PRELIMINARY GEOTECHNICAL REPORT
US101 PALO COMADO CANYON ROAD INTERCHANGE IMPROVEMENT PROJECT
BRIDGE NO. 53-1678, 07-LA-101 PM 33.0-34.4
CITY OF AGOURA HILLS, CALIFORNIA
CALTRANS EA NO. 07-257200

Prepared for:

KIMLEY-HORN AND ASSOCIATES, INC.
11060 White Rock Road, Suite 150
Ranco Cordova, California 95670

Prepared by:

KLEINFELDER WEST, INC.
2 Ada, Suite 250
Irvine, California 92618
Phone (949) 727-4466, Fax (949) 727-9242

Kleinfelder Project No. 106226

February 18, 2011

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February 18, 2011
Project No. 106226

Kimley-Horn and Associates, Inc.
11060 White Rock Road, Suite 150
Rancho Cordova, California 95670

Attention: Mr. Robert Blume, PE

Subject: Preliminary Geotechnical Report
US101 Palo Comado Canyon Road Interchange Improvement Project
Bridge No. 53-1678, 07-LA-101 PM 33.0-34.4
City of Agoura Hills, California
Caltrans EA# 07-257200

Dear Mr. Blume:

Kleinfelder West, Inc. (Kleinfelder) is pleased to present this Preliminary Geotechnical Report (PGR) for the proposed US101 Palo Comado Canyon Road interchange improvement project. This report has been prepared for the Project Report/Environmental Document (PR/ED) phase of the project. Presented herein is geotechnical information to be used for preliminary evaluation of the proposed project. This report is intended for use by the Project Development Team (PDT) to assess potential impacts and estimate construction costs. Review comments prepared by Caltrans dated February 2, 2011 have been incorporated into this report as applicable. A copy of the Caltrans comments and our responses to the comments are provided in Appendix C of this report.

The future designer will be responsible for performing additional geotechnical investigations necessary to meet Caltrans standards for PS&E design-level geotechnical reports. Revisions or modifications to our findings and recommendations may be required following completion of supplemental investigation and analysis during the design-level phase of this project.

We appreciate the opportunity to be of service on this project. If you have any questions, comments or require additional information, please do not hesitate to contact the undersigned.

Sincerely,

KLEINFELDER WEST, INC.

Scott G. Lawson, PE, GE
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
SCOTT G. LAWSON
No. GE 2829
Exp. 9/30/11
GEOTECHNICAL
STATE OF CALIFORNIA

Robert E. Lemmer, PG, CEG, CHG
Senior Engineering Geologist

PROFESSIONAL GEOLOGIST
ROBERT E. LEMMER JR.
NO. 2265
CERTIFIED ENGINEERING GEOLOGIST
EXP. 11-30-12
STATE OF CALIFORNIA

2 Ada, Suite 250, Irvine, CA 92618 p | 949.727.4466 f | 949.727.9242

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1.0 INTRODUCTION

1.1 Background

The California Department of Transportation (Caltrans) and The City of Agoura Hills (City), propose to construct improvements at the US101/Palo Comado Canyon Road interchange (PM 33.0/34.4), in Los Angeles County in the City of Agoura Hills (see Plate 1, Site Location Map). The project includes widening the Palo Comado Canyon Road and Palo Comado Canyon Road Overcrossing over US101 and modification of the interchange ramps in order to improve traffic circulation, safety, and bicycle/pedestrian access.

The purpose of the project is to:

- Reduce existing and forecasted traffic congestion within the project limits;
- Improve circulation at the US101/Palo Comado Canyon Road interchange and adjacent roadway network;
- Improve safety at the US101/Palo Comado Canyon Road interchange; and
- Accommodate pedestrian and bicycle traffic along Palo Comado Canyon Road.

1.2 Purpose and Scope of Services

The purpose of this desktop study is to evaluate available geotechnical and geologic information and assess the impacts of existing conditions upon the proposed improvements, as well as potential design and construction requirements. This report provides preliminary geotechnical data for use by the Project Development Team (PDT) to assess potential impacts and estimate construction costs.

Additional geotechnical investigations and the preparation of a Geotechnical Design Report (GDR), Materials Report (MR), and Structure Foundation Report (SFR) for the bridge widening, non-standard retaining walls or other structures to meet Caltrans requirements of Type Selection and Plans, Specifications and Estimates (PS&E) level geotechnical design studies will be required.

Proposed improvements considered in this report include widening of the existing bridge (bridge No. 53-1678), new retaining walls, grading for bridge approach embankments and lane and shoulder additions, ramp modifications, and new pavement.

Kleinfelder has also prepared a Draft Initial Site Assessment (ISA) and Preliminary Materials Report (PMR) for the project, which are provided separately.

2.0 EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

2.1 Existing Facilities

The US101/Palo Comado Canyon Road Overcrossing (also known as Chesebro Road Overcrossing, Bridge No. 53-1678) is a four-span bridge built in 1963. It provides one 12-foot lane and 4-foot shoulder in each direction. A 5-foot sidewalk is provided on the west side of the overcrossing. The minimum vertical clearance is 15 feet, which is located in the northeast corner of the structure over the northbound US101 freeway number four lane. The bridge was constructed with pre-stressed pre-cast “I” girders on three-column reinforced concrete bents and diaphragm abutments. The bridge has a total width of approximately 40 feet and a total length of approximately 234 feet measured along the bridge centerline.

The Palo Comado Canyon Road intersection with the US101 freeway is a non-standard interchange, with the existing southbound on- and off-ramps terminating a block west of the bridge on Dorothy Drive and Chesebro Road. The interchange is configured with tight diamond (L-1) ramps on the northbound side and hook ramps (L-6) on the southbound side.

2.2 Proposed Improvements

The proposed improvements within the project limits include a No Build alternative and one Build alternative described in the following sections. The No Build alternative provides a baseline for comparing the impacts associated with the Build alternatives since environmental reviews must consider the effects of not implementing the proposed project.

The Build alternative will include widening the entire length of Palo Comado Canyon Road, between Driver Avenue to the north and Chesebro Road to the south; from two to four lanes (see Proposed Build Alternative exhibit in Appendix A). Within these limits, the Palo Comado Canyon Road Overcrossing will be widened from one lane in each direction to provide two lanes in each direction, along with a dedicated lefthand turn lanes, for a total of five striped lanes. A Class II bike lane and sidewalks will be provided on both sides of the overcrossing. Based on the Advanced Planning Study drawing presented in Appendix A, the widened bridge will be constructed with pre-stressed pre-cast “I” girders. The widened bridge will have a total length of approximately 234 feet measured along the centerline of the bridge and have a maximum width of approximately 90 feet.

The Build alternative will maintain the existing layout of the interchange ramps; however, the northbound on- and off-ramps will be slightly re-configured, with an additional lane being provided on the northbound off-ramp at the Palo Comado Canyon Road intersection. The intersection of the northbound ramps and Palo Comado Road will be signalized; the remaining intersections will remain un-signalized.

Existing utilities will be protected in place during construction. The existing storm drain systems will remain in place. New inlets will be installed along the modified northbound off-ramp, as well as the northbound on-ramp. A new inlet system will be added to accommodate the widening of Palo Comado Canyon Road south of the bridge.

3.0 PERTINENT REPORTS AND INVESTIGATIONS

We reviewed the following pertinent reports and documents for this study:

- Project Study Report to Request for Conceptual Approval and Programming for Capital Cost, On Route US101 Between 0.9 mile West of Liberty Canyon Road and 1.3 mile East of Kanan Road, prepared by Parsons Transporation Group, Inc. for the City of Agoura Hills, dated February 2009.
- Preliminary Foundation Report, Palo Comado OC a.k.a. Chesebro Rd OC (Widen), Bridge No. 53-1678, 07-LA-101-PM 33.69, Agoura Hills, California, prepared by Group Delta Consultants, Inc., revision dated February 5, 2009, GDC Project No. L-783.

Additional references used are listed at the end of this report.

4.0 EXCEPTIONS TO POLICY

No exceptions to policy were taken for the preparation of this report. A Preliminary Foundation Report (PFR) was previously prepared by Group Delta Consultants, Inc. (Group Delta, 2009) for the proposed widening of Palo Comado Canyon Road OC (Bridge No. 53-1678). The PFR was revised based on review comments provided by Caltrans.

After the preparation of that PFR, Caltrans released the new 2009 Seismic Design Criteria, Appendix B. We have developed recommended seismic design parameters in accordance with new 2009 Seismic Design Criteria, Appendix B which may be used in lieu of the seismic design parameters presented in the Group Delta PFR. Our seismic design recommendations are presented in Section 10 of this report.

5.0 FIELD INVESTIGATION AND TESTING PROGRAM

No subsurface exploration was performed for this study.

6.0 LABORATORY TESTING PROGRAM

No laboratory testing was performed for this study.

7.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

7.1 Geologic Setting

The site is located at the northern flank of the Santa Monica Mountains within the Transverse Ranges Geomorphic Province of California. The Transverse Ranges Province is characterized by roughly east-west trending, convergent structural features, such as, folding and reverse/thrust faulting, in contrast to the predominant northwest-southeast strike-slip structural trend in the other geomorphic provinces in California (California Geological Survey [CGS], 2002). The convergent deformational features of the Transverse Ranges are a result of north-south crustal shorting due to plate tectonics.

Compressive folding results in the local uplift of the mountains and lowering of the intervening valleys, along with propagation of reverse/thrust faults (including blind thrusts) and filling of the valley basins with alluvial sediments.

The primary geologic units comprising the foothills bordering the project area include the middle Miocene age Topanga Group (11 to 16 million years) and the younger, late Miocene age Modelo Formation (5 million years old). The Topanga Group is comprised of approximately 19,700 feet (6,000 meters) of sedimentary and volcanic rock, including the Conejo Volcanics, Topanga Canyon and Calabasas Formations (Yerkes and Campbell, 2005; Loyd, 2002). The Modelo Formation generally overlies the Calabasas Formation unconformably, but is often adjacent to the Calabasas Formation where there is faulting.

7.2 Site Seismicity

The controlling fault with respect to the deterministic analysis is the Santa Monica fault (Fault ID No. 280), located approximately 7.0 miles (11.3 kilometers) due south of the site (Caltrans, 2009a). The Santa Monica fault is a reverse fault (R) capable of generating a maximum magnitude (Mw) 7.0 earthquake. The fault dip is 50 degrees and the dip direction is north.

The nearest active fault (i.e., a fault along which displacement has occurred within the past 11,000 years) is the Chatsworth fault, located approximately 5.3 miles (8.5 km) to the northeast of the site (Caltrans, 2009a). The Chatsworth fault is a reverse fault (R) and has a maximum magnitude (M_{Max}) of 6.6.
The Malibu Coast fault, located approximately 6.8 miles (11.0 kilometers) south of the site and is a left lateral strike-slip (LLSS) with a maximum magnitude (M_{Max}) of approximately 6.7 (Caltrans, 2009a).

7.3 Subsurface Conditions

No subsurface exploration or laboratory testing were performed for this study. The subsurface conditions at the site were preliminarily evaluated by reviewing the as-built Log of Test Borings (LOTBs) for Palo Comado OC (formerly Chesebro Road Overcrossing, Bridge No. 53-1878) and other available geologic literature. The borings shown on the LOTBs (presented in Appendix B) were drilled to depths ranging from approximately 15 to 40 feet below ground surface, corresponding to the lowest elevation explored of approximately 870 feet above Mean Sea Level (msl).

Based on the as-built plans and the geologic data reviewed, the project area is underlain by artificial fill, alluvium, and sedimentary bedrock belonging to the middle Miocene-age Calabasas Formation (Yerkes and Campbell, 2005). The lithologic units in the project area are shown on Plate 2, Geologic Map. Abutments 1 and 5 were constructed on artificial fill that is as thick as 10 to 20 feet below the bottom of footings, while the footings for Bents 2, 3 and 4 were constructed 5 to 10 feet beneath orginal grade.

7.3.1 Artificial Fill

The uppermost layer of soil beneath the bridge abutments generally consists of artificial fill. No information is currently available regarding the composition of the artificial fill. The artificial fill appears to have been placed during construction of the existing bridge over US101. Artificial fill may have also been placed during construction of the associated on- and off-ramps and surrounding developments.

7.3.2 Alluvial Deposits

The alluvial deposits underlying the project area were deposited along the margin of the active drainage of Cheseboro Canyon and are comprised of loose to moderately dense sand, silty sand, clayey silt and sandy silt (Wills and Barrows, 1997; Yerkes and Showalter, 1993). Based on the data reviewed, the alluvial deposits are as thick as 5 to 10 feet.

7.3.3 Calabasas Formation

Based on the LOTBs and the literature reviewed, the depth to bedrock is approximately 0 to 25 feet (an approximate elevation of 900 feet msl) beneath the existing bridge footings. The primary lithology underlying the project area is marine claystone, siltstone and shale of the Calabasas Formation. In other areas near the site, the Calabasas Formation consists of interbedded clayey to silty sandstone, siltstone, and silty shale with some local bed of breccia (Yerkes and Campell, 2005; Yerkes and Showalter, 1993).

The slopes adjacent to NB Palo Comado and the NB off-ramp of the US101 are mapped as Calabasas Formation (see Plate 2).

7.3.4 Modelo Formation

Modelo Formation does not directly underly the project area, but is mapped in the slopes immediately to the east near the US101. The Modelo Formation is marine sedimentary rock, generally consisting of thin bedded, diatomaceous mudstone, clay shale, and siltstone with some interbedded very fine- to coarse-grained sandstone (Yerkes and Campbell, 2005; McCrink et al., 1997; Yerkes and Showalter, 1993).

The Modelo Formation is susceptible to slope failures and many slopes with exposures of this formation are delineated as a landslide-prone area on the Calabasas quadrangle Seismic Hazard Zones (CGS, 1998) and discussed in the Seismic Hazard Zone report (McCrink et al., 1997).

7.4 Groundwater

Groundwater was not encountered in the borings performed at the site in June 1961 to the lowest elevation explored of approximately 870 feet above msl, corresponding to a depth of approximately 40 feet below US101 at the bridge site. Pile logs peformed during original construction of the bridge in 1962 indicate that perched groundwater was encountered in the shale in one boring at an approximate elevation of 887 feet msl, corresponding to an approximate depth of approximately 25 feet below US101.

The historic shallow groundwater level at the site is unknown based on the data reviewed. According to the California Geological Survey (Wills and Barrows, 1997), the historic shallow groundwater in the vicinity of the site is about 20 feet below grade.

Fluctuations of the groundwater level, localized zones of perched water, and variations in soil moisture content should be anticipated during and following the rainy season (late fall to early spring). Irrigation of landscaped areas on and adjacent to the site can also cause a fluctuation of local groundwater levels.

8.0 SCOUR AND EROSION

8.1 Scour

The proposed widened bridge will span US101. The proposed retaining wall alignments will not cross any significant drainage courses. Therefore, scour potential is not considered a design issue.

8.2 Erosion

Construction of the various elements of the project will likely result in alteration of the landform due to grading. Landform alterations may create erosional impacts to the existing terrain. The more extensive alterations will be from construction of the cuts for retaining walls and fill slopes associated with the ramp and roadway widening. Applying standard engineering techniques during design and construction to prevent erosion can mitigate these impacts. Typical erosion control mitigation measures include improved drainage control and implementation of landscaping after construction.

9.0 CORROSION EVALUATION

No corrosion test data is available at this time. Corrosion potential should be evaluated as part of site-specific geotechnical investigations during the final design phase of the project. Corrosion data should be analyzed and evaluated by qualified engineers with experience in corrosion protection.

Section 4.1 of the "Corrosion Guidelines" prepared by the Corrosion Technology Branch, Caltrans Office of Engineering and Testing Services (September 2003) defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2,000 ppm of sulfates, or has a pH of less than 5.5.

10.0 PRELIMINARY SEISMIC RECOMMENDATIONS

The most significant geologic hazard to this project is the potential for moderate to severe seismic shaking that is likely to occur during the design life of the proposed bridge structure. The bridge is located in the seismically active southern California region within the influence of several fault systems that are considered to be active or potentially active. These active and potentially active faults shown in Plate 3, Regional Fault Map, are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience strong ground acceleration as the result of moderate to large magnitude earthquakes.

10.1 Ground Surface Fault Rupture

The site is not located within a currently delineated State of California Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). No known active faults have been identified on the site, thus, the potential for future surface fault rupture at the site is considered to be “low.” While fault rupture would most likely occur along previously established fault traces, future fault rupture could occur at other locations.

10.2 Seismic Shaking

Seismic design parameters were presented in the Preliminary Foundation Report prepared by Group Delta Consultants, Inc. for the proposed bridge widening (Group Delta, February 5, 2009). Subsequent to that report, Caltrans released the 2009 Seismic Design Criteria, Appendix B (Caltrans 2009a, b, c and d). The Caltrans Implementation memorandum dated August 6, 2009 requires that the 2009 Seismic Design Procedures be used for bridges that receive type selection approval after September 30, 2009. We have developed the following recommended seismic design parameters in accordance with new 2009 Seismic Design Criteria, Appendix B.

The project site is located in a seismically active region. Based on mapping by the California Geologic Survey (Bryant and Hart, 2007), United States Geologic Survey (Yerkes and Campbell, 2005) and on the Caltrans ARS Online website (Caltrans, 2009a), the Santa Monica fault (fault database ID No. 280) has a rupture distance of approximately 7.0 miles (11.3 km) south of the bridge site and is the governing fault for deterministic seismic hazard analysis. According to the Caltrans (Caltrans, 2009b) fault database and errata report, this fault is reverse (R) fault dipping 50 degrees with an assigned Maximum Magnitude (M_{Max}) of 7.0. Additional fault characteristics are summarized in Table 1.

No exploratory borings were performed for the preparation of this report. Our preliminary estimate of the shear wave velocity in the upper 30 meters (V_{s30}) for the site is based on data presented in the as-built LOTB sheets, our literature review and the guidelines set forth in the Caltrans Geotechnical Services Manual version 1.0 (Caltrans 2009b). The site is not located within a California deep soil basin region as defined by Caltrans (2009a and c). Site characteristics are summarized in Table 1 below.

Table 1
Site Characteristics and Governing Fault Parameters

Site Coordinates	Latitude = 34.1431 degrees, Longitude = -118.7381 degrees
Shear Wave Velocity [1], $V_{s(30)}$	440 m/s
Depth to V_s=1.0 km/s, $Z_{1.0}$	168 m
Depth to V_s=2.5 km/s, $Z_{2.5}$	2 km
Fault Name and ID Number	Santa Monica fault, ID No. 280
Maximum Magnitude (M_{Max})	7.0
Fault Type	Reverse
Fault Dip	50 degrees
Dip Direction	North
Bottom of Rupture Plane	12 km
Top of Rupture Plane (Z_{tor})	0 km
R_{RUP} [2]	11.3 km
R_{jB} [3]	3.8 km
R_X [4]	14.7 km
F_{norm} (1 for normal, 0 for others)	0
F_{rev} (1 for reverse, 0 for others)	1
Peak Ground Acceleration (PGA)	0.50 (based on probabilistic response spectrum)

Notes:
[1]$V_{s(30)}$ estimate based on correlation between as-built LOTB SPT blowcounts and $V_{s(30)}$ (Caltrans, 2009b).
[2]R_{RUP} = Closest distance from the site to the fault rupture plane.
[3]R_{JB} = Joyner-Boore distance; the shortest horizontal distance to the surface projection of the rupture area.
[4]R_X = Horizontal distance from the site to the fault trace or surface projection of the top of the rupture plane.

10.3 Design Response Spectra

The deterministic response spectrum was calculated using the Caltrans Deterministic Spreadsheet (version dated 7/28/09) and checked using ARS Online as required by Caltrans (2009b). The probabilistic response spectrum was developed using the Caltrans Probabilistic Spreadsheet (version dated 8/4/09), and compared with results from ARS Online as required by Caltrans (2009b). The near-fault and basin amplification factors were applied as necessary to both the deterministic and probabilistic spectra.

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The probabilistic response spectrum was found to govern at this site since it was greater than the deterministic spectrum at all periods. The recommended design spectrum is presented graphically on Plate 4A and numerically on Plate 4B.

10.4 Liquefaction Potential

Seismically induced soil liquefaction generally occurs in loose, saturated, cohesionless soil when pore pressures within the soil increase during ground shaking. The increase in pore pressure transforms the soil from a solid to a semi-liquid state. The primary factors affecting the liquefaction potential of a soil deposit are: 1) intensity and duration of earthquake shaking, 2) soil type and relative density, 3) overburden pressures, and 4) depth to groundwater. Soils most susceptible to liquefaction are clean, loose, uniformly graded, fine-grained sands, and non-plastic silts that are saturated. Silty sands have also been shown to be susceptible to liquefaction. These soils typically lose a portion or all of their shear strength and regain strength sometime after shaking stops. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils and the reduced shear resistance of slopes.

According to the California Geological Survey (CGS, 1998), the site is located in a liquefaction hazard zone. We performed a preliminary screening level evaluation of the liquefaction potential at the project site using subsurface data from the as-built LOTBs and the simplified liquefaction analysis procedure by Youd and Idriss (Youd et al., 2001). Based on an assumed historical high groundwater elevation of approximately 890 feet msl and the density and composition of the subsurface materials (i.e., bedrock) below this elevation, it is our professional opinion that the potential for liquefaction to impact site is low.

The potential for liquefaction to occur at the project site should be re-evaluated during a later design phase of the project after the completion of exploratory borings at the site and laboratory testing in order to confirm or, if necessary, modify the conclusions presented herein.

10.5 Seismic Slope Stability

The embankment slopes at the abutments were engineered during bridge construction and are not located in an area mapped as an Earthquake-induced landslide hazard zone (CGS, 1998; McCrink et al., 1997); however, immediately east of Palo Comado OC the slopes consisting of bedrock from the Modelo Formation are mapped as an Earthquake-induced landslide hazard zone. Seismic slope stability at the abutments is not anticipated to be a significant concern. Stability of the abutment slopes should be evaluated in the Structure Foundation Report during a later design phase.

Based on Caltrans Guidelines for Structure Foundation Reports (Caltrans, 2009e), a seismic coefficient, k_h = 1/3 x Horizontal PGA and no more than 0.2g should be used in a pseudo-static slope stability analysis.

11.0 BRIDGE FOUNDATIONS

11.1 As-Built Bridge Foundation Data

The existing bridge is supported on 16-inch diameter cast-in-drilled-hole (CIDH) piles at the abutments and bents. The piles have a design compressive capacity of 90 kips per pile. The average pile length at the abutments is approximately 38 feet. The average pile length at the bents ranges from approximately 13 to 23 feet. Additional details regarding the existing bridge foundations is presented in the Preliminary Foundation Report prepared by Group Delta (2009).

The as-built plans indicate the existing embankments at the abutments have an approximate slope gradient of $1^{1}/_{2}$:1 (Horizontal:Vertical, H:V) beneath the bridge. The slopes beneath the bridge are un-paved.

11.2 Preliminary Bridge Foundation Recommendations

The Preliminary Foundation Report prepared by Group Delta recommends that the proposed bridge widening be supported by 16-inch diameter CIDH piles similar to the existing foundations. Based on our review of the Group Delta PFR, the results of our literature review and preliminary analysis performed for this report, and our current understanding of the proposed project, we generally concur with the bridge foundation recommendations provided in the Group Delta PFR. However, we recommend that the updated seismic design parameters provided in Section 10 of this report be used in lieu of those presented in the Group Delta PFR.

12.0 CUTS AND EXCAVATIONS

12.1 Cut Slopes

We anticipate that no permanent significant cut slopes will be required for the proposed improvements. Areas where the proposed ramp or roadway widening will encroach into existing slopes will be accommodated by the construction of new retaining walls.

South and West facing slopes adjacent to the northbound (NB) US101 off-ramp and NB Palo Comado north of the US101 are potentially unstable. However, based on the geologic maps reviewed, regional bedding dips steeply northeast, which is favorable with respect to the stability of the temporary cut slopes that may be required for retaining wall construction. Although bedding appears favorable, the presence of adversely oriented joints or discontinuities is uncertain. In later phases of the project, subsurface exploration, geologic mapping, and laboratory testing should be performed to characterize the geotechnical conditions (i.e., rock mass or soil strength, geologic structure, subsurface profile, and groundwater elevation) at these locations for use in further detailed analyses and design.

12.2 Excavation Characteristics

The rippability and excavatibility of rock is complex and largely governed by weathering, rock strength, and the nature of the discontinuities within the rock mass that can vary depending upon location and depth. In general, we anticipate the earth materials and artificial fill along the project alignment can be excavated using conventional earth moving equipment.

13.0 EMBANKMENTS/FILL SLOPES

New roadway embankments and fill slopes will be required for various portions of the project. Topic 304.1 of the Caltrans Highway Design Manual (2006) states that,

> *"For new construction, widening, or where slopes are otherwise being modified, embankment (fill) slopes should be 4:1 or flatter."*

We understand that embankment fill slopes with a gradient of 2:1 (H:V) may be planned for some areas of the project alignment. According to the Highway Design Manual, slopes steeper than 4:1 (H :V) must be approved by the District Landscape Architect. From a preliminary geotechnical design standpoint, the embankments and fills may be planned with slopes constructed at a slope gradient of 2:1 (H:V) or flatter. Embankment slopes may be designed at a gradient steeper than 2:1 (H:V) using soil reinforcement or engineered buttresses. Such embankment slopes should be evaluated during a later design phase.

All fill soils used for roadway embankments should be nearly free of organic or other deleterious debris. All material used for roadway embankments should meet the requirements outlined in Section 19 of the Caltrans Standard Specifications. A project specific Aerially Deposited Lead (ADL) Survey Report should be performed to identify native materials that require special handling and/or disposal at a permitted facility.

All imported borrow materials to be used for engineered fill should be sampled and tested by the project Geotechnical Engineer during later stages of the project prior to being approved for use along the alignment. Embankments should also be constructed in accordance with Section 19 of the Caltrans Standard Specifications. Aside from materials derived from the Calabasas Formation siltstone or shale lithofacies, which are fine-grained and potentially expansive, we anticipate native materials excavated along the alignment may be suitable for use as compacted fill. More detailed recommendations may be presented following a design level geotechnical investigation.

14.0 EARTH RETAINING SYSTEMS

Multiple retaining walls are planned for the proposed interchange improvement project. Preliminary locations, range of wall heights, anticipated foundation conditions for the proposed retaining walls along the alignment are summarized in Table 2. It is our understanding that the retaining wall locations presented in Table 2 and the Proposed Build Exhibit in Appendix A are approximate and subject to change as final revisions are made to the proposed interchange improvements.

Based on our preliminary estimate of the strength of the earth materials within the project alignment and the maximum wall heights and loading conditions, we have estimated the wall/foundation types that will likely be feasible for the proposed retaining walls. Depending on the final retaining wall heights and locations, deep foundations may be required for some retaining walls.

14.1 Feasible Retaining Wall Options

We anticipate that the proposed retaining walls can be constructed as Caltrans Standard Type 1 cantilevered retaining walls. For retaining walls constructed in cut conditions, standard Caltrans Type 1 walls are assumed to be constructable provided a temporary cut to construct the wall from the bottom up is feasible. Excavations should be performed in accordance with Caltrans Standard Specifications Section 19-3 (2006b) and Standard Plan A62B (2006c). All trenches and temporary excavations should be excavated in accordance with CALOSHA Construction Safety Orders and the Caltrans Trenching and Shoring Manual (2000) safety requirements.

For the proposed retaining wall located on the NB US101 off-ramp, an anchored retaining wall with a top-down construction method (i.e. soil nail or tieback wall) is also considered feasible. The final wall type will depend on the results of further analysis during the PS&E phase of the project and a cost and constructability comparison between an anchored retaining wall and a cast-in-place (CIP) retaining wall.

Shallow foundations are anticipated to be suitable for proposed Type 1 retaining walls 12-feet tall or less founded in alluvium or compacted fill materials. Sections of walls that are greater than 12-ft tall may require deep foundations. Similarly, for proposed Type 1 retaining walls founded in bedrock materials and 18-feet tall or less, shallow foundations are anticipated to be suitable, while sections that are greater than 18-ft tall may require deep foundations.

Because of the anticipated shallow depth to bedrock in some locations and the potential for difficult pile-driving conditions anticipated in this material, cast-in-drilled-hole (CIDH) piles will likely be required rather than driven piles. Retaining wall design should follow procedures described in Chapter 5 of Caltrans Bridge Design Specifications (2004) for gravity and semi-gravity walls.

During the final design phase of the project, site-specific geotechnical investigation and laboratory testing should be performed at proposed retaining wall locations to confirm the anticipated wall/foundation types provided in Table 2. Additional retaining walls not listed in Table 2 may be required during future phases of the project to allow for design optimization.

Table 2
Summary of Proposed Retaining Walls

Wall Location	Anticipated Wall Type[1]	Wall Grading	Anticipated Slope in Front of Wall (H:V)	Caltrans Load Case[2]	Approximate Wall Length (feet)	Wall Height Range (feet)	Anticipated Foundation Conditions	
							Materials[3]	Type
NB Palo Comado South of US101	Type 1	Fill	2:1	I	230	8 - 12	Af, Qa, Tcb	Spread Footings
SB Palo Comado South of US101	Type 1	Fill	2:1	I	535	7 - 16	Af, Qa, Tcb	Spread Footings/ Piles[4]
NB US101 Off-Ramp	Type 1 or Anchored	Cut	Level	I, II	920	2 - 26	Tcb	Spread Footings/ Piles[5] or Soil Nails/Tiebacks
NB Palo Comado North of US101	Type 1	Cut	Level	II	175	4 – 8	Tcb	Spread Footings
NB Palo Comado North of US101	Type 1	Fill	Level	I	220	1 – 3	Af, Qa, Tcb	Spread Footings
NB US101 On-Ramp (EAST)	Type 1	Fill	2:1	I	80	1 – 3	Af	Spread Footings
NB US101 On-Ramp (WEST)	Type 1	Fill	2:1	I	310	2 - 4	Af	Spread Footings
SB Palo Comado North of US101	Type 1	Fill	Level	I	200	1 – 4	Af, Qa	Spread Footings

Notes:

1. *Type 1 – Caltrans standard cantilever wall less than 36 ft; Anchored- retaining wall constructed using tiebacks or soil nails.*
2. *Load Cases: I – Level surface at crest of wall with constant surcharge of 240 pounds per square foot (psf); II – 2H:1V unlimited sloping surface at wall crest.*
3. *Af – artificial fill; Qa- alluvium; Tcb- Calabasas Formation siltstone or shale.*
4. *For retaining walls founded in alluvium and artificial fill, wall sections with retained heights greater than 12 ft may require pile supported foundations.*
5. *For retaining walls founded in competent bedrock material, wall sections with retained heights greater than 18 ft may require pile supported foundations.*

15.0 MATERIAL SOURCES

Limited borrow materials may be available from areas within the alignment with planned cuts. However, much of the cut material is anticipated to consist of Calabasas Formation siltstone and shale and soils derived from this geologic formation, which are generally unsuitable as engineered fill material. We anticipate that fill material needed to reach final design grades will have to be imported from off-site.

Kleinfelder has not conducted a preliminary search of borrow and disposal sites located in the vicinity of the proposed alignment. Additional investigation during later phases of design may be performed to identify potential borrow and also to evaluate the engineering properties and extent of suitable material for use as engineered fill for the project. Evaluation of disposal and borrow sites should be performed in accordance with Caltrans Design Information Bulletin 85 (Caltrans, 2007b). Construction materials such as aggregates, asphalt, Portland cement, and fly ash should be imported from local commercial sources.

16.0 CONSTRUCTION CONSIDERATIONS

16.1 Construction Considerations That Influence Design

The proposed interchange improvement project may be affected by or affect other projects planned in this area by the City of Agoura Hills, Caltrans, or other agencies.

Access for construction equipment must be planned which would allow for widening of the existing bridge and construction of the new retaining walls. Where applicable, falsework and shoring will also require additional consideration.

The feasibility of a temporary cut in the existing slope to construct a standard Caltrans Type 1 retaining wall along the NB US101 off-ramp should be further evaluated during the PS&E phase of design after the completion of a site-specific geotechnical investigation and laboratory testing.

16.2 Hazardous Waste Considerations

The scope of our geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands. A Draft Initial Site Assessment (ISA) for the project has been prepared under separate cover by Kleinfelder to address many of these issues.

We anticipate that an Aerially Deposited Lead (ADL) survey will be performed during a future phase of the project to provide recommendations for suitable locations within the project that lead-impacted soils may be re-used on-site. Such recommendations should be incorporated into the final plans and specifications for the project.

17.0 FUTURE GEOTECHNICAL INVESTIGATION

The data and preliminary assessment provided in this report summarize the geotechnical data reviewed and are intended to assist the Project Report team in preparing their preliminary engineering design and cost estimates. We recommend site-specific geologic and geotechnical investigations be completed to prepare a Structure Foundation Report (SFR), Geotechnical Design Report (GDR), and Materials Report (MR) in conformance with the latest Caltrans standards.

The geologic and geotechnical investigations should include in situ testing and laboratory testing to obtain site specific data appropriate for design. Surficial mapping should be performed in areas of retaining walls that will cut into existing hillsides. Typically, subsurface explorations are performed in areas where structures (e.g., retaining walls, bridge abutments, etc.) are proposed, where slope modifications are proposed, in areas of new pavement, or where unfavorable ground conditions are suspected.

In situ testing typically consists of standard penetration testing (SPT) and cone penetration testing (CPT), but can also include testing such as borehole shear test (BST), vane shear, pressure meter testing, permeability testing, and soil resistivity among others. For most highway projects, SPT and CPT data combined with laboratory testing is sufficient for design.

18.0 LIMITATIONS

This report has been prepared for the exclusive use of the City of Agoura Hills and its consultants for specific application to the proposed US101 Palo Comado Canyon Road Interchange Improvement project. It is intended solely for their use in the preliminary design of the project as described herein. It may not contain sufficient information for other uses or purposes of other parties. It is not considered sufficient for final design or construction of the project. The findings, conclusions, and recommendations presented in this report were prepared in a manner consistent with the standards of care and skill ordinarily exercised by members of its profession completing PR/ED studies practicing under similar conditions in the geographic vicinity and at the time the services were performed. No warranty or guarantee, express or implied, is made. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard. They are not incorporated into it or “included by reference,” as that latter term is used relative to contracts or other matters of law.

This report was based on the proposed project information provided to Kleinfelder. If any change (i.e., structure type, location, etc.) is implemented which materially alters the project, additional geotechnical services may be required, which could include revisions to the geotechnical recommendations presented herein.

This report may be used only by the client and only for the purposes stated within a reasonable time from its issuance, but in no event later than three years from the date of the report. Land or facility use, on and off-site conditions, regulations, design criteria, procedures, or other factors may change over time, which may require additional work. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold Kleinfelder harmless from any claim or liability associated with such unauthorized use or non-compliance.

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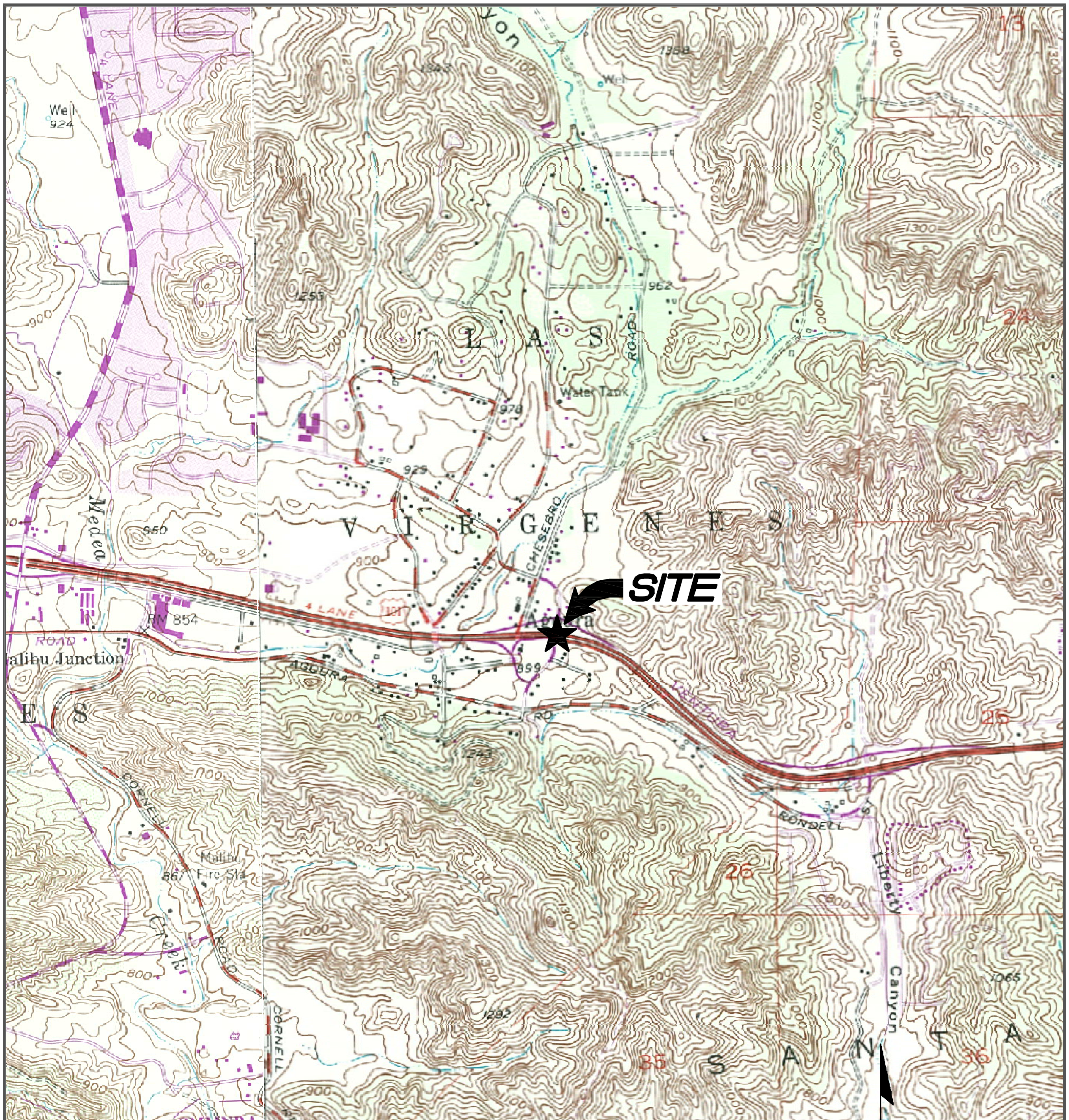
PLATES



SOURCE: U.S.G.S. 7.5' Topographic series, Calabasas and Thousand Oaks, California Quadrangle 1952 (1950), Photorevised 1967 (1981).

2,000 1,000 0 2,000

APPROXIMATE SCALE (feet)

PLOTTED: 13 Nov 2009, 3:12pm, MGriffin

CAD FILE: L:\2009\CADD\106226\ LAYOUT: 1

ATTACHED IMAGES: Images: topo-Calabasas.CA.JPG Images: topo-Thousand-Oaks.CA.JPG

ATTACHED XREFS:

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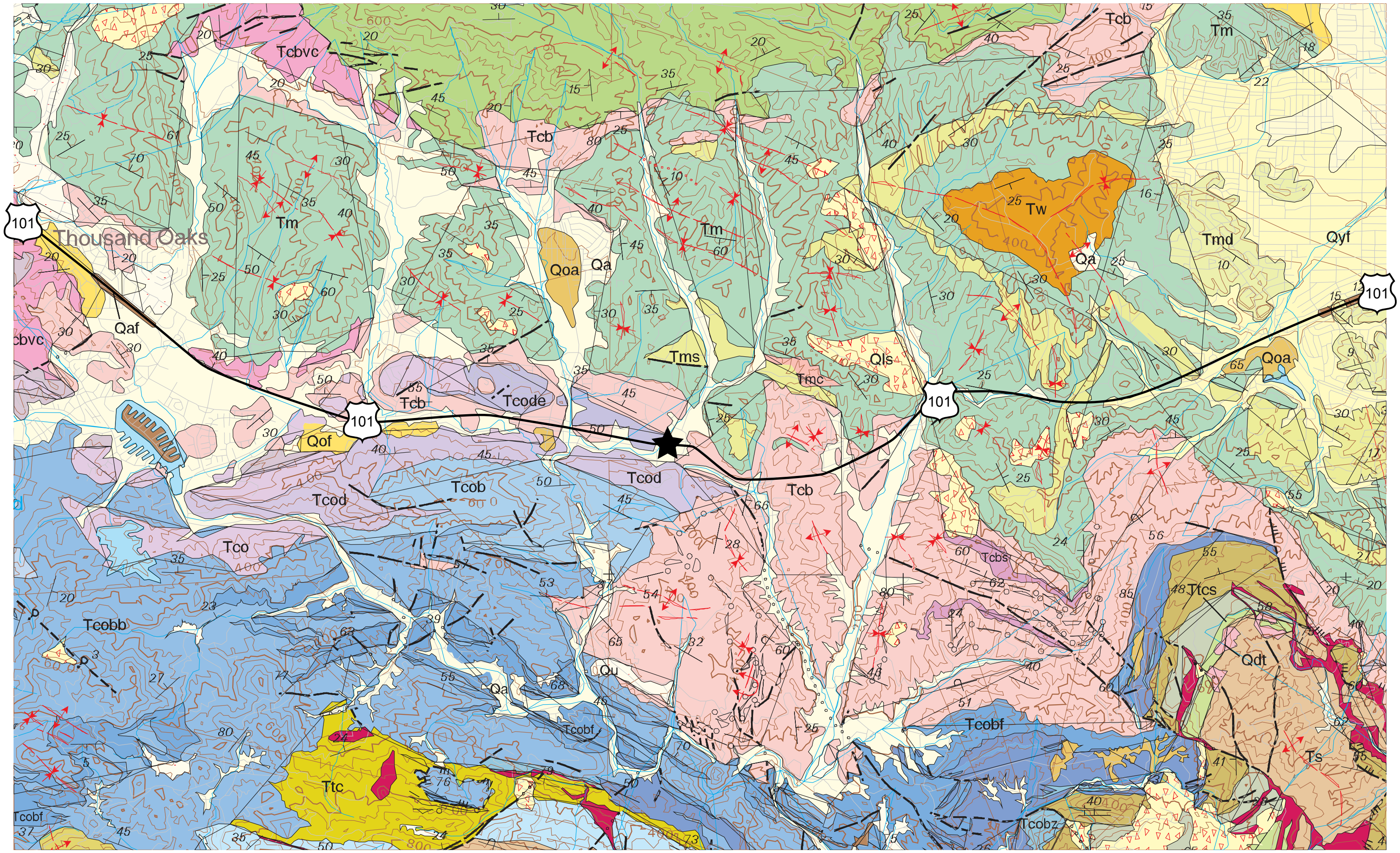
PROJECT NO. 106226		SITE LOCATION MAP	PLATE
DRAWN: 11/2009			1
DRAWN BY: MRG			
CHECKED BY: MC		US 101 PALO COMADO CANYON ROAD INTERCHANGE PROJECT AGOURA HILLS, CALIFORNIA	
FILE NAME: 106226p1.dwg			



N

0 1 2

miles

Contour Interval = 40 meters

Adapted from: Yerkes, R.F., and Campbell, R.H., 2005, Preliminary Geologic Map of the Los Angeles 30'x60' Quadrangle, Southern California, Version 1, United States Geologic Survey, Open File Report 2005-1019.

SYMBOLS

★ Project Area

Surface Fault

Concealed Fault

MAP UNITS DISCUSSED IN REPORT

Qls - landslide deposits

Qa - alluvium

Tm - Modelo Formation (undivided)

Tms - Modelo Formation (sandstone)

Tcb - Calabasas Formation (undivided)

Tcod - Conejo Volcanics (dacite)

Tcode - Conejo Volcanics (dacite w/ epiclastic lenses)

Tccob - Conejo Volcanics (basalt)

*Refer to Yerkes and Campbell (2005) for a complete explanation and list of units and symbols

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PROJECT NO.	106226
DRAWN:	12-15-10
DRAWN BY:	ASW
CHECKED BY:	REL
FILE NAME:	GEOMAP

GEOLOGIC MAP

PALO COMADO CANYON ROAD INTERCHANGE
CITY OF AGOURA HILLS, CALIFORNIA

PLATE

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