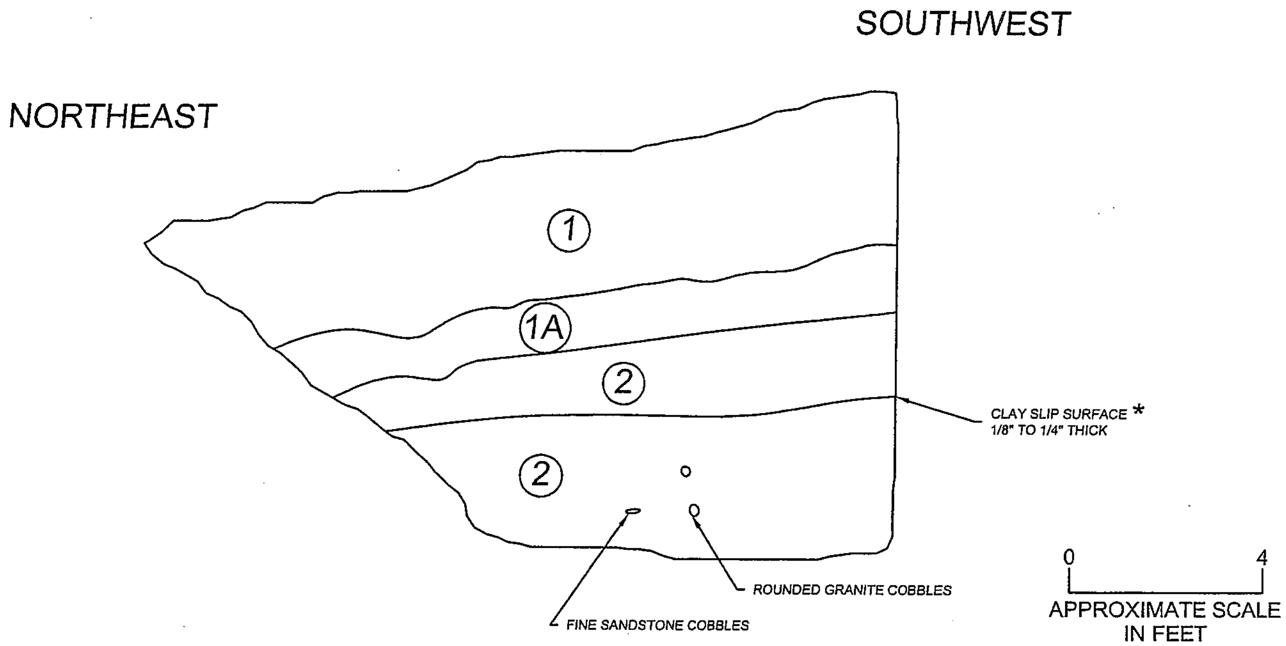
Unit 1: Organic rich top soil: Sandy silt, brown, with organics with abundant fine grass rootlets and occasional roots about 1/4 inch diameter to depths 2 to 2½ feet below site grade. Undulatory bottom surface grading to greyish tan silt (Unit 1A). Angular siltstone fragments from less than 1 inch to several inches in diameter.

Unit 2: Silt: Greyish tan, abundant siltstone, claystone, and porcelanite fragments 1/8 to 1 inch in diameter. No granitic pebbles/cobbles. Slight increase in clay content below about 4 feet below site grade.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: lc	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 9

TP-13



SLIP SURFACE STIKE AT ABOUT PIT ORIENTATION DIP TO WEST ABOUT 20 DEGREES

TP-13

Unit 1: Organic rich top soil: Sandy silt: dark brown to dark grey, with abundant fine grass rootlets and roots about 1/4 inch diameter to a depth of about 1½ feet below site grade. Undulatory bottom surface grading to greyish tan silt (Unit 1A). Some angular siltstone fragments from less than 1 inch to several inches in maximum dimension.

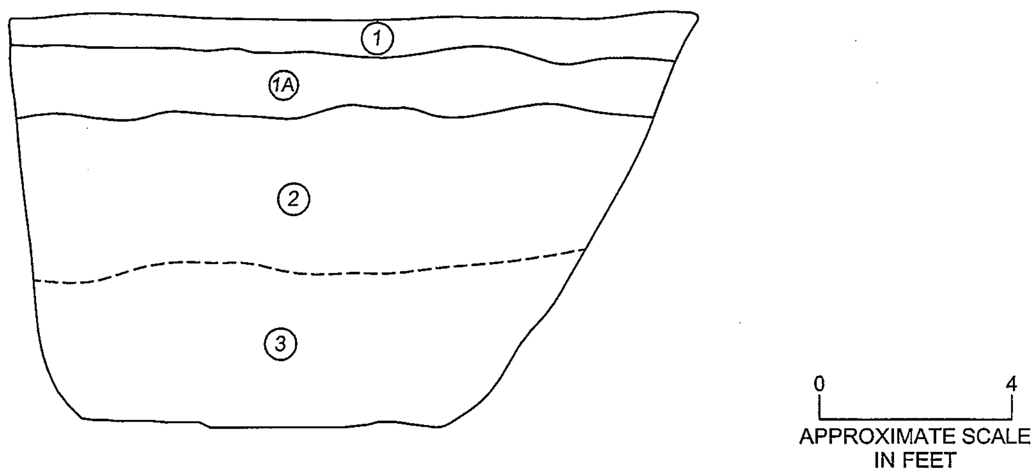
Unit 1A: Silt: greyish tan, some rootlets, abundant siltstone and porcelanite fragments, porous, appears loose.

Unit 2: Silt: tan to yellowish brown, abundant siltstone, claystone, and porcelanite fragments 1/8 to 1 inch in diameter. Increase in clay content below about 5 feet below site grade. Granitic boulder noted, rounded granitic and fine sandstone cobbles.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: LCL	
	PROJECT NO. D69201.01	DRAWING NO.	

Lot 10

TP-14



TP-14

Unit 1: Organic rich top soil: Sandy silt, brown, with abundant fine grass rootlets and occasional roots about 1/4 inch diameter to depths of about ½ to 1 foot below site grade. Undulatory bottom surface grading to greyish tan silt (Unit 1A). Angular siltstone fragments from less than 1 inch to several inches in diameter.

Unit 1A: Silt matrix: Greyish tan, blocky, some rootlets, abundant siltstone and porcelanite fragments generally less than 1 inch in diameter, some rootlets, porous, appears loose.

Unit 2: Silt: greyish tan, abundant siltstone, claystone, and porcelanite fragments ⅛ to 1 inch in diameter. No granitic pebbles/cobbles. Increase in clay content below about 5 feet below site grade.

Unit 3: Lean Clay, dark brown, with abundant siltstone and porcelanite fragments generally less than 1 inch in diameter.

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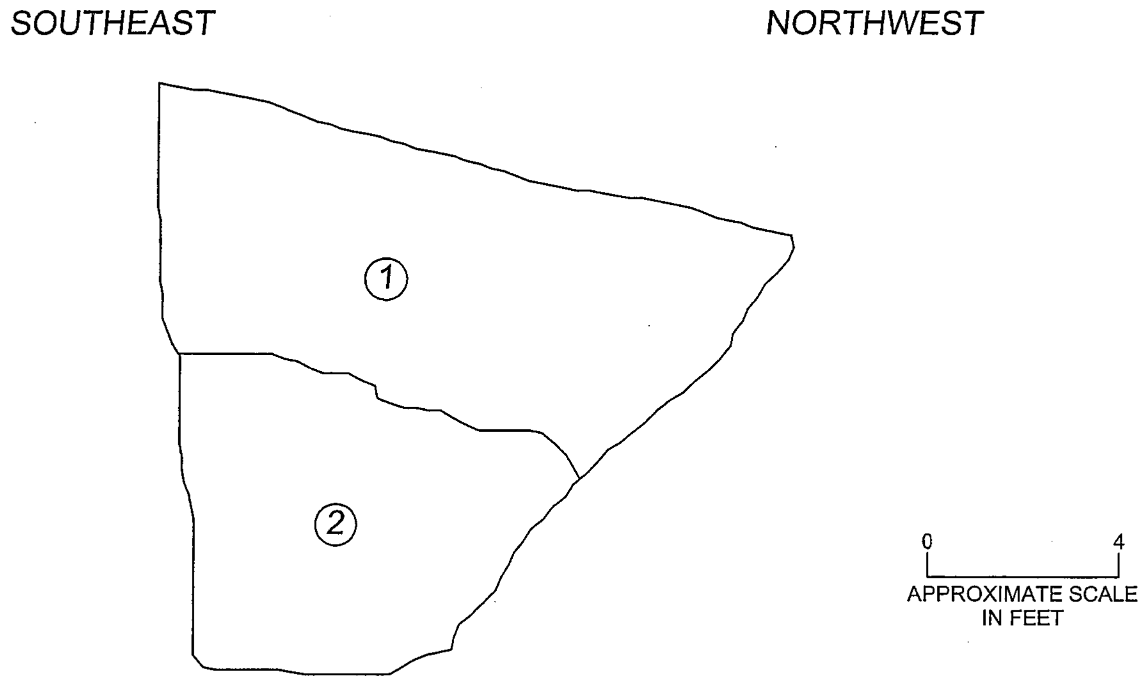
	DRAWN BY: RM	APPROVED BY: VC	
	PROJECT NO. D69201.01	DRAWING NO.	

LOT 10

TP-10



TP-10

Unit 1: Organic rich top soil: Sandy silt, dark brown to dark grey, appears to have high organic content to about 5 feet below site grade, abundant fine grass rootlets and some roots about 1/4 inch diameter to about 1½ feet below site grade. Undulatory bottom surface grading to tan silt and silty sand (Unit 2). Some angular siltstone fragments from less than 1 inch to several inches in diameter.

Unit 2: Silt: tan, abundant siltstone fragments 1/8 to 1 inch in diameter.

TEST PIT LOGS PRELIMINARY GEOTECHNICAL ENGINEERING AND GEOLOGIC INVESTIGATION PROPOSED TEN (10) LOT RESIDENTIAL SUBDIVISION CARMEL, MONTEREY COUNTY, CALIFORNIA	FILE NO. 69201-01-02X	DATE DRAWN: 10/04/07	MTA MOORE TWINING ASSOCIATES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY: LC	
	PROJECT NO. D69201.01	DRAWING NO.	

APPENDIX C

APPENDIX C

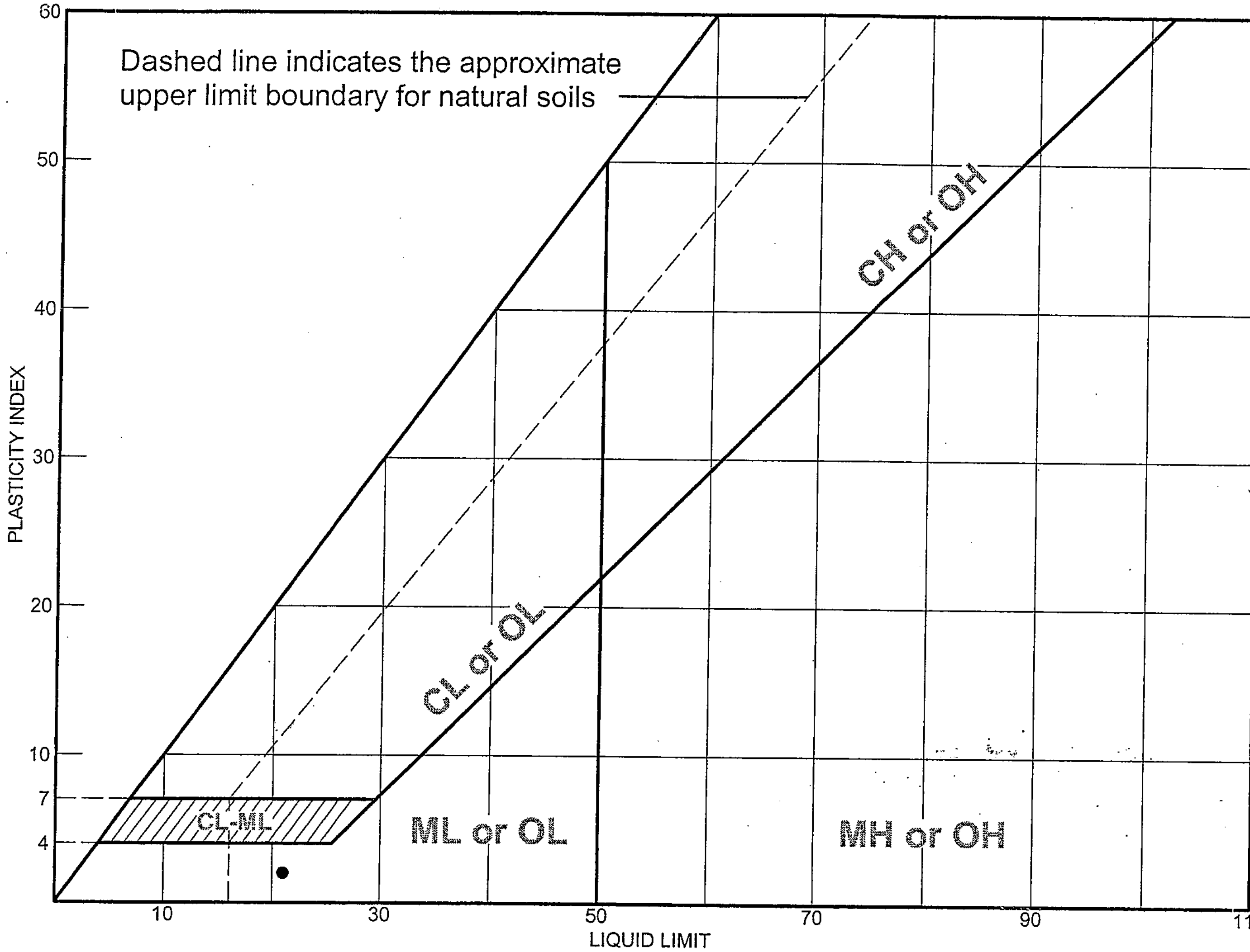
RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	Number of Tests:	To Determine:
Atterberg Limits (ASTM D4318)	2	The consistency and "stickiness," as well as the range of moisture contents within which the material is "workable".
Remolded Direct Shear (ASTM D3080)	1	Soil shearing strength of remolded soil sample under varying loads and/or moisture conditions.
Direct Shear (ASTM D3080)	2	Soil shearing strength of a ring sample of native soil under varying loads and/or moisture conditions.
Dry Density (ASTM D2216)	17	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Moisture Content (ASTM D2216)	19	Moisture contents representative of field conditions at the time the sample was taken.
Moisture-Density Relationship (ASTM D1557)	3	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.

These Included:	Number of Tests:	To Determine:
Expansion Index (UBC 18-2)	2	Swell potential of soil with increases in moisture content.
Loss-on-ignition	1	Percent organic content of soil sample by dry weight.
R-Value (ASTM CTM)	2	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (ASTM D4327)	2	Percentage of water-soluble sulfate as (SO_4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	2	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM D1125)	2	The potential of the soil to corrode metal.
pH (ASTM D4972)	2	The acidity or alkalinity of subgrade material.

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		21	19	2			

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 4.3-5.8'

Remarks:

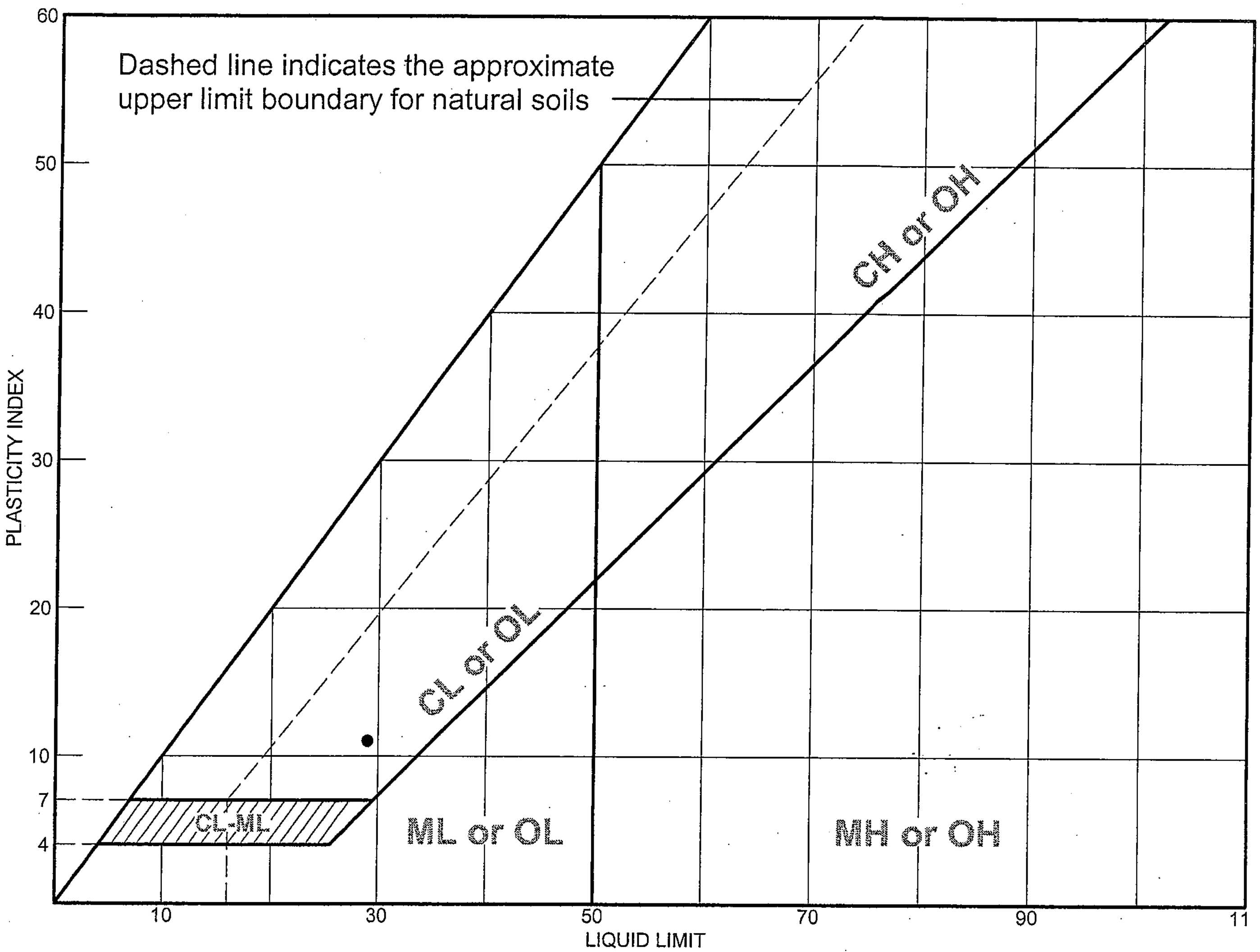
●

LIQUID AND PLASTIC LIMITS TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 1

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		29	18	11			

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 8-8.5'

Remarks:

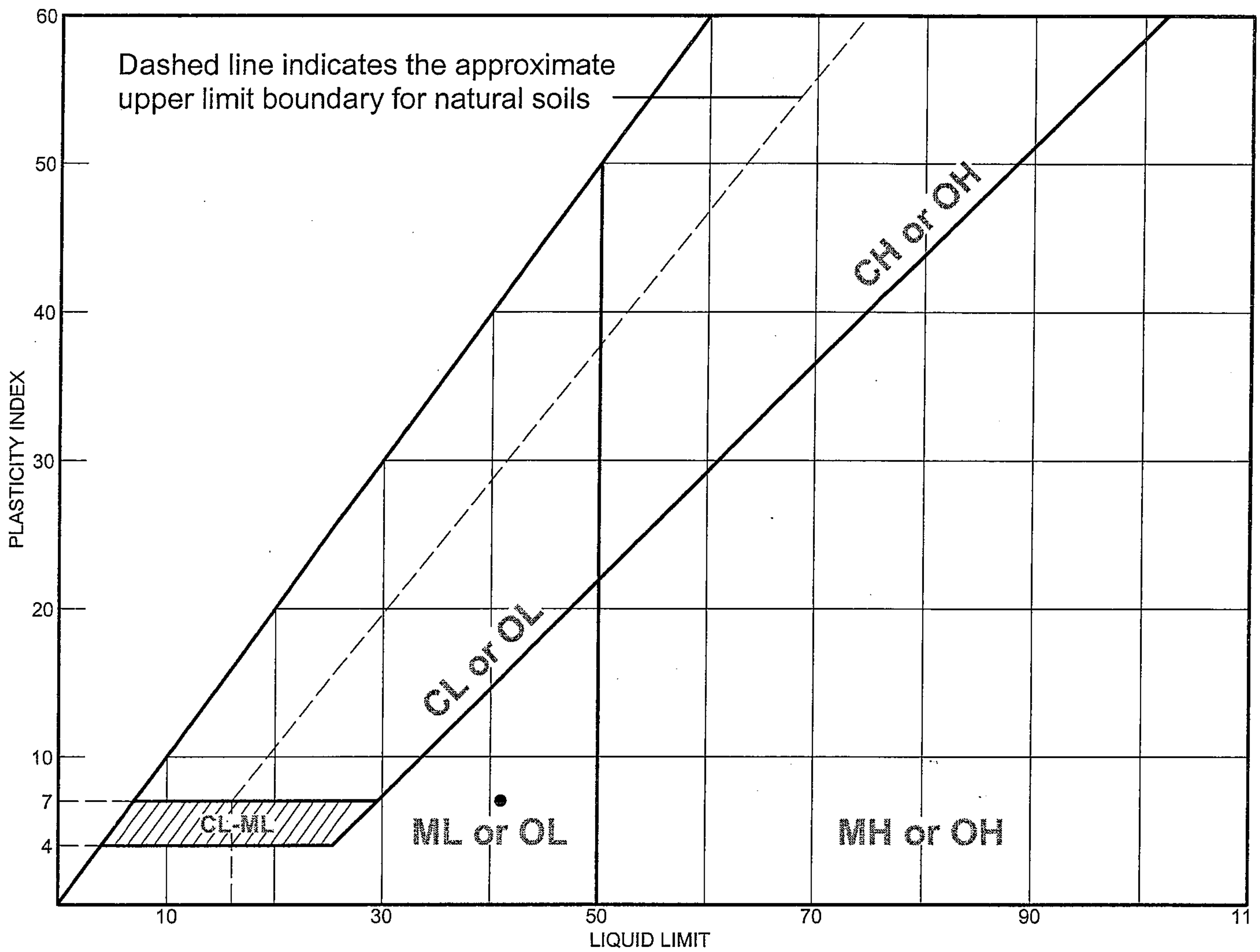
●

LIQUID AND PLASTIC LIMITS TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 2

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		41	34	7			

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 4-5'

Moore Twining Associates, Inc.

Fresno, CA

Remarks:

●

Figure 3

Moore Twining Associates, Inc.
2527 Fresno St.
Fresno, CA 93721
(559)268-7021

EXPANSION BUILDING CODE (UBC) 18-2

Project Number:	D69201.01	Project:	Jeff Taylor Properties
Sample Location:	TP-6, Lot 6 Unit 2	Depth:	4 - 11' BSG
Date Sampled:	5/24/2007	Sampled By:	K.C.

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
07-1067	**34.5**	**61.4**	**59.1**

Initial Thickness:	**1.0000**	Final Thickness:	**1.0000**
Expansion Index (EI):	**0**	Expansion Soil Classification:	**Very Low**

Table Number 18-2
Expansive Soil Classification

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Figure No. 4

Prepared 5/15/07

Moore Twining Associates, Inc.
2527 Fresno St.
Fresno, CA 93721
(559)268-7021

EXPANSION BUILDING CODE (UBC) 18-2

Project Number:	D69201.01	Project:	Jeff Taylor Properties, Carmel, CA
Sample Location:	B-4	Depth:	0-3' BSG
Date Sampled:	5/29/2007	Sampled By:	K.C.

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
07-1067	**17.4**	**30.0**	**87.1**

Initial Thickness:	**1.0000**	Final Thickness:	**1.0123**
Expansion Index (EI):	**12**	Expansion Soil Classification:	**Very Low**

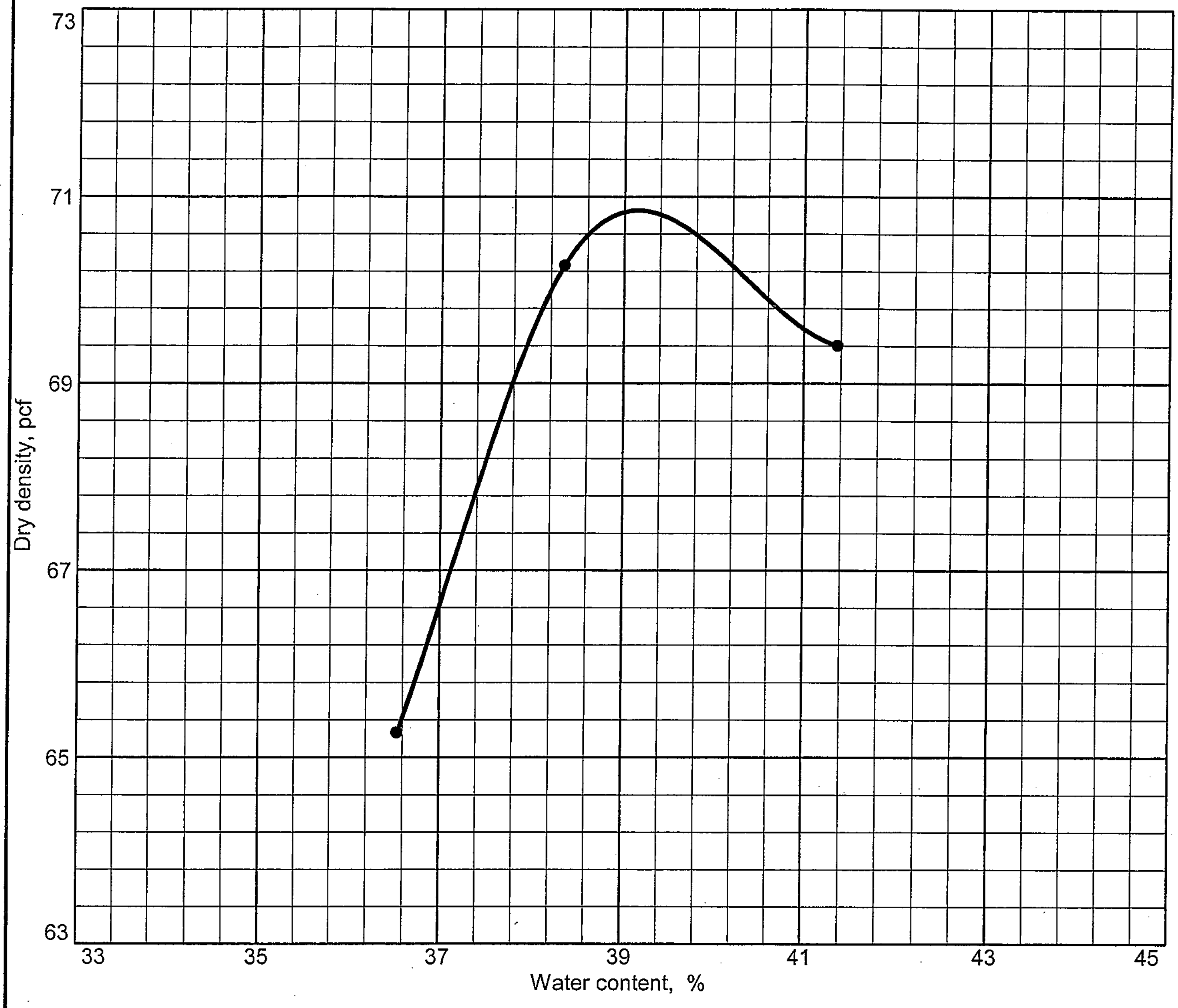
Table Number 18-2
Expansive Soil Classification

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Figure No. 5

Prepared 5/15/07

COMPACTION TEST REPORT



Test specification: ASTM D 1557-00 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
4-11' BSG								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 70.9 pcf Optimum moisture = 39.1 %	

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

● **Source:** **Sample** TP-6, Lot 6 **Elev./Depth:** 4-11' BSG

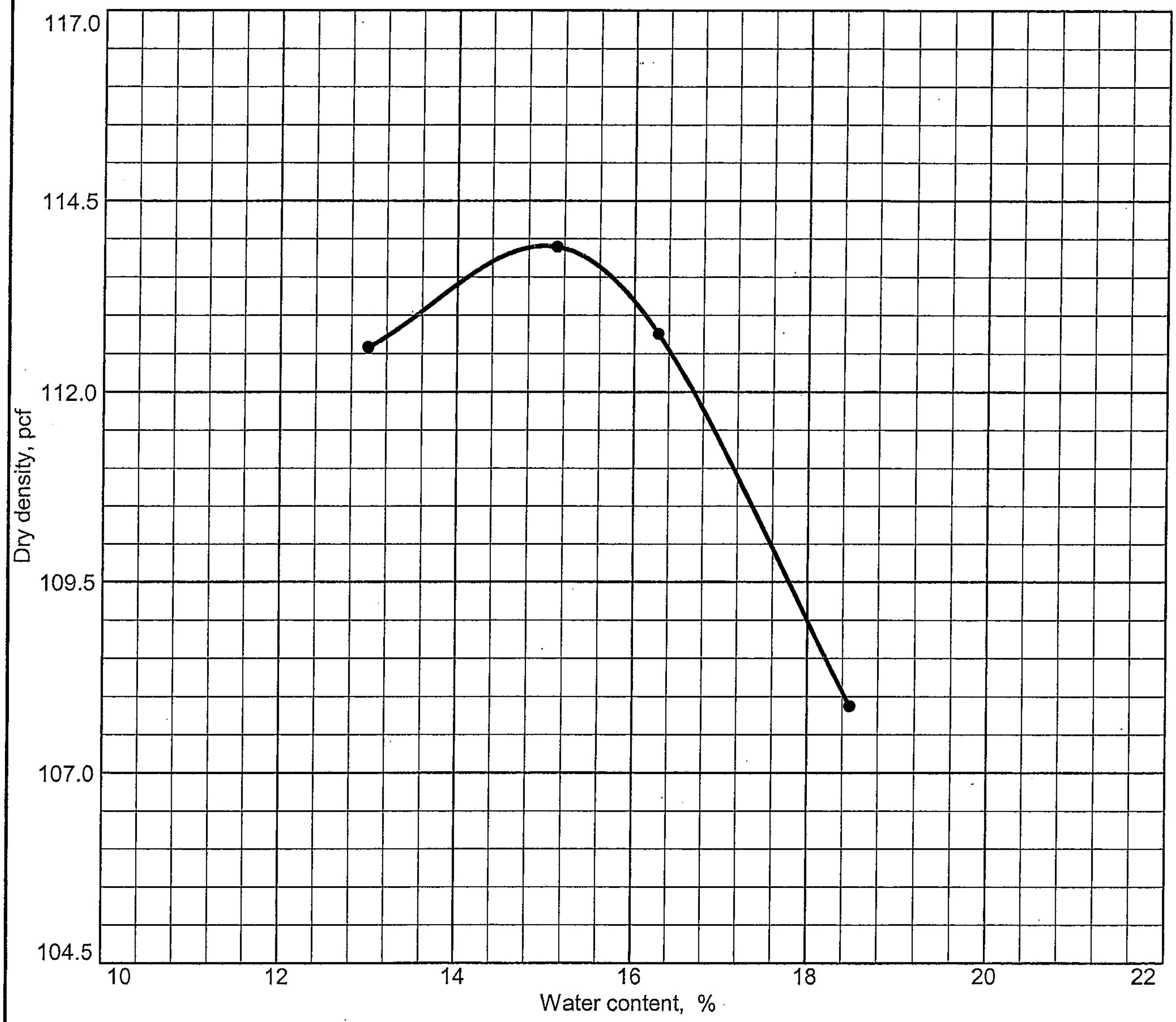
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure 6

PROCTOR TEST REPORT



Test specification: ASTM D 1557-00 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
48-60"								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 113.9 pcf Optimum moisture = 15.0 %	

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

Date:

● **Source:** **Sample No.:** TP-12 **Elev./Depth:** 48-60"

Remarks:

PROCTOR TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 7

PROCTOR TEST REPORT

Test specification: ASTM D 1557-00 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
24-36"								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 110.1 pcf Optimum moisture = 15.4 %	

Project No. D69201.01 **Client:** Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

Date:

● **Source:** **Sample No.:** TP-12 **Elev./Depth:** 24-36"

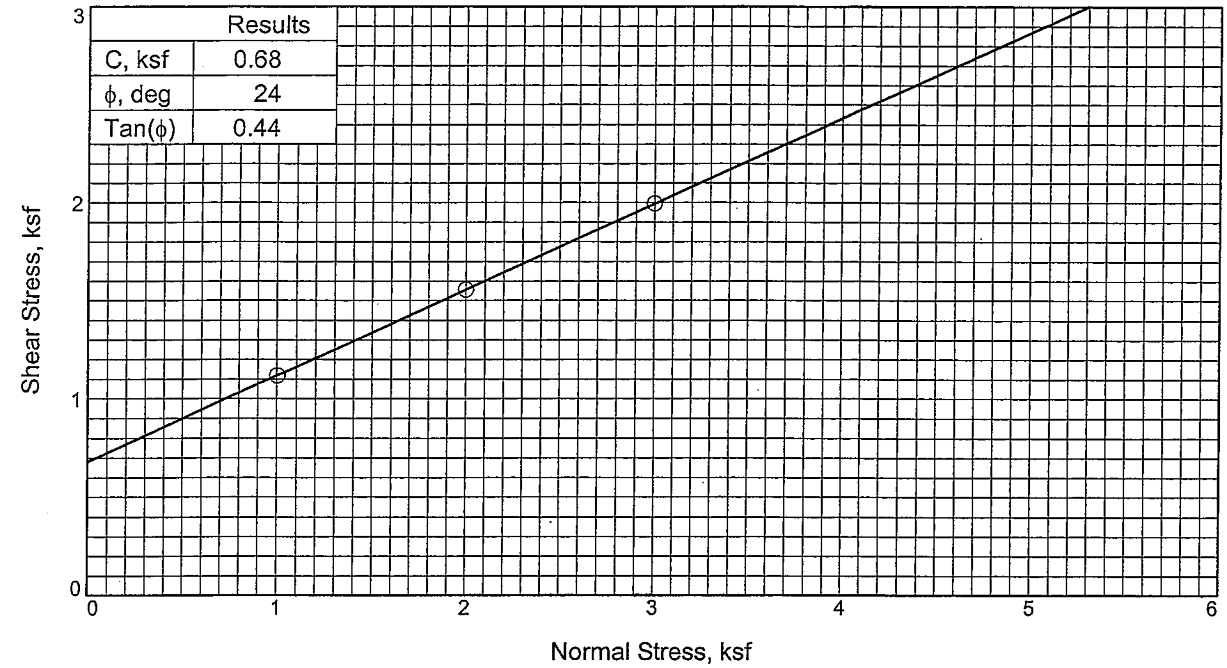
PROCTOR TEST REPORT

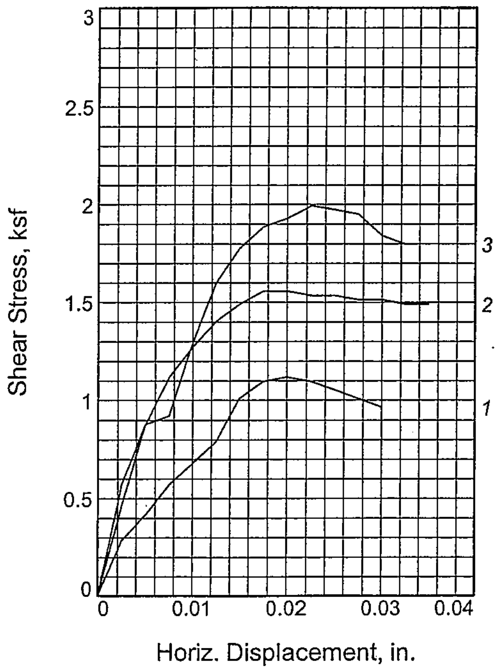
THE TWINING LABORATORIES, INC.

Remarks:

Figure No. 8

Results	
C, ksf	0.68
ϕ, deg	24
Tan(ϕ)	0.44





Sample No.		1	2	3
Initial	Water Content, %	39.1	39.1	39.1
	Dry Density, pcf	63.8	63.8	63.8
	Saturation, %	65.1	65.1	65.1
	Void Ratio	1.5915	1.5915	1.5915
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	63.1	56.8	58.7
	Dry Density, pcf	64.8	65.5	67.6
	Saturation, %	107.6	98.6	107.5
	Void Ratio	1.5526	1.5249	1.4479
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.97	0.94
Normal Stress, ksf		1.00	2.00	3.00
Shear Stress, ksf		1.12	1.56	1.99
Displacement, in.		0.02	0.02	0.02
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		0.8	0.7	0.9

Sample Type: Bulk
Description: Remolded to 90% relative Compaction (ASTM 1557)

Assumed Specific Gravity= 2.65
Remarks:

Figure 9

Client: Jeff Taylor Properties

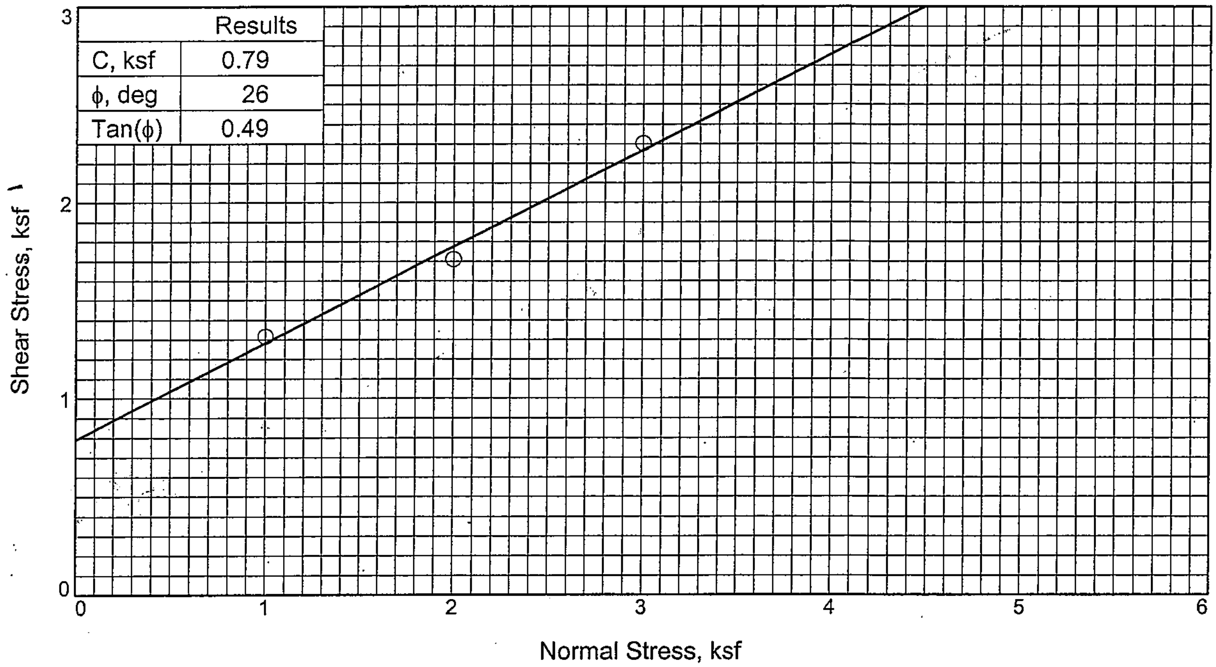
Project: Jeff Taylor Properties Carmel, CA

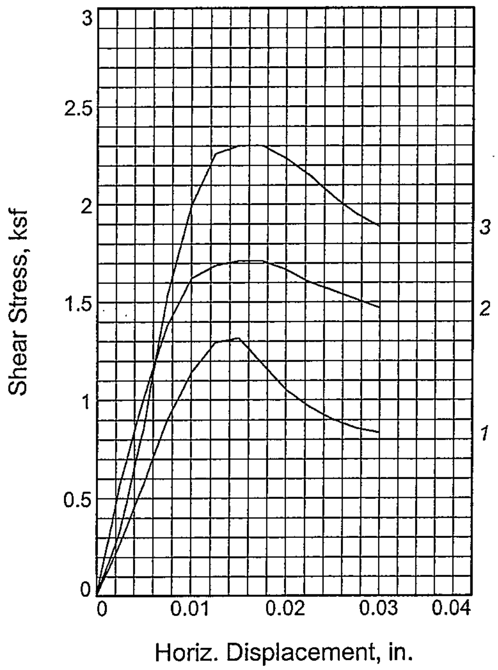
Sample Number: TP-6, Lot 6, Unit 2 **Depth:** 4-11' BSG

Proj. No.: D69201.01 **Date Sampled:**

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.





	Sample No.	1	2	3
Initial	Water Content, %	15.7	15.7	15.7
	Dry Density, pcf	106.5	109.0	106.5
	Saturation, %	75.1	80.3	75.0
	Void Ratio	0.5527	0.5171	0.5538
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	28.3	29.6	27.7
	Dry Density, pcf	108.8	115.6	109.9
	Saturation, %	144.1	181.9	145.2
	Void Ratio	0.5212	0.4316	0.5048
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.94	0.97
Normal Stress, ksf		1.00	2.00	3.00
Shear Stress, ksf		1.31	1.71	2.30
Displacement, in.		0.02	0.02	0.02
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		0.6	0.6	0.6

Sample Type: Ring
Description:

Assumed Specific Gravity= 2.65
Remarks:

Client: Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

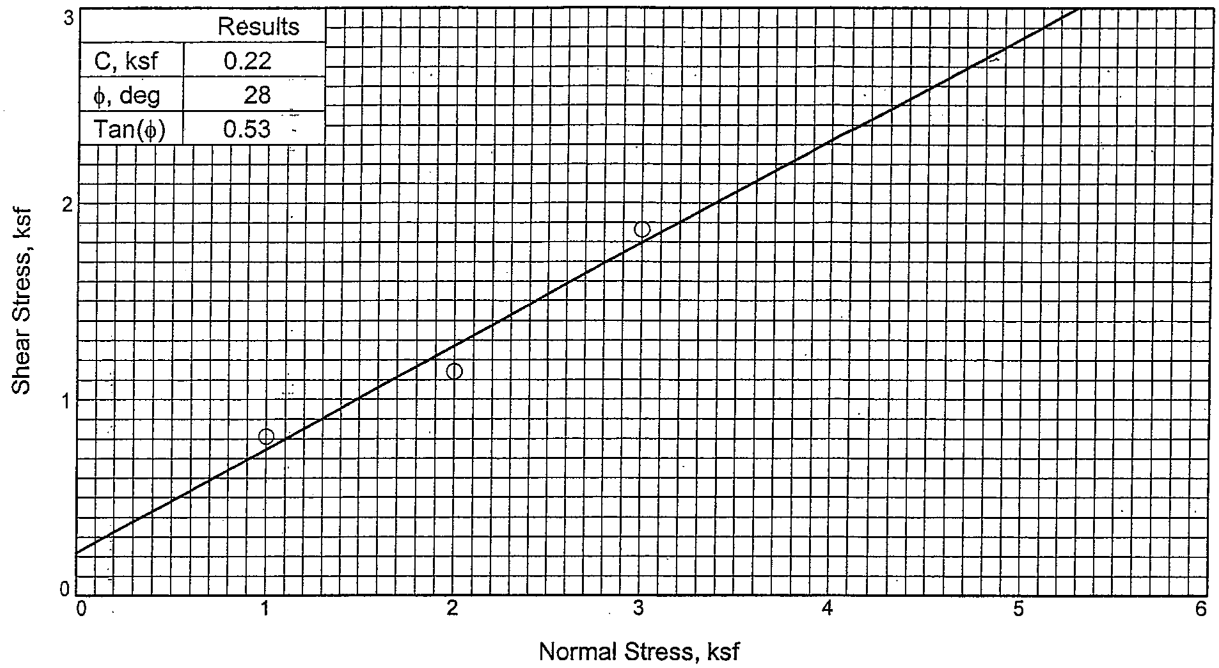
Sample Number: B-1 **Depth:** 3.5-5'

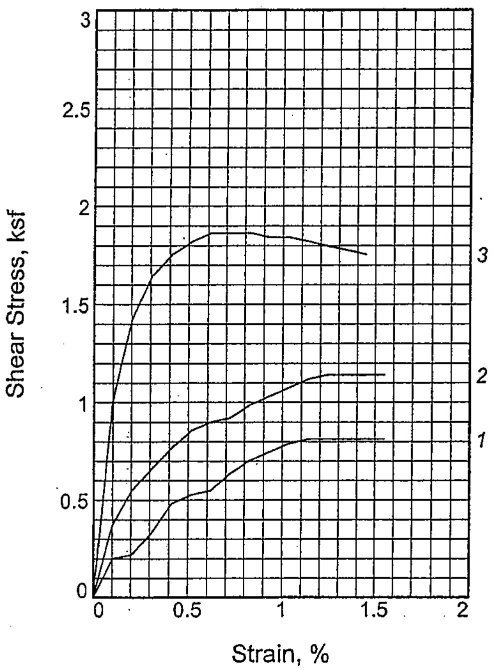
Proj. No.: D69201.01 **Date Sampled:**

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Figure 10





	Sample No.	1	2	3
Initial	Water Content, %	11.1	11.1	11.1
	Dry Density, pcf	111.8	111.8	111.8
	Saturation, %	61.4	61.4	61.4
	Void Ratio	0.4796	0.4796	0.4796
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	14.8	14.8	14.8
	Dry Density, pcf	114.4	121.1	122.1
	Saturation, %	88.1	107.4	110.7
	Void Ratio	0.4455	0.3656	0.3547
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.92	0.92
Normal Stress, ksf		1.00	2.00	3.00
Shear Stress, ksf		0.81	1.14	1.86
Strain, %		1.1	1.2	0.6
Ult. Stress, ksf				
Strain, %				
Strain at peak, %		1.1	1.2	0.6

Sample Type:

Description:
B-2 at 3.5-5, Reshear on relatively undisturbed ring sample.

Specific Gravity= 2.65

Remarks:

Figure 11

Client: Jeff Taylor Properties

Project: Jeff Taylor Properties Carmel, CA

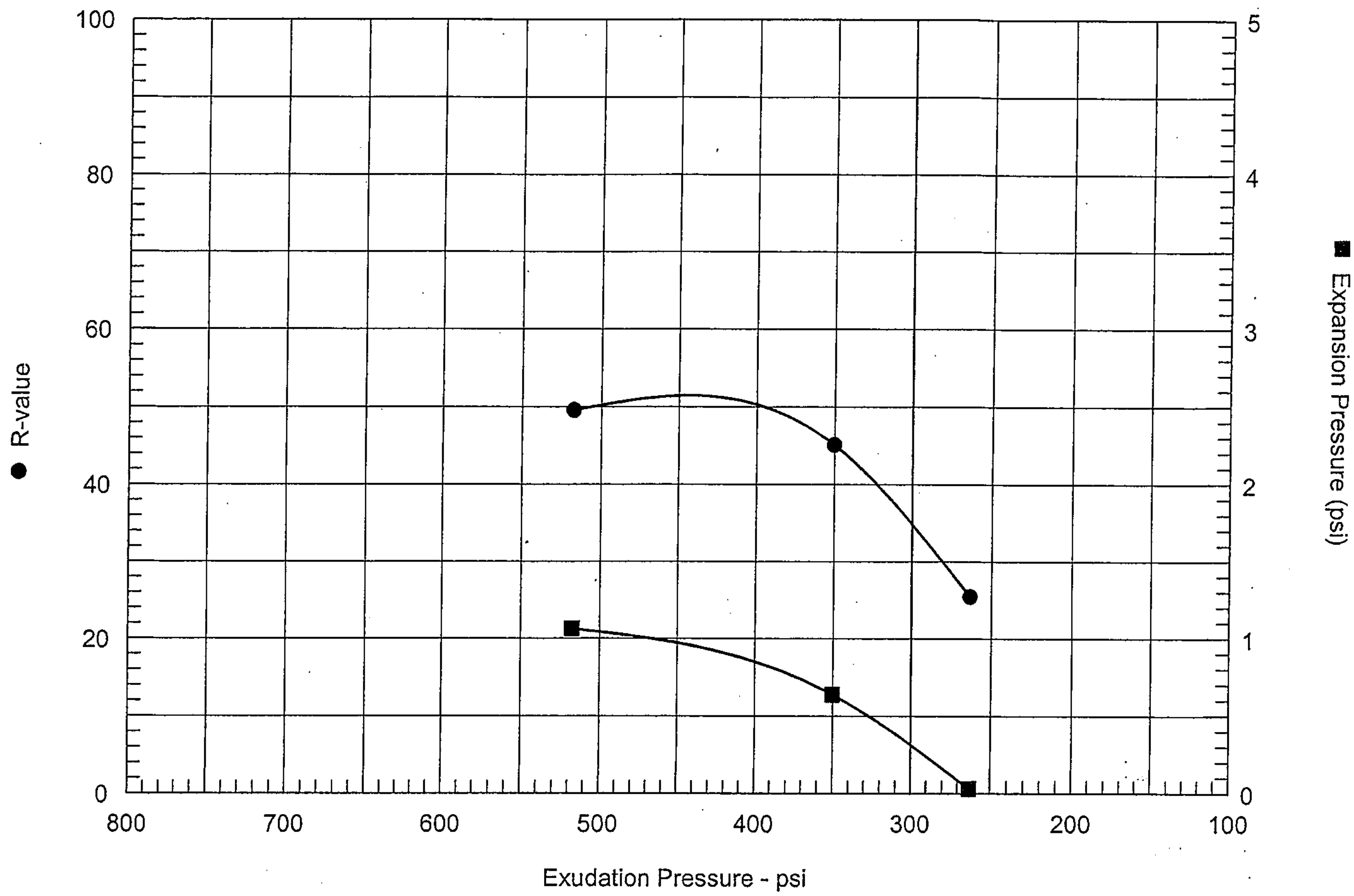
Sample Number: B-2 **Depth:** 3.5-4.3'

Proj. No.: D69201.01 **Date Sampled:**

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	350	95.4	25.4	1.06	54	2.50	517	50	50
2	275	93.5	26.5	0.64	66	2.55	350	45	45
3	150	91.6	27.6	0.03	97	2.38	263	28	25

Test Results	Material Description
R-value at 300 psi exudation pressure = 35 **Exp. pressure at 300 psi exudation pressure =** 0.31 psi	

Project No.: D69201.01
Project: Jeff Taylor Properties Carmel, CA
Sample Number: B-4 **Depth:** 0-3'

Date: 6/18/2007

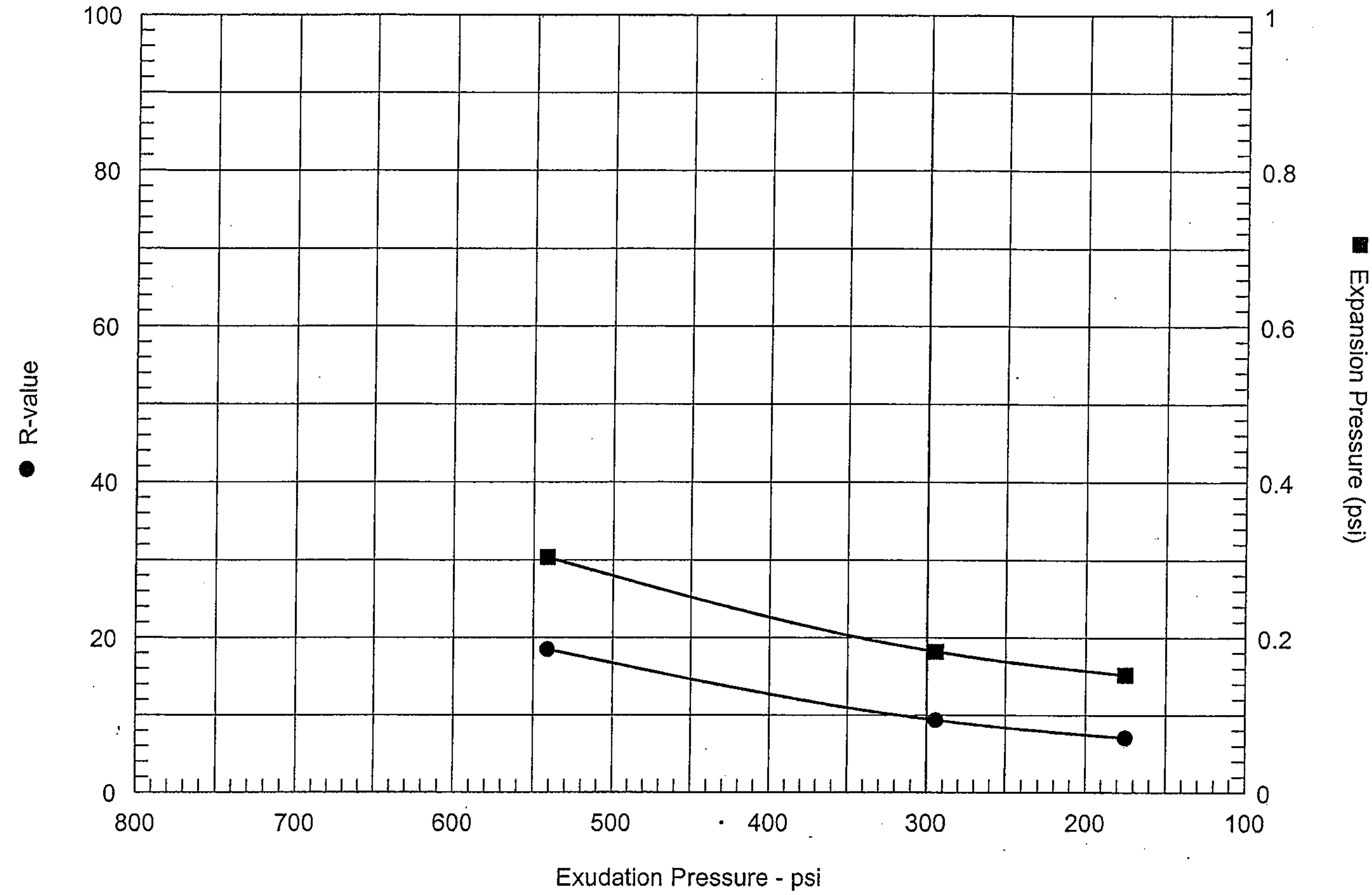
Tested by: 981
Checked by: 871
Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 12

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	120	97.7	24.1	0.30	116	2.65	541	17	18
2	120	96.8	25.2	0.18	135	2.69	294	8	9
3	60	95.6	26.3	0.15	142	2.55	175	7	7

Test Results	Material Description
R-value at 300 psi exudation pressure = 9 **Exp. pressure at 300 psi exudation pressure =** 0.18 psi	

Project No.: D69201.01

Project: Jeff Taylor Properties Carmel, CA

Sample Number: B-1 **Depth:** 0-3'

Date: 6/18/2007

Tested by: 981

Checked by: 871

Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. 13

MTA EST. 1898 ANALYTICAL CHEMISTRY DIVISION
MOORE TWINING ASSOCIATES, INC.
CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

May 31, 2007

Work Order #: 7E15023

Ken Clark
Twining Geotechnical Department
2527 Fresno Street
Fresno, CA 93721

RE: Taylor Property Lot 10

Enclosed are the analytical results for samples received by our laboratory on 05/15/07 . For your reference, these analyses have been assigned laboratory work order number 7E15023.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety. Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

Moore Twining Associates, Inc.

Ronald J. Boquist
Director of Analytical Chemistry

Figure 14

MOORE TWINING ASSOCIATES, INC.
EST. 1898 ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: Taylor Property Lot 10 Project Number: D69201.01 Project Manager: Ken Clark	**Reported:** 05/31/2007

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
TP-5 6-18 inches BSG	7E15023-01	Soil	05/15/07 15:12	05/15/07 15:12
TP-12 24-36 inches BSG	7E15023-02	Soil	05/15/07 15:12	05/15/07 15:12
TP-12 48-60 inches BSG	7E15023-03	Soil	05/15/07 15:12	05/15/07 15:12

Moore Twining Associates, Inc.

Ronald J. Boquist, Director of Analytical Chemistry
James H. Brownfield, Quality Assurance Manager

The results in this report apply to the samples analyzed in accordance with the chain custody document. This analytical report must be reproduced in its entirety.

MOORE TWINING ASSOCIATES, INC.
EST. 1898 ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: Taylor Property Lot 10 Project Number: D69201.01 Project Manager: Ken Clark	**Reported:** 05/31/2007

TP-5 6-18

7E15023-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
LOI (% Organic Matter)	**4.8**	0.10	%	T7E2906	05/29/07	05/30/07	ASTM D2974	

Moore Twining Associates, Inc.

Ronald J. Boquist, Director of Analytical Chemistry
James H. Brownfield, Quality Assurance Manager

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MOORE TWINING ASSOCIATES, INC.
CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: Taylor Property Lot 10 Project Number: D69201.01 Project Manager: Ken Clark	**Reported:** 05/31/2007

TP-12 24-36

7E15023-02 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	**640**	60	mg/kg	T7E2209	05/22/07	05/22/07	ASTM D-4327-84	
Chloride	**0.064**	0.0060	% by Weight	[CALC]	05/22/07	05/22/07	ASTM D4327-84	
Sulfate as SO4	**0.073**	0.0060	% by Weight	[CALC]	05/22/07	05/22/07	ASTM D4327-84	
pH	**6.8**	0.30	pH Units	T7E2209	05/22/07	05/22/07	ATSM D4972-89 Mod	
Resistivity	**690**		ohms/cm	T7E2209	05/22/07	05/22/07	ASTM D1125-82	
Sulfate as SO4	**730**	60	mg/kg	T7E2209	05/22/07	05/22/07	ASTM D4327-84	

Moore Twining Associates, Inc.

Ronald J. Boquist, Director of Analytical Chemistry
James H. Brownfield, Quality Assurance Manager

The results in this report apply to the samples analyzed in accordance with the chain custody document. This analytical report must be reproduced in its entirety.

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MOORE TWINING ASSOCIATES, INC.

CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: Taylor Property Lot 10 Project Number: D69201.01 Project Manager: Ken Clark	**Reported:** 05/31/2007

TP-12 48-60

7E15023-03 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	**180**	60	mg/kg	T7E2209	05/22/07	05/22/07	ASTM D-4327-84	
Chloride	**0.018**	0.0060	% by Weight	[CALC]	05/22/07	05/22/07	ASTM D4327-84	
Sulfate as SO4	**0.54**	0.030	% by Weight	[CALC]	05/22/07	05/22/07	ASTM D4327-84	
pH	**6.1**	0.30	pH Units	T7E2209	05/22/07	05/22/07	ATSM D4972-89 Mod	
Resistivity	**370**		ohms/cm	T7E2209	05/22/07	05/22/07	ASTM D1125-82	
Sulfate as SO4	**5400**	300	mg/kg	T7E2209	05/22/07	05/22/07	ASTM D4327-84	

Notes and Definitions

ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
RPD	Relative Percent Difference

Quality Control Data Available Upon Request

APPENDIX D

APPENDIX D

PHOTOGRAPHS

Photograph No. 1: View to northwest toward site. Wooded hillside with lots 8 and 10 (Lot 10 includes existing residence) in mid-ground with Quail Lodge Resort golf course beyond.

Photograph No. 2: Drilling at test boring B-2.

Photograph No. 3:   Drilling at test boring B-5, Lot 1.

Photograph No. 4: Test pit 4, Lot 4. A horizon soil underlain by E horizon soils, underlain by lean clay with abundant siltstone and porcelanite fragments.

Photograph No. 5: Test pit 12, Lot 1. Slickensides in blocky clay at about 4 feet below site grade.

Photograph No. 6: Test pit 13, Lot 9.

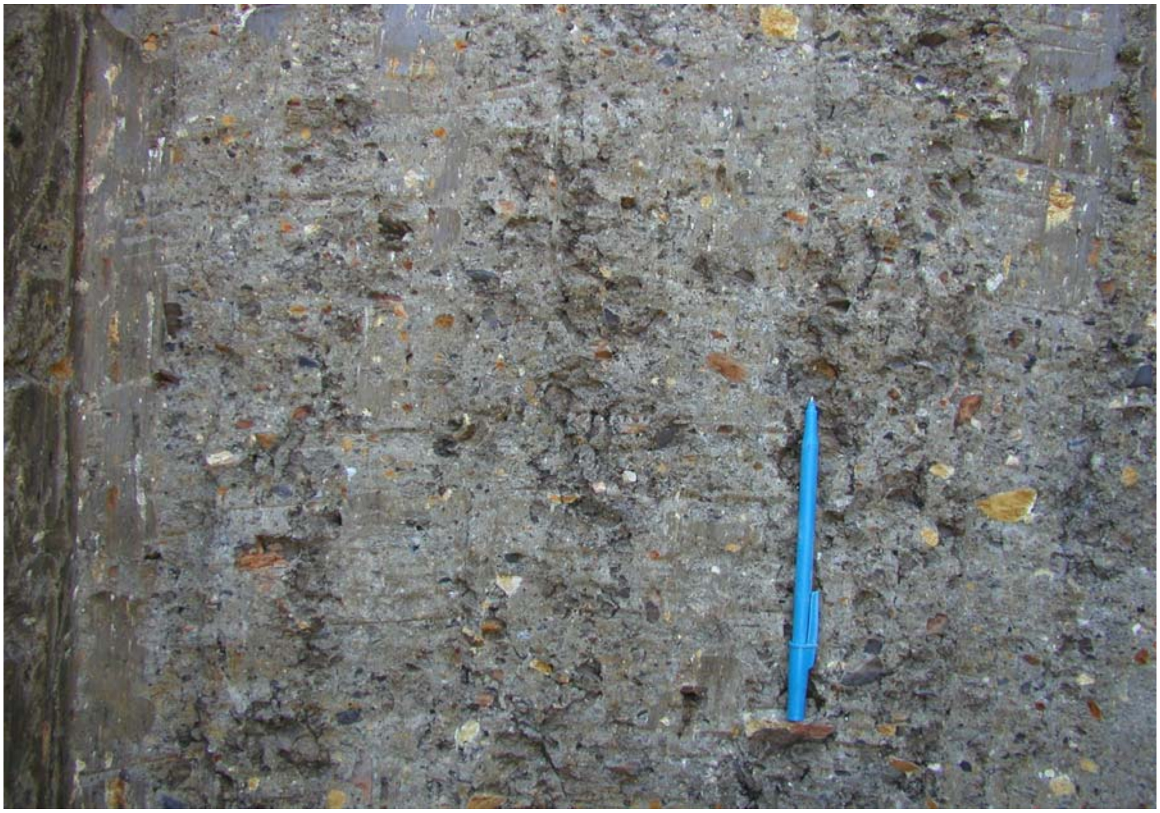

Photograph No. 7: Test pit 13, Lot 9. Pen at clay slip surface.


Photograph No. 8: Test pit 13, Lot 9. Head of pen on clay slip surface.

Photograph No. 9:   Test pit TP-1, Lot 5. Pick head at bottom of grayish tan slit with abundant siltstone and porcelanite fragments. Silt is underlain by lean clay with siltstone and porcelanite fragments and intensely fractured and sheared siltstone below the pick handle.

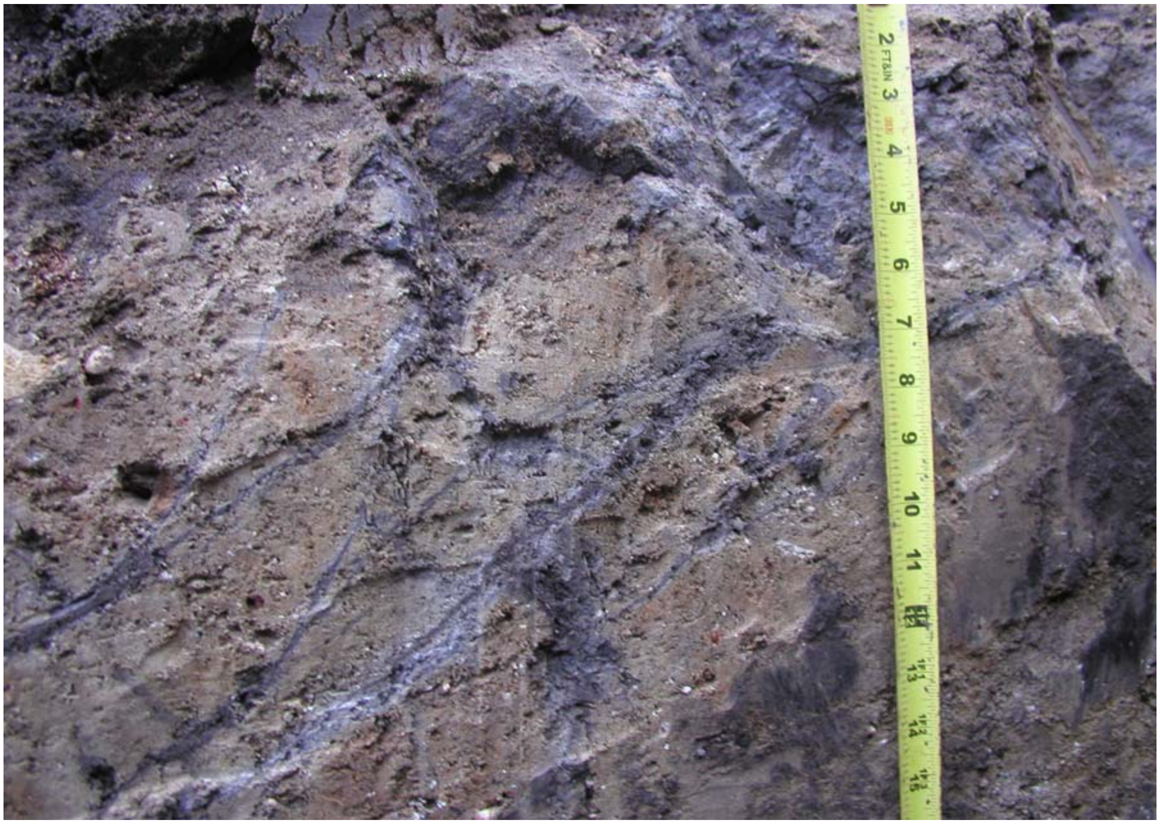
Photograph No. 10: Sheared Monterey Formation siltstone, Test Pit 11, Lot 7. Ground surface at top of photograph.

Photograph No. 11: Test pit 12, Lot 1. Approximately 2 to 3 foot thick A horizon top soil exposed at top of trench. Note granodiorite exposed near bottom of test pit.



Photograph No. 12: Test pit 4, Lot 4. Lean clay with abundant siltstone and porcelanite fragments.

Photograph No. 13: Surface condition near headscarp of small slide Lot 3.



Photograph No. 14: Clay shear surfaces exposed in TP-5, Lot 3.

Photograph No. 15: Clay shear surfaces exposed in TP-5, Lot 3.

Photograph No. 16: View toward the south across Lot 5 home site area and test pit TP-2.


Photograph No. 17: View toward west across Lot 6 showing test pits TP-6 and TP-7.

Photograph No. 18: View toward east across Lot 2.

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