Appendix D

Geotechnical Investigation Report and Geotechnical Recommendation Memorandum

GEOTECHNICAL INVESTIGATION REPORT
AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

May 25, 2012

KLEINFELDER
Bright People. Right Solutions.

May 25, 2012
Project No. 113541

Mr. Michael Choi
Kimley-Horn and Associates, Inc.
5550 Topanga Canyon Blvd., Suite 250
Woodland Hills, California 91367

Subject: **Geotechnical Investigation Report**
Agoura Road and Kanan Road Widening Project
City of Agoura Hills, California

Dear Mr. Choi:

Kleinfelder is pleased to present this report summarizing our geotechnical investigation for the subject project. The purpose of our geotechnical investigation was to evaluate subsurface soil conditions and provide geotechnical recommendations for the design and construction of the proposed roadway widening. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 6.

Kleinfelder previously provided you with a draft report on May 26, 2011. We understand that Kimley-Horn provided that draft report to the City of Agoura Hills for review and has used that report for preliminary planning purposes. As a result of ongoing project planning with the City of Agoura Hills, Kimley-Horn requested that Kleinfelder perform additional sampling and testing of existing pavement subgrade materials along Agoura Road. Kimley-Horn has also informed Kleinfelder that the proposed work along Kanan Road has changed and that Kanan Road improvements will no longer include widening and modification of existing slopes along Kanan Road. The following report has been updated to include pavement thickness design recommendations based on the additional subgrade sampling. This report has also been updated to include recommendations for rock catchment design along Kanan Road based on our understanding of the current project.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned at (951) 801-3681, or Eric Philips at (213) 622-3790.

Respectfully submitted,
KLEINFELDER WEST, INC.

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Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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EXECUTIVE SUMMARY

This report presents the results of our geotechnical investigation for the proposed roadway improvements located in Agoura Hills, California. The purpose of this investigation was to explore and evaluate the subsurface conditions along the project alignment and to provide geotechnical recommendations for design and construction.

The soils encountered in our explorations generally consisted of undocumented fill and native soils. In five of our borings, bedrock was encountered underlying the native and artificial fill soil. Undocumented fill was encountered in several borings to depths up to 5 feet below grade. The fill consisted of medium dense to dense silty sand, sandy gravel, clayey sand, and silty clay with varying amount gravels; deeper and/or poorer quality fill may exist between locations investigated. The native soils generally consist of medium dense to dense clayey to silty sand with gravel and some sandy clay.

Based on the results of our field investigation and laboratory testing, it is our professional opinion that the project is feasible from a geotechnical standpoint, provided that the recommendations contained in this report are incorporated into design and construction. Overexcavation of the undocumented fill and upper native soils is recommended to provide uniform support of pavement sections and retaining wall foundations. The primary geotechnical considerations for the proposed development are summarized below.

- The undocumented fill and upper native soils encountered during our investigation, in their current conditions, are not considered suitable for support of the proposed improvements. To provide better support of the proposed structures, we recommend that the existing soil be overexcavated and recompacted as engineered fill.
- Kleinfelder reviewed the referenced reports by GeoSoils Consultants and Gorian and Associates and the analyses presented are considered acceptable for Slopes 1 through 4. We judge that Slopes 1 through 4 may be designed with a maximum gradient of 2:1, horizontal to vertical (H:V).
- Slopes 5 through 7 were evaluated by Kleinfelder as cut slopes constructed at a 2:1 (H:V). We evaluated a cross section of the maximum slope height. The cross section of the maximum slope height, presented as A-A’, is located within Slope 7.

- Kanan Road Slope 8 is an engineered cut slope along the east side of Kanan Road. It is approximately 420 feet in length and a maximum of approximately 52 feet high. Although the slope gradient varies along the length observed, generally the cut maintains an overall approximately 55 degree slope, inclined to the west. The slope comprises andesite-dacite flow breccias and agglomerates of the Miocene-age Conejo Volcanics, a member of the Topanga Group. The deposits are very thickly-bedded (3-10 feet thick) and are uniformly inclined northward toward Agoura Road between 35 and 45 degrees. Although the deposits are in most cases well-indurated, cobble-size clasts (average diameter of 3-inches to 12-inches) and boulder-size clasts (average diameter of 12 inches to 3 feet and locally up to 7 feet) are abundantly present and were observed locally eroding out of the slope, presenting a potential rock fall hazard.
- Based on the geologic mapping, the apparent dip of the bedding of Slope 8 is into the slope. Based on our geologic mapping and estimated shear strengths, we calculated a static factor of safety greater than 1.5 for a 1.5:1 (H:V) slope configuration. A steeper slope configuration may also have a calculated factor of safety greater than 1.5 depending on the overall height of the slope cut. However, based on Federal Highway Administration (FHWA) Catchment Design Guide, rockfall mitigation could require relatively wide catchment areas to provide 90% catchment. Because of the proposed construction of a sidewalk near the base of the cut slope, space is not available for a rockfall catchment area. For rockfall catchment, we modeled a 12-foot high catchment fence installed at the toe of the existing cut slope. Kleinfelder should be provided the opportunity to evaluate the final proposed design when it is available.

The executive summary presented herein briefly summarizes results of our geotechnical investigation for the subject project and should be used only in conjunction with recommendations presented in the attached report. It is subject to the limitations included in Section 6 of this report and the ASFE (Association of Engineering Firms Practicing in the Geosciences) insert.

1.0 INTRODUCTION

Kleinfelder performed a geotechnical investigation for the proposed Agoura Road and Kanan Road widening project in Agoura Hills, California. This report summarizes the results of our field exploration, laboratory testing and engineering analysis and provides recommendations for design and construction for the subject project. The location of the project presented in this report is shown on Plate 1, Site Location Map. The purpose of our geotechnical investigation was to evaluate subsurface soil conditions and provide geotechnical recommendations for the design and construction of the proposed road widening project. The scope of our services was presented in our proposal dated June 25, 2010 (Revision 2, July 28, 2010) and proposals for additional work dated November 18, 2010, and November 11, 2011.

Our report includes a description of the work performed, a discussion of the geotechnical conditions observed at the site, and recommendations developed from our engineering analyses of field and laboratory data. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included. We recommend that all individuals utilizing this report read the limitations (Section 6.0) along with the attached ASFE document.

1.1 PROJECT DESCRIPTION

We understand that the project consists of widening portions of the existing Agoura Road and adding a sidewalk along Kanan Road in the City of Agoura Hills, California.

Agoura Road is an arterial city street in the City of Agoura Hills, California. Agoura Road is generally one block south of and parallel to U.S. Highway 101 . The project limits of the proposed Agoura Road widening and improvements are generally from the City limits on the west end to the intersection of Cornell Road on the east end. Areas near the center of the project limits have already been improved and are not a part (NAP) of the project. We understand that Agoura Road improvements will include widening the existing 2- to 3-lane roadway to 4 lanes (two lanes in each direction). Improvements will also include constructing curb and gutter and sidewalks along both sides of the street. Some areas of Agoura Road will include median improvements and/or new median construction. Other areas will include diagonal parking and construction of roundabouts to improve traffic flow and reduce traffic speeds. We anticipate new traffic lanes will be constructed with asphaltic concrete (AC). Unreinforced concrete will be used for sidewalks. Construction of the proposed improvements will include fill slopes up to approximately 15 feet in height, cut slopes up to approximately 60 feet in height, and conventional cantilever retaining walls up to approximately 5 feet in height supported on shallow spread foundations.

Kanan Road intersects Agoura Road towards the easterly end of the City of Agoura Hills. Kanan Road provides passage from Agoura Hills through the Santa Monica Mountains to the coast. The project limits of the proposed Kanan Road improvements are generally from the intersection of Agoura Road on the north end to the intersection of Cornell Road on the south end. We understand that the Kanan Road improvements will include construction of curb and gutter and sidewalks along the street. Previously, the project included widening Kanan road, cutting into an existing approximately 52-foot tall cut slope (Slope 8), and construction of a conventional cantilever retaining wall (approximately 6 feet tall) supported on a shallow spread foundation at the base of the slope. We understand that widening Kanan Road is no longer being considered as part of this project and that further grading of the slope will not be performed.

1.2 SCOPE OF SERVICES

The scope of our geotechnical investigation consisted of a literature review, subsurface explorations, limited geologic mapping, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this report. A description of our scope of services performed for the geotechnical portion of the project follows.

Task 1 – Background Data Review. We reviewed readily-available published and unpublished geologic literature in our files and the files of public agencies, including selected publications prepared by the California Geological Survey and the U.S. Geological Survey. We also reviewed readily available seismic and faulting information, including data for designated earthquake fault zones as well as our in-house database of faulting in the general site vicinity. In addition, we were provided with the following geotechnical reports.

- Gorian & Associates (2000), Geotechnical Update Report and Results of Geotechnical Investigation to Answer Review Letter by Bing Yen and Associates, Inc. (dated November 9, 1998), Proposed Development at 30300 Agoura Hills Road, City of Agoura Hills, County of Los Angeles; California, Dated May 4, 2000.

- Gorian & Associates (2006), Geotechnical Update, Proposed Development at 30300 Agoura Hills Road, City of Agoura Hills, County of Los Angeles; California, Dated January 6, 2006.
- GeoSoils Consultants, Inc. (2009), Geologic and Geotechnical Engineering Review of Preliminary Development Plans, Conrad N. Hilton Headquarters Campus, 30500 and 30440 Agoura Road, APN 2061-002-024 and -048, Agoura Hills, California, Dated January 13, 2009
- GeoSoils Consultants, Inc. (2010), Response to City of Agoura Hills Geotechnical Review Sheet (GDI#: 09.00103.0174) Planning/Feasibility Comments dated August 28, 2009, 30500 and 30440 Agoura Road, APN 2061-002-024 and -048, Agoura Hills, California, Dated February 3, 2010.
- GeoSoils Consultants, Inc. (2010), Response to City of Agoura Hills Geotechnical Review Sheet (GDI#: 09.00103.0174) Planning/Feasibility Comments dated August 28, 2009, 30500 and 30440 Agoura Road, APN 2061-002-024 and -048, Agoura Hills, California, Dated May 19, 2010.

Hereinafter in this report, these consultants will be referred to as Gorian and Geosoils. We are assuming the role of Geotechnical Engineer of Record for slopes 1 through 4 as described in this report.

Task 2 – Field Exploration. Kleinfelder supervised excavation of 14 borings. Eleven borings were located along Agoura Road and 3 borings were located along Kanan Road. The approximate locations of the borings are presented on the boring location maps, Plates 2A through 2H. The borings were excavated to provide general information in order to characterize subsurface materials and perform our analyses. Traffic control was set up (traffic control services were provided by RP Barricade of Newbury Park, California) to provide a safe workspace during execution of our fieldwork.

Prior to beginning subsurface exploration, each of the 14 boring locations were marked and Kleinfelder notified Underground Service Alert (USA) of our intent to dig in accordance with California State law. In addition to USA notification, Kleinfelder subcontracted GEOVision (a geophysical services company from Corona, California) to provide borehole geophysical clearance services.

Exploratory borings along Agoura Road were drilled and logged between December 13 and 15, 2010 and on March 2, 2012. Exploratory borings along Kanan Road were drilled and logged on April 26, 2011. The borings were advanced to depths ranging from approximately 1.5 to 20.5 feet below the existing ground surface (bgs) using either a hand-auger, or limited access, or truck-mounted drill rigs operated by CalPac Drilling of Calimesa, California. Bulk and drive samples were retrieved from the borings, sealed and transported to our laboratory for further evaluation. A staff professional of Kleinfelder supervised the sampling, logged and visually classified the excavated soil cuttings and samples retrieved. Bulk soil samples were generally collected within the upper 5 feet of each boring and drive samples were collected at approximate 5-foot intervals using split-spoon samplers. Upon completion of the borings, excavated soil cuttings were used to backfill the excavations. The holes in pavements from the borings were patched with rapid-set concrete. The Logs of Borings B-1 through B-11 are included in Appendix A, Field Exploration at the end of this report. A description of the materials encountered in the additional borings (B-12 through B-14) drilled on March 2, 2012 are presented in tabular form in Section 4.4, Pavement Sections, of this report. The approximate locations of the borings are shown on Plates 2A through 2H, Boring Location Maps.

Task 3 – Limited Geologic Mapping. Rock exposures were only observed at slope 8. To characterize the rock mass where the cut slope (Slope 8) is planned for Kanan Road, a Kleinfelder geologist (under direct supervision of a California- Registered and Certified Engineering Geologist) performed mapping of the exposed rock surface at the proposed cut slope. The rock mass conditions and rock discontinuities will were evaluated for use in slope stability analyses.

Task 4 – Laboratory Testing. Laboratory testing was performed on selected samples to provide parameters for engineering evaluation. Testing consisted of in-situ density and moisture content, wash sieve, sieve and hydrometer, plasticity index, direct shear, expansion index, maximum density and optimum moisture, and R-value. Descriptions of the laboratory tests performed and the results of the testing are presented in Appendix B, Laboratory Test Results.

Task 5 – Geotechnical Analyses. Field and laboratory data were analyzed in conjunction with our understanding of the proposed project to provide geotechnical recommendations for the design and construction. In addition, seismic parameters based on the 2010 California Building Code (CBC) are presented.

Task 6 – Report Preparation. This report summarizes the work performed, data acquired, and our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed improvements. Our report includes the following items:

- Site location map and site plan showing the approximate boring locations;
- Logs of borings, including approximate elevations (Appendix A);
- Results of laboratory tests (Appendix B);
- Discussion of general site conditions;
- Discussion of general subsurface conditions as encountered in our field exploration;
- Discussion of regional and local geology and site seismicity;
- Discussion of geologic and seismic hazards;
- Recommendations for site preparation, earthwork, temporary slope inclinations, fill placement, and compaction specifications, including excavation characteristics of subsurface soil deposits;
- Recommendations for retaining wall foundation design, allowable bearing pressures, and embedment depths;
- Recommendations for seismic design parameters in accordance with the 2010 CBC;
- Recommendations for cut-slope and fill-slope construction; and
- Preliminary evaluation of storm water infiltration.

2.0 SITE CONDITIONS

2.1 SITE DESCRIPTION

Agoura Road is an arterial city street in the City of Agoura Hills, California. Agoura Road is generally one block south of and generally parallels U.S. 101. The project limits of proposed Agoura Road widening and improvements are generally from the City limits on the west end to the intersection of Cornell Road on the east end.

Kanan Road intersects Agoura Road towards the easterly end of the City of Agoura Hills. Kanan Road provides passage from Agoura Hills through the Santa Monica Mountains to the coast. The project limits of proposed Kanan Road improvements are generally from the intersection of Agoura Road on the north end and the intersection of Cornell Road on the south end.

The following is a description of existing site conditions at slopes 1 through 8 shown on Plates 2A through 2H.

2.1.1 Slopes 1 through 4

Slopes 1 through 4 are engineered slopes, each approximately 300 to 600 feet in length, located along Agoura Road, approximately 0.20 mi. to 0.75 mi. west of Reyes Adobe Road. The north-facing slopes are inclined approximately 40 degrees to the north and are generally mapped as underlain by volcanic deposits that dip to the north and underlie varying thicknesses of older alluvium and in some cases artificial fill. In 2008, an investigation by GeoSoils, mapped volcanic breccia and interbedded volcanic sandstones and siltstones that dip 37 to 51 degrees throughout these slopes. Slopes 2 and 3 were found to have as much as 30 to 85 feet of older alluvium overlying the volcanic deposits, respectively. Conversely, Slope 4 did not exhibit any alluvial deposition.

2.1.2 Slopes 5 through 7

Slopes 5 through 7 are engineered slopes, each approximately 300 to 650 feet in length, located along Agoura Road, between Ladyface Court to the west and Roadside Drive to the east. The north-facing slopes are inclined approximately 40 degrees to the north and are generally mapped as underlain by basaltic breccias and flows that dip to the north and underlie younger alluvium and crop out south of Agoura Road. During

December 2010, Kleinfelder's field investigation included three borings; B-2, B-3 and B-4 drilled near Slope 5, Slope 6 and Slope 7, respectively. At B-2, several attempts to hand auger met refusal within the top two feet. At B-3, only partial recovery of very dense yellowish brown poorly graded sand with gravel was encountered to 15 feet below ground surface. At B-4 similar yellowish brown dense to very dense sand and gravel samples were retrieved. Based on regional mapping and the subsurface investigation these slopes are likely composed of basaltic breccias and flows that weather to a dark yellowish or olive brown. Surficial sedimentary deposits are probably minimal.

2.1.3 Slope 8

Kanan Road Slope 8 is an engineered cut slope along the east side of Kanan Road. It is approximately 420 feet in length and a maximum of approximately 52 feet high. Although the slope gradient varies along the length observed, generally the cut maintains an overall approximately 55 degree slope, inclined to the west. The slope comprises andesite-dacite flow breccias and agglomerates of the Miocene-age Conejo Volcanics, a member of the Topanga Group. The deposits are very thickly-bedded (3-10 feet thick) and are uniformly inclined northward toward Agoura Road between 35 and 45 degrees. Although the deposits are in most cases well-indurated, cobble-size clasts (average diameter of 3-inches to 12-inches) and boulder-size clasts (average diameter of 12 inches to 3 feet and locally up to 7 feet) are abundantly present and were observed locally eroding out of the slope, presenting a potential rock fall hazard. Other than the potential rock fall hazard, no evidence of gross instability, in the form of slumps, surface failures, cracking, etc., was observed.

3.0 GEOLOGY

3.1 REGIONAL GEOLOGIC SETTING

The site is located at the northern flank of the Santa Monica Mountains within the Transverse Ranges Geomorphic Province of California. The geologic setting is presented on Plate 3, Regional Geologic Map. 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 comprises 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, which is not observed within the project area generally overlies the Calabasas Formation unconformably, but is often adjacent to the Calabasas Formation where there is faulting.

3.2 SUBSURFACE CONDITIONS

Subsurface conditions at the project consist of artificial fill, and younger and older alluvial deposits overlying bedrock of the Miocene-age Conejo Volcanics. The Conejo Volcanics consist of a sequence of volcanic flow deposits of basalt, andesitic basalt and volcanic sandstones, and siltstones, generally inclined steeply between approximately 37 and 51 degrees to the north. During December and April 26, 2011, Kleinfelder drilled eleven borings to a maximum depth of 20.5 feet below ground surface and mapped a bedrock slope (Slope 8) on Kanan Road, south of Agoura Road. Additional explorations were made on March 2, 2012 by coring through the asphalt along Agoura Road in order to collect samples for R-value testing and provide additional support for pavement design recommendations.

The following is a general description of the subsurface conditions and the bedrock characteristics mapped that can be applied to subsurface conditions at the locations explored. Subsurface materials encountered at the locations explored generally consisted of a thin veneer of native older or younger alluvium or artificial fill overlying bedrock of the Conejo Volcanics. Detailed descriptions of the deposits are provided in our logs of borings presented in Appendix A.

3.2.1 Fill and Native Soils

Native and fill soils encountered generally consisted of medium dense to dense clayey to silty sand with gravel and some sandy clay. These soils were generally present locally within the upper 3 to approximately 5 feet. Laboratory testing of fill and native soil samples indicate that the soils encountered can generally be considered expansive. Laboratory testing of three bulk samples of subgrade soils collected at borings B-1, B-6, and B-9 resulted in R-values of 28, 16, and 5, respectively. Laboratory dry densities in borings B-9 and B-11 ranged from approximately 87 to 112 pounds per cubic-foot (pcf). Laboratory moisture contents ranged from approximately 11 to 15 percent.

3.2.2 Bedrock

Bedrock was mapped along Kanan Road at Slope 8. The slope predominantly comprised andesite-dacite clast-supported flow breccias and agglomerates of the Miocene-age Conejo Volcanics. These deposits were largely composed of cobble-size clasts (3-12” diameter) but also exhibited large boulder-size clasts (as large as 7 feet in diameter). These deposits were interbedded with matrix-supported volcanic breccias and sandstones. The deposits are very thickly-bedded (3-10 feet thick) and uniformly dip to the north toward Agoura Road between approximately 35 and 45 degrees. Bedrock materials encountered below native and fill soils were consistent with Conejo volcanics breccia and agglomerate flows with typical blow counts greater than 50 for 6 inches.

3.3 GROUNDWATER

Groundwater was not encountered in any of the borings performed at the site in December 2010 or in April 2011.According to the California Department of Conservation (2000) the historic shallow groundwater level at the site is within 10 feet below the ground surface. These shallow contours generally follow the natural alignment of Lindero Canyon and Medea Creek, which are also aligned with Agoura Road through much of the project site.

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.

3.4 FAULTING

There is a high potential for moderate to strong seismic shaking to occur during the design life of the project. The site is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as being a “sufficiently active and well defined fault” that has exhibited surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago). These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of earthquakes. Active faults without surface expression (blind faults) and other potentially active seismic sources, which are capable of generating earthquakes, are not currently zoned and are known to be locally present under the region.

According to the City of Agoura Hills General Plan (Agoura Hills, 2010) there are 6 minor faults have been identified within the City of Agoura Hills but are considered inactive. The closest active faults to the site are the Malibu Coast and Simi-Santa Rosa faults located approximately 7.5 and 9.5 miles from the site (Ziony and Jones, 1989).

3.5 ASSESSMENT OF POTENTIAL GEOLOGIC HAZARDS

3.5.1 Fault-Rupture Hazard

Faults identified by the State as being active are not known to be present at the surface at the site. The site is not located within a State of California Earthquake Fault Rupture Hazard Zone (Bryant and Hart, 2007). Based on our geologic literature review, no mapped active or potentially active fault traces are known to transect the project site.

3.5.2 Flood Hazard

The Federal Emergency Management Agency (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which cover the entire United States. These maps identify those areas which may be subjected to 100 year and 500-year cycle floods. Based on our review of FEMA map panel 1244F, the site intersects the 100-year floodplain at two FEMA designated floodways; Madea Creek and Lindero Canyon. Madea Creek is the more prominent of the two floodways.

According to the City of Agoura (Agoura Hills, 2010), seismic induced inundation in Agoura Hills is not expected to occur within the City. Within the City, Lake Lindero is the only sizeable body of water, and considered a low level hazard with respect to seismic induced inundation. Outside the City, several reservoirs are known to exist, including Bard Reservoir, Malibu Lake, Lake Sherwood, Westlake Lake, Las Virgenes Reservoir, and Lake Eleanor. To date these reservoirs have not been considered high priority through the State's Division of Safety of Dams of the Department of Water Resources, who investigates on a highest priority basis, those dams most likely to fail under seismic shaking.

3.5.3 Landsliding

Landslides are ground failures (several tens to hundreds of feet deep) in which a (mass of earth material, including debris and often portions of bedrock) large section of a slope detaches and slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass. Structures above the slide area are typically damaged by undermining of foundations. Areas below a slide mass can be damaged by being overridden and crushed by the failed slope material.

Several factors can increase the potential for landsliding; slope angle, rock or soil type, bedding and foliation orientation, persistence of fractures, fracture density, zones of shearing or faulting, weathering, clay content, seismicity, water content, groundwater and the presence or absence of vegetation.

Although the area of the project site is not identified as a landslide hazard zone, some of these risk factors for landslides do exist at the site. North-facing slopes along Agoura Road comprise bedrock known to have out of slope bedding dipping approximately 40 to 50 degrees to the north. Additionally, throughout the project area the presence of cobbles and boulders within the Conejo Volcanics breccias and agglomerates may create a rockfall hazard if engineering controls are not applied. The presence of faults or areas of shearing may intensify this affect.

3.5.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

Soils in the project area have been identified by the City of Agoura Hills as moderately to highly expansive. The upper fill and alluvial soils (approximately upper 10 feet) are generally considered expansive. Our laboratory testing performed on a single sample resulted in an Expansion Index of 40, which is considered low expansion potential. However, information obtained from the referenced previous investigations by others (Gorian and Geosoils) indicates that medium to highly expansive soils should be expected at the project site.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our field exploration, laboratory testing and engineering analyses conducted during this study, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this report are incorporated into the project design and construction. The primary geotechnical considerations for site development are the presence of expansive soils, stability of proposed slope cuts, and stability of proposed embankment fills.

The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in the borings, the results of the laboratory-testing program, and our engineering analyses performed. Our recommendations regarding the geotechnical aspects of the design and construction of the project are presented in the following sections.

4.2 SEISMIC DESIGN CONSIDERATIONS

4.2.1 CBC Seismic Design Parameters

CBC (2010) Seismic Design Parameters are summarized in the following Table 1. The Seismic Design Category for a structure may be determined in accordance with Section 1613.5.6 of the 2010 CBC.

Table 1
Agoura Road Alignment 2010 CBC Seismic Design Parameters

Design Parameter	Recommended Value for West End of Project Alignment (City Limit) Latitude: 34.144806 Longitude: -118.793711	Recommended Value for East End of Project Alignment (Cornell Road) Latitude: 34.143457 Longitude: -118.76241
Site Class	D	D
S_s (Figure 1613.5(3)) (g)	1.71	1.65
S_1 (Figure 1613.5(4)) (g)	0.72	0.65
F_a (Table 1613.5.3(1))	1.0	1.0

Table 1 (Continued)
Agoura Road Alignment 2010 CBC Seismic Design Parameters

Design Parameter	Recommended Value for West End of Project Alignment (City Limit) Latitude: 34.144806 Longitude: -118.793711	Recommended Value for East End of Project Alignment (Cornell Road) Latitude: 34.143457 Longitude: -118.76241
F_V (Table 1613.5.3(2))	1.5	1.5
S_{MS} (Equation 16-36) (g)	1.71	1.65
S_{M1} (Equation 16-37) (g)	1.07	1.03
S_{DS} (Equation 16-38) (g)	1.14	1.10
S_{D1} (Equation 16-49) (g)	0.72	0.69

4.2.2 Slope Stability

The results of slope stability analyses for slopes 1 through 4 are presented in reports prepared by Gorian and GeoSoils, referenced herein. We reviewed slope stability analysis performed by Gorian and GeoSoils for slopes 1 through 4 and generally concur with their slope stability calculations. Additional investigation and analysis for slopes 1 through 4 was excluded from our authorized scope of work.

Kleinfelder performed slope stability analyses to evaluate proposed cut-slopes 5 through 7, the existing slope 8 configuration, and proposed roadway embankment fill slopes. Conclusions and recommendations for slopes 1 through 8 are presented below.

4.2.2.1 Materials Strength Parameters

Due to the lack of detailed geologic data for slopes 5 through 7, the slopes were analyzed as homogenous soil slopes. Shear strength parameters used in our analysis for proposed cut slopes 5 through 7 are based on conservative averaged ultimate direct shear strengths (cohesion (C) = 200 pounds per square-foot and friction angle (Φ) = 28 degrees) resulting from laboratory testing of soil samples collected in Kleinfelder Boring B-4 and B-5, and from previously published data presented in referenced reports by GSC, 2009. Cross-section A-A' depicts the maximum anticipated cut slope height (46 feet) proposed for Slopes 5, 6, and 7.

Shear strength parameters used in our analysis for proposed roadway embankment fill slopes are based on assumed conservative direct shear strengths and the results of our field investigation. Qualification testing of fill materials during construction will be required to verify strength parameters greater than or equal to cohesion (C) = 175 pounds per square-foot and friction angle (Φ) = 32 degrees. Embankment fill slope parameters are generally based on surficial stability in order to achieve a minimum Factor of Safety (FS) of 1.5 with maximum slope gradients of 2:1 (H:V).

Shear strength parameters for slope 8 were based on the results of our limited geologic mapping along the base of the existing slope and published geologic data (Conejo Volcanics). Anisotropic strengths were assigned to the bedrock materials to account for the directional shear strength based on bedding strike and dip. Apparent dip of the bedding is estimated to be approximately 35 degrees into slope. Along bedding strengths used are (C) = 350 psf and (Φ) = 24 degrees. Cross bedding strengths used are (C) = 1,000 psf and (Φ) = 33 degrees.

4.2.2.2 Analysis Methodology

Global slope stability analyses were performed using the computer program SLIDE 5.044 developed by Rocscience. SLIDE is a two-dimensional, limit equilibrium slope stability analysis program. The program was used perform random failure surface searches using Bishop Simplified method. Both static and pseudo-static stability analyses were performed. Pseudo-static analysis was performed in general accordance with guidelines presented in California Geological Survey (CGC, 2008) Special Publication 117A (SP 117A), Guidelines for Evaluating and Mitigating Seismic Hazards in California.

In order to evaluate if a slope is anticipated to experience lateral deformation during a design level seismic event, SP 117A guidelines require that the horizontal seismic coefficient used in pseudo-static slope stability analyses be determined from a screening analysis procedure. Generally, if the calculated factor of safety (F.S.) is 1.0 or less, further analysis is necessary to calculate predicted lateral deformation. If the calculated F.S. is 1.0 or greater, then it is assumed that lateral deformation would be on the order of 6 inches or less during a design level seismic event.

Input for the screening procedure for estimating the horizontal seismic coefficient include the maximum horizontal acceleration at the site for a soft rock condition (MHAr) corresponding to a seismic hazard level with a return period of 475 years or 10% of probability of exceedance in 50 years, the mode earthquake magnitude (M) and distance (r) associated with this maximum horizontal acceleration, and the allowable slope displacement. Based on the CGS web site, site acceleration for a return period of 475 years and allowable slope displacements of approximately 6 inches, the estimated seismic coefficient keq is 0.16.

Kleinfelder also performed surficial slope stability analysis (infinite slope stability analysis). The assumptions made for our analyses are summarized below:

- Soil slopes with maximum slope gradients of 2:1 (H:V).
- The assumed slip surface is 4 feet from the slope surface and parallel to the slope surface.
- The soil is saturated to the depth of the assumed slip surface.
- There is water seepage downslope to a depth of 4 feet and the water seepage flows parallel to the slope surface.

The results of our analysis are presented in Appendix C of this report. The following presents our recommendations for construction of proposed slopes associated with the project.

4.2.2.3 Slopes 1 through 4

Based on our review of the analysis presented in the referenced GeoSoils and Gorian reports, we recommend that slopes 1 through 4 be constructed at a maximum slope gradient of 2:1 (H:V).

Detailed subsurface investigation of the conditions at each of these slopes was beyond our authorized scope of services. Though conditions could vary, we do not anticipate encountering adverse bedding conditions during grading. If adverse bedding conditions are encountered, redesign of the subject slopes may be necessary resulting in delays during construction. To reduce the risk of construction delays, confirmation borings could be excavated through the top of each proposed cut slope prior to construction. Kleinfelder can provide a proposal for additional scope and fee if this option is desired. Kleinfelder should be retained to provide full-time observation and geologic mapping during construction of Slopes 1 through 4 to verify and validate assumptions drawn from the results of this investigation.

4.2.2.4 Slopes 5 through 7

Due to the lack of detailed geologic data for slopes 5 through 7, the slopes were analyzed as a homogenous soil slopes. Proposed slopes were analyzed at a gradient of 2:1 (H:V), as shown on Plate 3, Cross Section A-A'. The results of our slope stability analyses indicate calculated factors of safety of 1.64 for the static condition, 1.19 for the pseudostatic condition, and 1.17 for the Screening Analysis based on Special Publication 117A (SP 117A). We recommend that slopes 5 through 7 be constructed at a maximum gradient of 2:1 (H:V).

Detailed subsurface investigation of the conditions at each of these slopes was beyond our authorized scope of services. Though conditions could vary, we do not anticipate encountering adverse bedding conditions during grading. If adverse bedding conditions are encountered, redesign of the subject slope may be necessary resulting in delays during construction. To reduce the risk of construction delays, confirmation borings could be excavated through the top of each proposed cut slope prior to construction. Kleinfelder can provide a proposal for additional scope and fee if this option is desired. Kleinfelder should be retained to provide full-time observation and geologic mapping during construction of Slopes 5 through 7 to verify and validate assumptions drawn from the results of this investigation.

4.2.2.5 Slope 8

Slope 8 was evaluated based on limited surface geologic mapping, one shallow boring (B-10) performed near the toe of the existing cut slope, and geologic research of the general bedrock parameters. Based on the geologic mapping, the apparent dip of the bedding is approximately 35 degrees into the slope. Based on our understanding that widening Kanan Road is no longer being considered as part of this project and that further grading of the slope will not be performed, Kleinfelder evaluated the existing slope configuration. The results of our slope stability analyses indicate a factor of safety greater than 1.5. However, cobble-size clasts and boulder-size clasts are abundantly present and were observed locally eroding out of the slope, presenting a potential rock fall hazard. Based on Federal Highway Administration (FHWA) Catchment Design

Guide, rockfall mitigation could require relatively wide catchment areas to provide 90% catchment. Because of the proposed construction of a sidewalk near the base of the cut slope, space is not available for a rockfall catchment area. For rockfall catchment, we modeled a 12-foot high catchment fence installed at the toe of the existing cut slope. Additional options for rockfall catchment are presented in Section 4.2.2.7, Rockfall Hazard Mitigation and Catchment Design.

Detailed subsurface investigation of the conditions at each of these slopes was beyond our authorized scope of services. Though conditions could vary, we do not anticipate encountering adverse bedding conditions during grading. If adverse bedding conditions are encountered, redesign of the subject slope may be necessary resulting in delays during construction. To reduce the risk of construction delays, confirmation borings could be excavated through the top of each proposed cutslope prior to construction. Kleinfelder can provide a proposal for additional scope and fee if this option is desired. Kleinfelder should be retained to provide full-time observation and geologic mapping during scaling to remove the larger and loose rock blocks and during construction of rockfall catchment to verify and validate assumptions drawn from the results of this investigation.

4.2.2.6 Proposed Embankment Fills

Based on the referenced project plans, embankment fills are anticipated along the proposed Agoura Road alignment. Embankment fills, anticipated to be less than 15 feet in height, should be supported on competent bedrock, native soil, or a minimum of 2 feet of engineered fill prepared in accordance with the recommendations presented in Section 4.6, Earthwork. Embankment fills should be keyed at the base of the slope at least 3 feet deep and benched horizontally into the existing slope at least 6 feet and every 4 feet vertically. Recommended stability fill details are presented on Plate 5, Benching Details.

4.2.2.7 Rock Fall Hazard Mitigation and Catchment Design

Based on observations during geologic mapping along cut-slope 8, rock fall may be considered a hazard during the life of the project. We performed computer assisted analyses modeling rock fall simulations for the existing slope 8. We used the computer program Colorado Rockfall Simulation Program (CRSP) for our analysis. The following describes input parameters used for our CRSP analysis.

Our simulations were modeled with 6-inch, one-foot, and two-foot diameter spherical-shaped rock blocks consistent with observed rock blocks along the existing cut-slope face. For each simulation, the model rolled 500 rock blocks. We chose normal coefficients, tangential coefficients, and surface roughness values for the CRSP runs based on field observations and recommendations from the CRSP Manual for different slope descriptions related to slope lithology (Jones, et. al., 2000). Table 2 summarizes the range of values used for the CRSP coefficients.

Table 2
Rock Slope Rock Fall Coefficient Ranges

Lithology	CRSP Slope Description	Normal Coefficient Range	Tangential Coefficient Range	Surface Roughness Coefficient Range
Competent Bedrock	Bedrock Slope	0.25-0.30	0.85-0.90	0.25-1.0
Weathered Bedrock/Colluvial	Bedrock Slope	0.20-0.25	0.80-0.85	0.50-1.0
Soil	Firm Soil Slope	0.15-0.25	0.75-0.80	0.50-1.0
Catchment Area	Firm Soil Slope	0.15-0.25	0.75-0.80	0.50-1.0
Catchment Fence	NA	0.60-0.80	0.85-0.95	0.20-0.30

The existing slope is approximately 52-feet in height and has an inclination of approximately 55 degrees (0.7H:1V). Because of the proposed construction of a sidewalk near the base of the cut slope, space is not available for a rockfall catchment area. For rockfall catchment, we modeled a 12-foot high catchment fence installed at the toe of the existing cut slope. Based on our analyses, the 12-foot high catchment fence provides approximately 90 percent rockfall catchment from the existing cut slope. The analysis indicates that the maximum kinetic energy generated from a two-foot diameter rock block is approximately 10,000 ft-lbs. The rockfall fence should be designed and sized to contain this energy level.

Another option to reduce the potential for rockfall is to install a wire mesh drape system on the cut slope. Typically, the wire mesh system is anchored at the top of the cut slope with rock anchors. The wire mesh is then draped down over the cut slope and hung to about 3-5 feet above the ground. If rockfall occurs, the wire mesh is designed to constrain the rockfall to slowly move down the slope and fall out the bottom of the drape. The rockfall can then be periodically cleaned up from the base of the slope. The wire mesh system should be designed for the rock block sizes present in the existing slope and the support anchors and wire mesh sized appropriately.

For either the catchment fence or wire mesh option discussed above, we recommend that the existing cut slope be scaled to remove the larger and loose rock blocks present. This will further reduce the potential for rockfall from the existing cut slope.

4.3 RETAINING WALLS

4.3.1 General

Based on the results of our field exploration, laboratory testing and geotechnical analyses, the proposed retaining walls may be supported on conventional spread foundations placed entirely on engineered fill or competent bedrock. If founded on engineered fill, spread foundations should be underlain by a minimum 2 feet of engineered fill constructed as recommended in Section 4.7. Recommendations for the design lateral earth pressures and design of spread foundations are presented below. Transitions from bedrock to engineered fill within a single footing should be avoided. If this condition exists, the bedrock portion should be overexcavated to provide the minimum fill thickness recommended above.

The recommended lateral earth pressures assume that drainage is provided behind the walls to prevent the buildup of hydrostatic pressures. Walls should be provided with drains to reduce the potential for the buildup of hydrostatic pressure. Drains may consist of a 2-foot-wide zone of ¾-inch rock wrapped in filter fabric located immediately behind the wall extending to within 1 foot of the ground surface. Perforated Schedule 40 PVC pipe should be installed within the rock at the base of the drain and sloped to discharge to a suitable collection facility. Commercially available drainage panels could be used as an alternative. The product manufacturer's recommendations should be followed in the installation of a drainage panel. Expansive soils should not be used as wall backfill material.

Where slope extend at inclinations greater than horizontal behind retaining walls, a minimum of a 2-foot drainage swale should be constructed at the top of the wall to limit the amount of surface water infiltrating behind the wall

4.3.2 Spread Footings

Spread footings founded on engineered fill may be designed for a net allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus sustained live loads. Spread footings founded entirely on bedrock may be designed for a net allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus sustained live loads. The footings should be established at a depth of at least 24 inches below the lowest adjacent exterior grade and at least 12 inches into the bedrock. A one-third increase in the above bearing pressures can be used for wind or seismic loads.

The structural engineer should design the footing dimension and reinforcement; however spread footings should have a minimum width 24 inches. Structurally continuous foundations should not be directly founded on both engineered fill and bedrock. If the proposed foundations are anticipated to directly bear on both engineered fill and bedrock, a structural break should be constructed in the foundation to limit the distress caused by differential settlement.

4.3.3 Estimated Settlements

We estimate total static settlement for foundations designed in accordance with the recommendations presented above and supported entirely on engineered fill or bedrock to be less than ½ inch.

4.3.4 Lateral Resistance

Lateral load resistance may be derived from passive resistance along the vertical sides of the footings, friction acting at the base of the footing, or a combination of the two. An allowable passive resistance of 250 psf per foot of depth may be used for design. Allowable passive resistance values should not exceed 1,500 psf. An allowable coefficient of friction value of 0.30 between the base of the footings and the engineered fill soils and competent bedrock can be used for sliding resistance using the dead load forces. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations.

4.3.5 Lateral Earth Pressures

Design earth pressures for retaining walls depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. The earth pressures provided assume that that a non-expansive backfill will be used and a drainage system will be installed behind the walls, so that external water pressure will not develop. If a drainage system will not be installed, the wall should be designed to resist hydrostatic pressure in addition to the earth pressure.
The recommended active lateral earth pressures for horizontal backfills using granular relatively non-expansive soils on walls that are free to rotate at least 0.1 percent of the wall height is 35 pcf. The recommended active lateral earth pressures for backfills sloping not steeper than 1.5:1 using granular relatively non-expansive soils on walls that are free to rotate at least 0.1percent of the wall height is 60 pcf.

The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (240 psf). Lateral load contributions from other surcharges located behind walls may be provided once the load configurations and layouts are known.

4.4 PAVEMENT SECTIONS

New pavement sections should be designed considering the parameters presented below. Laboratory testing of seven bulk samples of the pavement subgrade soils collected at borings B-1, B-6, B-8, B-9 and B-12 through 14 resulted in R-values ranging from 5 to 26. Due to uncertainty of soil conditions between the locations of our borings, the following table presents recommended pavement sections based on an average R-value of 16. Additional explorations were performed on March 2, 2012. The materials encountered in those explorations are described in the Table 3 below.

Table 3
Existing Conditions Encountered at Borings B-12 Through 14

Boring	Asphaltic Concrete (inches)	Aggregate Base (inches)	Comments Regarding Condition of Aggregate Base and Subgrade Soils
B-12	8	4	Boring extended approximately 39 inches below pavement surface; approximately 1 foot of cemented sand was encountered below the aggregate base (possible cement treated sub-base); and sandy clay was encountered below the cemented sand. An R-value of 16 resulted from laboratory testing of the clayey subgrade material. The sample was prepared to try to segregate sandy material that was potentially mixed by sampling through the overlying layer.
B-13	9.5	No Base	Boring extended approximately 24 inches below pavement surface. Aggregate base was not encountered below the asphalt. Sandy clay was encountered below the asphalt. An R-value of 13 resulted from laboratory testing of the clayey subgrade material.
B-14	5	14	Boring extended approximately 36 inches below pavement surface. Approximately 14 inches of aggregate base was encountered below the asphalt. Sandy clay with some gravel was encountered below the aggregate base. An R-value of 15 resulted from laboratory testing of the clayey subgrade material.

Recommended pavement sections were developed using Caltrans Highway Design Manual (last updated July, 2009) and presented in Table 4 below.

Table 4
Recommended Asphalt Pavement Sections
(Design R-Value = 16)

Traffic Index (TI)	Asphalt Concrete Pavement Thickness (inches)	Class 2 Aggregate Base Thickness* (inches)	Overall Pavement Section Thickness (inches)
7	4.5	13	17.5
	5.0	11.5	16.5
	5.5	10	15.5
	6.0	9	15
9	5.5	17.5	23
	6.0	16	22
	6.5	15	21.5
	7.5	13	20.5

Table 4 (Continued)
Recommended Asphalt Pavement Sections
(Design R-Value = 16)

Traffic Index (TI)	Asphalt Concrete Pavement Thickness (inches)	Class 2 Aggregate Base Thickness* (inches)	Overall Pavement Section Thickness (inches)
9.5	6.0	18	24
	6.5	17	23.5
	7.5	15	22.5
	8.0	14	22

*Aggregate base thicknesses should be increased by 15% to account for expansion potential of subgrade soils where clayey conditions are encountered during construction. Additional R-value testing may also be necessary where clayey conditions are encountered during construction.

A traffic study was not performed by Kleinfelder to generate the TI's presented above. The TI's above are assumed values based on our experience with similar projects and requests made by you. These TI's should be verified and validated by the project Civil Engineer. Changes in the traffic indices/volumes may affect the corresponding pavement sections.

The pavement sections provided above are contingent on the following recommendations being implemented during construction.

- Pavement sections should be underlain by a minimum of 24 inches of newly placed engineered fill, prepared as described within this report.
- The subgrade soils should be in a relatively stable, non-yielding condition at the time engineered fill and/or aggregate base materials are placed and compacted.
- Aggregate base materials should be placed at near optimum moisture content (within 3 percent) and compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.
- Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate baserock or "Greenbook" specifications for crushed aggregate base.
- Asphalt paving materials and placement methods should meet current Caltrans or "Greenbook" specifications for asphalt concrete.
- All concrete curbs separating pavement and landscaped areas should extend into the subgrade and below the bottom of adjacent, aggregate base materials.

- A representative of the geotechnical engineer should evaluate materials encountered during construction. Based on field observations during grading activities, additional R-Value testing may be needed. Modified pavement design recommendations may be presented after reviewing post-grading R-value test results.

4.5 CONCRETE FLATWORK

Prior to casting concrete flatwork, subgrade soils should be moisture conditioned and recompacted, as recommended in Section 4.7. Due to the potentially expansive soils at the site, the moisture content of the subgrade soils should be maintained at or above optimum prior to the placement of any flatwork. In the event that these subgrade soils are allowed to dry out, the exposed subgrade should be re-moisture conditioned.

Concrete walks for pedestrian traffic or landscape should be at least four inches thick. Weakened plane joints should be located at intervals of about 6 feet. Careful control of the water/cement ratio should be performed to avoid shrinkage cracking due to excess water or poor concrete finishing or curing.

4.6 STORM WATER INFILTRATION

The rate of infiltration is a function of saturated hydraulic conductivity, hydraulic gradient, and wetted area. Saturated hydraulic conductivity (permeability) of a soil, when considering infiltration system design, may be approximated by correlation with the grain size distribution. Correlations do not generally account for the in-situ compaction and/or density of the infiltrating soils. Vertical hydraulic gradient is estimated based on the depth to groundwater. Where groundwater, is deep (generally greater than 50 feet below the bottom of large infiltration ponds) and impermeable or low permeable layers are also deep, the saturated hydraulic conductivity is generally equivalent to the rate of infiltration. This is because the hydraulic gradient (i) can be considered to be equal to 1 so long as the wetting front moves vertically downward. This will be true only when depth to groundwater or low hydraulic conductivity soil layer is sufficient. When the wetting front encounters the groundwater table or a soil layer with low hydraulic conductivity the vertical hydraulic gradient can rapidly approach zero, resulting in greatly reduced infiltration and groundwater mounding.

Storm water infiltration systems are generally applicable for soil sites that have estimated long-term infiltration rates of at least ½ inch per hour. Based on the results of our field investigation, the majority of the near surface soils are fine-grained. Additionally, groundwater may be seasonably as shallow as 10 feet below ground surface. Therefore, long-term infiltration rates are anticipated to be much lower than a ½-inch per hour and infiltration systems may be subject to long-term ponding and/or overflow. We recommend that storm water infiltrations systems not be used for this project.

4.7 EARTHWORK

4.7.1 General

Site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications, and the recommendations included in this report. References to maximum unit weights are established in accordance with the latest version of ASTM Standard Test Method D1557. The earthwork operations should be observed and tested by a representative of Kleinfelder.

4.7.2 Site Preparation

Organic, inert and oversized materials (greater than 3 inches in maximum dimension) should be stripped and isolated prior to removal of reusable soils. Pavement should be stripped and disposed off-site or pulverized and mixed with the on-site soils and reused as fill material. Overexcavation should remove any loose or soft earth materials until a firm, unyielding or competent subgrade is exposed, as evaluated by the geotechnical engineer. Overexcavation must expose a firm, non-yielding subgrade that is free of significant voids and organics. The subgrade soils exposed at the bottom of overexcavation should be observed by a geotechnical engineer from our office prior to the placement of any fill. Prior to the placement of engineered fill, after site preparation, the bottom of the overexcavations should be proof-rolled and compacted to at least 90 percent relative compaction to the satisfaction of the geotechnical engineer-of-record. Additional removals, scarification and drying operations, and/or subgrade reinforcement may be required to stabilize soft, yielding subgrades.

The grading contractor should anticipate that additional processing and moisture conditioning of the onsite soils will be necessary during site grading to obtain material which is acceptable to be placed as engineered fill, as described in this report. The moisture conditioning of some of the soils will require significant drying and some soils will require the addition of moisture. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork operations.

The grading contractor should also anticipate encountering oversized material greater than 3 inches in maximum dimension within 5 feet of the existing subgrade. Quantifying the actual amount of oversize material that could be encountered requires additional investigation.

4.7.3 Fill Material

We anticipate that most of the on-site soils may be reusable as engineered fill once oversized materials greater than 3 inches in dimension (if encountered) have been removed and after organic and inorganic debris are cleared and disposed off-site. Fill should be placed in lifts no greater than 8 inches thick, loose measurement, and moisture conditioned to between 2 and 4 percent over optimum moisture. The engineered fill soil should be compacted to at least 90 percent relative compaction but generally no more than 92 percent relative compaction. In order to achieve 95 percent compaction of aggregate base, compaction of the upper 6 inches of pavement subgrade soils to 95 percent may be required and is considered acceptable.

If imported fill soils are to be used for engineered fill, they should be sampled and tested and approved by the geotechnical engineer prior to being transported to the site. In general, well-graded mixtures of gravel, sand and non-plastic silt are acceptable for use as import fill. Fine-grained soils should not be imported onsite for placement as engineered fill

4.7.4 Excavation Characteristics and Wet Soils

Our soil borings were performed with moderate effort using a hollow stem auger. The contractor should anticipate moderate excavation effort and plan accordingly. The Slope 8 bedrock may be moderately difficult to excavate and the grading contractor may require special equipment for this portion of the project. Kleinfelder did not perform borings within the Slope 8 bedrock so conclusions regarding the excavatability of the bedrock are based upon our knowledge of the material in the area. Actual site conditions may be different. The contractor should anticipate encountering particles greater than 3 inches in diameter and may need to crush the material to make it reusable from a particle size standpoint.

4.7.5 Temporary Excavations

Temporary cuts up to 10 feet high may be sloped back at an inclination of no steeper than 1.5:1 (horizontal to vertical) in existing site soils. Minor sloughing and/or raveling should be anticipated as they dry out. If signs of slope instability are observed, the inclination recommended above should be decreased until stability of the slope is obtained. In addition, at the first signs of slope instability, the geotechnical engineer should be contacted. Where space for sloped embankments is not available, shoring will be necessary. Shoring and/or underpinning of existing improvements that are to remain may be required to perform the demolition and overexcavation. Excavations within a 1.5:1 plane extending downward from a horizontal distance of 2 feet beyond the bottom outer edge of existing improvements should not be attempted without bracing and/or underpinning the improvements. Personnel from the geotechnical engineer should observe the excavations so that modifications can be made to the excavations, as necessary, based on variations in the encountered soil conditions. All applicable excavation safety requirements and regulations, including OSHA requirements, should be met.

Where sloped excavations are used, tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within a distance equal to the depth of the excavation. Greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Kleinfelder should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If temporary construction slopes are to be maintained during the rainy season, berms are recommended along the tops of the slopes to reduce runoff that may enter the excavation and erode the slope faces.

Temporary, shallow excavations with vertical side slopes less than 4 feet high should generally be stable, although sloughing may be encountered. Vertical excavations greater than 4 feet high should not be attempted without appropriate shoring to prevent local instability. All trench excavations should be braced and shored in accordance with good construction practice and all applicable safety ordinances and codes. The contractor should be responsible for the structural design and safety of the temporary shoring system, and we recommend that this design be submitted to the Kleinfelder for review to check that our recommendations have been incorporated. For planning purposes, the on-site soils may be considered Type C, as defined using the current OSHA soil classification.

Stockpiled (excavated) materials should be placed no closer to the edge of an excavation than a distance equal to the depth of the excavation, but no closer than 4 feet. All trench excavations should be made in accordance with OSHA requirements.

4.7.6 Trench Backfill

If relocation of utilities is necessary, pipe or utility bedding should consist of sand or similar granular material having a minimum sand equivalent value of 30. The sand should be placed in a zone that extends a minimum of 6 inches below and 12 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density. Trench backfill above pipe bedding may consist of approved, on-site or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density but no more than 92 percent of the maximum dry density. In order to achieve 95 percent compaction of aggregate base, compaction of the upper 6 inches of pavement subgrade soils to 95 percent may be required and is considered acceptable. Jetting of pipe bedding or trench backfill materials is not permitted.

4.7.7 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, perched groundwater, drought, or other factors and may cause unacceptable settlement or heave of pavements, sidewalks, curbs, gutters and other structures supported over these materials. The soils encountered during our investigation have a low expansion potential. However, based on previous work in the area and regional information, medium to highly expansive soils may be encountered with the limits of the project. The recommendations presented herein are intended to reduce the effects of the expansive nature of the site soils. The potential for negative impacts of expansive soils cannot be completely eliminated unless they are completely removed from the site or chemically altered.

5.0 ADDITIONAL SERVICES

5.1 ADDITIONAL GEOTECHNICAL INVESTIGATION

Our authorized scope included limited geotechnical investigation. Conditions could vary between the locations explored. We do not anticipate encountering adverse bedding conditions during grading. However, if adverse bedding conditions are encountered, redesign of proposed slopes may be necessary resulting in delays during construction. To reduce the risk of construction delays, confirmation borings could be excavated through the top of each proposed cut slope prior to construction. Kleinfelder can provide a proposal for additional scope and fee if this option is desired. Kleinfelder should be retained to provide full-time observation and geologic mapping during construction of all slopes constructed for this project.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that Kleinfelder perform a general review of the project plans and specifications before they are finalized to verify that our geotechnical recommendations have been properly interpreted and implemented during design. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations.

5.3 CONSTRUCTION OBSERVATION AND TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because we sample only a limited portion of the soils affecting the performance of the proposed structure, unanticipated or changed conditions can be encountered during grading. Proper geotechnical observation and testing during construction are imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Kleinfelder be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study.

Our services are typically needed at the following stages of grading.

- after demolition;
- during grading;
- after the overexcavation, but prior to scarification;
- during utility trench backfill;
- during base placement and site paving; and
- after excavation for foundations.

6.0 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

The scope of services was limited to performing 11 borings. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our field investigations, laboratory analysis, and engineering evaluations.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of pavements and placement of engineered fill;
- Confirm engineered fill is placed and compacted per the project specifications.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, installation of piles, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock, and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinions, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

7.0 REFERENCES

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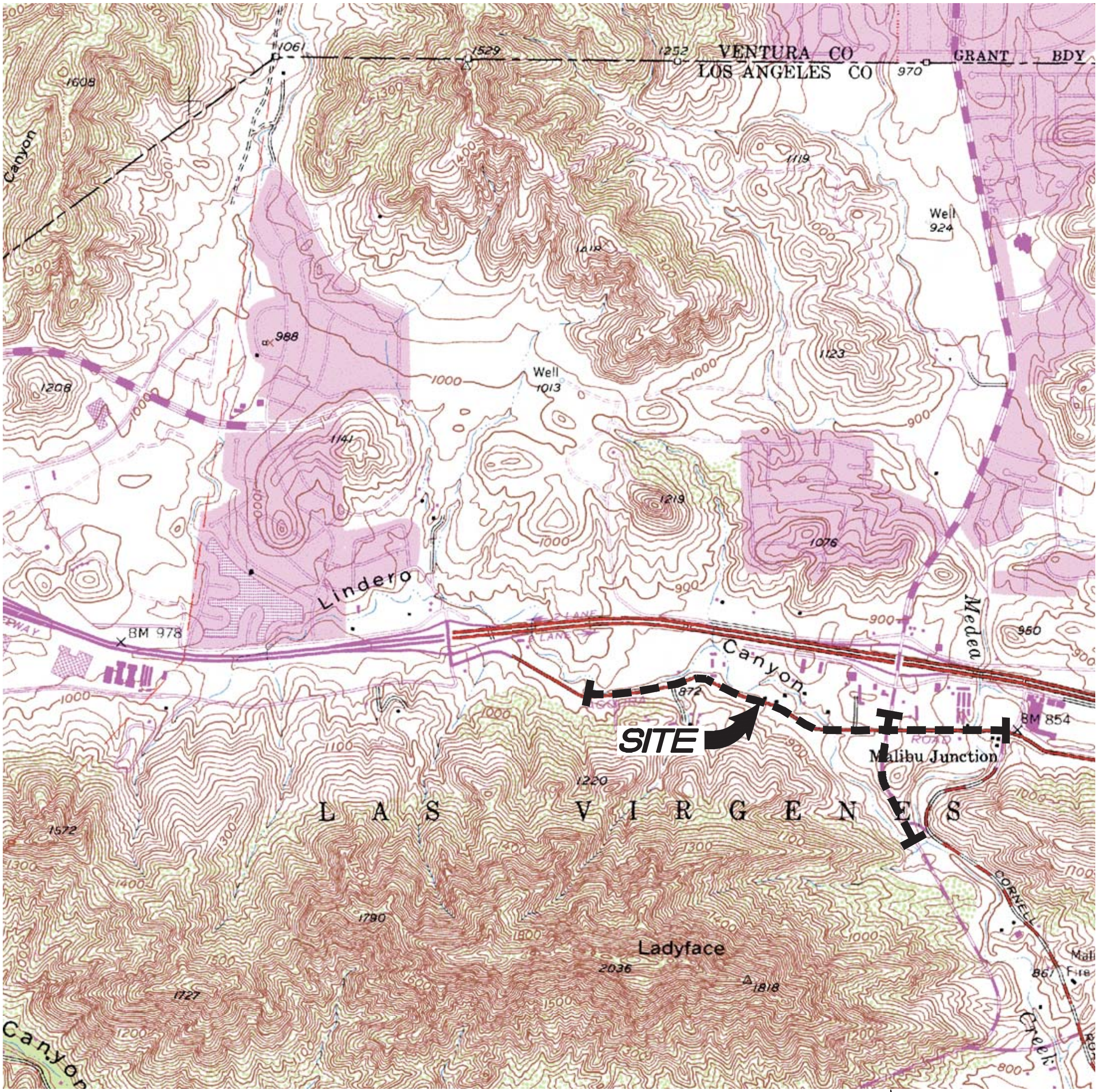
PLATES



SOURCE: U.S.G.S. □.5' Topographic series, Thousand Oaks, California
Quadrangle 1950, Photorevised 1981.

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ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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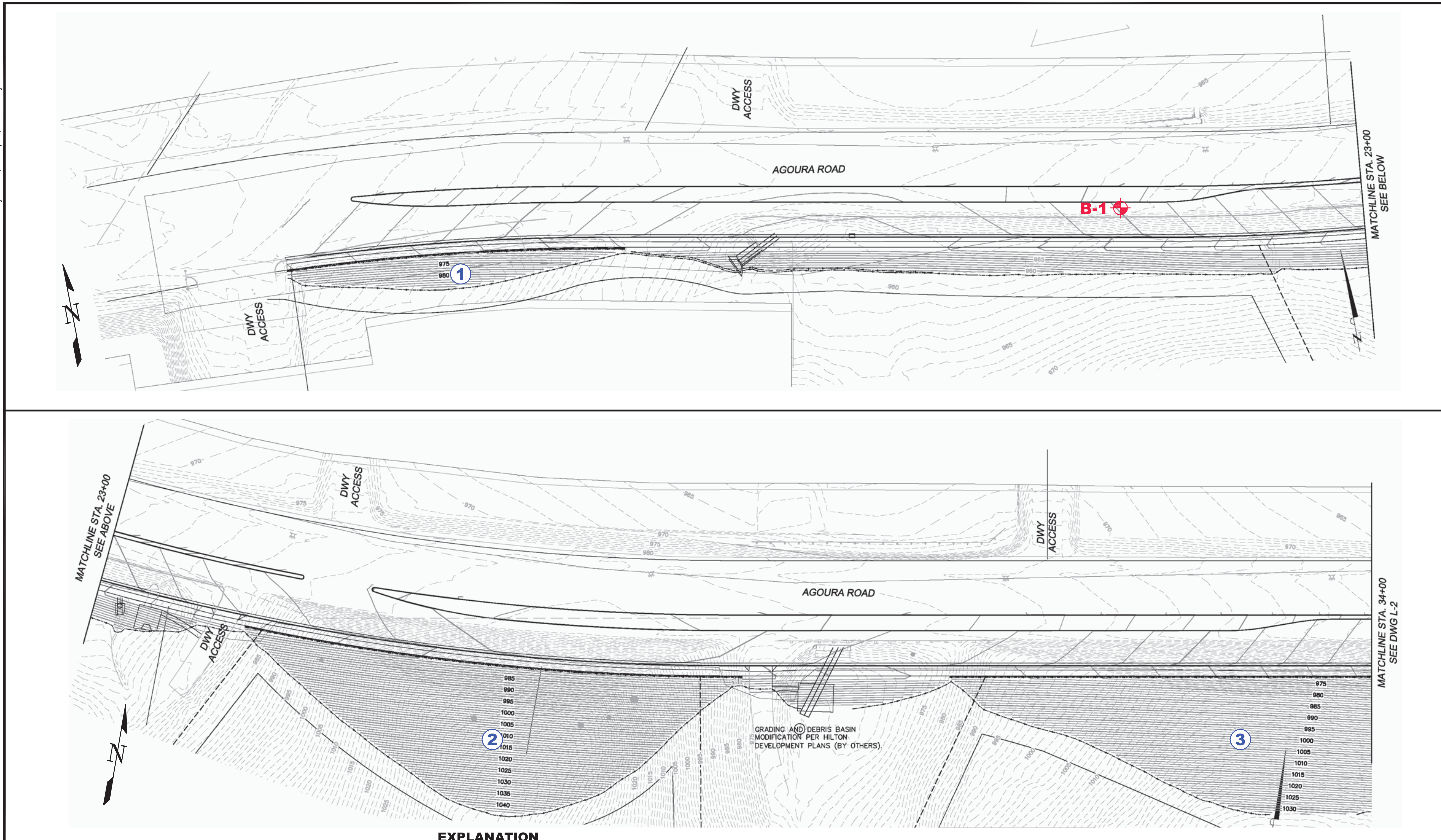
DRAWN BY:	DMF
CHECKED BY:	JDW
FILE NAME: 113541p1.dwg	

SITE LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE

1



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APPROXIMATE SCALE (feet)

EXPLANATION

B-1 APPROXIMATE SOIL BORING LOCATION

3 SLOPE NUMBER

REFERENCE: BASE MAP PROVIDED BY KIMLEY HORN AND ASSOCIATES, INC., UNDATED

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FILE NAME:	113541p2.dwg

BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE

2A

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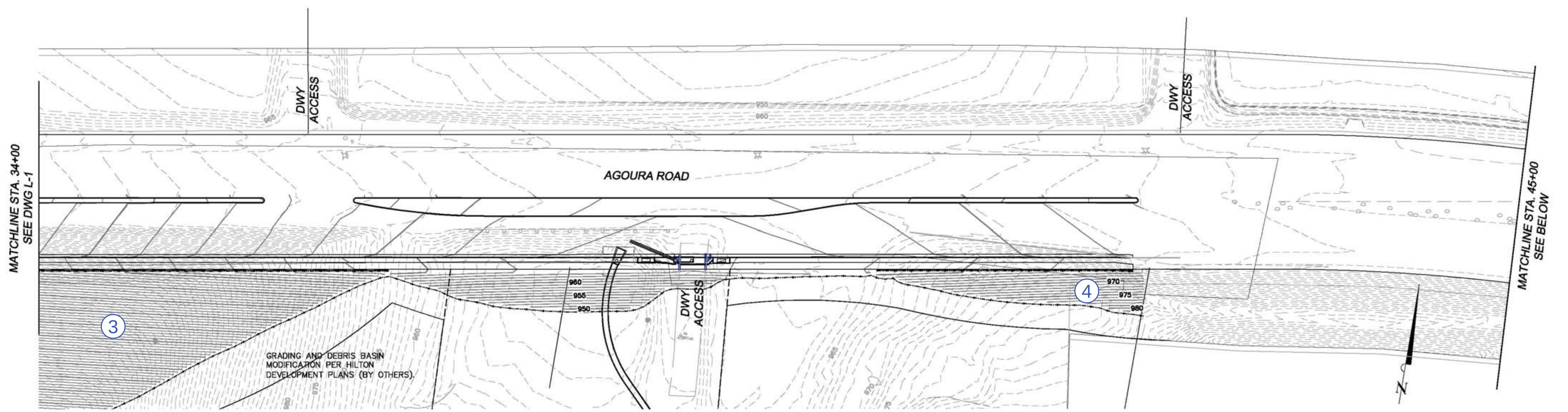

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80 40 0 80

APPROXIMATE SCALE (feet)

EXPLANATION

(4) SLOPE NUMBER

REFERENCE: BASE MAP PROVIDED BY KIMLEY HORN AND ASSOCIATES, INC., UNDATED

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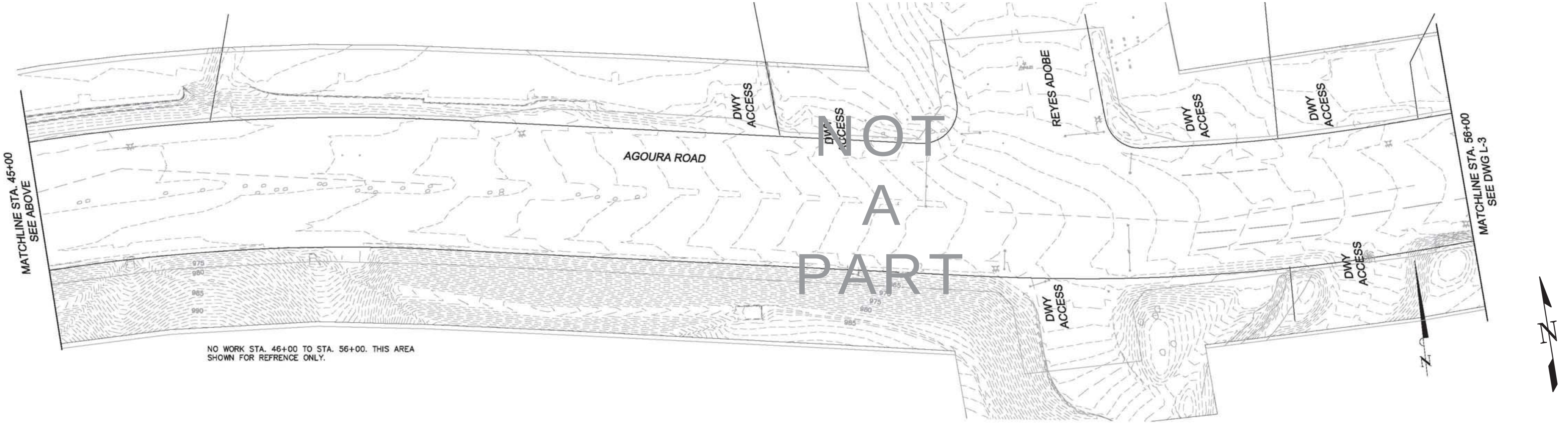
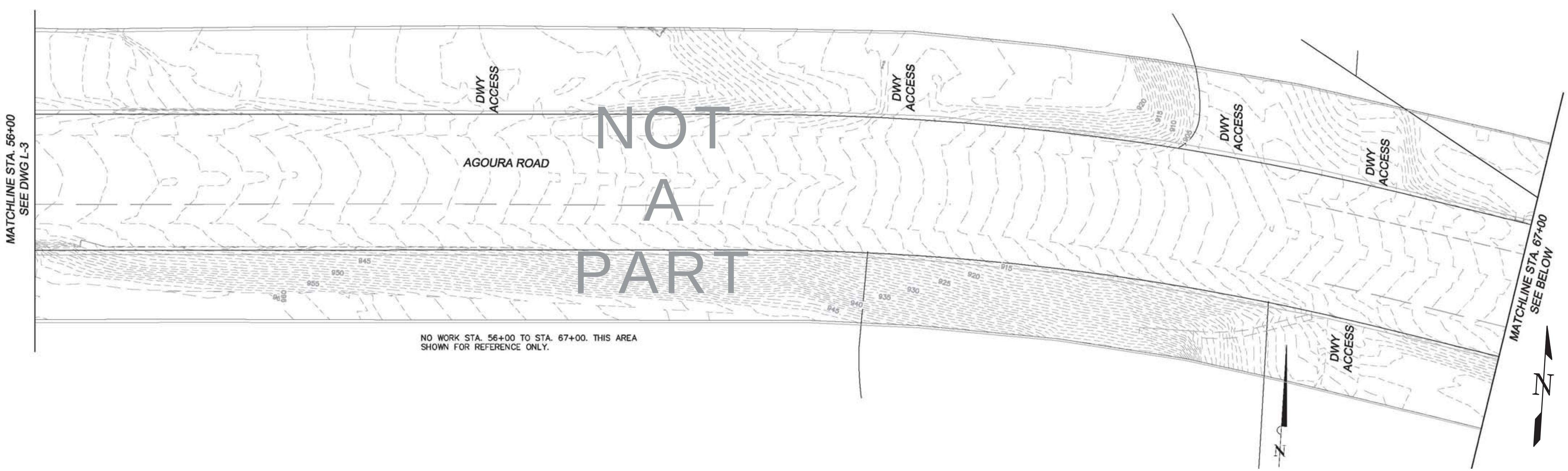
BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE

2B





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APPROXIMATE SCALE (feet)

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BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
2C

ATTACHED IMAGES: Images: Agoura Road Grading (3)_1.png Images: Agoura Road Grading (3)_2.png Images: Agoura Road Grading (3)_3.png Images: Agoura Road Grading (3)_4.png Images: Agoura Road G
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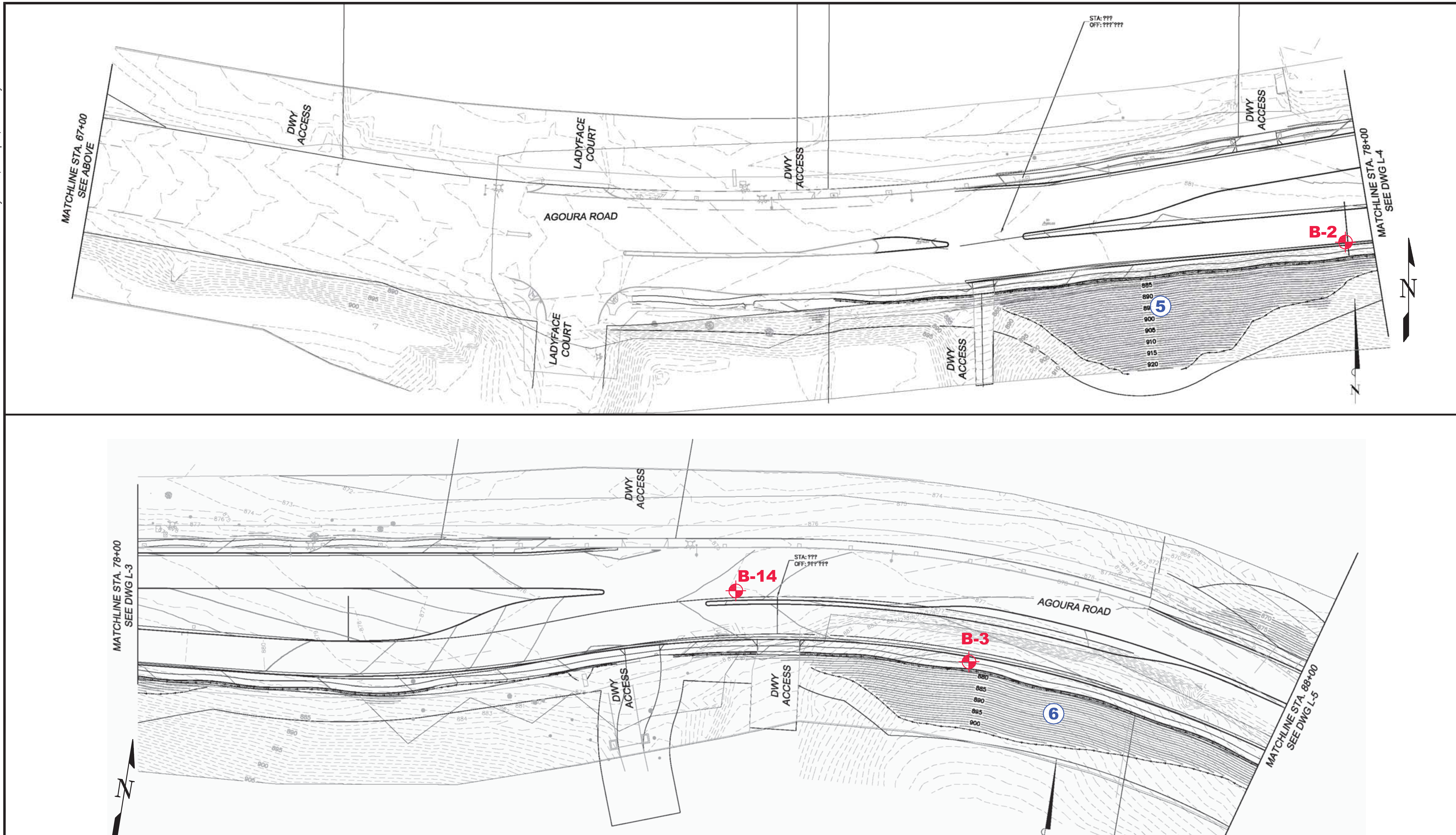

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APPROXIMATE SCALE (feet)

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EXPLANATION

B-3 APPROXIMATE SOIL BORING LOCATION

6 SLOPE NUMBER

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BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
2D

ATTACHED IMAGES: Images: Agoura Road Grading (3)_1.png Images: Agoura Road Grading (3)_2.png Images: Agoura Road Grading (3)_3.png Images: Agoura Road Grading (3)_4.png Images: Agoura Road
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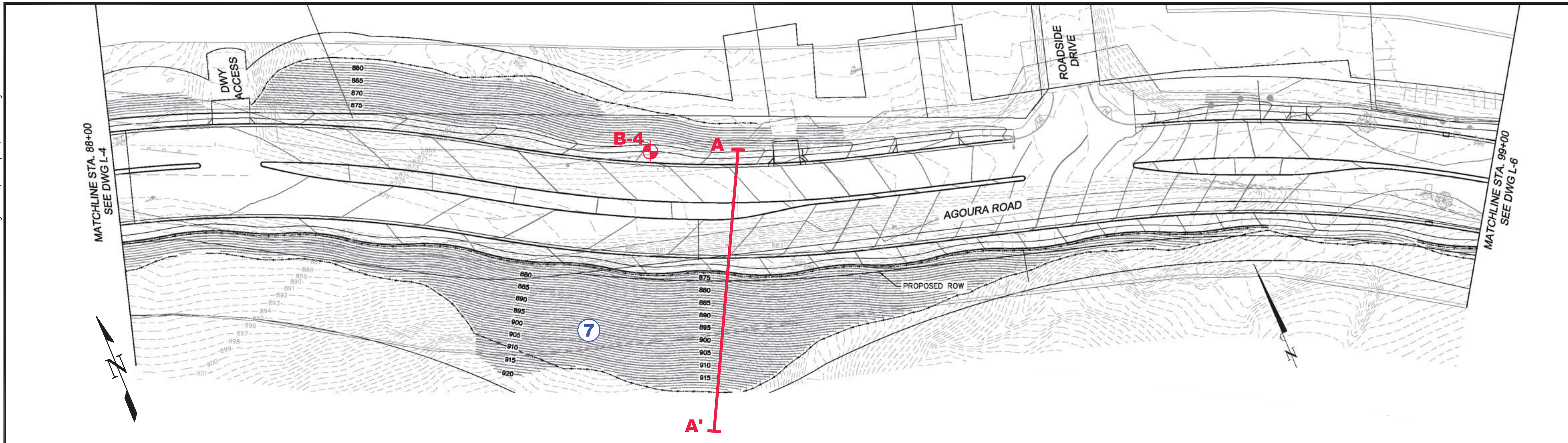

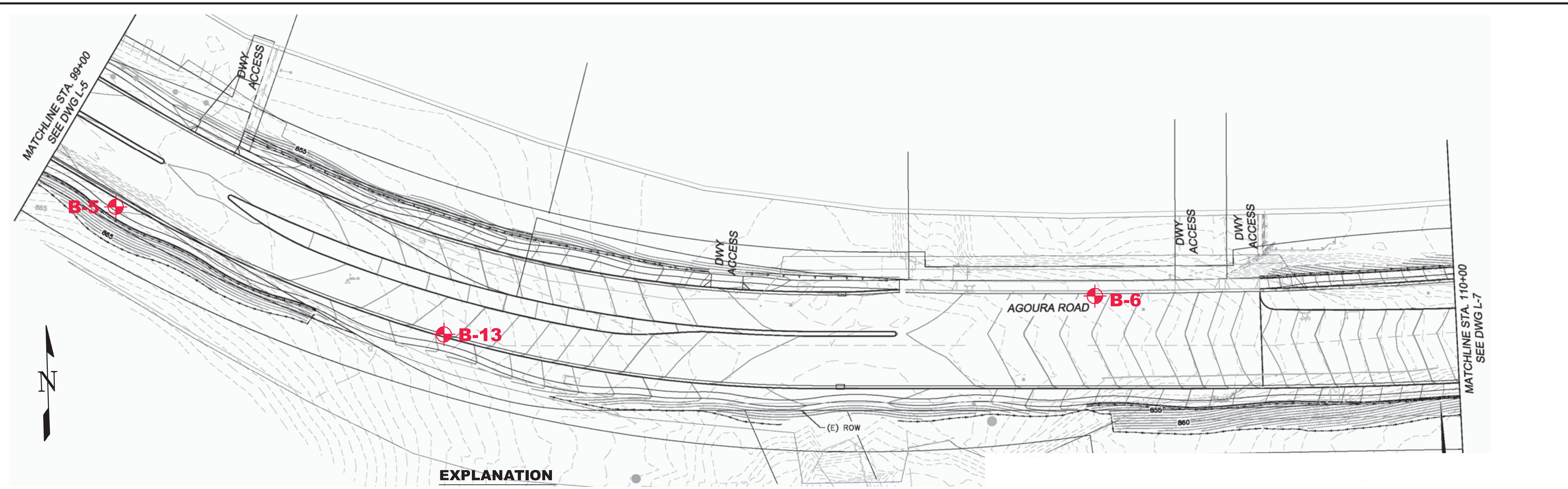

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APPROXIMATE SCALE (feet)

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EXPLANATION

B-13 APPROXIMATE SOIL BORING LOCATION

7 SLOPE NUMBER

A A' CROSS-SECTION LOCATION

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BORING LOCATION MAP

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WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE

2E

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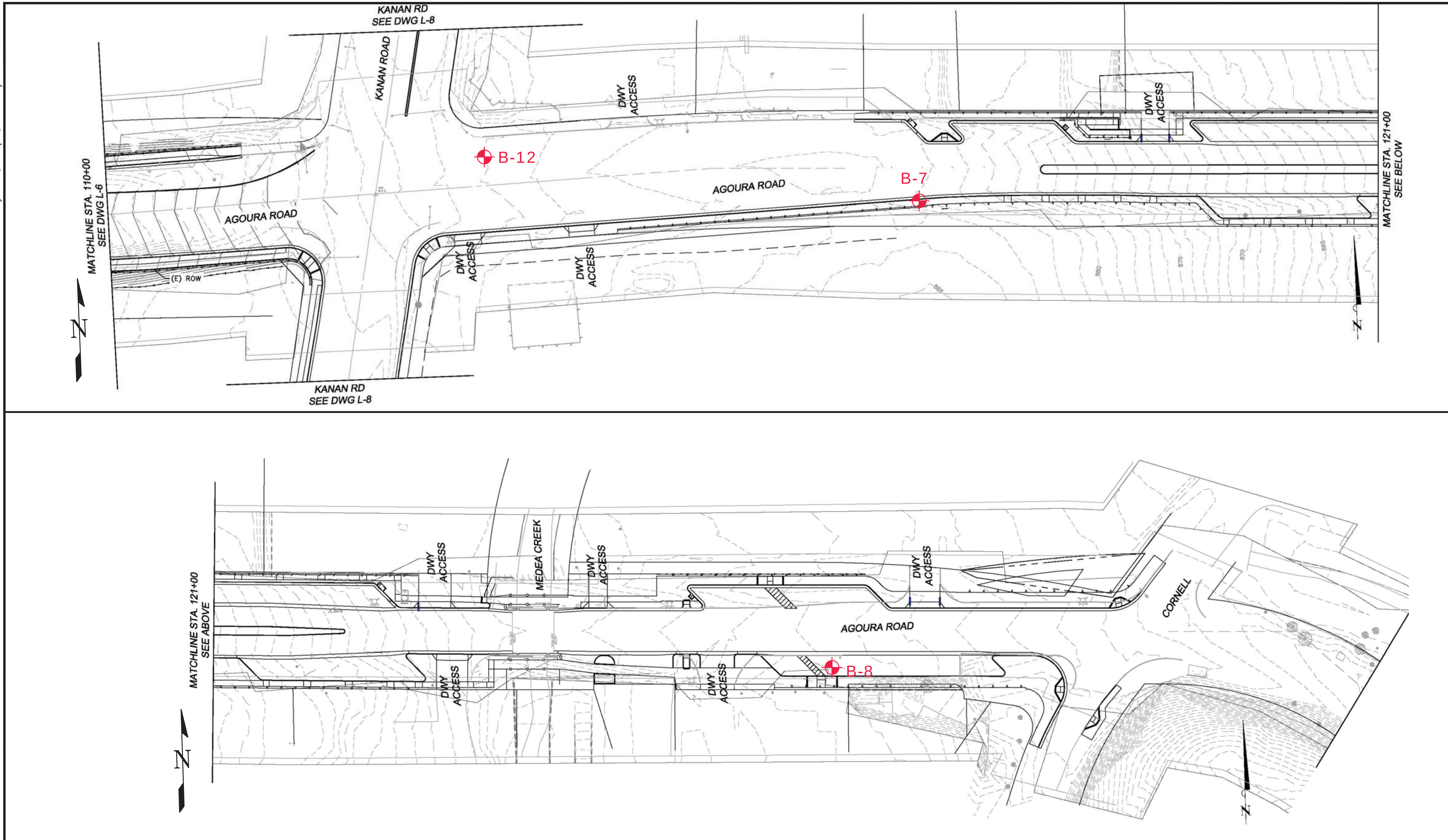

80 40 0 80

APPROXIMATE SCALE (feet)

EXPLANATION

B-12 APPROXIMATE SOIL BORING LOCATION

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BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
2F

ATTACHED IMAGES: Images: Agoura Road Grading (3)_1.png Images: Agoura Road Grading (3)_2.png Images: Agoura Road Grading (3)_3.png Images: Agoura Road Grading (3)_4.png Images: Agoura Road G
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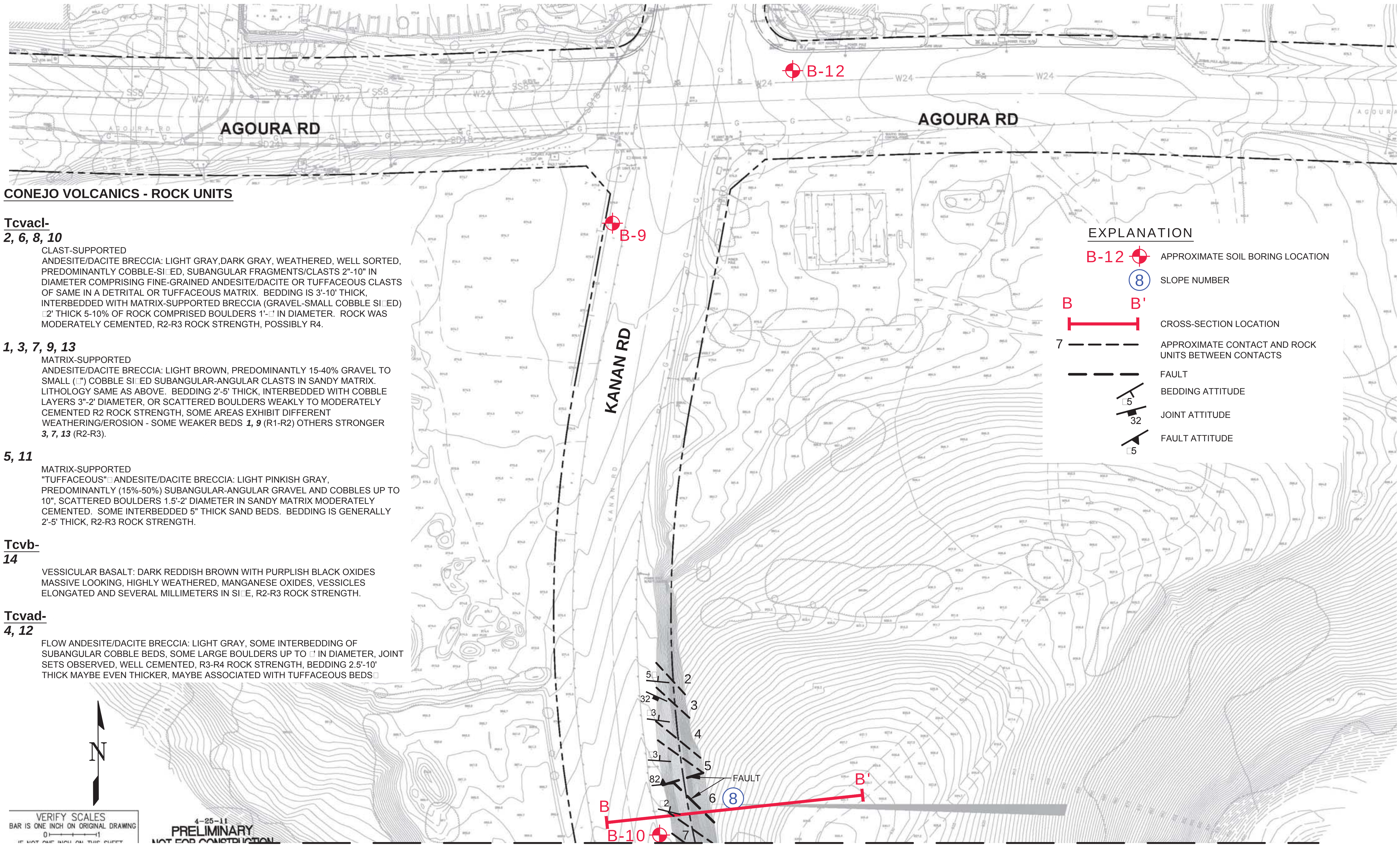

CONEJO VOLCANICS - ROCK UNITS

Tcvacl-
2, 6, 8, 10

CLAST-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT GRAY,DARK GRAY, WEATHERED, WELL SORTED, PREDOMINANTLY COBBLE-SI□ED, SUBANGULAR FRAGMENTS/CLASTS 2"-10" IN DIAMETER COMPRISING FINE-GRAINED ANDESITE/DACITE OR TUFFACEOUS CLASTS OF SAME IN A DETRITAL OR TUFFACEOUS MATRIX. BEDDING IS 3'-10' THICK, INTERBEDDED WITH MATRIX-SUPPORTED BRECCIA (GRAVEL-SMALL COBBLE SI□ED) □2' THICK 5-10% OF ROCK COMPRISED BOULDERS 1'-□' IN DIAMETER. ROCK WAS MODERATELY CEMENTED, R2-R3 ROCK STRENGTH, POSSIBLY R4.

1, 3, 7, 9, 13

MATRIX-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT BROWN, PREDOMINANTLY 15-40% GRAVEL TO SMALL (□") COBBLE SI□ED SUBANGULAR-ANGULAR CLASTS IN SANDY MATRIX. LITHOLOGY SAME AS ABOVE. BEDDING 2'-5' THICK, INTERBEDDED WITH COBBLE LAYERS 3"-2' DIAMETER, OR SCATTERED BOULDERS WEAKLY TO MODERATELY CEMENTED R2 ROCK STRENGTH, SOME AREAS EXHIBIT DIFFERENT WEATHERING/EROSION - SOME WEAKER BEDS ***1, 9*** (R1-R2) OTHERS STRONGER ***3, 7, 13*** (R2-R3).

5, 11

MATRIX-SUPPORTED
"TUFFACEOUS"□ ANDESITE/DACITE BRECCIA: LIGHT PINKISH GRAY, PREDOMINANTLY (15%-50%) SUBANGULAR-ANGULAR GRAVEL AND COBBLES UP TO 10", SCATTERED BOULDERS 1.5'-2' DIAMETER IN SANDY MATRIX MODERATELY CEMENTED. SOME INTERBEDDED 5" THICK SAND BEDS. BEDDING IS GENERALLY 2'-5' THICK, R2-R3 ROCK STRENGTH.

Tcvb-
14

VESSICULAR BASALT: DARK REDDISH BROWN WITH PURPLISH BLACK OXIDES MASSIVE LOOKING, HIGHLY WEATHERED, MANGANESE OXIDES, VESSICLES ELONGATED AND SEVERAL MILLIMETERS IN SI□E, R2-R3 ROCK STRENGTH.

Tcvad-
4, 12

FLOW ANDESITE/DACITE BRECCIA: LIGHT GRAY, SOME INTERBEDDING OF SUBANGULAR COBBLE BEDS, SOME LARGE BOULDERS UP TO □' IN DIAMETER, JOINT SETS OBSERVED, WELL CEMENTED, R3-R4 ROCK STRENGTH, BEDDING 2.5'-10' THICK MAYBE EVEN THICKER, MAYBE ASSOCIATED WITH TUFFACEOUS BEDS□

EXPLANATION

B-12	APPROXIMATE SOIL BORING LOCATION
8	SLOPE NUMBER
B — B'	CROSS-SECTION LOCATION
7	APPROXIMATE CONTACT AND ROCK UNITS BETWEEN CONTACTS
	FAULT
□5	BEDDING ATTITUDE
32	JOINT ATTITUDE
□5	FAULT ATTITUDE

80 40 0 80
APPROXIMATE SCALE (feet)

REFERENCE: BASE MAP PROVIDED BY AECOM, DATED 04/25/11

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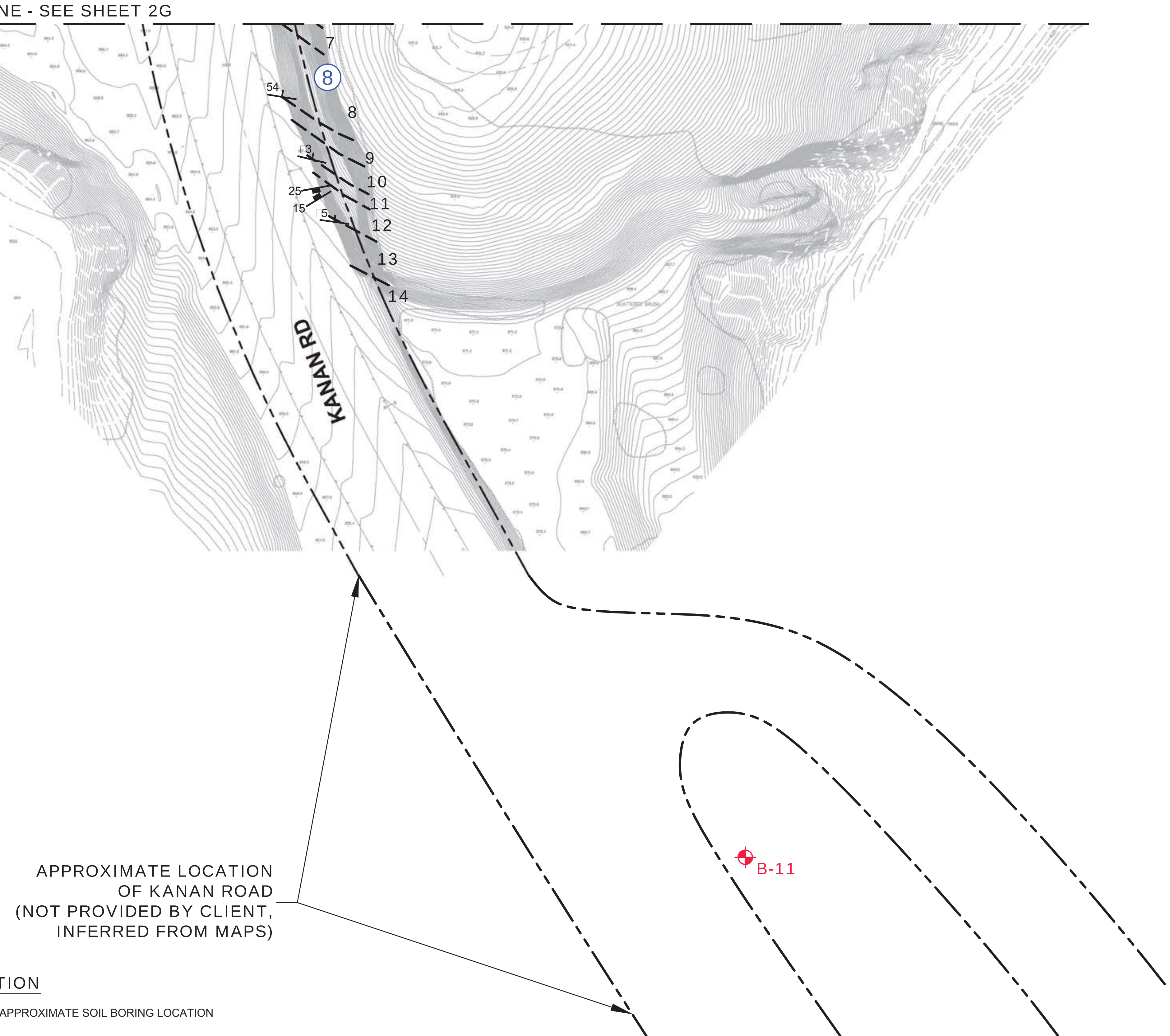
PROJECT NO.	113541	BORING LOCATION MAP
DRAWN:	5/2012	
DRAWN BY:	DMF	
CHECKED BY:	JDW	AGOURA ROAD AND KANAN ROAD WIDENING PROJECT CITY OF AGOURA HILLS, CALIFORNIA
FILE NAME:	113541p2.dwg	

PLATE
2G

ATTACHED IMAGES: Images: Agoura Road Grading (3)_1.png Images: Agoura Road Grading (3)_2.png Images: Agoura Road Grading (3)_3.png Images: Agoura Road Grading (3)_4.png Images: Agoura Road G
ATTACHED XREFS:
DIAMOND BAR, CA
CAD FILE: L:\2012\CADD\113541\ LAYOUT: 2G
PLOTTED: 22 May 2012, 9:28pm, dfahrney



CONEJO VOLCANICS - ROCK UNITS

Tcvacl-
2, 6, 8, 10

CLAST-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT GRAY,DARK GRAY, WEATHERED, WELL SORTED, PREDOMINANTLY COBBLE-SI□ED, SUBANGULAR FRAGMENTS/CLASTS 2"-10" IN DIAMETER COMPRISING FINE-GRAINED ANDESITE/DACITE OR TUFFACEOUS CLASTS OF SAME IN A DETRITAL OR TUFFACEOUS MATRIX. BEDDING IS 3'-10' THICK, INTERBEDDED WITH MATRIX-SUPPORTED BRECCIA (GRAVEL-SMALL COBBLE SI□ED) □2' THICK 5-10% OF ROCK COMPRISED BOULDERS 1'-□' IN DIAMETER. ROCK WAS MODERATELY CEMENTED, R2-R3 ROCK STRENGTH, POSSIBLY R4.

1, 3, 7, 9, 13

MATRIX-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT BROWN, PREDOMINANTLY 15-40% GRAVEL TO SMALL (□") COBBLE SI□ED SUBANGULAR-ANGULAR CLASTS IN SANDY MATRIX. LITHOLOGY SAME AS ABOVE. BEDDING 2'-5' THICK, INTERBEDDED WITH COBBLE LAYERS 3"-2' DIAMETER, OR SCATTERED BOULDERS WEAKLY TO MODERATELY CEMENTED R2 ROCK STRENGTH, SOME AREAS EXHIBIT DIFFERENT WEATHERING/EROSION - SOME WEAKER BEDS ***1, 9*** (R1-R2) OTHERS STRONGER ***3, 7, 13*** (R2-R3).

5, 11

MATRIX-SUPPORTED
"TUFFACEOUS"□ANDESITE/DACITE BRECCIA: LIGHT PINKISH GRAY, PREDOMINANTLY (15%-50%) SUBANGULAR-ANGULAR GRAVEL AND COBBLES UP TO 10", SCATTERED BOULDERS 1.5'-2' DIAMETER IN SANDY MATRIX MODERATELY CEMENTED. SOME INTERBEDDED 5" THICK SAND BEDS. BEDDING IS GENERALLY 2'-5' THICK, R2-R3 ROCK STRENGTH.

Tcvb-
14

VESSICULAR BASALT: DARK REDDISH BROWN WITH PURPLISH BLACK OXIDES MASSIVE LOOKING, HIGHLY WEATHERED, MANGANESE OXIDES, VESSICLES ELONGATED AND SEVERAL MILLIMETERS IN SI□E, R2-R3 ROCK STRENGTH.

Tcvad-
4, 12

FLOW ANDESITE/DACITE BRECCIA: LIGHT GRAY, SOME INTERBEDDING OF SUBANGULAR COBBLE BEDS, SOME LARGE BOULDERS UP TO □' IN DIAMETER, JOINT SETS OBSERVED, WELL CEMENTED, R3-R4 ROCK STRENGTH, BEDDING 2.5'-10' THICK MAYBE EVEN THICKER, MAYBE ASSOCIATED WITH TUFFACEOUS BEDS□

N

80 40 0 80
APPROXIMATE SCALE (feet)

EXPLANATION

B-11 APPROXIMATE SOIL BORING LOCATION
8 SLOPE NUMBER
14 APPROXIMATE CONTACT AND ROCK UNITS BETWEEN CONTACTS
FAULT
5 BEDDING ATTITUDE
32 JOINT ATTITUDE

REFERENCE: BASE MAP PROVIDED BY AECOM, DATED 04/25/11

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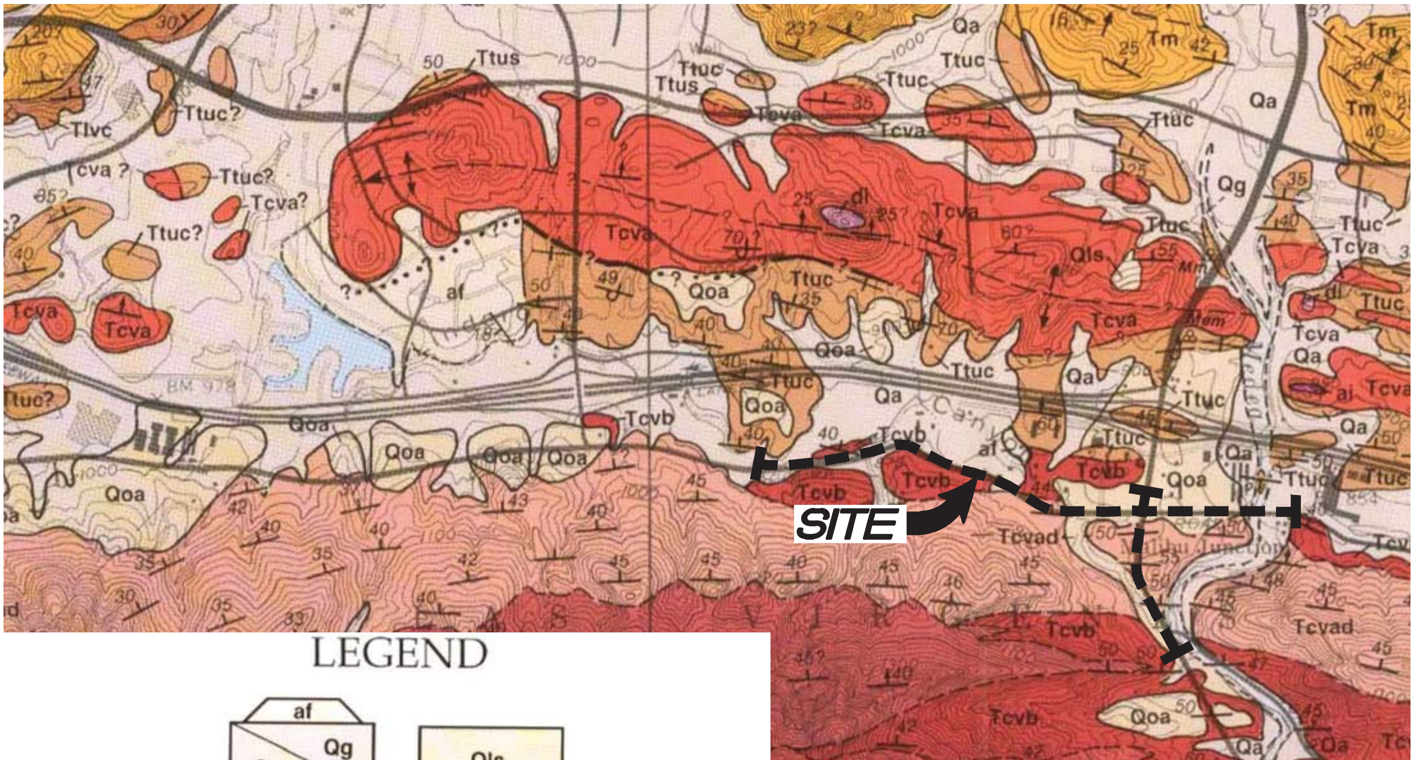
PROJECT NO.	113541
DRAWN:	5/2012
DRAWN BY:	DMF
CHECKED BY:	JDW
FILE NAME:	113541p2.dwg

BORING LOCATION MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
2H

ATTACHED IMAGES: Images: Agoura Road Grading (3)_1.png Images: Agoura Road Grading (3)_2.png Images: Agoura Road Grading (3)_3.png Images: Agoura Road Grading (3)_4.png Images: Agoura Road G
ATTACHED XREFS:
DIAMOND BAR, CA
CAD FILE: L:\2012\CADD\113541\ LAYOUT: 2H PLOTTED: 22 May 2012, 9:28pm, dfahrney



SOURCE: U.S.G.S. 7.5' Topographic series, Thousand Oaks, California Quadrangle 1950, Photorevised 1981. Dibblee Geologic series, dated 1993.

LEGEND

af
Qa
Qg
Qls

SURFICIAL SEDIMENTS

unconsolidated detrital sediments, undissected to partly dissected

Qg *gravel and sand of major stream channels*
Qa *alluvial gravel, sand and clay of valley areas*
af *artificial cut and fill*
Qls *landslide debris*

Qc
Qoa

OLDER SURFICIAL SEDIMENTS

unconsolidated to weakly consolidated alluvial sediments, dissected where elevated; late Pleistocene age

Qoa *older dissected alluvial gravel*
Qc *talus fan gravel and sand, locally indurated*

Tcvbb Tcva
Tcvs Tcvad Tcvar
Tcvab Tcvby
Tcvbp Tcvb

EXTRUSIVE ROCKS

Tcvad *andesite-dacite breccia of Westlake (designated as Tcvab in Calabasas quadrangle of Dibblee 1992); light colored (light pinkish gray to light brown), composed of moderately to poorly sorted, mostly cobble-boulder sized angular fragments of light colored, very fine grained feldspathic andesite-dacite in semi-coherent, detrital or tuffaceous (?) matrix of same rock; crudely stratified; in eastern exposures well stratified and many fragments subrounded; in western exposures near Lake Sherwood some of lower part of breccia is firmly indurated to form prominent, resistant outcrops; unit gradational into Tcva; probably deposited subaerially as laharic (rock and mudflow) and reworked detritus from nearby volcanic source to west; prominently exposed near Westlake Dam; lithologically similar to and may be equivalent to more coherent unit Tcvdb in Triunfo Pass and Newbury Park quadrangles (Dibblee and Ehrenspeck 1990a and b)*

Tcvb *basaltic flows and breccias, black to dark gray, weather dark olive-brown, fine grained, massive to vaguely bedded, incoherent and crumbly where weathered, weakly resistant to erosion; range from basalt to basaltic andesite composed of plagioclase feldspar (average An_{50}) and ferromagnesian minerals (hypersthene or augite, rarely olivine, [Weigand 1982]), in places amygdaloidal; in a few places, mostly in northwestern area, includes a few thin lenses up to a few meters thick of dark gray basaltic sandstone and siltstone; probably deposited subaerially, possibly in part under shallow sea*

N

2,000 1,000 0 2,000
APPROXIMATE SCALE (feet)

PLOTTED: 18 May 2011, 2:51pm, dfahrney
CAD FILE: L:\2011\CADD\113541\Agoura-Kanan_GEO\ LAYOUT: 1
Images: Thousand Oaks Quad geology (LA Ventura Counties) Dibblee DF-49.png
ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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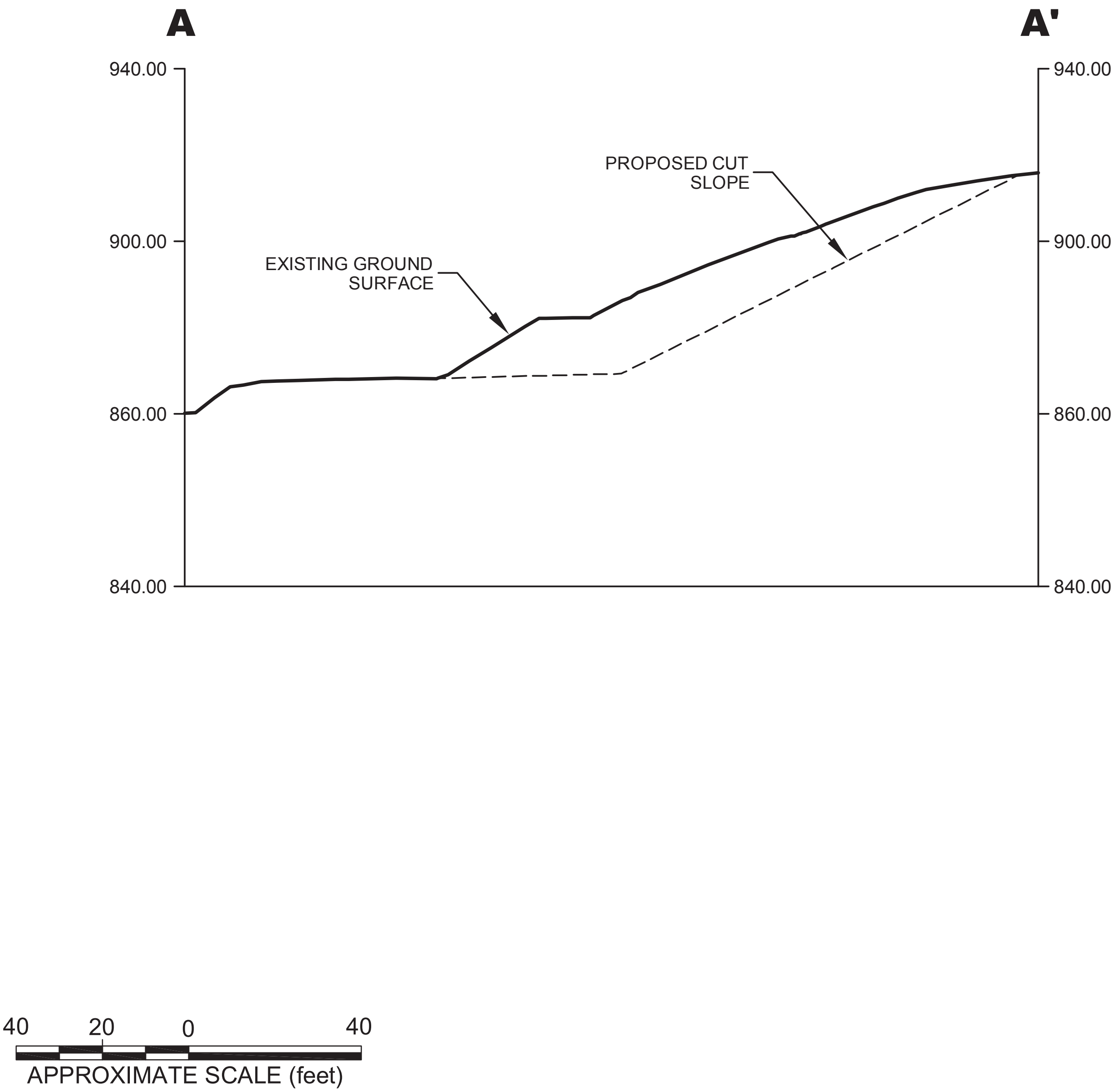
PROJECT NO.	113541
DRAWN:	5/2011
DRAWN BY:	DMF
CHECKED BY:	JDW
FILE NAME:	113541p3.dwg

REGIONAL GEOLOGIC MAP

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
3



PLOTTED: 26 May 2011, 5:09pm, dfahrney

CAD FILE: L:\2011\CADD\113541\Agoura-Kanan_GEO\ LAYOUT: 4A

Images: temp-delete_scan CS-B_pl4.jpg

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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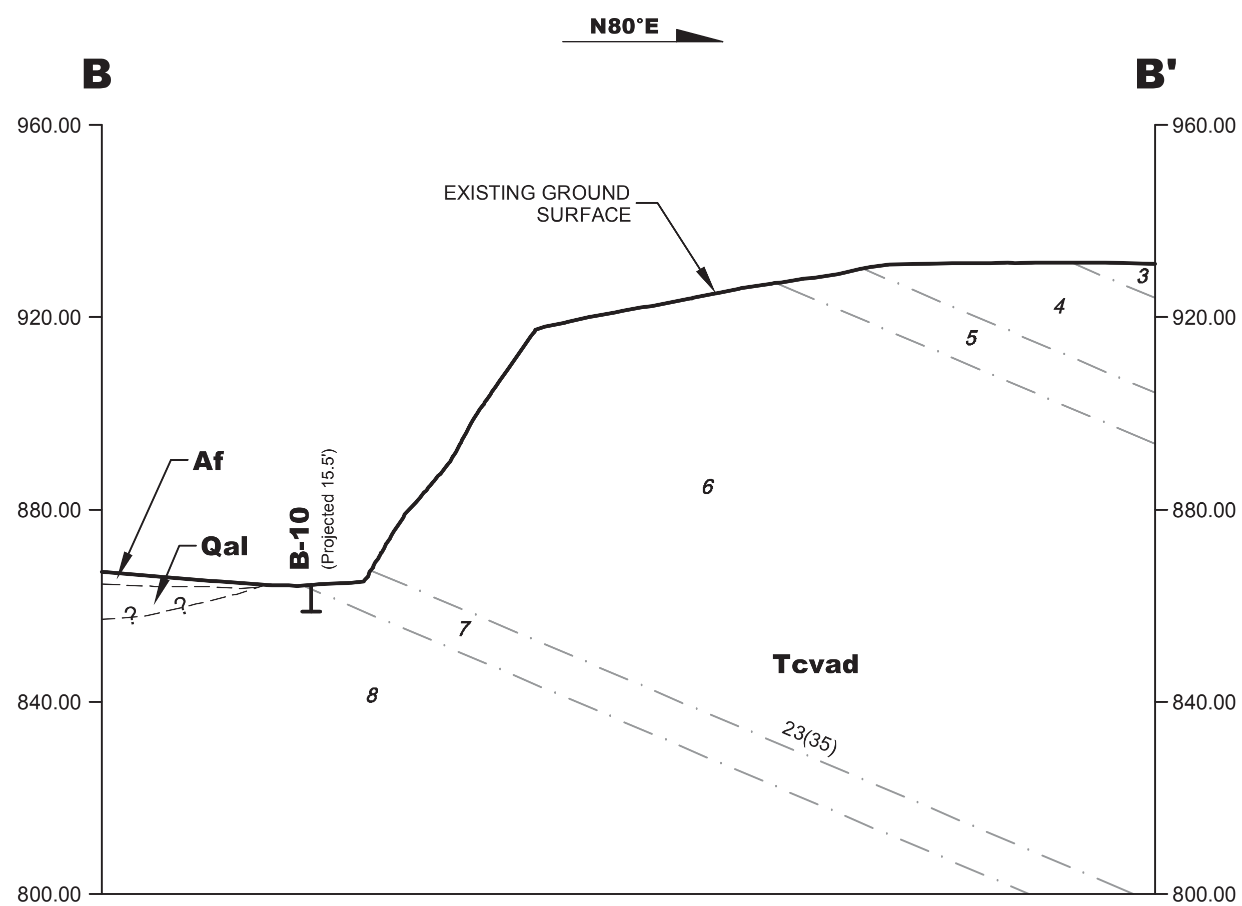
PROJECT NO. 113541	**CROSS-SECTION A-A'**	PLATE
DRAWN: 5/2011		**4A**
DRAWN BY: MRG		
CHECKED BY: EP	AGOURA ROAD AND KANAN ROAD WIDENING PROJECT CITY OF AGOURA HILLS, CALIFORNIA	
FILE NAME: 113541p4.dwg		



NOTE: LOCATION OF GEOLOGIC CONTACTS ON CROSS-SECTION ARE APPROXIMATE

EXPLANATION

af ARTIFICIAL FILL

Qal ALLUVIUM, CIRCLED WHERE BURIED

– – – –?– GEOLOGIC CONTACT, QUERIED WHERE INFERRED.

CONEJO VOLCANICS - ROCK UNITS

Tcvacl-
2, 6, 8, 10

CLAST-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT GRAY,DARK GRAY, WEATHERED, WELL SORTED, PREDOMINANTLY COBBLE-SIZED, SUBANGULAR FRAGMENTS/CLASTS 2"-10" IN DIAMETER COMPRISING FINE-GRAINED ANDESITE/DACITE OR TUFFACEOUS CLASTS OF SAME IN A DETRITAL OR TUFFACEOUS MATRIX. BEDDING IS 3'-10' THICK, INTERBEDDED WITH MATRIX-SUPPORTED BRECCIA (GRAVEL-SMALL COBBLE SIZED) ~2' THICK 5-10% OF ROCK COMPRISED BOULDERS 1'-7' IN DIAMETER. ROCK WAS MODERATELY CEMENTED, R2-R3 ROCK STRENGTH, POSSIBLY R4.

1, 3, 7, 9, 13

MATRIX-SUPPORTED
ANDESITE/DACITE BRECCIA: LIGHT BROWN, PREDOMINANTLY 15-40% GRAVEL TO SMALL (6") COBBLE SIZED SUBANGULAR-ANGULAR CLASTS IN SANDY MATRIX. LITHOLOGY SAME AS ABOVE. BEDDING 2'-5' THICK, INTERBEDDED WITH COBBLE LAYERS 3"-2' DIAMETER, OR SCATTERED BOULDERS WEAKLY TO MODERATELY CEMENTED R2 ROCK STRENGTH, SOME AREAS EXHIBIT DIFFERENT WEATHERING/EROSION - SOME WEAKER BEDS ***1, 9*** (R1-R2) OTHERS STRONGER ***3, 7, 13*** (R2-R3).

5, 11

MATRIX-SUPPORTED
"TUFFACEOUS"? ANDESITE/DACITE BRECCIA: LIGHT PINKISH GRAY, PREDOMINANTLY (15%-50%) SUBANGULAR-ANGULAR GRAVEL AND COBBLES UP TO 10", SCATTERED BOULDERS 1.5'-2' DIAMETER IN SANDY MATRIX MODERATELY CEMENTED. SOME INTERBEDDED 5" THICK SAND BEDS. BEDDING IS GENERALLY 2'-5' THICK, R2-R3 ROCK STRENGTH.

Tcvb-
14

VESSICULAR BASALT: DARK REDDISH BROWN WITH PURPLISH BLACK OXIDES MASSIVE LOOKING, HIGHLY WEATHERED, MANGANESE OXIDES, VESSICLES ELONGATED AND SEVERAL MILLIMETERS IN SIZE, R2-R3 ROCK STRENGTH.

Tcvad-
4, 12

FLOW ANDESITE/DACITE BRECCIA: LIGHT GRAY, SOME INTERBEDDING OF SUBANGULAR COBBLE BEDS, SOME LARGE BOULDERS UP TO 7' IN DIAMETER, JOINT SETS OBSERVED, WELL CEMENTED, R3-R4 ROCK STRENGTH, BEDDING 2.5'-10' THICK MAYBE EVEN THICKER, MAYBE ASSOCIATED WITH TUFFACEOUS BEDS?

40 20 0 40
APPROXIMATE SCALE (feet)

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PROJECT NO.	113541
DRAWN:	5/2011
DRAWN BY:	MRG
CHECKED BY:	EP
FILE NAME:	113541p4.dwg

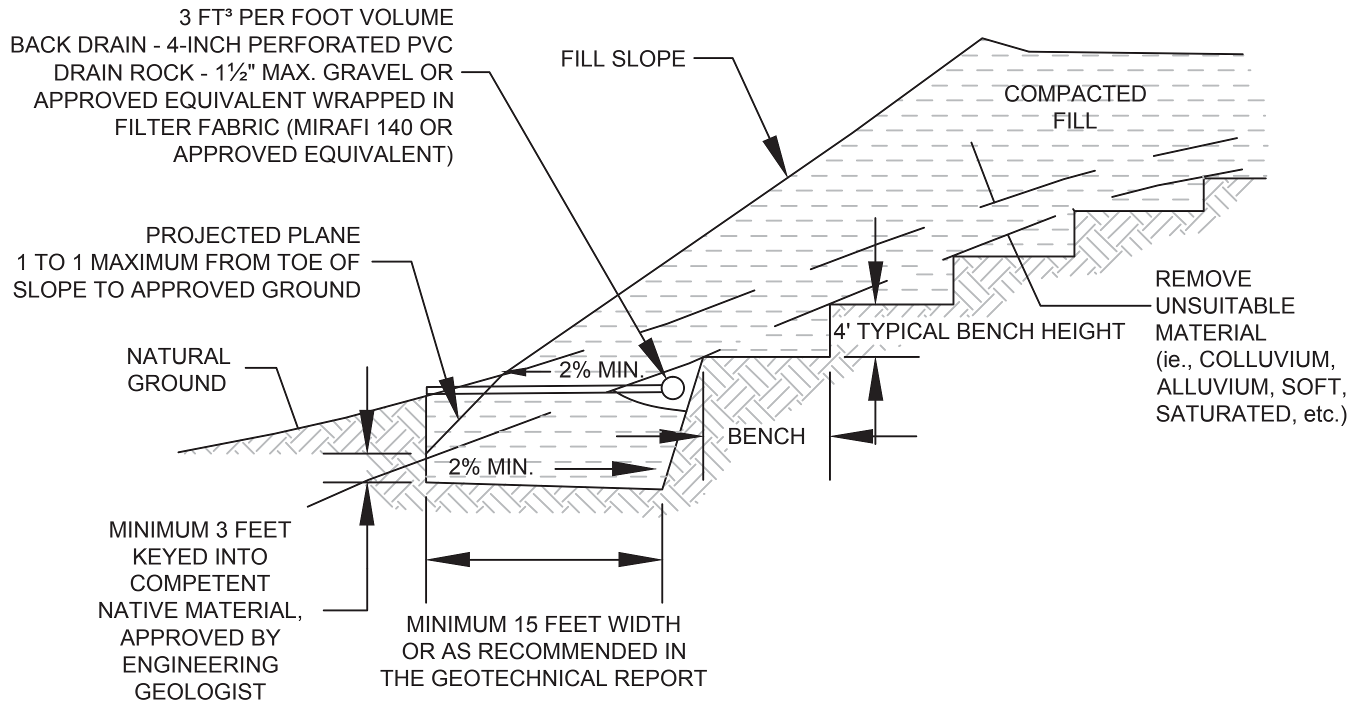
CROSS-SECTION B-B'

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
4B

PLOTTED: 26 May 2011, 3:43pm, dfahrney
CAD FILE: L:\2011\CADD\113541\Agoura-Kanan_GEO\ LAYOUT: 4B
ATTACHED IMAGES: Images: temp-delete_scan CS-B_pl4.jpg
ATTACHED XREFS:
DIAMOND BAR, CA

BENCHING DETAILS



NOTES:

1) LOWEST BENCH: DEPTH AND WIDTH SUBJECT TO FIELD CHANGE BASED ON CONSULTANT'S INSPECTION.

2) SUBDRAINAGE: BACK DRAINS MAY BE ELIMINATED AT THE DISCRETION OF THE GEOTECHNICAL CONSULTANT, BASED ON FIELD CONDITIONS.

3) SUBDRAIN INSTALLATION: SUBDRAIN PIPE SHALL BE INSTALLED WITH PERFORATIONS DOWN, OR AT LOCATIONS DESIGNATED BY THE GEOTECHNICAL CONSULTANT. OUTLET PIPE SHALL BE NON-PERFORATED PIPE.

4) SUBDRAIN TYPE: SUBDRAIN TYPE SHALL BE SCHEDULE 40 PVC PIPE OR AN APPROVED EQUIVALENT.

PLOTTED: 18 May 2011, 2:59pm, dfahrney
CAD FILE: L:\2011\CADD\113541\Agoura-Kanan_GEO\ LAYOUT: 1
ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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PROJECT NO.	113541
DRAWN:	5/2011
DRAWN BY:	DMF
CHECKED BY:	JDW
FILE NAME:	113541p5.dwg

BENCHING DETAILS

AGOURA ROAD AND KANAN ROAD
WIDENING PROJECT
CITY OF AGOURA HILLS, CALIFORNIA

PLATE
5

APPENDIX A

Field Explorations

APPENDIX A
Field Explorations

The subsurface exploration program for the proposed project consisted of excavating and logging a total of 11 hollow-stem auger borings and three pavement cores (called out as Borings B-12 through B-14). The borings were drilled with a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers, provided by Cal Pac Drilling of Calimesa, California. The approximate locations of the borings are shown on Plates 2A through 2H, Boring Location Map.

The logs of the borings are presented as Plates A-2 through A-12, Log of Borings. An explanation to the logs is presented on Plates A-1a and A-1b, Explanation of Logs. The logs of borings present a description of the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, boring elevation and the name of the logger and drilling subcontractor. A Kleinfelder staff professional logged the borings utilizing the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and drive samples of representative earth materials were obtained from the borings at maximum intervals of approximately 5 feet. At the conclusion of drilling, each boring was backfilled with soil cuttings. Borings in paved areas were patched with concrete.

A California sampler was used to obtain relatively undisturbed drive samples of the soil encountered. This sampler consists of a 3 inch O.D., 2.5 inch I.D. split barrel shaft that is driven a total of 18 inches into the soil at the bottom of the boring. The soil was retained in six 1-inch brass rings for laboratory testing. The sampler was driven using a 140-pound automatic hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings. Where the sample was driven less than 12 inches, the number of blows to drive the sample for each 6-inch segment, or portion thereof, is shown on the logs.

Bulk samples of the sub-surface soils were directly retrieved from the soil cuttings produced by the auger blades.

Date Drilled:
Drilled By:
Drilling Method:
Logged By:

Water Depth:
Date Measured:
Reference Elevation:
Datum:

Elevation (feet) Depth	Sample	Sample No.	Blow Count (Blows/ft.)	Graphic Log	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		1	6			108	10	DS, SE
		2	12					GS
5								
	(1)	(2)	(3)	(4)	(5)	(6)	(6)	(7)

NOTES ON FIELD INVESTIGATION

1. SAMPLE – Graphical representation of sample type as shown below.
   Split Spoon – Standard Penetration Test Sample (SPT)
   Drive Sample – California Sample (Cal)
   Bulk Sample – Obtained by collecting cuttings in a plastic bag
   Tube Sample – Shelby/Pitcher Tube Sample

2. SAMPLE NO. – Sample Number

3. BLOWS/FT – Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil at the bottom of the hole with a standard (140 lb) hammer dropping a standard 30 inches. Drive samples collected in bucket auger borings may be obtained by dropping non–standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D–1586.

   SCR/RQD – Sample Core Recovery (SCR) in percent (%) and Rock Quality Designation (RQD) in percent (%). RQD is defined as the percentage of core in each run which the spacing between natural fractures is greater than 4 inches. Mechanical breaks of the core are not considered.

4. GRAPHIC LOG – Standard symbols for soil and rock types, as shown on plate A–1b.

5. GEOTECHNICAL DESCRIPTION

   Soil – Soil classifications are based on the United Soil Classification System per ASTM D–2987, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

   Rock – Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.

   Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil.

6. DRY DENSITY, MOISTURE CONTENT: As estimated by laboratory or field testing.

7. ADDITIONAL TESTS – (Indicates sample tested for properties other than the above):

MAX – Maximum Dry Density	SG – Specific Gravity	PP – Pocket Penetrometer
GS – Grain Size Distribution	HA – Hydrometer Analysis	WA – Wash Analysis
SE – Sand Equivalent	AL – Atterberg Limits	DS – Direct Shear
EI – Expansion Index	RV – R–Value	CP – Collapse Potential
CHEM – Sulfate and Chloride Content, pH, Resistivity	CN – Consolidation	UC – Unconfined Compression
PM – Permeability	CU – Consolidation Undrained Triaxial	T – Torvane
UU – Unconsolidated Undrained Triaxial	CD – Consolidated Drained Triaxial	PI – Plasticity Index

8. ATTITUDES – Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceeded by a one–letter symbol denoting nature of discontinuity as shown below.

   B: Bedding Plane J: Jointing C: Contact F: Fault S: Shear

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EXPLANATION OF LOGS

PLATE
A-1a

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COURSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COURSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COURSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY - COARSE - GRAIN SOIL

RELATIVE DENSITY	SPT * (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very Dense	>50	85 - 100

CONSISTENCY-FINE-GRAIN SOIL

		TORVANE	POCKET ** PENETROMETER
CONSISTENCY	SPT (# blows/ft)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium Stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very Stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM-1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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EXPLANATION OF LOGS

PLATE
A-1b

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	970 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						Asphaltic Concrete (AC): approximately 8 inches thick; aggregate base not observed			
			G1	N/A		**Sandy Lean Clay (CL):** brown to yellowish brown, moist, fine to coarse sand, fine gravel, chunks of asphalt		14.3	RV = 26 CHEM, MAX
965	5					**Silty Sand with Gravel (SM):** gray to reddish brown, moist, dense, fine to coarse sand, fine to coarse gravel up to 2-inch diameter			
			1	46			111	15.3	GS, HA, WA = 33%
						Silty Gravel with Sand (GM): mottled gray, dark gray, moist, medium dense, fine to coarse sand, fine gravel			
960	10								
			2A				100	15.8	GS, WA = 25%
			2B	37			102	17.4	

Boring terminated at a depth of 11.5 ft below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings and capped with concrete.

GEOTECH DB AGOURA RD WIDENING GINT.GPJ KA_RDLND.GDT 5/17/11

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PROJECT NO. 113541

Agoura Road and Kanan Road Widening Project
City of Agoura Hills, California

LOG OF BORING B-1

PLATE A-2

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	Hand Auger	Elevation:	881 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
880		G1	N/A		**Gravelly Silt (ML):** dark brown, moist to wet, gravel up to 2-inch diameter		23.5	GS, HA, WA = 52%
					Boring terminated at a depth of 1.7 ft below existing site grade. Tried hand augering in 3 locations, all reached refusal at 1.7 ft on a gravel layer. Groundwater was not encountered. Boring was backfilled with soil cuttings.			

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City of Agoura Hills, California

LOG OF BORING B-2

PLATE

A-3

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/14/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/14/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	888 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			G1	N/A		**Artificial Fill:** **Sandy Gravel (GP):** brown/grey, slightly moist, gravel to 3-inch diameter			
885						**Silty Sand/ Clayey Sand (SM/SC):** yellowish brown, dry to moist, very dense, fine to coarse sand			
	5		1	50/5"		**Bedrock:** **Conejo Volcanics:** Andesite/Dacite Breccia, gray, very dense, moderately cemented, cobble sized clasts, detrital matrix, subangular clasts, recovered as sand with gravel	104	18.9	
880									
	10		2	60/6"			95	10.5	WA = 26% LL = NP
875									
	15		3	50/4"		-- becomes mottled black, yellow brown	100	19.6	

Boring terminated at a depth of 15.3 ft below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings.

GEOTECH DB AGOURA RD WIDENING GINT.GPJ KA_RDLND.GDT 5/22/11

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City of Agoura Hills, California

PLATE

A-4

PROJECT NO. 113541

LOG OF BORING B-3

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/15/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/15/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	871 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
870			G1	N/A		**Sandy Clay (CL):** olive brown to olive gray, moist, fine to coarse sand, few fine gravel		14.5	GS, HA, WA = 53%
	5					**Clayey Sand (SC):** yellowish brown, dry to moist, medium dense, fine to coarse sand			
865			1	37		**Sandy Clay (CL):** dark gray mottled with red, moist, hard	105	12.4	DS
	10					**Clayey Sand/Sandy Lean Clay (SC/CL):** yellowish brown, dry, dense/ very hard, fine to coarse sand			
860			2A 2B	52			103	16.2	
						Bedrock: **Conejo Volcanics:** Andesite/Dacite Breccia, gray, very dense, moderately cemented, cobble sized clasts, detrital matrix, subangular clasts, recovered as silty gravel and clayey gravel			
	15		3	50/4"					
855									
	20		4	50/4"					
						Boring terminated at a depth of 20.3 ft below existing site grade. Groundwater was not encountered. Boring was backfilled with soil cuttings.			

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PLATE

A-5

PROJECT NO. 113541

LOG OF BORING B-4

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	855 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			G1	N/A		**Sandy Clay with Gravel (CL):** olive brown, dry to moist, fine to coarse sand, fine gravel		16.2	GS, HA, WA = 50%
850	5					**Bedrock:** **Conejo Volcanics:** Andesite/Dacite Breccia, gray, very dense, moderately cemented, cobble sized clasts, detrital matrix, subangular clasts, recovered as clayey sand, silty sand with gravel, and silty gravel with sand			
			1	90/10"			105	17.4	DS
845	10		2A 2B	50/5"			100	21.0	
840	15		3	50/4"					
						Boring terminated at a depth of 15.3 ft below existing site grade. Groundwater was not encountered. Boring was backfilled with soil cuttings.			

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PLATE

A-6

PROJECT NO. 113541

LOG OF BORING B-5

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	848 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet) Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
845		G1	N/A		**Clayey Sand (SC):** yellowish brown, moist, fine to coarse sand, fine gravel		11.9	GS, HA, WA = 33% RV = 18 MAX
5 840		1	76		**Sandy lean Clay/ Clayey Sand (CL/SC):** light brownish gray, olive yellow, moist, hard/ very dense, fine to coarse sand	112	14.2	EI = 40
10		2A 2B	75/11"		-- white stringers	109	18.6	
					Boring terminated at a depth of 11.4 ft below existing site grade. Groundwater was not encountered. Boring was backfilled with soil cuttings.			

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LOG OF BORING B-6

PLATE

A-7

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	881 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
880			G1	N/A		**Clayey Sand (SC):** yellowish brown, olive brown, moist, fine to coarse sand, few fine gravel		11.7	GS, HA, WA = 35% CHEM
875	5		1	52		**Silty Sand (SM):** yellowish brown, gray, moist, dense, medium to coarse sand, trace fine gravel	106	12.9	
870	10		2	53		-- becomes mottled yellow, white, gray, yellowish brown, fine to coarse sand, 2-inch diameter gravel in sampler shoe	100	12.8	

Boring terminated at a depth of 11.5 ft below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings.

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PLATE A-8

PROJECT NO. 113541

LOG OF BORING B-7

Drafted By: Reviewed By: **Legend To Logs On Plate A-1**

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	12/13/10	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	12/13/2010
Drilling Method:	L 10 T Hollow-Stem Auger	Elevation:	848 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			G1	N/A		**Asphaltic Concrete (AC):** approximately 4 inches thick; aggregate base not observed		19.4	GS, HA, WA = 46%
845						**Clayey Sand (SC):** dark brown, mottled with yellowish brown, moist, fine to coarse sand, trace gravel up to 3-inch diameter, organic material -- laminated gravel layer, some gravel, trace cobbles up to 4-inch diameter			
	5		1	21		**Silty Sand (SM):** yellowish brown, dry to moist, medium dense, gravel up to 1-inch diameter			
840						**Clayey Sand with Gravel (SC):** yellowish brown, mottled with yellow, olive, brown black, moist, very dense, coarse sand, gravel up to 2-inch diameter			
	10		2A 2B	75/11"					

Encountered hand auger refusal at 3.1 feet, moved over and advanced this boring.
Boring terminated at a depth of 11.4 feet below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings and capped with concrete.

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PLATE A-9

PROJECT NO. 113541

LOG OF BORING B-8

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	4/26/11	Water Depth:	Not Encountered
Drilled By:	CalPac Drilling	Date Measured:	4/26/2011
Drilling Method:	Hollow Stem Auger	Elevation:	843 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			1			**Artificial Fill:** **Clayey Sand (SC):** brown to yellow brown, moist, fine to coarse sand, trace fine gravel			RV=5, PI=24
			2			**Sandy Clay (CL):** mottled olive gray and olive yellow, fine to medium sand, moist, firm			
840			3						
	5		4	23		**Native:** **Silty Sand (SM):** light grayish brown, moist, medium dense, fine to coarse grained, with fine gravel, weakly cemented, some iron oxide staining			
835						**Bedrock:** **Conejo Volcanics:** Andesite/Dacite Breccia, gray, very dense, moderately cemented, cobble sized clasts, detrital matrix, subangular clasts			
	10		5	20, 70					

Boring terminated at a depth of 11 ft below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings.

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	4/26/11	Water Depth:	Not Encountered
Drilled By:	CalPac Drilling	Date Measured:	4/26/2011
Drilling Method:	Hollow Stem Auger	Elevation:	876 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
					Asphaltic Concrete (AC): approximately 12 inches thick; aggregate base not observed			
875			1		**Artificial Fill:** **Clayey Sand (SC):** brown, moist, fine to coarse sand, with gravel up to 2-inch diameter			MAX, DS
	5		2		**Bedrock:** **Conejo Volcanics:** Andesite/Dacite Breccia, gray, very dense, moderately cemented, cobble sized clasts, detrital matrix, subangular clasts			
870					Boring terminated at a depth of 5.5 ft below existing site grade. Groundwater was not encountered. Boring was backfilled with soil cuttings and capped with quickset concrete.			
	10							
865								

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LOG OF BORING B-10

PLATE A-10

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	4/26/11	Water Depth:	Not Encountered
Drilled By:	CalPac Drilling	Date Measured:	4/26/2011
Drilling Method:	Hollow Stem Auger	Elevation:	864 feet (approx.)
Logged By:	K. Sarwold	Datum:	MSL

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						Artificial Fill:			
			1			**Silty Sand (SM):** yellow to olive yellow, dry, fine to coarse sand, with gine gravel			MAX
						Asphaltic Concrete (AC): approximately 6 inches thick, aggregate base not observed			
860	5					**Artificial Fill:** **Clayey Sand (SC):** brown to yellow brown, moist, medium dense, fine to coarse sand, with fine gravel			
			2	29 17			87	15.7	
						Native: **Clayey Sand (SC):** brown to yellow brown, moist, medium dense, fine to coarse sand, with fine gravel. -- brown, increase in clay content			
			3			-- coarse gravel observed.	105	12.3	

Boring terminated at a depth of 11.5 ft below existing site grade.
Groundwater was not encountered.
Boring was backfilled with soil cuttings.

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Agoura Road and Kanan Road Widening Project
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PLATE

A-12

PROJECT NO. 113541

LOG OF BORING B-11

GEOTECH DB KANAN ROAD GINT.GPJ KA_RDLND.GDT 5/19/11

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

APPENDIX B

Laboratory Testing

APPENDIX B
Laboratory Testing

GENERAL

Laboratory tests were performed on selected samples as an aid in classifying the soils and to evaluate physical properties of the soils that may affect foundation design and construction procedures. The tests were performed in general conformance with the current ASTM or California Department of Transportation (Caltrans) standards. A description of the laboratory-testing program is presented below.

Laboratory tests were performed on representative relatively undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with one of the following references:

1. Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951
2. Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970
3. ASTM Standards for Soil Testing, latest revisions
4. State of California Department of Transportation, Standard Test Methods, latest revisions.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on selected soil samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The results are presented on the Logs of Borings and are summarized in Table B-1, Moisture Content and Unit Weight.

WASH SIEVE

The percent passing the #200 sieve of nine soil samples was performed by wash sieving in accordance with ASTM Standard Test Method D422-63. The test results are summarized in Table B-2, Wash Sieve Test Results.

SIEVE ANALYSIS

Sieve analyses were performed on eight samples of the materials encountered at the site to evaluate the grain size distribution characteristics of the soils and to aid in their classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented as Plates B-1 and B-2, Grain Size Distribution.

HYDROMETER

Hydrometer testing was performed on eight selected soil samples to determine the gradation characteristics of the fine grain soil passing the #200 sieve, and to aid in the classification of the soil. The tests were performed in general accordance with ASTM Test Method D 422. Results of the testing are presented on Plates B-1 and B-2.

EXPANSION INDEX

Expansion index testing was performed on one bulk samples of the near-surface soils to evaluate their expansion characteristics. The test was performed in accordance with UBC Standard No. 18-2, Expansion Index Test Method. The test result is presented in Table B-3, Expansion Index Test Results and may be compared to the table presented below to qualitatively evaluate the expansion potential of the near-surface site soils.

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

PLASTICITY INDEX

Plasticity index testing was performed on two selected samples of the on-site soils to determine plasticity characteristics and to aid in the classification of the soil. The tests were performed in accordance with ASTM Standard Test Method D 4318. The results are presented on Plate B-3, Plasticity Index Test.

DIRECT SHEAR

Direct shear testing was conducted on three samples to evaluate the shear strength parameters of representative on-site soils. The sample from B-10 was taken from a bulk sample and remolded to 90% relative compaction for the test. Each sample was tested in a saturated state in general accordance with ASTM Test Method D3080-90. The test results are presented on Plate B-4 through B-6, Direct Shear Test.

MAXIMUM DENSITY/OPTIMUM MOISTURE TEST

Four maximum density/optimum moisture tests were performed on select bulk samples of the on-site soils to determine compaction characteristics. The tests were performed in accordance with ASTM Standard Test Method D-1557-91. The test results are presented in Table B-4, Maximum Density / Optimum Moisture Test Results.

R-VALUE TEST

Three resistance value (R-value) tests were performed to evaluate support characteristics of the near-surface onsite soils. R-value testing was performed in accordance with Caltrans Standard Test Method 301. The test results are presented in Table B-5, R-Value Test Results.

CORROSIVITY TESTS

A series of chemical tests were performed on two representative soil samples collected from the borings to estimate pH, sulfate content, chloride content, and electrical resistivity. The test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to the construction materials. The results of the tests are presented in Table B-6, Corrosion Test Results.

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Table B-1
Moisture Content and Unit Weight

Boring	Depth (ft)	Moisture Content (%)	Dry Unit Weight (pcf)
B – 1	0 – 5	14.3	--
B – 1	6	15.3	111
B – 1	10.5	15.8	100
B – 1	11	17.4	102
B – 2	0 – 1.6	23.5	--
B – 3	5.5	18.9	104
B – 3	11	10.5	95
B – 3	15	19.6	100
B – 4	0 – 4	14.5	--
B – 4	6	12.4	105
B – 4	11	16.2	103
B – 5	0 – 5	16.2	--
B – 5	6	17.4	105
B – 5	10.5	21.0	100
B – 6	1 – 5	11.9	--
B – 6	6	14.2	112
B – 6	11	18.6	109
B – 7	0 – 5	11.7	--
B – 7	6	12.9	106
B – 8	11	12.8	100
B – 8	0.3 – 5	19.4	--
B – 11	6	15.7	87
B – 11	11	12.3	105

– denotes dry unit weight test was not performed due to sample disturbance

Table B-2
Wash Sieve Test Results

Boring	Depth (ft)	Percent Passing No. 200 Sieve
B – 1	6	33
B – 1	11	25
B – 2	0 – 1.5	52
B – 3	10	26
B – 4	0 – 4	53
B – 5	0 – 5	50
B – 6	1 – 5	33
B – 7	0 – 5	35
B – 8	1 – 4	46

Table B-3
Expansion Index Test Results

Boring	Depth (ft)	Expansion Index	Expansion Potential
B – 6	6	40	Low

Table B-4
Maximum Density/Optimum Moisture Test Results

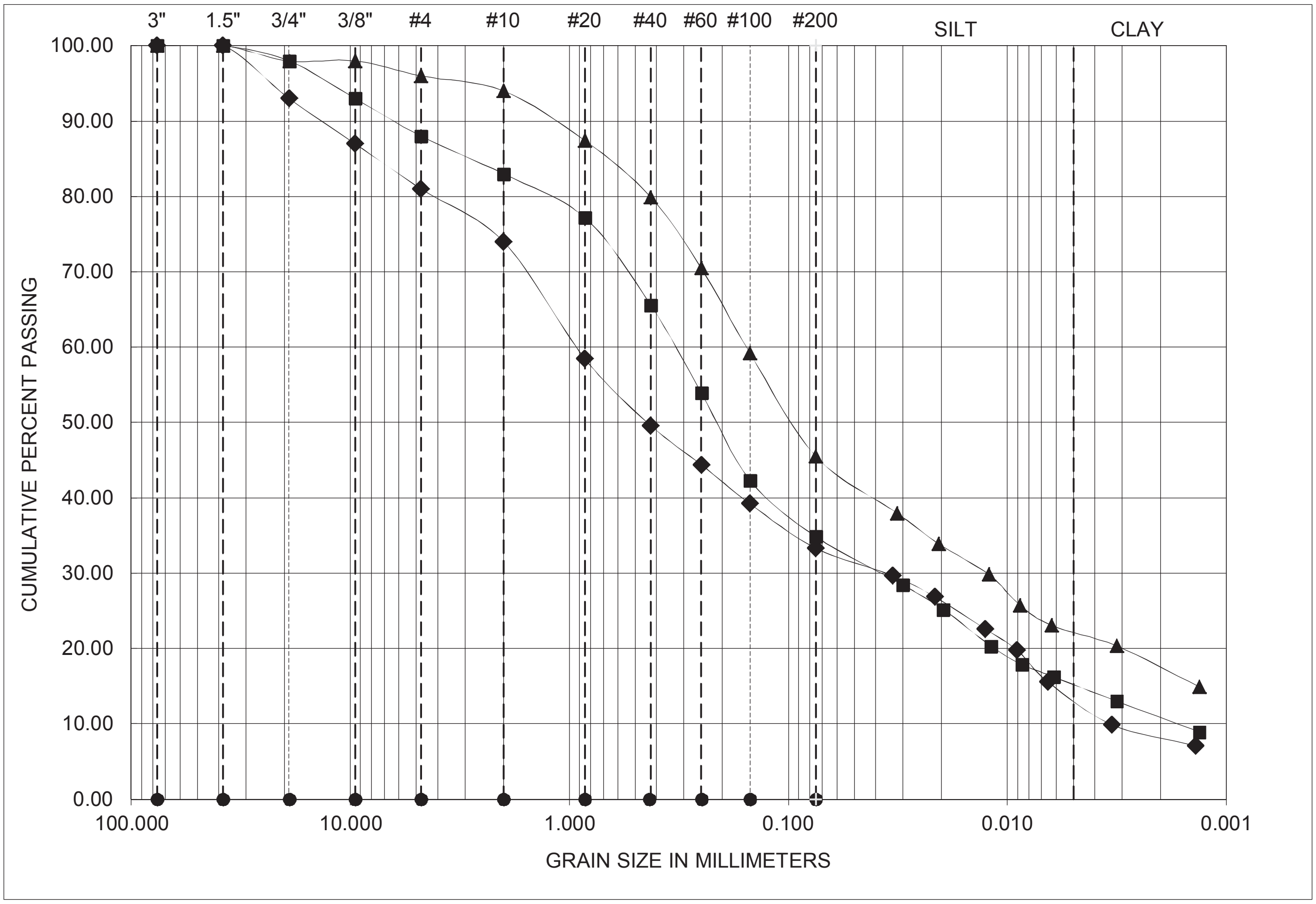
Boring	Depth (ft)	Maximum Density (pcf)	Optimum Moisture (%)
B – 1	1 – 5	113.7	14.9
B – 6	1 – 5	119.2	13.6
B – 10	1 – 3	120.0	12.0
B – 11	1 – 3.5	123.4	5.8

Table B-5
R-Value Test Results

Boring	Approximate Depth (ft)	R-Value
B – 1	1 – 5	26
B – 6	1 – 5	18
8	0.5 – 3.5	23
B – 9	1.5 – 4	5
B-12	0.5 - 2	16
B-13	2 - 3	13
B-14	1.5 - 3	15

Table B-6
Corrosion Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
B – 1	1 – 5	8.5	14	174	920
B – 7	0 – 5	8.0	32	138	460



COBBLE	GRAVEL	SAND	SILT	CLAY

SYMBOL	SAMPLE IDENTIFICATION			PERCENTAGES			ATTERBERG LIMITS			SOIL CLASSIFICATION	USCS TOTAL SAMPLE
	BORING NO.	SAMPLE NO.	DEPTH (ft.)	GRAVEL	SAND	FINES	LL	PL	PI		
♦	B-6	G1	1-5	19	48	33.3	--	--	--	Clayey Sand w/Gravel	SC
■	B-7	G1	0-5	12	53	34.9	--	--	--	Clayey Sand	SC
▲	B-8	G1	0-5	4	51	45.5	--	--	--	Clayey Sand	SC

Performed in general accordance with ASTM D 422

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City of Agoura Hills, California

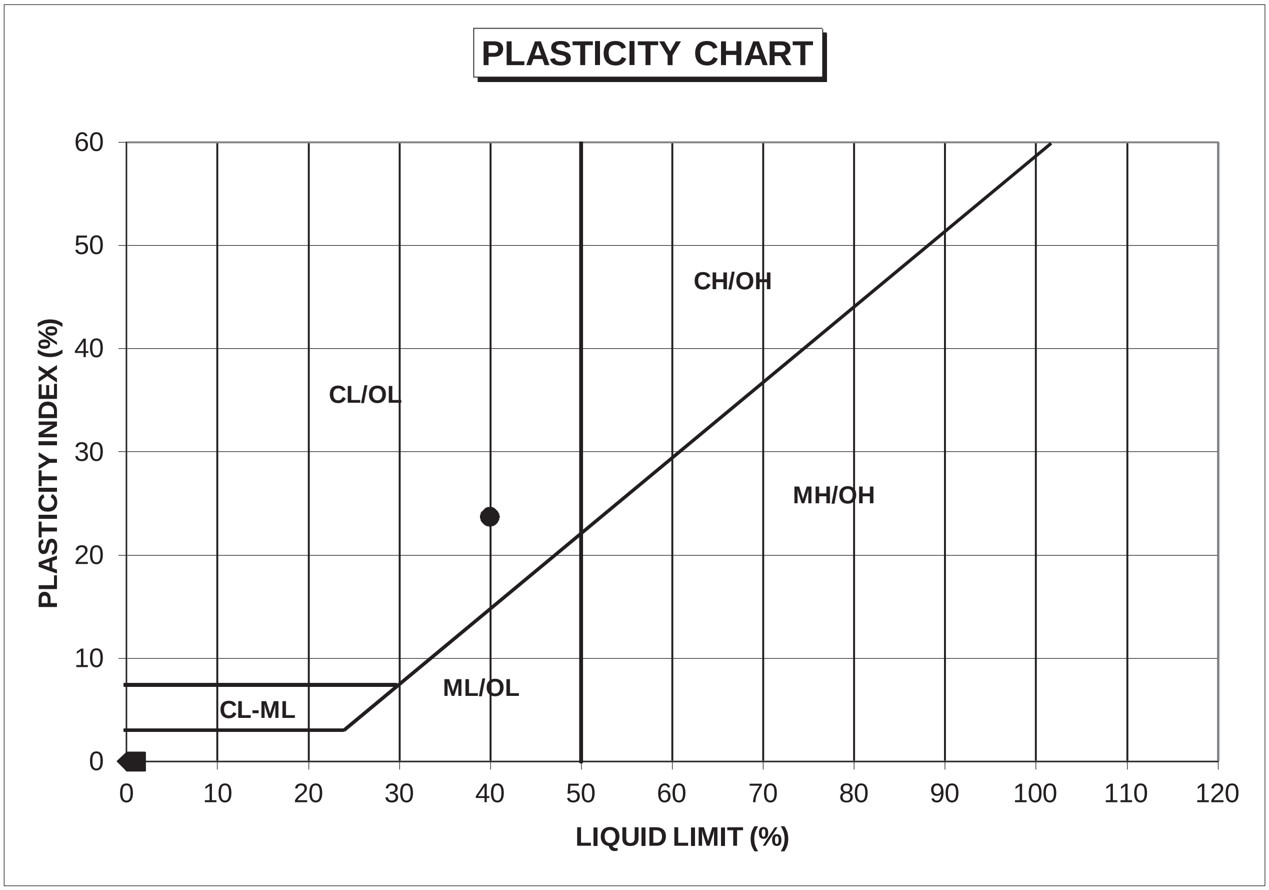
PLATE

B-2

PROJECT NO 113541

GRAIN SIZE DISTRIBUTION

SVHYD B-7 THRU B-8



SYMBOL	SAMPLE IDENTIFICATION			ATTERBERG LIMITS			SOIL CLASSIFICATION	USCS TOTAL SAMPLE
	BORING NO.	SAMPLE NO.	DEPTH (ft)	LL	PL	PI		
♦	B-3	2	11	NP	NP	NP	SILTY SAND	SM
●	B-9	2	1.5-4.0	40	16	24	CLAY	CL

Performed in general accordance with ASTM D 4318

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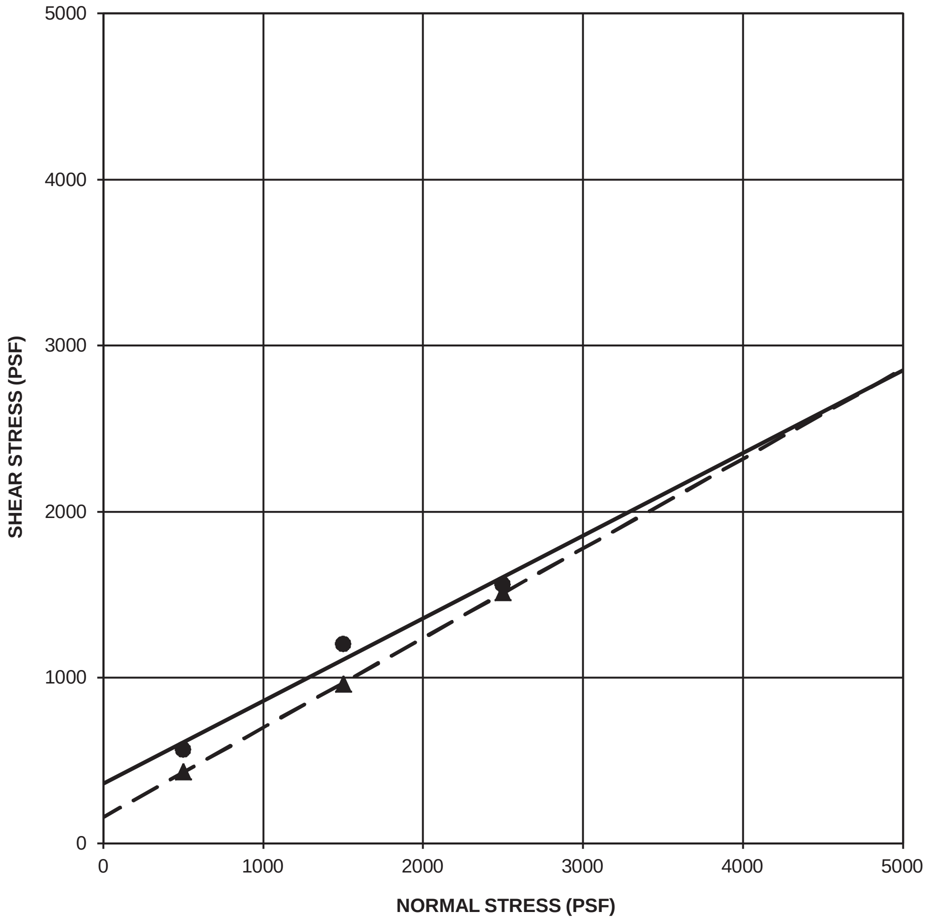
City of Agoura Hills, California

PLASTICITY INDEX TEST

PLATE B-3

PROJECT NO. 113541



SYMBOL		BORING NO.	SAMPLE NO.	DEPTH (ft)	COHESION (psf)	FRICTION ANGLE (deg)	SOIL CLASSIFICATION	USCS TOTAL SAMPLE
PEAK*	●	B-4	1	6	360	26	Sandy Clay	CL
ULTIMATE*	▲	B-4	1	6	160	28	Sandy Clay	CL

INITIAL MOISTURE(%): 16.8

INITIAL DRY DENSTIY(PCF): 105

FINAL MOISTURE(%): 26.6

Normal Stress (psf)	500	1500	2500
Peak Stress (psf)	564	1200	1560
Ultimate Stress (psf)	432	960	1512

Performed in general accordance with ASTM D 3080

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Agoura Road and Kanan Road Widening Project

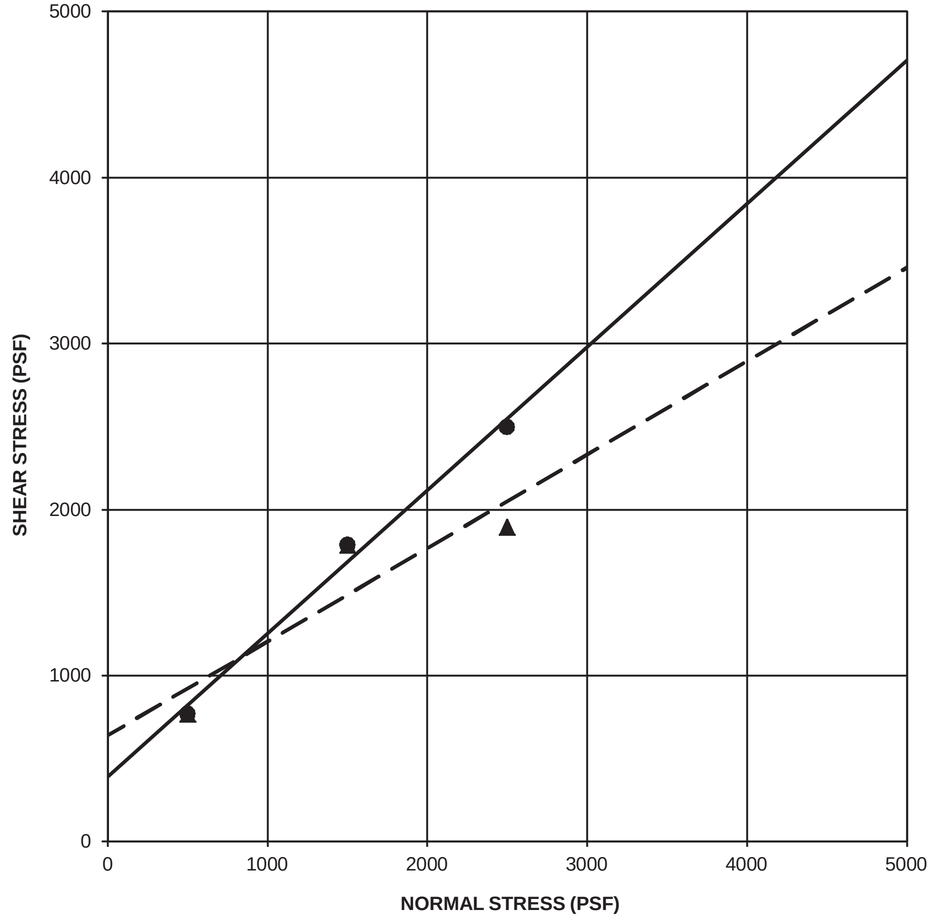
City of Agoura Hills, California

DIRECT SHEAR TEST

PLATE

B-4

PROJECT NO. 113541



SYMBOL		BORING NO.	SAMPLE NO.	DEPTH (ft)	COHESION (psf)	FRICTION ANGLE (deg)	SOIL CLASSIFICATION	USCS TOTAL SAMPLE
PEAK*	●	B-5	1	6	390	41	Clayey Sand with Gravel	SC
ULTIMATE*	▲	B-5	1	6	640	29	Clayey Sand with Gravel	SC

INITIAL MOISTURE(%): 17.4

INITIAL DRY DENSTIY(PCF): 105

FINAL MOISTURE(%): 37.9

Normal Stress (psf)	500	1500	2500
Peak Stress (psf)	768	1788	2496
Ultimate Stress (psf)	768	1788	1896

Performed in general accordance with ASTM D 3080

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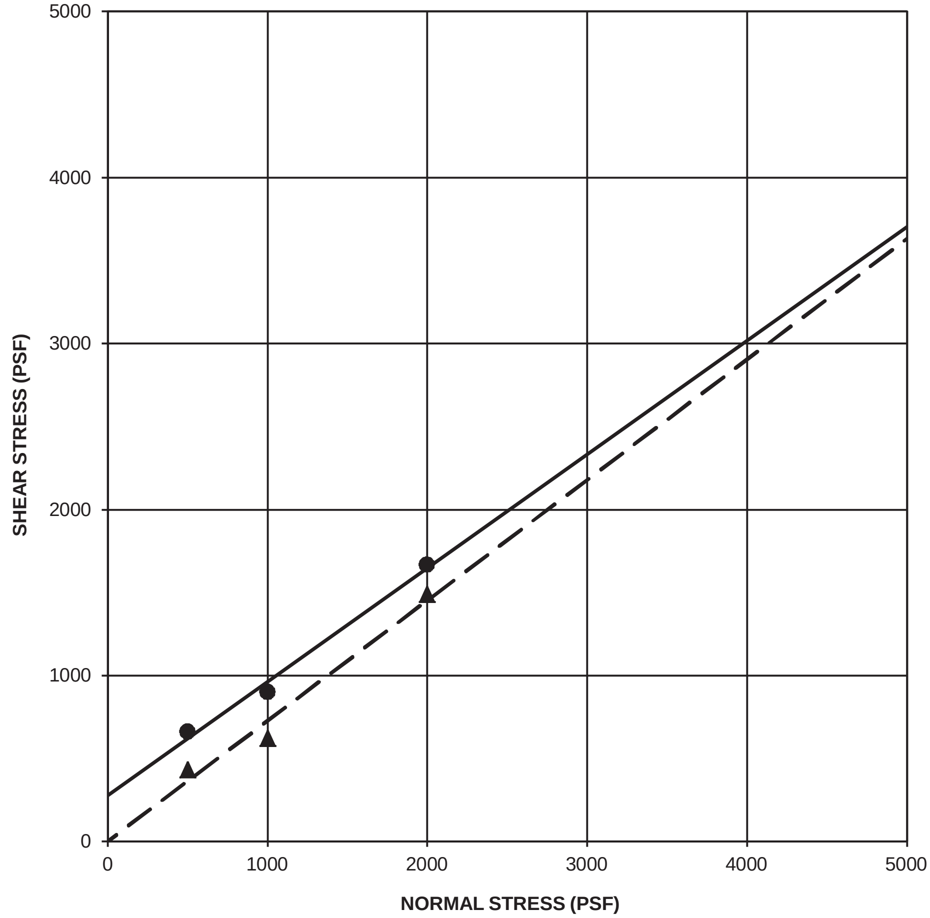
PLATE

B-5

PROJECT NO. **113541**

DIRECT SHEAR TEST



SYMBOL		BORING NO.	SAMPLE NO.	DEPTH (ft)	COHESION (psf)	FRICTION ANGLE (deg)	SOIL CLASSIFICATION	USCS TOTAL SAMPLE
PEAK*	●	B-10	1	1-3	280	34	Clayey Sand	SM
ULTIMATE*	▲	B-10	1	1-3	0	36	Clayey Sand	SM

INITIAL MOISTURE(%):	12.0
INITIAL DRY DENSTIY(PCF):	120
FINAL MOISTURE(%):	17.8

Normal Stress (psf)	500	1000	2000
Peak Stress (psf)	660	900	1668
Ultimate Stress (psf)	432	624	1488

Sample tested was remolded to 90 percent of the ASTM D 1557 Maximum Dry Density - Optimum Moisture Content Result
Performed in general accordance with ASTM D 3080

KLEINFELDER
Bright People. Right Solutions.

Agoura Road and Kanan Road Widening Project

City of Agoura Hills, California

DIRECT SHEAR TEST

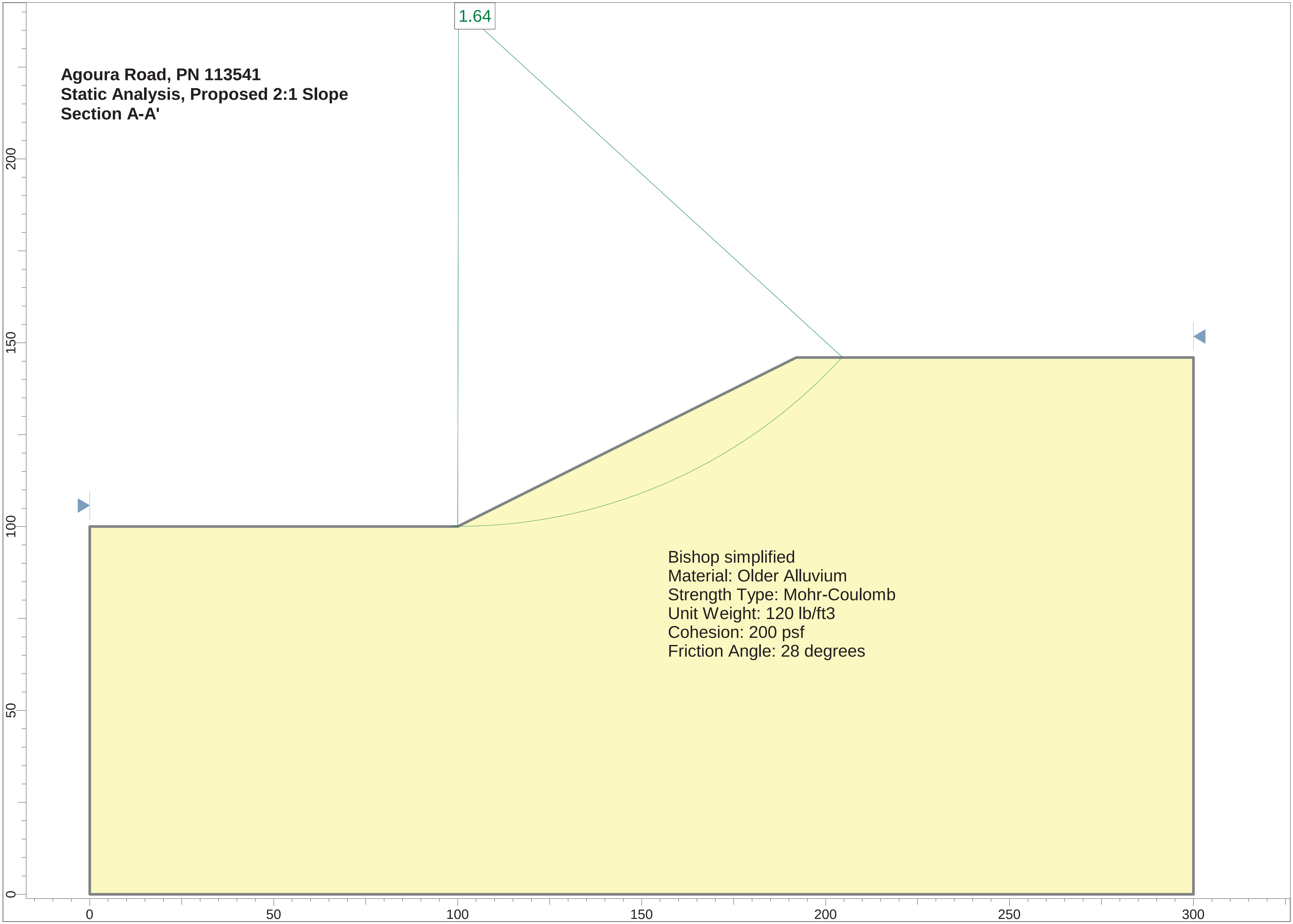
PLATE

B - 6

PROJECT NO. 113541

APPENDIX C

Slope Stability Analysis


1.64
Agoura Road, PN 113541
Static Analysis, Proposed 2:1 Slope
Section A-A'
Bishop simplified
Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
0
50
100
150
200
250
300
0
50
100
150
200

Slide Analysis Information

Document Name

File Name: Section A-A Agoura Road

Project Settings

Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius increment: 10
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Material Properties

Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
Water Surface: None

Global Minimums

Method: bishop simplified
FS: 1.644170
Center: 100.244, 241.409
Radius: 141.400
Left Slip Surface Endpoint: 100.019, 100.009

Right Slip Surface Endpoint: 204.604, 146.000
Resisting Moment=1.35847e+007 lb-ft
Driving Moment=8.26233e+006 lb-ft

Valid / Invalid Surfaces

Method: bishop simplified
Number of Valid Surfaces: 112145
Number of Invalid Surfaces: 66
Error Codes:
Error Code -108 reported for 63 surfaces
Error Code -109 reported for 3 surfaces

Error Codes

The following errors were encountered during the computation:

-108 = Total driving moment
or total driving force < 0.1. This is to
limit the calculation of extremely high safety
factors if the driving force is very small
(0.1 is an arbitrary number).

-109 = Soiltype for slice base not
located. This error should occur very rarely,
if at all. It may occur if a very low number of
slices is combined with certain soil geometries,
such that the midpoint of a slice base is
actually outside the soil region,even though
the slip surface is wholly within the soil region.

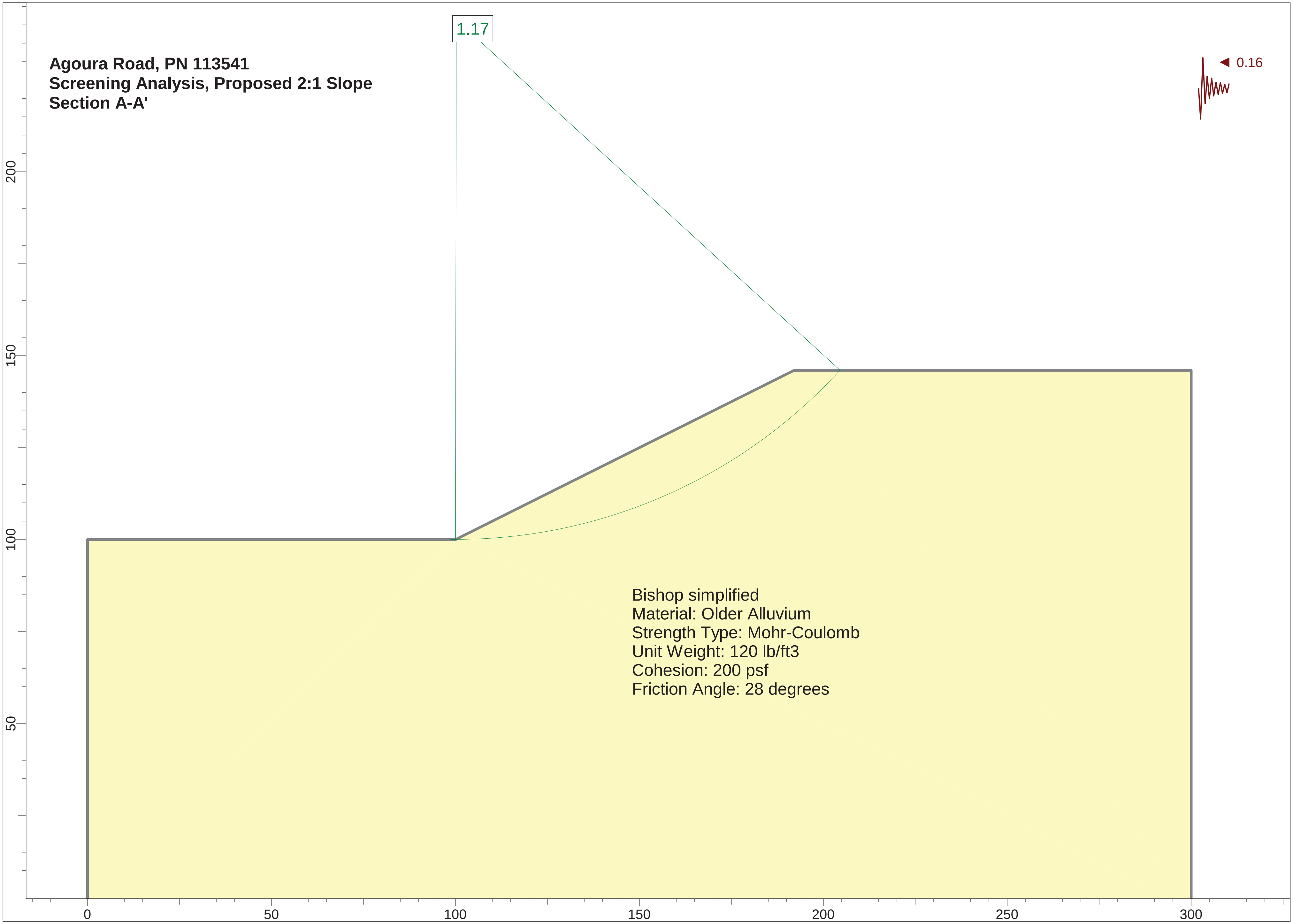
List of All Coordinates

Search Grid

72.994	180.769
150.851	180.769
150.851	307.103
72.994	307.103

External Boundary

300.000	0.000
300.000	146.000
192.000	146.000
100.000	100.000
0.000	100.000
0.000	0.000



Agoura Road, PN 113541
Screening Analysis, Proposed 2:1 Slope
Section A-A'
1.17
0.16
Bishop simplified
Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
0
50
100
150
200
250
300
50
100
150
200

Slide Analysis Information

Document Name

File Name: Section A-A Agoura Road SA

Project Settings

Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius increment: 10
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.16

Material Properties

Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
Water Surface: None

Global Minimums

Method: bishop simplified

FS: 1.168050
Center: 100.244, 241.409
Radius: 141.400
Left Slip Surface Endpoint: 100.019, 100.009
Right Slip Surface Endpoint: 204.604, 146.000
Resisting Moment=1.29176e+007 lb-ft
Driving Moment=1.10591e+007 lb-ft

Valid / Invalid Surfaces

Method: bishop simplified
Number of Valid Surfaces: 112208
Number of Invalid Surfaces: 3
Error Codes:
Error Code -109 reported for 3 surfaces

Error Codes

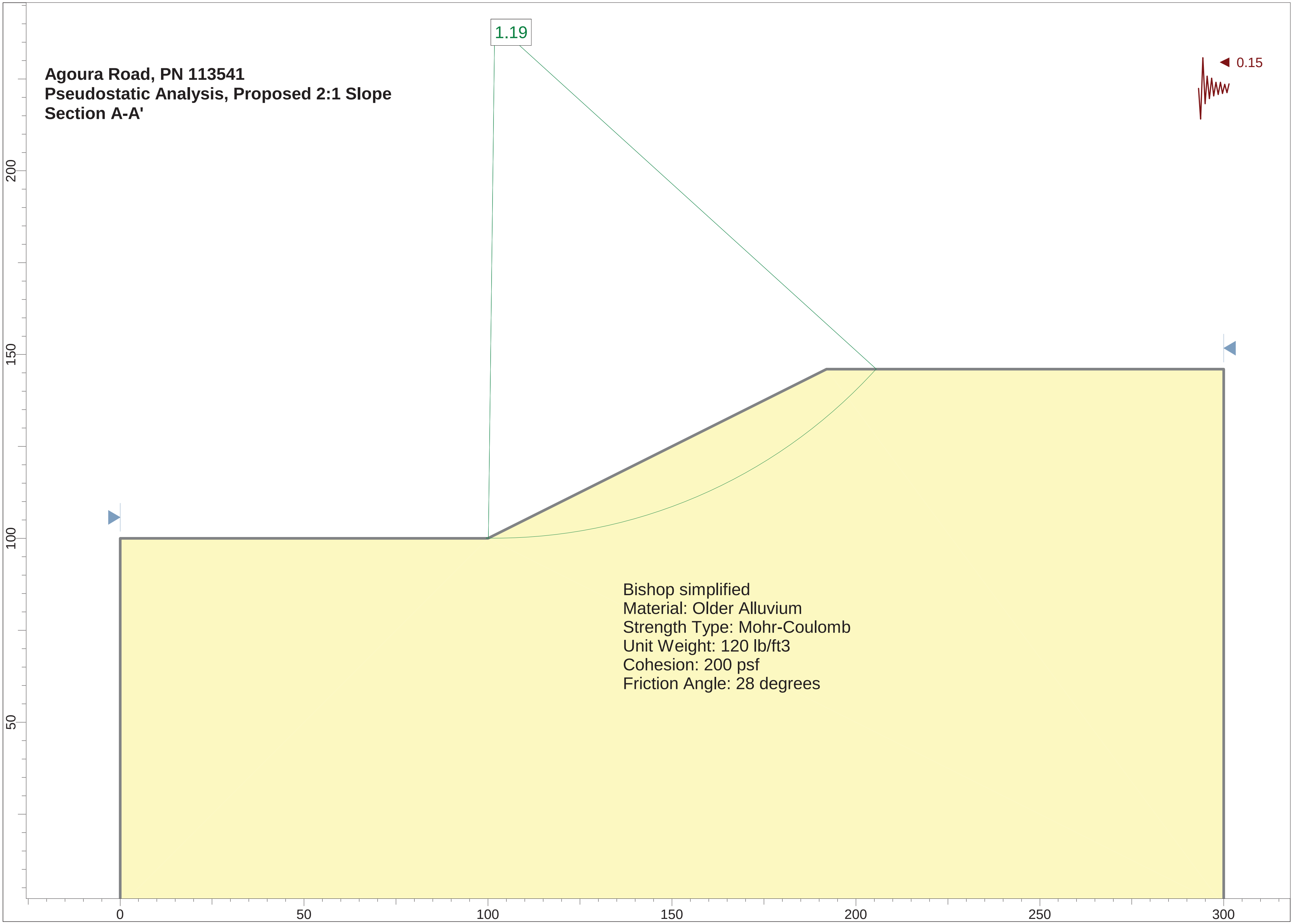
The following errors were encountered during the computation:

-109 = Soiltype for slice base not
located. This error should occur very rarely,
if at all. It may occur if a very low number of
slices is combined with certain soil geometries,
such that the midpoint of a slice base is
actually outside the soil region,even though
the slip surface is wholly within the soil region.

List of All Coordinates

External Boundary

300.000	0.000
300.000	146.000
192.000	146.000
100.000	100.000
0.000	100.000
0.000	0.000



Agoura Road, PN 113541
Pseudostatic Analysis, Proposed 2:1 Slope
Section A-A'
1.19
0.15
Bishop simplified
Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
0
50
100
150
200
250
300
50
100
150
200

Slide Analysis Information

Document Name

File Name: Section A-A Agoura Road PS

Project Settings

Project Title: SLIDE - An Interactive Slope Stability Program
Failure Direction: Right to Left
Units of Measurement: Imperial Units
Pore Fluid Unit Weight: 62.4 lb/ft3
Groundwater Method: Water Surfaces
Data Output: Standard
Calculate Excess Pore Pressure: Off
Allow Ru with Water Surfaces or Grids: Off
Random Numbers: Pseudo-random Seed
Random Number Seed: 10116
Random Number Generation Method: Park and Miller v.3

Analysis Methods

Analysis Methods used:
Bishop simplified

Number of slices: 25
Tolerance: 0.005
Maximum number of iterations: 50

Surface Options

Surface Type: Circular
Search Method: Grid Search
Radius increment: 10
Composite Surfaces: Disabled
Reverse Curvature: Create Tension Crack
Minimum Elevation: Not Defined
Minimum Depth: Not Defined

Loading

Seismic Load Coefficient (Horizontal): 0.15

Material Properties

Material: Older Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 200 psf
Friction Angle: 28 degrees
Water Surface: None

Global Minimums

Method: bishop simplified

FS: 1.190670
Center: 101.801, 240.146
Radius: 140.120
Left Slip Surface Endpoint: 100.073, 100.036
Right Slip Surface Endpoint: 205.581, 146.000
Resisting Moment=1.33153e+007 lb-ft
Driving Moment=1.1183e+007 lb-ft

Valid / Invalid Surfaces

Method: bishop simplified
Number of Valid Surfaces: 112208
Number of Invalid Surfaces: 3
Error Codes:
Error Code -109 reported for 3 surfaces

Error Codes

The following errors were encountered during the computation:

-109 = Soiltype for slice base not located. This error should occur very rarely, if at all. It may occur if a very low number of slices is combined with certain soil geometries, such that the midpoint of a slice base is actually outside the soil region,even though the slip surface is wholly within the soil region.

List of All Coordinates

External Boundary

300.000	0.000
300.000	146.000
192.000	146.000
100.000	100.000
0.000	100.000
0.000	0.000

APPENDIX D

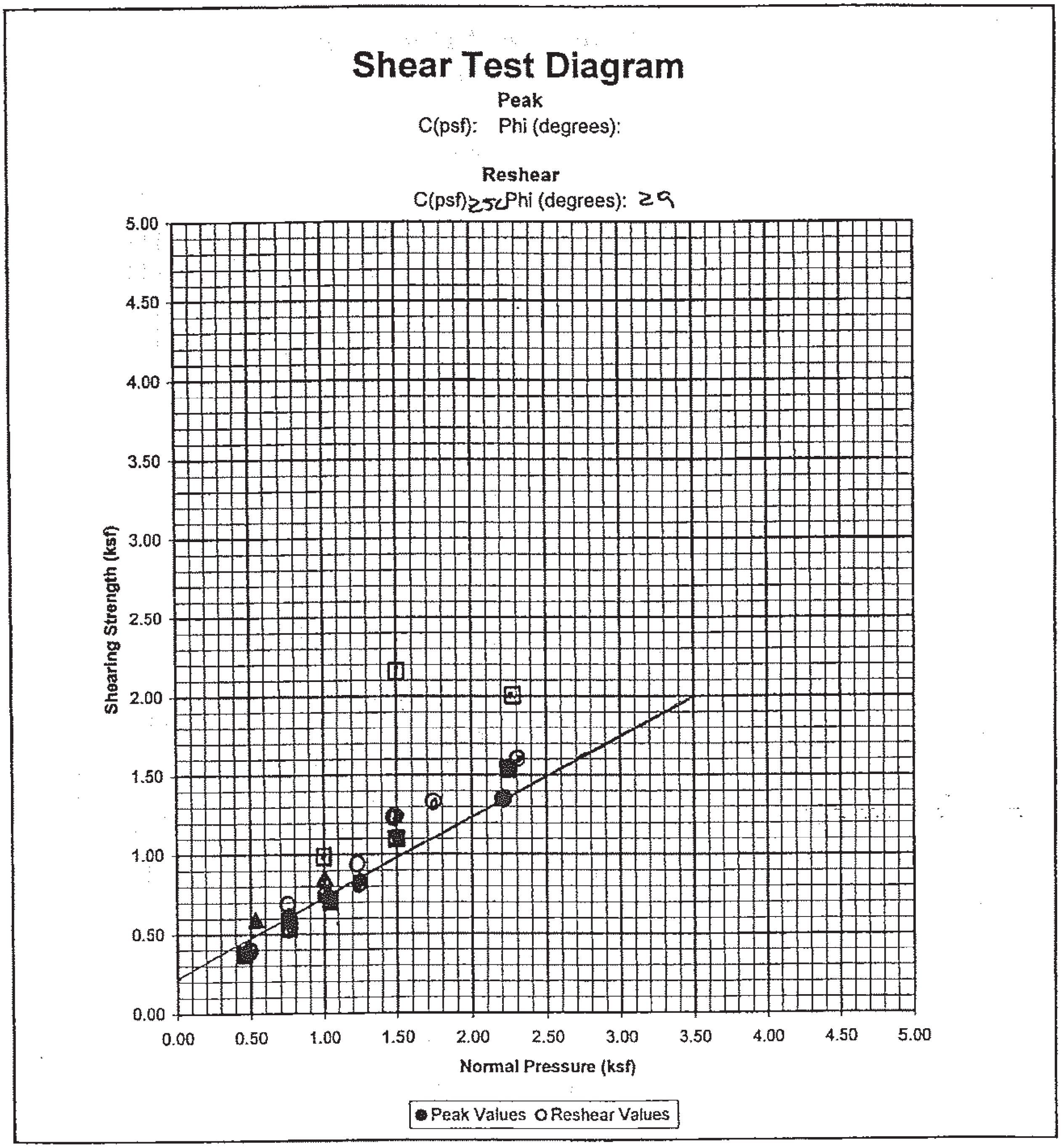
Pertinent Data from Previous Reports

Hilton

GeoSoils Consultants, Inc.

PLATE SH-2A

Geotechnical Engineering * Engineering Geology



● Cut slope #1

■ cut slope #2

▲ B-8 @ 10'

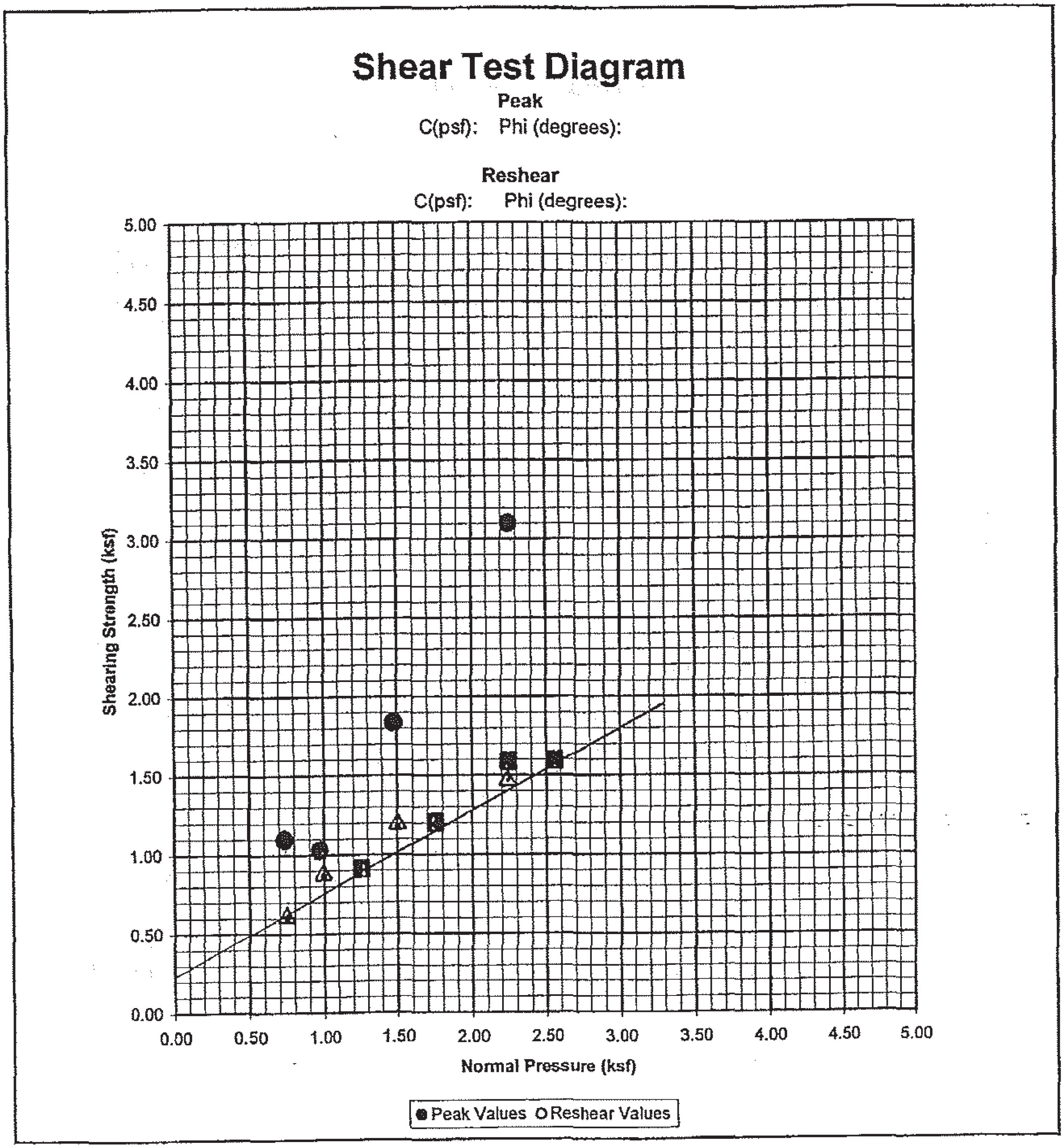
⊙ B-8 @ 30'

Hilton

GeoSoils Consultants, Inc.

Geotechnical Engineering * Engineering Geology

PLATE SH- Fill



● B-1 @ 10'
△ B-2 @ 5'
■ B-7 @ 5'

Existing Fill.

TWINING

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twininginc.com

April, 4, 2016
Project No. 150702.4

Mr. Justin Gatza
Kimley-Horm
660 South Figueroa Street, Suite 2050
Los Angeles, CA 90017

Subject: Geotechnical Recommendations
Proposed Kanan Road/ Agoura Road Ultimate Intersection Improvements
City of Agoura Hills, California

References:

Fugro West, Inc., 2008. Geotechnical Study, Agoura Hills Roundabout, Agoura Hills, California, Project No. 3044.071, dated July 25.

Kleinfelder, 2012, Geotechnical Investigation Report, Agoura Road and Kanan Road Widening Project, City of Agoura Hills, California, dated May 25.

Leighton, 2016, Agoura Road Widening Project; Review of Tensar Proposed Pavement Section Redesign, dated February 18.

Kimley-Horn, 2015, Design Drawings for Kanan Road and Agoura Road Intersection Improvements, dated July 2.

Dear Mr. Gatza:

Twining, Inc. (Twining) is pleased to present our geotechnical recommendations for the proposed Kanan Road and Agoura Road Ultimate Intersection Improvements project in the City of Agoura Hills, California. The proposed improvements include widening the existing roads, constructing new center medians, striping, HMA pavement and flexible vehicular brick paving, short retaining walls, and relocating existing oak trees.

To prepare this report, we have performed a site reconnaissance to observe surface conditions in October, 2015. We reviewed above-referenced geotechnical reports and civil drawings. The current site conditions are essentially as described in the 2012 Kleinfelder report.

Based on our review of previous geotechnical reports, subgrade soils under the existing pavement consist of clayey sand and sandy clay. The clayey soils have a low R- value ranging from 5 to 16. It is our understanding that geotextile enhancement to reduce the overall pavement structural section is proposed.

Based on the results of our literature review and the field observations, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

RECOMMENDATIONS TO SUBGRADE GEOTEXTILE ENHANCEMENT

It is our understanding that the following pavement structural section with geotextile enhancement (listed in vertical descending order) is proposed:

- 1-5/8" Asphalt Rubber Hot Mix (ARHM)
- 4.5" Hot Mixed Asphalt (HMA)
- 6" Class 2 Aggregate Base (AB)
- Sheet-layer of Tensar TX5 geogrid
- 6" Class 2 Aggregate Base (AB)
- Sheet-layer of Tensar TX5 geogrid
- Scarification of 6" of native subgrade and proof-rolling and compacted to 95%

We have reviewed this pavement structural section in accordance with Caltrans Highway Design Manual, and concluded this design is adequate for a Traffic Index (TI) of 9.5 from a geotechnical standpoint.

We have also performed the analysis for the proposed flexible vehicular brick paving in accordance with Design Guide for Vehicular Brick Pavements (Brick Industry Association, 2003). The following pavement structural section with geotextile enhancement (listed in vertical descending order) is recommended:

- 2-5/8" Brick Paver (Minimum thickness)
- 3/4" Bituminous Setting Bed
- 2" HMA (Minimum thickness)
- 6" Class 2 Aggregate Base (AB)
- Sheet-layer of Tensar TX5 geogrid
- 6" Class 2 Aggregate Base (AB)
- Sheet-layer of Tensar TX5 geogrid
- Scarification of 6" of native subgrade and proof-rolling and compacted to 95%

Prior to placement of geogrid, the exposed native subgrade should be proof-rolled and inspected by Twining. Additional removals may be recommended if loose or soft soils are exposed. The exposed ground surface should then be scarified to a depth of approximately 6 inches and watered or dried, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 95 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

RECOMMENDATIONS FOR RETAINING WALLS

It is our understanding that short retaining walls are proposed along the north side of Agoura Road. For a cantilevered wall that is free to rotate at the top, the following active pressures can be used for structural design.

Backslope ratio (Horizontal : Vertical)	Active Pressure in terms of EFP (pcf)
Level	30
3 : 1	37
2 : 1	46

The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the walls and that external hydrostatic pressure will not develop behind the wall. The values presented above do not include surcharge loads. The additional horizontal pressure acting on the wall can be estimated as approximately 30% of the magnitude of the vertical surcharge pressure for the “active” conditions.

Adequate backdrain system (i.e. drain pipe or weepholes) is essential in order to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. The walls should be appropriately waterproofed in accordance with the recommendations of the project design engineer. The backdrain consisting of a 4-inch-diameter perforated pipe encased in 1 square foot per foot of ¾-inch open-graded crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140N or equivalent) should be placed continuously along the bottom of the retaining side of the wall. The pipe should be sloped at least 1 percent and discharge through a solid pipe to an appropriate outlet. A weephole should consist of a 3-inch-diameter solid PVC pipe encased in a minimum 1-cubic-foot, 3/4-inch drain rock, with center-to-center spacing of 15 feet.

Any imported backfill material should consist of granular, non-expansive material with an expansion index no greater than 30, and should be approved by the project geotechnical engineer prior to importing to the site.

The wall footing should be at least 18 inches in width and 12 inches in depth below the lowest adjacent grade. A soil bearing capacity of 2,000 psf can be used for design. Bearing capacity can increase 300 psf for each additional foot of width and 450 psf for additional foot of depth to a maximum allowable capacity of 3,500 psf. The allowable bearing values may be increased by one-third when considering wind or earthquake loading. Allowable coefficient of friction can be assumed to be 0.3, and lateral passive resistance in terms of equivalent fluid pressure (EFP) of 300 pcf can be used for design.

The excavated footing subgrade should be inspected by Twining. Additional removals and/or compaction may be required if loose or soft soils are exposed.

RECOMMENDATIONS FOR TRANSPLANT OF OAK TREES

Several oak trees will be transplanted to the new location near the intersection of Kanan Road and Cornell Road. We recommend an arborist supervise the oak trees transplant operations. The holes created by the removal of trees should be backfilled with compacted fill. The holes should be inspected by a geotechnical engineer to ensure all the loose materials are removed prior to placement of backfill. As an alternative, Controlled-Low-Strength-Material (CLSM) may be used to backfill holes deeper than 3 feet. In such a case, we recommend the hole be backfilled with CLSM consisting of 2-sack cement slurry up to 2 feet below the proposed subgrade, and then capped with compacted fill to the subgrade elevation.

Based on our field observations, a descending slope is located at the west side of the proposed transplant location on the west shoulder of Kanan Road. We recommend a minimum 3 feet setback distance from the drip line of trees to the top of the slope be provided. An arborist should be retained to evaluate the site topographical and horticultural soil conditions as well as supervise the transplant operations.

LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of available background documents, and on information obtained from our recent field observations. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Twining performed its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either express or implied, is made as to the conclusions and recommendations contained in this report.

We trust that this information meets your needs at this time. Please do not hesitate to contact the undersigned with any questions at 562-426-3355.

Respectfully submitted,
TWINING, INC.

Sean C. Lin, GE 2921
Chief Geotechnical Engineer