Appendix E

Geological and Geotechnical Resources

Appendix E.1

Geological and Geotechnical Investigation Report: Proposed Buchanan Road Bypass (2008)

Prepared for **RBF Consulting**

GEOLOGICAL AND GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

January 9, 2008
File No.: 75856/PWGEO

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Mr. William J. Conyers, Senior Associate
RBF Consulting
1981 North Broadway, Suite 235
Walnut Creek, California 94596-3817

SUBJECT: Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass in Pittsburg, California

Dear Mr. Conyers:

We are pleased to submit four copies of our report containing the results of our geologic and geotechnical investigation report for the proposed Buchanan Road Bypass in Pittsburg, California.

The enclosed report provides a description of the investigation performed along the route selected by you and the City of Pittsburg, our conclusions pertaining to site preparation and compaction requirements, and recommendations for construction considerations. The purpose of our investigation was to perform a geological and geotechnical investigation in accordance with our revised scope of work and cost estimate dated June 8, 2006. Our scope of work consisted of performing exploratory soil borings, exploratory rock cores, geologic trenches/test pits, seismic refraction traverses, additional geologic mapping, as needed, laboratory testing, engineering analysis, and preparation of this report. The purpose of our investigation was to conduct a design level geotechnical investigation of the proposed Buchanan Road Bypass alignment.

Some of the geological and geotechnical constraints evaluated during this investigation included faulting and seismicity, landslide deposits, adverse bedrock bedding, compressible and expansive soils, erosion, bedrock rippability, cut and fill slope stability, foundation conditions, and settlement of deep fills. We have presented in this report a description of such constraints, and their impacts to the planned roadway design.

We appreciate the opportunity of providing services to you on this project and trust that this report meets your needs at this time. If you have any questions concerning the information presented, please contact Mike Majchrzak at (925) 484-1700 or Fernando Silva at (925) 427-6477.

Sincerely,

KLEINFELDER WEST, INC.

Lianna Serrano
Staff Engineer

Michael Majchrzak, P.E., G.E. #555
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
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LS/MFM/jmk

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KLEINFELDER 7133 Koll Center Parkway, Suite 100, Pleasanton, CA 94566-3101 (925) 484-1700 (925) 484-5838 fax

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.

ASFE
The Best People on Earth

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GEOLOGICAL AND GEOTECHNICAL INVESTIGATION REPORT
BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

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PLATES

EXECUTIVE SUMMARY

The City of Pittsburg is currently planning the proposed Buchanan Road Bypass that will be an east-west limited access arterial roadway in the undeveloped hills located south of the City. The new roadway will extend from near the western limits of the City of Antioch at Somersville Road to a new intersection with Kirker Pass Road. The proposed project will consist of constructing an approximately 4.0 km (2.5 mile) section of new roadway along one of three optional alignments, and the project site spreads over an area measuring approximately 4.5 square km (1.75 square miles).

This report presents the results of Kleinfelder's engineering geology and geotechnical constraints investigation as part of the overall planning process by RBF Consulting, the project civil engineers. The intent of our investigation was to perform an investigation of the subsurface conditions along the alignment selected by the City of Pittsburg, and to identify significant geologic, seismic, and geotechnical constraints that could impact the design of the roadway. Our investigation consisted of geologic reconnaissance, drilling of borings, rock coring, test pits, seismic refraction analysis, laboratory testing, slope stability evaluations, engineering analysis, review of pertinent information presented in reports by us and other geotechnical engineers for nearby projects, and the preparation of this report.

Based on our field investigation and analysis, there are a number of geologic considerations that can impact the design of the roadway. These include, amount others, existing landslides, adverse bedding of rock and formational material, expansive soils, and slope stability. We have discussed these items with representatives of RBF Consulting, and they have included in the design aspects to address these considerations. Some of these are discussed as follows:

- It is our understanding that it is desired that most of the cut and fill slopes have inclinations that are at approximately 2:1 (horizontal to vertical). We have evaluated the major slopes at these inclinations. In most cases, the 2:1 inclination is acceptable. In cut areas at this inclination, most of the slopes will need a buttress fill to address adverse bedding of the exposed rock and formational material. For a few slopes, inclinations gentler than 2:1 will be needed because the planned cuts are extremely high, and the soils generally weaker. A summary of the slopes evaluated, along with the steepest inclination recommended, and whether a buttress fill is needed, is presented on Table 7.1.4-1 with the locations of the identified slopes shown on Plates 2A and 2B.

- There is an existing landslide designated as Landslide 4 on Plate 2B. This landslide is an ancient one with some minor movement of the surfacial material. It is extremely deep. Our evaluation indicates that the landslide can be left in place provided that it is re-shaped, and that a debris encatchment be provided near the base of the landslide. Most of this encatchment is already included as a fill embankment to support the road. An additional embankment may be needed at the western portion of this area. The portion of the landslide beneath the planned filled for the roadway will need to be removed. This excavation will require special methods as discussed in Section 7.4, "Slope Stability Results" of this report. Even with these special methods, there is a distinct possibility that movement of the landslide may occur. As a result, additional excavation of the landslide material may be needed.

- Fill slope can be placed at an inclination no steeper than 2:1 (horizontal to vertical). Where fills are greater than 5 feet in height, keys at the toe of the fills will be required. Where existing colluvium or landslide debris exists, it will need to be removed. Specially, the landslide debris will need to be removed at all fill locations. We estimate that there will be about 10 to 20 feet of colluvum material that will need to be in non-landslide areas where fill is to be placed.

- The on-site soils can be reused as fill material, including landslide material that is excavated. A majority of the soils is clay with medium to high expansive potential. As such, additional effort might be needed to reach the desired moisture content for placement, as well as achieving the appropriate compaction. There is a variety of material at the site, which their locations and designation are approximately on Plates 2A and 2B. We have discussed the placement characteristics of each of these materials in this report.

- The seismic refraction lines performed in the field indicated shear wave velocities between 900 and 5400 ft/s. These values are within that typically for ripping based on using a D-9 dozer or larger. Blasting is not anticipated. Localized hard ripping might be needed.

- As with any hillside development, especially with significant cut and fill slopes as currently envisioned for this project, there will be long term settlement, sloughing of soils on slope, erosion, and downward creep of soils on slopes, all of which will require future maintenance. Aspects can be included in the design to reduce the impact of such items, but will not eliminate them. As such, maintenance of the road will need to be anticipated.

- Future consultations will be needed by us to "fine tune" the design. Therefore, the conclusions and recommendations presented in this report should be considered general. In addition, evaluation of remedial grading (keyways, subdrains, etc.), along with assistance in establishing hidden qualities (overexcavation for example), will need to be provided by us.

- Due to the variety of materials at the site, as well as variation in bedding inclinations, all cuts should be evaluated by us during grading for both short and long term stability. In addition, amble observation and testing during grading by us should be also be provided.

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

1 INTRODUCTION

This report presents the results of Kleinfelder's engineering geology and geotechnical investigation of the site for the proposed Buchanan Road Bypass (Bypass) project located in Pittsburg, California. The general location of the Bypass is shown on Plate 1, Site Vicinity Map. This investigation was performed to provide design-level conclusions and recommendations for the proposed Bypass alignment as part of the overall planning process by RBF Consulting.

The purpose of our investigation has been to assess the potential impact of landslides, stability of manufactured (cut and fill) sloes, unstable or compressible colluvial and alluvial soil areas, rippability of bedrock, and proposed grading at the site. The intent of this report is to provide design level recommendations for the Bypass alignment selected with respect to geological and geotechnical issues. The results of our field and laboratory investigations and engineering analysis, as well as our conclusions and recommendations, are presented in this report.

Our services were provided in accordance with our revised scope of work submittal dated June 8, 2006. Future consultations will be needed by us to "fine tune" the design. Therefore, the conclusions and recommendations presented in this report should be considered general. In addition, evaluation of remedial grading (keyways, subdrains, etc.), along with assistance in establishing hidden qualities (overexcavation for example), will need to be provided by us.

1.1 PROJECT DESCRIPTION

The proposed Buchanan Road Bypass project will consist of constructing an approximately 2.4 km (1.5 miles) section of new roadway. The new roadway will extend from Kirker Pass Road to Ventura Drive. The Bypass will be an east/west, limited-access arterial roadway in the undeveloped hills south of the City. The preliminary plans by RBF Consultants indicate that east of Ventura Drive the roadway will be constructed by others as part of the Sky Ranch and Black Diamond Ranch residential developments, and will extend to James Donlon Boulevard west of Somersville Road. We understand that while the bypass roadway will ultimately be widened to four lanes, the current project will be only a two-lane road; however, the right-of-way and grading for all four lanes will be included.

Besides the roadway and associated drainage facilities, other project features associated with the proposed bypass road include the following:

- Five culverts along smaller stream crossings and two culverts along Kirker Canyon Creek.
- Cut slopes and embankment fills with heights ranging up to about 60 meters (190 feet).
- Approximately 4,300 to 5,500 linear meters (14,000 to 18,000 linear feet) of soundwall.

The bypass road alignment is anticipated to encounter geologic materials consisting of undocumented fill, alluvium, colluvium, active and dormant landslides, Tulare Formation claystone, Lawlor Tuff, Neroly Formation sandstone, Cierbo Formation sandstone, and Kirker Formation tuffaceous materials. The lateral limits and contacts of these geologic units are shown on Plates 2A and 2B.

1.2 BACKGROUND

Services provided to date for this project include our Phase I geological and geotechnical evaluation for three potential roadway alignments connecting Kirker Pass and Somersville Roads. The results of that evaluation were presented in our report dated September 20, 2002 and titled "Geological and Geotechnical Constraints Evaluation Report for the Proposed Buchanan Road Bypass in Pittsburg, California."

Our Phase I investigation studied three possible bypass-road alignments and concluded that the Central Alignment will require the least mitigation to address geologic, seismic and geotechnical constraints/considerations. This conclusion was based on observations as follows:

- The Central Alignment crosses the smallest number of landslides, whether active or dormant.
- It does not cross the large dormant landslide deposits (Landslides "3" and "4" depicted on Plate 3B, of the Phase I report) mapped along the Northern Alignment, nor does it cross the active landslide complex mapped in the southeast corner of the project area along the Southern Alignment.
- This route should require the least grading volume because it will not require mitigation of the two large dormant landslides along the Northern Alignment and the active landslide complex along the Southern Alignment.
- The Central Alignment does not extend along areas underlain by the tuff/tuffaceous materials more than the other alignments. Accordingly, it will require potential less mitigation of such materials than the other two alignments.

Based in part on a review of the findings of the Phase I report, the City of Pittsburg and RBF Consulting have concluded that the Central Alignment is desired alignment, and requested that Kleinfelder provide this design-level geotechnical investigation of that alignment.

1.3 PURPOSE AND SCOPE OF SERVICES

Our field investigation included advancing auger and rock-core borings, excavation of exploratory trenches and test pits, performance of seismic refraction traverses, and geologic mapping. We performed a laboratory-testing program to evaluate the physical characteristics and engineering properties of the encountered soil and rock materials, slope stability analyses based on the grading plans provided by RBF titled "Final Central Alignment" dated July 9, 2007 and prepared this written report addressing the geologic feasibility, rock rippability, landslide and colluvial remediation, and geotechnical issues related to the currently planned grading. Our completed report includes geologic maps and structural cross-sections depicting our geologic modeling of the road alignment. Boring logs and laboratory test results for soil and rock samples are included in this report. Recommendations pertaining to the design of the soundwall, drainage and earthwork are provided.

1.4 AUTHORIZATION

This preliminary geological and geotechnical investigation was performed in accordance with our contract with RBF dated September 18, 2006.

1.5 SITE DESCRIPTION AND CONDITIONS

The site area is situated along the northern foothills of Mount Diablo generally along the elevated ridgelines bordering relatively flat alluvial plains underlying the City. The San Joaquin River near its junction with Sacramento River is located to the north beyond the alluvial plains. The north/south trending ridgelines are separated by several prominent drainage courses that drain northward towards the relatively flat plains.

The alignment has a topographic relief measuring approximately 118 meters (387 feet) with ground surface elevations varying from approximately 54 meters (177 feet) near the northeastern portion of the alignment to approximately 172 meters (564 feet) along the southernmost section of the alignment. The topographic conditions (as provided by RBF) are presented on Plates 2A and 2B.

The site surface area is generally covered by wild grasses and scattered oak trees along the ridge lines. Numerous dirt roads cross the site. High voltage transmission lines that are supported by more than 20 power towers cross the site area in a generally east/west direction. The site area is generally free of buildings except near the mouth of the prominent drainage course situated to the south of the termination point of Suzanne Drive where the Thomas residence and other ranch related structures are located north of the alignment. Several fence lines and associated gates are present throughout the site area. The site area is currently being used for animal grazing activities. An underground gas easement housing high pressure gas lines parallels the entire length of the overhead high voltage transmission lines and towers along their southern side.

Kirker Creek crosses the western edge of the alignment. The creek is located within a canyon approximately 60 feet in depth. The creek runs approximately parallel to Kirker Pass Road.

A cut area that could have been used as a borrow site with a vertical slope measuring about 6 to 8 meters (20 to 26 feet) in height was observed near the south side of the alignment from about station 36+00 to about 37+00. Several corrugated metal culverts were noted across the site where drainage swales or courses are crossed by dirt roads.

2 GEOLOGY

2.1 REGIONAL GEOLOGY

The site is located in Contra Costa County the majority of which lies within the Coast Range Geomorphic Province in Central California. This geomorphic province contains a more or less discontinuous series of northwest-trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in regional studies by Schlocker (1970), Wagner and others (1990), Chin and others (1993), and Ellen and Wentworth (1995).

The dominant structural feature in the Coast Range Geomorphic Province is the San Andreas fault which is a strike-slip fault, with a right-lateral sense of motion. Numerous fault traces such as the Hayward, Calaveras and San Gregorio, among others, comprise the San Andreas fault system in the tectonic context of the seismically active San Francisco Bay Area. The San Andreas fault trace is the boundary between two tectonic plates, the Pacific Plate to the west of the fault and the North American Plate to the east of the fault. These two crustal plates are moving past each other in a generally northwest/southeast direction at approximately 5 cm/year (2 inch/year) at the mouth of the Gulf of California and 1 to 3 cm/year (0.4 to 1.2 inch/year) in the central and northern parts of California (Brown, 1990).

In the San Francisco Bay Area, movement along this plate boundary is concentrated on the San Andreas fault; however, it is also distributed, to a lesser extent across a number of the other near-by faults. The northwest trend of the faults within the San Andreas fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area.

The basement rocks east of the San Andreas fault are Jurassic to Cretaceous age (195-65 million years before present) rocks of the Franciscan Complex. This Complex is generally comprised of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks. West of the San Andreas fault, the basement rocks are composed of the Cretaceous age (140 to 65 million years old) granitic Salinian block. The basement rocks on both sides of the San Andreas fault are overlain by Cretaceous, Tertiary (66 to 1.6 million years old) and Quaternary age (1.6 million years or younger) marine and continental sedimentary and local volcanic rocks. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely as a result of movement along the San Andreas fault system which has been ongoing for about the last 25 million years. The inland valleys, as well as the structural depression containing the San Francisco Bay, are filled with unconsolidated to semi-consolidated surficial deposits of Quaternary age (about the last 1.6 million years or younger). Surficial continental deposits (alluvium, colluvium and landslide deposits) consist of varying mixtures of unconsolidated to semi-consolidated sand, silt, clay and gravel while the Bay deposits typically consist of very soft organic rich silt and clay (Bay mud) or sand.

2.2 SITE RECONNAISSANCE AND GEOLOGY

The site lies on the northern foothills of Mount Diablo in central Contra Costa County. Over the last approximately 40 years, the geology of Contra Costa County has been extensively mapped by the United States Geological Survey (USGS) and numerous resultant published geologic maps are available. Published geologic literature and maps reviewed for this study are listed in the "References" section (Section 9 of this report). A discussion of the site's geology is presented below.

The geology of the site and adjacent areas have been mapped by the Division of Mines (1954), Brabb (1976), Dibblee (1980), Wagner and others (1990), Chin and others (1993), Graymer and others (1994), Ellen and Wentworth (1995), and Helley and Graymer (1997). The maps generally agree on the distribution of bedrock formations in the immediate vicinity of the site.

The mapped bedrock units are listed below starting with the youngest units:

- The Pliocene age (about 2 to 4 million years old) Tulare Formation (map symbol **Ttu**) which is largely comprised of continental sandy claystone, sandstone, and conglomerate;

- The Pliocene age (about 4 to 5 million years old) Lawlor Tuff (map symbol **Tlt**) which is chiefly comprised of pyroclastic (volcanic ejecta) pumice lapilli tuff and water-reworked tuff deposited in a sedimentary setting;
- The Miocene age (about 20 million years old) Neroly Formation (map symbol **Tn**) which is mostly comprised of continental blue andesitic sandstone;
- The Miocene age (about 20 million years old) Cierbo Formation (map symbol **Tc**) which is generally comprised of marine pebbly and fossiliferous sandstone;
- The Oligocene age (about 30 million years old) Kirker Formation (map symbols **Tkt** and **Tks**) which is mainly comprised of volcanic tuff and tuffaceous sandstone; and
- The late Eocene age (about 40 million years old) Markley Formation (map symbol **Tmk**) which is represented by marine, arkosic (chiefly feldspar minerals) sandstone, shale, and siltstone.

2.3 BEDROCK UNITS

The bedrock units observed at the site were mapped by us and their approximate locations are shown on our Site Geologic Map and Subsurface Explorations Point, Plates 2A and 2B. Based on our site reconnaissance and mapping, a review of published geologic reports, as well as other information contained in site-specific studies conducted by other consultants and in our files, we have prepared the following descriptions of the bedrock formations encountered along the planned alignment.

2.3.1 Tulare Formation

The Tulare formation is Pliocene age (about 2 to 4 million years old) sedimentary formation that is largely comprised of continental (non-marine) light sandy yellowish brown claystone, sandstone, and conglomerate. These units are derived from the older sedimentary and volcanic units to the south. According to Ellen and Wentworth (1995), the sandy claystone constitutes more than 75 percent of the formation but in hard topography areas the clayey sandstone and conglomerate represent more than 50 percent of the unit's composition. The claystone and the sandstone units contain pebbly stringers and clasts from the older units such as lapilli tuff from the Lawlor, blue sandstone and petrified wood from the Neroly, and fossiliferous sandstone fragments from the Cierbo Formations. The sandy claystone is thickly and crudely bedded, lenticular, irregular, and has a low intergranular permeability. Bedrock and mantle materials are highly to severely expansive.

The Tulare formation units were encountered and their exposures were observed by our geologists during subsurface investigations and grading activities (Kleinfelder, 1999 and 2001) at Units 10, 11, 13, 15, and 16 of the Mira Vista Hills residential development in Antioch (southeast corner of James Donolon Boulevard and Somersville Road, located approximately one mile to the east of the eastern end of the alignment). The sandy claystone constituted the majority of the unit but localized clean sandy zones were encountered. The sandy claystone, described by ENGEO, Inc. (1991) as "gummy", is generally weak to plastic, highly weathered, of low hardness (firm), and thickly bedded. These characteristics resulted in the observed topography of low rounded hills. The older sections of this formation (farthest south) appear to form intermediate to hard topography implying the presence of clean sandstone and conglomerate along those zones. Colluvial expansive soils blanket the bedrock materials.

This formation is susceptible to soil creep and landslide activities. North-facing slopes (whether they are natural or cut) along with areas adjacent to the geologic contact with the underlying Lawlor Tuff units are especially susceptible to landsliding because of the weak nature of the material, the adverse northward dip of the beds and the geologic contacts.

2.3.2 Lawlor Tuff

The Pliocene age Lawlor Tuff (dated by Sarna-Wojcicki in 1976 to be approximately 4.5 million years old) is chiefly comprised of pyroclastic (volcanic ejecta) pumice lapilli tuff and water-reworked tuff that was deposited in a sedimentary setting. This unit forms distinctive resistant outcrops that can be traced almost continuously along their northwestern strike for nearly 8 miles in the general vicinity of Antioch and Pittsburg (Chesterman and Schmidt, 1956).

Ellen and Wentworth (1995) noted that the pyroclastic pumiceous lapilli tuff constitutes approximately 25 to 35 percent of the unit and that the remainder of the unit is comprised of firm tuffaceous sedimentary rock and minor basalt beds. The relatively light density pumiceous lapilli tuff is andesitic in composition (Vitt, 1936) and contains angular broken fragments of white and grayish white pumice supported by a white to pink matrix of pumicite. The tuff contains broken crystals of feldspars and olivie-basalt gravels locally. It is well compacted and forms prominent exposures and cavernous outcrops. The materials forming the Lawlor Tuff are fractured and have a low to moderate intergranular permeability where weathered and high where fresh. These units weather to an approximate depth of 20 feet and they are usually expansive where weathered with the surficial mantle severely expansive.

Based on observations recorded by our geologists during the subsurface exploration and grading activities at the Mira Vista Hills project in Antioch (Kleinfelder, 1999 and 2000), the pumiceous lapilli tuff appeared firm to hard, cohesive, brittle, and appeared clayey where weathered. At depth, zones of light density and high permeability vitric tuff were encountered in some of our borings at the Mira Vista Hills project. Such zones could react when they come into contact with concrete, collapse when wetted or loaded and are moderately to highly expansive.

The Lawlor Tuff units generally have a low to moderate landslide susceptibility. It is underlain by the Neroly Formation sandstone and overlain by the high clay content Tulare Formation. The upper surface of the Lawlor Tuff is usually less permeable (when welded or solidified) and it acts as a medium to transmit water northward beneath the base of the Tulare Formation contributing to landslide activities in the Tulare Formation near the contact with the lower tuff units.

2.3.3 Neroly Formation

The Neroly Formation is mostly comprised of continental blue andesitic sandstone and siltstone with interbeds of gravel and is commonly cross-bedded. Their blue appearance results from a translucent coating on the sand/silt grains. The coating was previously thought to be opal, but Lerbemko (1956) showed it to be a fibrous mineral instead. Snow (1957) concluded that the interstitial coating is made up of bound montmorillonitic clay.

This formation forms prominent "hogbacks" and ledges, providing good outcrop exposures. The materials comprising the formation are generally thickly bedded, moderately hard, friable, porous, and weakly cemented. The sandstone forms cavernous weathering patterns and wind sculpturing is common. Some of the upper beds are rusty and resistant due to cementation by iron oxide. Clark (1912) described large concretions (zones of concentrated cementation) within the sandstone that could reach many meters in diameter. The bedrock is fractured in a perpendicular fashion to the beds.

According to Ellen and Wentworth (1995) the sandstone has a moderate intergranular permeability while that of the siltstone is low. The rocky units may not appear clayey but weathering of the sandstone and siltstone frees clay particles bound up around the grains to where the soils derived from the bedrock are expansive. The undisturbed bedrock is considered to have low expansion but the soil mantle is highly to severely expansive.

2.3.4 Cierbo Formation

The Miocene Cierbo Formation is generally comprised of marine pebbly and fossiliferous (clam and oyster shells) sandstone. The arkosic (pre-dominantly composed of feldspar minerals) sandstone ranges from fine to coarse-grained with stringers of pebbles. Tuffaceous and diatomaceous shale and lignite seams are common. Sandstone concretions as large as 3 meters in diameter are also common at depth. The sandstone varies from strongly cemented and resistant forming hard topography to firm and nonresistant forming swales and bold rounded hills.

Ellen and Wentworth (1995) indicate that more than 60 percent of the formation is composed of the nonresistant sandstone. During our field reconnaissance, we observed the upper (younger) portion of the formation to be cemented by calcite which is crystalline, thus forming hard topography. The lower portions, however, tended to be less resistant. Conglomerate beds with hard clasts and shale partings are common within the sandstone. Spheroidal weathering patterns were noted within the strongly cemented portions. The nonresistant sandstone has a moderate intergranular permeability while the hard sandstone portion displays a very low intergranular permeability. The bedrock generally has a low expansion potential but the weathered portions and surficial mantle are considered expansive.

2.3.5 Kirker Formation

The Kirker Formation is mainly comprised of marine volcanic tuff and tuffaceous sandstone and mudstone. The tuff is vitric (composed of crystals) and lithic (composed of minute rock fragments) and is fissile.

According to Ellen and Wentworth (1995) the tuff constitutes approximately 20 percent of the formation, while the tuffaceous sandstone and mudstone make up the remaining portion. The units are firm, brittle, friable, fractured, scaly, laminated, and have a relatively light density. They also display a low to moderate intergranular permeability. Additionally, the units are considered expansive where weathered or mechanically broken and their soil mantles are severely expansive. Limited field exposures suggest that the tuff is nonresistant and the sandstone to be intermediate. The tuff was observed along the base of the drainage swale below the "cut area" near the center of Plate 2A and along the base of the hillock (small hill) situated near the extreme central western portion of the site between the Kirker Creek channel and Kirker Pass Road. The sandstone and tuff generally form swales, rounded knobs, and subdued topography and erode easily.

ENGEO, Inc. (1990) noted that these tuffaceous units react adversely with concrete, are expansive and have low strengths.

2.4 QUATERNARY SURFICIAL DEPOSITS

Quaternary surficial deposits such as alluvium, colluvium/slope wash, and landslide deposits were also mapped along some portions of the site. The approximate locations of the surface mapped Quaternary deposits are shown on our Plates 2A and 2B. These surficial deposits will be discussed further below.

2.4.1 Quaternary Alluvium

Helley and Graymer (1997) prepared a geologic map differentiating Quaternary (1.7 million years old and younger) alluvial deposits. They differentiated between Pleistocene (11,000 to 1.7 million years old) and Holocene (Recent to 11,000 years old) alluvial deposits and identified their depositional modes. These alluvial deposits are generally comprised of varying amounts of clay, silt, sand, and gravel and are transported and laid in place by running water. Helley and Graymer (1997) mapped the relatively flat topographic areas in the vicinity of Kirker Canyon Creek and along the base of the foothills as Pleistocene alluvial fan deposits (map symbol **Qpaf**). During our reconnaissance of the site, we noted that the creek channels and prominent drainage courses contained younger Holocene deposits in addition to the slightly elevated Pleistocene deposits usually bordering the creek channels. We designated the younger Holocene deposits on the Plates 2A and 2B as **Qal.**

These alluvial deposits could measure in excess of 6 m (20 feet) along the northern portions of the prominent drainage courses crossing the site. Additionally, these deposits could measure in excess of 15m (50 feet) in depth along the relatively flat and broad plain to the west and northwest of Markley Canyon Creek channel. These alluvial deposits could consist of soft and wet sediment at depth. Soft alluvial sediments are usually removed from areas to receive engineered fills to lessen subsequent settlement of the fills.

2.4.2 Colluvium and Slope Wash

Colluvium is comprised of loose, heterogeneous soil and rock material that is deposited by natural mass-wasting processes. Slope wash is made up of soil and rock materials that are or have been transported down a slope by mass-wasting processes assisted by running water. Both of these surficial deposits tend to creep down steep slope faces and are usually present along the axis of drainage swales and topographic hollows. The slope wash deposits are delineated on the Exploratory Sample Locations maps (Plates 2A and 2B) as **SR**. Colluvial deposits are generally present on nearly all of the moderately steep to steep slope faces that are covered with residual soil, and at the flatter areas at the toes of the hills. Because the distribution of the colluvial deposits is wide across the site area, we chose to delineate their presence with squiggly arrows indicating approximate direction of creep. We did not show them as a distinctive geologic unit on our Plates 2A through 2B so that the underlying bedrock geology is not obscured. Typically the upper portions of the colluvial material have undergone shrinking and swelling cycles, and are usually susceptible to consolidation when loads (such as from new fills).

These colluvial deposits are more abundant within the clayey Tulare Formation occupying the northern and central portions of the site. There are, however, more slope wash deposits along the more granular bedrock units such as the Neroly and Cierbo Formations. The colluvial and slope wash deposits tend to be more clayey towards the north and more granular southward. These deposits are relatively shallow, approximately 1.5m (5 feet), but could range between 4.6 to 6.1 m (15 to 20 feet) depending on the topographic shape of the swales housing them. These deposits are usually subexcavated and incorporated into engineered fills in proposed fill areas, and are rebuilt if exposed on cut slope faces to help stabilize them.

Two moderate size slope wash deposits underlie portions of the alignment along the southern portion of the prominent drainage swale bordering the Thomas residence to the west. The slope wash deposits generally occur in the Neroly and Cierbo Formation materials.

2.4.3 Landslides

Numerous landslide deposits were identified within the site area during our aerial photograph interpretation, reconnaissance and literature review performed as part of this study. Active and dormant landslide deposits were mapped and their approximate lateral boundaries and degree of activity are indicated on Plates 2A and 2B. We marked the active landslides with a circled "**A**" to differentiate them from the mapped dormant ones we marked with a circled "**D**". In addition, slope wash deposits were mapped and their approximate lateral boundaries are indicated on Plates 2A and 2B. Slope wash deposits are marked with a circled "**SR**".

The active landslides are those that display recent topographic signs of activation such as head scarps, soil cracking outlining portions or all of the landslide, hummocky terrain, dirt road distortions, striated head scarps, and raised toe areas. Dormant landslides are those that lack indications of recent movement. Their outlines are generally subdued, they lack head scarps and the slide mass itself seems to be less angular. The dormant landslides tend to be stable in their current configurations but may reactivate if existing conditions are altered. Dormant landslide reactivation could occur due to seismic activities, increase in groundwater flow, grading operations undermining the toe area or loading the higher portions of the landslide with fill. Slope wash deposits are shallow instances of soil creep.

The majority of the landslide deposits mapped (active and dormant) are situated within the Tulare Formation. The four largest landslide deposits do not underlie the alignment, however several significant lanslides are anticipated to effect the alignment. The locations and a brief discussion of the landslide deposits that will impact the alignment are as follows (stationing is reference to the Final Central Alignment" dated July 9, 2007 dated map by RBF):

- A large landslide deposit was mapped underlying the alignment from about Station 26+00 to about Station 26+50. It is delineated on Plate 2A as "1", and is shown on the western portion of this plate. It measures approximately 60 meters (190 feet) in width and 50 meters (160 feet) in length and is mapped as dormant.
- A second landslide deposit (designated as "2" on Plate 2A) underlies the alignment from about Station 33+25 to about Station 33+55. The landslide's approximate location and lateral limits are shown along near the center of Plate 2A. It measures approximately 30 meters (85 feet) in width and 90 meters (290 feet) in length, is mapped active and is currently failing.
- A third landslide deposit (designated as "3" on Plate 2A) underlies the alignment from about Station 41+50 to Station 42+50. The landslide measures approximately 130 meters (415 feet) in width and 100 meters (320 feet) in length and is mapped as dormant. The landslide is shown on the eastern portion of Plate 2A, on the western side of a large drainage swale.
- A fourth landslide deposit (designated as "4" on Plate 2B) is mapped on the western portion of Plate 2B. The toe of the landslide underlies the alignment from about Station 46+15 to about Station 46+90, and the majority of the landlside is located to the south (upslope) of the alignment. The landslide is shown on the western portion of Plate 2B. The landslide measures approximately 140 meters (450 feet) wide and

180 meters (580 feet) long and is mapped as dormant.

- A fifth landslide deposit (designated as "5" on Plate 2B) is mapped to the north of the alignment and landslide "4", on the western portion of Plate 2B. The scarp of the landslide is located adjacent to the toe of the alignment fill. The landslide measures approximately 45 meters (145 feet) in width and 85 meters (270 feet) in length, and is mapped as dormant.

Several additional landslide deposits that are relatively smaller were mapped along portions of the alignment. We did not observe large offsite landslide deposits that could encroach on the site and adversely effect the alignment.

2.5 SITE SOILS AND SOIL SURVEY MAPS

The soils within the site area have been classified and described by the U.S. Soil Conservation Service (1977). They utilized aerial photographs as base maps. Their maps show the majority of the northern portion (underlain by the Tulare Formation) to belong to the Altamont Group. These materials are described as having a high shrink and swell potential and are corrosive. They indicate that care must be taken when using these materials for roadway construction due to their high expansion potential and low strengths.

According to the Soil Survey Maps, the more resistant outcrop and peak areas are underlain by soils of the Lodo Series. These materials are described as having a very thin veneer of soil covering rocky terrain. Their expansion and corrosion potentials are more moderate than the Altamont Group soils. The disadvantages associated with this type of soils are their limited supply and possibly low strengths.

Along the relatively flat areas surrounding the channels of Markley and Kirker Creek channels, the soil survey maps show soils belonging to the Rincon Soil Series. These soils tend to be similar in their engineering characteristics to the Altamont Group soils. Table (~~6~~.5-1) below presents more detailed characteristics of the soils discussed above.
2

The soil types discussed above were not shown on our Preliminary Site Geologic Maps. We chose not to delineate them on our maps so that the geological units and contacts are not obscured.

TABLE 2.5-1
Engineering Characteristics of the Soils within the Project Area

Soil Series or Group	Depth to Bedrock (meters) (feet)	Plasticity Index	Shrink/Swell Potential	Corrosivity	Permeability
Altamont	31/2 - 5 1.1 – 1.5	25-30	High	High	Slow
Lodo	1 – 11/2 0.3 – 0.5	15-20	Moderate	Moderate	Slow
Rincon	>5 .1.5	15-25	High	High	Slow

Note: Derived from the Soil Survey Publication for Contra Costa County (USDA, 1977)

2.6 GEOLOGIC STRUCTURE

The older bedrock formations such as the Markley, Kirker, and Cierbo were initially deposited in a horizontal fashion atop each other by sedimentary processes in a marine environment. A period of erosion followed after each layer was deposited but before the following layer was laid. Mountain building and uplift episodes caused the deposited layers to form a part of the continent and become exposed. A period of erosion along the exposed surface of the deposited and uplifted layers followed, after which a depositional period of continental sedimentary and volcanic formations such as the Neroly, Lawlor Tuff, and Tulare occurred. The units were subsequently tilted northward rendering the units stacked in their current position becoming older towards the south.

The thickness and width of the exposed portions of the various formations varied laterally because the units weathered differentially along their exposed surfaces before the next layers were deposited.

The units generally strike northwestward at an angle ranging between 55 and 75 degrees and have associated dips varying between 30 to 40 degrees to the east of north. The bedding of the Tulare formation is generally subdued and obscured but bedding readings recorded by our geologists at the Mira Vista Hills Subdivision indicate that the bedding angles may be as gentle as 20 degrees to the north.

The geologic units and formations mapped across the site dip northeastward. Accordingly, cut slopes steeper than 20 degrees could potentially present adverse bedding (dip slope) conditions that could contribute to or promote slope instability and failure. Planned cut slopes steeper than 20 degrees may need to be rebuilt as engineered fill.

2.7 AERIAL PHOTOGRAPH INTERPRETATION

Aerial photographic stereo pairs purchased specifically for our previous study were reviewed as part of our geologic investigation of the site. The reviewed photographs are listed below.

Date	Source	Flightline/Frames	scale	Color
6/29/99	PAS	CC-AV-6100-127 (6, 7 & 8)	1:12000	B&W
6/29/99	PAS	CC-AV-6100-128 (6, 7 & 8)	1:12000	B&W
6/29/99	PAS	CC-AV-6100-129 (7 & 8)	1:12000	B&W

These photographs were reviewed for the presence of terrain features indicative of active or dormant landslides and for features characteristic of fault zones, particularly lineaments. A lineament is seen on a stereo aerial photograph pair as a feature with tonal contrast on either side. These differences may be indicative of changes in soil types, vegetation, groundwater levels or sedimentary bedding characteristics. Lineaments are often indicative of the presence of geologic structures such as folds and fault.

A buried gas line forming a linear feature extending from the southwest corner immediately south of and parallel with the power transmission towers to the northeast corner of the site at a point where Markley Creek crosses Somersville Road is visible on the aerial photographs.

The northern area underlain by the Tulare Formation appeared dark-toned and exhibited soft (rounded) topography that lacks ribs (angular). The elevated southern portions of this formation near the contact with Lawlor Tuff displayed intermediate topography which could represent the location of the more resistant sandstone and conglomerate units within the formation. Numerous active and dormant landslide deposits were observed within the Tulare formation. Some of these deposits are very large and dormant and others are relatively large and are part of a whole landslide complex encompassing the entire slope faces at those locations.

The area where the Lawlor Tuff was mapped appeared light-toned with alternating dark layers. The unit appeared to exhibit variable resistant with some individual beds forming prominent outcrop exposures. The uppermost portion (youngest) of the unit is represented by a well-compacted (possibly partly welded) tuff bed that extends nearly across the whole site area in a northwestern trend atop which lies the Tulare Formation units.

The area where the Neroly Formation was mapped prominently stands out as hard topography with abundant outcrop exposures. The sandstone outcrops form hard crests and bands representing bedding. The beds appeared to alternate between light and dark gray tones possibly indicating degrees of differential weathering. The beds are stacked and nearly continuously extend across the site area along their northwestern strike line.

The area occupied by the Cierbo Formation sandstone appeared faintly banded and somewhat resistant especially near the contact with the overlying Neroly Formation sandstone beds. The central and southern sections of the unit appeared non-resistant with subdued topographic expression forming swales and bold rounded hills.

The Kirker Formation units generally appeared intermediate to non-resistant forming swales and drainage gullies and soft knobs along white bands. The zone where the Markley Formation was mapped appeared fairly non-resistant forming ridges and hill slopes rather than swales and gullies. The area exhibited subdued "hogback" pattern with subtle banding.

We did not observe features indicative of faulting in the vicinity of the possible northward trending fault shown on ENGEO Inc.'s (1990) geologic map generally along the center of the site. ENGEO Inc. (1990) showed displaced beds in a left lateral sense of motion along the trend of the mapped fault. The mapped feature may represent a shear zone or an inactive bedrock fault. Dibblee (1980) and Graymer and others (1994) did not map a fault in that vicinity.

2.8 GROUNDWATER

Groundwater was not encountered in our exploratory soil or rock core borings. No springs or seeps indicative of shallow groundwater were observed during our reconnaissance. Groundwater may be encountered along the drainage courses during the grading operations, especially if cuts are made along these drainage courses. Groundwater levels may be higher during the rainy season.

3 FAULTING AND SEISMICITY

The project site is located in a region which is traditionally characterized by moderate to high seismic activity. Based on the information provided in Hart and Bryant (1997), the site is not located within an Alquist-Priolo Earthquake Fault Zone where special studies addressing the potential of surface fault rupture are required. The closest fault is the Greenville-Marsh Creek fault located at a distance of about 5.6 km (3.5 miles) towards the southwest. A major earthquake on this fault could cause significant ground shaking at the site. A list of the significant regional faults and their seismic parameters are presented in Table 3-1 below. Some of the faults located in the region (not considered independent seismogenic sources) and not listed in the table are the Antioch fault, the Livermore fault, the Pleasanton fault, and the Veronas-Las Positas fault. Plate 3 is a map showing the location of some of the regional faults in relation to the site.

The locations of the faults and associated parameters presented on Table 7-1 below are based on data presented by Real and others (1978), Toppozada and others (1978), Hart and others (1984), Wesnousky (1986), Working Group on California Earthquake Probabilities (1999), Schwartz (1994), Jennings (1994), Frankel and others (1996), and Petersen and others (1996). The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Kanamori (1977).

TABLE 3-1: SIGNIFICANT FAULTS

Fault Name and Geometry (1) and Location (2)	Fault Length (km)	Closest to Site (km)	Magnitude of Maximum Earthquake * (3)	Slip Rate (mm/yr)
Greenville-Marsh Creek (rl-ss) (SW)	73 ± 7	5.6	6.9	2 ± 1
Great Valley 6 (r,15,W) (NE)	45 ± 5	10	6.7	1.5 ± 1
Concord-Green Valley (rl-ss) (W)	66 ± 7	13	6.9	6 ± 3
Great Valley 5 (r,15,W) (NE)	28 ± 3	16	6.5	1.5 ± 1
Mt. Diablo Thrust (r,W) (SW)	25 ± 5	18	6.7	3 ± 2
Calaveras (northern) (rl-ss) (SW)	52 ± 5	20	6.8	6 ± 2
Hayward (rl-ss) (SW)	86 ± 9	35	7.1	9 ± 1
Great Valley 4 (r,15,W) (N)	42 ± 4	36	6.6	1.5 ± 1
West Napa (rl-ss)	30 ± 3	38	6.5	1 ± 1
Rodgers Creek (rl-ss) (NW)	63 ± 6	42	7.0	9 ± 2
Great Valley 7 (r,15,W) (SE)	45 ± 5	43	6.7	1.5 ± 1

Hayward (SE Extension) (rl-r-o) (S)	26 ± 3	58	6.4	3 ± 2
Hunting Creek-Berryessa (rl-ss) (NW)	60 ± 6	60	6.9	6 ± 3
Calaveras (southern) (rl-ss) (S)	106 ± 11	61	6.2	15 ± 2
San Andreas (1906 Event) (rl-ss) (SW)	470 ± 47	64	7.9	24 ± 3
San Gregorio	129 ± 13	70	7.3	5 ± 2
Monte Vista-Shannon (r,45,E) (SW)	41 ± 4	72	6.8	0.4 ± 0.3
Great Valley 3 (r,15,W) (N)	55 ± 6	77	6.8	1.5 ± 1
Point Reyes (rl-ss) (NW)	47 ± 5	85	6.8	0.3 ± 0.2
Great Valley 8 (r,15,W) (SE)	41 ± 4	88	6.6	1.5 ± 1
Sargent (rl-r-o) (S)	53 ± 5	95	6.8	3 ± 1.5
Ortigalita	66 ± 7	97	6.9	1 ± 0.5
Zayante-Vergeles	56 ± 6	100	6.8	0.1 ± 0.1

(1) ss = strike slip; r = reverse; n = normal; rl = right lateral; o = oblique; 15 W = Dip angle and direction
(2) W = West; E = East; SW = Southwest; NE = Northeast; S = South
(3) Moment Magnitude based on rupture area regressions from Wells and Coppersmith (1994) need reference

The project site and its vicinity are located in an area traditionally characterized by high seismic activity. A number of large earthquakes have occurred within this area in the past years. Some significant regional earthquakes include the 1889 (M6.3) Antioch earthquake, the 1868 (M7) Hayward earthquake, the 1906 (M7.9) San Francisco earthquake, the 1838 (M7) San Francisco/San Mateo earthquake, the 1858 (M6.1) Mission Peak area earthquake, the 1861 (M5.7) San Ramon Valley earthquake, the two 1903 (M5.5) San Jose earthquakes, the July 1911 (M6.6) Calaveras earthquake, the 1957 (M5.3) Daly City earthquake, the 1980 (M5.8) Livermore earthquake, and the 1989 (M6.9) Loma Prieta earthquake.

The earthquake database used in our search contains in excess of 5,500 seismic events and covers the period from 1800 through July 2002. The earthquake database is principally comprised of an earthquake catalog for the State of California prepared by the California Geological Survey, CGS (formerly known as the California Division of Mines and Geology, CDMG). The original CGS (CDMG) catalog (Real and others 1978) is a merger of the University of California at Berkeley and the California Institute of Technology instrumental catalogs (Hileman and others 1973). The combined catalog contains earthquake records from January 1,1900 through December 31, 1974. Updates prepared by CGS (CDMG) in 1979 and 1982 extend the coverage through 1982. In addition to the CGS (CDMG) updates, the data for earthquakes for the period between 1910 and July 2002 have been obtained from a composite catalog by Council of the National Seismic System (CNSS). The CNSS catalog is a worldwide earthquake catalog that is created by merging the master earthquake catalogs from contributing CNSS member networks and then removing duplicate events, or non-unique solutions from the same event. The CNSS network includes Northern and Southern California Seismic Networks, Pacific Northwest Seismic Network, University of Nevada, Reno Seismic Network, University of Utah Seismographic Stations and US National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900. This subset of the earthquake database was derived from Seeburger and Bolt (1976) and Toppozada and others (1978, 1981). In addition, we have also utilized the data from CGS (CDMG) Map Sheet 49 (Toppozada and others 2000).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km [62 miles] of the site), dates (1800 through July 2002), and magnitudes (M>4). A summary of the results of the historical search is presented below.

Time Period (1800 to July 2002)	202+ years
Maximum Magnitude	M8+
Approximate distance to nearest historical M> 4 earthquake	2 km (1.2 miles)
Number of events exceeding magnitude 4 within search area	142

4 FIELD INVESTIGATION

Field investigation of the alignment was performed to explore the subsurface conditions, and to supplement the previous investigation. Our previous field investigation consisted of a site reconnaissance and geologic mapping. Our current investigation consisted of site reconnaissance, seismic refraction, exploratory borings, exploratory cores and trenching. The subsurface exploration consisted of drilling 8 solid stem auger borings using a truck-mounted drill rig, drilling 4 wire-line rock cores using a track-mounted drill rig, and excavating 19 test pits and one trench with a backhoe.

Our field professionals selected the test pit locations, test pit depths, seismic traverse locations, boring locations, boring depths, sampling intervals, and observed the drilling operations. Our field professionals logged the borings, trenches and cores on a full-time basis. Sample classifications, blow counts recorded during sampling, RQDs and other related information were recorded on the boring logs, core logs and test pit logs. A key to the logs of borings and cores is presented on Plate A-1 of this appendix. Logs of Borings are presented on Plates A-2 through A-9. Logs of Rock Cores are presented on Plates A-10 through A-13. Logs of Test Pits are presented on Plates B-1 through B-12. In addition to the test pits, a fault investigation trench was excavated. The log of the fault investigation trench is presented on Plate B-13. The velocity profiles interpreted from the seismic refraction lines are presented on Plates C-1 through C-5. The approximate locations of the borings, cores, test pits, trench and seismic traverses are presented on Plates 2A and 2B. The boring, core, trench and traverse locations were estimated by our field professional based on visual sightings and/or pacing from existing site features.

A more detailed discussion of the various field exploration elements are presented below.

4.1 HOLLOW STEM AUGER BORINGS

Eight hollow stem auger soil borings were drilled and sampled by Frontier Drilling of Linden, California using a truck mounted drill rig equipped with 6-inch outside diameter solid stem augers.

Relatively undisturbed samples of the subsurface soils were recovered using a 2.0-inch inside diameter and 2.5-inch outside diameter stainless steel or brass tube lined Modified California Sampler within cohesive soils and a 1.4-inch inside diameter and 2.0-inch outside diameter unlined Standard Penetration Test Sampler within granular soils. The samples were driven by a 140 point hammer falling 30 inches. The number of blows required to drive the last 12 inches of an 18 inch drive are recorded on the Penetration Resistance (blows/foot) on the boring logs. The blow counts on the logs have not been corrected to Standard Penetration Resistance (N-Counts). When each sampler was withdrawn from the boring the samples were removed, examined for logging purposes, sealed, labeled, and subsequently transported to our laboratory for testing.

4.2 WIRE-LINE ROCK CORES

Four rotary wash borings (C-1 through C-4) were drilled and sampled by Gregg Drilling Company of Martinez, California using a track mounted drill rig equipped for wire-line rock coring.

Rock cores were obtained used rock-coring equipment configured for the wire-line drilling method. In the wire-line system, the five-foot long drill rods act as a casing and fluid is circulated from the bit through the annulus between the drill hole wall and drill rod. The inner core barrel is removed and replaced without removing the drill rods, allowing for continuous coring. The inner barrel assembly is locked into the lead section of the wire-line drill rod by means of a retrievable overshot latching mechanism. After the core run, the overshot mechanism is lowered through the rods and latched onto a spearhead on the top of the core barrel assembly that is then hoisted to the surface with a cable and wire-line winch. Two inner core barrel assemblies are used for maximum productivity in continuous coring. The wire-line drilling method was used under the general guidelines of ASTM D 2113 "Standard Practice for Diamond Core Drilling for Site Investigation".

Where rock coring was used, continuous sampling using HQ core barrel samplers was performed using a Sentex carbide bit. The inside diameter of the HQ core barrel is 2 3/8-inches and the set reaming shell (hole diameter) is 3 3/4-inches. The cores were typically retrieved in five-foot runs. Upon retrieval, the core barrel was split apart and the sample removed. The samples were placed into core boxes and examined for classification characteristics and quality. Rock Quality Designations (RQD) were measured and recorded under the general guidelines of ASTM D6032 "Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core". The RQD is defined as the cumulative length of core pieces longer than 4 inches in a run divided by the total length of the core run. The total length of core includes all lost core sections. Any mechanical breaks caused by the drilling process or in extracting the core from the core barrel are ignored.

Logs of the borings based on visually classified materials encountered and obtained relatively undisturbed samples of the subsurface materials have been prepared and are presented in Appendix B. Rock classifications made in the field from cores were re-evaluated in the laboratory after further examination and laboratory testing.

4.3 TRENCHING

Nineteen test pits and one fault trench were excavated from July 9 through July 12, 2007. The trenches were excavated using a four wheel-drive backhoe equipped with a 30-inch bucket at the approximate locations shown on Plates 2A and 2B.

The exploratory trenches ranged in depth from about 2 to 15 feet and in length from about 10 to 55 feet. One wall of each trench was cleaned of "backhoe bucket smear" utilizing hand tools, and the exposed surfaces were subsequently logged by a Kleinfelder staff geologist.

The logs of our geologic trenches are presented in Appendix B as Plates B-1 through B-12. A fault investigation trench was also performed, and the log is presented in Appendix B as Plate B-13. At the conclusion of the trenching operations, the trench excavations were backfilled with soil cuttings. It should be noted that the trench backfill was compacted

4.4 SEISMIC-REFRACTION SURVEY

4.4.1 General

To evaluate bedrock rippability at selected sites along the alignment of the Buchanan Road extension, five seismic-refraction lines (SRL-1 through SRL-5) were run on July 12, 2007. The locations of the refraction lines were chosen to obtain relative rippability along ridge crests and slopes where bedrock is inferred to be more resistant.

A seismic-refraction survey consists of inducing shear waves from an energy source such as an explosive shot or sledgehammer blow into the earth along an array of signal receivers (geophones). The shock waves enter the earth at the shot point as omni-directional waveforms. The velocity with which the waves move through the earth is dependent on the density and strength parameters of the earth materials it encounters. Shallow, relatively slow velocity soil and weathered rock will transmit the wave to the closest geophones first. Waves within faster and deeper velocity materials will overtake waves in slower materials and register at geophones farther away from the shot point before the slower waves arrive. Interpreting the resulting shear-wave's first arrival times is used to develop a numerical and graphic model of subsurface conditions.

The speed with which rock transmits shock waves is controlled by its strength and degree of induration and these characteristics materially affect the rock's rippability. Conditions that are favorable for seismic-wave transmission and therefore unfavorable for rippability include:

- Massive or homogeneous rock units
- Absence of planes of structural weakness
- High degree of cementation
- High compressive strength
- High rock quality determination (RQD)

Rock conditions that are favorable for rippability include:

- Presence of fractures, faults, and planes of weakness

- Weathering
- Brittleness
- High degree of stratification or lamination
- Large, loosely cemented grains
- Low compressive strength
- Low rock quality determination (RQD)

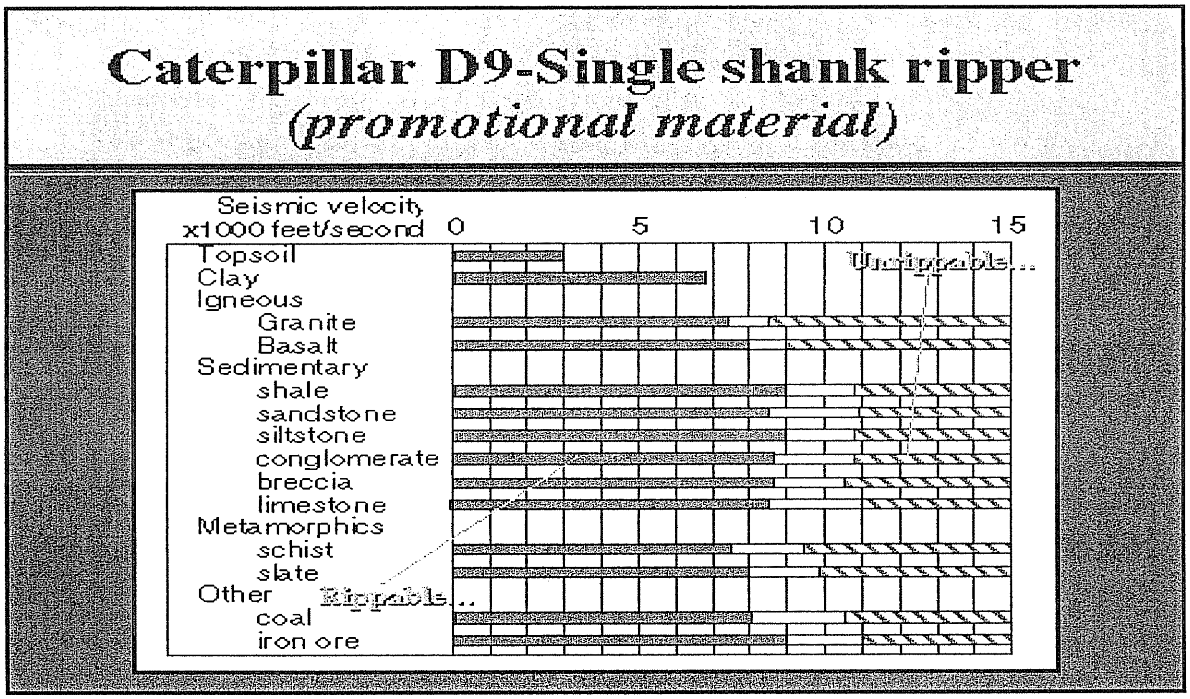
Each of the seismic-refraction lines conducted for this project consisted of an array of twelve 14hz vertical geophones equally spaced over a distance of 110 feet. The recording instrument was a 12-channel, Geometrics S-12 seismograph. Energy was applied to the earth along a five-point shot array with a ten-pound sledgehammer fitted with a trigger mechanism that actuated the seismograph-receiving window. Surface profiles along the seismic lines were derived from the topographic site plan provided by Final Central Alignment" dated July 9, 2007. Data were reduced using "SeisImage," a software program developed by Geometric Inc. of San Jose, California.

The approximate locations of the five seismic-refraction lines are presented on Plates 2A and 2B. The velocity profiles interpreted from them are presented in Appendix C.

4.4.2 Rippability Evaluation

In evaluating the seismic-refraction velocities with respect to rippability, we used Caterpillar Tractor Company, Handbook of Rippability for heavy duty ripper performance. This table is as follows:

TABLE 4.4.2 – 1



The table provided by Caterpillar Tractor Company is for a large track bulldozer with a single ripper hook attached. This rippability rating was used as an indicator of the relative difficulty anticipated in excavating rock at the selected sites along the Buchanan Road alignment and should be adjusted based on the equipment selected by the contractor for this project. For conditions where seismic-refraction data is within the rippable range, it should be expected that hard areas may be encountered.

It is important to note that the operator's experience, working condition of excavation equipment, and the selection of excavation tools used will be critical factors in the excavatability of rock. During construction, modifications to tool selection or replacement of equipment being used may be necessary to improve performance and production rates. It is recommended that the contractors who use the rippability data in this report visit the proposed location of the roadway to observe bedrock conditions. It is recommended that the contractor have options available in order to deal with differing bedrock conditions. Table 4.4.2- 2 below presents the interpreted results of the seismic-refraction lines.

Table 4.4.2 - 2: Seismic Refraction Data

SRL*	PLATE	Geologic Material; Approximate Seismic Velocity	Rippability Conditions†
1	C-1	Soil and weathered rock - 0 to about 20 ft. thick (3,600 ft/s) over sandstone (5,400 ft/s)	Rippable
2	C-2	Soil and weathered rock - 0 to about 10 ft. thick (3,000 ft/s) over sandstone (4,300 ft/s)	Rippable
3	C-3	Soil and weathered rock - 0 to about 30 ft. thick (3,000 ft/s)	Rippable
4	C-4	Soil and weathered rock - 0 to about 20 ft. thick (2,200 ft/s) over sandstone (5,200 ft/s)	Rippable
5	C-5	Soil and highly weathered rock - 0 to about 10 ft. thick (<1,000 ft/s) over weathered sandstone (5,200 ft/s)	Rippable

* : Seismic refraction line
† : Based on Caterpillar Tractor Company, Handbook of Rippability for Heavy Duty Ripper Performance

The seismic velocities obtained during our seismic-refraction survey are necessarily averages across differing soil and rock conditions. Bedrock units can have coherent rock masses separated by discontinuities (fractures, bedding planes, joints, highly weathered zones, etc). This fact should be considered and allowed for when using seismic-refraction to estimate rippability. Seismic waves travel through coherent rock relatively fast and travel through intervening discontinuities relatively slow. The resulting seismic velocity through rock units will be the sum of velocities across coherent rock masses and discontinuities and will not be a true velocity through either rock type. As a result, variable rippability or excavation conditions both harder and softer can be encountered along the survey line.

The "layers" shown on Plates C-1 through C-5 are velocity layers and reflect interpreted zones of relatively consistent velocities and may not represent rock contacts or other physical characteristics. The bedrock in the area of the seismic lines dips at about 20 to 30 degrees to the north. This structural orientation was not observed in the seismic profiles.

5 LABORATORY TESTING

A laboratory testing program was performed on samples retrieved. The laboratory testing program was conducted to assess the moisture content, unit weight, particle size, unconfined compressive strength, direct shear strength, percent swell, and/or corrosivity of the soils and rock sampled. The laboratory test results performed on samples collected from the site are presented on the individual boring and test pit logs of Appendices A and B. Graphic presentation of the Atterberg Limits, particle size, consolidation and swell tests are presented on Plates D-1 through D-7 in Appendix D. Results of the corrosivity testing, performed by Cerco Analytical, Inc. of Pleasanton, California, are presented in Appendix F.

6 SUBSURFACE CONDITIONS

The subsurface soils encountered in the borings and pits excavated at the site generally consist of medium stiff to very stiff sandy and silty clays. These clayey soils are moderately to highly expansive. Thin layers of sand, silty sand, clayey sand and clayey gravel were interlayered within the cohesive clayey soils. These granular materials were generally medium dense to dense. The native soils are underlain by bedrock materials consisting of heterogeneous layers of claystone, siltstone, and sandstone. The upper portion of the bedrock materials were generally described as weathered and moist. The claystone materials exhibited moderate to very high expansive characteristics. Free groundwater was not encountered in the borings, rock cores or test pits.

The above is a general description of soil, rock, and groundwater conditions encountered at the site in the borings, rock cores and test pits excavated during this study. A more detailed description of the soil, rock, and groundwater conditions encountered at the site is presented in Section 2 "Geology" of this report and on the logs of borings, and test pits in Appendices A and B.

Soil and groundwater conditions can deviate from those conditions encountered at the boring locations and be influenced by weather (drought seasons), pumping, grading and other factors. Should this be revealed during construction, Kleinfelder should be notified immediately for possible revisions to the recommendations that follow.

7 CUT SLOPE STABILITY ANALYSIS

Our stability analysis of the planned cut slopes included developing representative soil/rock strength parameters, cross-sections showing the geology and groundwater levels, and modeling of static and pseudo-static conditions utilizing horizontal seismic coefficients. A brief discussion of these items is presented below.

7.1 SOIL/ROCK STRENGTH PARAMETERS

We utilized nine different soil/bedrock units in the geologic modeling for our slope stability analyses. These units consist of landslide deposit materials, basal landslide plane, Alluvium, the Lawlor Tuff bedrock, the Tulare Formation bedrock, the Neroly Formation Bedrock, the Cierbo Formation bedrock, the Kirker Formation bedrock, and engineered fill. In addition, we varied the strength parameters of the soil and rock properties, groundwater conditions, final slope inclination, and the need for buttress fills during our analysis to evaluate for sensitivity of these items. The cross-sections evaluated were based on height and type of material. The sections are designated as B, D, E, F and G, and are shown approximately on Plates 2A and 2B. We analyzed three conditions that included: 1) Static stability of the final graded condition without groundwater; 2) The impact on the static stability if the groundwater (although not encountered during our exploration) would rise to the bottom of subdrainage system to be incorporated into the planned grading, and 3) Stability during a seismic event. The properties used for our stability analysis is summarized in the following table:

TABLE 7.1 – 1 Summary of Stability Analyses Material Properties

Soil/Rock Type	Unit Weight (pcf)	Cohesion (psf)	Frictional Angle (degree)
Engineered Fill	124	500	28
Landslide Deposit	125	800	18
Basal Landslide Plane	120	0	15
Alluvium (Qpaf)	120	800	18
Tulare (Ttu)	125	800	20
Neroly (Tn)	120	1750	25
Cierbo (Tc)	120	1750	25
Kirker (Tks, Tkt)	115	1000	25
Lawler (Tlt)	115	500	36

A discussion on the derivation of the strength parameters used for the nine units is provided below.

7.1.1 Engineered Fill

Engineered fill will be used to construct keyways for the slope repairs, buttresses for cut slopes and fill slopes. As such, we selected strength parameters for the engineered fill based on the laboratory results presented in Table 1 (see Appendix E) of our November 29, 1999 geological and geotechnical investigation report titled "Geologic and Geotechnical Investigation Report, Subdivision 6824, Mira Vista Hills Units 10, 11, 13, and 15, Antioch, California," (File No. 11-2721-00), and on our experience with similar soil types in the San Francisco Bay Area (Bay Area). We selected a Friction (ϕ) angle of 28 degrees and a Cohesion (c) of 500 pounds per square foot (psf) for the engineered fill in our analyses.

7.1.2 Landslide Deposit and Alluvial Materials

We assigned strength parameters to the landslide deposit and alluvial materials based on the direct shear test results presented in Table 1 of our November 29, 1999 report, and our experience with similar materials within the Bay Area. The landslide materials are generally comprised of alluvial materials. Accordingly, we modeled the landslide deposits using a cohesion of 500 pounds per square foot (psf) and an internal friction angle of 28 degrees during our analyses.

7.1.3 Basal Landslide Plane

Strength parameters for the Basal Landslide plane were developed based on the results of laboratory testing performed during previous studies at near-by sites, (Kleinfelder, 2001) and our experience with similar soil types within the Bay Area. We chose a Friction angle of 15 degrees and a Cohesion value of 0 psf for the Basal Landslide plane in our analyses.

7.1.4 Tulare Formation (Bedrock)

Strength parameters for the Tulare Formation bedrock unit were developed based on the results of laboratory testing performed during previous studies at near-by sites, (Kleinfelder, 1999) and our experience with similar material types within the Bay Area. We chose a Friction angle of 20 degrees and a Cohesion value of 800 psf for the Lawlor Tuff in our analyses.

7.1.5 Lawlor Tuff (Bedrock)

Strength parameters for the Lawlor Tuff bedrock unit was developed based on the results of laboratory testing performed during previous studies at near-by sites, (Kleinfelder, 1999) and our experience with similar soil types within the Bay Area. We chose a Friction angle of 36 degrees and a Cohesion value of 500 psf for the Lawlor Tuff in our analyses.

7.1.6 Neroly and Cierbo Formations (Bedrock)

Strength parameters for the Neroly and Cierbo formations were developed based on the direct shear test results performed on samples of the Nerole formation during our investigation at a nearby site (Kleinfelder, 2004). The direct shear performed on this material indicates an ϕ of 25 degrees and a C value of 1750 psf. For simplicities sake, we used these values for both the Neroly and Cierbo bedrock material.

7.1.7 Kirker Sandstone and Tuff (Bedrock)

Strength parameters for the Kirker formation were developed based on our experience with similar material types in the adjacent formations. We chose a Friction angle of 25 degrees and a Cohesion value of 1000 psf for the both the Kirker Tuff and Sandstone in our analyses.

7.1.8 Lawlor Tuff (Bedrock)

Strength parameters for the Lawlor Tuff bedrock unit were developed based on the results of laboratory testing performed during previous studies at near-by sites, (Kleinfelder, 1999) and our experience with similar material types within the Bay Area. We chose a Friction angle of 36 degrees and a Cohesion value of 500 psf for the Lawlor Tuff in our analyses.

7.2 STATIC ANALYSES

In order to evaluate the long term performance of the proposed cut and fill slopes discussed in this report, we performed slope stability analyses on Cross Sections B, D, E, F and G using the limit equilibrium computer code SLOPEW, developed by Geo-Slope, International. The analyses used the Spencer Method to evaluate the cross sections. The objective of our analysis was to evaluate whether the proposed slopes would yield a factor of safety (FOS) under static conditions of 1.5 or higher after grading. According to current standards of practice used in geotechnical engineering for the state of California, FOS values of 1.5 or higher for static conditions are indicative of stable slopes.

7.3 PSEUDO-STATIC (DYNAMIC) ANALYSES

Besides evaluating the stability of the proposed slope repairs under static conditions, we evaluated the recommended permanent repairs under seismic (pseudo-static) conditions. According to current standards of practice used in geotechnical engineering for the State of California, FOS values higher than 1.0 for seismic conditions are indicative of stable slopes.

A seismic coefficient of 0.23 which is typically required by many state and local agencies in California (Division of Mines and Geology – Department of Conservation, 1997) was used to model dynamic loads. Material properties for the landslide deposit material, landslide plane, Lawlor Tuff, Tulare Formation, and engineered fill were not changed for the pseudo-static analyses.

7.4 SLOPE STABILITY RESULTS

The result of our static and seismic slope stability analyses is presented in the table below. It was our understanding that the desired inclination of the planned cuts is to be at 2:1 (horizontal to vertical). Our analyses began at that inclination. For some of the planned cut slopes, gentler inclinations were needed to achieve a static factor of safety of approximately 1.5 or higher. In addition, for slope D-D', the static factor of safety with groundwater was 1.4, which is slightly below the desired 1.5. Without groundwater, the static factor of safety of over 1.8. Since it is not feasible for such a high cut to have groundwater buildup throughout the entire height of the cut, and that the top of the cut is at a knob of a hill with minimum area for water infiltration, it was concluded that the static factor of safety is acceptable. Based on the bedding of the materials to be encountered in some of the cuts, buttress fills will be needed. A buttress fill is engineered fill that is placed in front of the planned cut slope. This will require the overexcavation of the face of the slope, installation of a keyway and subdrainage, and the placement of fill over the entire height of the cut. A more detailed discussion of buttress fills is presented in Section 8.5.1 "Cut Slopes" of this report.

The summary of the slope stability analyses area as follows:

TABLE 7.4 – 1
Slope Stability Results

Slope Location	Slope Inclination (horizontal to vertical)	Static		Seismic	Temporary
		With Groundwater	Without Groundwater		
Cross-Section B-B' (Station 4+00 to 47+50) (1)	2.5:1	1.6		1.1	
Cross-Section D-D' (Station 43+00 to 46+00)	3.25:1	1.4	1.8	1.8	
Cross-Section E-E' (Station 32+00 to 39+00)	2:1	1.5	2.0	1.3	
Cross-Section F-F' (Station 28+00 to 30+00)	2:1	1.5	2.0	1.25	
Cross-Section G-G' (Station 25+00 to 27+00)	2:1	1.5	1.6	1.0	
Cross Section B-B' (Station 46+00 to 27+50)	2:1	-	-	-	0.9

Note: (1) Based on RBF Consulting map dated 7/9/07

The cross-section at B-B' includes grading that consists of cutting within an existing landslide. This landslide is designated as "4" on Plate 2B. This landslide also extends below the planned fills for the bypass road. Our slope stability analysis indicates that the global stability of the landslide where to be cut has a static factor of safety of 1.6 (greater than the desired 1.5) with groundwater. As such, the entire landslide does not need to be removed other than to achieve an inclination at the face of 2.5:1. At this inclination, there is potential that the surfacial soils of the landslide may slough and move downward toward the planned bypass. Most of the bypass in this area is on fill, which results in a raised section. As such, there is a built in debris encatchment area in case there is sloughing of the slope. Toward the west end of the landslide, the bypass road is near grade. Because of the lack of a debris encatchment area, an encatchment area will need to be created. This can be achieved with the use of an encatchment berm. We will need to work with the Civil Engineer in providing such an encatchment area. Consideration of the impact of future soil movement to the placement of drainage structures at the toe of the landslide. The portion of the landslide beneath the planned filled for the bypass road will need to be removed as part of the grading. We estimate the maximum thickness of the landslide debris to be approximately 75 feet.

In all analyses, the factor of safety under seismic loading for the long term slopes was greater than 1.0. The results of the slope stability (static factor of safety without water, static factor of safety with water, and seismic factor of the safety) with the recommended slope inclination are presented in the table below. The table also indicated which slopes will require buttress fills.

TABLE 7.4 – 2
Recommended Cut Slope Inclination

Approximate Cut Location (Stations)	Recommended Slope Inclination (h:v)	Buttress Thickness (ft)	Notes
25+00 to 27+00	2:1	NA	
28+00 to 30+00	2:1	NA	
37+00 to 39+00	2:1	30	Buttress required on southern (north facing) slope only.
43+00 to 46+00	3.25:1	30	
46+00 to 47+50	2.5:1	30	Include debris encatchment, remove and replace landslide material beneath fill
52+00 to 56+00	2:1	30	

Notes: 1) Buttress thickness is measured perpendicular to slope
2) Fill slopes can be placed at 2:1 (horizontal to vertical)
3) All slopes will need to be assessed by an Engineering Geologist during grading
4) Stations based RBF Consulting, Aerial Map, 7/08/02, and are in meters

In addition to the above analyses, we also performed a slope stability analysis along Cross-section B-B' where the toe of the existing landslide is to removed for placement of the fill for the alignment. We have assumed a temporary cut of 1:1 (horizontal to vertical) for this analysis. The results of the analysis indicate that there is a static factor of safety below 1.0. Therefore, we are recommending that the temporary cut be at an inclination of 2:1. In addition, to further reduce the potential for movement of the landslide, it is recommended that lowest 30 feet of the cut be removed in 50 foot long sections and replaced with engineered fill prior to excavating other "lower 30 feet". It should be emphasized that the transition from the cut to the replacement fill will need to be completed on a continuous basis. The placement of fill should start the day following completion of each section of the excavation. Once a section is rebuilt, then a new section can be started. The width of the section should be a minimum of 50 feet wide. Even with these special measures, there is a probability that the landslide may move. This will result in additional material to be excavated and replaced.

As with any hillside development, some sloughing of new cuts and fills will occur that will require maintenance.

8 CONCLUSIONS AND RECOMMENDATIONS

Based on our investigation, we have identified a number of geologic, seismic, and geotechnical constraints and/or considerations at the project site. These constraints and/or considerations are typical for most roadway construction on hillsides in the San Francisco Bay Area and will need to be addressed during the design and construction of the Bypass. Presented below in separate subsections are the identified constraints and/or considerations for geologic, seismic, and geotechnical aspects of this project.

8.1 GEOLOGIC CONSIDERATIONS

The identified geologic constraints and/or considerations are as follows:

- Landslide Deposits
- Adverse Bedrock Bedding
- Colluvial and Slope Wash Deposits

A discussion of each one of these constraints/considerations is presented below.

8.1.1 Landslide Deposits

Numerous active and dormant landslide deposits have been mapped within the site area. The pertinent landslide deposits that underlie portions of the optional alignments have been discussed further in the Quaternary Surficial Deposits (Section 2.4) section of this report.

The landslide deposits underlying the alignment should be removed. Specifically, we identified that the toe of Landside 4 needs to be removed where located beneath the planned alignment. Generally we recommend that the landslide deposits underlying fill areas of the alignment be overexvacated and replaced as engineered fill, and that landslide deposits underlying cut areas of the alignment be evaluated in the field by our geologist during grading operations to determine if they need to be remediated after the cuts have been performed. Excavating of the landslide material at Landslide 4 will require special procedures as discussed in Section 7.4 "Slope Stability Results" of this report. In addition, buttressing or complete repair should be performed for landslide "5", which is located on a slope immediately down slope of the Bypass alignment. If that landslide is not remediated, then it is likely it will undermine the toe of the roadway fill. We do not anticipate that landslides not underlying the alignment or otherwise discussed in this report will impact the proposed roadway.

In some cases, collection areas (debris benches and basins) for landslide materials should be established between the edge of the slope and the roadway where complete removal of the landslide materials is not possible. This is further discussed in Section 7.1.4 Slope Stability Results of this report.

8.1.2 Adverse Bedrock Bedding

All the mapped bedrock formations mapped within this site dip northeastward. North/northeast facing cut slopes could undermine exposed bedding, creating a dip slope condition. Such a dip slope condition could render all proposed north/northeast-facing cut slopes as adverse since slope failures could occur. Cut slopes proposed either lower or higher than the alignment can potentially fail and adversely affect the alignment. The angle of the bedrock beds dip should be evaluated for all proposed cut slopes and where possible the angle of the slope face should not be steeper than that of the bedrock dip to prevent the beds from day lighting on the slope face. In some instances, where the bedding angle is considered adverse or the soil exposed is either weak or too sandy, it may be necessary to repair some of the cut slope faces and subdrain them. The slope that will require buttress fills are indicated in Section 7.1.4 Slope Stability Results of this report. In addition, cut and fill slopes should be observed by our Certified Engineering Geologist during grading to evaluate the exposed bedrock dip, the nature of the exposed materials, and how that relates to the proposed gradient. Additional remediation of the slope faces may be needed based on the results of the insitu conditons.

8.1.3 Colluvial and Slope Wash Deposits

A number of colluvial and slope wash deposits were mapped within the site area during this and our previous study. The approximate lateral extents and locations of the slope wash deposits are shown on Plates 2A and 2B. The colluvial deposits generally exist overlying the bedrock or firmer soil along the entire alignment, varying in thickness from about 2 feet to greater than 15 feet. These deposits were discussed further in Section 2.4 "Quaternary Surficial Deposits" of this report. The grading activities may remove the slope wash and colluvial deposits but additional excavations may be needed. Care should be taken not to create cuts in such deposits or place fills atop them which could cause them to fail causing fresh landslides. These deposits should be removed from fill areas and repaired (subexcavated and placed back as engineered fill) where exposed along proposed cut areas.

8.2 SEISMIC RELATED HAZARDS

The activity level of fault systems in the region places the project site, and all of the San Francisco Bay Area, in a seismically active setting. The major active faults in the region include the San Andreas (64 km/40 mi), Hayward (35 km/22 mi), Calaveras (20 km/12 mi), and San Gregorio (70 km/43 mi). Some of the closer active faults zoned by the CDG are the Greenville-Marsh Creek (5.6 km/3.5 mi) and the Concord (13 km/8.1 mi). In addition to these active faults, there are other faults (active and inactive) in the region that are not considered independent seismogenic sources such as the Antioch, Livermore, Pleasanton, Verona and Las Positas faults. However, no active fault traces or extensions zoned by the CGS are known to exist within the limits of the project site.

The primary seismic hazards that could affect the project site are related to strong earthquakes that will likely occur during the design life of the project. The potential earthquake related hazards to be considered include ground shaking, surface rupture, possible ground failure due to liquefaction, ground lurching or lateral spreading, and seismically-induced landsliding or settlement. In general, the seismicity of the San Francisco Bay Area and the nature of subsurface conditions at the site will combine to make certain earthquake hazards to the project are very real and require significant design considerations.

8.2.1 Ground Shaking

Historic earthquake records indicate that the site vicinity (along with the entire San Francisco Bay Area) is subject to strong ground shaking as a result of the relatively close proximity to the active faults in the region. The two major events affecting the San

Francisco Area occurred on October 17, 1989 (Loma Prieta earthquake, moment magnitude of 6.9) and the Great San Francisco earthquake of 1906 (moment magnitude 7.9).

The potential ground shaking intensity at the site is likely to be strong to very strong as a result of a major earthquake occurring on one of the nearby active faults in the region. Structures should be designed for the strong ground shaking anticipated, especially those structures which may be sensitive to strong ground shaking, particularly bridge or overpass structures. Factors that influence ground shaking intensity include distance to the earthquake epicenter, duration and magnitude of the event, and subsurface conditions at the site. In general, amplification of ground motions is not likely to be a significant factor for the project because the site is generally underlain by dense soil or shallow bedrock (compared to sites with soft sediments such as Bay Mud and/or loose fill).

8.2.2 Ground Surface Rupture

The absence of known active fault traces crossing through the project site results in very low potential for ground surface rupture to occur as a result of fault movements. It appears unlikely that fault rupture will occur directly across the proposed road alignments.

ENGEO, Inc. (1990) mapped an approximately north/south trending bedrock fault or shear zone that was shown on their site geologic map roughly parallel to the prominent drainage swale (along its western side) located to the west of the Thomas residence. One fault trench was performed during our field investigation to evaluate the potential for an active fault in this area. The results of our fault trench are shown on Plate B-13. No evidence of faulting was noted during our field investigation.

No features indicative of faulting were noted during our aerial photograph review and reconnaissance mapping, however. Additionally, authors such as Dibblee (1980) and Graymer and others (1994) did not map that fault or any other faults within the site area.

Based on that information, we conclude that the potential for fault-related surface rupture is low.

8.2.3 Liquefaction, Ground Lurching or Lateral Spreading

Liquefaction is a phenomenon in which saturated, granular, cohesionless soils lose strength because of build-up of excess pore water pressure under cyclic loading such as induced by earthquakes. Soils most susceptible to liquefaction are loose, clean, uniformly graded, fine-grained sands. Liquefaction can cause embankment displacement and/or building structural damage as a result of shallow foundation failures and/or large vertical and/or lateral displacements.

Bedrock units underlie the majority of the site area. Helley and Graymer (1997) mapped old alluvial fan deposits along the relatively flat area present along the northeastern portion of the site. Such deposits are more cemented and dense than the younger alluvial (Holocene) deposits. The Holocene alluvial deposits mapped around or within channels of prominent drainage swales are generally anticipated to be thin. Based on that information, we conclude that the potential for liquefaction at this site is low.

Ground lurching and lateral spreading of sloping ground surfaces (or towards areas of sloping ground) are related to liquefaction. Vertical and lateral ground movements due to this phenomena have been known to result in damage to near surface improvements such as pavements, infrastructure and flatwork. Based on our investigation of the site conditions, we judge that the risk associated with these types of ground failure at the site is considered relatively low. However, the Kirker Creek channel banks measure in excess of 12 meters (39 feet) in depth. Because this width is significant, ground lurching may occur. Therefore, the creek banks should be observed and mapped and their relative stability evaluated.

8.2.4 Seismically Induced Settlement

Rapid earthquake-induced densification has been known to result in settlement of loose granular fill soils above and below the water table at various sites, resulting in erratic ground settlements due to the variable nature of fill material types and densities. This could result in potential damage to infrastructure and surface improvements, and creation of voids beneath structures supported on deep foundations. However, based on our investigation of the site conditions, and the fact that proposed embankments will be constructed as "engineered fills", we judge that the risk of this type of ground failure at the site is relatively low.

8.2.5 Seismically Induced Landslides

The primary form of ground failure expected during strong earthquake shaking in the project area is seismically-induced landsliding. Areas which are susceptible to landsliding may experience slippage during earthquake ground shaking. The magnitude of seismically-induced landsliding will be influenced by the level of ground shaking and the amount of ground saturation from rainfall. Multiple pre-existing landslides have been identified at the site that impact the alignment. These landslides are described in additional detail in Section 2.4.3 "Landslides" of this report. It is uncertain whether or not any of these slides are directly related to seismic shaking.

In general, relatively few massive slope failures have occurred around the San Francisco Bay Area during earthquakes, but there have been some. On the other hand, many superficial (shallow) slides have been induced by seismic loading. The performance of earth slopes or embankments subjected to strong ground shaking is best measured in terms of deformation. The potential for seismically induced landslides at the site is considered to be low to moderate.

8.3 GEOTECHNICAL CONSIDERATIONS

The geotechnical considerations evaluated for this project are as follows:

- Rock Excavation and Bedrock Rippability
- Grading
- Settlement of Deep Fills
- Use of Tuffaceous Soil/Rock Materials
- Expansive Soils
- Vertical Loading on Culverts
- Erosion Control
- Foundation Considerations
- Soil Corrosion Potential

- Site Drainage and Groundwater Control

These items as discussed below.

8.3.1 Rippability Evaluation

Deep/large cuts are necessary for the Bypass alignment. These cuts could encounter resistant, strongly cemented bedrock or large isolated calcite-cemented concretions due to the relatively shallow depth to bedrock observed across the majority of the site. Therefore we performed seismic refraction analyses to assess rippabilty.

Rippability is the ability of materials to be excavated by ripper teeth mounted on standard earthmoving equipment, or manipulated without the need for explosive blasting. In evaluating the seismic-refraction velocities with respect to rippability, we used Caterpillar Tractor Company, Handbook of Rippability for heavy duty ripper performance. The table provided by Caterpillar Tractor Company is for a large track bulldozer with a single ripper hook attached. This rippability rating was used as an indicator of the relative difficulty anticipated in excavating rock at the selected sites along the Buchanan Road alignment and should be adjusted based on the equipment selected by the contractor for this project. For conditions where seismic-refraction data is within the rippable range, it should be expected that hard areas may be encountered.

Such hard areas include strongly cemented bedrock beds, welded tuff beds, and cemented sandstone concretions measuring up to 3.7 meters (12 feet) in diameter may be encountered during the grading operations. Such conditions may require large excavating conventional equipment or even blasting. The sandstone concretions and oversized bedrock fragments dislodged by conventional equipment may need to be mechanically broken down by hydraulic hammers, demolition balls or air operated jacks so that they are manageable to transport or for inclusion into deep fills.

It is important to note that the operator's experience, working condition of excavation equipment and the selection of excavation tools used will be critical factors in the excavatability of rock. During construction, modifications to tool selection or replacement of equipment being used may be necessary to improve performance and production rates. It is recommended that the contractors who use the rippability data in this report visit the proposed location of the roadway to observe bedrock conditions. It is recommended that the contractor have options available in order to deal with differing bedrock conditions. The results of our seismic refraction data are presented in Section 4.4 "Seismic Refraction Survey" of this report. The seismic velocity of the rock units were found to vary from about 3,000 to 5,200 feet per second.

The seismic velocities obtained during our seismic-refraction survey are necessarily averages across differing soil and rock conditions. Bedrock units can have coherent rock masses separated by discontinuities (fractures, bedding planes, joints, highly weathered zones, etc). This fact should be considered and allowed for when using seismic-refraction to estimate rippability. Seismic waves travel through coherent rock relatively fast and travel through intervening discontinuities relatively slow. The resulting seismic velocity through rock units will be the sum of velocities across coherent rock masses and discontinuities and will not be a true velocity through either rock type. As a result, variable rippability or excavation conditions both harder and softer can be encountered along the survey line.

8.3.2 Grading

A number of landslide deposits present along the optional Bypass alignments will generally be eliminated by the grading operations. All landslide material should be removed from the foundation or supporting area of the proposed embankment fills.

Inclinations for specific cut slopes are presented in Section 7.1.4 "Results of Slope Stability" of this report, and they vary from about 2:1 to 3.25:1. These inclinations represent the steepest allowable inclination for each final cut, however gentler inclinations may be used. Inclinations for other cut slopes should be no steeper than 2:1. Inclinations for fill slopes created by the proposed grading process should generally have inclinations of 2:1 (horizontal to vertical); although we should review all locations and heights of planned fills prior to finalizing drawings.

Cut slopes over 3 m (10 feet) in height should be observed by a Kleinfelder engineering geologist at the time of excavation to confirm that adverse geologic conditions are not exposed which might lead to future landsliding. Where adverse conditions are exposed, supplemental remedial measures should be performed which may include complete removal of the area or additional buttressing and subdrainage. Careful compaction control during grading operations to construct "engineered fill" embankments and/or replacement landslide backfill prisms should be maintained on all slope areas. Compaction along the face of the proposed fill slope may be necessary to retard shallow sloughing of materials. Fill slopes should be overbuilt and then trimmed back to reveal a well compacted slope face.

The excavated soil material generated during site grading is anticipated to consist primarily of clay from colluvial, slope wash and alluvial sources. Additionally, the excavated bedrock materials generated during grading are anticipated to consist of various mixtures of soils derived from the sedimentary sandstone (some are tuffaceous), siltstone, and claystone and volcanic tuff layers. Soil that contains more than 3% organic matter should not be used as engineered fill unless mixed together with on-site soils such that the organic percentage does not exceed 3 percent by weight, or may be stockpiled for use in landscape areas, if this is acceptable to the landscape architect.

Much of the excavated soil and rock materials along the Bypass alignment are expected to be generally suitable for reuse as common/general fill for roadway embankments. However the potentially expansive clay soils will likely not be suitable for reuse in the upper zone of fill prisms underlying flatwork and foundations. Depending on the size of rock fragments following excavation, ripped rock may also be considered unsuitable for reuse as common/general fill for embankments and underlying structures. This is primarily due to the potentially large size of strongly cemented sandstone concretions and ripped or shot rock that may be resistant to break down into smaller manageable-sized fragments or pieces for workability purposes and incorporation into fills.

If the project planning and design team intend to reuse excavated rock as fill, then provisions to mix/blend the rock fragments with soil material will need to be developed in order to avoid potential voids associated with nesting of rock fragments. Evaluation of suitability for incorporation into engineered fill embankments during mass grading should be performed during construction.

It is important to note that native in-place materials will experience shrinkage by approximately 5 to 10 percent in their volumes when they are subexcavated and placed in other areas as engineered fill. This estimate of shrinkage is highly variable and should be considered very rough.

The significant amount of site grading required to construct the Bypass roadway will also create a need for large quantities of construction-related water to moisture condition the fill materials and for dust control purposes.

8.3.3 Settlement and Movement of Deep Fills

Vertical and horizontal movement in deep clayey or sandy fills anticipated along portions of the project area is likely to occur. The settlement of cohesive clay soil consists of the sum of three components; (1) immediate settlement occurring as the load is applied, (2) consolidation settlement occurring gradually as excess pore pressures generated by loads are dissipated, and (3) secondary compression essentially controlled by the composition and structure of the soil skeleton. The settlement of coarse-grained granular soils subjected to loads occurs primarily from the compression of the soil skeleton due to rearrangement of particles. The permeability of coarse-grained soil is large enough to justify the assumption of immediate excess pore pressure dissipation upon application of load. Settlement of coarse-grained soil can also be induced by vibratory ground motion due to earthquakes, blasting, or by soaking and submergence.

Geotechnical literature has documented that substantial movements may occur in deep fill areas. It has been estimated by previous consultants (ENGEO, Inc., 1990) that 30-meter (100-foot) thick fills could undergo vertical settlements of 0.3 to 0.6 m (1 to 2 feet) over time. These amounts of settlement relate to about 1 to 2 percent of the height of the fill. In addition, where canyon areas are filled, the edges of the fill may undergo extension while the central portion of the fill may undergo horizontal compression. Furthermore, the Bypass alignment crosses five stream channels where soft, compressible materials could be present. The weight of the heavy fill embankment could cause the compression of these materials below the roadway if the weak materials are not entirely removed. The movements from any of these sources may cause damage to improved surfaces, buried pipes, or other developed facilities.

If the majority of settlement can be completed during the early stages of construction, before constructing more settlement-sensitive features (such as flatwork, foundations, pavements), then settlement generally will not contribute significantly to large scale distress.

Methods of reducing or accelerating settlement include removal or displacement of compressible material and pre-consolidation by surcharge loading in advance of final construction. For compressible foundation soils in deep fill areas, mitigation of soft underlying materials should be performed by complete removal (overexcavation). We estimate from the exploration performed at the site that the amount of removal of softer material is between 10 and 15 feet in depth. Additional excavated may be needed in localized areas. Consideration should also be given to allowing the fill placement to set dormant for a winter, which will enable some of the settlement to occur prior to placement of permanent improvements.

Effective implementation of these measures should reduce the potential for significant settling of deep fill areas to within tolerable limits. Regardless of which mitigation method is used, surface improvements and subsurface systems should be designed to accommodate 15 to 30 cm (6 to 12 inches) of settlement where fills in excess of 15 m (50 feet) are planned. Gradients for both surface and subsurface drainage should be carefully designed to accommodate such settlements. Planning provisions should also consider providing flexible couplings for buried utility pipelines at critical locations so that they are not disrupted by the anticipated settlement.

8.3.4 Use of Tuff/Tuffaceous Rock/Soil Materials as Fill

Some tuff/tuffaceous materials have been identified within the project area. A limited amount of tuffaceous material is likely to be excavated during the grading operations. Previous consultants (ENGEO, Inc., 1990) indicated that "these rocks may react with concrete and may also be compressible and expansive." If the tuff/tuffaceous materials are used in the bypass embankment, deterioration of concrete in contact with these materials could occur. In addition, it may be difficult to achieve proper compaction of these materials. In addition, some adverse swelling/expansion of the Tuff materials could occur over time if not mitigated in advance.

The tuff/tuffaceous materials should generally be placed in the lower portions of any proposed fill materials, but not within 10 feet of buried culverts. No tuff/tuffaceous materials should be allowed near the surface of any fill embankment. In addition, the material should generally be mixed with substantial amounts of other soil and rock materials to dilute its adverse properties. A ratio of about 1 of the tuff to 10 of other materials should be considered where located next to buried structures.

8.3.5 Expansive Soils

The surficial soils predominantly consist of clay and exhibit moderate to severe plasticity and expansion potential. That is, they tend to shrink and swell with fluctuations in moisture content. The potentially expansive clay subgrade soils could cause differential movements of pavement, flatwork and shallow foundations if not mitigated in advance. To mitigate this expansion potential, we judge that limited overexcavation and/or use of careful moisture conditioning and subgrade preparation in areas of these project features should be adequate.

This soil type is also sensitive to changes in moisture content that could result in "workability" problems if the soil is too wet (common during the rainy winter months). If the clay soil is too wet then earthwork activities to grade the site could become increasingly difficult with regard to excavation, placement, and compaction of general fill in accordance with project requirements. If site grading will be conducted during periods of (or following) wet weather, the grading contractor should anticipate these conditions. The preferred approach to grading would be to conduct site earthwork during drier weather periods when the surficial clay soils are sufficiently dry and firm.

Some vertical movement of exterior flatwork should be anticipated and will occur as a result of moisture content variations of the supporting soil below. Discussions of measures to reduce the impact of these expansive soils are presented below.

8.3.6 Vertical Loads on Culverts

The culverts selected should be capable of supporting vertical loads due to the soil overburden (trench backfill) and surcharge, including traffic loads. An in-place density of 130 pounds per cubic foot may be assumed for the trench backfill, and Marston's

Formula may be used (Marston and Anderson, 1913). The vertical pressure on the pipe due to an H-20 live load, as defined in the "American Iron and Steel Institute, Handbook of Steel Drainage and Highway Construction Products", may be taken as follows:

Height of Cover Over Pipe (feet)	Vertical Pressure On Pipe (psf)
1	1800
2	800
4	400
6	200
8	100
>8	Neglect live load

Additional surcharge loads on the pipe should be considered in the design if they are located above the pipe or within a distance of H away from the pipe, where H is the depth from the ground surface to the pipe invert. For these additional surcharges, the load on the pipe may be estimated using a load distribution slope of ½ to 1 (horizontal to vertical).

8.3.7 Erosion Control

Site soils are potentially subject to moderate to high rates of erosion, hence soil erosion on new cut or fill slopes could be significant if not mitigated in advance. This could be especially significant where grading activities take place and the finished slopes contain no vegetation. Such slopes may deposit sediment on the lower portions of the slope or on the Bypass roadway surface during heavy rainfall periods. This may result in clogging of drainage facilities and ponding of water or sediment on to the roadway surface.

Slopes that are anticipated to be susceptible to erosion by wind and rainfall should be protected. Protection is also necessary for slopes subjected to water flow action (undercutting) as in the creek banks. In some cases, provision must also be implemented against burrowing animals. Terracing and landscaping measures are commonly used in order to control erosion. Significant erosion still can occur on slopes that have inclinations of 3:1. Other experience indicates that slope gradient steeper than 2:1 do experience greater erosion than those that are 3:1 or flatter.

Drainage terraces at 9 to 15 m (30 to 50 foot) vertical intervals (depending on the final slope gradient) should be established to help control surface runoff even if slopes of 3:1 or flatter are utilized. All graded soil slopes should be planted with fast-growing, deep-rooted, low water tolerant vegetation to retard erosion. Other possible slope protection measures include a layer of rock or cobbles or other commercially available erosion control products. Protection from flowing water action in the creeks may be provided by hand placed rock riprap, concrete pavement, pre-cast concrete blocks, soil-cement, or other commercially available erosion control products as approved by the appropriate government agency.

8.3.8 Foundation Conditions

The planned soundwalls and other low retaining walls will likely be supported on spread footing foundations bearing on undisturbed native soils or bedrock, or on engineered fills. Some limited overexcavation of potentially expansive clay soils will be necessary below lightly loaded shallow spread footings to reduce the potential for differential movements due to heave as described above. It is anticipated that the soundwalls can be designed in accordance with Caltrans Standards.

8.4 EARTHWORK

8.4.1 General

All site preparation and grading should be performed in accordance with the recommendations contained in this report. We recommend that all earthwork be observed by Kleinfelder. Final depths of stripping, over-excavations, benching and keying should be assessed in the field by us at the time of grading.

After the rough grading phase, the surface of all building and garage pads should be observed and tested Kleinfelder. Rough grading plans and typical lot drainage details should be reviewed by us to assess conformance to the design recommendations prior to construction bidding.

In general, site preparation and grading should be performed in accordance with the site specific recommendations which follow. A brief summary of compaction recommendations is presented in Exhibit 1. Additional earthwork recommendations are presented in related sections of this report.

8.4.2 Site Clearing and Stripping

Prior to the start of site grading, all surface vegetation and other deleterious matter should be removed from areas to be graded. The depth of stripping is generally estimated to be on the order of 3 inches, but may extend deeper in some areas. Where existing trees are removed, their stumps and major root systems should also be excavated and disposed of off-site. The organic stripping materials that is free of organic particles larger than 1/4 inch in size may be reused in landscaped areas or used in deeper fills if mixed together with on-site soils such that the organic percentage does not exceed 3 percent by weight.

8.4.3 Removal of Existing Fill

Test pits were excavated during our field investigation at the site. These excavations were filled with loose backfill. Where planned grading will not remove this loose backfill, it is important that the backfill be removed in its entirety. The excavations should then be backfilled with engineered fill.

The areas to be graded should first be cleared of vegetation and stripped of the upper few inches of soil containing organic matter. The organic laden soil may be blended with other soils for use as engineered fill at a ration of 3 percent organic laden to non organic laden soil. The surfaces exposed by stripping and/or excavation should be scarified to a depth of at least 20 cm (8 inches), moisture conditioned to at least 2 to 4 percent over optimum moisture content prior to fill placement to close shrinkage cracks to their full depth, and compacted to at least 90 percent relative compaction. Following subgrade preparation, the ground surface should be kept moist to avoid excessive moisture loss. Extended presoaking or sprinkling may be necessary unless grading is performed after winter rains.

8.4.4 Excavation and Surface Preparation

In most areas, rough grading is expected to be accomplished with conventional grading equipment such as D-8 and D-10 bulldozers and scrapers. Because of the significant depth of cuts in various areas of the project, heavy ripping and extensive bulldozer work may be needed before cut material can be moved. Blasting is not anticipated. However, isolated hard concretions may be encountered during grading that may require more extensive mechanical equipment to break up. During rough grading, this will need to be evaluated in order to provide cost effective recommendations for installation of utilities.

After stripping and clearing operations have been completed to the satisfaction of Kleinfelder, areas to receive fill should be further prepared by excavating any weak, cracked, compressible, and creep-susceptible soils. Weak, compressible, creep susceptible soils are generally the near surface soils blanketing the natural slopes and include the colluvial deposits within swales, such as mapped on Plates 2A and 2B (Geologic Unit Qc). The depth of excavation is generally expected to range from about 2 feet to 15 feet, but may extend deeper in some areas. It should be anticipated that overexcavations to depths of about 10 to 15 feet in fill areas should be anticipated. Excavations will also be required to remove any existing undocumented fills, and to create keyways and benches prior to placement of fill where the area to be filled has an inclination of 5:1 (horizontal to vertical) or steeper. Recommendations for keyway and bench excavations are presented in Section 8.4.6 "Keyways and Benches" of this report.

After the stripping, clearing and over-excavation portions of the site preparation have been completed, the exposed ground surface in areas to be filled should be scarified to a depth of about 8 inches, moisture conditioned and compacted in accordance with the recommendations presented in Exhibit 1.

8.4.5 Engineered Fill

In general, the on-site soils with an organic content of less that 3 percent, by volume, and free of any deleterious materials or hazardous substances, may be used as engineered fill to achieve project grades. Ideally, expansive soils should not be used as fill within 5 feet of finished grade beneath pavements. However, due to the lack any significant amount of less expansive soils at the site, this is not practical unless the less expansive soils are imported. Therefore, development of the site will need to accommodate expansive soils at the surface.

Rocks or concrete fragments larger than 12 inches in maximum dimension should not be placed in fill areas within 10 feet of finished rough subgrade. Oversized rocks, larger than 12 inches, may be placed in the deeper fills provided additional effect is expended during placement to obtain dense, well-compacted fill all around the oversized material. Rocks larger than 12 inches in diameter should be spaced at least an equipment width apart to permit compaction of fill material completely around the oversized rocks. Fill with oversized rocks may require filtering to reduce the potential for soil migration. One or more filter layers may be required. Proposed locations and materials for rock fills should be reviewed by us during grading. Recommendations for filters should be made at that time.

In the event that "non-expansive" fill material is needed (such as beneath pavements), it should be primarily granular with a Plasticity Index of 15 or less, should contain 10 to 40 percent passing the number 200 sieve, and should not contain rocks or lumps larger than 3 inches in greatest dimension with no more that 10 percent of the material larger than 1 inch. All imported fill material should be sampled, tested, and approved by Kleinfelder prior to being transported to the site.

8.4.6 Keyways and Benches

Where fill is to be placed on hillside slopes steeper than 5 to 1 (horizontal to vertical), keyways and benches should be excavated into the exposed ground surface to provide support for the fill. The keyways and benches should extend into competent natural soil or bedrock. Typically, keyways should be excavated at least 5 feet into competent material and have a minimum width of 15 feet, 10 feet for debris benches. Typical keyway and benching details are presented on Plate 5. Horizontal benches should be excavated into competent materials typically at 5-foot vertical intervals as the fill placement progresses up the slope.

Subsurface drainage should be provided in keyways, on intermediate benches as appropriate, and in natural seepage areas and existing drainage courses to be filled. Recommendations regarding subsurface drainage are presented below in the "Drainage" section of this report.

8.4.7 Fill Placement and Compaction

Upon the completion of site clearing and excavation of weak soils, areas to receive fill should be scarified to a depth of about 8 inches, moisture conditioned and compacted according to the recommendations outlined in Exhibit 1. Any compressible material encountered in a cut area should be excavated to a firm and stable subgrade. Upon the completion of excavation or any required over-excavation in cut areas, the exposed ground surface should be scarified to a depth of about 8 inches, and then moisture conditioned in accordance with the recommendations presented in Exhibit 1. The excavation should then be backfilled with engineered fill.

The finished grading including subgrade preparation and placement of aggregate base within the future pavement areas should be performed in accordance with the City of Pittsburg specifications.

Grading operations during the wet season or in areas where the soils are saturated may require provisions for drying the soil prior to compaction. If the project necessitates fill placement and compaction in wet conditions, we can provide alternative recommendations for drying the soil. Conversely, additional moisture may be required during the dry months. Water trucks should be available in sufficient number to provide adequate water during compaction.

8.4.8 Trenches

Prior to the placement of underground utilities, the trenches should be checked for subsurface seepage. If seepage is encountered or suspected, we should be consulted so that recommendations for subsurface drainage can be provided. A subdrain system may need to be incorporated into some utility trenches. The subdrain should help reduce the potential for the build-up of hydrostatic pressure resulting from the collection of ground water in permeable pipe bedding backfill materials. Excavation for trenches in significant cut areas may be difficult; see preceding "Excavation and Subgrade Preparation" subsection of this report.

The utility pipes should be bedded and shaped in accordance with the requirements of the City of Pittsburg, or the particular utility company. If quarry certifications are not available, imported bedding and shading material should be sampled, tested, and accepted by Kleinfelder prior to being transported to the site.

Backfill above the shading may consist of on-site soils or imported granular backfill materials outside of street right-a-way. If imported sand is used for backfilling trenches, the sand should be capped with a minimum 12 inch thick layer of compacted soils similar to the adjoining soils at the sides of the excavation. Backfill material should be placed in lifts not exceeding 8 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting to attain compaction should not be permitted. Refer to Exhibit 1 for moisture conditioning and compaction recommendations for backfill materials.

Care should be taken during the placement and compaction of utility trench backfill materials in street pavement areas. Poor compaction may cause excessive settlements which can result in damage to the pavement structural section.

Utility trenches located in landscaped areas should also be capped with a minimum 12 inches compacted on-site clayey soils.

8.4.9 Irrigation Trenches or Backfill Slopes

Proper backfill of irrigation trenches located on or above slopes is very important. If the backfill is placed and left loose (uncompacted) the backfill will settle. Should this occur, the trench will act as both a surface and subsurface collector of water which is undesirable. This water could percolate into the slope weakening the soils and a local slope failure could result. Irrigation line trench backfill should be placed as an engineer fill. To reduce the potential of saturated slopes as a result of irrigation line installation, we recommend pressure testing of all lines both prior to and after completion of trench backfill placement.

All trenches should be excavated and shored in accordance with all applicable OSHA regulations.

8.5 CUT AND FILL SLOPES

8.5.1 Cut Slopes

Inclinations for specific cut slopes are presented in Section 7.1.4 "Results of Slope Stability" section of this report, and they vary from about 2:1 to 3.25:1. These inclinations are the steepest for each cut, and more gentler inclinations can be used. Inclinations for other cut slopes should be no steeper than 2:1.

Cut slopes greater than 30 feet in height should be constructed with 6-foot wide terrace drain benches (surface feature) at vertical increments of 30 feet or less, in accordance with Chapter 33 of the 1997 Edition of the Uniform Building Code. From a stability aspect, the benches can be eliminated if the slopes are no steeper than 3 to 1 and do not exceed 40 feet in height. Permanent cut slopes higher than 40 feet will require intermittent benches. The benches will also require periodic maintenance. Where compressible or creep susceptible material are encountered in a cut area or remain in-place due to relatively shallow excavation, over-excavation and reconstruction of the slope may be required. Cut slope areas where fragmented bedrock, out of slope bedding and/or unstable soils are encountered during construction will require the construction of earth buttresses, stability fills (slope reconstruction) or other remedial measures.

The general approach to reducing the potential for failures of cut slopes in weak materials is to remove the outer portion feet of material from the face of the slope and to reconstruct the slope with integral subsurface drainage and compacted fill. A detail of a typical buttress fill and a debris bench at a reconstructed cut slope area is presented on Plate 5. In general, all on-site soils with an organic content of less than 3 percent may be used in reconstruction of the slopes. The slopes should be reconstructed in accordance with the recommendations for fill slopes presented below.

All cut slopes should be observed by our Engineering Geologist and Geotechnical Engineer at the time of grading to assess the applicability of our recommendations and to make supplemental recommendations, if necessary. Supplemental recommendations may include slope flattening, installation of internal drainage, or slope reconstruction in areas where geologic weaknesses or local anomalies are encountered during site earthwork.

Where cut slopes extend higher than 30 feet, subdrains should be installed at the toe of the slope. The subdrains should extend to a depth of about 6 feet, at a distance of approximately 5 feet from the toe. Additional subdrains further up on the cut slope may be required.

Some of the cut slopes may encroach close to the existing electrical towers. Where the cut slopes (inclusive of the cut for a buttress fill, if needed) is closer than 40 feet horizontally, we should be contacted to evaluate the impact on the tower.

The tops of all cut slopes should be rounded to reduce sloughing and erosion of the top of the cut.

8.5.2 Fill Slopes

Fill slopes of up to about 100 feet in height are planned. These slopes should be constructed at gradients no steeper than 2 to 1 (horizontal to vertical). The fill material must be compacted to the face of the slopes. To accomplish this, we recommend that slopes be over-built a minimum of three feet horizontally and then trimmed to design grades. Other methods may also provide the desired compaction. Proposed alternative methods should be submitted to us for review. Although deep-seated failures should not occur in properly compacted fill slopes, even properly designed and constructed fill slopes have a potential for shallow failures or surficial sloughage, particularly during periods of wet weather. All slopes should be constructed with drainage and terraced in accordance with Chapter 33 of the 1997 Edition of the Uniform Building Code.

All fill slopes should be keyed and benched into the foundation and back slope soils as shown on Plate 5. Vertical distance between benches should not exceed 5 feet. All keyways and intermediate benches on fill slopes should be drained. Additional subsurface drainage will be required in areas where seepage is evident or suspected and in areas where surface drainage exists. Subdrains should be constructed in accordance with the recommendations presented in Section 8.6, "Drainage", below.

The toe of fill slopes may encroach close to the existing electrical towers. If the toe of the fill is closer than 40 feet horizontally, then we should review the impact of the fill on the tower.

8.5.3 Fill Over Cut Slopes

Although not anticipated, special mitigation may be needed if fill is place above the top of planned cuts. The condition may overload the cut slope and create instability. Special mitigation, such as a set-back of the toe of the slope, change in inclination of the cut or fill or both, installation of a keyway at the toe of the fill, and other methods may be needed. If this condition is required by the grading scheme, we should be contact during design to evaluate whether special conditions are needed.

8.6 DRAINAGE

This section discusses subsurface drainage, surface drainage and erosion control, and near-surface seepage control.

8.6.1 Subsurface Drainage

We recommend that subsurface drainage be provided in the natural drainage areas to be filled, in major keyway excavations, in areas of observed or suspected seepage, and in areas of slope repair or construction. Detailed recommendations for locations of subdrains can be provided during remedial grading design when grading plans are finalized. The need for additional subdrains may be required and should be evaluated by our Engineering Geologist and Geotechnical Engineer during the grading of the site.

Although we do recommend general locations for subsurface drains in this report and the need for additional subdrains will likely be identified and addressed during mass grading, the potential for new springs and seeps to develop after completion of site development remains one potential source of this water is excess landscape water.

Landscape water features should be constructed with a double liner and integral subsurface drainage.

The drainage systems should consist of a combination of drainage blankets or interceptors consisting of perforated pipe surrounded by drain rock wrapped in filter fabric. Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68) may be an acceptable alternative to drain rock wrapped in filter fabric in areas were the volume of seepage is anticipated to be low.

The subdrains should be installed at the back of the key or bench and the base of the keys or benches should have a minimum slope of 2 percent towards the subdrain. Clean-outs should be installed on all main line subsurface drains. The perforated pipe should be installed with the perforations down and should be connected to a closed pipe leading to a suitable discharge point such as the storm system. Typical installation details are presented on Plates 4, 5 and 6.

Subdrain pipes should consist of either ABS (SDR-23.5), or PVC (Schedule 40 minimum) pipe meeting Caltrans and ASTM standards. Lateral drains should have a minimum diameter of four inches. Laterals should be connected to a main line with a minimum diameter of six inches. The actual locations, depth and extent of the subsurface drainage systems should be assessed by us in the field at the time of construction.

8.6.2 Surface Drainage and Erosion Control

Good surface drainage is essential to intercept and control surface water runoff to reduce slope erosion and subsurface infiltration. Effective erosion-control landscaping is also important. Measures to control surface water and erosion include placement of berms within cut and fill slopes, construction of berms at the top of slopes, installation of both soil and concrete V-ditches, landscaping of slopes, and control of watering on slopes. These items are discussed below.

V-ditches should be constructed along the toe of all slopes greater than 5 feet in height, including slopes above retaining walls, to direct all surface water to a suitable discharge. In lieu of the V-ditches, contouring the grade at the bottom of the slope is permissible to drain surface water into area drains.

Concentrated water should not be allowed to flow uncontained across any slope face. Areas above slopes should be graded to a 2 percent gradient or greater to direct surface water away from the top of the slopes and toward a suitable point of discharge such as erosion controlled ditches or closed pipes. In addition to grading areas above slopes to drain away from the slopes, berms may be constructed at the top of all slopes. Measures approved by local agencies should be constructed to reduce siltation during construction. Erosion controlled V-ditches, brow ditches, or intermediate benches, should be constructed above all slopes which have tributary areas. Lined ditches and temporary silt fences should also be considered at the toe of both cut and fill slopes equal to or greater than 20 feet in total height.

Where the roadway is located at the toe of slopes greater than 15 feet in height, area drains at the toe of the slopes should be considered to reduce the accumulate excess surface water during the rain season. These drains should be directed to a storm drainage system.

Slopes should be planted with appropriate ground cover vegetation to reduce the potential for future erosion and possible slope deterioration. Planting should occur sometime prior to the start of the rainy season. As a minimum, the planting should consist of hydroseeding. The type and time of placement of the hydroseeding should be shown on the erosion control plan. Generally, the hydroseeding should be placed such that the slopes are exposed to rain water before the seeds are allowed to dry, but before the start of the more substantial winter rains. Additional hydroseeding may be needed if the initial planting is partially or totally unsuccessful. This should be evaluated by professional landscaper. Other types of planting may be used. Deep root plants should be considered, as well as concentrated watering, such as a drip system. Even with planting, areas of erosion should be anticipated. In lieu of plating, other mitigating measures such as temporary silt fences might be necessary, depending upon the susceptibility of the exposed materials to erosion.

We recommend that irrigation of all slopes be limited. Overwatering of slope surfaces could result in surficial instability and downward creep of the shallow earth materials soils. For slopes that must be irrigated we recommend the use of a low volume system such as drip irrigation.

8.6.3 Near-Surface Seepage Control

Where pavements abut against landscaped areas, some method should be used to protect the aggregate base layer and subgrade soils against saturation from water in the landscaped areas. If landscape water or surface runoff is allowed to seep into the pavement section, the service life of the pavement may be reduced. Methods of reducing seepage into the pavement sections may include vertical curbs extending at least 2 inches below the base rock/subgrade interface, or subdrains behind or below curbs in landscape areas. Also, care should be taken to prevent over-watering of landscaped areas adjacent to pavements.

Vertical cut-offs, such as a deepened curb section or equivalent extending at least 2 inches below the subgrade/base rock interface, will help reduce the amount of lateral seepage under pavements or slabs from adjacent landscaped areas. Cut-offs must be carefully constructed such that they extend below the aggregate base section and are cast neat against undisturbed native soil or compacted clayey fill. The cut-offs should be continuous and any utility trenches (irrigation lines, electrical conduit, etc.) that extend through, or under the curbs, should be sealed with compacted clayey soil or cast in-place concrete.

Alternatively, seepage can be reduced by installing subdrains behind or below curbs. Typically, subdrains consist of 3 to 4-inch diameter perforated pipes installed in one-foot wide trenches surrounded by Class 2 Permeable Material (State of California Standard Specifications, Section 68). The top of the subdrain pipe should be positioned at least 2 inches below the aggregate base/subgrade contact. The subdrain should be sloped to drain to a closed pipe under pavements. Impervious soil or concrete should be placed around the pipe at the transition between the perforated and closed sections of pipe. The impervious plug should fill the trench to help reduce the potential for water from the subdrain system seeping through the trench backfill in the pavement areas.

Where utility lines extend through or beneath curbs adjacent to pavement areas, permeable backfill should be terminated at least one-foot from the footing or curb. Compacted clayey soils should be used around the pipe to act as a seepage cutoff. This will help reduce the amount of water seeping through the previous trench backfill and collecting under the building or pavements.

8.7 SITE MAINTENANCE

An effective program of post-construction maintenance is essential to reduce the potential for deterioration of the site. Drainage inlets and ditches, particularly in gully areas and on slopes, should be periodically inspected and cleaned of debris and sediments that could restrict the flow of runoff and seepage. Also, any detected cracks in lined ditches should be promptly sealed or patched.

Slopes should be monitored on a regular basis (at least annually) for signs of instability or seepage. Prompt attention to local instability or seepage is strongly recommended. Where seeps are observed, we should be consulted to assess the need for the installation of additional subsurface drainage.

Irrigation of the site should be carefully controlled, and over-irrigation practices should be avoided. A program of detection and prompt repair of underground pipe leaks, including the irrigation systems, should be implemented. In the event that irrigation line trench backfill where trenches are located on a slope, the surface of the slope should be restored to maintain surface drainage. If depressions are left on slope, they may collect water which could result in a localized slope failure.

In any hillside development, it is impossible to detect and mitigate all potential slope or soil-related problems during design and construction. The risk can be reduced by providing proper drainage and slope maintenance. Stringent rules and approval procedures for altering the existing slopes or developing the areas above the slopes should be established. Site maintenance should include the prompt detection and correction of any such problems. Professional consultation should be obtained as appropriate.

8.8 CORROSION POTENTIAL

Four soil samples were submitted to Cerco Analytical of Pleasanton, California for corrosion potential analysis. The test program included measurement of pH, soluble sulfate and chloride content. CERCO Analytical's report and results are presented in Appendix F. Based upon the resistivity measurements, three samples are classified as "corrosive" and one sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion. Since we are not corrosion specialists, a corrosion testing firm should be contacted for specific design details, if necessary.

A more detailed investigation may include more or fewer concerns and should be directed by a corrosion expert. Soils actually in contact with concrete should be sampled and tested for sulfate content during construction and the concrete mixes used should comply with the requirements of the 2007 CBC based on these results. Consideration should also be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of the investigation.

As an alternative or in addition to meeting CBC mix requirements, your Structural Engineer, architect or corrosion expert may choose to isolate the concrete from the corrosive soils or from ground or surface water that may leach corrosive materials from the soils and contact the concrete. These alternatives may include waterproofing, capillary breaks, vapor barriers and removal of corrosive soils.

8.9 PAVEMENTS

Pavements for this project will consist of roadways. We have calculated our pavement designs assuming the pavement subgrade soil will be similar to the soils located about 3 to 8 feet below existing ground surface described in the boring logs. If site grading exposes soil other than that assumed, or import fill is used to construct pavement subgrades, we should perform additional tests to confirm or revise the recommended pavement sections for actual field conditions. This may result in a savings in construction costs.

To date, no Traffic Index requirements have been provided to us. We have assumed Traffic Indices between 4 and 10 to cover the improvements. The appropriate traffic Index used for each area should be verified by the project Civil Engineer.

Four bulk samples of the topsoil from about 5 to 8 feet below ground surface and one bulk sample of re-worked Tulare formation claystone was obtained from the site. The samples were tested to assess the Resistance (R) value of the soil for use in the design of flexible pavements. The results of the tests indicated an R-value of less than 5. For design purposes, we have based our calculations on an R-value of 5 and used the Caltrans Flexible Pavement Design with a factor of safety of 0.2 feet. Two alternative pavement sections are presented for different Traffic Indices (T.I.) including an asphalt concrete over aggregate baserock design, and full depth asphalt concrete. Each T.I. represents a different level of use. The owner or designer should determine which level of use best reflects the project and select appropriate pavement sections.

RECOMMENDED PAVEMENT SECTIONS
ASPHALT CONCRETE PAVEMENT DESIGN (R-Value = 5)

T.I.	Alternative 1		Alternative 2
	AC	AB	Full Depth AC
4	2.5	7.5	7.0
5	2.5	11.0	8.5
6	3.0	13.5	9.0
7	4.0	15.5	11.5
8	4.5	18.5	12.5
9	5.5	20.5	14.0
10	6.0	23.5	16.0

NOTES: THICKNESSES SHOWN ARE IN INCHES

AC=Type A asphalt concrete in roadways and truck lanes, else use Type B
AB= Class II aggregate base (minimum R-value = 78)

We recommend that the subgrade soil over which the pavement sections are to be placed be moisture conditioned and compacted according to the recommendations in Exhibit 1. Subgrade preparation should extend a minimum of 2 feet laterally beyond the face of curbs.

Parking areas should be sloped and drainage gradients maintained to carry all surface water off the site. Surface water ponding should not be allowed anywhere on the site during or after construction. We recommend that the pavement section be isolated from vacant lots or intrusion of irrigation water from landscaped areas at all locations at which pavements are adjacent to landscaped areas. Curbs should extend a minimum of 2 inches below the baserock and into the subgrade to provide a barrier against migration of landscape water into the pavement section. For long-term performance, a subdrain system should be constructed behind the curbs at the interface of pavement and landscaping to collect excessive water from landscaping irrigation and to convey it to a storm drain or other drainage facility. As an alternative to subdrains, but not as effective, weep holes can be installed in the curbs at 4 feet on-center. The success of this alternative will largely depend on the ability of the contractor to properly perform the work.

In addition, we recommend that all pavements conform to the following criteria:

- All trench backfills, including utility and sprinkler lines, should be properly placed and adequately compacted to provide a stable subgrade.
- An adequate drainage system should be provided to prevent surface water or subsurface seepage from saturating the subgrade soil.
- The aggregate base and asphalt concrete materials should conform to ASTM test procedures and work should be performed in accordance with Caltrans Standard Specifications, latest edition.

8.10 SEISMIC DESIGN CRITERIA

On July 1, 2007, the new 2007 CBC was adopted and implementation began after January 1, 2008. According to the 2007 CBC, the mapped spectral response accelerations S_S and S_1 for the site are 1.50g and 0.57g, respectively, which were obtained based on the Java ground motion parameter calculator developed by the U.S. Geological Survey (USGS, 2007) using ASCE 7. The Site Class type can be classified as C, which is defined as a very dense soil and soft rock profile per Table 1613A.5.2 of the 2007 CBC. The site coefficients Fa and Fv are 1 and 1.3, respectively, per Tables 1613A.5.3(1) and 1613A.5.3(2) of the 2007 CBC.

Note that S_S and S_1 are based on Site Class type B (defined as a rock soil profile) and therefore need to be multiplied by the site coefficients Fa and Fv to obtain the maximum considered earthquake spectral response accelerations SM_S and SM_1, respectively. The design spectral response accelerations SD_S and SD_1 are computed by multiplying SM_S and SM_1, respectively, by 2/3.

9 ADDITIONAL SERVICES

Kleinfelder's current scope of services does not include development of remedial grading plans or hidden quantities analyses. If needed at a later date, we would be pleased to provide a scope of services and cost estimate for these tasks.

In addition, the review of plans and specifications and field observation and testing by Kleinfelder of earthwork related construction activities are an integral part of the conclusions and recommendations made in this report. It is recommended that Kleinfelder be present at the pre-bid meeting with the prospective grading contractors to clarify any issues and to address any questions regarding the recommendations presented in this report. If Kleinfelder is not retained for these services, the client will be assuming Kleinfelder's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein. The recommended tests, observations, and consultation by Kleinfelder prior to and during construction include, but are not limited to:

- Review of plans and specifications;
- Observations of all earthwork from site clearing and stripping through final grading and utility trench backfill;
- Observation of foundation excavations and foundation construction; and
- Construction observation and in-place density testing of fills, backfills; and finished subgrades.

10 LIMITATIONS

Our limited study specifically excluded any environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands. The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been performed according to generally accepted geotechnical engineering practices that existed in the San Francisco Bay Area at the time this report was written. No warranty is expressed or implied. This report is issued with the understanding that the owner chooses the risk he wishes to bear by the expenditures involved with the construction alternatives and scheduling that is chosen.

The conclusions and recommendations provided herein are for the subject roadway Bypass project described in this report. These conclusions and recommendations should be used for design of final grading plans. Further investigations for remedial grading plans will be required.

The conclusions recommendations presented in this report are based on information obtained from the following:

- Review of published and unpublished maps and data,
- 8 exploratory soil borings,
- 4 rock cores,
- 19 test pits,
- 1 fault trench,
- The observations of our engineering geologist and geotechnical engineer at the site during our reconnaissance,
- Laboratory testing,
- Slope stability analyses of five cross sections, and
- Our experience in the area.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on-site and off-site) or other factors may change over time, and additional work may be required.

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12 EXHIBIT 1 - SUMMARY OF COMPACTION RECOMMENDATIONS

Area	Compaction Recommendations (See Notes 1 through 6)
Subgrade Preparation and Placement of General Engineered Fill, Including Imported "Non-expansive" Fill	Compact upper 12 inches of subgrade and entire fill to a minimum of 90 percent compaction at an over optimum moisture content for granular soils, and to 90 percent compaction at a minimum of 2 percent over optimum moisture content for on-site clayey soils.
Trenches [6]	Compact trench backfill to a minimum of 90 percent compaction at an over optimum moisture content for granular soils, and to 90 percent compaction at a minimum of 2 percent over optimum moisture content for on-site clayey soils. Where trenches will be under flatwork or paving, the upper 8 inches should be compacted as recommended below.
Exterior Flatwork	Compact upper 8 inches of subgrade to a minimum of 90 percent compaction at an over optimum moisture content for granular soils and "non-expansive" fill, and to between 88 to 92 percent compaction at a minimum of 2 percent over optimum moisture content for on-site clayey soils. Compact baserock to a minimum of 90 percent compaction at an over optimum moisture content. Where exterior flatwork is exposed to vehicular traffic, compact baserock to a minimum of 95 percent compaction.
Roadways	Compact upper 8 inches of subgrade to a minimum of 92 percent compaction at above optimum moisture content. Compact baserock to a minimum of 95 percent compaction at an over optimum moisture content.

<u>Notes</u>:

(1) Depths are below finished subgrade elevation.

(2) All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D1557 (latest version). All lifts to be compacted shall be a maximum of 8 inches loose thickness.

(3) All compacted surfaces, such as fills, subgrades, and backfills need to be firm and stable, and should be unyielding under compaction equipment.

(4) Includes building pads.

(5) Where fills are deeper than 7 feet, the portion below 7 feet should be compacted to a minimum of 95 percent.

(6) In landscaping areas, this percent compaction in trenches may be reduced to 85 percent.

PLATES

EXPLANATION

APPROXIMATE GEOLOGIC CONTACT - dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SLOPE WASH (arrows indicate direction of movement)

ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)

DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

1 — 1 of 5 LANDSLIDES DISCUSSED IN REPORT

Qaf	ARTIFICIAL FILL
Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)

B B' — SLOPE STABILTY CROSS SECTION B-B' (SECTIONS A & C NOT USED IN STABILITY ANALYSIS THERFORE ARE NOT SHOWN)

SOIL BORING

ROCK CORE

TEST PIT OR EXPLORATORY FAULT TRENCH

SEISMIC TRAVERSE

NOTE:
CONTOUR INTERVAL IS IN METERS.

N

0 200 400 600 FEET

0 50 100 150 200 METERS

REFERENCE:
RBF Consulting, "Final Central Alignmant, dated 07/09/07, Aerial Topographic Map," dated 07/08/02. Plan reflects proposed grading.

DRAWN BY: JDS/LGS	EXPLORATORY SAMPLE LOCATIONS: WEST	KLEINFELDER	PLATE
REVISED BY:		7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566	2A
CHECKED BY: LS	BUCHANAN ROAD PITTSBURG, CALIFORNIA	PH. (925) 484-1700 FAX. (925) 484-5838	
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856-PWGEO / FILE NAME: KA_ALIGN.dwg	www.kleinfelder.com	

PLOTTED: 11 Jan 2008, 3:45pm, Jsala

EXPLANATION

	APPROXIMATE GEOLOGIC CONTACT - dotted where buried
	CREEPING COLLUVIUM
	INCISED EROSION RILLING OR SHALLOW SLUMP
	SLOPE WASH (arrows indicate direction of movement)
	ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)
	DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)
1	1 of 5 LANDSLIDES DISCUSSED IN REPORT

Qaf	ARTIFICIAL FILL
Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)

B B'	SLOPE STABILITY CROSS SECTION B-B' (SECTIONS A & C NOT USED IN STABILITY ANALYSIS THERFORE ARE NOT SHOWN)
	SOIL BORING
	ROCK CORE
	TEST PIT OR EXPLORATORY FAULT TRENCH
	SEISMIC TRAVERSE

NOTE:
CONTOUR INTERVAL IS IN METERS.

N

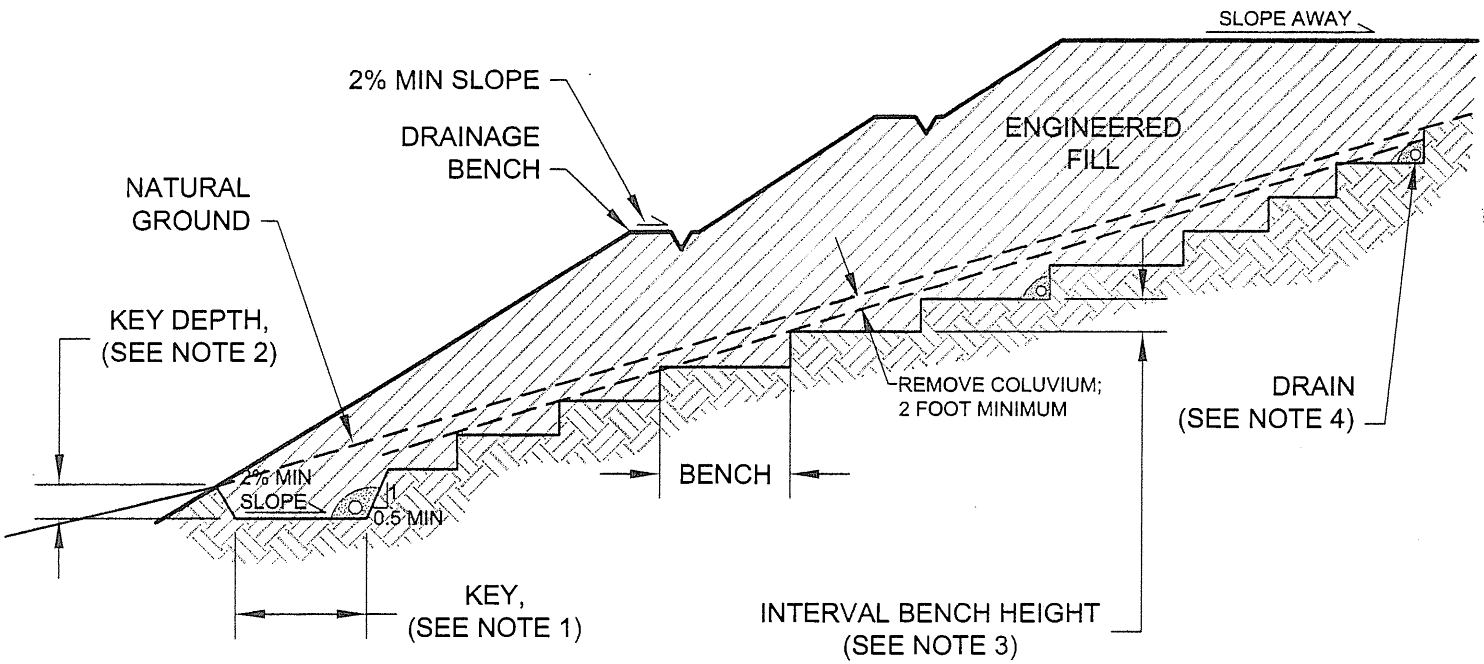
0 200 400 600 FEET
0 50 100 150 200 METERS

REFERENCE:
RBF Consulting, "Final Central Alignmant, dated 07/09/07, Aerial Topographic Map," dated 07/08/02. Plan reflects proposed grading.

DRAWN BY: JDS/LGS REVISED BY: CHECKED BY: LS DATE: JAN 2008 / APPROVED BY:	EXPLORATORY SAMPLE LOCATIONS: EAST BUCHANAN ROAD PITTSBURG, CALIFORNIA PROJECT NO. 75856-PWGEO / FILE NAME: KA_ALIGN.dwg	KLEINFELDER 7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	PLATE 2B

PLOTTED: 11 Jan 2008, 3:42pm, jsala

ATTACHED IMAGES: Images: SWALE.jpg Images: Topo with road.jpg
ATTACHED XREFS: XRef: Style A_08x11
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: FILL SLOPE



NOTES:

1. KEYWAY WIDTH: 1/2 VERTICAL HEIGHT OF FILL; 10' MINIMUM., 40' MAXIMUM.

2. KEYWAY DEPTH: MINIMUM 3 FEET INTO COMPETENT SOIL OR BEDROCK, MIMIMUM 10 FEET TOTAL DEPTH.

3. INTERVAL BENCHES: MINIMUM 2-FOOT WIDTH AND HEIGHTH; MAXIMUM 5-FOOT WIDTH AND HEIGHT.

4. SEE ATTACHED DRAIN DETAIL. LOCATION OF DRAINS WILL VARY. ONE DRAIN AT KEYWAY AND, AS A MINIMUM, ONE EVERY 25 FEET OF VERTICAL HEIGHT.

5. DRAINAGE BENCHES: WIDTH AND LOCATIONS BASED ON LOCAL CODES. STANDARD SPACING IS APPROXIMATELY 25 VERTICAL FEET; FOR SLOPES WITH ONLY ONE BENCH, THE BENCH SHALL BE AT MID-HEIGHT. SLOPES OVER 30' HIGH MUST HAVE 6'-WIDE DRAINAGE TERRACES ON 25' VERTICAL INTERVALS IN ACCORDANCE WITH CONTRA COSTA COUNTY GRADING ORDINANCE.

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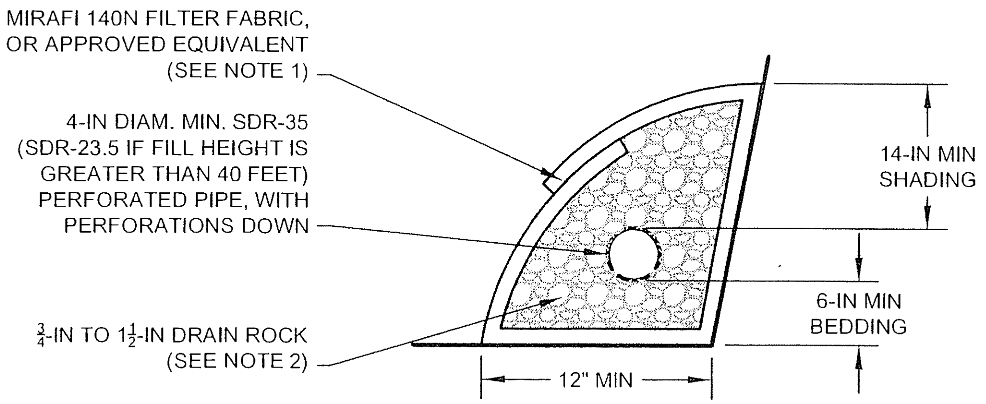
TYPICAL FILL SLOPE DETAIL

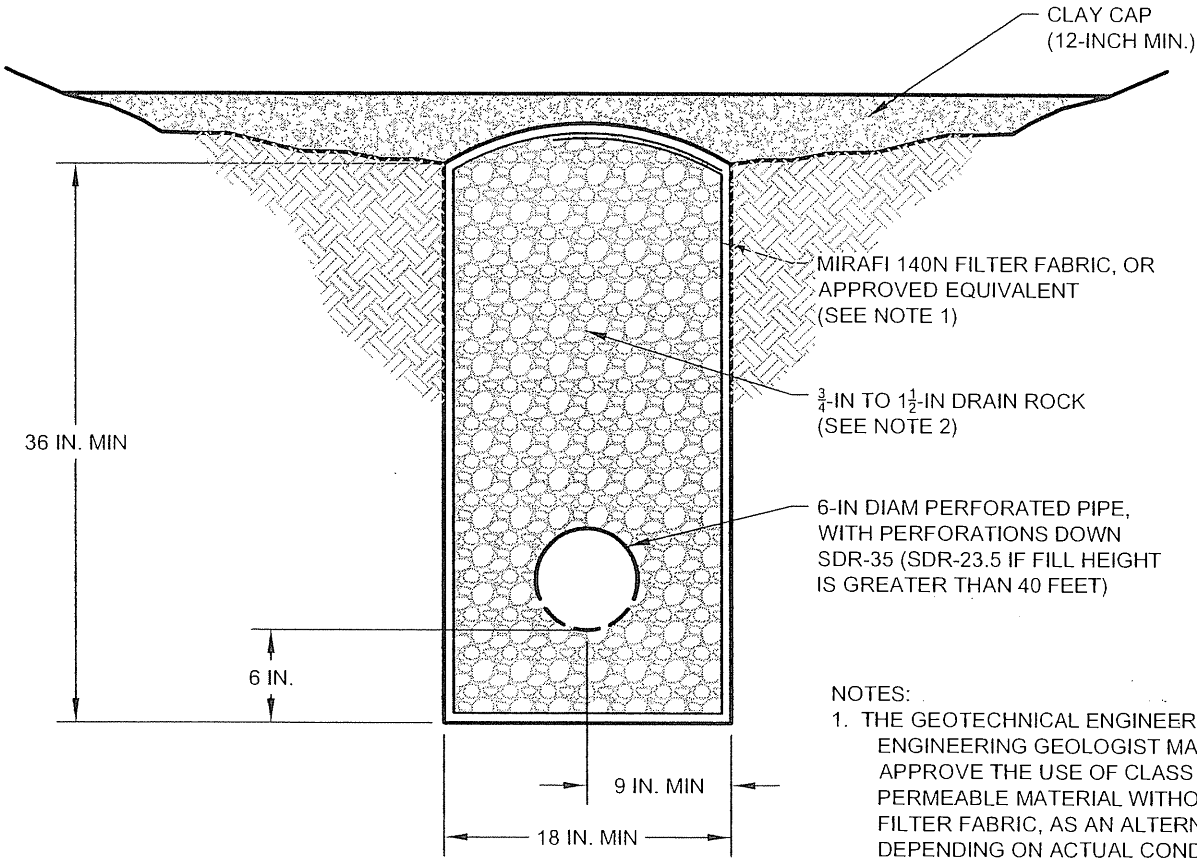
BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

DRAWN BY:	JDS
REVISED BY:	
CHECKED BY:	LS
PLATE	3

DRAWN: JAN 2008 | APPROVED BY: | PROJECT NO. 75856 | FILE NAME: SBDRNS.dwg



TYPICAL KEYWAY AND BENCH SUBDRAIN



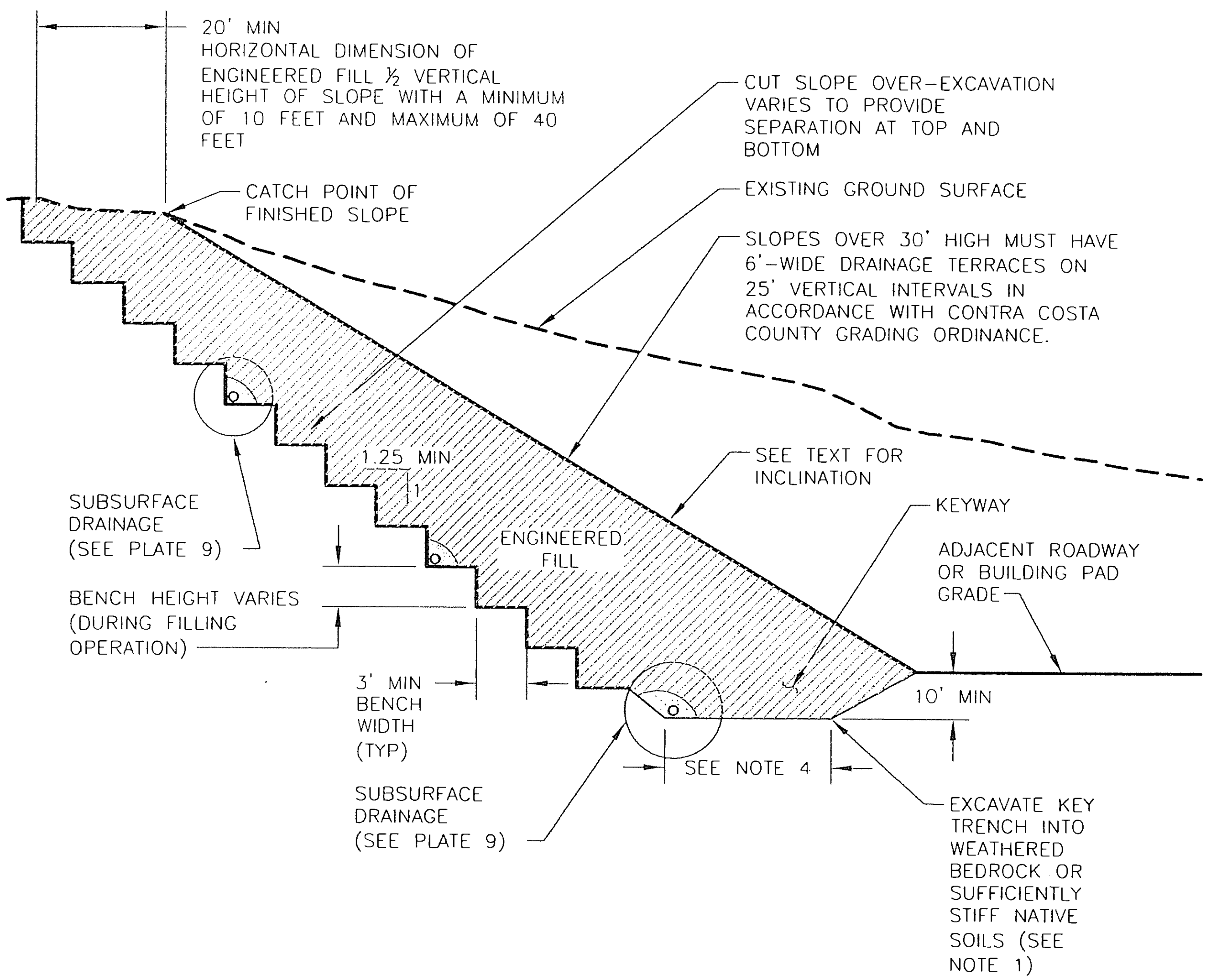
NOTES:

1. THE GEOTECHNICAL ENGINEER/ ENGINEERING GEOLOGIST MAY APPROVE THE USE OF CLASS 2 PERMEABLE MATERIAL WITHOUT FILTER FABRIC, AS AN ALTERNATIVE, DEPENDING ON ACTUAL CONDITIONS ENCOUNTERED.
2. DRAIN ROCK SHOULD BE CLEAN AND FREE DRAINING CALTRANS CLASS 1 TYPE B.
3. DRAWING IS NOT TO SCALE.

TYPICAL TRENCH SUBDRAIN

KLEINFELDER 981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	TYPICAL SUBDRAIN DETAILS	DRAWN BY: JDS REVISED BY: CHECKED BY: LS
	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	PLATE 4
DRAWN: JAN 2008 | APPROVED BY:	PROJECT NO. 75856 | FILE NAME:SBDRNS.dwg	

ATTACHED IMAGES: Images: SWALE.jpg Images: Topo with road.jpg
ATTACHED XREFS: XRef: Style A_08x11
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: SUBDRAN-TRENCH



NOTES:
1. FINAL BENCHING AND KEYWAY EXCAVATION DEPTHS AND DETAILS SUBJECT TO EVALUATION OF FIELD CONDITIONS.

2. DIMENSIONS AND LOCATIONS OF SUBDRAINAGE ARE APPROXIMATE. ACTUAL INSTALLATION TO BE BASED ON FIELD CONDITIONS.

3. GRADE PIPES TO DRAIN TO AN APPROPRIATE DISCHARGE FACILITY.

4. KEYWAY WIDTH = ½ OF VERTICAL HEIGHT WITH MIMIMUM OF 10 FEET AND MAXIMUM OF 40 FEET

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981 Garcia Avenue, Suite A
Pittsburg, CA 94565
PH. (925) 427-6477 FAX. (925) 427-6478
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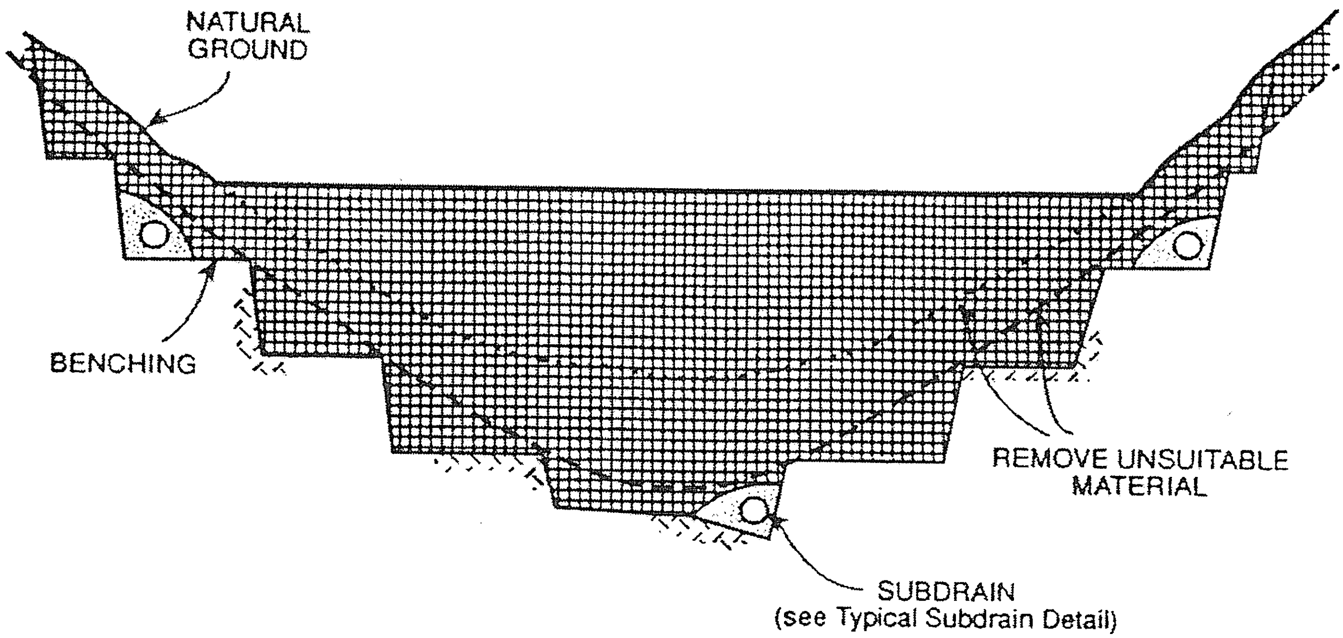
TYPICAL KEYWAY AND BUTTRESS FILL

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

DRAWN: JAN 2008	APPROVED BY:	PROJECT NO. 75856	FILE NAME: KEYWAY.dwg

DRAWN BY: JDS
REVISED BY:
CHECKED BY: LS
PLATE 5

ATTACHED IMAGES:
ATTACHED XREFS: XRef: Style A_08x11
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: Layout1



DRAWING IS NOT TO SCALE.

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SWALE SUBDRAIN DETAIL

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

DRAWN BY:	JDS
REVISED BY:	
CHECKED BY:	LS
PLATE	6
DRAWN:	JAN 2008
APPROVED BY:	
PROJECT NO.	75856
FILE NAME:	SBDRNS.dwg

ATTACHED IMAGES: Images: SWALE.jpg Images: Topo with road.jpg
ATTACHED XREFS: XRef: Style A_08x11
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: SWALE

APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW	Well-graded gravels or gravel with sand, little or no fines
		GP	Poorly-graded gravels or gravel with sand, little or no fines
		GM	Silty gravels, silty gravel with sand mixture
		GC	Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sand
		SC	Clayey sand
FINE GRAINED SOILS	SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL	Inorganic lean clays of low to medium plasticity, gravelly clays sandy clays, silty clays.
		OL	Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH	Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH	Inorganic fat clays (high plasticity).
		OH	Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

ROCK CLASSIFICATION

ROCK TYPE
BRECCIA
CLAYSTONE
CONGLOMERATE
SILTSTONE
SHALE
SANDSTONE

Physical Properties Criteria for Rock Descriptions

FRACTURE SPACING

VERY WIDELY FRACTURED	Greater than 6.0 feet
WIDELY FRACTURED	2.0 to 6.0 feet
MODERATELY FRACTURED	8.0 inches to 2.0 feet
CLOSELY FRACTURED	2.5 to 8.0 inches
INTENSELY FRACTURED	0.75 to 2.5 inches
CRUSHED	Less than 0.75 inches

BEDDING OR LAYERING

VERY THICK OR MASSIVE	Greater than 4.0 feet
THICK	2.0 to 4.0 feet
THIN	0.2 to 2.0 feet
VERY THIN	0.05 to 0.2 feet
LAMINATED	0.01 to 0.05 feet
THINLY LAMINATED	Less than 0.01 feet

WEATHERING

FRESH - No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.

SLIGHTLY WEATHERED - Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than in its fresh condition.

MODERATELY WEATHERED - Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.

HIGHLY WEATHERED - More than half of the rock material is decomposed and/or distintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.

COMPLETELY WEATHERED - All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

STRENGTH

PLASTIC - Can be remolded with hands.

FRIABLE - Can be crumbled between fingers or peeled by pocket knife.

WEAK - Can be peeled by a knife with difficulty, shallow indentations made by firm blow with point of geological hammer.

MEDIUM STRONG - Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geological hammer.

STRONG - Specimen requires more than one blow of geological hammer to fracture it.

VERY STRONG - Specimen withstands several blows of geological hammer without breaking.

EXTREMELY STRONG - Specimen can only be chipped with a geological hammer.

$$\text{RQD (Rock Quality Designation)} = \frac{\text{(Length of Solid Core Pieces 4 inches or Longer)}}{\text{(Total Length of Core Run)}}$$

Key to Test Data

Bulk Sample

Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Shelby Tube 3.0 inch O.D.

Continuous Rock Core

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Pitcher Barrel

0745, 5/31 — Approximate water level first observed in boring. Time recorded in reference to a 24-hour clock.

0800, 5/31 — Approximate water level observed in boring following drilling. Time recorded in reference to a 24-hour clock.

pp — Pocket Penetrometer reading, in (tsf) tons per square foot

LL	LIQUID LIMIT
PI	PLASTICITY INDEX
Passing-#200	SIEVE ANALYSIS (MINUS #200 SCREEN)
DD	DRY DENSITY (PCF)
UC	UNCONFINED COMPRESSIVE STRENGTH
WC	WATER CONTENT

Notes: Biscuiting indicates a horizontal fracture in a cored sample caused by adhesion of the sample to the inside of the core barrel.

Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

KLEINFELDER

ROCK AND SOIL LEGEND

PLATE **A-1**

Drafted By: J. Sala
Date: 9/17/2007
Project No.: 75856-PWGEO
File Number:

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

Date Completed: 7/10/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 269 feet. (MSL) Total Depth: 31.5 feet Groundwater Depth: I ∇ F ▼

Approx. Elevation: 269 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SANDY CLAY (CL) - olive-brown, dry, low to medium plasticity, fine grained sand (Pleistocene Alluvium)	
265	5			1	43				WC=8.1 DD=111.8	- sandy clay, brown, dry, hard, low to medium plasticity, fine grained sand	pp = 4.5
				2	24					- moist, with trace of fine gravel	
260	10			3	17				Passing -#200=56%	- light brown	pp = 3.0
255	15			4	17				LL=47; PI=34	**CLAY with SAND (CL)** - olive-brown, moist, stiff, medium plasticity, fine grained sand (Pleistocene Alluvium)	pp = 2.0
250	20			5	13				WC=36.9 DD=83.6		pp = 1.5
245	25			6	13					- ~6 inch clayey sand lense	pp = 3.0
240	30			7	15					- medium stiff	pp = 1.0
										Boring terminated at 31.5 feet. Boring backfilled with grout. No groundwater encountered.	

KLEINFELDER

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/11/2008 File Number:

LOG OF BORING B-1

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-2

RX-SOIL_PLSTN 75856.GPJ 1/11/08

Date Completed: 7/11/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 269 feet. (MSL) Total Depth: 31.5 feet Groundwater Depth: I ∇ F ▼

Approx. Elevation: 269 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
	0									**CLAYEY SAND (SC)** - brown, moist, medium dense, low plasticity, fine grained sand, with caliche (Pleistocene Alluvium)	
265				1	16						
	5			2	20				Passing -#200=72%		
260	10			3	19				WC=18.5 DD=93.0		
255	15			4	20					**CLAY (CL)** - black mottled with dark brown, moist, very stiff, medium plasticity, trace fine grained sand (Pleistocene Alluvium)	pp = 3.5
250										- olive-brown, stiff	
	20			5	17				LL=42; PI=30	- ~ 6" clayey sand lense	pp = 2.0
245	25			6	11						pp = 1.0
240	30			7	8				WC=34.7 DD=89.6		pp = 1.0
										Boring terminated at 31.5 feet. Boring backfilled with grout. No groundwater encountered.	

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

LOG OF BORING B-2

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-3

Date Completed: 7/9/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 314 feet. (MSL) Total Depth: 15.4 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 314 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SANDY CLAY (CL) - dark brown, dry, very hard, low to medium plasticity, fine grained sand, trace fine to coarse gravel (Pleistocene Alluvium)	pp = >4.5
				1	50/6"					- sandstone cobble	
310	5										
				2	50/6"						
305	10										
				3	50/5"				~~WC=12.8~~ DD=84.4	**SANDSTONE** - red-gray, highly weathered, friable to weak, fine grained sand (Neroly Formation)	
300	15										
				4	50/5"						
										Boring terminated at 15.4 feet. Boring backfilled with grout. No groundwater encountered.	
295	20										
290	25										
285	30										

KLEINFELDER

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

LOG OF BORING B-3

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-4

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Date Completed: 7/9/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 371 feet. (MSL) Total Depth: 20.5 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 371 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
370										**SANDY CLAY (CL)** - red-gray, moist, very stiff, low to medium plasticity, fine to medium grained sand (Quaternary Alluvium)	
				1	16				WC=19.3 DD=93.9		pp = 3.0
365	5			2	26					- hard	pp = >4.5
360	10			3	50/6"					**SANDSTONE** - gray mottled with limonite staining, moderate weathered, weak, fine grained sand (Cierbo Formation)	
355	15			4	50/5"					- 6-12" tuffaceous layer, highly weathered	
350	20			5	50/5"					- gravelly, with iron oxide staining, gravels are rounded, fine grained	
										Boring terminated at 20.5 feet. Boring backfilled with grout. No groundwater encountered.	
345	25										
340	30										

K L E I N F E L D E R

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

LOG OF BORING B-4

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-5

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Date Completed: 7/9/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 275 feet. (MSL) Total Depth: 25.4 feet Groundwater Depth: I ∇ F ▼

Approx. Elevation: 275 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V ROUGH

WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SANDY CLAY (CL) - brown, dry, very stiff, low to medium plasticity, fine to medium grained sand (Colluvium/Alluvium)	
				1	28						pp = 2.0
270	5										
				2	33				Swell (See plate D-6) WC=15.1 DD=85.6	- moist, with fine gravel	pp = 3.5
										CLAYEY SAND (SC) - gray-brown mottled with yellow, moist, low to medium plasticity, fine grained sand (Colluvium/Alluvium)	
265	10										
				3	36				WC=19.8 DD=93.9		
260	15										
				4	50/6"				Passing -#200=45%		
255	20										
				5	40				WC=3.9 DD=84.1	- fine to coarse grained sand	
250	25										
				6	50/5"					**SANDY CLAY (CL)** - gray-brown, moist, very stiff, low plasticity, fine grained sand with >2" max diameter gravel in base of liner (Alluvium)	pp = 3.25
										Boring terminated at 25.4 feet. Boring backfilled with grout. No groundwater encountered.	
245	30										

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

LOG OF BORING B-5

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE 1 of 1 **A-6**

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

Date Completed: 7/10/2007 Logged By: T. Nguyen

Boring Location: Steep - Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 288 feet. (MSL) Total Depth: 31 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 288 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SANDY CLAY (CL) - brown, moist, very stiff, low to medium plasticity, fine grained sand (Colluvium/Alluvium)	
285				1	29						pp = 1.0
	5			2	33				WC=20.6 DD=89.0		pp = 3.5
				3	30						
				4	23				WC=21.8		pp = 3.0
280				5	42				DD=99.0 LL=58; PI=43	**CLAYSTONE** - brown, completely weathered, friable, high plasticity, with caliche (Tulare Formation)	
	10			6	34						
				7	22				UC=4.5 tsf		
275				8	37				WC=18.4 DD=101.0		
	15			9	36						
				10	21						
270				11	29				WC=23.2 DD=101.2	- yellow-brown	
	20			12	27					fine to medium grained sand	
				13	23					- sandy	
265				14	32					- less sand, brown	
				15	37				LL=39; PI=26	**CLAYEY TUFFACEOUS SANDSTONE** - brown, highly weathered, friable (Tulare Formation)	
	25			16	64					**CLAYSTONE** - yellow-brown, completely weathered, friable, medium plasticity, with caliche (Tulare Formation) - highly weathered, weak	
				M	50/6"						
260				18	67				UC=11.5 tsf WC=11.9 DD=121.0		
	30			19	69						
				20	50/6"						
255										Boring terminated at 31 feet. Boring backfilled with grout. No groundwater encountered.	

RX-SOIL_PLSTN 75856.GPJ 1/11/08

KLEINFELDER

LOG OF BORING B-6

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-7

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/11/2008 File Number:

Date Completed: 7/10/2007 Logged By: T. Nguyen

Boring Location: Flat dirt, Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 198 feet. (MSL) Total Depth: 21.5 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 198 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										CLAYEY SAND (SC) - brown, moist, medium dense, low plasticity, fine grained sand (Alluvium)	
195				1	18						pp = >4.5
	5			2	24				WC=15.6 DD=99.9	- mottled with yellow-brown, with caliche	pp = >4.5
190											
	10			3	15				WC=24.6 DD=94.4	- yellow-brown, fine to medium grained sand	pp = >4.5
185											
	15			4	9				LL=34; PI=21 Passing -#200=48%	- wet, medium stiff, fine grained sand	pp = 0.5
180									Swell (See plate D-7) WC=29.7 DD=88.6		
	20			5	12				WC=33.0 DD=98.1	- with organics (grass - like vegetation)	pp = 0.5
										Boring terminated at 21.5 feet. Boring backfilled with grout. No groundwater encountered.	
175											
	25										
170											
	30										
165											

KLEINFELDER

LOG OF BORING B-7

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE 1 of 1

A-8

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Date Completed: 7/9/2007 Logged By: T. Nguyen

Boring Location: Steep - Weeds

Driller: Frontier Drilling Drilling Method/Size: 6" Solid Stem Auger

Elev. Top of Hole: Approximately 282 feet. (MSL) Total Depth: 26.5 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 282 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
282	0									**SANDY CLAY (CL)** - dark brown, dry, hard, medium plasticity, fine to medium grained sand (Alluvium)	
280		■		1	26						pp = >4.5
	5	■		2	27				WC=18.9 DD=101.9	- moist	pp = 4.25
275											
	10	■		3	37				LL=43; PI=32		pp = >4.5
270											
	15	■		4	35				WC=17.6 DD=113.6	**SANDY CLAYSTONE** - yellow-brown, highly weathered, friable, medium to high plasticity (Tulare Formation)	pp = 3.0
265											
	20	■		5	20					- stiff	pp = 2.5
260											
	25	■		6	23				WC=14.1 DD=107.2		
255										Boring terminated at 26.5 feet. Boring backfilled with grout. No groundwater encountered.	
	30										
250											

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

Drafted By: J. Sala Project No.: 75856-PWGEO

Date: 1/10/2008 File Number:

LOG OF BORING B-8

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 1
A-9

Date Completed: 7/12/2007 Logged By: L. Serrano

Boring Location:

Driller: Gregg Drilling Drilling Method/Size: H Q Wireline/4"

Elev. Top of Hole: Approximately 320 feet. (MSL) Total Depth: 29 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 320 feet (msl)
Approx. Latitude: 37.98696
Approx. Longitude: -121.87472

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
				1		94				**SANDY LEAN CLAY (CL)** - dark brown, moist, hard, low to medium plasticity, fine to coarse grained sand, trace gravel, silty, gravels are chert and sandstone (Landslide Deposit)	
315	5			2		95					
310	10									- 1.5" gravel	
				3		95					
305	15									**NO RECOVERY**	material in shoe same as above
300	20										1-2" rounded to subangular gravels, basalt, sandstone, chert in shoe
295	25										same gravels in shoe
290	30									Boring terminated at 29 feet. Boring backfilled with grout. No groundwater encountered.	

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

LOG OF BORING C-1

PLATE 1 of 1

A-10

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

Date Completed: 7/11/2007 Logged By: L. Serrano

Boring Location:

Driller: Gregg Drilling Drilling Method/Size: H Q Wireline/4"

Elev. Top of Hole: Approximately 554 feet. (MSL) Total Depth: 76 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 554 feet (msl)
Approx. Latitude: 37.98851
Approx. Longitude: -121.89367

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SANDSTONE - olive, friable, completely weathered, massive, fine grained sand (Cierbo Formation)	
550	5			1		100	60			- weak, highly weathered - friable, completely weathered - mottled with iron oxide	J-PL
545	10			2		100	53		D.S. Ø=27° C=2.2 ksf WC=17.1 DD=103		B-PL angle 55°; iron oxide on bedding surfaces B-PL angle 55°
										- weak, highly weathered - 4" seam of laminated bedding	B-PL angle 55°
540	15			3		75	33				
											B-PL-R angle 50°
535	20			4		80	33			- shell seam - fine grained sand, olive gray	B-PL-R angle 50° B-PL-R angle 40°
											no recovery; sample stuck in inner barrel
530	25			5		0					
										MUDSTONE - olive-gray, weak, highly weathered, closely fractured	iron oxide on fracture surfaces
525	30			6		100	27		D.S. Ø=24° C=2.1 ksf WC=48.6 DD=68	**SANDSTONE** - olive-gray, shelly, weak, highly weathered	
										- gray	slicken-sided fractures

KLEINFELDER

LOG OF BORING C-2

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 3
A-11

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
520	35			7		85	68			**SANDSTONE** - (continued) - gray-brown, less sand, very closely fractured	
515	40			8		100				**CLAYSTONE** - black, highly weathered, weak, massive bedding	J-PL; infilled fractures cemented with gypsum no RQD due to biscuiting
510	45			9		100				**CLAYEY SANDSTONE** - black, highly weathered, friable to weak, massive bedding, closely to moderately fractured, fine grained sand	no RQD due to biscuiting
505	50			10		97	33			- completely weathered, friable - less clay, moderately weathered, weak	
500	55			11		92	37			- yellow-brown - black	B-PL J-PL-R; with iron-oxide staining B-PL
495	60			12		90	0			**SANDSTONE** - olive-brown mottled with rust, highly weathered, weak, medium grained sand	
490	65			13		88	88				B-PL B-PL
485	70			14		77	55			- laminated, thin bedding - red-gray - claystone lense ~6" - gravelly layer, with 1/2" maximum	J(S)-PL-K

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER	LOG OF BORING C-2	PLATE 2 of 3
Drafted By: J. Sala — Project No.: 75856-PWGEO Date: 1/10/2008 — File Number:	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	A-11

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
480	75			15		100	62			diameter **SANDSTONE** - (continued) - black - olive with iron-oxide mottling	
475	80									Boring terminated at 76 feet. Boring backfilled with grout. No groundwater encountered.	
470	85										
465	90										
460	95										
455	100										
450	105										
445											

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

Drafted By: J. Sala
Date: 1/10/2008
Project No.: 75856-PWGEO
File Number:

LOG OF BORING C-2

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
3 of 3
A-11

Date Completed: 7/10/2007 Logged By: L. Serrano

Boring Location:

Driller: Gregg Drilling Drilling Method/Size: H Q Wireline/4"

Elev. Top of Hole: Approximately 528 feet. (MSL) Total Depth: 55 feet Groundwater Depth: I ▽ F ▼

Approx. Elevation: 528 feet (msl)
Approx. Latitude: 37.98381
Approx. Longitude: -121.88313

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										MUDSTONE - gray-brown, friable, sandy (Neroly Formation)	
525				1		62	16			**SANDSTONE** - gray-brown, closely fractured, medium to coarse grained sand	
	5									**MUDSTONE** - gray-brown, very closely fractured, weak, iron-oxide staining on fracture surfaces	
										SANDSTONE - brown, very closely fractured, medium grained sand, iron-oxide staining on fracture surfaces	
520	10			2		95	0			**MUDSTONE** - green, weak, very closely fractured, iron-oxide staining - red-brown	
515	15			3		90	54			**SANDSTONE** - gray-brown, closely fractured, weak to moderately strong, laminated to thin bedding	
510	20			4		100	27				
										CLAYSTONE - yellow-brown, friable, completely weathered	
505	25			5		100	30			**SANDSTONE** - gray-brown, friable to weak, very closely to closely fractured, laminated to thin bedding, magnesium-oxide staining on fracture surfaces - near vertical fractures	
500	30			6		83	12				
495										- friable lense	

KLEINFELDER

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

LOG OF BORING C-3

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
1 of 2
A-12

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
				7		100	52			**SANDSTONE** - (continued)	
										- laminated cross bedding	
	35			8		100	50				
490				9		100	38			- laminated cross bedding	
	40									- wood fragments	
485				10		100	8				
	45										
480				11		100	15				
	50									- wood fragments	
475				12		100	19				
	55									Boring terminated at 55 feet. Boring backfilled with grout. No groundwater encountered.	
470											
	60										
465											
	65										
460											
	70										

KLEINFELDER

Drafted By: J. Sala
Date: 1/10/2008

Project No.: 75856-PWGEO
File Number:

LOG OF BORING C-3

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE
2 of 2
A-12

RX-SOIL_PLSTN 75856.GPJ 1/10/08

Date Completed: 7/12/2007 Logged By: L. Serrano

Boring Location:

Driller: Gregg Drilling Drilling Method/Size: H Q Wireline/4"

Elev. Top of Hole: Approximately 288 feet. (MSL) Total Depth: 55 feet Groundwater Depth: I ∇ F ▼

Approx. Elevation: 288 feet (msl)

J-JOINT PL-PLANAR P-POLISHED
F-FAULT C-CURVED K-SLICKENSIDED
S-SHEAR U-UNDULATING SM-SMOOTH
B-BEDDING ST-STEPPED R-ROUGH
F-FOLIATION I-IRREGULAR VR-V. ROUGH
WEATHERING INDEX (FR,SW,MW,HW,CW) FRACTURE SPACING (VC,C,M,W,VW)

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
										SAND CLAY (CL) - dark brown, moist, very stiff, medium plasticity, fine to coarse grained sand (Landslide Deposit)	
285				1		57					pp = 3.75
	5									- brown	loss of circulation
											samples stuck in inner liner
280											slow drilling due to loss of circulation
	10										
275											
	15										
										CLAYSTONE - yellow-brown, completely weathered, friable, trace sand, high plasticity, massive, with caliche (Tulare Formation/Landslide?)	
270				2		95	85			- seam of gray	
	20										S-P-R angle 45°
										- sandy, fine grained sand	
265				3		98	93				
	25										
260				4		48	48				
	30										S-P-R angle 45°
255											

RX-SOIL_PLSTN 75856.GPJ 1/10/08

KLEINFELDER

LOG OF BORING C-4

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

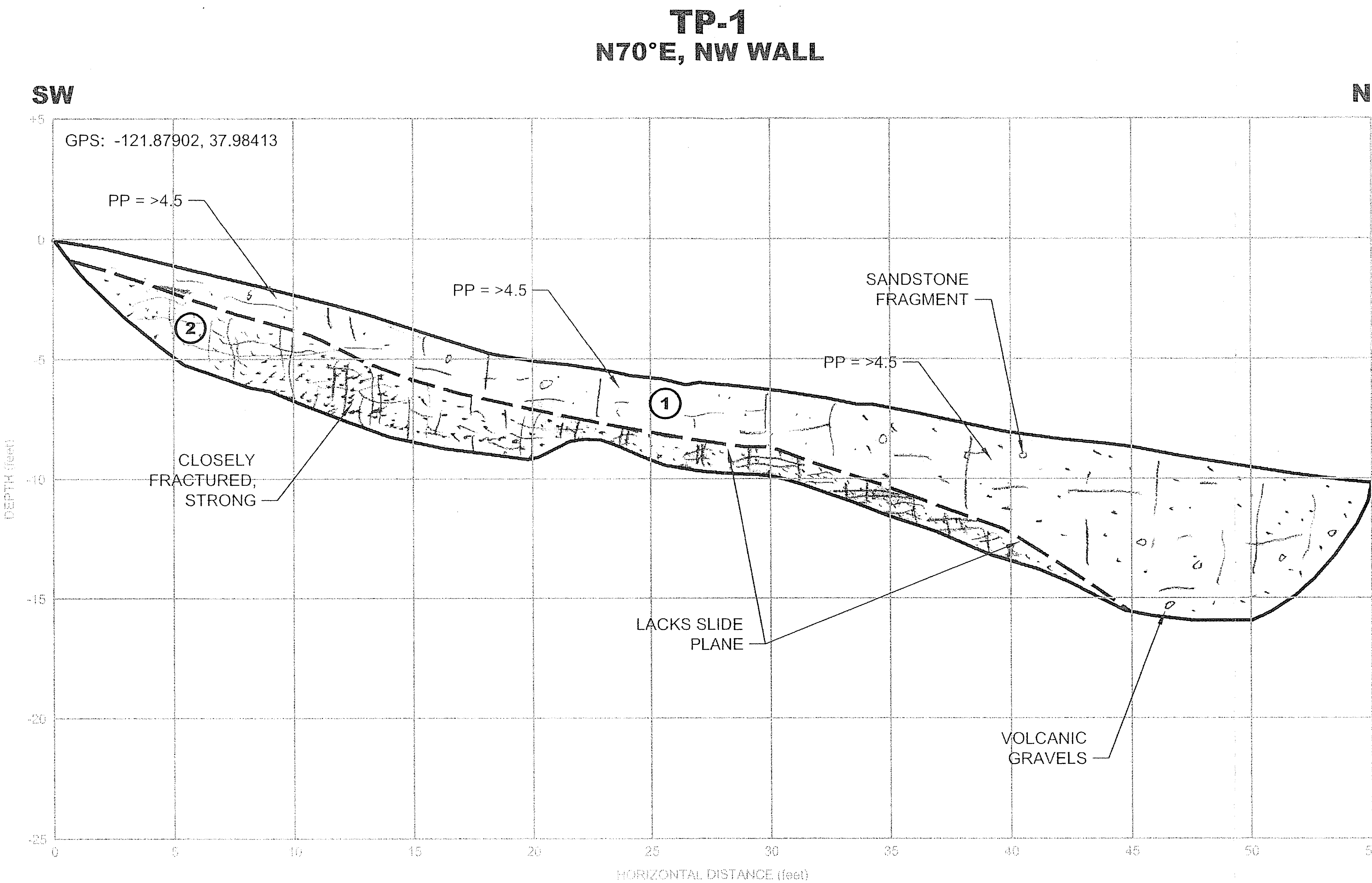
PLATE 1 of 2

A-13

Drafted By: J. Sala Project No.: 75856-PWGEO
Date: 1/10/2008 File Number:

Elevation (ft., msl)	Depth (feet)	SAMPLE TYPE	Time/Core Run Length	SAMPLE/BOX NUMBER	BLOWS PER FOOT	% RECOVERY	RQD	Graphic Log	Laboratory Tests	SOIL/ROCK DESCRIPTION	DISCONTINUITY DATA/ DRILLER'S NOTES
				5		20	20			**CLAYSTONE** - (continued)	
	35									- less sand	
250				6		60	55				
	40										
245				7		58	57				
	45										
240				8		10	0				sample stuck in inner liner, 6" from shoe recovered
	50										
235						60	46			- more carbonate	S-U-P angle 45°
	55									Boring terminated at 55 feet. Boring backfilled with grout. No groundwater encountered.	
230											
	60										
225											
	65										
220											
	70										

APPENDIX B



① **SANDY CLAY (CL)** - brown, moist, hard, rootlets within, lower portion is lighter in color and sandier (Colluvium). Note: Hard digging

② **SANDSTONE** - bluish-gray, closely fractured, very slightly weathered, strong (Neroly Formation)

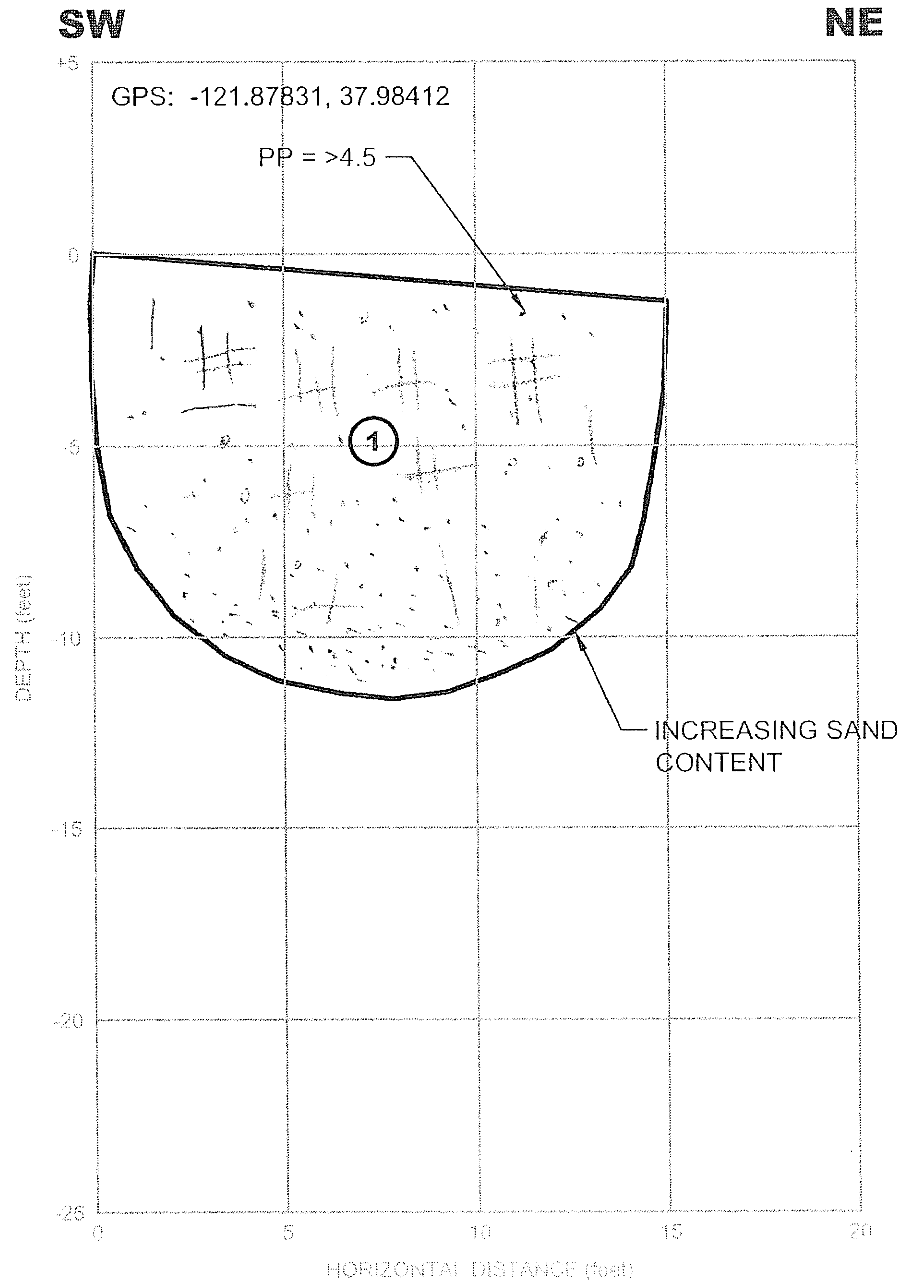
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	**TEST PIT TP-1**	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A, Pittsburg, CA 94565	**B-1**
CHECKED BY: LS	BUCHANAN ROAD BYPASS, PITTSBURG, CALIFORNIA	PH. (925) 427-6477 FAX. (925) 427-6478	
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg	www.kleinfelder.com	

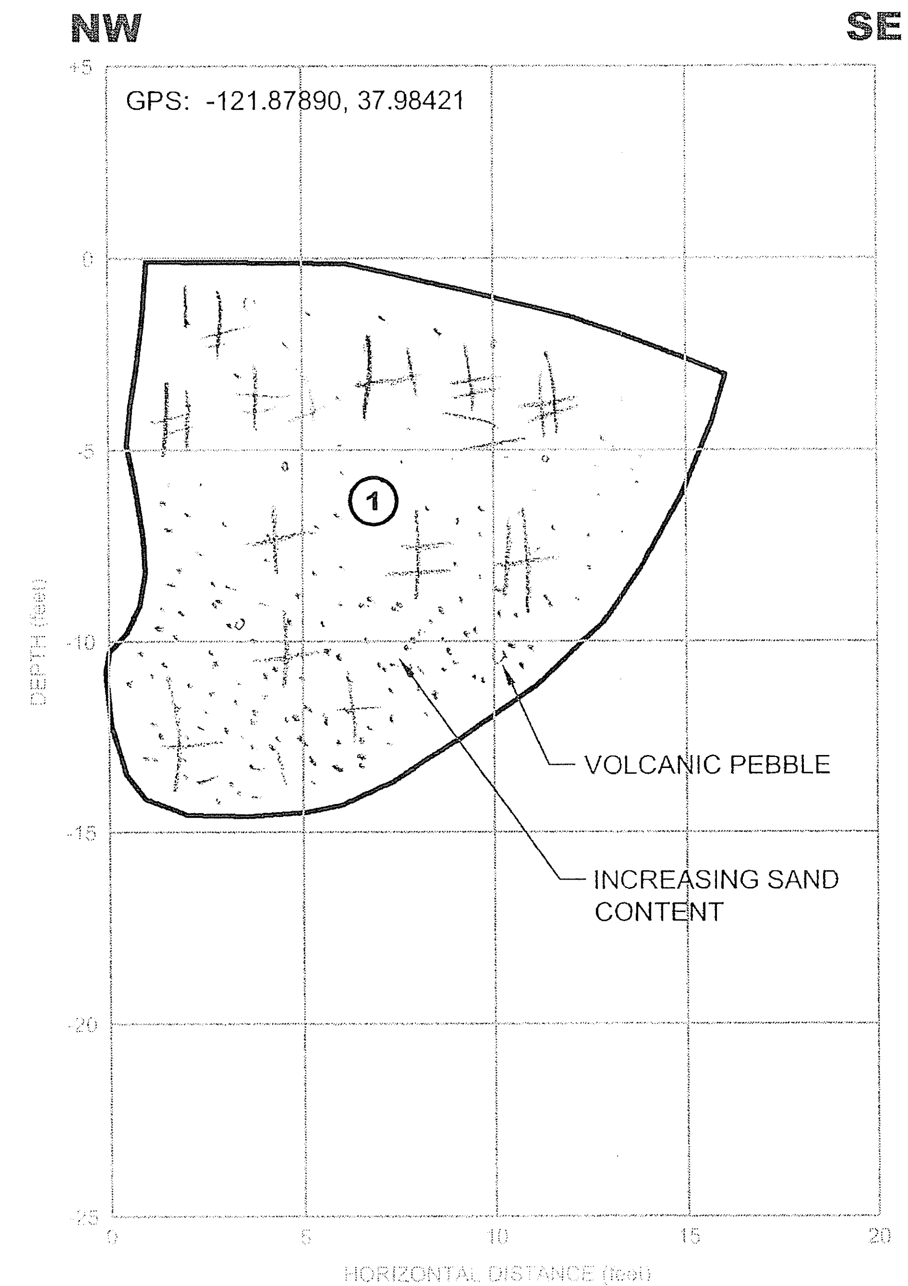
PLOTTED: 10 Jan 2008, 1:41pm, jsala

ATTACHED XREFS: XRef: Style A_11x17
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: TP-1

TP-2
N80°E, NW WALL



TP-3
N61°W, NE WALL



(1) **SANDY CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

NOTE: Bedrock not encountered.

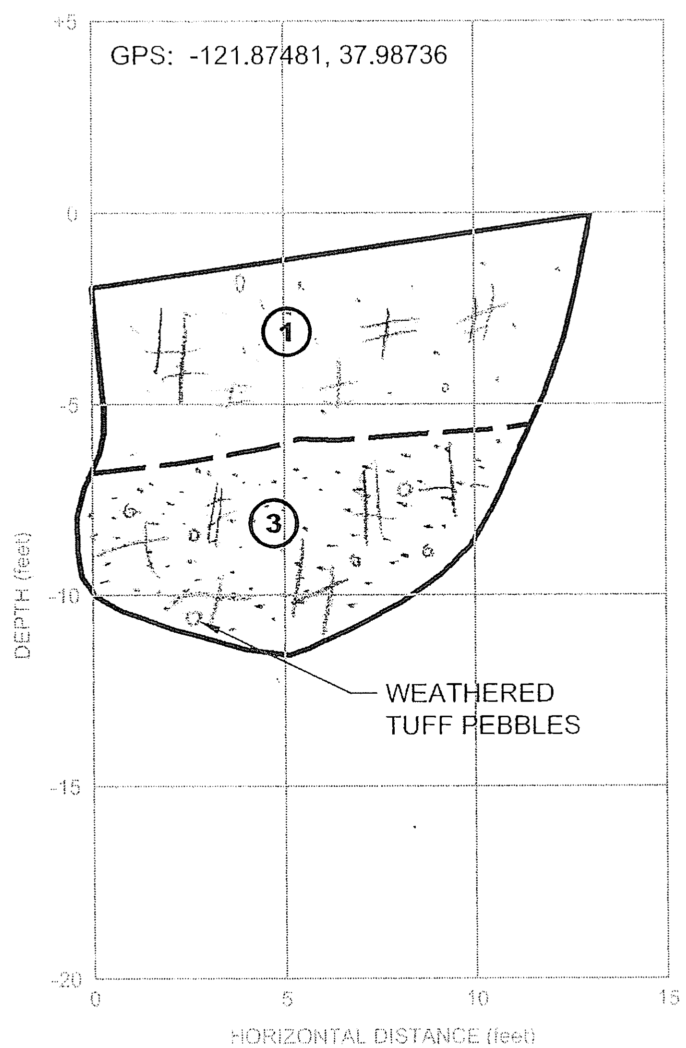
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	**TEST PITS TP-2 AND TP-3**	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	**B-2**
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

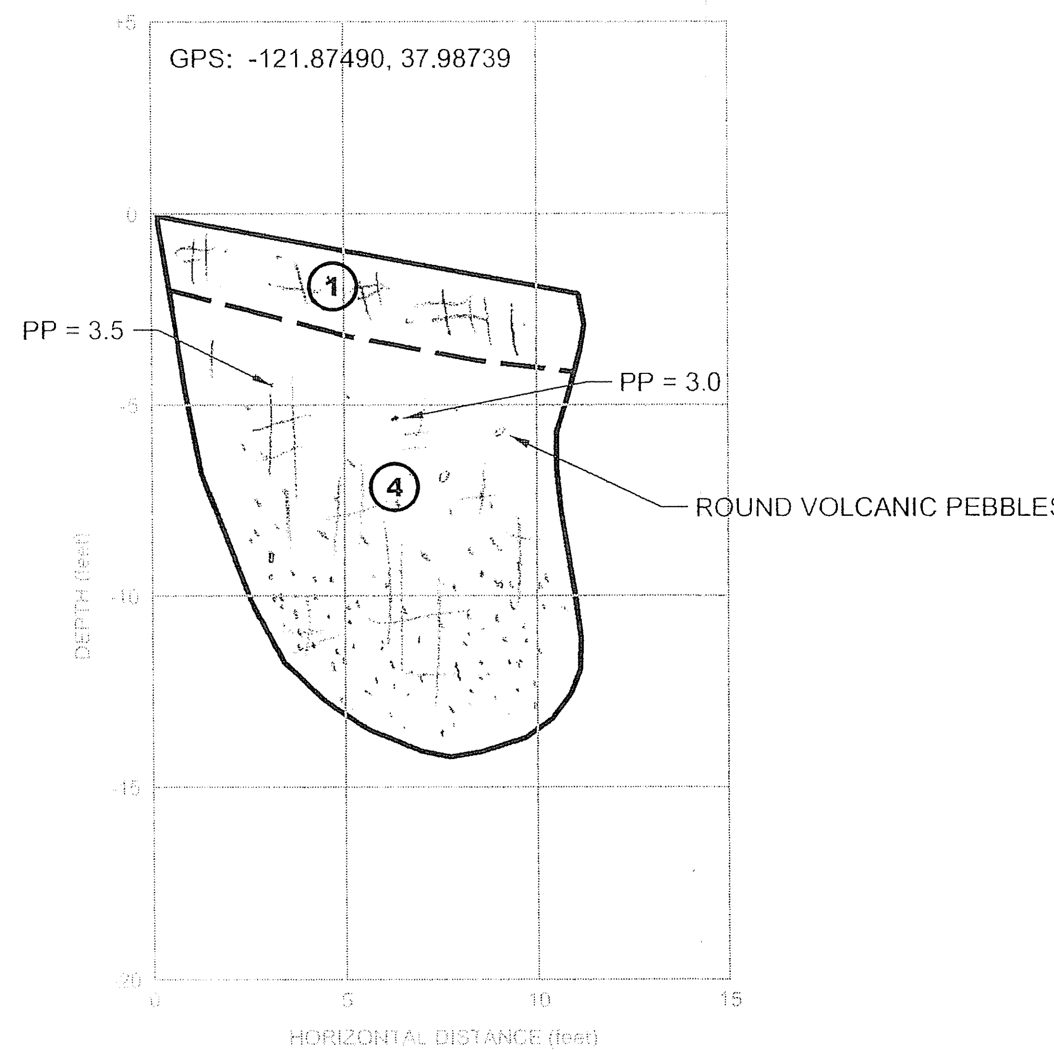
PLOTTED: 10 Jan 2008, 1:42pm, jsala

ATTACHED XREFS: XRef: Style A_11x17 LAYOUT: TP-2_TP-3
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICS\PWGEO\TP and Trench Logs\

TP-4
N27°W, NE WALL



TP-5
N5°W, SW WALL



(1) **SANDY CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

(3) **SANDY CLAY (CL)** - red-brown, moist, stiff, with silt (Colluvium)

(4) **SANDY/SILTY CLAY (CL)** - dark red-brown, moist, stiff to very stiff near surface; lighter and softer with depth, some pebble and gravel (Colluvium). Note: Easy digging

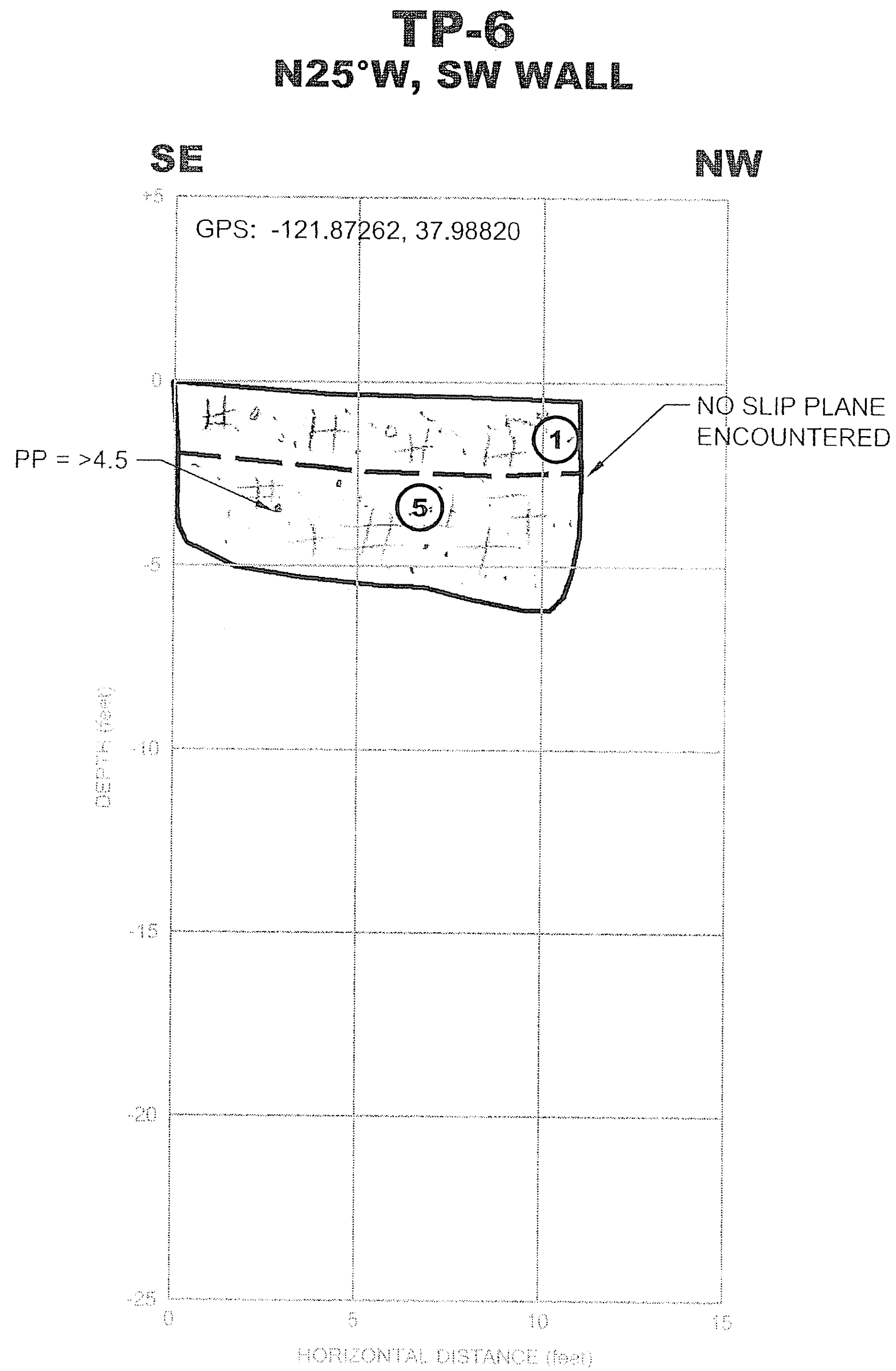
NOTE: Bedrock not encountered.

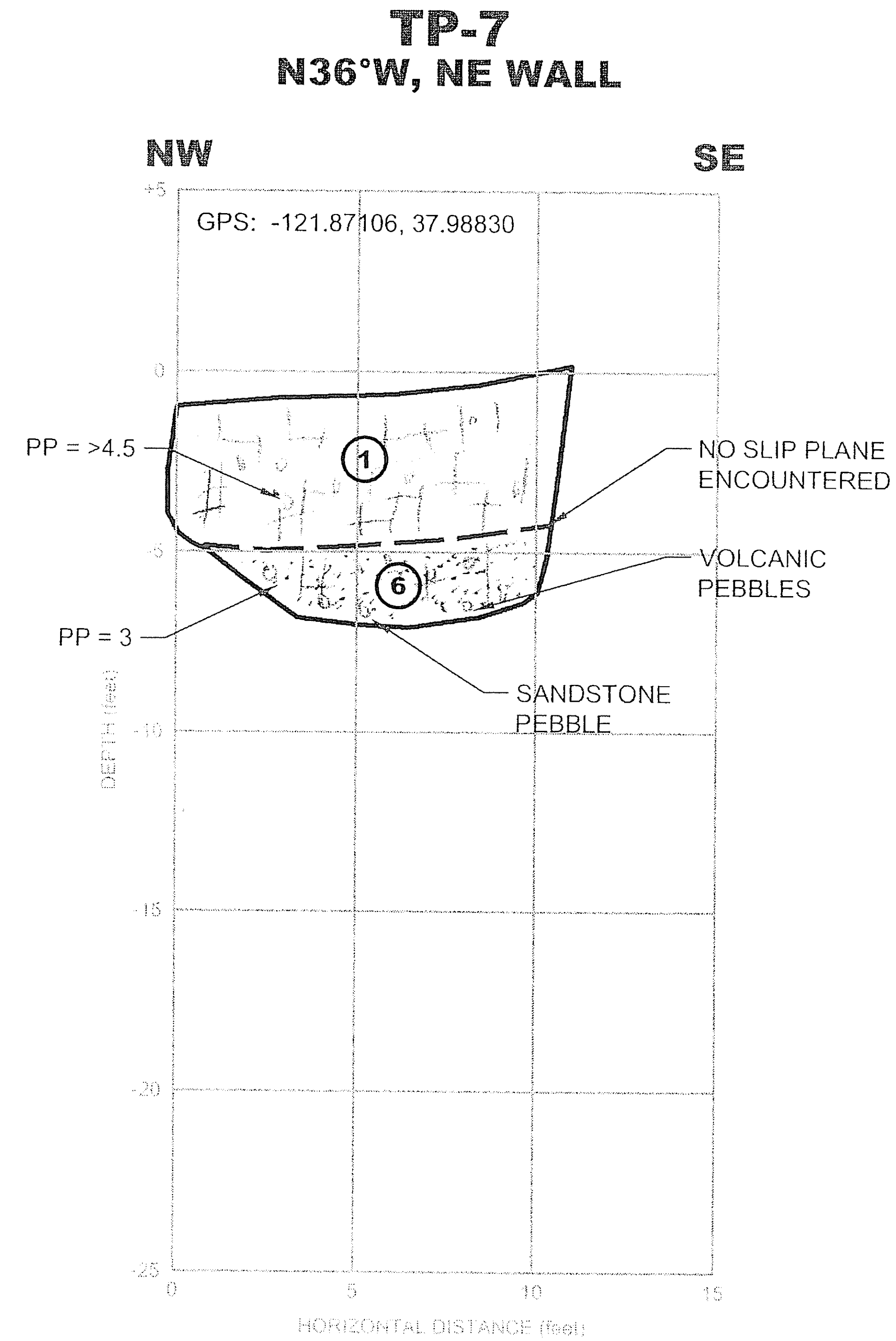
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	TEST PITS TP-4 AND TP-5		KLEINFELDER	PLATE
REVISED BY:			981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	B-3
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA			
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856	FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 10 Jan 2008, 1:43pm, jsala

ATTACHED XREFS: XRef: Style A_11x17
File-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: TP-4_TP-5





(1) **SANDY CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

(5) **CLAYSTONE** - tan, moist, hard, sandy, silty lean clay (Tulare Formation)

(6) **CLAYEY SAND** - tan, moist, stiff, with pebbles and gravel (Alluvium)

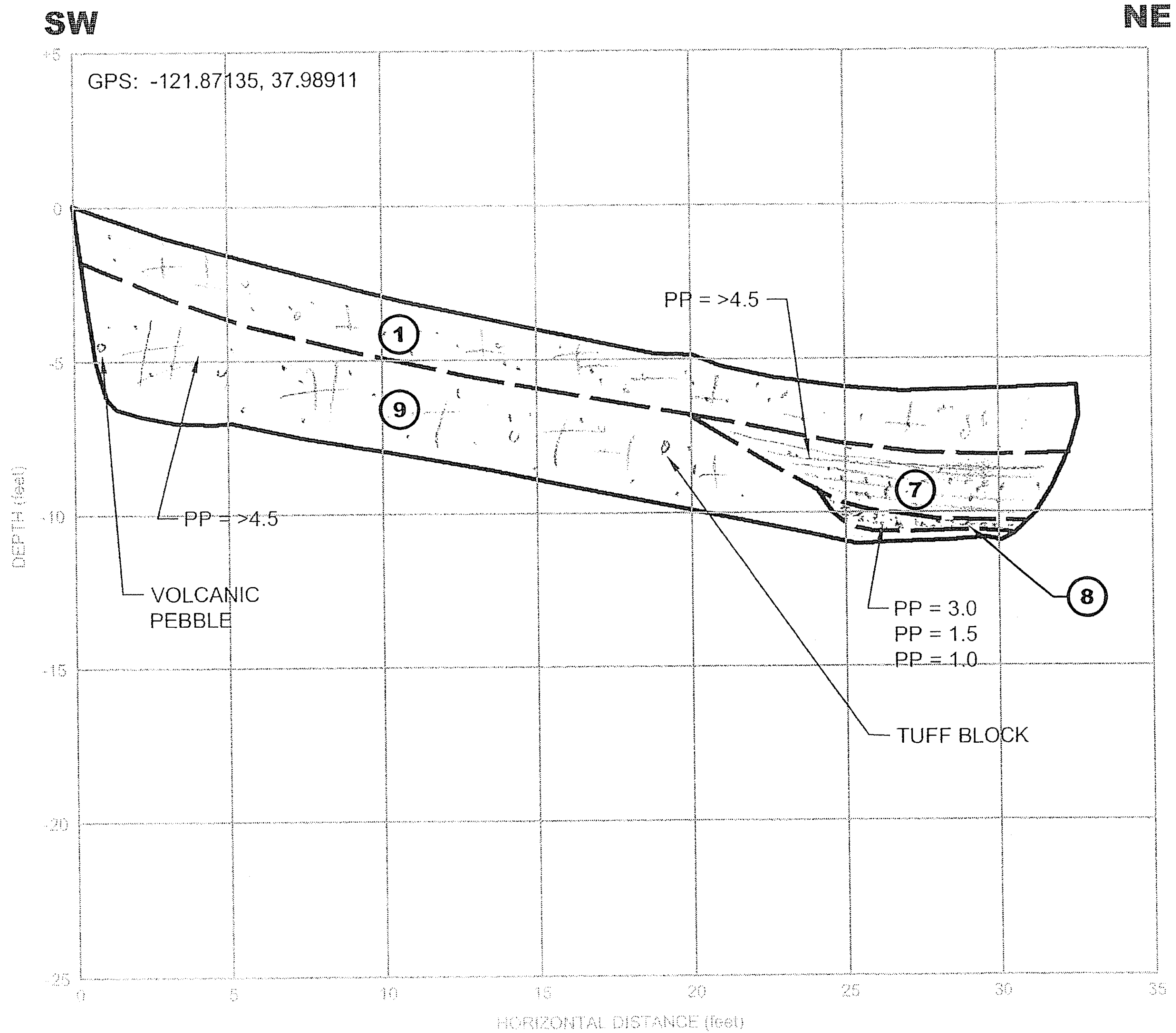
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	TEST PITS TP-6 AND TP-7	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	B-4
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 10 Jan 2008, 1:44pm, jsala

ATTACHED XREFS: XRef: Style A_11x17
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICS\PWGEO\TP and Trench Logs\ LAYOUT: TP-6_TP-7

TP-8
N89°E, NW WALL



(1) **CLAY with SAND (CL)** - dark brown, moist (dry near top), hard, with silt stained with white prismatic evaporate material (Colluvium)

(7) **SANDY CLAY (CL)** - tan and dark brown, laminated, moist, hard, some gravel of various composition (Alluvium)

(8) **SAND (SP)** - tan, moist, weak, medium grained, some gravel of various composition, inversely graded (Alluvium)

(9) **CLAY (CL)** - dark brown, moist, hard,some gravel of various composition (Colluvium)

NOTE: Bedrock not encountered.

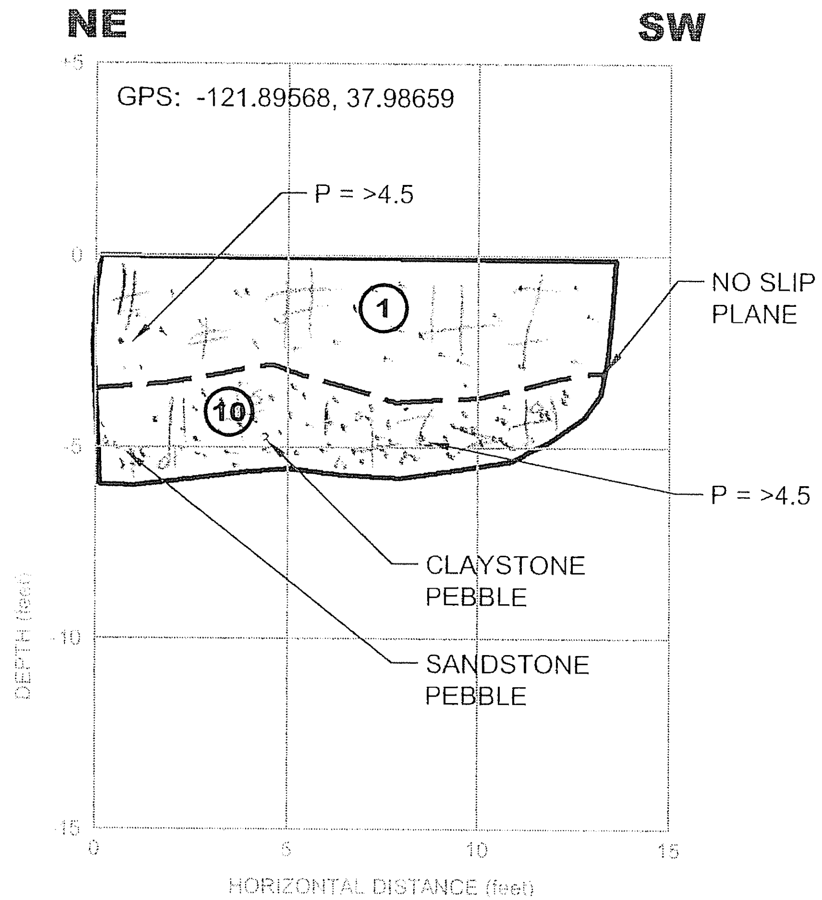
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS		TEST PIT TP-8	KLEINFELDER 981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	PLATE **B-5**
REVISED BY:				
CHECKED BY: LS		BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008	APPROVED BY:	PROJECT NO. 75856	FILE NAME: T-TP_LOGS.dwg	

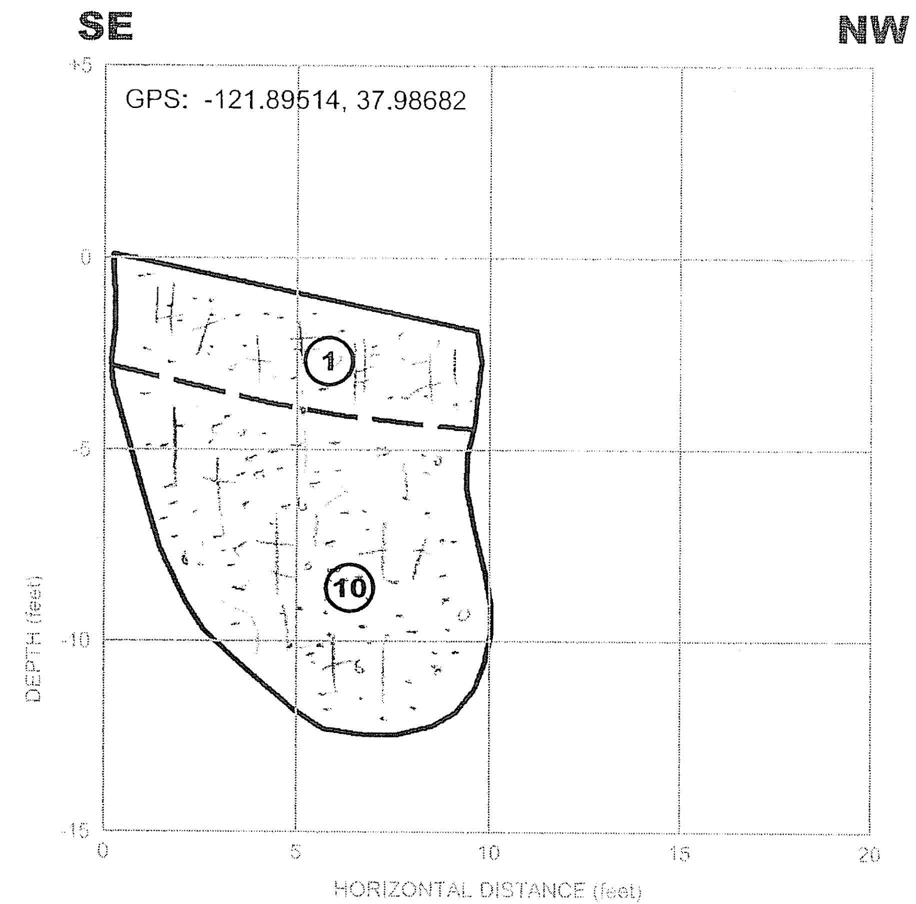
PLOTTED: 10 Jan 2008, 1:56pm, jsala

ATTACHED XREFS: XRef: Style A 11x17
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: TP-8

TP-9
N11°E, SE WALL



TP-10
N84°W, SW WALL



(1) **SANDY CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

(10) **SANDY CLAY (CL)** - dark red-brown, moist, very stiff (Colluvium)

NOTE: Bedrock not encountered.

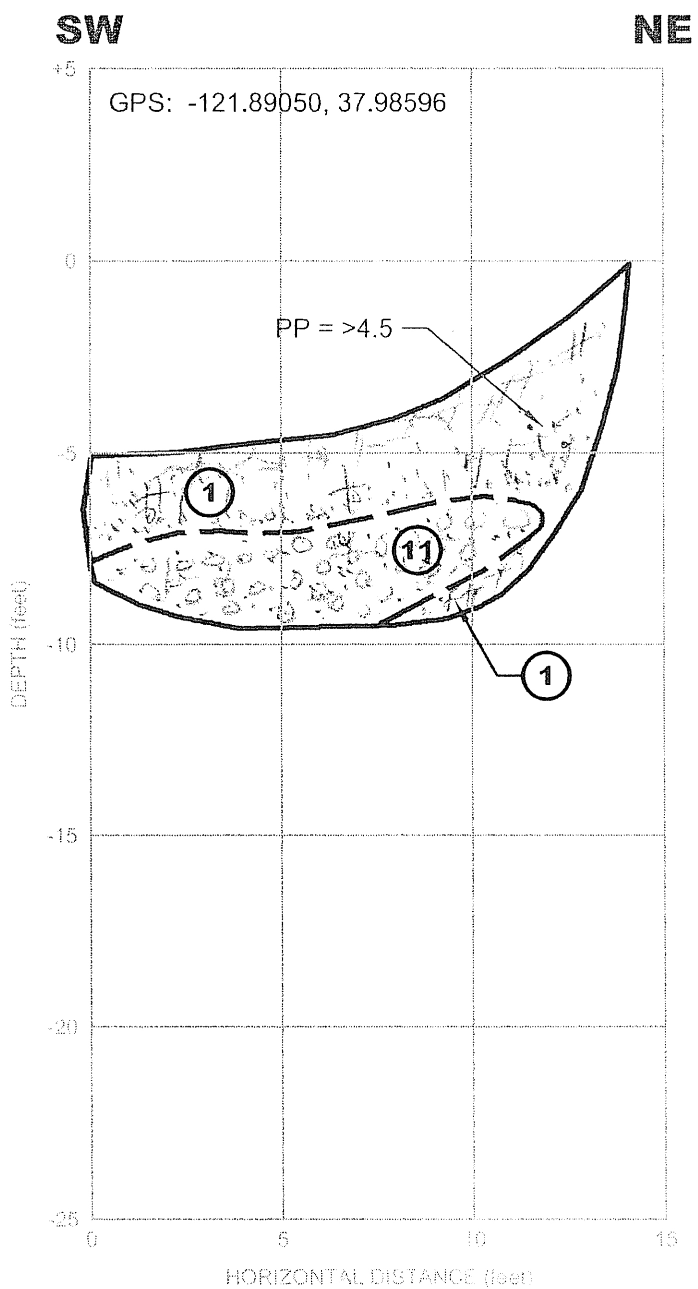
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	TEST PITS TP-9 AND TP-10	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	B-6
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

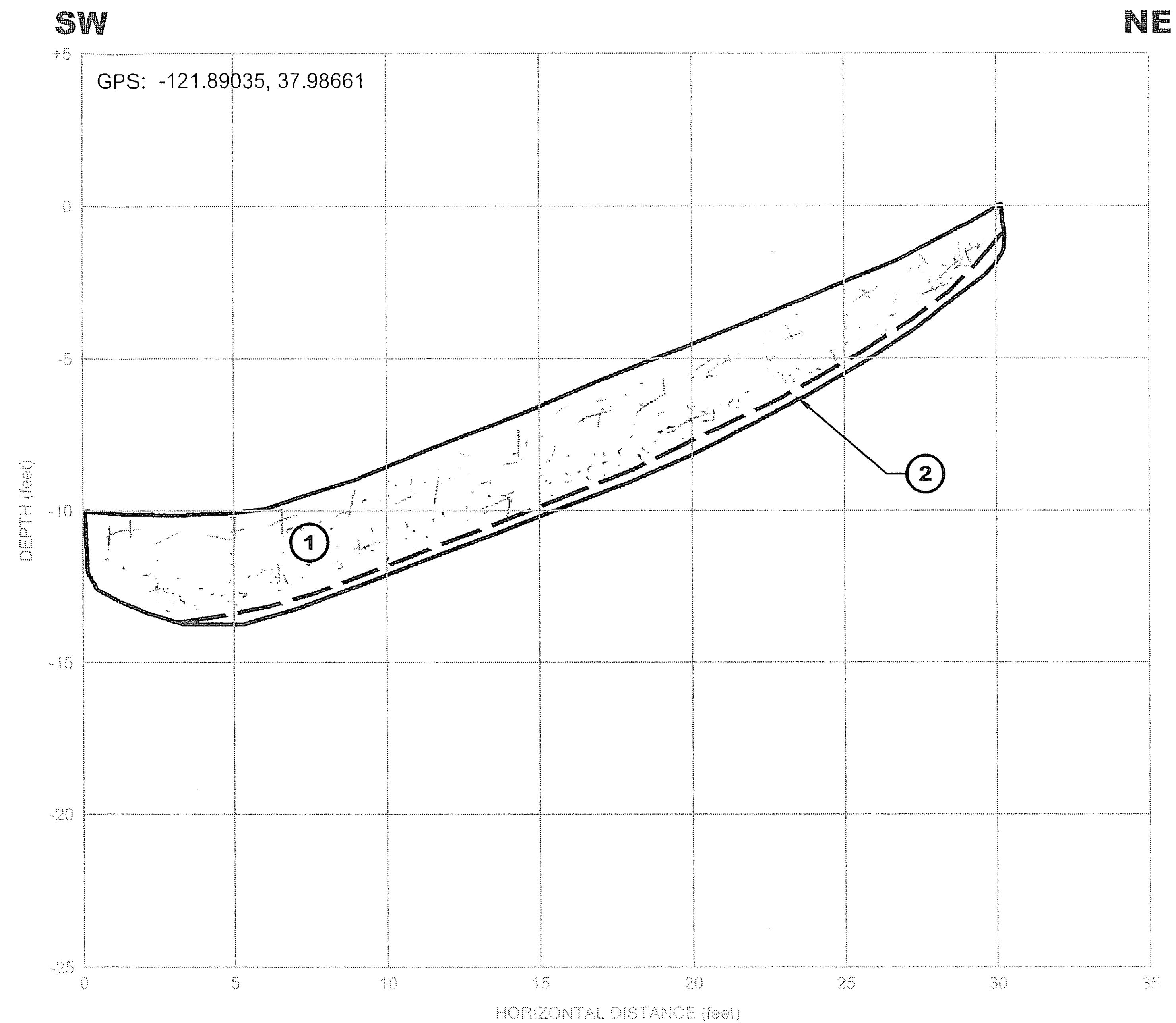
PLOTTED: 10 Jan 2008, 1:58pm, jsala

ATTACHED XREFS: XRef: Style A 11x17 LAYOUT: TP-9_TP-10
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICS\PWGEO\TP and Trench Logs\

TP-11
N55°E, NW WALL



TP-12
N65°E, NW WALL



(1) **SANDY LEAN CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

(2) **SANDSTONE** - blue-gray, very strong, no clear fractures, slightly weathered to weathered (Neroly)

(11) **GRAVEL** - moist, stiff, friable, with clayey sand, well-rounded gravel consists of volcanics and sandstone (Alluvium)

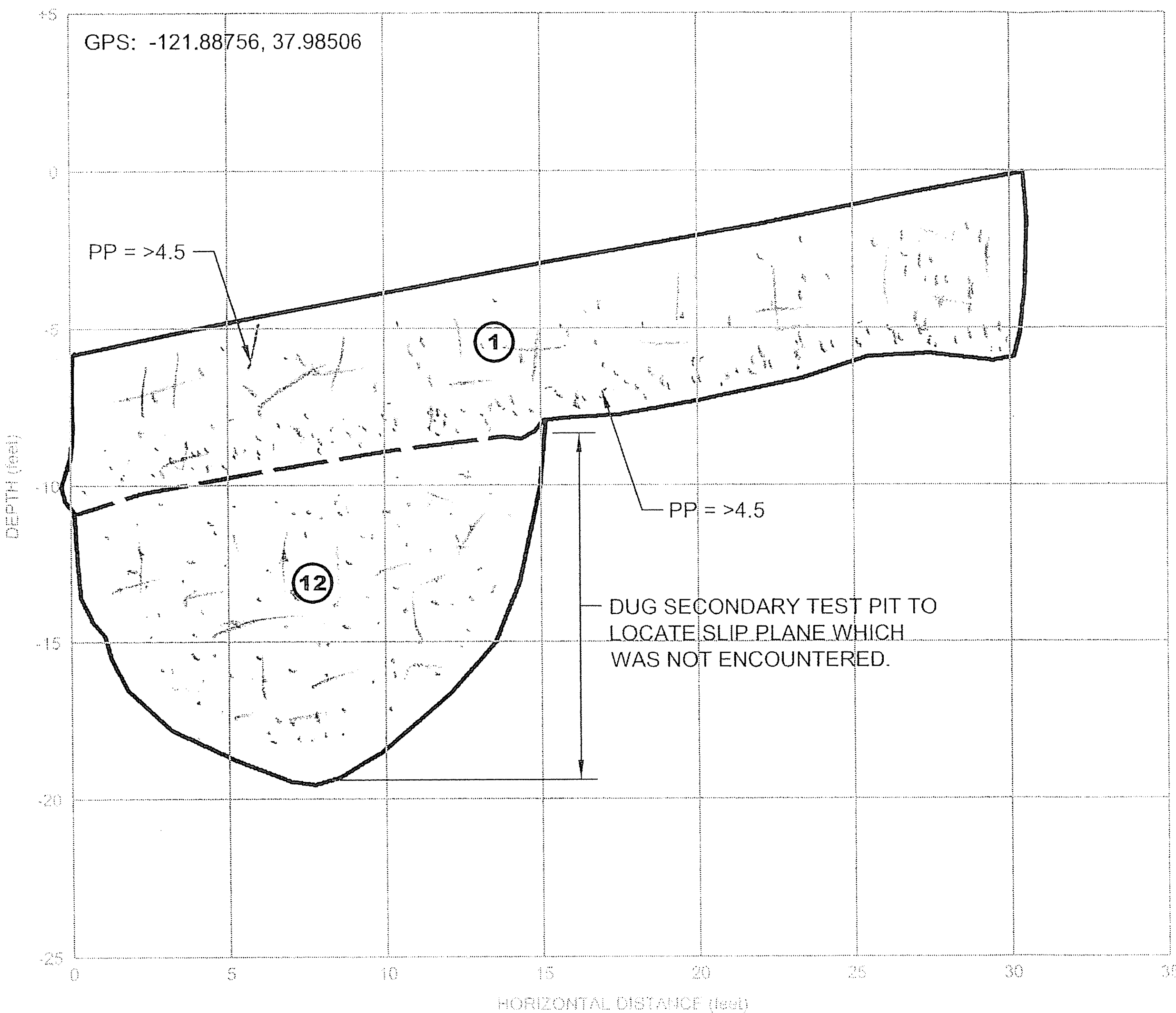
LOGGED BY: R. Roatch, July 9-12, 2007

ATTACHED XREFS: XRef: Style A_11x17 LAYOUT: TP-11_TP-12
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\

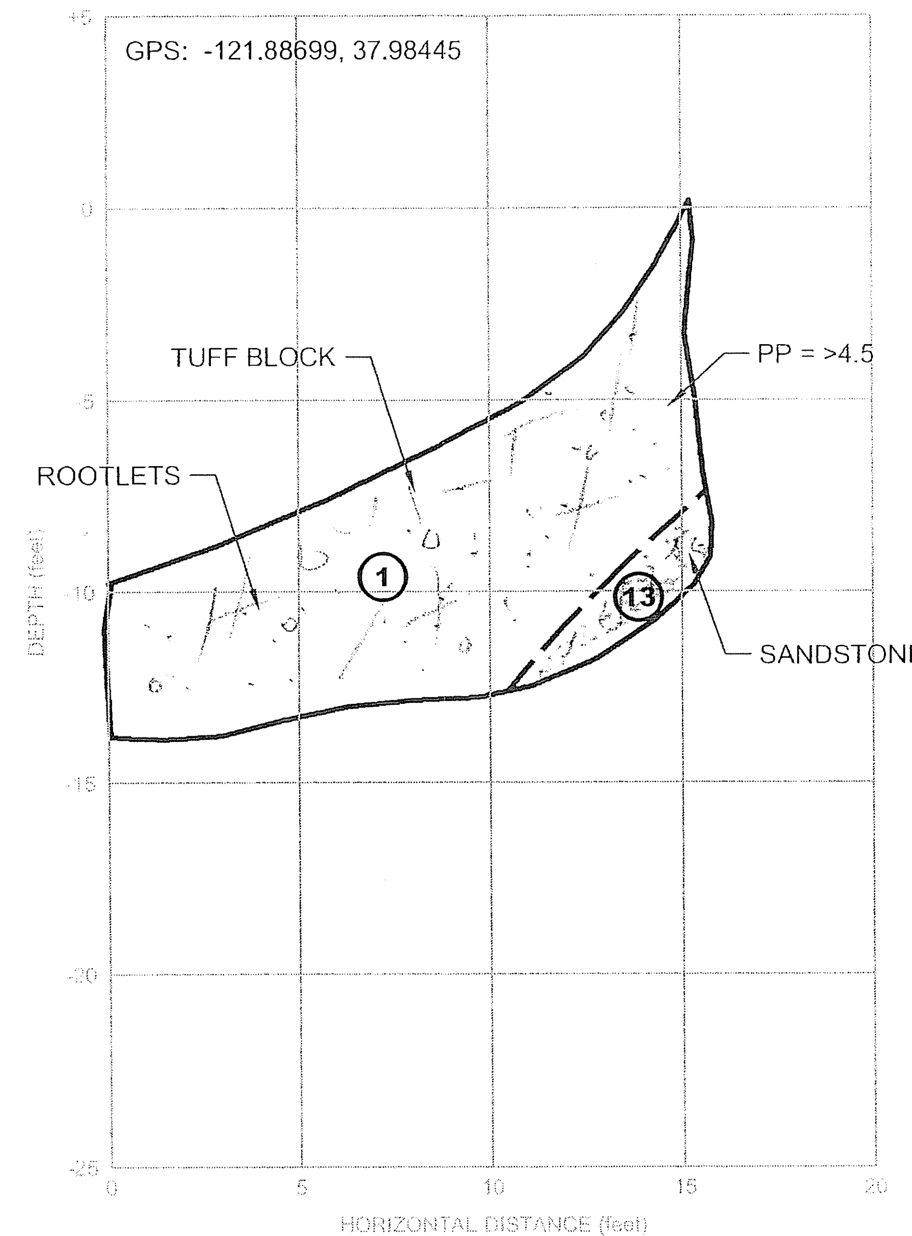
DRAWN BY: LGS	TEST PITS TP-11 AND TP-12	KLEINFELDER
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg	PLATE **B-7**

PLOTTED: 10 Jan 2008, 2:00pm, jsala

TP-13
N40E, NW WALL



TP-14
N10°W, SW WALL



(1) **SANDY CLAY (CL)** - brown, moist, hard, increasing sand content with depth (Colluvium)

(12) **SANDY CLAY (CL)** - dark brown, moist, very stiff, sand content increases with depth (Colluvium)

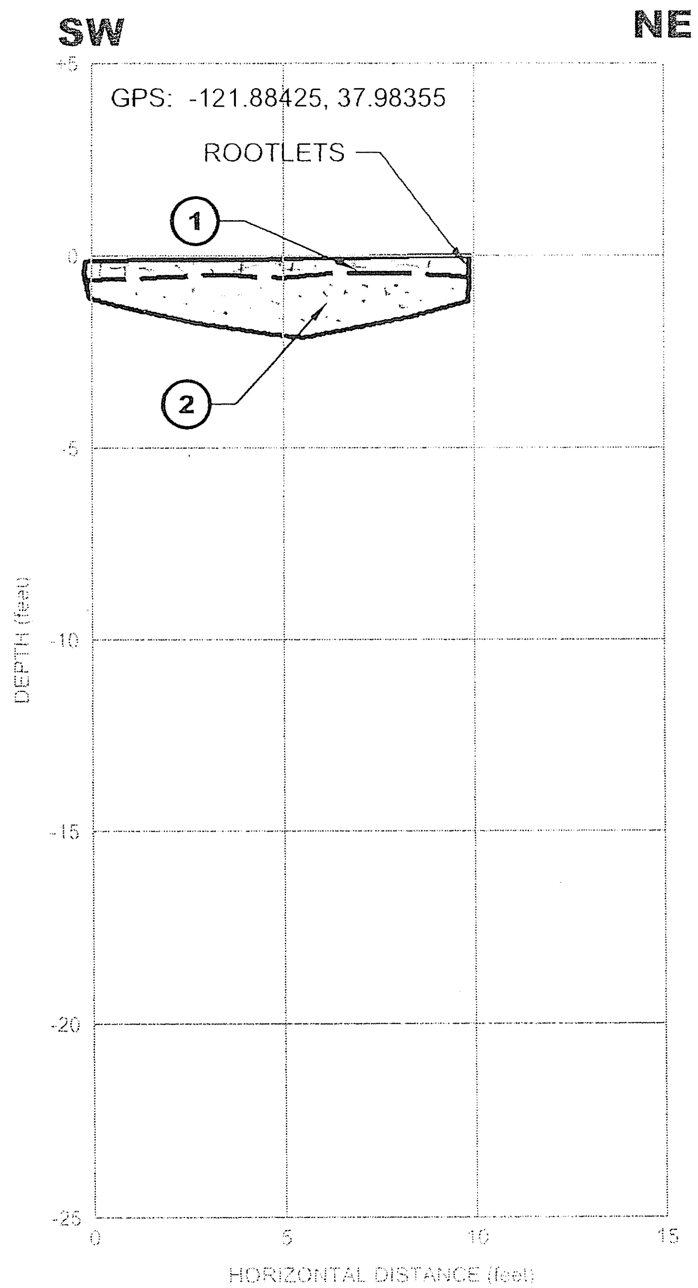
(13) **SANDSTONE** - brown to tan, moist, weathered, weak, medium grained, lightly fractured (Cierbo Formation)

LOGGED BY: R. Roatch, July 9-12, 2007

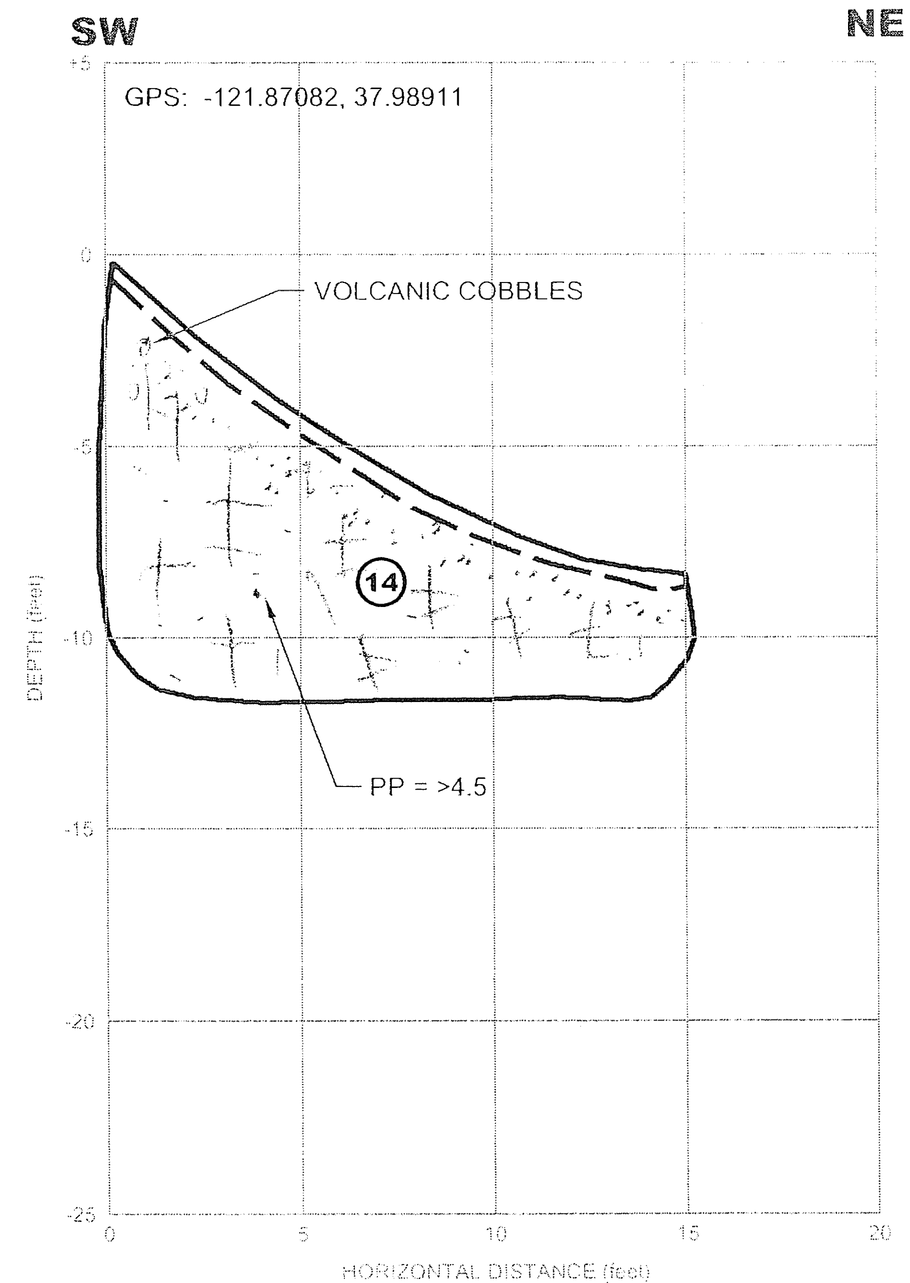
DRAWN BY: LGS	**TEST PITS TP-13 AND TP-14**		KLEINFELDER	PLATE
REVISED BY:			981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	**B-8**
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA			
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856	FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 10 Jan 2008, 2:01pm, jsala

TP-15
N40°W, SW WALL



TP-16
N84°W, SW WALL



(1) **CLAY with SAND and SILT (CL)** - dark brown, moist (dry near top), hard, stained with white prismatic evaporate material (Colluvium)

(2) **SANDSTONE** - bluish-gray, closely fractured, very slightly weathered, strong (Neroly Formation)

(14) **CLAYSTONE** - brown to tan, moist, very stiff (Tulare Formation). Note: Easy digging

NOTE: Bedrock not encountered in TP-15.

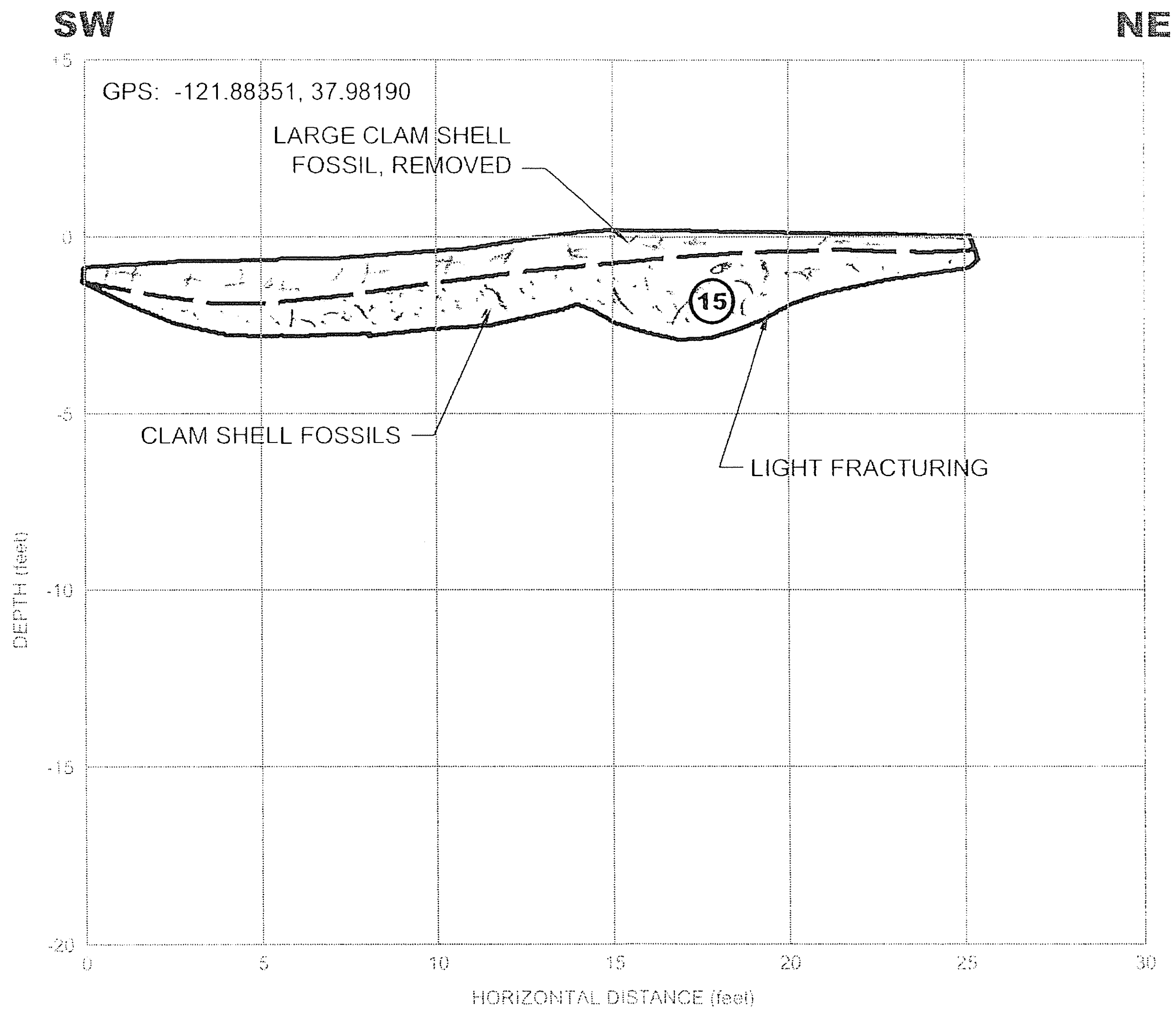
LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS	TEST PITS TP-15 AND TP-16	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	B-9
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 11 Jan 2008, 4:19pm, jsala

ATTACHED XREFS: XRef: Style A_11x17
File-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICS\PWGEO\TP and Trench Logs\ LAYOUT: TP-15_TP-16

TP-17
N88°E, NW WALL



(1) **CLAY with SAND (CL)** - dark brown, moist (dry near top), hard, with silt, stained with white prismatic evaporate material (Colluvium)

(15) **SANDSTONE** - light greenish-gray, dry, hard, fossiliferous, light weathering near surface (Cierbo)

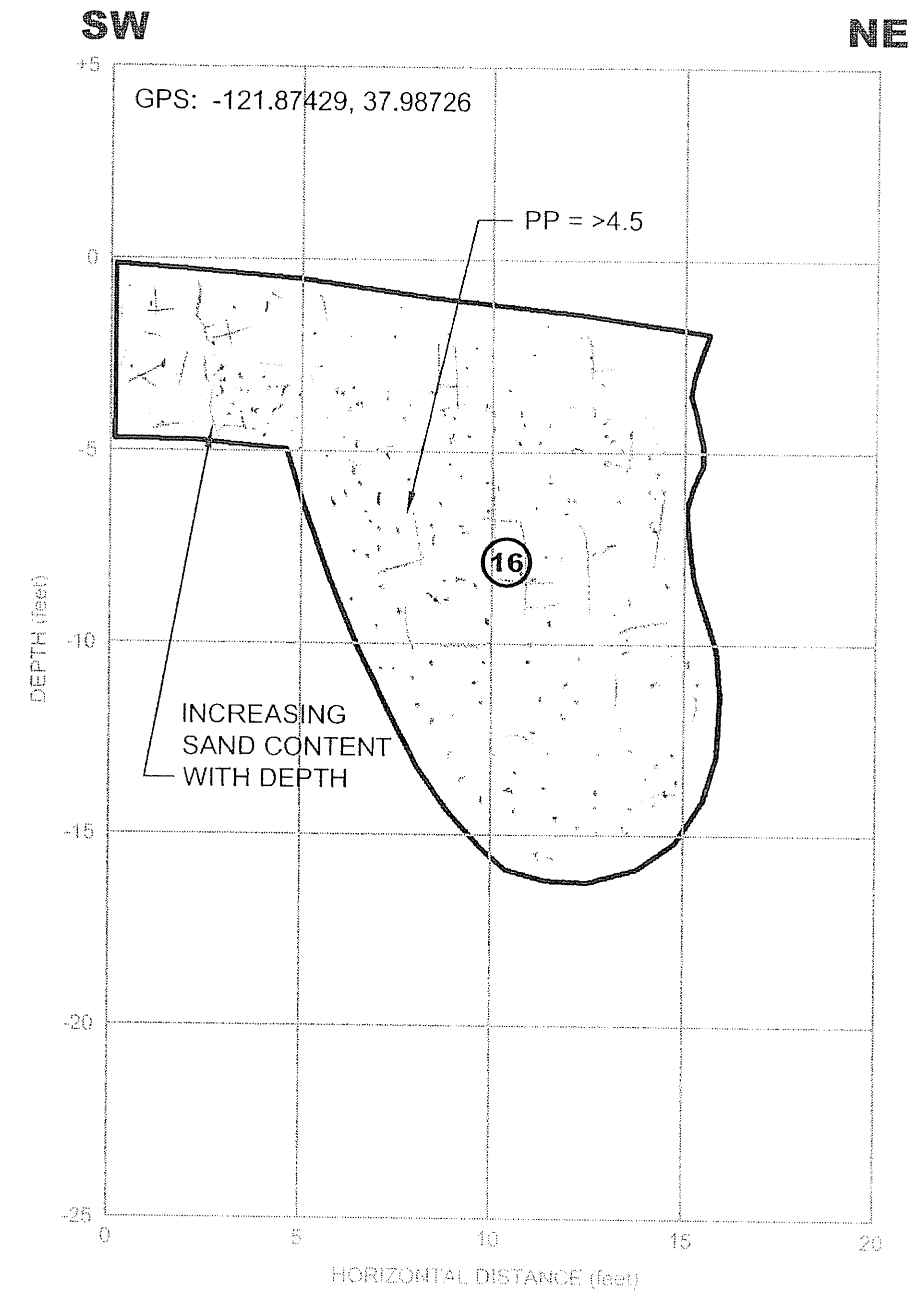
LOGGED BY: R. Roatch, July 9-12, 2007

ATTACHED XREFS: XRef: Style A_11x17 LAYOUT: TP-17
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICS\PWGEO\TP and Trench Logs\

DRAWN BY: LGS REVISED BY: CHECKED BY: LS	**TEST PIT TP-17**	KLEINFELDER	PLATE
DATE: JAN 2008 / APPROVED BY:	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	**B-10**
	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 11 Jan 2008, 4:20pm, jsala

TP-18
N21°E, SE WALL



(16) **SANDY CLAYSTONE** - tan to brown, with dark-colored rootlets, shrinkage cracks in top 2.5 feet, moist below 2.5 feet, very stiff (Tulare Formation)

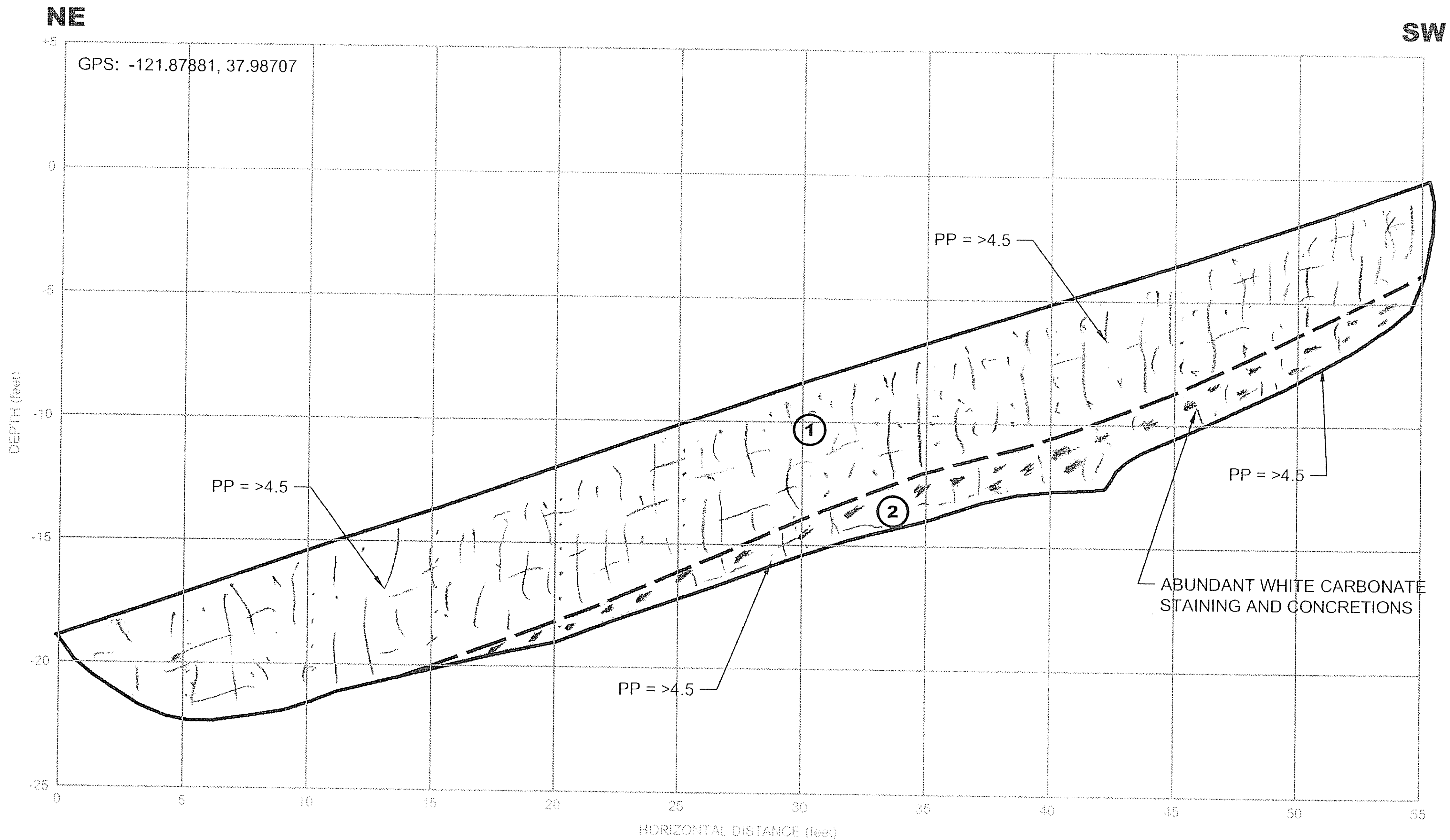
LOGGED BY: R. Roatch, July 9-12, 2007

ATTACHED XREFS: XRef: Style A 11x17
Ple-L:\2007\07PROJ CAD FILE: D:_PROJECTS\75856\GRAPHICs\PWGEO\TP and Trench Logs\ LAYOUT: TP-18

DRAWN BY: LGS	**TEST PIT TP-18**	KLEINFELDER	PLATE
REVISED BY:		981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	**B-11**
CHECKED BY: LS	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 10 Jan 2008, 2:03pm, jsala

F-1
N82°E, SE WALL



(1) **CLAY with SAND (CL)** - dark brown, moist (dry near top), hard, with silt stained with white prismatic evaporate material (Colluvium)

(2) **CLAYSTONE** - yellow-brown, moist, highly to completely weathered, weak, plastic, abundant carbonate staining and veinlets (Tulare Formation)

LOGGED BY: R. Roatch, July 9-12, 2007

DRAWN BY: LGS		**FAULT INVESTIGATION TRENCH F-1**	KLEINFELDER 981 Garcia Avenue, Suite A Pittsburg, CA 94565 PH. (925) 427-6477 FAX. (925) 427-6478 www.kleinfelder.com	PLATE **B-13**
REVISED BY:				
CHECKED BY: LS		BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA		
DATE: JAN 2008	APPROVED BY:	PROJECT NO. 75856 / FILE NAME: T-TP_LOGS.dwg		

PLOTTED: 10 Jan 2008, 2:05pm, jsala

ATTACHED XREFS: XRef: Style A 11x17
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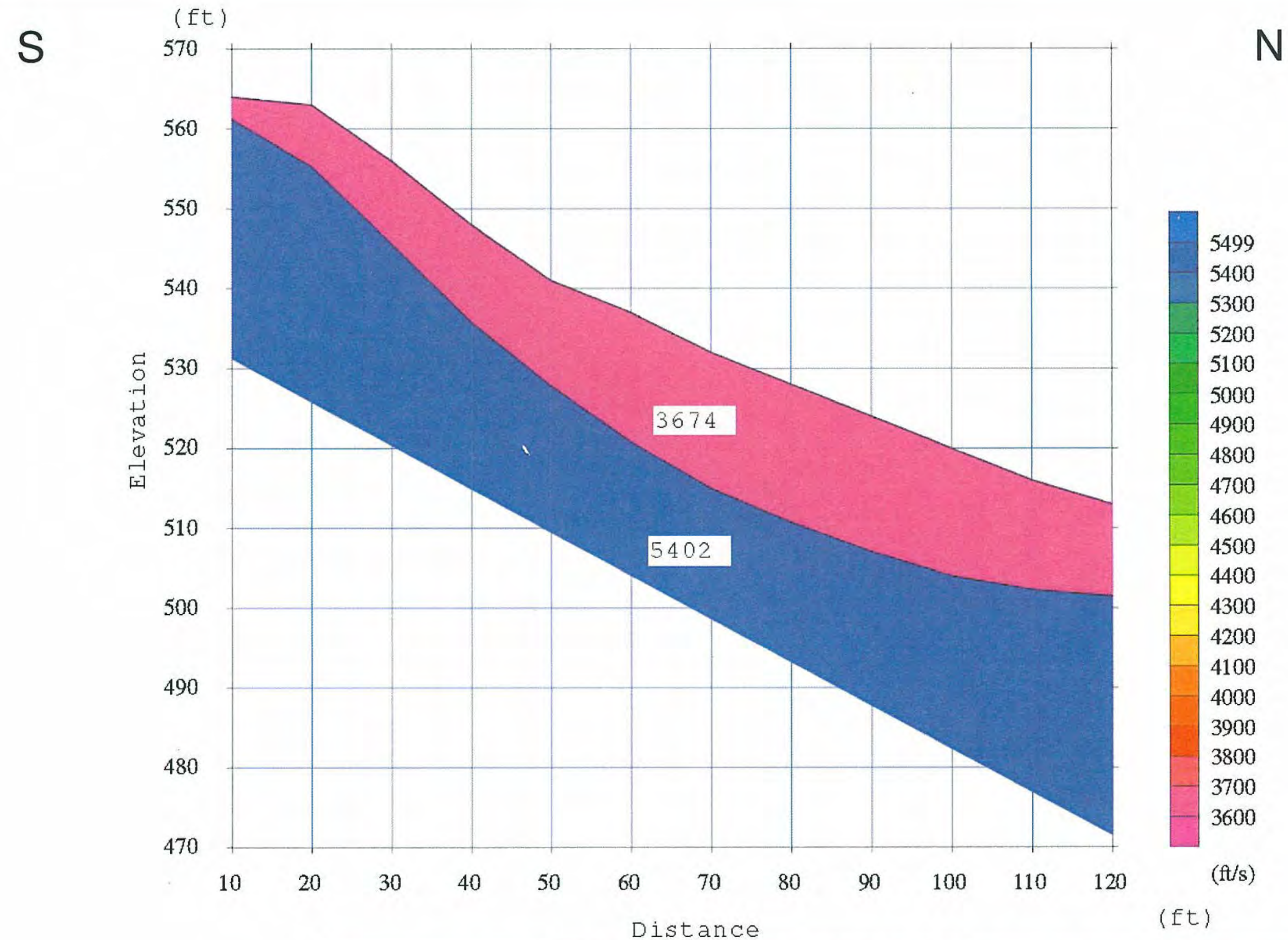
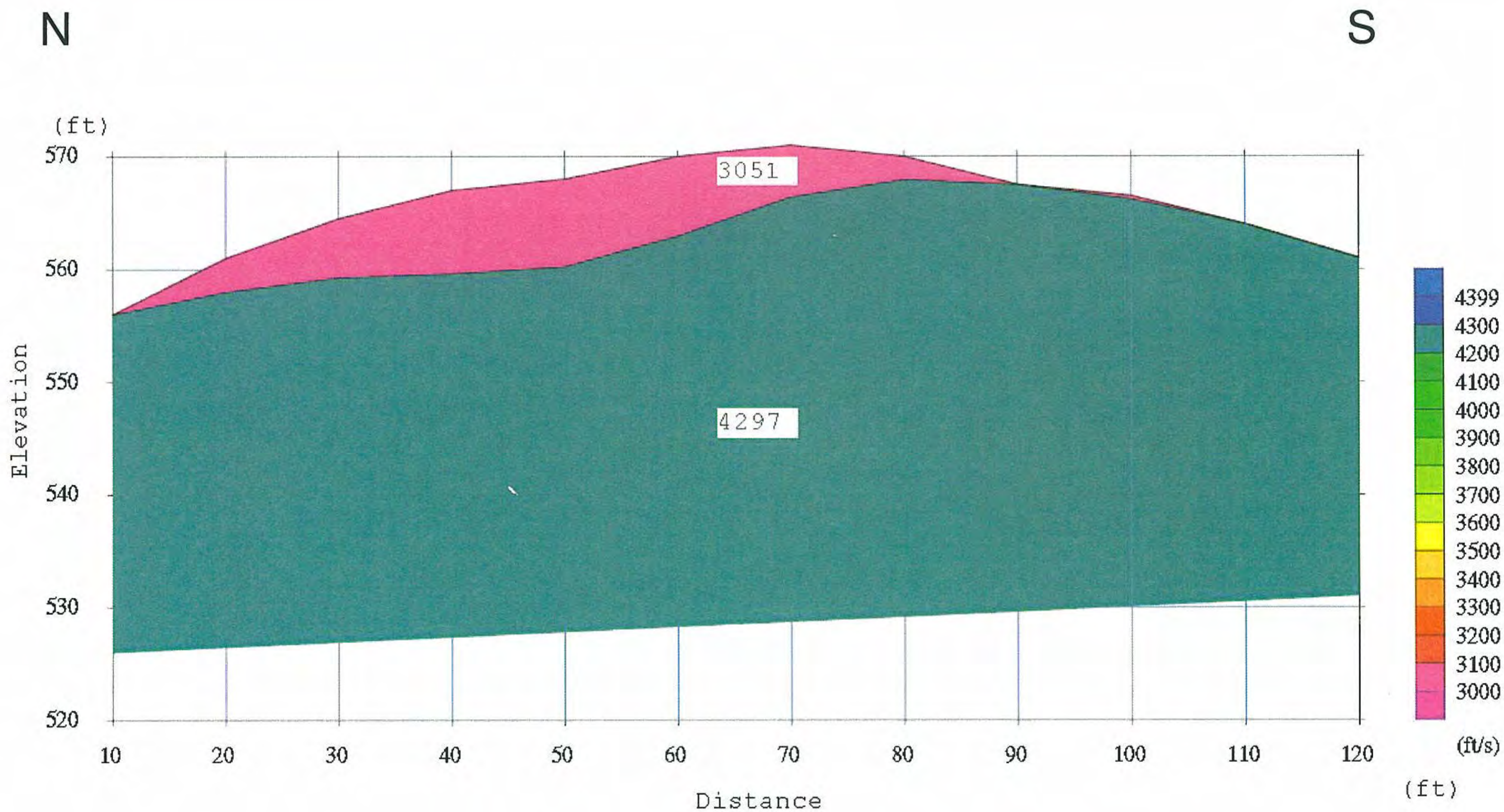
APPENDIX C



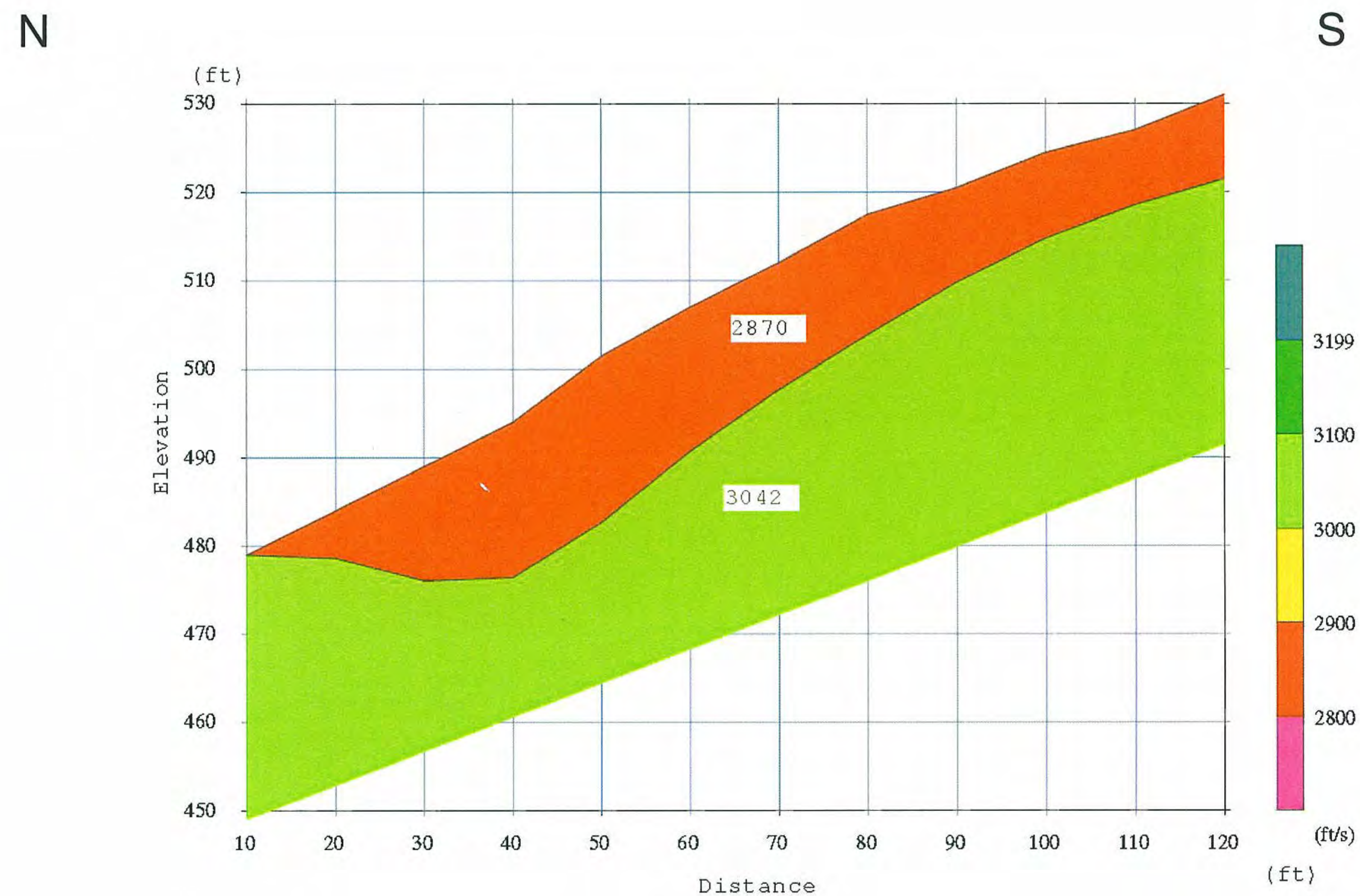
DRAWN BY: M. Clark REVISED BY:	**SEISMIC-REFRACTION SURVEY** **Line SRL-1**	**KLEINFELDER** 981 Garcia Avenue, Suite A Pittsburg, California 94565 Ph. (925) 427-6477 Fax. (925) 427-6478 www.kleinfelder.com	**PLATE** **C-1**
CHECKED BY: M. Majchrzak DATE: July 2007 / APPROVED BY: M. Majchrzak	Buchanan Road Bypass Pittsburg, California PROJECT NO: 75856 / FILE NAME:11 x 8½ plates.pdf		

PLOTTED: 14 January 2008, 10:27 AM



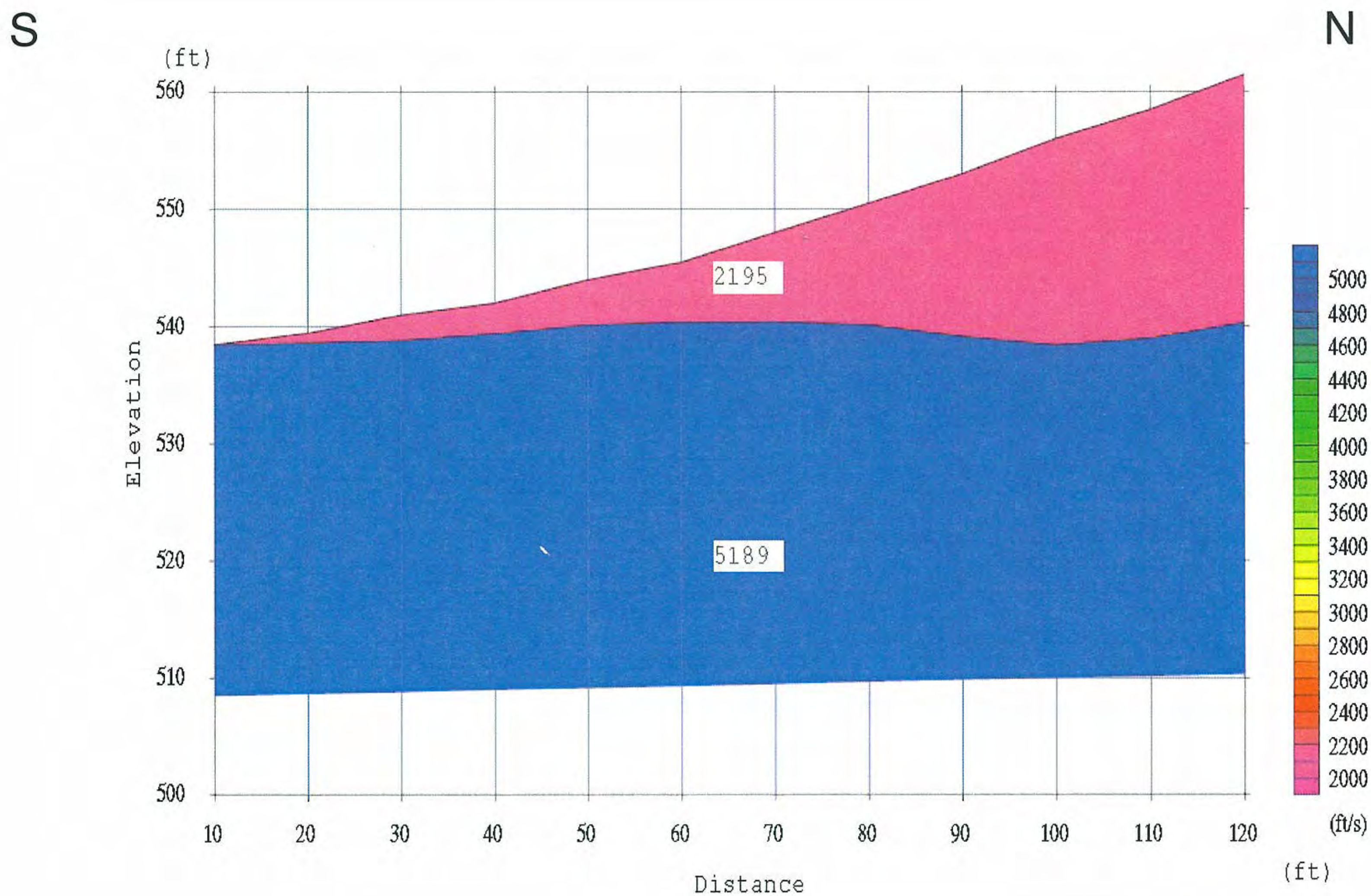
DRAWN BY: M. Clark REVISED BY:	**SEISMIC-REFRACTION SURVEY** **Line SRL-2**	KLEINFELDER	**PLATE**
CHECKED BY: M. Majchrzak DATE: July 2007 / APPROVED BY: M. Majchrzak	Buchanan Road Bypass Pittsburg, California PROJECT NO: 75856 / FILE NAME:11 x 8½ plates.pdf	981 Garcia Avenue, Suite A Pittsburg, California 94565 Ph. (925) 427-6477 Fax. (925) 427-6478 www.kleinfelder.com	**C-2**

PLOTTED: 14 January 2008, 10:27 AM



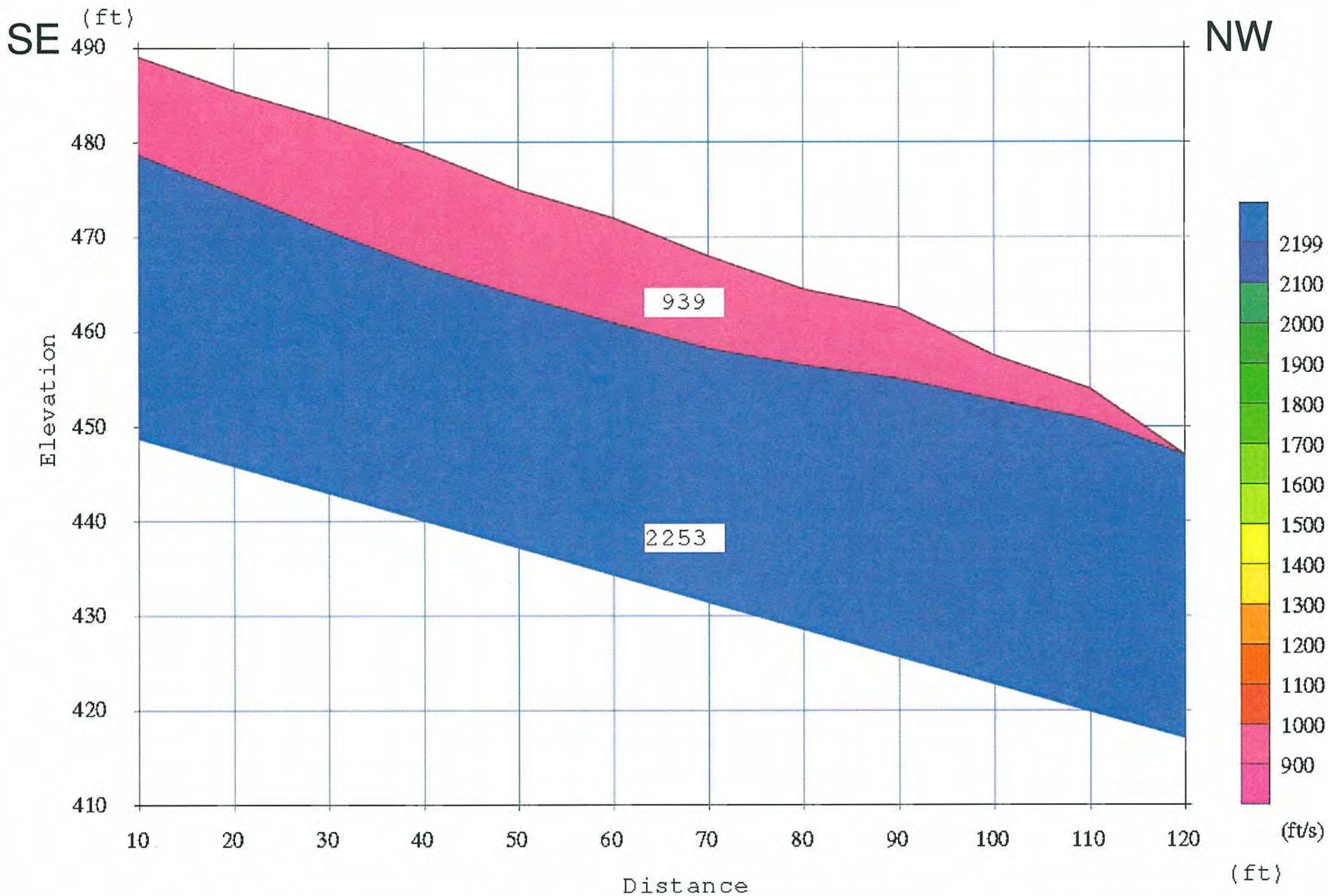
DRAWN BY: M. Clark REVISED BY: CHECKED BY: M. Majchrzak DATE: July 2007 APPROVED BY: M. Majchrzak	**SEISMIC-REFRACTION SURVEY** **Line SRL-3** Buchanan Road Bypass Pittsburg, California PROJECT NO: 75856 FILE NAME:11 x 8½ plates.pdf	KLEINFELDER 981 Garcia Avenue, Suite A Pittsburg, California 94565 Ph. (925) 427-6477 Fax. (925) 427-6478 www.kleinfelder.com	**PLATE** **C-3**

PLOTTED: 14 January 2008, 10:27 AM



DRAWN BY: M. Clark REVISED BY:	**SEISMIC-REFRACTION SURVEY** **Line SRL-4**	**KLEINFELDER** 981 Garcia Avenue, Suite A Pittsburg, California 94565 Ph. (925) 427-6477 Fax. (925) 427-6478 www.kleinfelder.com	**PLATE** **C-4**
CHECKED BY: M. Majchrzak DATE: July 2007 APPROVED BY: M. Majchrzak	Buchanan Road Bypass Pittsburg, California PROJECT NO: 75856 FILE NAME:11 x 8½ plates.pdf		

PLOTTED: 14 January 2008, 10:27 AM



DRAWN BY: M. Clark REVISED BY:	**SEISMIC-REFRACTION SURVEY** **Line SRL-5**	KLEINFELDER	**PLATE**
CHECKED BY: M. Majchrzak DATE: July 2007 / APPROVED BY: M. Majchrzak	Buchanan Road Bypass Pittsburg, California	981 Garcia Avenue, Suite A Pittsburg, California 94565 Ph. (925) 427-6477 Fax. (925) 427-6478 www.kleinfelder.com	**C-5**
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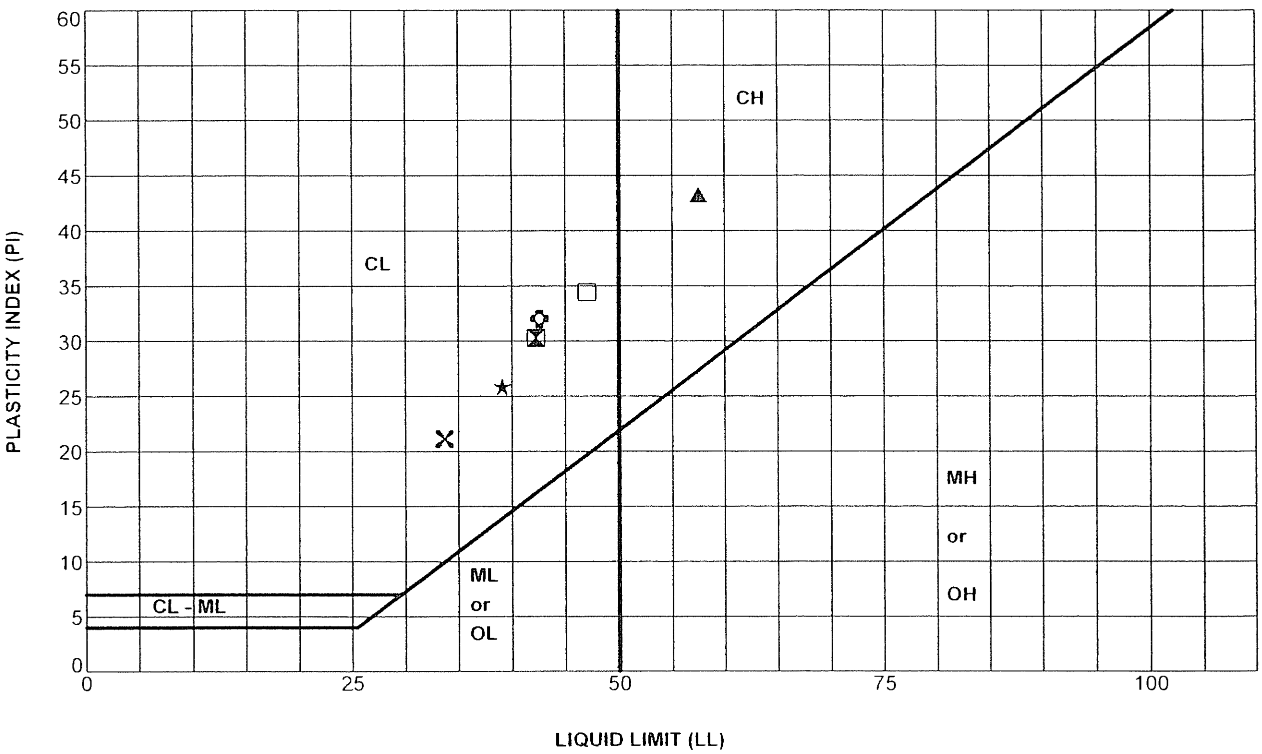
PLOTTED: 14 January 2008, 10:27 AM

APPENDIX D



SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
□	B-1	15.5	47	13	34	Olive-Brown Clay with Sand (CL)
⊠	B-2	20.5	42	12	30	Olive-Brown Sandy Clay (CL)
▲	B-6	8.5	58	14	44	Brown Claystone
★	B-6	23.5	39	13	26	Yellow-Brown Claystone
✕	B-7	15.5	34	13	21	Yellow-Brown Clayey Sand (SC)
✿	B-8	10.5	43	11	32	Dark Brown Sandy Clay (CL)

Unified Soil Classification

Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (DRY PREP)

KLEINFELDER

PROJECT NO. 75856-PWGEO

ATTERBERG LIMITS*

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-1

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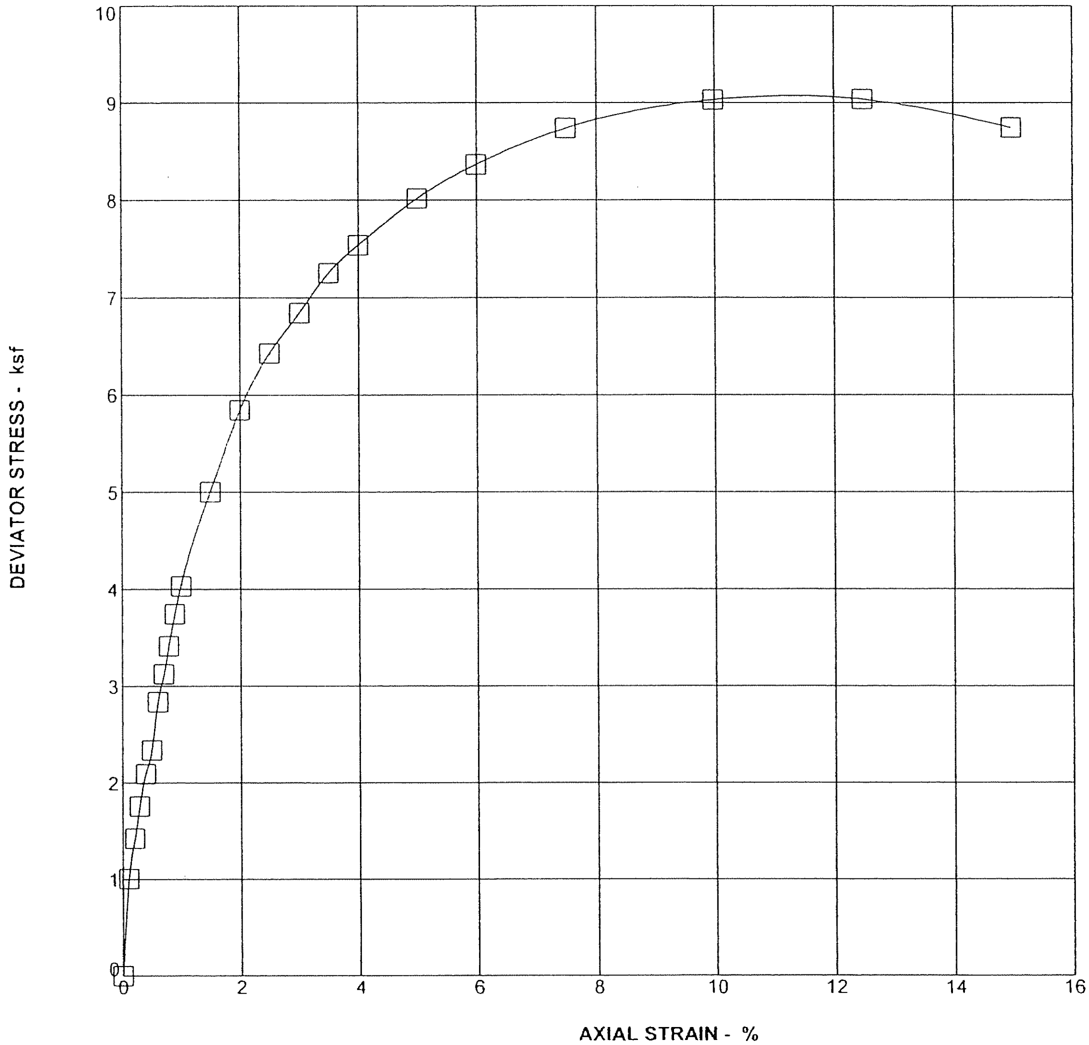

BORING NO.	B-6	**DRY DENSITY - pcf**	101
DEPTH - ft	12.0	**WATER CONTENT - %**	18.4
SAMPLE DESCRIPTION	Brown Claystone		
CONFINING STRESS - ksf	2.4		

MAXIMUM DEVIATOR STRESS= 9.04 ksf at 12.5 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850

KLEINFELDER

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-2

PROJECT NO. *75856-PWGEO*

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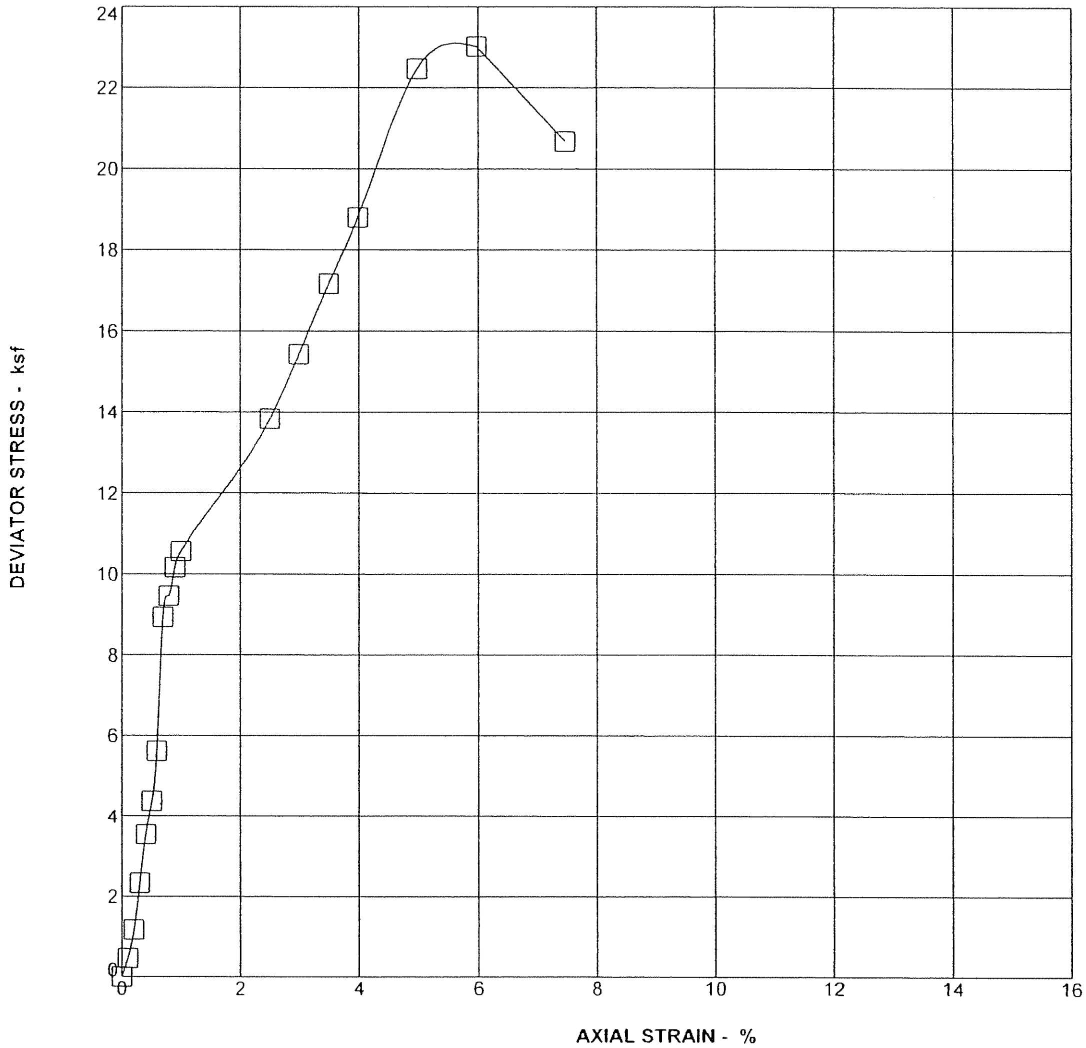

BORING NO.	B-6	**DRY DENSITY - pcf**	121
DEPTH - ft	28.0	**WATER CONTENT - %**	11.9
SAMPLE DESCRIPTION	Yellow Brown Claystone		
CONFINING STRESS - ksf	4.0		

MAXIMUM DEVIATOR STRESS= 23.02 ksf at 6.0 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850

KLEINFELDER

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-3

PROJECT NO. *75856-PWGEO*

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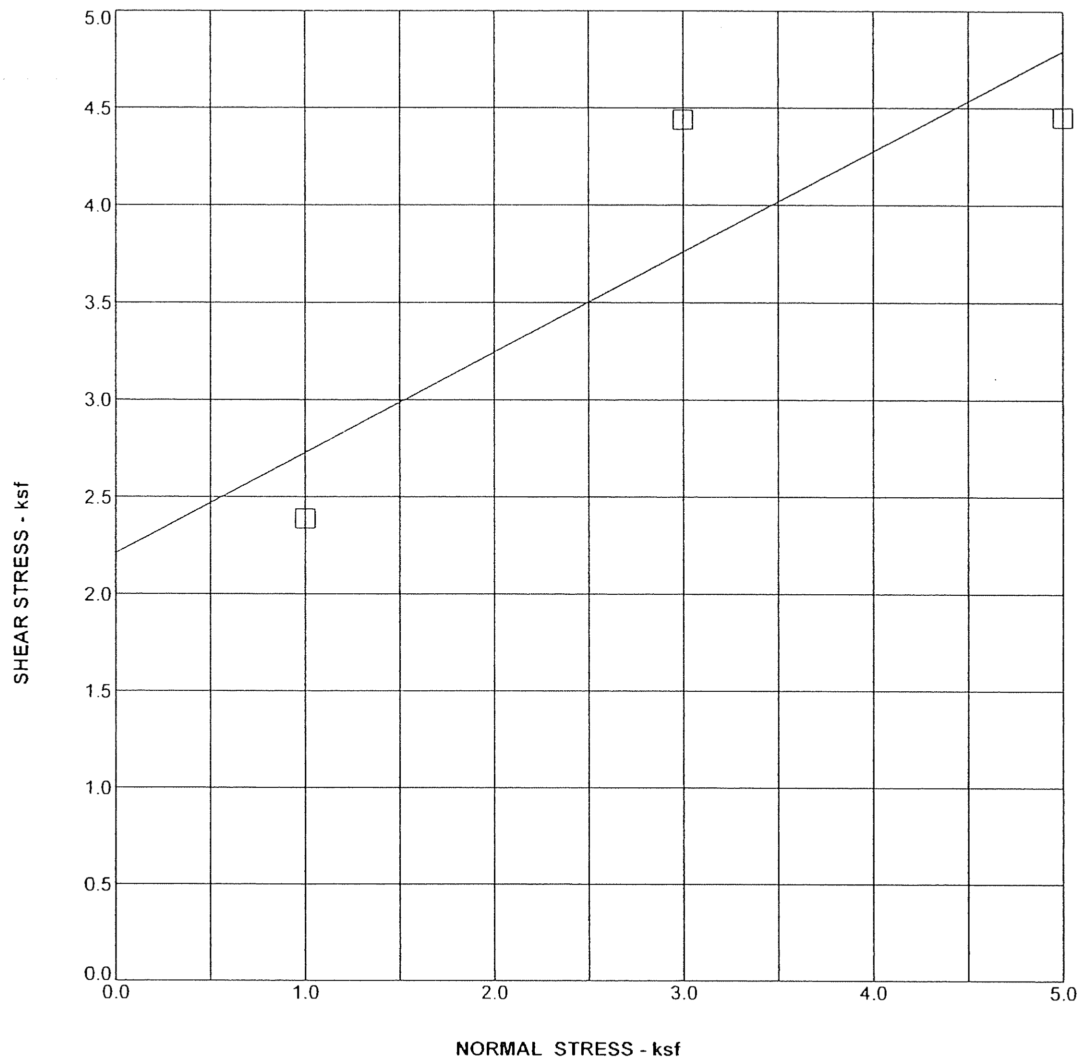

BORING NO: C-2

DEPTH: 7.0 ft

SAMPLE DESCRIPTION: Olive Sandstone

DRY DENSITY - pcf	103	101	98
INITIAL WATER CONTENT - %	17.1	16.5	16.2
FINAL WATER CONTENT - %	24.0	23.4	23.2
NORMAL STRESS - psf	1000	3000	5000
MAXIMUM SHEAR - psf	2388	4440	4452

FRICTION ANGLE = 27 degrees

COHESION= 2.21 ksf

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

KLEINFELDER

DIRECT SHEAR*

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-4

PROJECT NO. *75856-PWGEO*

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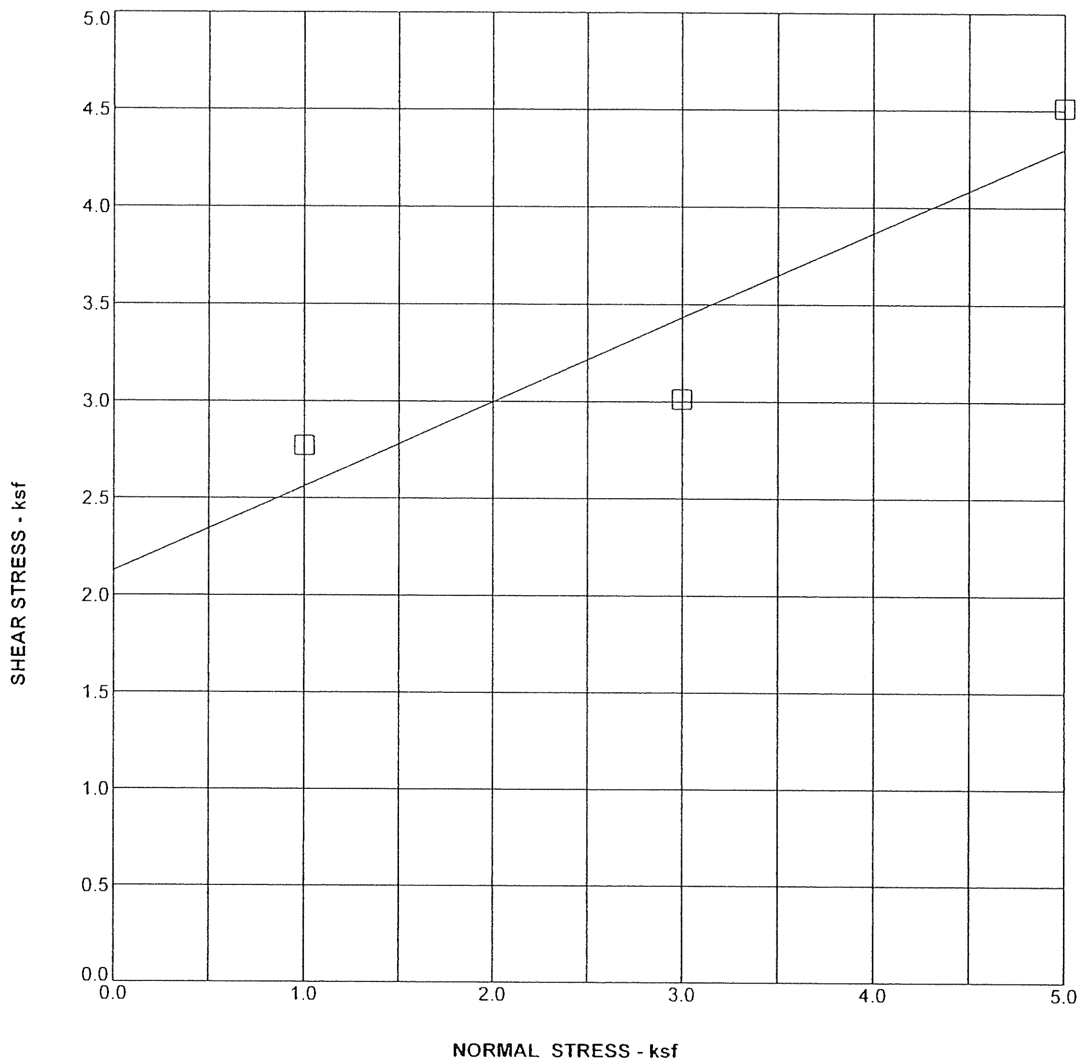

BORING NO: C-2

DEPTH: 27.0 ft

SAMPLE DESCRIPTION: Olive-Gray Mudstone

DRY DENSITY - pcf	68	75	86
INITIAL WATER CONTENT - %	48.6	41.7	30.1
FINAL WATER CONTENT - %	59.8	51.6	36.6
NORMAL STRESS - psf	1000	3000	5000
MAXIMUM SHEAR - psf	2772	3012	4512

FRICTION ANGLE = 24 degrees

COHESION= 2.13 ksf

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

KLEINFELDER

DIRECT SHEAR*

PLATE

D-5

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PROJECT NO. *75856-PWGEO*

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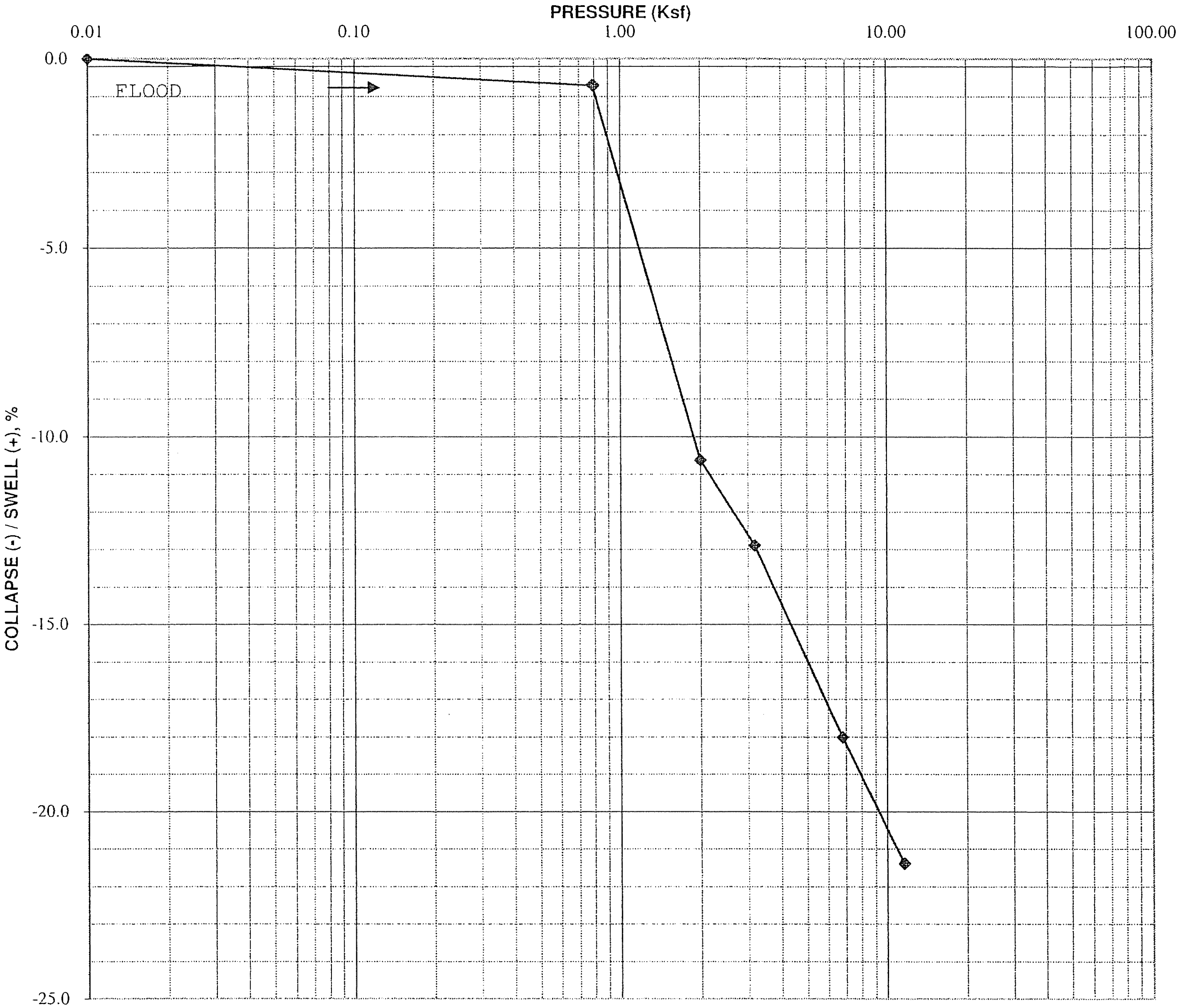


BORING NO. B-5

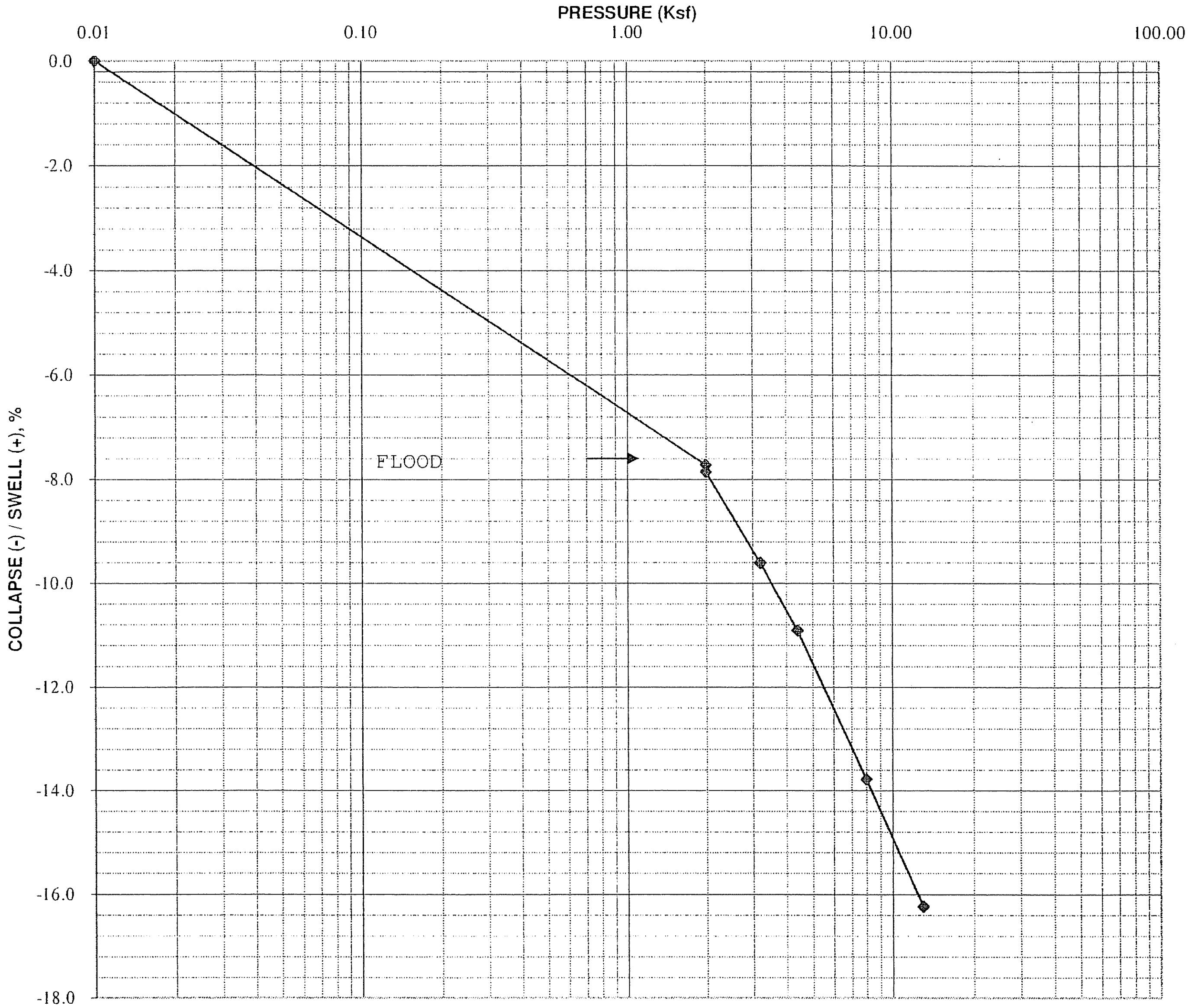
DEPTH, ft 5

SAMPLE DESCRIPTION Brown Sandy Clay (CL)

	INITIAL	FINAL
DRY DENSITY, psf	85.6	108.4
WATER CONTENT, %	15.1	22.9
SAMPLE HEIGHT, in.	0.6750	0.5306

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

KLEINFELDER	**ONE DIMENSIONAL SWELL***	*PLATE*
Project No.: *75856*	BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	**D-6**



BORING NO. B-7

DEPTH, ft 16

SAMPLE DESCRIPTION Yellow Brown Clayey Sand (SC)

	INITIAL	FINAL
DRY DENSITY, psf	88.6	105.0
WATER CONTENT, %	29.7	27.5
SAMPLE HEIGHT, in.	0.6730	0.5637

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

KLEINFELDER

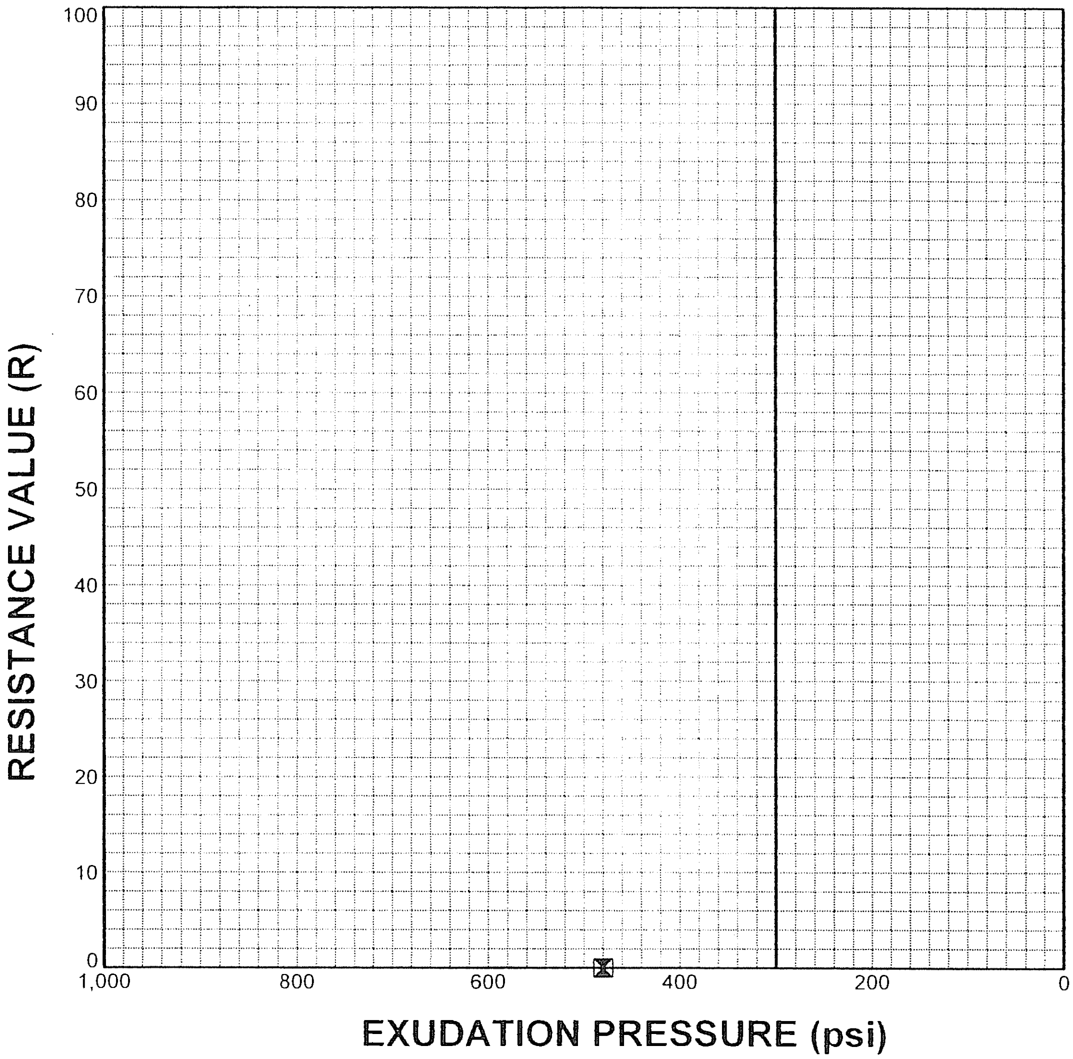
Project No.: *75856*

ONE DIMENSIONAL SWELL*

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-7



SPECIMEN NO.	⊠		
MOISTURE CONTENT (%)	23.7		
DRY DENSITY (PCF)	97.6		
EXUDATION PRESSURE (PSI)	480		
EXPANSION PRESSURE (PSF)	65		
RESISTANCE VALUE (R)	0		

Date Received: 8/2/2007

SAMPLE SOURCE	CLASSIFICATION	SAND EQUIVALENT	EXPANSION PRESSURE	R-VALUE
(PL11308-1)	Brown Clayey Silt	---	---	<5

NOTE: Sample No.: B-2

ASTM D 2844, Cal Test 301

KLEINFELDER

PROJECT NO. 75856-PWGEO

RESISTANCE VALUE TEST DATA

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-8

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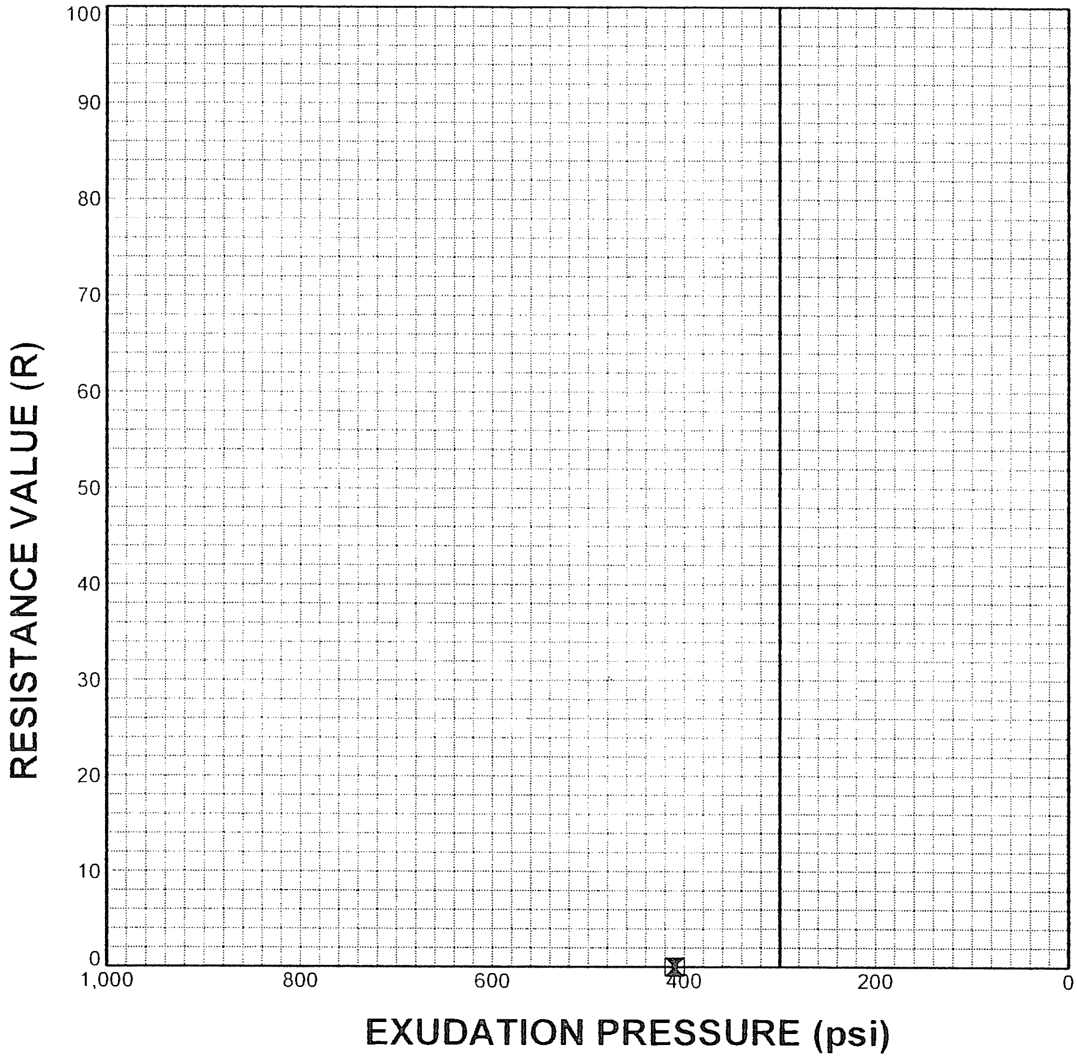

SPECIMEN NO.	⊠		
MOISTURE CONTENT (%)	23.2		
DRY DENSITY (PCF)	98.7		
EXUDATION PRESSURE (PSI)	410		
EXPANSION PRESSURE (PSF)	61		
RESISTANCE VALUE (R)	0		

Date Received: 8/2/2007

SAMPLE SOURCE	CLASSIFICATION	SAND EQUIVALENT	EXPANSION PRESSURE	R-VALUE
(PL11308-2)	Gray Brown Clayey Silt	---	---	<5

NOTE: Sample No. B-4

ASTM D 2844, Cal Test 301

KLEINFELDER

RESISTANCE VALUE TEST DATA

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

PLATE

D-9

PROJECT NO. 75856-PWGEO

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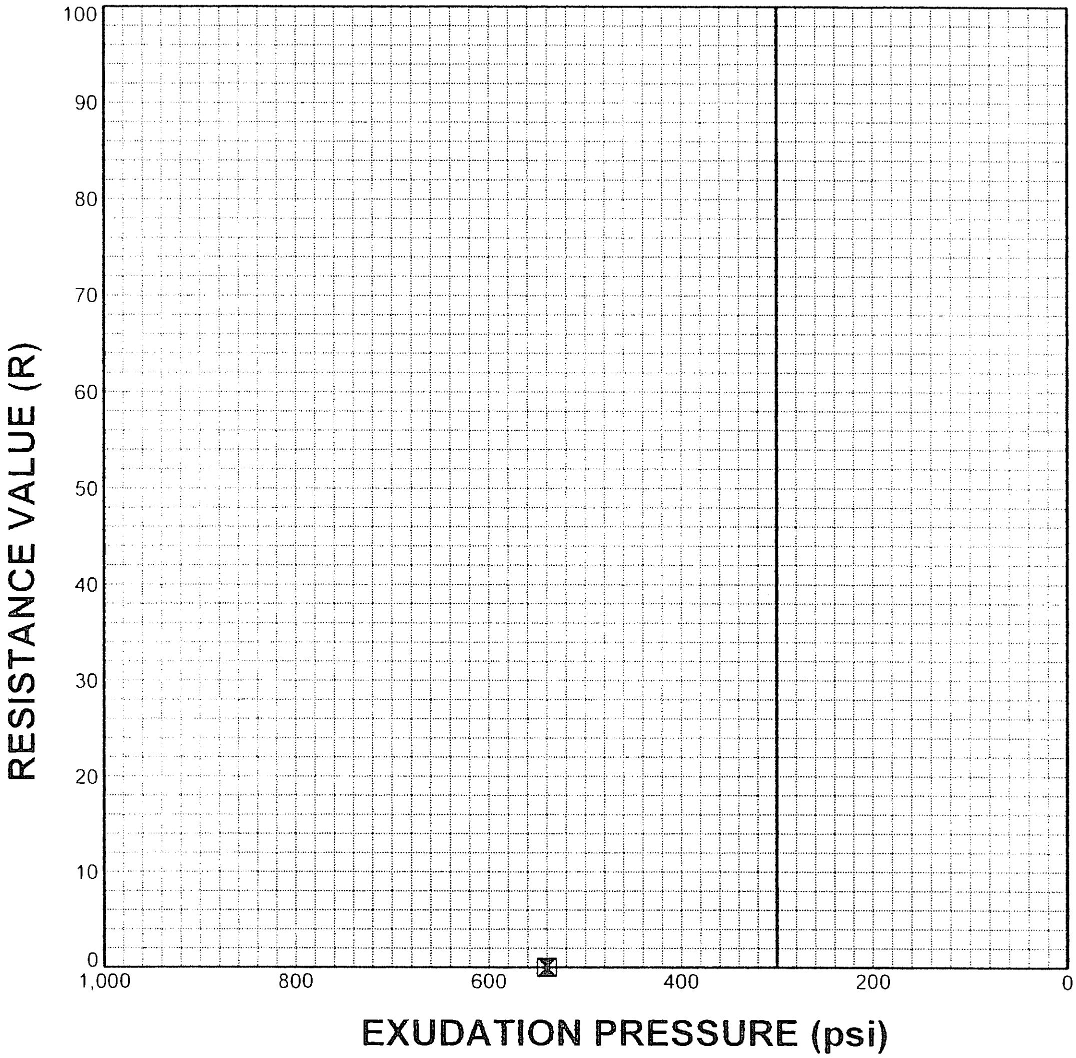

SPECIMEN NO.	⊠		
MOISTURE CONTENT (%)	22.6		
DRY DENSITY (PCF)	100.0		
EXUDATION PRESSURE (PSI)	540		
EXPANSION PRESSURE (PSF)	43		
RESISTANCE VALUE (R)	0		

Date Received: 8/2/2007

SAMPLE SOURCE	CLASSIFICATION	SAND EQUIVALENT	EXPANSION PRESSURE	R-VALUE
(PL11308-3)	Dark Brown Clayey Silt	---	---	<5

NOTE: Sample No. B-5

ASTM D 2844, Cal Test 301

KLEINFELDER	**RESISTANCE VALUE TEST DATA** BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	*PLATE* **D-10**
PROJECT NO. 75856-PWGEO		

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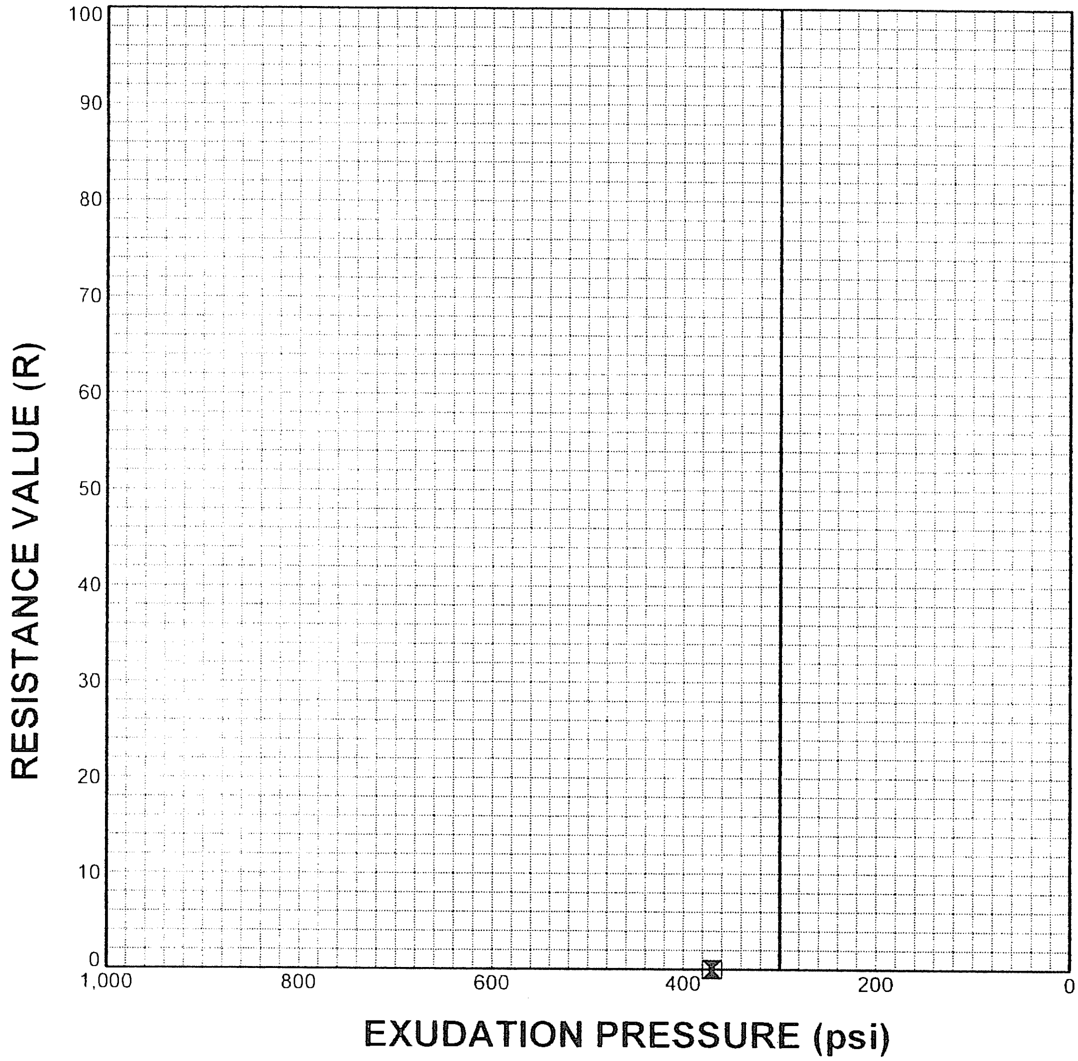

SPECIMEN NO.	⊠		
MOISTURE CONTENT (%)	24.3		
DRY DENSITY (PCF)	96.2		
EXUDATION PRESSURE (PSI)	370		
EXPANSION PRESSURE (PSF)	74		
RESISTANCE VALUE (R)	0		

Date Received: 8/2/2007

SAMPLE SOURCE	CLASSIFICATION	SAND EQUIVALENT	EXPANSION PRESSURE	R-VALUE
(PL11308-4)	Dark Brown Clayey Silt	---	---	<5

NOTE: Sample No. B-8

ASTM D 2844, Cal Test 301

RESISTANCE VALUE TEST DATA

BUCHANAN ROAD BYPASS
PITTSBURG, CALIFORNIA

KLEINFELDER

PLATE

D-11

PROJECT NO. 75856-PWGEO

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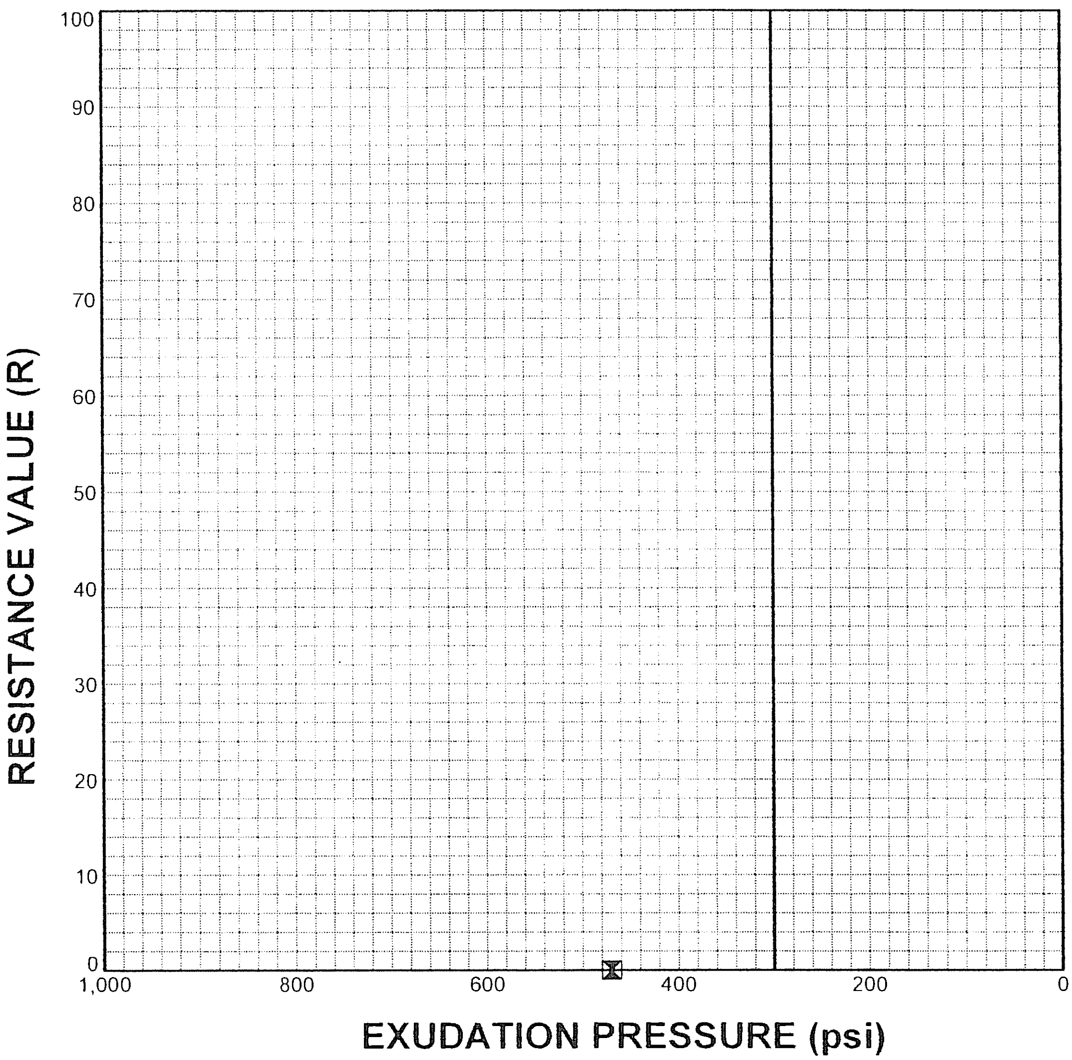

SPECIMEN NO.	☒		
MOISTURE CONTENT (%)	25.7		
DRY DENSITY (PCF)	95.6		
EXUDATION PRESSURE (PSI)	470		
EXPANSION PRESSURE (PSF)	78		
RESISTANCE VALUE (R)	0		

Date Received: 8/2/2007

SAMPLE SOURCE	CLASSIFICATION	SAND EQUIVALENT	EXPANSION PRESSURE	R-VALUE
(PL11308-5)	Yellow Brown Clay (Tulare)	---	---	<5

NOTE: Sample No. C (Tulare Composite)

ASTM D 2844, Cal Test 301

KLEINFELDER	RESISTANCE VALUE TEST DATA BUCHANAN ROAD BYPASS PITTSBURG, CALIFORNIA	PLATE **D-12**
PROJECT NO. 75856-PWGEO		

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APPENDIX E

California State Certified Laboratory No.2153

CERCO analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

8 October, 2007

Job No.0709151
Cust. No.10527

Mr. King Wong
Kleinfelder
7133 Koll Center Parkway
Pleasanton, CA 94566

Subject: Project No.: 75856-PWGEO
Project Name: Buchanan Road
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Wong:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on September 20, 2007. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Samples No.001, No.002 and No.004 are classified as "corrosive" and Sample No.003 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration ranges from none detected to 66 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration ranges from 24 to 300 mg/kg and are determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soils range from 6.9 to 7.6 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 440 to 460-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No.2153

CERCO analytical, inc.
3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 75856-PWGEO
Client's Project Name: Buchanan Road
Date Sampled: Not Indicated
Date Received: 20-Sep-07
Matrix: Soil
Authorization: Signed Chain of Custody and P.O. No.R9895

Date of Report: 8-Oct-2007

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0709151-001	B-7	440	7.6	-	1,600	-	N.D.	24
0709151-002	B-1	450	7.4	-	3,400	-	21	25
0709151-003	B-3	460	6.9	-	1,400	-	66	300
0709151-004	B-6	440	7.5	-	1,500	-	N.D.	28

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	21-Sep-2007	24-Sep-2007	-	5-Oct-2007	-	25-Sep-2007	25-Sep-2007

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMiller
Cheryl McMiller
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Appendix E.2

Engineering Geologic and Geotechnical Feasibility Report: Four Proposed James Donlon Boulevard Extension Alternatives (2012)

KLEINFELDER
Bright People. Right Solutions.

Prepared for **RBF Consulting**

ENGINEERING GEOLOGIC AND GEOTECHNICAL FEASIBILITY REPORT FOUR PROPOSED JAMES DONLON BOULEVARD ALIGNMENT EXTENSION ALTERNATIVES PITTSBURG, CALIFORNIA

March 7, 2012
File No.: 123561/PWGEO

KLEINFELDER
Bright People. Right Solutions.

March 7, 2012
File No. 123561/PWGEO

Mr. William J. Conyers, Senior Associate
RBF Consulting
500 Ygnacio Valley Road, Suite 270
Walnut Creek, California 94596-3847
wconyers@rbf.com

SUBJECT: Engineering Geologic and Geotechnical Feasibility Report for the Four Proposed James Donlon Boulevard Alignment Extension Alternatives, in Pittsburg, California

Dear Mr. Conyers:

We are pleased to submit four copies of our engineering geologic and geotechnical feasibility evaluation report for the four proposed James Donlon Boulevard alignment extension alternatives planned along the southern hilly portion of Pittsburg, California.

Kleinfelder initially issued a draft report on January 31, 2012 titled Engineering Geologic and Geotechnical Feasibility Report for the Three Proposed James Donlon Boulevard Alignment Extension Alternatives, in Pittsburg, California (File No. 123561/PWGEO [PLE12R008]). The three alignments evaluated in the January 31st report were identified by RBF Consultants as the Original Alignment (C1), Middle Alignment (C2), and Northern Alignment (C3).

The above noted draft report was subsequently revised and issued on February 23, 2012 under the title Revised Draft *Engineering Geologic and Geotechnical Feasibility Report for the four Proposed James Donlon Boulevard Extension Alternatives, in Pittsburg, California* (File No. 123561/PWGEO [PLE12R008]. The revision was made to include a fourth alignment alternative (Middle C2-Low), which has been added for consideration by RBF Consulting after the issuance of the January 31st report. The location of the added alignment (C2-Low) generally matched that of the Middle Alignment (C2) except that its overall elevation is lower. We understand that the lower elevations for the fourth added Middle Alignment (C2-Low) were developed to achieve a balance between cut and fill earth material volumes to avoid importing fill materials.

We are finalizing our February 23, 2012 report herein without making any significant changes or modifications to its technical content or plates.

This report presents the results of our current feasibility assessment which was performed in accordance with our scope of work presented in our proposal dated November 10, 2011. Some of the geological and geotechnical constraints evaluated during this investigation include faulting and seismicity, fault-related ground surface rupture, landslide deposits, soil types, expansive soils, soil corrosion, erosion and drainage, liquefaction and lateral spreading, dynamic compaction, flooding, naturally-occurring asbestos, and adverse bedrock bedding.

This assessment indicates that the four currently proposed alignment alternatives: Original Alignment (C1), Middle Alignment (C2), Middle Alignment (C2-Low), and Northern Alignment (C3) are feasible from an engineering geologic and geotechnical perspective provided that the roadway alignment to be selected is designed and constructed in accordance with our recommendations, which will be provided in an upcoming updated geological and geotechnical investigation report once an alignment is selected by the City of Pittsburg (City) and RBF Consulting (RBF). This assessment recommends the selection of the Middle Alignment (C2-Low) alternative, which should allow for balanced cut/fill volume quantities and hence avoid or minimize the need for importing fill soils during the grading of the project. In our opinion, this alternative encounters the least engineering geologic and geotechnical constraints compared to the other assessed alternatives. These constraints are associated with the site topography and underlying geologic formations to be encountered, landslide deposits, bedrock rippability, adverse bedrock bedding, magnitude of associated grading, and anticipated stability of cut slopes in the various bedrock formations.

We appreciate the opportunity of providing our services to you on this project and trust that this report meets your needs at this time. If you have any questions concerning the information presented, please contact Sadek Derrega at (925) 484-1700 or Fernando Silva at (925) 427-6477.

Sincerely,

KLEINFELDER WEST, INC.

PROFESSIONAL GEOLOGIST
SADEK M. DERREGA
No. 2175
Exp. 03/31/13
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Sadek M. Derrega, PG, CEG #2175
Principal Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
CRISTIANO B.C. DE MELO
No. 2756
Exp. 09-30-12
GEOTECHNICAL
STATE OF CALIFORNIA

Cristiano Melo, PE, GE#2756
Project Geotechnical Engineer

SMD/CM/jmk

ENGINEERING GEOLOGIC AND GEOTECHNICAL FEASIBILITY REPORT FOUR PROPOSED JAMES DONLON BOULEVARD ALIGNMENTEXTENSION ALTERNATIVES PITTSBURG, CALIFORNIA

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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.

ASFE
The Best People on Earth

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ENGINEERING GEOLOGIC AND GEOTECHNICAL FEASIBILITY REPORT FOUR PROPOSED JAMES DONLON BOULEVARD ALIGNMENT EXTENSION ALTERNATIVES PITTSBURG, CALIFORNIA

1 INTRODUCTION

This report presents the results of our current engineering geologic and geotechnical feasibility assessment, which was performed in accordance with our scope of work presented in our proposal dated November 10, 2011. Some of the geological and geotechnical constraints evaluated during this investigation included faulting and seismicity, fault-related ground surface rupture, landslide deposits, soil types, expansive soils, erosion and drainage, liquefaction and lateral spreading, dynamic compaction, flooding, naturally-occurring asbestos, and adverse bedrock bedding.

This assessment recommends the selection of the Middle Alignment (C2-Low) extension alternative based on the least engineering geologic and geotechnical constraints encountered, including type of underlying bedrock to be encountered, bedrock rippability, landslide deposits, adverse bedrock bedding, magnitude of associated grading, and anticipated stability of cut slopes in the various bedrock formations. In addition, the grading associated with this recommended alternative alignment should generate sufficient earth materials to balance cut/fill quantities and minimize the potential for fill soil importation.

1.1 PROJECT LOCATION AND DESCRIPTION

The general location of the proposed extension of James Donlon Boulevard is shown on Plate 1, Site Vicinity Map. This evaluation was performed to assess the feasibility of four roadway extension alignment alternatives identified herein as Original Alignment (C1), Middle Alignment (C2), Middle Alignment (C-2 Low), and Northern Alignment (C3), and to provide input to the City and RBF to select one of the four noted alignment alternatives. The four roadway alignment alternatives are approximately delineated on Plate 2, Aerial Site Plan.

The proposed James Donlon Boulevard extension project will consist of constructing an approximately 1.6-mile long section of new roadway. The new roadway will extend from Kirker Pass Road, marking the western project limit, to the western property line of the planned Sky Ranch residential development, which marks the eastern terminal point of the roadway extension alignment. The proposed alignment extension will be an east/west, limited-access arterial roadway in the undeveloped hills south of the City. We understand that east of the western property line of the Sky Ranch development the roadway alignment will be constructed by others.

Besides the roadway and associated drainage facilities, other project features associated with the proposed roadway extension will include the following:

- Five culverts along five smaller stream crossings.
- Two bridges across the Kirker Creek channel.
- Cut slopes and embankment fills with heights exceeding 170 feet.
- Several thousand linear feet of sound walls are anticipated.

The road alignment extension is anticipated to encounter geologic materials consisting of alluvium, colluvium/slope wash, active and dormant landslides, Tulare formation claystone, Lawlor Tuff, Neroly formation sandstone and siltstone, Cierbo formation sandstone, and Kirker formation tuffaceous siltstone and sandstone materials.

1.2 BACKGROUND

Kleinfelder previously prepared a report titled *Geological and Geotechnical Constraints Evaluation Report for the Proposed Buchanan Road in Pittsburg, California* dated September 20, 2002 (File 16656/GE1). Our 2002 report evaluated three optional alignments (Northern, Central, and Southern) developed by the City and RBF. Our evaluation was based on field mapping by our Certified Engineering Geologist (CEG) and review of aerial photographs and published and unpublished geologic and seismic reports and maps covering the site area. Our 2002 report concluded that the Central Alignment would require the least amount of mitigation needed to address the geologic, seismic, and geotechnical constraints and considerations identified during our evaluation. No subsurface exploration was performed as part of our noted 2002 assessment.

The City and RBF subsequently selected the Central Alignment and prepared a preliminary grading scheme, which they provided to us. Kleinfelder conducted a subsurface exploration program as part of a design-level geotechnical investigation that was designed to identify and characterize the subsurface conditions along the selected Central Alignment extension and to evaluate the feasibility of the noted grading scheme and the stability of cut and fill slopes proposed along the alignment. The results, conclusions, and recommendations of our study were presented in a report titled *Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass in Pittsburg, California* dated January 9, 2008 (File No. 75856/PWGEO).

Since the issuance of our referenced 2008 report, the above-noted and previously selected Central Alignment has been re-labeled as the "Original Alignment (C1)" and is referred to in this report as such. In addition to the Original Alignment (C1), three additional alignment alternatives to the Original Alignment (C1) have been developed by RBF and the City. They are identified as "Middle Alignment (C2)", "Middle Alignment (C2-Low)", and "Northern Alignment (C3)". These three additional alternative alignments extend east/west in a parallel fashion to the selected Original Alignment (C1) and are situated immediately to the north of it. Note that current version of the Original Alignment includes minor modifications to the original grading plan included in our 2008 report.

It is important to note that the Middle Alignment (C2) and the Northern Alignment (C3) do not extend along the entire length of the proposed Original Alignment (C1) and are only shown on topographic base maps generated by RBF and transmitted to us on December 12, 2011 to extend between approximate Stations 30+00 and 74+00 of the Original Alignment (C1). However, topographic base maps transmitted to us by RBF on February 9, 2012 for the Middle Alignment (C2-Low) show the noted alignment to extend along the entire length of the proposed Original Alignment (C1). Beyond Stations 30+00 and 74+00 of the Original Alignment (C1), the alignments for alternatives C1 and C2-Low are generally a match. Please note that Reference Station 10+00 marks the western terminal end of the roadway extension and is located at the intersection of the

Original Alignment (C1) with Kirker Pass Road based on project plans prepared by RBF.

1.3 PURPOSE AND SCOPE OF SERVICES

Our current scope included the following items:

- Background and document review including published geologic and seismic literature, aerial photographs, and our referenced reports prepared in 2002 and 2008;
- Review of the four alignment layouts and their proposed grading magnitude;
- Field reconnaissance of the alignment by our CEG and one of our Registered Geotechnical Engineers; and
- Preparation of this report presenting the results of our evaluation of the feasibility of the four alternatives relating to faulting and seismicity, fault-related ground surface rupture, landslide deposits, expansive soils, erosion and drainage, liquefaction and lateral spreading, dynamic compaction, flooding, naturally-occurring asbestos, and adverse bedrock bedding. In addition, this report recommends the selection of one of the four alignment alternatives, C1, C2, C2-Low, and C3 based on our interpretation of the least engineering geologic and geotechnical constraints encountered and the desire to achieve balanced cut/fill volume quantities during grading. These geologic and geotechnical constraints include the engineering geologic characteristics of the underlying geologic formations, bedrock rippability, landslide deposits, adverse bedrock bedding, magnitude of associated grading, and anticipated stability of cut slopes in the various bedrock formations.

1.4 AUTHORIZATION

This feasibility assessment was performed in accordance with our contract with RBF dated November 18, 2011.

1.5 SITE DESCRIPTION AND CONDITIONS

The site area is situated along the northern foothills of Mount Diablo, generally along the northern end of elevated ridgelines bordering the southern boundaries of the City. Beyond the ridgelines to the north, alluvial plains extend northward under the City, to the southern shores of the San Joaquin River channel near its junction with the Sacramento River. The north/south trending ridgelines are separated by several prominent drainage courses that drain northward towards the relatively flat alluvial plains.

The alignment has a topographic relief measuring approximately 390 feet with ground surface elevations varying from approximately 180 feet near the northeastern portion of the alignment to approximately 565 feet along the southernmost section of the alignment area. The site topography as it relates to the location of the three roadway extension alignments is shown on topographic base maps provided by RBF on Plates 3 through 7A and 7B.

The site surface area is generally covered by wild grasses and scattered oak trees along the ridgelines. Numerous dirt roads cross the site. High voltage overhead transmission lines that are supported by several steel towers extend across the site area in a generally east/west direction. The site area is generally free of buildings except near the mouth of the prominent drainage course situated to the south of the termination point of Suzanne Drive where the Thomas residence and other ranch related structures are located to the north of the three alignments. Several fence lines and associated gates are present throughout the site area. The site area is currently being used for animal ranching and grazing activities. An underground gas easement housing high pressure gas lines parallels the entire length of the overhead high voltage transmission and towers along their southern side.

Kirker Creek crosses the western edge of the alignment immediately east of Kirker Pass Road. The creek is located within a canyon that measures up to about 60 feet in depth locally. The creek flows northward along the east side of Kirker Pass Road. Several corrugated metal culverts were noted across the site where drainage swales or courses are crossed by dirt roads.

2 GEOLOGY

2.1 REGIONAL GEOLOGY

The site is located in Contra Costa County, the majority of which lies within the Coast Range geomorphic province in Central California. This geomorphic province contains a more or less discontinuous series of northwest-trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in regional studies by Schlocker (1970), Wagner and others (1990), Chin and others (1993), and Ellen and Wentworth (1995).

The dominant structural feature in the Coast Range Geomorphic Province is the San Andreas Fault which is a strike-slip fault, with a right-lateral sense of motion. Numerous fault traces such as the Hayward, Calaveras and San Gregorio, among others, comprise the San Andreas Fault system in the tectonic context of the seismically active San Francisco Bay Area. The San Andreas Fault trace is the boundary between two tectonic plates, the Pacific Plate to the west of the fault and the North American Plate to the east of the fault. These two crustal plates are moving past each other in a generally northwest/southeast direction at approximately 5 cm/year (2 inch/year) at the mouth of the Gulf of California and 1 to 3 cm/year (0.4 to 1.2 inch/year) in the central and northern parts of California (Brown, 1990).

In the San Francisco Bay Area, movement along this plate boundary is concentrated on the San Andreas Fault; however, it is also distributed, to a lesser extent across a number of the other near-by faults. The northwest trend of the faults within the San Andreas Fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area.

The basement rocks east of the San Andreas fault are Jurassic to Cretaceous age (195-65 million years before present) rocks of the Franciscan Complex. This Complex is generally comprised of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks. West of the San Andreas Fault, the basement rocks are composed of the Cretaceous age (140 to 65 million years old) granitic Salinian block. The basement rocks on both sides of the San Andreas fault are overlain by Cretaceous, Tertiary (66 to 1.8 million years old) and Quaternary age (1.8 million years or younger [USGS, 2006]) marine and continental sedimentary and local volcanic rocks. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely as a result of movement along the San Andreas fault system which has been ongoing for about the last 25 million years. The inland valleys, as well as the structural depression containing the San Francisco Bay, are filled with unconsolidated to semi-consolidated surficial deposits of Quaternary age. Surficial continental deposits (alluvium, colluvium, and landslide deposits) consist of varying mixtures of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the Bay deposits typically consist of very soft organic rich silt and clay (Bay mud) or sand.

2.2 SITE GEOLOGY AND RECONNAISSANCE

The site lies along the northern foothills of Mount Diablo in central Contra Costa County. Over the last approximately 40 years, the geology of Contra Costa County has been extensively mapped by the U.S. Geological Survey (USGS) and the California Geological Survey (CGS). Published geologic and seismic literature and maps reviewed for this study are listed in the "References" Section of this report. A description of the site's geology is presented below.

The geology of the site and adjacent areas has been mapped by the Division of Mines (1954), Brabb (1976), Dibblee (1980), Wagner et al. (1990), Chin et al. (1993), Graymer et al. (1994), Ellen and Wentworth (1995), and Helley and Graymer (1997). The maps generally agree on the distribution of bedrock formations in the immediate vicinity of the site. The portion of Graymer et al. (1994) geologic map that covers the site area is included herein as Plate 8.

The mapped bedrock formations are listed and discussed below starting with the northernmost (youngest) units:

- The Pliocene age (about 2 to 4 million years old) Tulare formation (map symbol **Ttu**) which is largely comprised of continental sandy claystone, sandstone, and conglomerate;

- The Pliocene age (about 4 to 5 million years old) Lawlor Tuff (map symbol **Tlt**) which is chiefly comprised of pyroclastic (volcanic ejecta) pumice lapilli tuff and water-reworked tuff deposited in a sedimentary setting;
- The Miocene age (about 20 million years old) Neroly formation (map symbol **Tn**) which is mostly comprised of continental blue andesitic sandstone and siltstone;
- The Miocene age (about 20 million years old) Cierbo formation (map symbol **Tc**) which is generally comprised of marine pebbly and fossiliferous sandstone and conglomerate; and
- The Oligocene age (about 30 million years old) Kirker formation (map symbols **Tkt** and **Tks**) which is mainly comprised of volcanic tuff and tuffaceous siltstone and sandstone.

2.3 BEDROCK UNITS

During our site reconnaissance visits, the bedrock units and surficial geologic deposits observed at the site were mapped by our CEG and their approximate locations are shown superimposed on the attached Plates 3 through 7A and 7B. Based on our site reconnaissance and mapping, a review of stereoscopic aerial photographs and published geologic maps and reports, as well as other information contained in site-specific studies conducted by other consultants, we have prepared descriptions of the bedrock formations encountered along the planned alignment. These descriptions are discussed in the sections below.

2.3.1 Tulare Formation

The Tulare formation is Pliocene age (about 2 to 4 million years old) sedimentary formation that is largely comprised of continental (non-marine) light sandy yellowish brown claystone, sandstone, and conglomerate. These units are derived from the older sedimentary and volcanic units to the south. According to Ellen and Wentworth (1995), the sandy claystone constitutes more than 75 percent of the formation but in hard topography areas the clayey sandstone and conglomerate represent more than 50 percent of the unit's composition. The claystone and the sandstone units contain pebbly stringers and clasts derived from the older units such as lapilli tuff from the Lawlor, blue sandstone and petrified wood from the Neroly, and fossiliferous sandstone fragments from the Cierbo formations. The sandy claystone is thickly and crudely bedded, lenticular, irregular, and has a low intergranular permeability. Bedrock and mantle materials are highly to severely expansive.

The Tulare formation units were encountered and their exposures were observed by our CEG during subsurface investigations and grading activities (Kleinfelder, 1999, 2001, and 2005 through 2007) in Antioch and Pittsburg. The sandy claystone generally constitutes the majority of the unit but localized clean sandy zones were encountered. The sandy claystone, described by ENGEO, Inc. (1991) as "gummy", is generally weak to plastic, highly weathered, of low hardness (firm), and thickly bedded. These characteristics resulted in the observed soft topography of low rounded hills. The older sections of this formation (farthest south) appear to form intermediate to hard topography implying the presence of clean sandstone and conglomerate along those zones. Colluvial expansive soils blanket the bedrock materials.

This formation is susceptible to soil creep and landslide activities. North-facing slopes (whether they are natural or cut) along with areas adjacent to the geologic contact with the underlying Lawlor Tuff units are especially susceptible to landslide activities because of the weak nature of the material, the adverse northward dip of the beds, and the subsurface water directed to the base of the Tulare formation along the underlying upper surface of the less permeable Lawlor Tuff. Relatively large landslides occur within this formation.

2.3.2 Lawlor Tuff

The Pliocene age Lawlor Tuff (dated by Sarna-Wojcicki in 1976 to be approximately 4.5 million years old) is chiefly comprised of pyroclastic (volcanic ejecta) pumice lapilli tuff and water-reworked tuff that was deposited in a sedimentary setting. This unit forms distinctive resistant outcrops that can be traced almost continuously along their northwestern strike for nearly 8 miles in the general vicinity of Antioch and Pittsburg (Chesterman and Schmidt, 1956).

Ellen and Wentworth (1995) noted that the pyroclastic pumiceous lapilli tuff constitutes approximately 25 to 35 percent of the unit and that the remainder of the unit is comprised of firm tuffaceous sedimentary rock and minor basalt beds. The relatively light density pumiceous lapilli tuff is andesitic in composition (Vitt, 1936) and contains angular broken fragments of white and grayish white pumice supported by a white to pink matrix of pumicite. The tuff contains broken crystals of feldspars and olivine-basalt gravels locally. It is well compacted but somewhat porous and forms prominent exposures and cavernous outcrops. The materials forming the Lawlor Tuff are widely fractured and have a low to moderate intergranular permeability where weathered and high where fresh. These units weather to an approximate depth of 20 feet and they are usually expansive where weathered with the surficial mantle severely expansive.

Based on observations recorded by our CEG during the subsurface exploration and grading activities at the Mira Vista Hills project in Antioch (Kleinfelder, 1999 and 2000) and Vista Del Mar project in Pittsburg (Kleinfelder, 2007), the pumiceous lapilli tuff appeared firm to hard, cohesive, brittle, and appeared clayey where weathered. At depth, zones of light density and high permeability vitric tuff were encountered in some of our borings at the Mira Vista Hills project. Such zones could react when they come into contact with concrete, collapse when wetted or loaded and are moderately to highly expansive.

The Lawlor Tuff units generally have a low to moderate landslide susceptibility. It is underlain by the Neroly formation siltstone and sandstone and overlain by the clayey Tulare formation units. The upper surface of the Lawlor Tuff is usually less permeable (when welded or solidified) and it acts as a medium to transmit water northward beneath the base of the Tulare formation forming relatively large landslide deposits within the Tulare formation near its contact with the lower underlying Lawlor Tuff units.

2.3.3 Neroly Formation

The Neroly formation is mostly comprised of continental blue andesitic sandstone and siltstone with interbeds of gravel and is commonly cross-bedded. Their blue appearance results from a translucent coating on the sand/silt grains. The coating was previously thought to be opal, but Lerbemko (1956) showed it to be a fibrous mineral instead.

Snow (1957) concluded that the interstitial coating is made up of bound montmorillonitic clay.

This formation forms prominent "hogbacks" and ledges, providing good outcrop exposures. The materials comprising the formation are generally thickly bedded, moderately hard, friable, porous, and moderately cemented. The sandstone forms cavernous weathering patterns and wind sculpturing is common. Some of the upper beds are rusty and resistant due to cementation by iron oxide. Clark (1912) described large concretions (zones of concentrated cementation) within the sandstone that could reach many feet in diameter. The bedrock is fractured in a perpendicular fashion to the bedding. Locally, the uppermost portion of the Neroly formation is comprised of laminated siltstone beds that are highly and closely fractured and are friable and expansive when broken down mechanically.

According to Ellen and Wentworth (1995) the sandstone has a moderate intergranular permeability while that of the siltstone is low. The rocky units may not appear clayey but weathering of the sandstone and siltstone frees clay particles bound up around the grains to where the soils derived from the bedrock are expansive. The undisturbed bedrock is considered to have low expansion but the soil mantle is highly expansive.

2.3.4 Cierbo Formation

The Miocene Cierbo formation is generally comprised of marine pebbly and fossiliferous (clam, gastropod, and oyster shells) sandstone and conglomerate. The arkosic (predominantly composed of feldspar minerals) sandstone ranges from fine to coarse-grained with stringers of pebbles. Tuffaceous and diatomaceous shale and lignite seams are common. Sandstone concretions as large as 10 feet in diameter are also common at depth. The sandstone varies from strongly cemented (with calcium carbonate) and resistant forming hard topography to firm and non-resistant forming swales and bold rounded hills.

Ellen and Wentworth (1995) indicate that more than 60 percent of the formation is composed of the non-resistant sandstone. During our field reconnaissance visits, we observed the upper (younger) portion of the formation to be cemented by calcite which is crystalline, thus forming hard topography. The lower portions, however, tended to be less resistant. Conglomerate beds with hard clasts and shale partings are common within the sandstone. Spheroidal weathering patterns were noted within the strongly cemented portions. The non-resistant portion of the sandstone has a moderate intergranular permeability while the hard sandstone portion displays a very low intergranular permeability. The bedrock generally has a low expansion potential but the weathered portions and surficial mantle are considered expansive.

2.3.5 Kirker Formation

The Kirker formation is mainly comprised of marine volcanic tuff and tuffaceous sandstone and mudstone. The tuff is vitric (composed of crystals) and lithic (composed of minute rock fragments) and is fissile.

According to Ellen and Wentworth (1995) the tuff constitutes approximately 20 percent of the formation, while the tuffaceous sandstone and mudstone make up the remaining portion. The units are firm, brittle, friable, fractured, laminated, and scaly, and have a relatively light density. They also display a low to moderate intergranular permeability. Additionally, the units are considered expansive where weathered or mechanically broken and their soil mantles are highly expansive. Limited field exposures suggest that the tuff is non-resistant and the sandstone to be intermediate. The tuff was observed along the base of the drainage swales and along the base of the hillock (small hill) situated near the extreme western portion of the site between the Kirker Creek channel and Kirker Pass Road. The sandstone and tuff generally form swales, rounded knobs, and subdued topography and erode easily.

ENGEO, Inc. (1990) noted that these tuffaceous units react adversely with concrete, expansive when broken down, and have low strengths.

2.4 QUATERNARY SURFICIAL DEPOSITS

Quaternary surficial deposits such as alluvium, colluvium/slope wash, and landslide deposits were also mapped along some portions of the site. The approximate locations of the surficial mapped Quaternary deposits are shown on our Plates 3 through 7A and 7B. These surficial deposits are discussed further below.

2.4.1 Quaternary Alluvium

Helley and Graymer (1997) prepared a geologic map differentiating Quaternary alluvial deposits. They differentiated the Pleistocene (11,000 to 1.8 million years old) and Holocene (Recent to 11,000 years old) alluvial deposits and identified their depositional types, and modes. These alluvial deposits are generally comprised of varying amounts of clay, silt, sand, and gravel and are transported and laid in place by running water. Helley and Graymer (1997) mapped the relatively flat topographic areas in the vicinity of Kirker Canyon Creek and along the northern base of the foothills as Pleistocene alluvial fan deposits (map symbol **Qpaf**). During our reconnaissance of the site, we noted that the creek and prominent drainage course channels contained relatively thin, localized younger Holocene deposits within their channels while the Pleistocene alluvium formed slightly elevated terraces bordering the creek channels. We designated the younger Holocene deposits on the Plates 3 through 7A and 7B as **Qal.**

The Pleistocene (Qpaf) alluvial deposits could measure in excess of 50 feet along the banks of the Kirker Creek channel and up to about 20 feet along the northern portions of the prominent drainage courses crossing the site. However, the Holocene (Qal) alluvial deposits are relatively thin within the drainage channels traversing the site area. Where relatively thick, these alluvial deposits could consist of soft and locally wet sediment at depth. Soft alluvial sediments are usually removed from areas to receive engineered fills to lessen subsequent settlement of the fills.

2.4.2 Colluvium and Slope Wash

Colluvium is comprised of loose, heterogeneous soil and rock material that is deposited by natural mass-wasting processes. Slope wash is made up of soil and rock materials that are or have been transported down a slope by mass-wasting processes assisted by running water. Both of these surficial deposits tend to creep down steep slope faces and are usually present along the axis of drainage swales and topographic hollows. The slope wash deposits are delineated on Plates 3 through 7A and 7B as **SR**. Colluvial deposits are generally present on nearly all of the moderately steep to steep slope faces that are covered with residual soil, and at the flatter areas at the toes of hills. Because the lateral distribution of the colluvial deposits is wide across the hilly site area, we chose to delineate their concentrated presence with squiggly arrows indicating approximate direction of creep. We did not show them as a distinctive geologic unit on our Plates 3 through 7A and 7B where they are relatively thin so that the underlying bedrock geology is not obscured. Typically the upper portions of the colluvial material have undergone shrinking and swelling cycles, and are usually susceptible to consolidation from loads (such as from new fills).

These colluvial deposits are more abundant within the clayey Tulare formation occupying the northern portion of the site. There are, however, more slope wash deposits along the more granular bedrock units such as the Neroly and Cierbo formations. The colluvial and slope wash deposits tend to be more clayey towards the north and more granular southward. These deposits are relatively shallow, measuring approximately between 5 and 10 feet, but could reach 15 to 20 feet depending on the topographic shape of the swales housing them and their source areas. These deposits are usually subexcavated and incorporated into engineered fills in proposed fill areas, and are rebuilt if exposed on cut slope faces to help stabilize them.

Two moderate-size slope wash deposits underlie portions of the alignment along the southern portion of the prominent drainage swale adjacent to the Thomas residence. The slope wash deposits generally occur in the Neroly and Cierbo formation materials.

2.4.3 Landslides

Numerous landslide deposits were identified within the site area during our aerial photograph interpretation, reconnaissance, and literature review performed as part of this study. Active and dormant landslide deposits were mapped and their approximate lateral boundaries and degree of activity are indicated on Plates 3 through 7A and 7B. We marked the active landslides with a circled "**A**" to differentiate them from the mapped dormant ones we marked with a circled "**D**".

The active landslides are those that display recent topographic signs of activation such as head scarps, soil cracking outlining portions or all of the landslide, hummocky terrain, dirt road distortions, striated head scarps, and raised toe areas. Dormant landslides are those that lack indications of recent movement. Their outlines are generally subdued, they lack head scarps and the slide mass itself seems to be less angular. The dormant landslides tend to be stable in their current configurations but may reactivate if existing conditions are altered. Dormant landslide reactivation could occur due to seismic activities, rise in groundwater levels, grading operations undermining the toe area or loading the higher portions of the landslide with fill.

The majority of the landslide deposits mapped (active and dormant) are situated within the Tulare formation. The four largest landslide deposits do not underlie the alignment. However several significant landslides are situated in close proximity to the planned alignment and they could impact its construction. The locations of the landslide deposits that could impact the alignment are shown on Plates 3 through 7A and 7B. The potential impact of the landslide deposits on the selected alignment will be discussed in our updated geotechnical evaluation to be prepared once a particular alignment is chosen.

2.5 SITE SOILS AND SOIL SURVEY MAPS

The soils within the site area have been classified and described by the U.S. Soil Conservation Service (1977). They utilized aerial photographs as base maps. Their maps show the majority of the northern portion (underlain by the Tulare formation) to belong to the Altamont Group. These materials are described as having a high shrink and swell potential and are corrosive. They indicate that care must be taken when using these materials for roadway construction due to their high expansion potential and low strengths.

According to the Soil Survey Maps, the more resistant outcrop and peak areas are underlain by soils of the Lodo Series. These materials are described as having a very thin veneer of soil covering rocky terrain. Their expansion and corrosion potentials are more moderate than the Altamont Group soils. The disadvantages associated with this type of soils are their limited supply and possibly low strengths.

Along the relatively flat areas surrounding the channel of Kirker Creek, the soil survey maps show soils belonging to the Rincon Soil Series. These soils tend to be similar in their engineering characteristics to the Altamont Group soils. Table (2.5-1) below presents more detailed characteristics of the soils discussed above.

TABLE 2.5-1
Engineering Characteristics of the Soils within the Project Area

Soil Series or Group	Depth to Bedrock (meters) (feet)	Plasticity Index	Shrink/Swell Potential	Corrosivity	Permeability
Altamont	3 ½ - 5 1.1 – 1.5	25-30	High	High	Slow
Lodo	1 – 11/2 0.3 – 0.5	15-20	Moderate	Moderate	Slow
Rincon	>5 .1.5	15-25	High	High	Slow

Note: Derived from the Soil Survey Publication for Contra Costa County (USDA, 1977)

2.6 GEOLOGIC STRUCTURE

The older bedrock formations such as the Kirker, and Cierbo were initially deposited in a horizontal fashion atop each other by sedimentary processes in a marine environment. A period of erosion followed after each formation was deposited but before the following unit was laid. Mountain building and uplift episodes caused the deposited layers to form a part of the continent and become exposed. A period of erosion along the exposed surface of the deposited and uplifted layers followed, after which a depositional period of continental sedimentary and volcanic formations such as the Neroly, Lawlor Tuff, and Tulare occurred. The units were subsequently tilted northward rendering the units stacked in their current position becoming younger northward up the geologic section.

The thickness and width of the exposed portions of the various formations varied laterally because the units weathered differentially along their exposed surfaces before the next formations were deposited.

The units generally strike northwestward at an angle ranging between 55 and 75 degrees and have associated dips varying between 15 to 40 degrees to the east of north. The bedding of the Tulare formation is generally subdued and obscured, but bedding readings recorded by our geologists at the Mira Vista Hills (in Antioch) and Vista Del Mar (in Pittsburg) subdivisions indicate that the bedding angles may be as gentle as 12 degrees to the northeast.

The geologic units and formations mapped across the site dip northeastward. Accordingly, cut slopes steeper than about 15 degrees could potentially present adverse bedding (dip slope) conditions that could contribute to or promote slope instability and failure. Planned cut slopes steeper than 15 to 20 degrees may need to be rebuilt as engineered fill depending on the underlying formation, degree of weathering, height of the slope, and the encountered localized conditions.

2.7 GROUNDWATER

Groundwater was not encountered in our previous exploratory soil or rock core borings. Four borings were recently drilled at the west side of the Original Alignment as part of our current studies. Three of these borings were drilled near the banks of Kirker Creek to depths of about 50 feet and only one boring encountered groundwater at a depth of about 50 feet. A forth boring was drilled near Kirker Pass Road to a depth of 26 feet and encountered water at a depth of about 19 feet, but we suspect that this water was perched and was not representative of the groundwater level in the area. No springs or seeps indicative of shallow groundwater were observed during our reconnaissance. Groundwater may be encountered along the drainage courses during the grading operations, especially if cuts are made along these drainage courses. Groundwater levels may be higher during the rainy season.

3 FAULTING AND SEISMICITY

The project site is located in a region which is traditionally characterized by moderate to high seismic activity. Based on the information provided in Hart and Bryant (1997), the site is not located within an Alquist-Priolo Earthquake Fault Zone where special studies addressing the potential of surface fault rupture are required. The closest fault is the Greenville-Marsh Creek fault located at a distance of about 5.6 km towards the southwest. A major earthquake on this fault could cause significant ground shaking at the site. A list of the significant regional faults and their seismic parameters are presented in Table 3-1 below. Some of the faults located in the region (not considered independent seismogenic sources) and not listed in the table are the Kirker Pass fault, Clayton fault, he Antioch fault and the Livermore fault. Plate 9 presents the fault map showing the location and activity of some of the regional faults in relation to the site.

The locations of the faults and associated parameters presented on Table 3-1 below are based on data presented by Real and others (1978), Toppozada and others (1978), Hart and others (1984), Wesnousky (1986), Working Group on California Earthquake Probabilities (1999), Schwartz (1994), Jennings (1994), Frankel and others (1996), and Petersen and others (1996). The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Kanamori (1977).

TABLE 3-1: SIGNIFICANT FAULTS

Fault Name and Geometry (1) and Location (2)	Fault Length (km)	Closest to Site (km)	Magnitude of Maximum Earthquake * (3)	Slip Rate (mm/yr)
Greenville-Marsh Creek (rl-ss) (SW)	73 ± 7	5.6	6.9	2 ± 1
Great Valley 6 (r,15,W) (NE)	45 ± 5	10	6.7	1.5 ± 1
Concord-Green Valley (rl-ss) (W)	66 ± 7	13	6.9	6 ± 3
Great Valley 5 (r,15,W) (NE)	28 ± 3	16	6.5	1.5 ± 1
Mt. Diablo Thrust (r,W) (SW)	25 ± 5	18	6.7	3 ± 2
Calaveras (northern) (rl-ss) (SW)	52 ± 5	20	6.8	6 ± 2
Hayward (rl-ss) (SW)	86 ± 9	35	7.1	9 ± 1
Great Valley 4 (r,15,W) (N)	42 ± 4	36	6.6	1.5 ± 1
West Napa (rl-ss)	30 ± 3	38	6.5	1 ± 1

Fault Name and Geometry (1) and Location (2)	Fault Length (km)	Closest to Site (km)	Magnitude of Maximum Earthquake * (3)	Slip Rate (mm/yr)
Rodgers Creek (rl-ss) (NW)	63 ± 6	42	7.0	9 ± 2
Great Valley 7 (r,15,W) (SE)	45 ± 5	43	6.7	1.5 ± 1
Hayward (SE Extension) (rl-r-o) (S)	26 ± 3	58	6.4	3 ± 2
Hunting Creek-Berryessa (rl-ss) (NW)	60 ± 6	60	6.9	6 ± 3
Calaveras (southern) (rl-ss) (S)	106 ± 11	61	6.2	15 ± 2
San Andreas (1906 Event) (rl-ss) (SW)	470 ± 47	64	7.9	24 ± 3
San Gregorio	129 ± 13	70	7.3	5 ± 2
Monte Vista-Shannon (r,45,E) (SW)	41 ± 4	72	6.8	0.4 ± 0.3
Great Valley 3 (r,15,W) (N)	55 ± 6	77	6.8	1.5 ± 1
Point Reyes (rl-ss) (NW)	47 ± 5	85	6.8	0.3 ± 0.2
Great Valley 8 (r,15,W) (SE)	41 ± 4	88	6.6	1.5 ± 1
Sargent (rl-r-o) (S)	53 ± 5	95	6.8	3 ± 1.5
Ortigalita	66 ± 7	97	6.9	1 ± 0.5
Zayante-Vergeles	56 ± 6	100	6.8	0.1 ± 0.1

(1) ss = strike slip; r = reverse; n = normal; rl = right lateral; o = oblique; 15 W = Dip angle and direction
(2) W = West; E = East; SW = Southwest; NE = Northeast; S = South
(3) Moment Magnitude based on rupture area regressions from Wells and Coppersmith (1994) need reference

The project site and its vicinity are located in an area traditionally characterized by high seismic activity. A number of large earthquakes have occurred within this area in the past years. Some significant regional earthquakes include the 1889 (M6.3) Antioch earthquake, the 1868 (M7) Hayward earthquake, the 1906 (M7.9) San Francisco earthquake, the 1838 (M7) San Francisco/San Mateo earthquake, the 1858 (M6.1) Mission Peak area earthquake, the 1861 (M5.7) San Ramon Valley earthquake, the two 1903 (M5.5) San Jose earthquakes, the July 1911 (M6.6) Calaveras earthquake, the 1957 (M5.3) Daly City earthquake, the 1980 (M5.8) Livermore earthquake, and the 1989 (M6.9) Loma Prieta earthquake.

The earthquake database used in our search contains in excess of 5,500 seismic events and covers the period from 1800 through December 2011. The earthquake database is principally comprised of an earthquake catalog for the State of California prepared by the California Geological Survey, CGS (formerly known as the California Division of Mines and Geology, CDMG). The original CGS (CDMG) catalog (Real and others 1978) is a merger of the University of California at Berkeley and the California

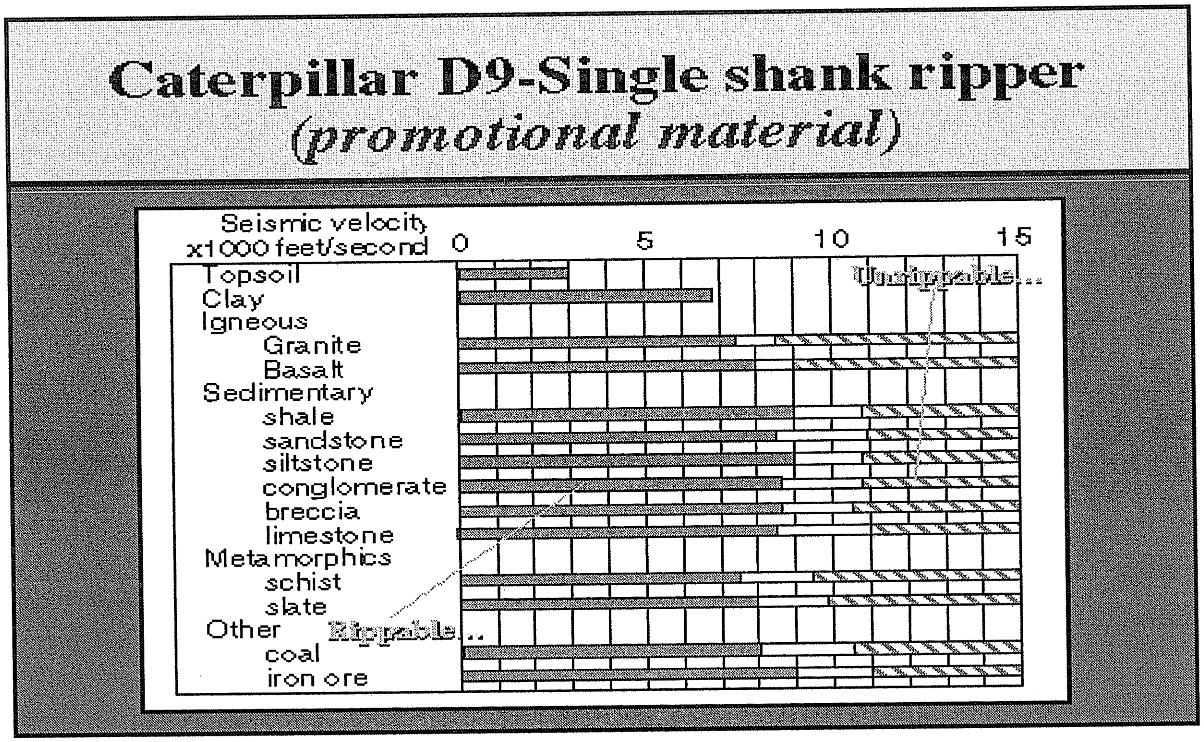
Institute of Technology instrumental catalogs (Hileman and others 1973). The combined catalog contains earthquake records from January 1, 1900 through December 31, 1974. Updates prepared by CGS (CDMG) in 1979 and 1982 extend the coverage through 1982. In addition to the CGS (CDMG) updates, the data for earthquakes for the period between 1910 and July 2002 have been obtained from a composite catalog by Council of the National Seismic System (CNSS). The CNSS catalog is a worldwide earthquake catalog that is created by merging the master earthquake catalogs from contributing CNSS member networks and then removing duplicate events, or non-unique solutions from the same event. The CNSS network includes Northern and Southern California Seismic Networks, Pacific Northwest Seismic Network, University of Nevada, Reno Seismic Network, University of Utah Seismographic Stations and US National Earthquake Information Service. The earthquake database also consists of earthquake records between 1800 and 1900. This subset of the earthquake database was derived from Seeburger and Bolt (1976) and Toppozada and others (1978, 1981). In addition, we have also utilized the data from CGS (CDMG) Map Sheet 49 (Toppozada and others 2000).

The parameters used to define the limits of the historical earthquake search include geographical limits (within 100 km [62 miles] of the site), dates (1800 through December 2011), and magnitudes (M>4). A summary of the results of the historical search is presented below.

Time Period (1800 to December 2011)	211+ years
Maximum Magnitude	M8+
Approximate distance to nearest historical M> 4 earthquake	2 km (1.2 miles)
Number of events exceeding magnitude 4 within search area	over 142

4 RIPPABILITY EVALUATION

In evaluating the seismic-refraction velocities with respect to rippability, we used Caterpillar Tractor Company, Handbook of Rippability for heavy duty ripper performance. This table is as follows:



The table provided by Caterpillar Tractor Company is for a large track bulldozer with a single ripper hook attached. This rippability rating was used as an indicator of the relative difficulty anticipated in excavating rock at the selected sites along the alignments and should be adjusted based on the equipment selected by the contractor for this project. For conditions where seismic-refraction data is within the rippable range, it should be expected that hard areas may be encountered.

It is important to note that the operator's experience, working condition of excavation equipment and the selection of excavation tools used will be critical factors in the excavatability of rock. During construction, modifications to tool selection or replacement of equipment being used may be necessary to improve performance and production rates. It is recommended that the contractors who use the rippability data in this report visit the proposed location of the roadway to observe bedrock conditions. It is recommended that the contractor have options available in order to deal with differing bedrock conditions.

The seismic velocities obtained during our seismic refraction survey performed as part of our previous 2008 study are necessarily averages across differing soil and rock conditions. Bedrock units can have coherent rock masses separated by discontinuities (fractures, bedding planes, joints, highly weathered zones, etc). This fact should be considered and allowed for when using seismic-refraction to estimate rippability. Seismic waves travel through coherent rock relatively fast and travel through intervening discontinuities relatively slow. The resulting seismic velocity through rock units will be the sum of velocities across coherent rock masses and discontinuities and will not be a true velocity through either rock type. As a result, variable rippability or excavation conditions both harder and softer can be encountered along the survey line.

Our limited seismic refraction survey results obtained in 2008 indicate that the explored areas within the Neroly and Cierbo formations show the highest velocities but are generally considered to lie in the rippable range. Our previous experience gained while observing grading operations within these two formations also indicates that they are generally rippable. However, localized strongly cemented zones within the two formations resulting in large diameter concretions have proven nearly non-rippable. Hydraulic hammers or blasting may be required if such strongly cemented zones are encountered.

5 ALTERNATIVE ALIGNMENT COMPARISON

The four optional roadway extension alignments being currently considered are identified as the Original Alignment (C1), Middle Alignment (C2), Middle Alignment (C2-Low), and Northern Alignment (C3) are shown collectively on Plate 3 and separately on Plates 4A and 4B through 7A and 7B, respectively.

Our geologic mapping showing the various underlying geologic formations and the surficial geologic deposits consisting of alluvium, colluvium/slope wash, and landslides has been delineated on the topographic maps used as a base for Plates 3 through 7A and 7B.

The easternmost (between Stations 76+00 through 99+75 of the Original Alignment) and westernmost portions (between Stations 10+00 through 30+00 of the Original Alignment) are shared by all four alternative alignments and these two sections have been assessed previously as part of our 2002 and 2008 studies. Our current assessment generally compares the portions of the roadway extension where there are three distinctively separate options as is delineated on Plates 3 through 7A and 7B.

5.1 ORIGINAL ALIGNMENT (C1)

The pertinent portion of this alignment being compared to the two other alternatives extends between approximate Stations 30+00 and 76+00 of the Original Alignment. It is mostly underlain by the Neroly and Cierbo formations although its eastern end encroaches onto the Tulare and Lawlor Tuff units and its southernmost part extends onto the Kirker formation near Stations 45+00 and 55+00 of the Original Alignment.

Cuts up to about 110 feet deep are proposed as part of the grading scheme for this alignment option and potentially adverse north-facing and relatively high cut slopes are proposed within the Cierbo and Kirker formations. As with the other alignment alternatives C2, C2-Low, and C3, a deep fill is proposed across the creek channel that extends northward to near the Thomas residence.

5.2 MIDDLE ALIGNMENT (C2)

This alignment alternative extends between approximate Stations 32+40 and 73+00 of the Middle Alignment extends mostly across terrain that is underlain by the Neroly formation although its eastern end encroaches on the Tulare formation (as do the other two alternatives) and Lawlor Tuff, and its western portion between approximate Stations 32+40 and 51+00 encroaches onto the Cierbo formation. A south-facing cut slope is planned into the Neroly formation between Stations 47+00 and 59+00 of the Middle Alignment. A relatively high north-facing cut slope measuring about 120 feet in height is proposed along this alignment and it would be made into the Neroly formation between Stations 49+00 and 58+00 of the Middle Alignment. A second relatively high north-facing slope within the Tulare formation is planned between approximate Stations 70+50 and 73+00. While this cut is considered adverse, it is also planned as part of the other two alternatives C1 and C3. We understand that the cut quantities that would be generated during the grading of this alternative alignment would be lower than the required fill quantities and would therefore require fill importation.

5.3 MIDDLE ALIGNMENT (C2-LOW)

The location of this alternative alignment generally matches that of the Middle Alignment C-2, but its overall elevations are lower. Accordingly, cuts associated with this alignment alternative are generally lower and fills are less thick than those of the Middle Alignment C2. Relatively higher cut slopes proposed as part of the Middle Alignment (C2-Low) will generally be made into the Neroly formation where the potential for adverse bedding impacts and erosion are considered lower than across other formations. The advantage of this alignment alternative over the Middle Alignment C2 is that its proposed magnitude of cut and fill should balance out and the need for fill material importation would either be avoided or significantly reduced.

5.4 NORTHERN ALIGNMENT (C3)

This alignment option is mostly underlain by the Neroly formation although it encroaches onto the Tulare formation and Lawlor Tuff at its eastern end. Its eastern portion between approximate Stations 39+00 and 45+00 (of the Northern Alignment) extends onto the Cierbo formation where fill is planned. A south-facing cut slope is planned into the Tulare formation and Lawlor Tuff between approximate Stations 46+00 and 56+00 and a north-facing cut slope is planned between approximate Stations 46+00 and 57+00 of the Northern Alignment. A less significant south-facing cut slope will be made into the Neroly formation between approximate Stations 35+00 through 39+00.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 FAULT-RELATED GROUND SURFACE RUPTURE

No known active faults have been mapped crossing the site area and the site is not situated within an Alquist-Priolo Earthquake Fault Zone established by the CGS around active fault traces where the site-specific fault evaluations are required. The closest active fault traced zoned by the CGS is the Greenville fault and it is situated more than 3.5 miles from the site. The absence of known active fault traces crossing through the project site results in very low potential for ground surface rupture to occur as a result of fault movements. It appears unlikely that fault rupture will occur directly across the proposed road alignments. Based on this information, it is our opinion that the potential for fault-related ground surface rupture to occur in the vicinity of the alignments is low.

6.2 LANDSLIDE DEPOSITS

The landslide deposits that underlie the three alignment alternatives are shown on Plates 3 through 7A and 7B. An active landslide is mapped underlying portions of the Original and Middle alignments at near Station 40+00 (of the Original Alignemnt). Another mapped landslide at near Station 85+00 is a dormant deposit which will not require mitigation since it does not show signs of movement and fill will be placed along its toe portion, which will help increase its stability.

Several smaller landslides were mapped and they would either be blanketed by planned fills or be cut out and removed during the grading.

6.3 EXPANSIVE SOILS

The surficial soils predominantly consist of clay and exhibit moderate to severe plasticity and expansion potential. That is, they tend to shrink and swell with fluctuations in moisture content. The potentially expansive clay subgrade soils could cause differential movements of pavement, flatwork, and shallow foundations if not mitigated in advance. To mitigate this expansion potential, we judge that limited overexcavation and/or use of careful moisture conditioning and subgrade preparation within areas where these project features are anticipated should be adequate.

This soil type is also sensitive to changes in moisture content that could result in "workability" problems if the soil is too wet (common during the rainy winter months). If the clay soil is too wet, then earthwork activities to grade the site could become increasingly difficult with regard to excavation, placement, and compaction of general fill in accordance with project requirements. If site grading will be conducted during periods of (or following) wet weather, the grading contractor should anticipate these conditions. The preferred approach to grading would be to conduct site earthwork during drier weather periods when the surficial clay soils are sufficiently dry and firm.

Some vertical movement of exterior flatwork should be anticipated and will occur as a result of moisture content variations of the supporting soil below. Discussions of measures to reduce the impact of these expansive soils will be presented in our upcoming updated geotechnical report.

6.4 SOIL CORROSION

Corrosion potential analysis performed by an independent contractor as part of our 2008 geotechnical evaluation included measurement of pH, soluble sulfate and chloride content. Based upon the resistivity measurements, three samples were classified as "corrosive" and one sample is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron will need to be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion. Since we are not corrosion specialists, a corrosion testing firm should be contacted for specific design details, if necessary.

6.5 DRAINAGE AND EROSION CONTROL

The site area drains towards several north-flowing drainage courses including Kirker Creek. These drainage courses will be crossed by the selected alignment. Two bridges will be constructed across the Kirker Creek channel. Drainage culverts will be installed under the planned fills before they are placed across the remaining prominent drainage courses. Accordingly, the natural drainage pattern or damming is not anticipated.

Site soils are potentially subject to moderate to high rates of erosion, hence soil erosion on new cut or fill slopes could be significant if not mitigated in advance. This could be especially significant where grading activities take place and the finished slopes contain no vegetation. Such slopes may deposit sediment on the lower portions of the slope or on the Bypass roadway surface during heavy rainfall periods. This may result in clogging of drainage facilities and ponding of water or sediment on to the roadway surface.

Slopes that are anticipated to be susceptible to erosion by wind and rainfall should be protected. Protection is also necessary for slopes subjected to water flow action (undercutting) as in the creek banks or along drainage swales where fills are planned. In some cases, provision must also be implemented against burrowing animals. Terracing and landscaping measures are commonly used in order to control erosion. Significant erosion still can occur on slopes that have inclinations steeper than 3H:1V (horizontal to vertical). Our experience indicates that slope gradient steeper than 2H:V1 do experience greater erosion than those that are 3H:1V or flatter.

Six-foot wide drainage terraces should be constructed on slopes steeper than 3H:1V at every 30 vertical foot interval except where the slope heights exceed 120 feet in vertical height in which case a 12-foot wide mid-slope terrace should be constructed along with 6-foot wide benches at every 30-foot vertical intervals.

All graded soil slopes should be planted with fast-growing, deep-rooted, low water tolerant vegetation to retard erosion. Other possible slope protection measures include a layer of rock or cobbles or other commercially available erosion control products. Protection from flowing water action in the creeks may be provided by hand placed rock riprap, concrete pavement, pre-cast concrete blocks, soil-cement, or other commercially available erosion control products as approved by the appropriate government agency.

6.6 LIQUEFACTION AND LATERAL SPREADING

Liquefaction is a phenomenon in which saturated, granular, cohesionless soils lose strength because of build-up of excess pore water pressure under cyclic loading such as induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movement if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements, resulting in reduction of foundation stiffness and capacities. Liquefaction can also cause embankment displacement and/or building structural damage as a result of shallow foundation failures and/or large vertical and/or lateral displacements.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. This phenomenon typically occurs adjacent to free faces such as slopes and creek channels. Vertical and lateral ground movements due to this phenomenon have been known to result in damage to near surface improvements such as pavements, infrastructure, and flatwork.

According to Youd and Hoose (1978), there were no reported signs of liquefaction and lateral spreading within about 10 miles of the project limits as a result of past earthquakes in the region. According to Holzer (1998), the site is over 23 miles away from the nearest recorded signs of liquefaction and lateral spreading associated with the 1989 M6.9 Loma Prieta earthquake. It should be noted that the mapping by Holzer(1998) focused primarily on the inner coastline of San Francisco Bay and the Monterrey Bay coastline.

Bedrock units underlie the majority of the site area, but alluvial soils are present along the margins of five drainage courses that cross the proposed road alignments. Helley and Graymer (1997) mapped old alluvial fan deposits along the relatively flat area present along the northeastern portion of the site. Such deposits are more cemented and dense than the younger alluvial (Holocene) deposits. The Holocene alluvial deposits mapped around or within channels of prominent drainage swales are generally anticipated to be thin.

The groundwater was not encountered in the 12 borings drilled by Kleinfelder at the site in 2007[1] as part of our 2008 investigation, which ranged from about 15 to 76 feet in depth. Five of these borings were drilled along the margins of drainage courses. Four borings were recently drilled at the west side of the alignment as part of our current studies. Three of these borings were drilled near the banks of Kirker Creek to depths of about 50 feet and only one boring encountered groundwater at a depth of about 50 feet. A forth boring was drilled near Kirker Pass Road to a depth of 26 feet and encountered water at a depth of about 19 feet, but we suspect that this water was perched and was not representative of the groundwater level in the area. According to a groundwater monitoring report by Shaw Environmental, Inc. (2010)[2] for a site located at the northeast corner of the intersection of Buchanan Road with Loveridge Road, groundwater was encountered at depths of 50 feet or greater. That site is located about ¾ to 1mile from the proposed road alignments.

Based on the above information, we conclude that the potential for liquefaction and lateral spreading to occur at this site is low.

6.7 DYNAMIC COMPACTION

Another type of seismically induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. Based on the subsurface conditions encountered at the site, we conclude that potential for this type of seismic distress to affect the site is low.

[1] Presented in the report titled Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass, Pittsburg, California, dated January 9, 2008 (File No. 75856/PWGEO).

[2] Available at the GeoTracker website (http://geotracker.waterboards.ca.gov), which is maintained by the State of California's Water Resources Control Board.

6.8 FLOODING

The site area is situated at approximate elevations ranging between 180 and 565 feet above Mean Sea Level. It is located at a moderate distance from the San Joaquin River and the San Francisco Bay. Accordingly, the potential for flooding resulting from storm-related events, tsunami, or dam failure inundation is considered low.

6.9 NATURALLY-OCCURRING ASBESTOS

None of the formations underlying the three alternative alignments is comprised of ultrabasic rock and none of the mapped formations contains serpentine or asbestos-bearing minerals. In addition, no bedrock formations that contain asbestos-bearing minerals are mapped to the south where some of the drainage courses originate. Based on this information, it is our opinion that the potential for asbestos-bearing mineral to be encountered during the construction of this project is low.

6.10 ADVERSE BEDROCK BEDDING

The mapped bedrock formations mapped within this site dip northeastward. North/northeast facing cut slopes could undermine exposed bedding, creating a dip slope condition. Such a dip slope condition could render all proposed north/northeast-facing cut slopes as adverse since slope failures could occur. Cut slopes proposed either lower or higher than the alignment elevations can potentially fail and adversely affect the alignment. The dip angle of the bedrock beds should be evaluated for all proposed cut slopes and, where possible, the angle of the slope face should not be steeper than that of the bedrock dip to prevent the beds from daylighting on the slope face. In some instances, where the bedding angle is considered adverse or the soil exposed is either weak or too sandy, it may be necessary to repair some of the cut slope faces by overexcavation and placement of subdrained, buttressed fill slopes.

6.11 ALIGNMENT ALTERNATIVE SELECTION

Based on our feasibility assessment, it is our opinion that the four optional roadway extension alignment alternatives Original (C1), Middle (C2), Middle C2-Low, and Northern (C3) are geologically and geotechnically feasible. However, the Middle Alignment (C2-Low) has the following advantages:

- It is mostly underlain by the Neroly formation, which is considered most stable and less susceptible to landslide activities than the other formations found at the site;
- Its only prominent north-facing cut planned between Stations 47+00 and 59+00 (of the Middle Alignment [C2-Low]) will encounter the Neroly formation;
- Cut materials generated from the Neroly formation are considered suitable fill materials and will most likely require less compaction effort than material generated from other formations underlying the site;
- Cut slopes into the Neroly formation would be less susceptible to slope instability and erosion than cuts into other formations found at the site; and
- Its proposed magnitude of cut and fill should balance out and the need for fill material importation would either be avoided or significantly reduced.
- In our opinion, this alignment alternative would be better suited to receive and support deep fills than the other two alternatives.

Based on the above, we recommend that the Middle Alignment (C2-Low) be selected.

6.12 ADDITIONAL SUBSURFACE EXPLORATION

The grading scheme for all four alignment alternatives indicates a significant cut in the Tulare formation near approximate Stations 73+00 through 80+00 of the Original Alignment (C1). The planned cut will most likely expose a relatively high cut and adverse bedrock dip. This planned cut slope will exceed 160 feet in vertical height and will require reconstruction and buttressing.

In addition, the cut will be made along the base of the Tulare formation, which is underlain by the more indurated Lawlor Tuff. Infiltrating subsurface moisture is anticipated to migrate northward atop the Lawlor Tuff and along the base of the Tulare, which could significantly contribute to landslide causation. To better characterize the subsurface conditions and evaluate the geologic structure at this planned cut area, we recommend that two continuously-sampled rock core borings be advanced to extend to the base of the planned cut. This could require performing additional laboratory testing to evaluate strength properties and soil/rock characteristics of the samples retrieved from these two borings.

7 ADDITIONAL SERVICES

Kleinfelder's current scope of services includes updating the geotechnical report prepared in 2008 by Kleinfelder and the development of remedial grading plans.

The review of plans and specifications and field observation and testing by Kleinfelder of earthwork related construction activities are an integral part of the conclusions and recommendations made in this report. It is recommended that Kleinfelder be present at the pre-bid meeting with the prospective grading contractors to clarify any issues and to address any questions regarding the recommendations presented in this report. If Kleinfelder is not retained for these services, the client will be assuming Kleinfelder's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein and/or our upcoming update report. The recommended tests, observations, and consultation by Kleinfelder prior to and during construction include, but are not limited to:

- Review of plans and specifications;
- Observations of earthwork operations spanning site clearing and stripping through final grading and utility trench backfill;
- Observation of foundation excavations and foundation construction; and
- Construction observation and in-place density testing of fills, backfills; and finished subgrades.

8 ADDITIONAL SERVICES

Our limited study specifically excluded any environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater or atmosphere, or the presence of wetlands. The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been performed according to generally accepted geotechnical engineering practices that existed in the San Francisco Bay Area at the time this report was written. No warranty is expressed or implied. This report is issued with the understanding that the owner chooses the risk he wishes to bear by the expenditures involved with the construction alternatives and scheduling that is chosen.

The conclusions and recommendations provided herein are for the subject roadway extension project described in this report. These conclusions and recommendations should be used for the selection of the alignment to be constructed and the design of final grading plans. Further investigations for remedial grading plans will be required and additional input for design of the project will be provided in our upcoming updated report.

The conclusions recommendations presented in this report are based on information obtained from the following:

- Review of our previous reports for the site,
- Review of published and unpublished maps and data,
- Aerial photograph review,
- The observations of our engineering geologist and geotechnical engineer at the site during our reconnaissance,
- Our experience in the area.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the site have changed. Land use, site conditions (both on-site and off-site) or other factors may change over time, and additional work may be required.

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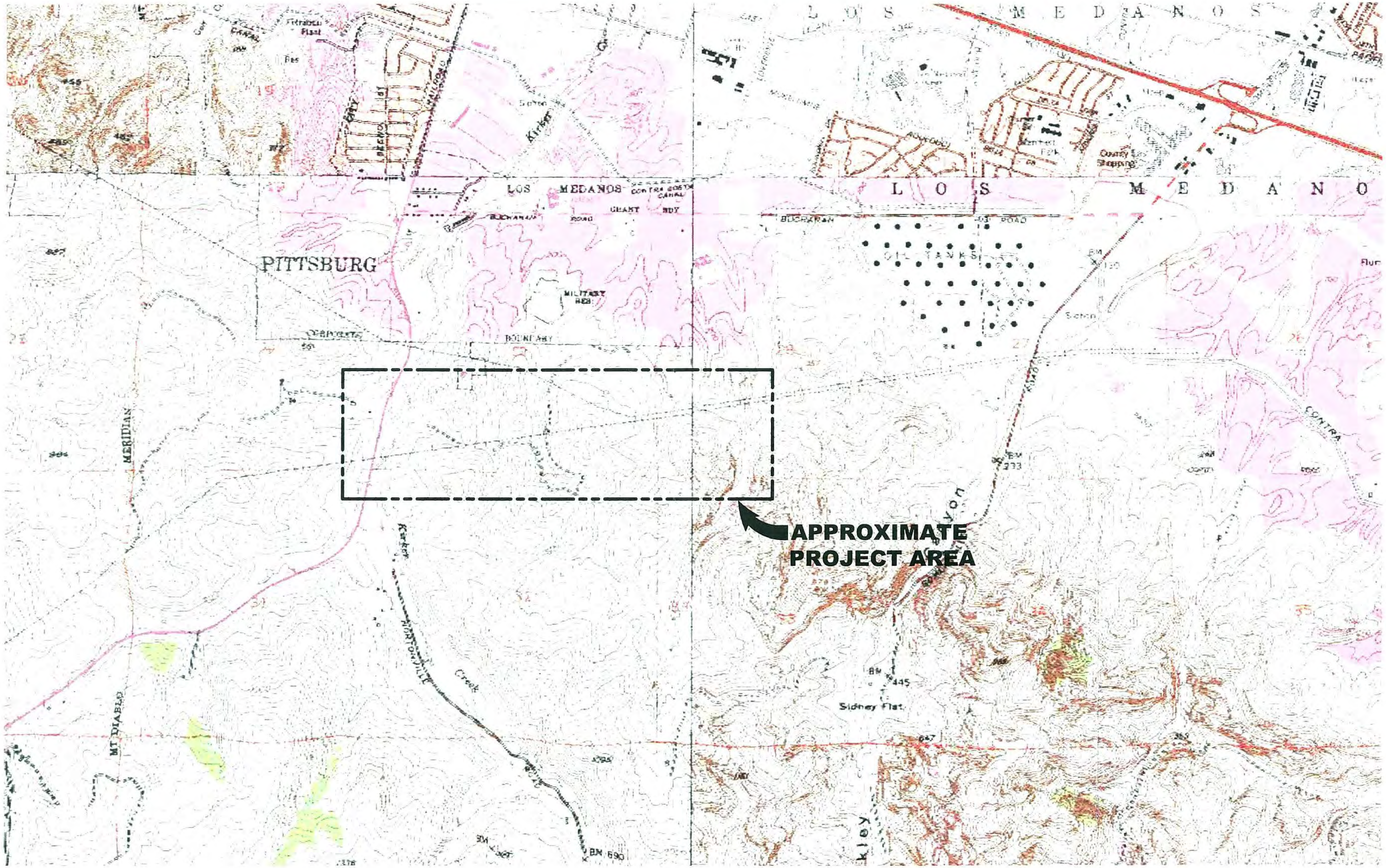
PLATES



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1800 900 0 1800

APPROXIMATE SCALE (feet)

REFERENCE:
USGS 7.5 Minute Series (Topographic)
Antioch South Quadrangle, dated 1953, photorevised 1980;
Clayton Quadrangle, dated 1953, photorevised 1980, revised 1994

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FILE NAME: VIC-PLAN.dwg	

SITE VICINITY MAP

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
1

ATTACHED IMAGES: Images: 20120120172112_00001.jpg Images: AREA GEOLOGY LEGEND.jpg Images: SITE-VIC.jpg Images: SITEPLAN.jpg
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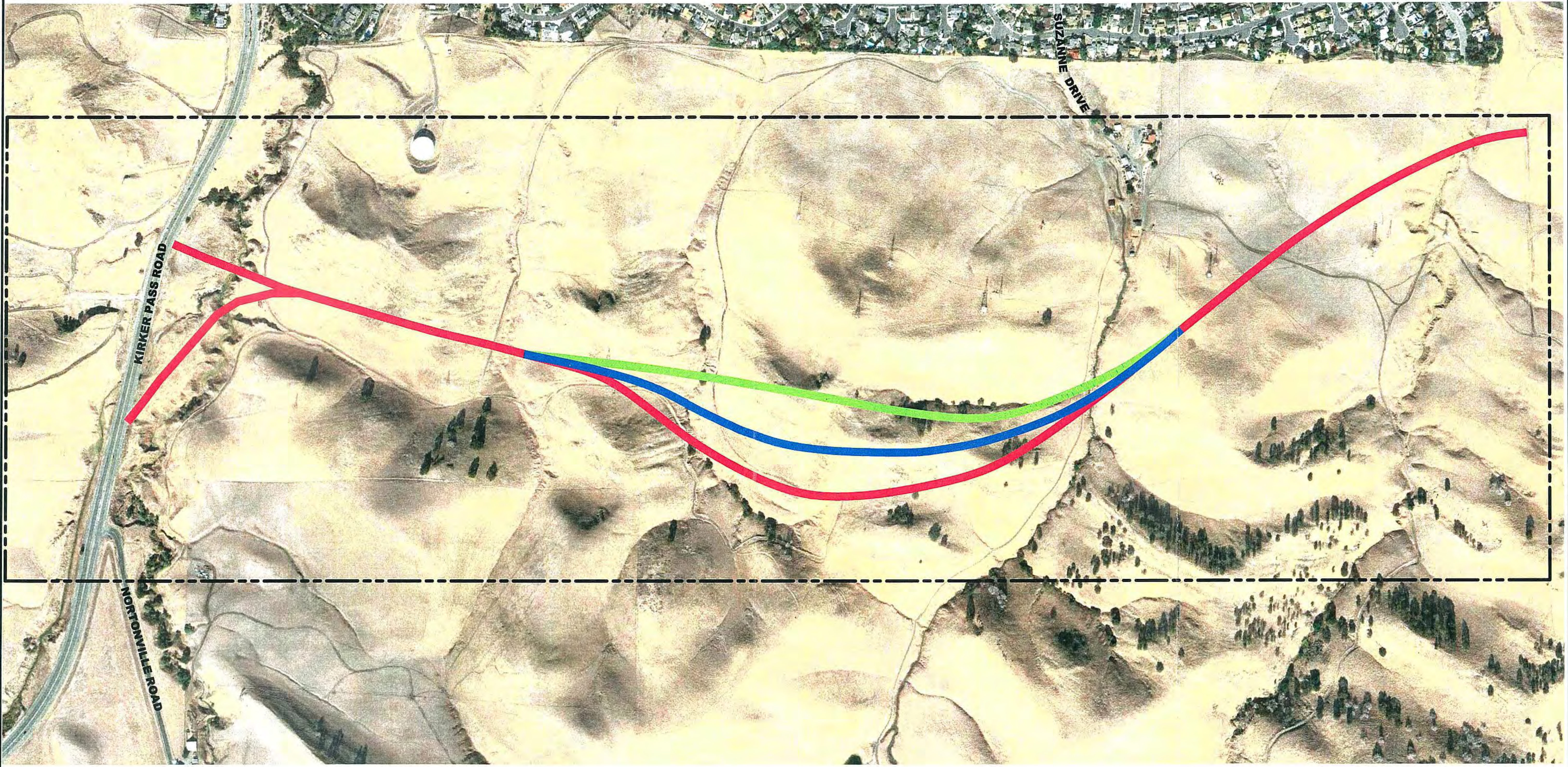

LEGEND

APPROXIMATE PROJECT AREA

ORIGINAL ALIGNMENT (C1)

MIDDLE ALIGNMENTS (C2) AND (C2-LOW)

NORTHERN ALIGNMENT (C3)

NOTE: Locations are approximate.

500 250 0 500

APPROXIMATE SCALE (feet)

REFERENCE:
Google Earth Pro., Imagery date 10-29-2011

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FILE NAME: VIC-PLAN.dwg	

AERIAL SITE PLAN

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
2

ATTACHED IMAGES: Images: 20120120172112_00001.jpg Images: AREA GEOLOGY LEGEND.jpg Images: SITE-VIC.jpg Images: SITEPLAN.jpg
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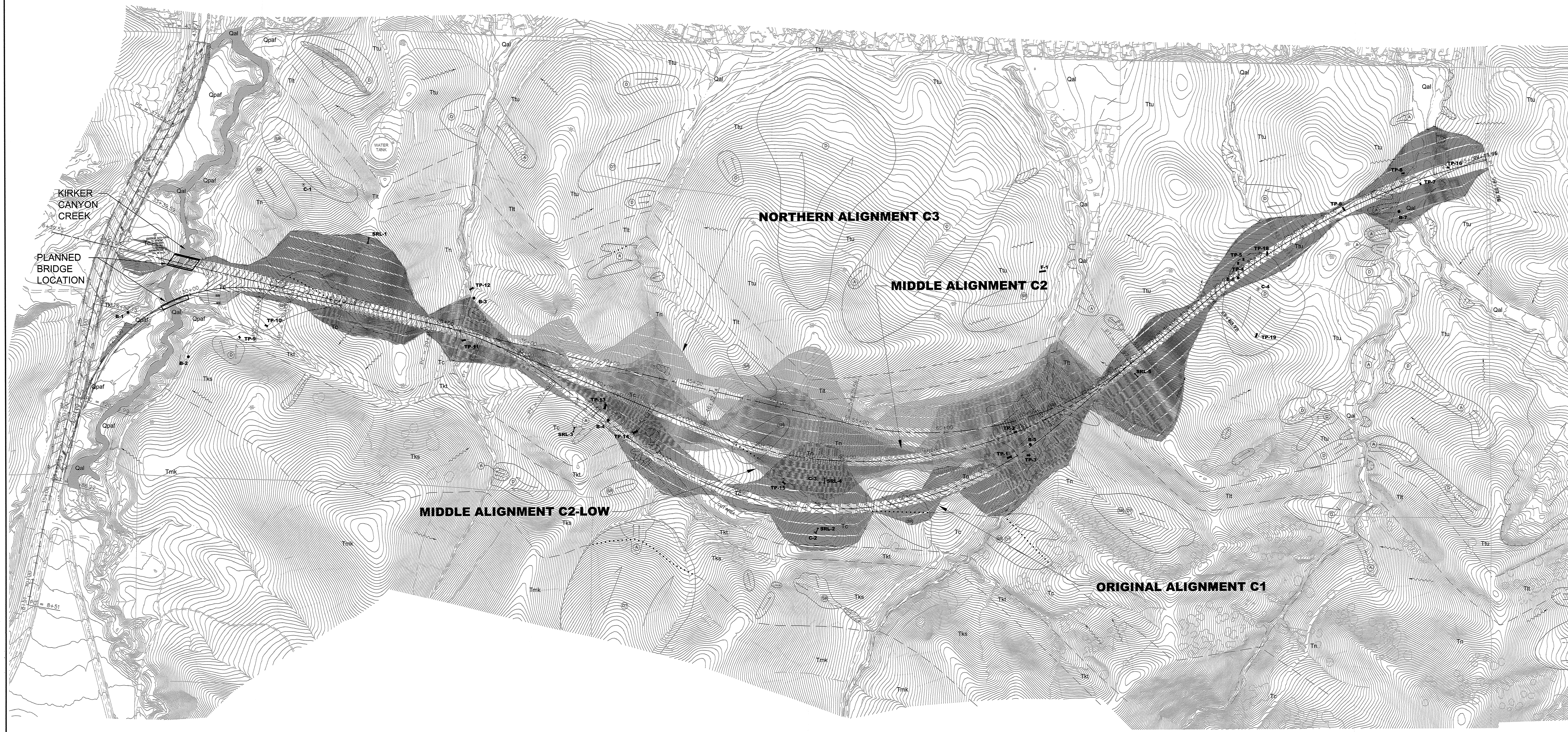

EXPLANATION

APPROXIMATE GEOLOGIC CONTACT – dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SR SLOPE WASH (arrows indicate direction of movement)

A ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)

D DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)

B-7	SOIL BORING (By Kleinfelder, 2008)
C-4	ROCK CORE (By Kleinfelder, 2008)
TP-19	TEST PIT OR EXPLORATORY FAULT TRENCH (By Kleinfelder, 2008)
SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)

N

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APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map" dated 12/10/11 and 12/13/11:
File Emailed By RBF Consulting, "0129-GRBS-C2-Low Profile-Full drawing" dated 2/9/12.

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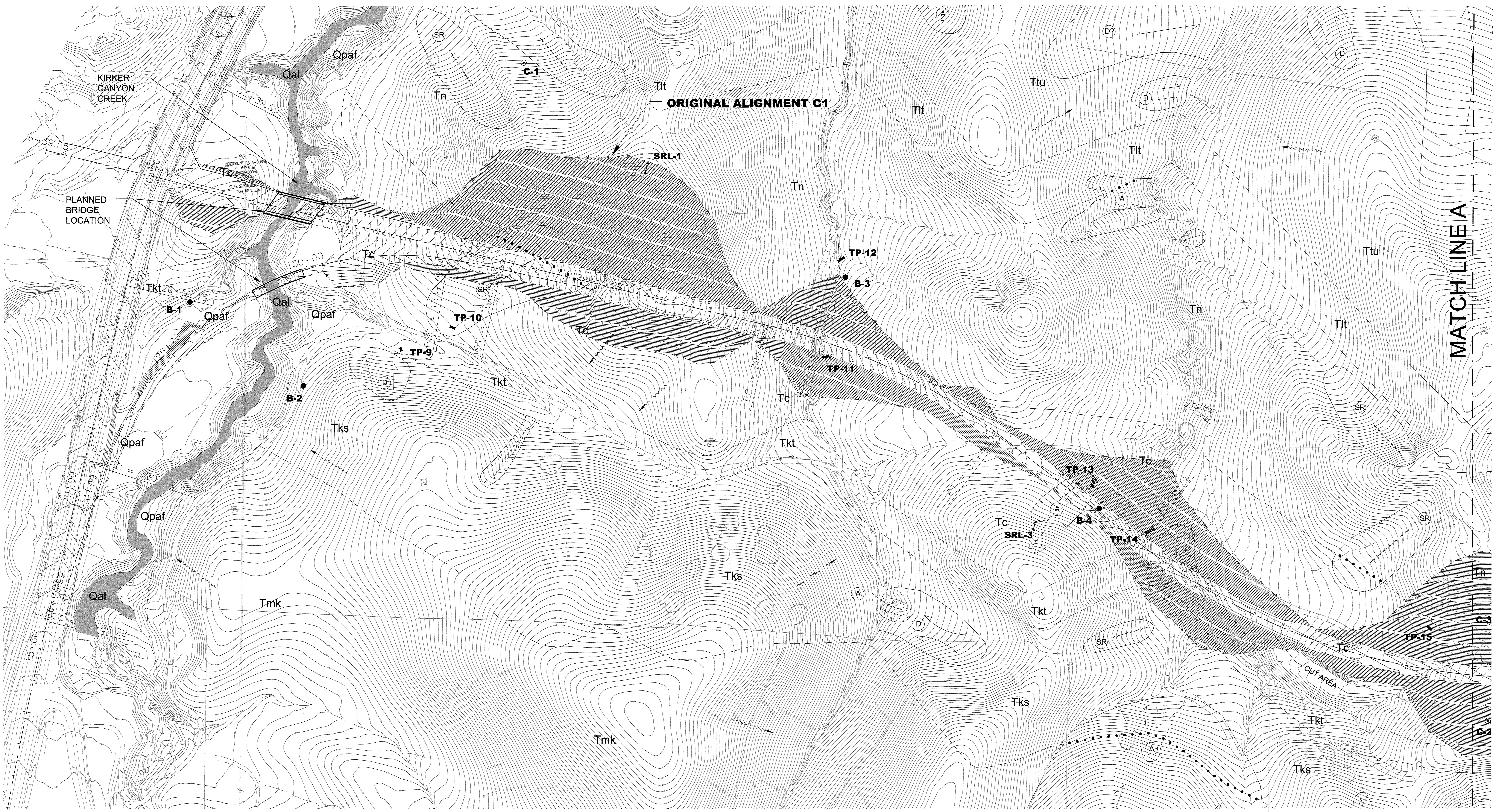
FILE NAME:	BASE MAP w LOW profile.dwg

ALTERNATIVES ALIGNMENTS ORIGINAL (C1), MIDDLE (C2), MIDDLE (C2-LOW) & NORTHERN (C3) LAYOUT PLAN

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
3



EXPLANATION

	APPROXIMATE GEOLOGIC CONTACT – dotted where buried
	CREEPING COLLUVIUM
	INCISED EROSION RILLING OR SHALLOW SLUMP
	SLOPE WASH (arrows indicate direction of movement)
	ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)
	DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)

B-7	SOIL BORING (By Kleinfelder, 2008)
C-4	ROCK CORE (By Kleinfelder, 2008)
TP-19	TEST PIT OR EXPLORATORY FAULT TRENCH (By Kleinfelder, 2008)
SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)

N

100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11.

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123561	ORIGINAL ALIGNMENT (C1)	PLATE 4A
	DRAWN: MAR 2012		
	DRAWN BY: JDS		
	CHECKED BY: SD	JAMES DONLON BOULEVARD EXTENSION PITTSBURG, CALIFORNIA	
	FILE NAME: BASE MAP w LOW profile.dwg		

EXPLANATION

APPROXIMATE GEOLOGIC CONTACT — dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SLOPE WASH (arrows indicate direction of movement)

ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)

DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)

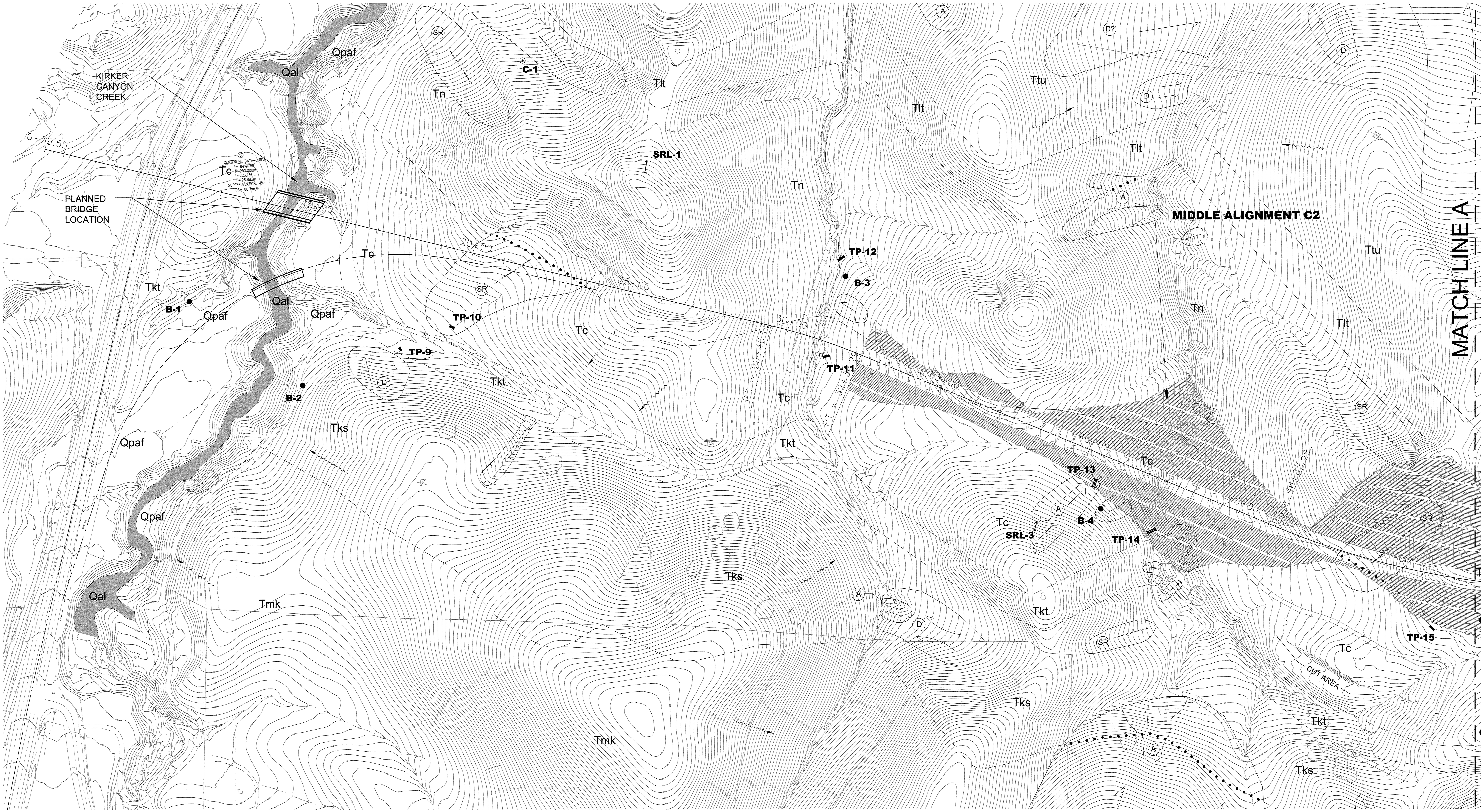
B-7 ●	SOIL BORING (By Kleinfelder, 2008)
C-4 ⊙	ROCK CORE (By Kleinfelder, 2008)
TP-19	TEST PIT OR EXPLORATORY FAULT TRENCH (By Kleinfelder, 2008)
SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)

N

100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11.

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123561 DRAWN: MAR 2012 DRAWN BY: JDS CHECKED BY: SD FILE NAME: BASE MAP w LOW profile.dwg	ORIGINAL ALIGNMENT (C1) JAMES DONLON BOULEVARD EXTENSION PITTSBURG, CALIFORNIA	PLATE 4B



EXPLANATION

	APPROXIMATE GEOLOGIC CONTACT – dotted where buried
	CREEPING COLLUVIUM
	INCISED EROSION RILLING OR SHALLOW SLUMP
SR	SLOPE WASH (arrows indicate direction of movement)
A	ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)
D	DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

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SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)

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100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11.

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123561	MIDDLE ALIGNMENT (C2)	PLATE 5A
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	DRAWN BY: JDS		
	CHECKED BY: SD	JAMES DONLON BOULEVARD EXTENSION PITTSBURG, CALIFORNIA	
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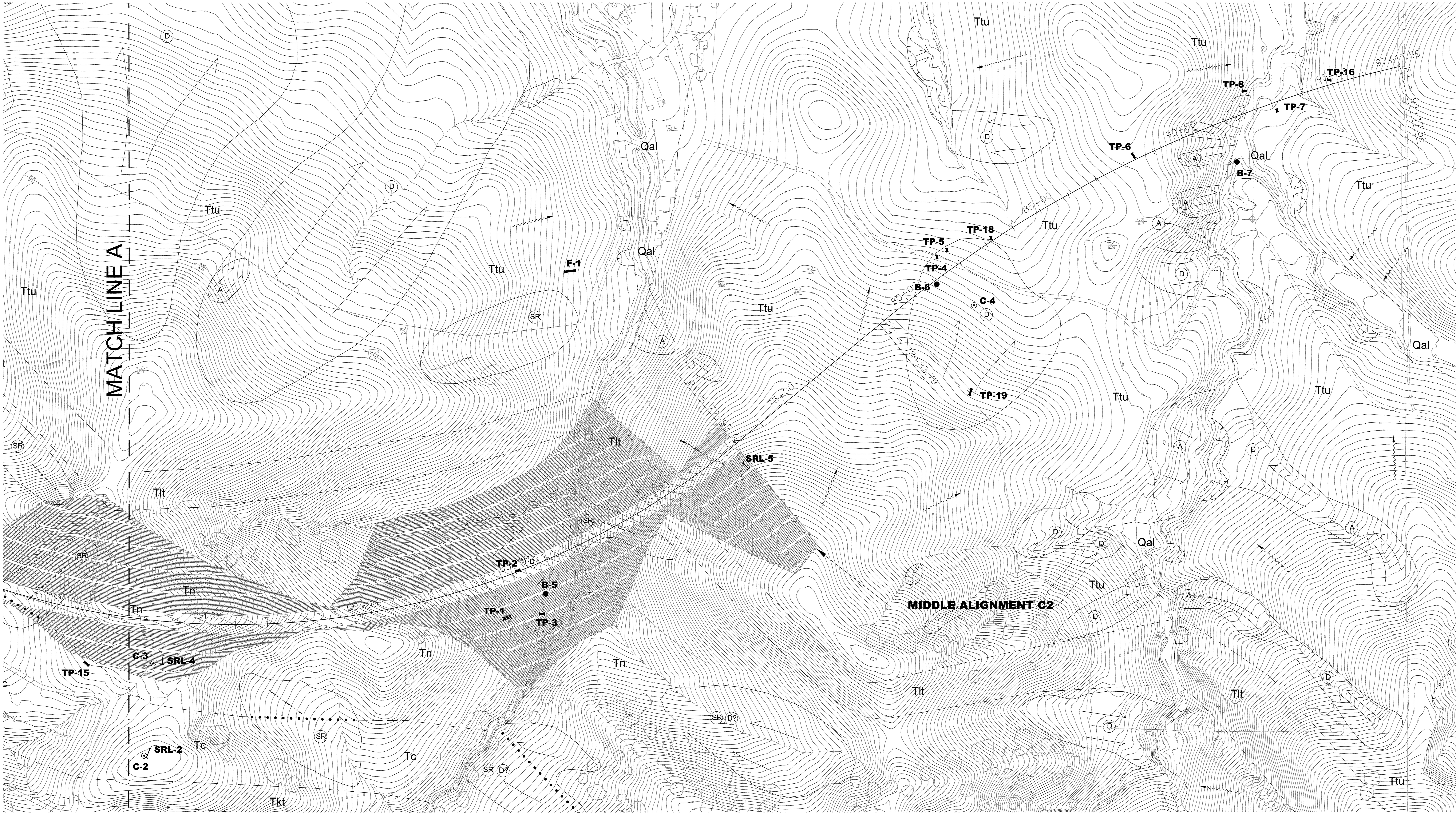

EXPLANATION

— — —······	APPROXIMATE GEOLOGIC CONTACT — dotted where buried
	CREEPING COLLUVIUM
	INCISED EROSION RILLING OR SHALLOW SLUMP
SR	SLOPE WASH (arrows indicate direction of movement)
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N

100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
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PROJECT NO.	123561
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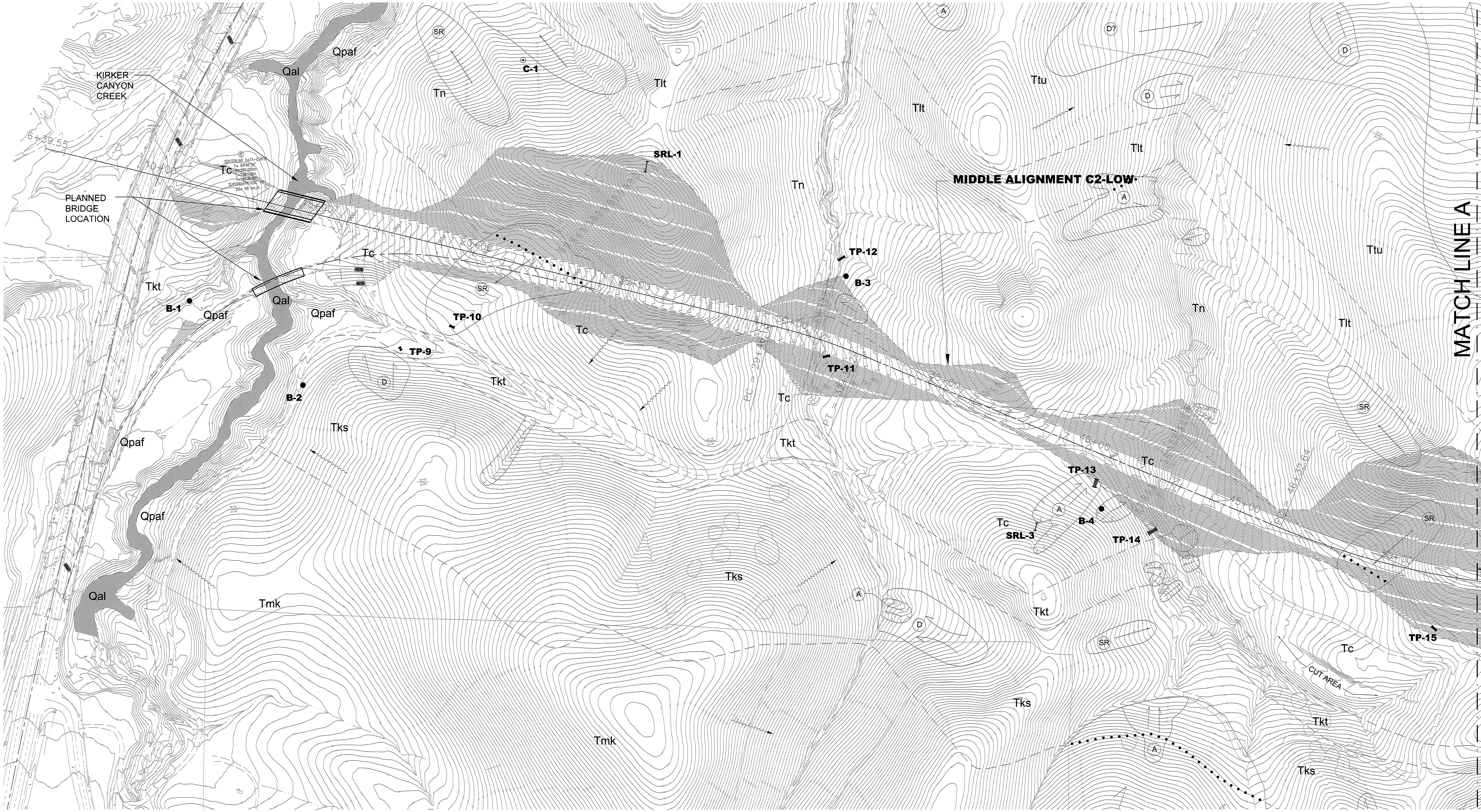
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	BASE MAP w LOW profile.dwg

MIDDLE ALIGNMENT (C2)

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
5B



EXPLANATION

APPROXIMATE GEOLOGIC CONTACT – dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SLOPE WASH (arrows indicate direction of movement)

ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)

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N

100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11;
File Emailed By RBF Consulting, "0129-GRBS-C2-Low Profile-Full drawing" dated 2/9/12.

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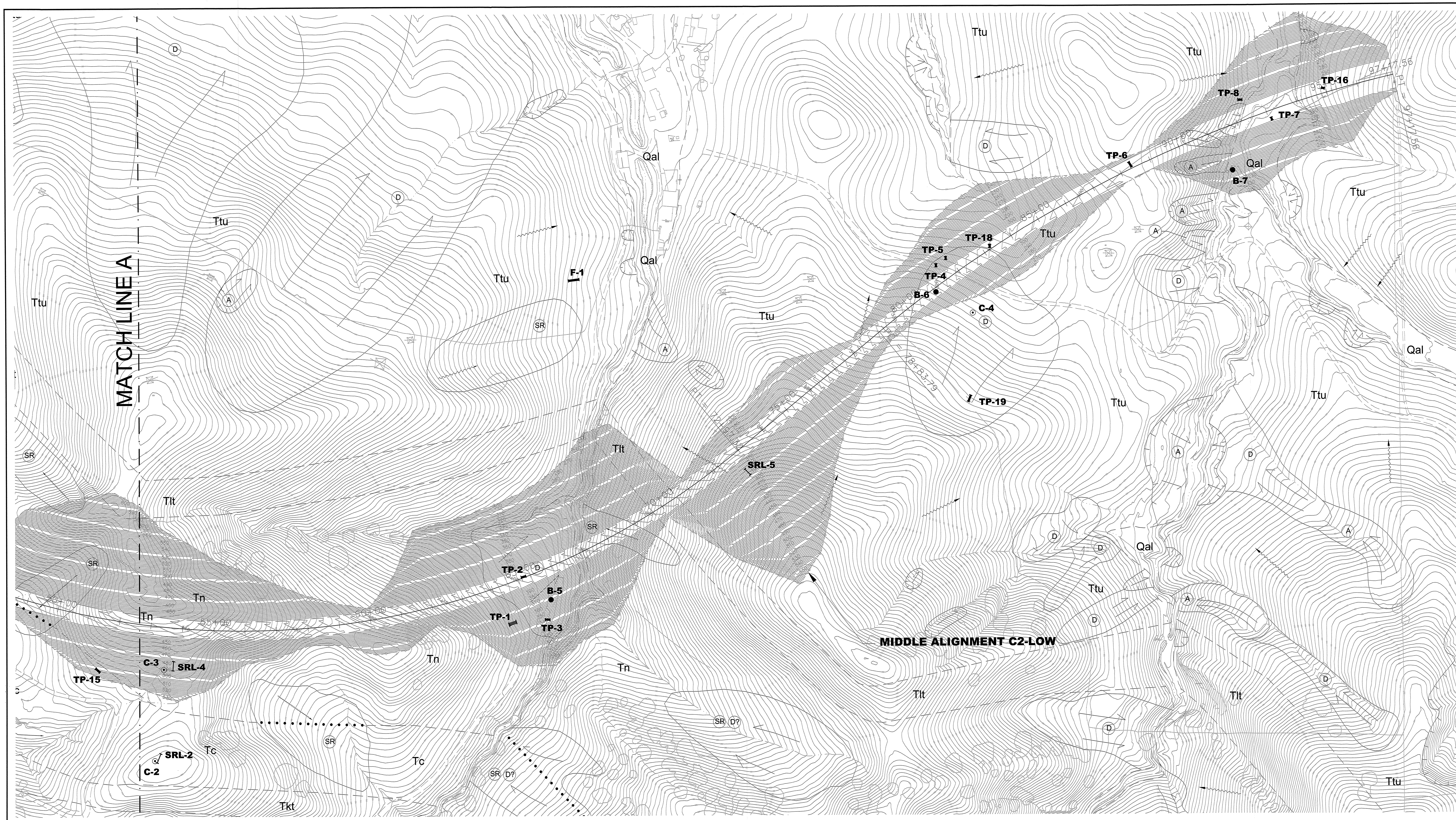
PROJECT NO.	123561
DRAWN:	MAR 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	BASE MAP w LOW profile.dwg

MIDDLE ALIGNMENT (C2-LOW)

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
6A



EXPLANATION

— — ······ APPROXIMATE GEOLOGIC CONTACT — dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SLOPE WASH (arrows indicate direction of movement)

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APPROXIMATE SCALE (feet)

REFERENCE:
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KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123561	MIDDLE ALIGNMENT (C2-LOW)	PLATE
	DRAWN: MAR 2012		6B
	DRAWN BY: JDS		
	CHECKED BY: SD		
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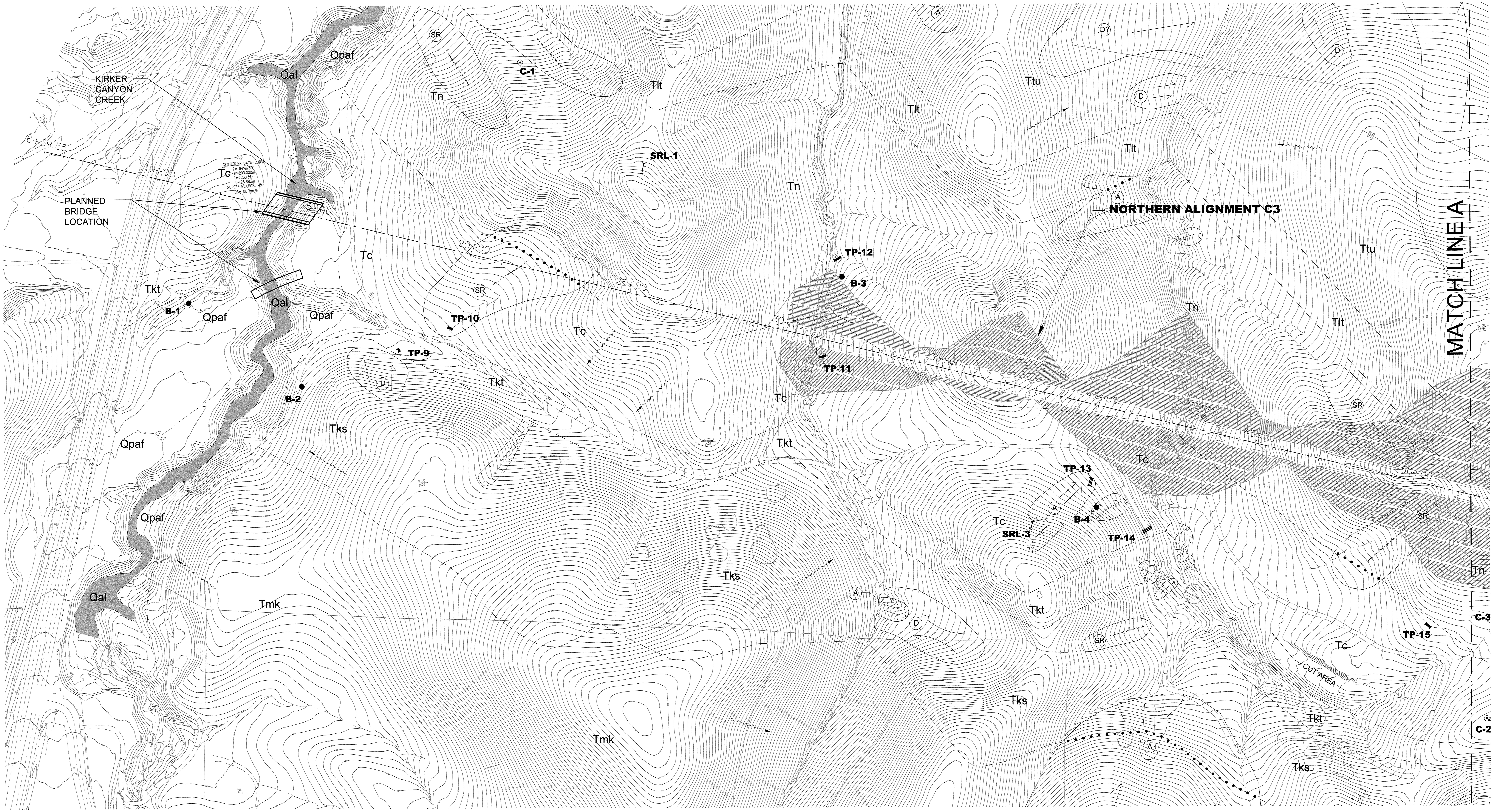

EXPLANATION

APPROXIMATE GEOLOGIC CONTACT – dotted where buried	
CREEPING COLLUVIUM	
INCISED EROSION RILLING OR SHALLOW SLUMP	
SLOPE WASH (arrows indicate direction of movement)	
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APPROXIMATE SCALE (feet)

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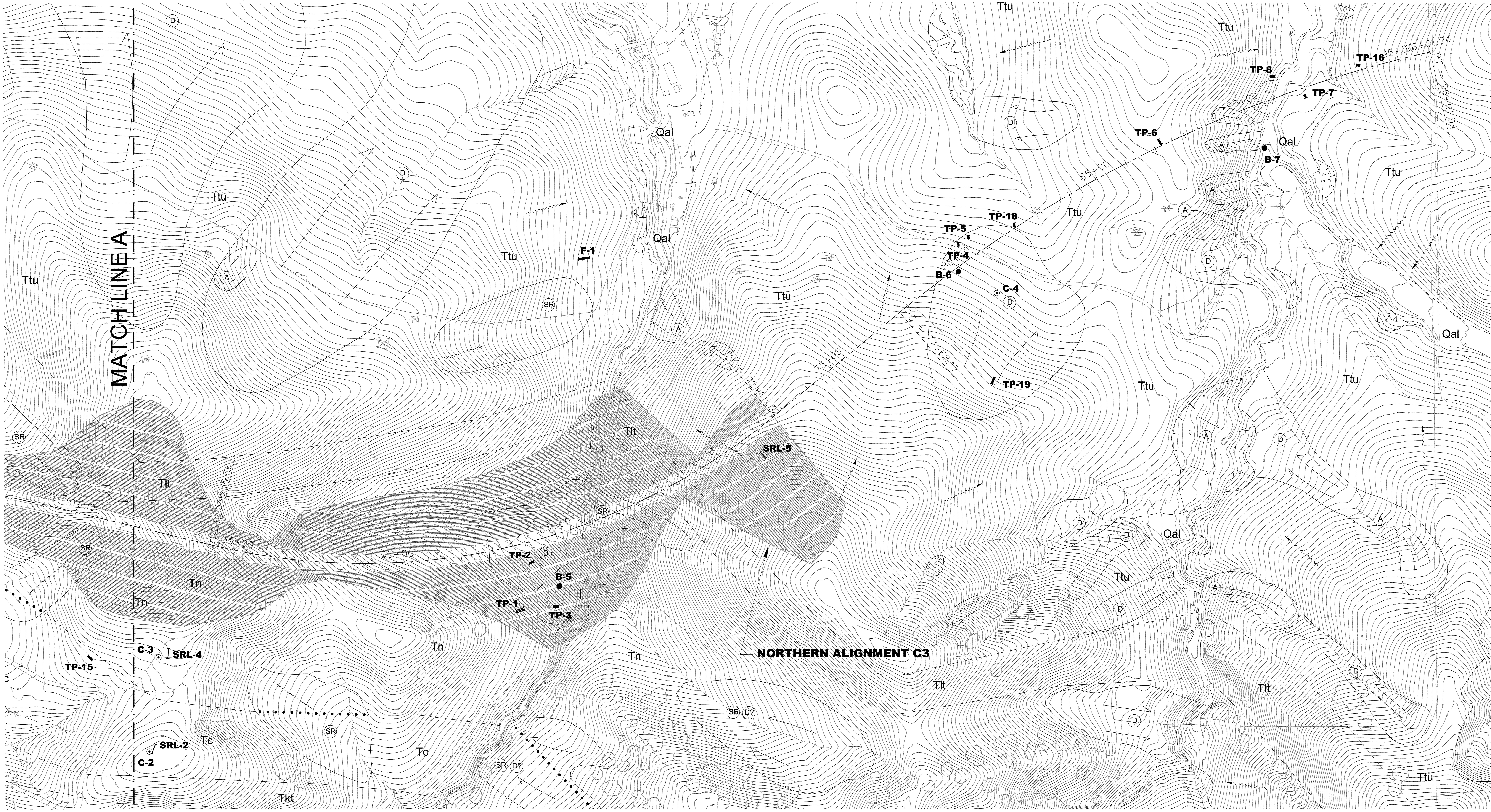
FILE NAME:	BASE MAP w LOW profile.dwg

NORTHERN ALIGNMENT (C3)

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
7A



EXPLANATION

APPROXIMATE GEOLOGIC CONTACT — dotted where buried

CREEPING COLLUVIUM

INCISED EROSION RILLING OR SHALLOW SLUMP

SLOPE WASH (arrows indicate direction of movement)

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SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)

N

100 50 0 100
APPROXIMATE SCALE (feet)

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11.

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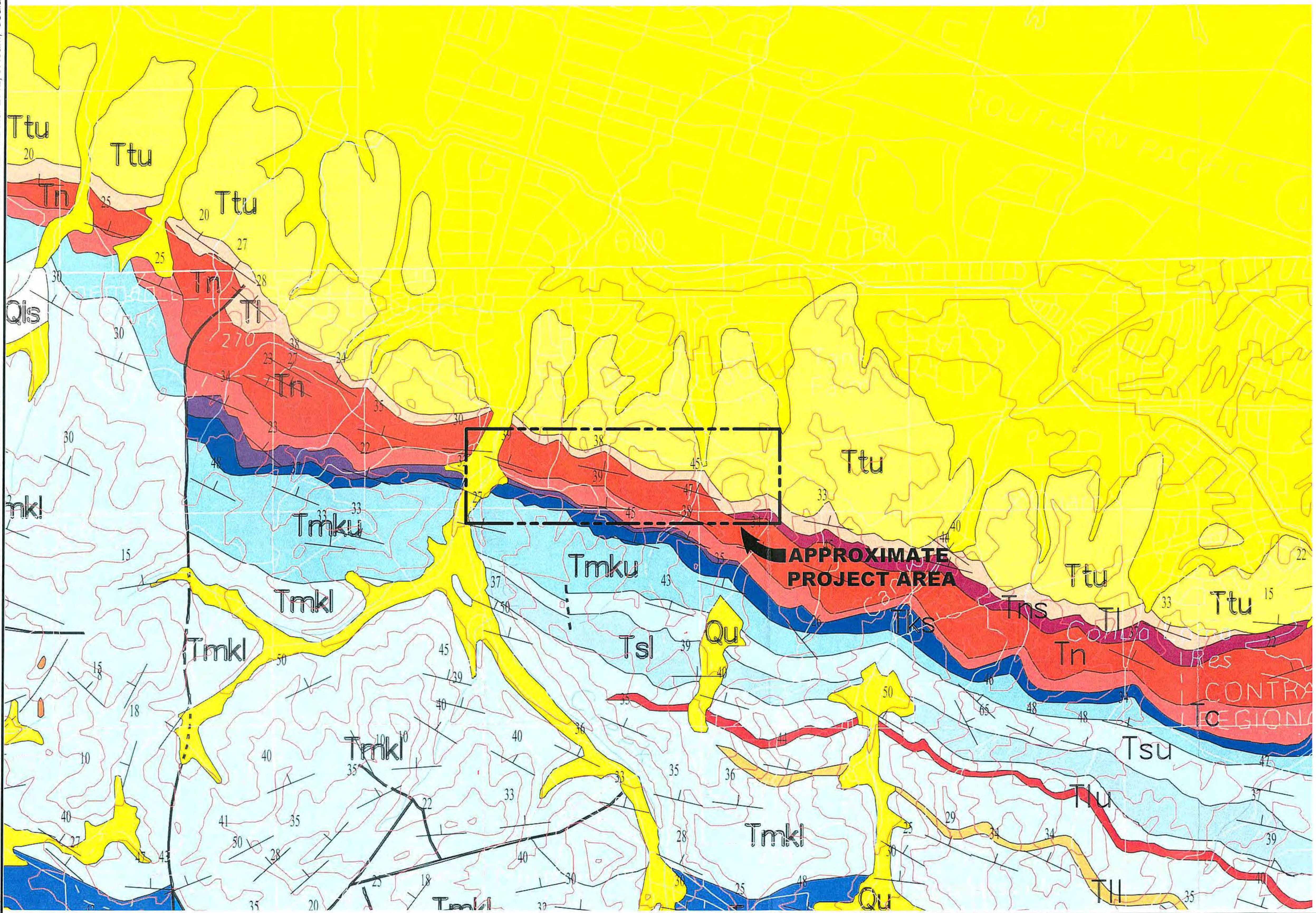
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	BASE MAP w LOW profile.dwg

NORTHERN ALIGNMENT (C3)

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
7B



MAP EXPLANATION

- Qu – Undivided Quaternary deposits
- Qls – Landslide deposits
- Ttu – Tulare Formation
- Tl – Lawlor tuff
- Tns – Neroly Formation, siltstone beds
- Tn – Neroly Sandstone
- Tc – Cierbo Sandstone
- Tkt – Kirker Tuff
- Tks – Kirker Tuff, sandstone beds
- Tmku – Markley Formation, upper member
- Tsu – Markley Formation, upper Sidney Flat Shale member
- Tsl – Markley Formation, lower Sidney Flat Shale member
- Tmkl – Markley Formation, lower member
- Tlu – Markley Formation, lower member, upper siltstone beds
- Tll – Markley Formation, lower member, lower siltstone beds

- Contact
- Contact, approximately located
- Contact, inferred
- Contact, uncertain
- Contact, concealed
- Fault
- Fault, approximately located
- Fault, inferred
- Fault, uncertain
- Fault, concealed
- Fault, concealed and uncertain
- Fold axis
- Fold axis, inferred
- Fold axis, concealed
- Strike and dip of bedding

N

2500 1250 0 2500

APPROXIMATE SCALE (feet)

REFERENCE:
Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California
Compiled by R.W. Graymer, D.L. Jones, and E.E. Brabb, 1994

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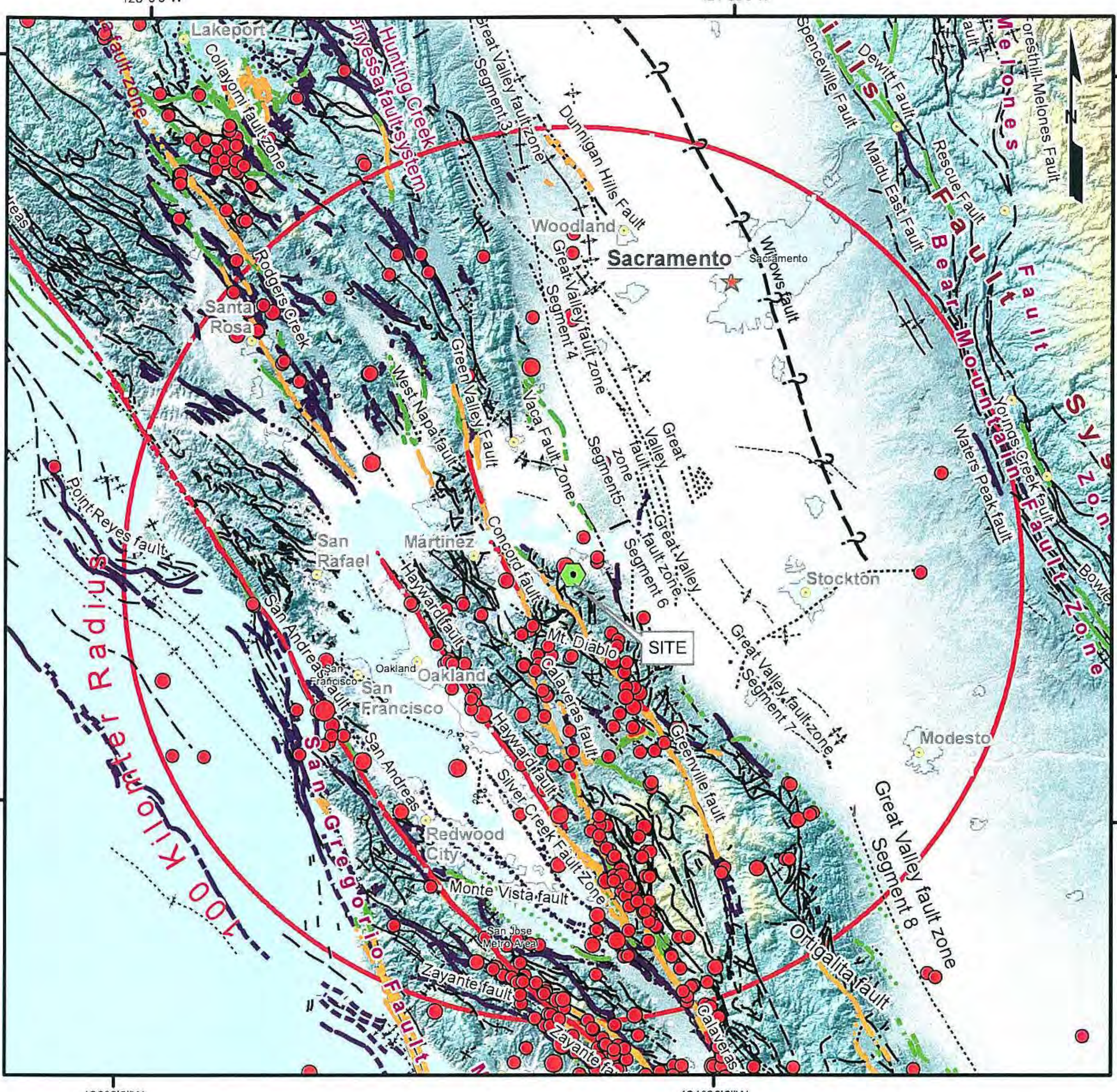
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	VIC-PLAN.dwg

AREA GEOLOGY MAP

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
8

ATTACHED IMAGES: Images: 20120120172112_00001.jpg Images: AREA GEOLOGY LEGEND.jpg Images: SITE-VIC.jpg Images: SITEPLAN.jpg
ATTACHED XREFS: XRef: Eng-B_11x17_L_StyleA
PLEASANTON, CA CAD FILE: C:\Users\JSala\Documents_Archive_PROJECTS\123561-\ LAYOUT: AREA GEOLOGY PLOTTED: 06 Mar 2012, 9:40am, JSala



Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 5.0
- 5.1 - 6.0
- 6.1 - 7.0
- 7.1 - 8.0
- 8.1 - 9.0

35 17.5 0 35
Kilometers

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PROJECT NO.	123561
DRAWN:	2/21/12
DRAWN BY:	I.McGovern
CHECKED BY:	S.Derrega
FILE NAME:	123561.MXD

FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - DECEMBER 2011)

JAMES DONLON BOULEVARD EXTENSION
PITSBURG, CALIFORNIA

PLATE
9

Appendix E.3

Supplemental Engineering Geologic and Geotechnical Report James Donlon Boulevard Alignment Extension Middle Alignment (C2-Low) Alternative

Prepared for **RBF Consulting**

REVISED SUPPLEMENTAL ENGINEERING GEOLOGIC AND GEOTECHNICAL REPORT JAMES DONLON BOULEVARD ALIGNMENT EXTENSION MIDDLE ALIGNMENT (C2-LOW) ALTERNATIVE PITTSBURG, CALIFORNIA

November 30, 2012
File No.: 123460/PWGEO

KLEINFELDER
Bright People. Right Solutions.

November 30, 2012
File No. 123460/PWGEO

Mr. William J. Conyers, Senior Associate
RBF Consulting
500 Ygnacio Valley Road, Suite 300
Walnut Creek, California 94596-3847
wconyers@rbf.com

SUBJECT: Revised Supplemental Engineering Geologic and Geotechnical Report for the James Donlon Boulevard Alignment Extension, Middle Alignment (C2-Low) Alternative, Pittsburg, California

Dear Mr. Conyers:

We are pleased to submit four (4) bound copies of our revised supplemental engineering geologic and geotechnical report for the James Donlon Boulevard Alignment Extension, Middle Alignment (C2-Low) Alternative, planned along the southern hilly portion of Pittsburg, California. This report was previously issued as a draft on September 6, 2012. This report was subsequently finalized and issued on October 2, 2012 and it is being revised herein to respond to RBF Consulting review comments of our draft report.

The discussions, conclusions, and recommendations presented in this report are intended to supplement those presented in our report titled *Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass in Pittsburg, California*, dated January 9, 2008 (File No. 75856/PWGEO) and where there is a conflict, the recommendations presented herein supersede relevant discussions and recommendations contained in our referenced 2008 report.

Based on our findings, it is our opinion that the proposed Middle Alignment (C2-Low) is feasible provided that the conclusions and supplemental recommendations presented in this report and our January 9, 2008 report are incorporated into the design and construction of the project.

In summary, the proposed south-facing cut slopes extending from approximately Stations 18+00 to 29+00 may be steepened from the proposed 2H:1V (horizontal to vertical) to a 1.75H:1V gradient and the south-facing cut slope extending from approximately Stations 46+50 to 61+00 may be steepened from the proposed 2H:1V to a 1.5H:1V gradient. We are not recommending that the south-facing cut slopes along the selected alignment be overexcavated and rebuilt as fill buttresses. However, we are recommending that nearly all north-facing cut slopes be overexcavated and rebuilt as fill slopes that are supported on subdrained base keyways. Our attached remedial grading plans delineate the limits of the recommended remedial grading where cut slopes are to be overexcavated and rebuilt. Depending on the height of fill and cut slopes, we are recommending that 6- to 12-foot wide benches be constructed and concrete V-ditches be installed along the inboard side of the noted drainage benches.

In addition, the proposed slope gradient of the fill embankment planned from approximately Stations 60+00 to 70+00 may be steepened to 1H:1V if the fill is supported by a mechanically stabilized earth geosynthetic system similar to that described in this report. All the remaining proposed fill slopes are considered feasible and we are recommending herein that all planned fill slopes be supported on base keyways that are subdrained and extended into bedrock or unyielding firm soils.

We are not including remedial grading recommendations for the planned cut slopes between approximately Stations 70+00 and 78+00 because of several factors that we identify in this report. We are however recommending further subsurface investigation and characterization of the subsurface geologic conditions of these proposed cut slopes. Additional discussions and specific recommendations regarding these and other aspects of the project are contained in the report.

On a preliminary basis, the proposed retaining walls planned near the western end of the roadway alignment are considered feasible. We recommend performing additional site-specific field exploration and laboratory testing to characterize the subsurface conditions anticipated in our preliminary evaluation of the walls, evaluate depth to bedrock, and provide retaining wall design parameters and recommendations.

The conclusions and recommendations presented in this report are based on limited field reconnaissance, document review, subsurface exploration and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil/bedrock conditions may be found in localized areas of the site during construction.

Kleinfelder should review the project plans and specifications prior to construction bidding, to confirm that they are in compliance with the recommendations presented in this report and our January 9, 2008 report.

It is also recommended that Kleinfelder be retained during construction so that our engineering geologist is able to observe the subsurface conditions encountered during grading and, if deemed necessary, to modify our design recommendations presented herein. In addition, Kleinfelder's representatives should be provided the opportunity during construction to observe earthwork operations and the installation of foundations associated with the bridge and retaining walls to allow us to make adjustments, if needed, to our recommendations if varying subsurface conditions are encountered.

We appreciate the opportunity of providing our services to you on this project and trust that this report meets your needs at this time. If you have any questions concerning the information presented, please contact Sadek Derrega at (925) 484-1700 or Fernando Silva at (925) 427-6477.

Sincerely,

KLEINFELDER WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
CRISTIANO B.C. DE MELO
No. 2756
Exp. 09-30-14
GEOTECHNICAL
STATE OF CALIFORNIA

Cristiano Melo, PE, GE #2756
Project Geotechnical Engineer

PROFESSIONAL GEOLOGIST
SADEK M. DERREGA
No. 2175
Exp. 03/31/13
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Sadek M. Derrega, PG, CEG #2175
Principal Engineering Geologist

Fernando J. Silva, CE, GE #2519
Senior Project Manager

CM/SMD/FJS/jmk

REVISED SUPPLEMENTAL ENGINEERING GEOLOGIC AND GEOTECHNICAL RECOMMENDATIONS REPORT
JAMES DONLON BOULEVARD ALIGNMENT EXTENSION
MIDDLE ALIGNMENT (C-2 LOW) ALTERNATIVE
PITTSBURG, CALIFORNIA

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APPENDIX D – Proposed Retaining Walls (RBF Exhibit B-4)

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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REVISED SUPPLEMENTAL ENGINEERING GEOLOGIC AND GEOTECHNICAL RECOMMENDATIONS REPORT
JAMES DONLON BOULEVARD ALIGNMENT EXTENSION
MIDDLE ALIGNMENT (C2-LOW) ALTERANTIVE
PITTSBURG, CALIFORNIA

1 INTRODUCTION

This report presents our revised supplemental engineering geologic and geotechnical recommendations for the James Donlon Boulevard Alignment Extension in accordance with our scope of work presented our July 29, 2011 proposal[1] (File No. 120125) and our April 26, 2012 request for budget increase letter[2] (File No. 120125). This report was previously issued as a draft on September 6, 2012.

The findings, discussions, and recommendations contained in this report apply to the currently proposed alignment, identified as the Middle Alignment (C2-Low) which has been selected for final design. All stations referenced in this report are based on the stationing system for the Middle Alignment (C2-Low) as presented on the grading prepared by RBF for the noted alignment. The conclusions and recommendations presented in this report are intended to supplement the conclusions and recommendations contained in our referenced January 9, 2008 report, and where conflicts arise they are to supersede relevant discussions and recommendations contained in our January 9, 2008[3] report. The discussions and recommendations presented in our January 9, 2008 report remain applicable to the project where they are not in conflict with this supplemental report.

[1] Titled *Proposal to Updated Geotechnical Investigation Report, James Donlon Boulevard Extension, Pittsburg, California.*

[2] Titled *Request for Budget Increase to Provide Preliminary Retaining Wall Evaluation, Update Geotechnical Investigation Report, Proposed James Donlon Boulevard Extension, Pittsburg, California.*

[3] Titled *Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass in Pittsburg, California*, dated January 9, 2008 (File No. 75856/PWGEO).

1.1 PROJECT LOCATION AND DESCRIPTION

The general location of the proposed James Donlon Boulevard roadway alignment extension is shown on Plate 1, Site Vicinity Map. The Middle Alignment (C2-Low) is shown on Plate 2, Aerial Site Plan.

The proposed James Donlon Boulevard extension project will consist of constructing an approximately 1.6-mile long section of new roadway. The new roadway will extend from Kirker Pass Road, marking the western project limit, to the western property line of the planned Sky Ranch residential development, which marks the eastern terminal point of the roadway extension alignment. The proposed alignment extension will be an east/west, limited-access arterial roadway in the undeveloped hills south of the City of Pittsburg (City). We understand that east of the western property line of the Sky Ranch development the roadway alignment will be constructed by others.

Besides the roadway and associated drainage facilities, other project features associated with the proposed roadway extension will include the following:

- Five culverts along five stream crossings;
- Two bridges across the Kirker Creek channel;
- Cut slopes that measure more than 190 feet in height and embankment fills with heights exceeding 160 feet; and
- Several hundred lineal feet of retaining walls are anticipated.

The roadway alignment extension is anticipated to encounter geologic materials consisting of alluvium, colluvium/slope wash, active and dormant landslides, Tulare formation claystone, Lawlor Tuff, Neroly formation sandstone and siltstone, Cierbo formation sandstone, and Kirker formation tuffaceous siltstone and sandstone materials.

1.2 BACKGROUND AND DOCUMENT REVIEW

Kleinfelder previously prepared a report titled *Geological and Geotechnical Constraints Evaluation Report for the Proposed Buchanan Road in Pittsburg, California*, dated September 20, 2002 (File 16656/GE1). Our 2002 report evaluated three optional alignments (Northern, Central, and Southern) developed by the City and RBF Consulting (RBF). Our evaluation was based on field mapping by our Certified Engineering Geologist (CEG) and review of aerial photographs and published and unpublished geologic and seismic reports and maps covering the site area. Our 2002 report concluded that the Central Alignment would require the least amount of mitigation needed to address the geologic, seismic, and geotechnical constraints and considerations identified during our evaluation. No subsurface exploration was performed as part of our noted 2002 assessment.

The City and RBF subsequently selected the Central Alignment and prepared a preliminary grading scheme, which they provided to us. Kleinfelder conducted a subsurface exploration program as part of a design-level geotechnical investigation that was designed to identify and characterize the subsurface conditions along the selected Central Alignment extension and to evaluate the feasibility of the noted grading scheme and the stability of cut and fill slopes proposed along the alignment. The results, conclusions, and recommendations of our study were presented in a report titled *Geological and Geotechnical Investigation Report for the Proposed Buchanan Road Bypass in Pittsburg, California*, dated January 9, 2008 (File No. 75856/PWGEO).

Since the issuance of our referenced 2008 report, the above-noted and previously selected Central Alignment has been re-labeled as "Alignment (C1)" and is referred to in this report as such. In addition to Alignment (C1), four additional alignment alternatives to Alignment (C1) have been developed by RBF and the City. They are identified as “Alignment (C1-Low)”, "Middle Alignment (C2)", "Middle Alignment (C2-Low)", and "Northern Alignment (C3)". Three of these additional alternative alignments extend east/west in a parallel fashion to Alignment (C1) and are situated immediately to the north of it. Alignment (C1-Low) generally matches Alignment (C1), but its overall elevations are lower. Note that the current version of Alignment (C1) includes minor modifications to the original grading plan included in our 2008 report.

It is important to note that the Middle Alignment (C2) and the Northern Alignment (C3) do not extend along the entire length of the proposed Alignment (C1) and are only shown on topographic base maps generated by RBF and transmitted to us on December 12, 2011 to extend between approximate Stations 30+00 and 74+00 of Alignment (C1). However, topographic base maps transmitted to us by RBF on February 9, 2012 for the Middle Alignment (C2-Low) show the noted alignment to extend along the entire length of proposed Alignment (C1). Beyond Stations 30+00 and 74+00 of Alignment (C1), the alignments for alternatives C1 and C2-Low are generally a match. Please note that Reference Station 10+00 marks the western terminal end of the roadway extension and is located at the intersection of Alignment (C1) with Kirker Pass Road based on project plans prepared by RBF.

Further discussion of the five alignment options is presented in the following documents:

- Report titled *Engineering Geologic and Geotechnical Feasibility Report, Four Proposed James Donlon Boulevard Alignment Extension Alternatives, Pittsburg, California*, dated March 7, 2012 (File No. 123561/PWGEO); and
- Letter titled *Limited Geological and Geotechnical Feasibility Study for Proposed Stream Crossing Alternative Original Alignment C1-Low, James Donlon Boulevard Extension Project, Pittsburg, California*, dated May 31, 2012 (File No. 123561/PWGEO).

Based on the conclusions and recommendations presented in our above-referenced March 7, 2012 report and May 31, 2012 letter and additional recommendations provided by other members of the project design team the Middle Alignment (C2-Low) has been selected by RBF and the City for the final design of the project.

In addition to the reports discussed above, Kleinfelder has recently issued two preliminary foundation reports for two planned bridges to span the Kirker Creek channel near the western end of the project. These reports are titled as follows:

- *Foundation Report, James Donlon Boulevard Bridge, James Donlon Boulevard Extension, Pittsburg, California*, dated July 20, 2012 (File No. 123066/PWGEO); and
- *Final Foundation Report, Ramp Bridge, James Donlon Boulevard Extension, Pittsburg, California*, dated May 22, 2012 (File No. 123066/PWGEO).

1.3 PURPOSE AND SCOPE OF SERVICES

Our current scope included the following items:

- Background and document review including published geologic and seismic literature, aerial photographs, and our referenced reports prepared in 2002 and 2008;
- Review of the Middle Alignment (C2-Low) layout and its proposed grading magnitude;
- Field reconnaissance of the alignment by our CEG and one of our Registered Geotechnical Engineers;
- Evaluation of rock outcrop cut reduction;
- Evaluation of the feasibility of steepening the slope gradient for the proposed fill embankment proposed between approximately Stations 60+00 and 70+00 using earth reinforcement;
- Preliminary evaluation of proposed retaining walls at the western limit of the alignment;
- Preparation of plans depicting the recommended remedial grading limits, keyways, and subdrain lines;
- Recommendations for additional field investigation; and
- Preparation of this report presenting our supplemental findings and recommendations.

1.4 AUTHORIZATION

This supplemental engineering geologic and geotechnical study was performed in accordance with our contract with RBF dated October 6, 2011.

2 FIELD RECONNAISSANCE AND GEOLOGIC MAPPING

Our Certified Engineering Geologist (CEG), Sadek M. Derrega, visited the site on December 1, 2, and 13, 2011 to conduct a limited field reconnaissance along the entire proposed alignment. Particular attention was given to the west end of the alignment where the two proposed bridges will cross the Kirker Creek channel and the two prominent proposed cut and fill areas planned along the alignment. Our Geotechnical Engineer (GE), Cristiano Melo, also participated in the December 13, 2011 field reconnaissance.

3 SUPPLEMENTAL FIELD EXPLORATION

We performed a supplemental field exploration as part of our work to collect site-specific subsurface information for the two proposed bridges that will cross Kirker Creek channel and also to evaluate the subsurface conditions for the proposed fill along the west approach to the Ramp Bridge. Our exploration did not pertain to the actual C-2 Low alignment beyond the noted bridges to the east and we are only including the subsurface information and laboratory testing results herein for completeness purposes.

Our field investigation was performed in two phases with hollow stem auger borings drilled on November 29 and 30, 2011 and a test pit excavated on December 1, 2011. We drilled four (4) soil borings (designated B-1 through B-4) to depths ranging from approximately 27 to 52 feet below existing ground surface. One test pit (designated TP-1) was excavated to a maximum depth of approximately 5 feet. The boring and test pit locations are shown on the Remedial Grading Plan, Plate 3A.

3.1 BORINGS

The soils were classified in general accordance with the Unified Soil Classification System presented on the Boring Log Legend, Plate A-1 in Appendix A. Logs of the borings are presented on Plates A-2 through A-5. Water level readings, where encountered or observed, were taken at each boring prior to grouting. Prior to sealing the samples, strength characteristics of the cohesive soil samples recovered were evaluated using a hand-held pocket penetrometer. The results of these tests are shown adjacent to the samples on the boring logs.

3.2 TEST PIT

The test pit was excavated using a rubber-tired backhoe equipped with a 30-inch wide bucket. Excavation depth was about 5 feet below existing grade and the length of the test pit was about 10 feet. The test pit log is presented in on Plate A-6 in Appendix A.

3.3 LABORATORY TESTING

The laboratory testing program included unit weight and moisture content, Atterberg limits, unconsolidated-undrained triaxial, consolidation testing. Most of the laboratory test results are presented on the boring logs. The results of the Atterberg Limits, unconsolidated-undrained triaxial, and consolidation tests are presented graphically on Plates B-1 through B-12, in Appendix B.

Corrosion test results are presented in Plate B-13 in Appendix B. Based upon the resistivity measurements, both samples were classified as “severely corrosive” by CERCO. For additional discussion regarding corrosivity, please refer to CERCO’s report in Appendix B and to Section 8.8 of our January 9, 2008 report.

4 SLOPE STABILITY ANALYSIS

The purpose of our slope stability analysis was to evaluate the possibility of steepening the proposed south-facing prominent cut slopes extending between approximately Stations 18+00 to 29+00 and Stations 46+50 to 61+00 in order to reduce the magnitude of the planned cuts and impacts to the rock outcrop areas near these planned cut slopes. These slopes are currently proposed to be cut at approximate gradients of 2H:1V. As part of our analysis, we developed two slope stability cross sections (identified as 1-1' and 2-2' in Appendix C) through representative portions of these slopes. Slope Stability Cross Section 1-1' was performed at approximately Station 24+30 and Slope Stability Cross Section 2-2' was performed at approximately Station 51+70. Factors of safety computed via slope stability analysis are dependent on the slope configuration, geologic model and the strength parameters of the various geologic units used as well as the elevation of the groundwater table.

We analyzed the stability of the subject slopes for static (long-term) and seismic (short-term) conditions. A brief discussion of our slope stability methodology is presented below.

4.1 SELECTED SOIL STRENGTH PARAMETERS

Our slope stability analysis considered six different bedrock units consisting of: Tertiary Tulare formation (map symbol Ttu), Tertiary Lawlor Tuff (map symbol Tlt), Tertiary Neroly formation (map symbol Tn), Tertiary Cierbo formation (Tc), Tertiary Kirker formation – Volcanic Tuff (Tkt), and Tertiary Kirker Formation – Tuffaceous Sandstone (Tks). The unit weight and strength parameters used in our slope stability analysis are listed in Table 4.1-1 below, and on the individual stability runs presented on Plates C-1 through C-6 in Appendix C. The parameters used in our analysis were obtained from Section 7.1 of our January 9, 2008 report with one exception, which consisted of lowering the cohesion value for the Tertiary Cierbo formation to 1,000 psf from the previously-used cohesion value of 1,750 psf, based on recent experience with the noted formation.

Table 4.1-1: Unit Weight and Strength Parameters

Unit Description	Unit Weight (pcf)	Cohesion (psf)	Phi** (psf)
Tertiary Tulare formation (Ttu)	125	800	20
Tertiary Lawlor Tuff (Tlt)	115	500	36
Tertiary Neroly formation (Tn)	120	1,750	25
Tertiary Cierbo formation (Tc)	120	1,000*	25
Tertiary Kirker formation – Volcanic Tuff (Tkt)	115	1,000	25
Tertiary Kirker formation – Tuffaceous Sandstone (Tks)	115	1,000	25

Notes:
* Based on our experience, this value was intentionally lowered from the 1,750 psf provided in Section 7.1 of our January 9, 2008.
** Phi = internal angle of friction.

4.2 GROUNDWATER

For our slope stability analyses, the groundwater level was assumed to be about 10 and 50 feet below the ground surface for long-term (static) conditions and about 50 feet below the ground surface for short-term (seismic) conditions.

4.3 ANALYSIS METHODOLOGY

The Morgenstern-Price (half-sine function) method and the slope stability program SlopeW were used to perform our analyses. The Morgenstern-Price method is a limit-equilibrium method that rigorously satisfies static equilibrium. The seismic slope stability analysis was based on the pseudo-static method. Based on the methodology provided by the Southern California Earthquake Center (SCEC, 2002) and Special Publication 117A (CGS, 2008), we used a horizontal seismic coefficient of 0.16 in our analyses for seismic conditions. This value was based on an estimated maximum horizontal acceleration of 0.42g, an estimated mode magnitude and distance of M6.6 and 14½ km, respectively, of the causative earthquake, and a displacement threshold of 15 centimeters. The earthquake parameters used to estimate the seismic coefficient were based on a 10 percent probability of exceedance in 50 years (equivalent to 475-year return period) based on the USGS's online deaggregation tool available at https://geohazards.usgs.gov/deaggint/2008/.

4.4 SLOPE STABILITY RESULTS

According to SCEC (2002)[4] and Special Publication 117A (CGS, 2008)[5], slopes are considered stable when their factors of safety (FOS) are greater than or equal to 1.5 and 1.0 under static and seismic conditions, respectively. Our slope stability results are presented graphically on Plates C-1 through C-6 in Appendix C and are summarized in Table 4.4-1 below.

Table 4.4-1: Slope Stability Results

Cross Section (Approximate Station)	Slope Gradient Used	Condition	Approximate Groundwater Level Used (feet)	FOS	Plate Number
1-1' (Sta. 24+30)	1.75H:1V	Static	50	1.6	C-1
	1.75H:1V	Static	10	1.3	C-2
	1.75H:1V	Seismic	50	1.2	C-3
2-2' (Sta. 51+70)	1.5H:1V	Static	50	1.6	C-4
	1.5H:1V	Static	10	1.4	C-5
	1.5H:1V	Seismic	50	1.2	C-6

Based on our slope stability results, the proposed south-facing prominent cuts may be steepened to 1.75H:1V between approximate Stations 18+00 and 29+00 and to 1.5H:1V between approximate Stations 46+50 and 61+00. Although Plates C-2 and C-5 indicate FOS values lower than 1.5, these values are associated with an assumed groundwater level of about 10 feet below the ground surface. We believe that this assumed groundwater level is very conservative and it is unlikely to occur at these slopes even during above-average wet years. Based on our previous and recent

[4] Southern California Earthquake Center (SCEC), 2002, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California*.
[5] California Geological Survey (CGS, 2008), *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A, 2008.

subsurface exploration points, and the elevation of the nearby prominent drainage courses and creeks, we believe that even assuming a groundwater level of 50 feet below the ground surface for the pertinent slopes is conservative and deeper groundwater levels could be used in our analysis.

5 ROCK OUTCROP REDUCTION

As is noted in the “Slope Stability Analysis” section above and in the “Corrective (Remedial) Grading” section below, we have recommended cutting the two prominent, relatively high south-facing cut slopes (Approximate Stations 18+00 to 29+00 and 46+50 to 61+00) to steeper gradients than those proposed on the civil grading plans. Our recommendations were made based on the favorable bedding, nature of the Neroly formation, and our slope stability analysis results. The steepening of the two noted slopes will help reduce the magnitude of the initially planned cuts and will reduce encroachment onto rock outcrops.

6 DRAINAGE TERRACES (BENCHES)

Refer to the "Corrective (Remedial) Grading" section on next page.

7 CORRECTIVE (REMEDIAL) GRADING

Corrective (remedial) grading is needed almost along the entire length of the selected C-2 Low alignment. All proposed fill and cut slopes where overexcavation and rebuilding is recommended will need to be supported on keyway excavations that extend a minimum of 10 feet into the underlying bedrock or firm unyielding soil. The keyway depth may need to be adjusted based on the engineering geologist's observations in the field during grading. Plate 4 shows a typical fill slope buttress detail, while Plate 5 presents a typical keyway buttress fill for cut slope reconstruction.

Perforated subdrain pipes encased in Caltrans Class 2 permeable material will need to be placed along the entire length of the keyway heels and directed to provide positive gravity flow where approximately delineated on the attached Remedial Grading Plans (Plates 3A and 3B). Plate 6 presents typical subdrain details for keyway and canyon fill areas. Subdrain pipe sections should be glued where they intersect, or where they are raised for cleanouts, 45-degree angled connectors should be installed. Subdrain lines should be added above the base of the keyway elevation at approximate intervals of about 25 vertical feet of fill. Subdrain pipe Type SDR-35 should be used where fill thickness is less than about 30 feet vertically. Where the fill thickness exceeds 30 vertical feet, subdrain pipe Type SDR-23.5, with thicker walls, should be used instead. For fill areas planned within v-shaped drainage courses, all the recommended lateral tributary perforated subdrain pipes should be connected to a solid 12-inch diameter subdrain collector pipe that extends along the axis of the swale in a parallel fashion to the perforated line recommended along the axis of drainage swale and daylighted further down the drainage swale. The upper ends of the solid collector pipes should be capped and the lower ends of all subdrain lines (solid or perforated) should be fitted with grates to prevent animal access and debris introduction.

Fill placement and preparation of areas to receive fill should be performed in accordance with the recommendations presented in our January 9, 2008 report.

In general, the keyway widths recommended and shown on Plates 3A and 3B for both proposed fill slope areas and cut slope areas to be overexcavated and rebuilt, measure about half of the vertical height of the proposed slopes. This selected keyway width aspect ratio is based on slope stability analysis, experience gained on previous mass grading projects, and on the type of soil and fill material to be derived from the bedrock formations in this general area. We anticipate the depth of the keyway excavations to extend about 10 feet into the underlying bedrock or into firm unyielding soils. Actual keyway depths will need to be established by our engineering geologists in the field during grading based on the encountered subsurface conditions.

Remedial grading details pertaining to each proposed fill and cut slope location are discussed below starting from the west end of the project and are approximately delineated on Plates 3A and 3B. The approximate grading limits are also delineated on the noted plates.

It is important to note that the width of the fill buttresses proposed at cut slope areas, where we recommend such slopes be overexcavated and rebuilt, should be maintained the same as that of the recommended keyway width throughout their entire height. This implies that the temporary backcut slope should be cut at the same gradient as the final slope face (2H:1V).

7.1 PROPOSED FILL SLOPES (BETWEEN APPROXIMATE STATIONS 29+00 AND 35+00)

The roadway alignment at this location will cross a prominent drainage swale that drains northward. Fill placed across the drainage swale will create south- and north-facing fill slope portions that will measure approximately 65 and 75 feet in vertical height, respectively. The two fill slopes will be wedge-shaped and their widths will increase with height. As is delineated on Plate 3A, we are recommending that the south-facing fill slope portion planned along the south side of the roadway be supported on a 30-foot wide keyway excavation while the north-facing fill slope portion to the north of the roadway is supported on a 40-foot wide keyway excavation.

Perforated subdrain pipes should be placed along the heels of both keyway excavations and also be extended along the axis of the drainage swale daylighting to the north of the north-facing slope portion. The outfall pipe to be located beyond the toe of the north-facing fill slope portion should be non-perforated, solid, and rigid. A non-perforated, solid subdrain collector pipe should be extended along the axis of the drainage course (parallel to the axial perforated subdrain line) and all tributary perforated subdrain lines should be connected to this collector pipe as shown on Plate 3A. Perforated subdrain pipes Type SDR-35 should be installed within the keyway excavations while Type SDR-23.5, with thicker walls, should be installed along the axis of the swale where the fill depth increases as previously discussed. Subdrain cleanout risers should be installed at all upper/terminal ends of the perforated pipes where approximately delineated on Plate 3A.

Because the proposed north- and south-facing fill slopes are not very wide along their toes, we are recommending a single, 12-foot wide drainage terrace (bench) at mid-height, which would facilitate the surficial drainage along the planned slope faces and allow for equipment access. A concrete V-ditch should be constructed along the inboard side of the drainage bench.

This planned fill area will be underlain by Neroly and Cierbo sandstone formations, which are well consolidated and granular. In addition, we anticipate the bedrock materials to be present at relatively shallow depths along the swale axis. Accordingly, we do not anticipate relatively thick wet and soft alluvial sediments to be present along the drainage course axis. Subsequent settlement of the supporting soils/bedrock induced by fill placement is not anticipated to be significant. Refer to Section 8.3.3 of our January 9, 2008 report for further discussion of anticipated settlements in deep fill areas.

7.2 PROPOSED FILL SLOPES (BETWEEN APPROXIMATE STATIONS 38+50 AND 47+00)

The roadway alignment at this location will cross a second prominent drainage swale that also drains northward. Fill placed across the drainage swale will create south- and north-facing fill slope portions that will measure approximately 80 and 100 feet in vertical height, respectively. The two fill slopes will be wedge-shaped and their widths will increase with height. As is delineated on Plate 3A, we are recommending that the south-facing fill slope portion planned along the south side of the roadway be supported on a 40-foot wide keyway excavation while the north-facing fill slope portion to the north of the roadway is supported on a 50-foot wide keyway excavation.

Perforated subdrain pipes should be placed along the heels of both keyway excavations and also be extended along the axis of the drainage swale daylighting to the north of the north-facing slope portion. The outfall pipe located beyond the toe of the north-facing fill slope portion should be non-perforated, solid, and rigid. A non-perforated, solid subdrain collector pipe should be extended along the axis of the drainage course (parallel to the axial perforated subdrain line) and all tributary perforated subdrain lines should be connected to this collector pipe as shown on Plate 3A. Subdrain pipes Type SDR-35 should be installed within the keyway excavations while Type SDR-23.5, with thicker walls, should be installed along the axis of the swale where the fill depth increases as previously discussed. Subdrain cleanout risers should be installed at all upper/terminal ends of the perforated pipes where approximately delineated on Plates 3A and 3B.

A single, 12-foot wide drainage terrace (bench) is recommended at mid-height of the approximately 80-foot high, south-facing fill slope portion proposed along the south side of the roadway, which would facilitate the surficial drainage along the planned slope face and allow for equipment access.

For the north-facing fill slope portion planned to the north of the roadway, we are recommending that a 12-foot wide drainage bench be constructed at mid-height. In addition, two 6-foot wide drainage benches should also be constructed; one between the top of the planned fill slope and the noted 12-foot wide drainage bench (at mid-height) and the second between the toe of the slope and the noted 12-foot drainage bench (at mid-height).

Concrete V-ditches should be constructed along the inboard side of all drainage benches.

This planned fill area will be underlain mostly by the Neroly and Cierbo sandstone formations, except for the southernmost portion of the keyway excavation for the south-facing slope, which will encroach onto the Kirker volcanic tuff bedrock. All three bedrock formations are well consolidated although the Kirker volcanic tuff materials are expected to be highly expansive and corrosive when broken down mechanically. Accordingly, fill materials generated from the Kirker Tuff should be placed at depth and not near the ground surface where the roadway pavement and improvements are planned. We anticipate the bedrock materials to be present at relatively shallow depths along the swale axis. Based on this information, we do not anticipate relatively thick, wet and, soft alluvial sediments to be present along the drainage course axis. Subsequent settlement of the supporting soils/bedrock induced by fill placement is not anticipated to be significant. Refer to Section 8.3.3 of our January 9, 2008 report for further discussion of anticipated settlements in deep fill areas.

The southern and southwestern portions of the planned fill in this area will encroach on several relatively shallow and active landslides, the largest of which is labeled as Landslide 2 on Plate 3A. These landslides are not anticipated to impact the planned fill construction and special measures should not be needed before or during the planned grading.

7.3 PROPOSED FILL SLOPES (BETWEEN APPROXIMATE STATIONS 60+00 AND 70+00)

The roadway alignment at this location will cross the third and most prominent drainage swale, which also drains northward. Fill placed across the drainage swale will create south- and north-facing fill slope portions that will measure approximately 120 and 160 feet in vertical height, respectively. The two fill slopes will be trapezoidal-shaped and their widths will increase with height. As is delineated on Plate 3B, we are recommending that the south-facing fill slope portion planned along the south side of the roadway be supported on a 60-foot wide keyway excavation while the north-facing fill slope portion to the north of the roadway is supported on a 70-foot wide keyway excavation.

Perforated subdrain pipes should be placed along the heels of both keyway excavations and also be extended along the axis of the drainage swale daylighting to the north of the north-facing slope portion. The outfall pipe located beyond the toe of north-facing fill slope portion should be non-perforated, solid, and rigid. A non-perforated, solid subdrain collector pipe should be extended along the axis of the drainage course (parallel to the axial perforated subdrain line) and all tributary perforated subdrain lines should be connected to this collector pipe as shown on Plate 3B. Subdrain pipes Type SDR-35 should be installed within the keyway excavations while Type SDR-23.5, with thicker walls, should be installed along the axis of the swale where the fill depth increases as previously discussed. Subdrain cleanout risers should be installed at all upper/terminal ends of the perforated pipes where approximately delineated on Plates 3A and 3B.

For both south- and north-facing slope portions, we are recommending that a single, 12-foot wide drainage terrace (bench) be constructed at mid-height, which would facilitate the surficial drainage along the planned slope faces and allow for equipment access. In addition, two 6-foot wide drainage benches should be constructed; one between the top of the planned fill slope and the noted 12-foot wide drainage (at mid-height) bench and the second between the toe of the slope and the noted 12-foot drainage bench (at mid-height), for both the south- and north-facing fill slope portions.

Concrete V-ditches should be constructed along the inboard side of all drainage benches.

The planned fill in this location is shown on Plate 3B to have an approximate gradient of 2H:1V. Our proposed remedial grading scheme, which includes the locations of the keyway excavations and subdrain pipes, is shown along the toe of the noted slopes. However, as part of our current study (and as discussed later in this report), we evaluated the feasibility of constructing mechanically stabilized (reinforced) earth geosynthetic fill slopes with approximate gradients of 1H:1V. If this alternative is implemented, the position of the toes of both the south- and north-facing slopes could be shifted significantly towards the roadway. Accordingly, the position of the keyway excavations and associated subdrain lines would also need to be shifted although the keyway geometries will mostly remain unaltered.

We mapped a relatively large landslide deposit, delineated as Landslide 3 on Plate 3B, along the east-facing western swale bank and generally extending between Station 64+00 and the axis of the swale near Station 66+90. This landslide, which occurred in the Neroly sandstone is considered dormant and is not showing signs of current movement. We do not anticipate that a significant concentration of soft landslide material or water seepages will be encountered during grading within the landslide limits. Hence, we believe the potential for subsequent settlement of the supporting soils/bedrock induced by fill placement is considered to be very low in this area. However, the area will still require preparation to receive fill in accordance with our January 9, 2008 report recommendations. The fill placement atop and around the mapped landslide mass should effectively encapsulate it and should reduce the risk for it to reactivate in the future.

Furthermore, during our repeated site reconnaissance and mapping visits to the site, we observed bedrock outcrops exposed along the axis of the drainage swale, which indicates that the central portion of the swale lacks relatively deep soft alluvial sediment.

Nearly this entire proposed fill area will be underlain by the favorable Neroly sandstone and Lawlor Tuff materials, which are well consolidated and granular. However, the northeast corner of the keyway excavation to underlie the toe portion of the north-facing fill slope planned to the north of the roadway is anticipated to be underlain by clay-rich Tulare formation sediments. Nonetheless, if the 1H:1V slope gradient configuration is selected and the keyway excavation in that area is shifted southward, then the entire fill area (including the keyway) should be underlain by favorable Neroly and Lawlor bedrock materials.

7.4 PROPOSED FILL SLOPES (BETWEEN APPROXIMATE STATIONS 78+50 AND 87+50)

The roadway alignment at this location will cross near the head of a fourth, less prominent northward-trending topographic swale that also drains northward. Fill placed across the drainage swale will create south- and north-facing fill slope portions that will measure approximately 50 and 80 feet in vertical height, respectively. The two fill slopes will be oval-shaped and their widths will increase with height. As is delineated on Plate 3B, we are recommending that the south-facing fill slope portion planned along the south side of the roadway be supported on a 30-foot wide keyway excavation while the north-facing fill slope portion to the north of the roadway is supported on a 40-foot wide keyway excavation.

Perforated subdrain pipes should be placed along the heels of both keyway excavations and also extended along the axis of the drainage swale daylighting to the north of the north-facing slope portion. The outfall pipe beyond the toe of the north-facing fill slope portion should be non-perforated, solid and rigid. A non-perforated, solid subdrain collector pipe should be extended along the axis of the drainage course (parallel to the axial perforated subdrain line) and all tributary perforated subdrain lines should be connected to this collector pipe as shown on Plate 3B. Subdrain pipes Type SDR-35 should be installed within the keyway excavations while Type SDR-23.5, with thicker walls, should be installed along the axis of the swale where the fill depth increases as previously discussed. Subdrain cleanout risers should be installed at all upper/terminal ends of the perforated pipes where approximately delineated on Plate 3B.

A single 6-foot wide drainage bench, which would facilitate the surficial drainage along the planned slope face and allow for equipment access, is recommended across the approximately 50-foot high, south-facing fill slope portion proposed along the south side of the roadway. In addition, a single, 12-foot wide drainage bench at mid-height is recommended across the approximately 80-foot high, north-facing fill slope portion proposed along the north side of the roadway.

Concrete V-ditches should be constructed along the inboard side of all drainage benches.

This entire planned fill area will be underlain by the clay-rich Tulare formation sediments, which are considered expansive.

The proposed fill prism is planned across a topographic swale and measures to maintain the natural runoff flow northward within the drainage swale should be considered and implemented to prevent damming of the noted swale.

A dormant landslide labeled as Landslide 5 on Plate 3B is mapped downslope of a north-facing fill slope proposed in this area. The lower edge of the planned keyway in this area does not encroach on the southern limit of the noted landslide and the landslide has not shown signs of reactivation during the last ten years. Based on this information, we are not recommending that this landslide be mitigated and depending on the exposed conditions during grading, the edge of the keyway opposite the landslide may need to be deepened during construction.

The recommended keyway excavation for the south-facing slope will encroach onto a dormant landslide deposit labeled as Landslide 4 on Plate 3B. The keyway excavation should penetrate the landslide debris and be founded into in-place bedrock materials. Our engineering geologist should be provided the opportunity to assess the depth of the planned keyway excavation and the noted landslide during grading to evaluate the needed depth of excavation. The landslide materials mapped within the fill limits should be overexcavated and replaced with engineered fill during grading prior to the placement of the fill material. The removal of the landslide debris and excavation for the proposed keyway across the landslide should be staged and performed in two sections with the first half subdrained and backfilled before the second half is excavated to reduce the risk of having the mass mobilize during grading. Our 2008 report indicated the need for an encatchment berm on the south side of the roadway, especially along the western margin of the landslide where the landslide limit is near grade. However, after further evaluation it is our opinion that the south-facing fill slope to be constructed across the lower portion of the landslide will function as an encatchment berm and it should suffice to collect subsequent debris before it reaches the roadway. This conclusion is based on the minimal depth of the landslide along its western limit and the anticipated northeast direction of failure, which is controlled by the topography.

We anticipate the bedrock materials to be present at relatively deeper depths reaching up to about 10 feet below the ground surface in this area. We do not anticipate soft alluvial sediments to be present along the drainage course axis or where the keyways are planned. Subsequent settlement of the supporting soils/bedrock induced by fill placement is not anticipated to be significant.

7.5 PROPOSED FILL SLOPES (BETWEEN APPROXIMATE STATIONS 89+00 AND 97+20)

This is the easternmost fill area proposed for the roadway alignment and it will cross one of the most prominent northward-trending drainage courses within the project limits. Fill placed across this northward-flowing drainage course will also create south- and north-facing fill slope portions that will measure between approximately 80 and 90 feet in vertical height. The south-facing slope portion will also be constructed across a less prominent drainage swale that forms a tributary to the more prominent northward-flowing drainage course. The two fill slopes will be oval-shaped and their widths will increase with height. As is delineated on Plate 3B, we are recommending that the south-facing fill slope portion planned along the south side of the roadway be supported on 30- and 40-foot wide keyway excavations (across the tributary swale and the more prominent course, respectively) while the north-facing fill slope portion to the north of the roadway be supported on a 50-foot wide keyway excavation.

Perforated subdrain pipes should be placed along the heels of both keyway excavations and also extended along the axis of the drainage course and the tributary swale daylighting to the north of the north-facing slope portion. The outfall pipe beyond the toe of the north-facing fill slope portion should be non-perforated, solid, and rigid. A non-perforated, solid subdrain collector pipe should be extended along the axis of the drainage course (parallel to the axial perforated subdrain line) and all tributary perforated subdrain lines should be connected to this collector pipe as shown on Plate 3B. Subdrain pipes Type SDR-35 should be installed within the keyway excavations while Type SDR-23.5, with thicker walls, should be installed along the axis of the swale where the fill depth increases as previously discussed. Subdrain cleanout risers should be installed at all upper/terminal ends of the perforated pipes where approximately delineated on Plate 3B.

A single 12-foot wide drainage bench, which would facilitate the surficial drainage along the planned slope face and allow for equipment access, is recommended across the approximately 80- to 90-foot high, south- and north-facing fill slope portions proposed along the south and north sides of the roadway. Concrete V-ditches should be constructed along the inboard side of all drainage benches.

As is shown on Plate 3B, this entire planned fill area will be underlain by the clay-rich Tulare formation sediments, which are considered expansive.

The proposed fill prism is planned across a prominent drainage course and its associated tributary topographic swale and measures to maintain the natural runoff flow northward within the drainage course and swale should be considered and implemented to prevent damming of the noted drainage courses.

The planned fill prism will overlie several relatively smaller landslide deposits that are considered active. Most of these landslides are erosional features created by the seasonal flow along the prominent drainage course, which undercuts and undermines the toe of the west-facing bank of the noted swale. The axis of the swale and the mapped surficial slope failures should be overexcavated and any soft sediment present within their limits removed prior to fill placement. Our engineering geologist should be provided the opportunity to assess the depth of the planned keyway excavation to evaluate the removal of any soft sediment encountered.

We anticipate the bedrock materials to be present at relatively deeper depths within the swale axis in this area. Culvert intakes along the base of the south-facing slope planned to the south of the roadway should be raised to help avoid potential blockage due to material shedding by the mapped active landslide deposits in that immediate vicinity.

7.6 PROPOSED CUT SLOPES (BETWEEN APPROXIMATE STATIONS 10+50 AND 13+00)

This is the westernmost cut proposed and it is an approximately 30-foot high cut planned across a topographic knob present along the west side of the proposed James Donlon Bridge, which will span the Kirker Creek canyon. The cut slope will encounter highly and closely fractured pebbly sandstone belonging to the Cierbo formation, which dips adversely northward. As is delineated on Plate 3A, we are recommending that this north-facing cut planned along the south side of the roadway be overexcavated and rebuilt as a fill slope with a slope gradient not exceeding 2H:1V that is supported on a 15-foot wide keyway excavation. This selected keyway width is based on an aspect ratio of approximately half of the slope height, which is based on slope stability analysis, experience gained on previous mass grading projects and on the type of soil and fill material to be derived from the bedrock formations in this general area.

A perforated subdrain pipe should be placed along the heel of the keyway excavation and be either directed to drain toward the creek channel or along the shoulder of Kirker Pass road. The outfall pipe should be non-perforated, solid, and rigid. A subdrain pipe Type SDR-35 should be installed within the keyway. A subdrain cleanout riser should be installed at the opposite end of the outfall pipe.

The corrective grading for the planned cut slope will result in extending the actual grading limit by approximately 15 to 20 feet southward beyond the grading limits shown on the civil plans. The approximate limits of the corrective remedial grading are delineated on Plate 3A. No mid-slope drainage bench is recommended across the north-facing slope face because its height is not anticipated to exceed 30 vertical feet.

7.7 PROPOSED CUT SLOPES (BETWEEN APPROXIMATE STATIONS 18+00 AND 29+00)

This cut slope will have north- and south-facing portions that vary in vertical height between about 60 and 170 feet, respectively. The proposed north-facing, 60-foot high cut slope will mostly encounter highly and closely fractured pebbly sandstone belonging to the Cierbo formation, which dips adversely northward. As is delineated on Plate 3A, we are recommending that this north-facing cut planned along the south side of the roadway be overexcavated and rebuilt as a fill slope with a slope gradient not exceeding 2H:1V that is supported on a 35-foot wide keyway excavation. This selected keyway width is based on an aspect ratio of approximately half of the slope height, which is based on slope stability analysis, experience gained on previous mass grading projects, and on the type of soil and fill material to be derived from the bedrock formations in this general area. The planned roadway, associated off ramp, and the planned north-facing cut will encroach on a relatively shallow northeast/southwest-trending slope wash area. The civil and remedial grading plan scheme proposed in this area will most likely remove the majority of the noted deposit. We do not anticipate that the noted deposit will impact the planned grading or improvements.

A perforated subdrain pipe should be placed along the heel of the keyway excavation and directed to drain southwestward toward the creek tributary as is shown on Plate 3A. If this area is utilized for a borrow site, the subdrain may be directed eastward to connect with the subdrain system planned along the abutting fill area situated immediately to the east. The outfall pipe should be non-perforated, solid, and rigid if directed westward. A subdrain pipe Type SDR-35 should be installed within the keyway. A subdrain cleanout riser should be installed at the opposite end of the outfall pipe.

The corrective grading for the planned cut slope will result in extending the actual grading limit by approximately 35 to 40 feet southward beyond the grading limits shown on the civil plans. The approximate limits of the corrective remedial grading are delineated on Plate 3A. A single 6-foot wide drainage bench, which would facilitate the surficial drainage along the planned slope face and allow for equipment access, is recommended at the mid-height of the slope. A concrete V-ditch should be constructed along the inboard side of all drainage benches.

The south-facing, 170-foot high cut slope planned along the north side of the roadway will mostly encounter favorably-bedded gray sandstone belonging to the Neroly formation except for the southwestern corner of the cut where Cierbo sandstone may be encountered. Based on the noted favorable bedrock bedding, the nature of the Neroly formation sandstone, our field observations, and our slope stability analysis performed as part of this supplemental report, it is our opinion that this south-facing cut planned along the north side of the roadway may be steepened to an approximate slope gradient not exceeding 1.75H:1V. No remedial grading is needed except for the recommended drainage benches discussed below. If a 1.75H:1V slope gradient is implemented, the grading limit on the civil plan shown on Plate 3A would shift southward and the magnitude of the cut would be decreased.

The planned cut will remove the head section of the mapped slope wash deposit shown on Plate 3A. The northward-dipping contact separating the overlying Neroly formation sandstone from the underlying Cierbo sandstone mapped along the southwest corner of the south-facing cut slope planned along the north side of the roadway may shift northward after the cut is made. Depending on the exposed field conditions during grading, our engineering geologist may recommend overexcavating and rebuilding that slope portion.

A single 12-foot wide drainage bench should be constructed at the mid-slope height coupled with two additional 6-foot wide drainage benches: one at the mid-height between the top of the slope and the 12-foot wide bench and the other at the mid-height between the toe of the south-facing slope and the 12-foot wide drainage bench.

7.8 PROPOSED CUT SLOPE (BETWEEN APPROXIMATE STATIONS 34+00 AND 38+00)

This approximately 40-foot high, south-facing slope proposed along the north side of the roadway will most likely expose Neroly sandstone that is favorably bedded. Although we are not recommending that this cut slope be overexcavated and rebuilt, the planned cut may result in shifting the contact with the underlying Cierbo formation northward exposing more Cierbo on the cut. Depending on the nature of the exposed conditions and the final location of the noted geologic contact, our geologist may recommend overexcavating and rebuilding this south-facing slope. If this cut needs to be rebuilt, a 15- to 20-foot wide keyway with a subdrain will be needed along the toe of the slope, and the subdrain will most likely be combined with the subdrain for the proposed adjacent fill slope situated immediately to the west. Whether this cut is rebuilt or not, a single 6-foot wide drainage bench should be constructed at the mid-height of this 40-foot high south-facing slope to facilitate surficial drainage and equipment access.

7.9 PROPOSED CUT SLOPES (BETWEEN APPROXIMATE STATIONS 46+50 AND 61+00)

This cut slope will have north- and south-facing portions that vary in vertical height between about 90 and 160 feet, respectively. The proposed north-facing, 90-foot high cut slope will mostly encounter Neroly sandstone and also highly and closely fractured pebbly sandstone belonging to the Cierbo sandstone, both of which dip adversely northward. The position of the mapped geologic contact separating the two formations and shown on Plates 3A and 3B will most likely shift northward after the cut is made. As is delineated on Plates 3A and 3B, we are recommending that this north-facing cut planned along the south side of the roadway be overexcavated and rebuilt as a fill slope with a slope gradient not exceeding 2H:1V that is supported on a 45-foot wide keyway excavation.

A perforated subdrain pipe should be placed along the heel of the keyway excavation and directed to drain southwestward toward the adjacent drainage course as shown on Plate 3A. The outfall pipe should be non-perforated, solid, and rigid if directed westward. Subdrain pipe Type SDR-35 should be installed within the keyway. Additional tributary perforated subdrain lines should be installed at about 25 feet in vertical height. A subdrain cleanout riser should be installed at all upper ends of the perforated subdrain lines as is approximately shown on Plate 3B.

The corrective grading of the planned cut slope will result in extending the actual grading limit by approximately 45 to 50 feet southward beyond the grading limits shown on the civil plans. The approximate limits of the corrective remedial grading are delineated on Plates 3A and 3B. A single 12-foot wide drainage bench, which would facilitate the surficial drainage along the planned slope face and allow for equipment access, is recommended at the mid-height of the slope. A concrete V-ditch should be constructed along the inboard side of all drainage benches.

The south-facing, approximately 160-foot high cut slope planned along the north side of the roadway will mostly encounter favorably-bedded gray sandstone belonging to the Neroly formation except for the southwestern corner of the cut where Cierbo sandstone may be encountered. Based on the noted favorable bedrock bedding, the nature of the Neroly formation sandstone, our field observations, and our slope stability analysis performed as part of this update, it is our opinion that this south-facing cut planned along the north side of the roadway may be steepened to an approximate slope gradient not exceeding 1.5H:1V. No remedial grading is anticipated except for the recommended drainage benches discussed below. If a 1.5H:1V slope gradient is implemented, the grading limit on the civil plan shown on Plates 3A and 3B would shift southward and the magnitude of the cut would be decreased.

The planned cut slope should remove a mapped slope wash deposit in this area shown on Plates 3A and 3B. The northward-dipping contact separating the overlying Neroly formation sandstone from the underlying Cierbo sandstone mapped along the southwest corner of the south-facing cut slope planned along the north side of the roadway may shift northward after the cut is made. Depending on the exposed field conditions during grading, our engineering geologist may recommend overexcavating and rebuilding that slope portion.

A single 12-foot wide drainage bench should be constructed at the mid-slope height coupled with two additional 6-foot wide drainage benches: one at the mid-height between the top of the slope and the 12-foot wide bench and the other at the mid-height between the toe of the south-facing slope and the 12-foot wide drainage bench.

The north-facing cut slope portion along the south side of the roadway in the vicinity of Station 60+00 is shown to be about 30 feet in vertical height. This slope portion should also be overexcavated and rebuilt. A 15-foot wide keyway should be constructed. Because this is an elevated area, we are not recommending the installation of a subdrain line in this keyway. However, a 6-foot wide drainage bench should be constructed at mid-slope height.

7.10 PROPOSED CUT SLOPES (BETWEEN APPROXIMATE STATIONS 70+00 AND 78+00)

The proposed easternmost cut slopes, which consist of an approximately 200-foot high, south-facing slope and a 40-foot high, north-facing slope, are considered the most difficult slopes to be graded across the entire roadway alignment. This conclusion is based on the following geologic conditions:

- The north-facing slope portion is the highest cut planned along the alignment;
- Both north- and south-facing slope portions are underlain by and will expose landslide-prone Tulare formation;
- The north-facing cut slope portion will expose adversely bedded Tulare sandy claystone;
- The location of the north-facing cut is situated along the adversely-dipping geologic contact separating the overlying Tulare formation from the underlying Lawlor Tuff;
- The northward-dipping Lawlor Tuff unit underlying the proposed north-facing high cut is well consolidated, considered stable, and will likely transmit subsurface water flows northward beneath the Tulare clayey sediments, which could cause them to fail;
- A relatively large and subdued landslide deposit, the outline of which appears to have been rounded and masked by erosion and slope mass wasting, may be present in this area;
- Remedial grading for the north-facing slope portion may need to be as wide as 100+ feet, which would result in shifting the limit of the grading southward by approximately 100 additional feet; and
- The area lacks specific subsurface information which we consider of utmost importance to evaluate the feasibility and stability of the proposed north-facing cut.

Based on the above information, we recommend that further investigation of this area be performed to assess its subsurface conditions, geologic structure, thickness of geologic formations, and its short- and long-term stability.

8 PRELIMINARY EVALUATION OF PROPOSED RETAINING WALLS

According to the preliminary retaining wall layout and profile plan titled Exhibit B-4 shown in Appendix D, prepared by RBF, six retaining walls are planned near the western limits of the proposed James Donlon Boulevard Extension project. These walls are anticipated to consist of cantilevered, soldier pile and lagging, sheet pile, MSE, and cantilevered walls supported on piles. Our preliminary evaluation of these walls is discussed below.

8.1 MECHANICALLY STABILIZED WALLS

As shown in Appendix D, two 190-foot long mechanically stabilized earth (MSE) walls (numbered 123 and 127) are proposed along the western approach to the Ramp Bridge. These walls are expected to be up to 25 feet high. Based on the available exploration points and the geology shown on Plate 3A, the subsurface conditions at the wall foundations are expected to consist of Pleistocene age alluvial fan deposits (map symbol Qpaf on Plate 3A). These deposits are generally moist, firm to hard in consistency, and are comprised of yellowish brown sandy fat to lean clays with varying concentrations of sandy and gravelly zones.

Both retaining walls are anticipated to be founded on shallow foundations. Retaining wall 127 will be located adjacent to west end of the Ramp Bridge. We expect the Ramp Bridge abutment and its wing walls will be supported on deep foundations. Therefore, differential settlement between the bridge structure (abutment and wing walls) and retaining wall 127 should be expected during construction. Structures supported on properly constructed pile foundations typically experience negligible settlement in comparison to structures that are supported on shallow foundations. Relatively deep fills are planned for the construction of the bridge embankments. These fills can potentially experience significant settlements over time. This could result in significant differential settlement between pile-supported and shallow foundation-supported structures that abut each other, and between pile-supported structures adjacent to deep fill embankments. Section 10 of this report (Settlement of Deep Fills) discusses mitigation measures for addressing this differential settlement potential.

A preliminary allowable bearing capacity of 3,000 pounds per square foot (psf) may be used for the conceptual design of spread footings for these retaining walls. A one-third increase for transient loads, such as seismic loads, may be used. This preliminary bearing capacity is based on the assumption that the footings will be supported on engineered fill and/or firm/undisturbed subgrade and will have a minimum embedment depth of 3 feet. This bearing capacity will need to be reduced where the wall footings are close to a slope. More detailed analysis is needed to allow us to provide a final design bearing capacity.

For the design of the retaining walls, we recommend using a minimum soil reinforcement length of 70 percent of the wall height with a length of not less than 8 feet. The wall footings should have a minimum embedment depth of 3 feet or 0.1H (where H is the wall height), whichever is greater. The base width for the MSE wall should be evaluated at several height sections along the length of the wall using applicable software like MSEW©. A minimum horizontal berm of 4 feet, or 0.1H wide, whichever is greater, should be provided in front of walls founded on slopes.

Based on the subsurface information available, the proposed MSE walls appear to be geotechnically feasible. During the final design phase of the project, site-specific geotechnical investigation and laboratory testing should be performed at proposed MSE wall locations to confirm the anticipated foundation type discussed above, to assess potential geotechnical issues such as estimated settlements, and to provide final recommendations. In addition, global stability and sliding analysis should be performed to assess the stability of the retaining structures. This report provides recommendations for a supplemental investigation for the proposed walls.

8.2 CAST-IN-PLACE WALLS

Three Cast-In-Place (CIP) retaining walls are proposed for this project as shown in Appendix D. These retaining structures consist of Caltrans Standard Type 1 walls (cantilevered retaining walls less than 36 feet high). The three walls can be described as follows:

- Retaining wall 17 is about 590 feet in length and will be located on the west side of Kirker Pass Road, opposite the planned exit ramp to James Donlon Boulevard. This wall is expected to be up to 15 feet high;
- Retaining wall 22 is about 375 feet in length and will be located on the east side of Kirker Pass Road, south of the intersection with James Donlon Boulevard and opposite to MSE wall 123. This wall is expected to be up to 15 feet high; and
- Retaining wall 34 is about 185 feet in length and will be located on the east side of Kirker Pass Road, north of the intersection with James Donlon Boulevard. This wall is expected to be up to 10 feet high.

The proposed walls appear to be geotechnically feasible at the proposed locations. Based on the available exploration points and the geology shown on Plate 3A, the subsurface conditions at the wall foundations are expected to consist of Pleistocene age alluvial fan deposits (map symbol Qpaf on Plate 3A), except for the northern segment of wall 22, which is expected to overlie the Tertiary Kirker Formation (Volcanic Tuff, map symbol Tkt). The Kirker formation (map symbol Tkt) is mainly comprised of marine volcanic tuff and tuffaceous sandstone, and mudstone. The tuff is vitric (composed of crystals) and lithic (composed of minute rock fragments) and is fissile. The bedrock is expected to be weathered and generally rippable, although concretions may be encountered, which could require more extensive mechanical equipment to break up, such as jackhammer-equipped backhoes and excavators. For additional discussion on the rippability of the site soils and bedrock, refer to Section 8.4.4 of our January 9, 2008 report.

The preliminary plans show that retaining walls 22 and 34 would be founded on shallow foundations. Both walls are situated close to sloping ground and measures will likely be needed to increase the global stability of the walls. These measures could include, but are not limited to, deepening the footings and increasing the footing distance from the slope face to shift the footing pressure bulb away from the slope face, thus lowering the potential for surcharging the slope. Where the above recommendations cannot be met due to topography and design constraints, and global stability minimum requirements are not met, deep foundation systems such as cast-in-drilled-hole (CIDH) piers should be considered. During the final design phase, a comprehensive stability analysis should be performed to evaluate both the static and seismic factors of safety against sliding, overturning, eccentricity, and global failure surfaces for these walls. The preliminary retaining wall layout and profile shown in Appendix D indicate retaining wall 17 being supported by deep foundations where wall the height is greater than 12 feet. When the wall design is finalized and loading information is available, a comprehensive analysis should be performed to estimate pile lengths and sizes to accommodate the proposed axial and lateral loads.

For preliminary analysis purposes, piles may be estimated to have an embedment depth of about 1.5 to 2 times the exposed wall height. This embedment is subject to change during the design phase based on several factors such as wall axial and lateral loads, number of piles, pile sizes, subsurface conditions, and footing distance to slope face. At the time of the preparation of this report and due to the lack of subsurface information at the proposed wall locations, the depth to bedrock is not known. Some of the piles may extend into bedrock during installation. The bedrock is expected to be weathered and generally drillable, although concretions and hard zones may be encountered, which could require more powerful drilling equipment. Depending on the depth of bedrock, driven piles may not be a feasible option for deep foundations and cast-in-drilled-hole piers may need to be used instead. As part of the additional investigation being recommended in this report, the proposed wall locations should be explored in order to evaluate the depth and drillability of bedrock, and develop geotechnical design parameters for the walls, including deep foundations.

All three proposed CIP retaining walls cross over tributaries to Kirker Creek. Design consideration should be given in accommodating the existing/new culvert boxes proposed within the walls or passing underneath the walls. Where culverts pass underneath the walls, the design team should consider various options so that new loading from the proposed retaining wall does not impact the box culverts. For preliminary purposes, a 1H:1V imaginary plane projected from the bottom of the footings may be used to evaluate the influence zone for surcharge loads imposed by the wall footings on the culverts. The final design may include construction of a loading platform consisting of reinforced ground improvement (RGI) to transfer the load from the wall to deeper soil outside the influence zone of the culvert.

8.3 SOLDIER PILE/SHEET PILE WALLS

Retaining wall 16 is 250 feet in length and will be located on the east side of Kirker Pass Road, near the entrance to exit ramp to James Donlon Boulevard. According to the preliminary retaining wall layout and profile plan shown in Appendix D, the wall is anticipated to consist of a soldier pile or sheet pile wall. This wall is expected to be up to 10 feet high.

For preliminary analysis purposes, soldier piles usually consist of 2.5- to 3.5-foot diameter augered holes with steel, wide-flange beam reinforcing, backfilled with lean concrete and/or structural grout. Typical pile spacing is 6 to 8 feet on-center with typical embedment depths of 1.5 to 2 times the exposed wall height for cantilever walls. The embedment depth should be based on the minimum depth to achieve force and moment equilibrium.

The subsurface soil at the proposed wall location is expected to consist of alluvium underlain by bedrock. Due to the lack of subsurface data, the depth to bedrock is not known at this location. Some of the piles may extend into bedrock during installation. The bedrock is expected to be weathered and generally drillable, although concretions and hard zones may be encountered, which could require more powerful drilling equipment. Depending on the depth of bedrock, sheet piles may not be a feasible option for deep foundations and pre-drilled soldier piles may need to be used instead. As part of the additional investigation being recommended in this report, the proposed wall location should be explored in order to evaluate the depth and drillability of bedrock, and develop geotechnical design parameters for the wall, including deep foundations.

9 SLOPE GRADIENT OF FILL EMBANKMENT BETWEEN STATIONS 60+00 AND 70+00

As requested by RBF, we evaluated the use of a reinforced slope system to increase the slope gradient from 2H:1V (horizontal to vertical) to 1H:1V for the proposed roadway fill embankment located between Stations 60+00 and 70+00. The embankment height ranges from about 120 to 160 feet at this location. As part of our evaluation, we discussed the proposed embankment with engineers from TenCate Geosynthetics Americas (TenCate), a leading supplier of geosynthetics and industrial fabrics. With our input, TenCate performed a preliminary analysis of the embankment using a mechanically stabilized earth geosynthetic system. Based on their preliminary results, a 1H:1V slope gradient is feasible provided the embankment is reinforced with properly designed and constructed geogrid reinforcement. For TenCate's preliminary analysis, they used a geogrid reinforced embankment system composed of Miragrid 24XT and Miragrid 8XT for the lower 130 feet and upper 30 feet of the embankment, respectively. The geogrid layers were spaced every 3 feet vertically. The geogrid layers used ranged between approximately 40 and 200 feet in length.

The design of geosynthetic systems and their properties are supplier dependent. For this reason, if RBF and the City wish to further explore this option, we recommend that they secure the services of a leading geosynthetics supplier, such as TenCate, to develop a design for the mechanically stabilized earth geosynthetic system for this embankment. To provide increased surficial slope stability and lower the potential for erosion to occur along the embankment slope face, we recommend that a face wrap such as Miramesh GR or equivalent be placed between successive geogrid layers.

As previously noted, we recommend that drainage benches be placed across the embankment slope faces for drainage purposes and equipment access. A 12-foot wide bench is recommended at the mid-height of the embankment slopes and two 6-foot wide benches: one at the mid-height between the top of the slope and the 12-foot wide bench and the second at the mid-height between the toe of the slope and the 12-foot wide bench. A 3-foot wide V-ditch should be installed on each bench. Each bench should be sloped at a minimum gradient of 2 percent towards the back to allow water to flow towards the V-ditches. We anticipate these benches could be exposed to maintenance equipment traffic in the future. For this reason, surcharge loads for such equipment should be incorporated into the design of the mechanically stabilized earth geosynthetic system for the embankment.

As an alternative to a geosynthetic system, mechanically stabilized earth (MSE) retaining walls could be used to construct the fill embankment. However, to reduce loading, the walls would need to be designed in tiers which could affect its cost effectiveness. Crib walls may not be considered a viable alternative for steepening the embankment's slopes because these walls are typically not cost-effective to build beyond wall heights of about 20 to 30 feet.

10 SETTLEMENT OF DEEP FILLS

Section 8.3.3 of our January 9, 2008 report includes a discussion about the amount of anticipated/estimated settlement that could take place in deep fill areas of the site, such as the fill embankment planned between Stations 60+00 and 70+00 and the fill abutments for the proposed bridges, and provides recommendations for mitigating such settlement. To reduce the potential for settlement distress on guard rails, catch basins, storm drains, pavements, and other structures located within these new fills, construction of these improvements should be delayed as much as possible (at least 60 to 90 days) after the embankments and abutments have been constructed. If this waiting period is not feasible from a construction schedule standpoint, we should be consulted to provide alternative solutions.

Settlements of deep fills can be reduced by compacting the portion of the fill below a depth of 10 feet from finished grade to a minimum of 95 percent compaction at least 2 percent over optimum moisture content for clayey soils and at near optimum moisture content for granular soils. Compaction should be based on the maximum dry unit weight per ASTM D1557. In areas of abrupt changes in fill thickness, consideration should be given to also using a reinforcing geotextile fabric such as Mirafi RS380i or equivalent at the base of the aggregate base layer for pavements and concrete flatwork. Consideration should also be given to the implementation of a surcharge program.

To reduce the potential for abrupt changes in surface elevations that could develop overtime where bridge decks meet bridge abutments, consideration should be given to constructing structural approach slabs at bridge abutments. These slabs help provide a smoother transition between the bridge deck and the bridge abutments. The structural approach slabs could consist of typical Caltrans structural approach types N (30S), R (30S), N (30D), R (30D), and/or N (45D).

11 UPDATED SEISMIC DESIGN CRITERIA

The seismic design criteria provided in our January 9, 2008 report was based on the 2007 California Building Code (CBC), which has since been superseded by the 2010 CBC.

Portions of the roadway alignment lie within relatively deep alluvium of 30+ feet in thickness, such as the west end where the Kirker Creek bridges are proposed, while other portions lie or will lie (after cuts) on weathered bedrock. Also, some areas of the site will receive a significant amount of fill, such as the large fill embankment proposed at Station 67+00. Based on this, we recommend using Site Class C (defined as very dense soil and soft rock per Table 1613.5.3 of the 2010 CBC) for the areas of the site where shallow bedrock is present and no significant amount of fill (i.e., greater than 10 feet thick) is planned. Areas of the site covered by more than 10 feet of alluvium or fill should be classified as Site Class D (defined as stiff soil profile per Table 1613.5.3 of the 2010 CBC). Where the designer is not sure if shallow bedrock is present or not, the Site Class that results in more stringent seismic parameters should be used.

According to the 2010 California Building Code (CBC), the mapped spectral response accelerations S_S and S_1 for the site are 1.50g and 0.58g, respectively, which were obtained based on the Java ground motion parameter calculator developed by the U.S. Geological Survey (USGS, 2011) using ASCE 7. The site coefficients Fa and Fv, which are based on Tables 1613.5.3(1) and 1613.5.3(2) of the 2010 CBC, are 1.0 and 1.3, respectively, for Site Class C and 1.0 and 1.5, respectively, for Site Class D.

Note that S_S and S_1 are based on Site Class type B (defined as a rock soil profile per Table 1613.5.3 of the 2010 CBC) and therefore need to be multiplied by the site coefficients Fa and Fv to obtain the maximum considered earthquake spectral response accelerations SM_S and SM_1, respectively. The design spectral response accelerations SD_S and SD_1 are computed by multiplying SM_S and SM_1, respectively, by 2/3. Table 11-1 below presents the seismic parameters for each Site Class.

Table 11-1 – 2010 CBC Seismic Parameters

Seismic Parameter	Site Class C	Site Class D
Fa	1.0	1.0
Fv	1.3	1.5
SM_S	1.50	1.50
SM_1	0.75	0.87
SD_S	1.00	1.00
SD_1	0.50	0.58

12 RECOMMENDATIONS FOR ADDITIONAL INVESTIGATIONS

As previously discussed, limited subsurface information is available for the western limits of the James Donlon Boulevard Alignment where the proposed retaining walls will be located. Although bedrock is anticipated in the area, its depth is unknown where the walls will be located. The presence of bedrock could affect the type, depth, and constructability of the foundations for the walls. Therefore, we recommend performing a supplemental field exploration to investigate the proposed wall locations. The supplemental investigation would allow us to provide final geotechnical design recommendations for the walls and evaluate the depth of bedrock. At this time, we recommend performing at least one to two exploration points per wall.

The proposed easternmost cut slopes, which consist of an approximately 200-foot high, south-facing slope and a 40-foot high, north-facing slope portions between Stations 70+00 and 78+00, are considered the most difficult slopes to be graded across the entire roadway alignment. This conclusion is based on the following geologic conditions:

- The north-facing slope portion is the highest cut planned along the alignment;
- Both north- and south-facing slope portions are underlain by and will expose landslide-prone Tulare formation;
- The north-facing cut slope portion will expose adversely bedded Tulare sandy claystone;
- The location of the north-facing cut is situated along the adversely-dipping geologic contact separating the overlying Tulare formation from the underlying Lawlor Tuff;
- The northward-dipping Lawlor Tuff unit underlying the proposed north-facing high cut is well consolidated, considered stable, and will transmit subsurface water flows northward beneath the Tulare clayey sediments, which could cause them to fail;
- A relatively large and subdued landslide deposit, the outline of which appears to have been rounded and masked by erosion and slope mass wasting may be present in this area;

- Remedial grading for the north-facing slope portion may need to be as wide as 100+ feet, which would result in shifting the limit of the grading southward by approximately 100 additional feet; and
- The area lacks specific subsurface information which we consider of utmost importance to evaluate the feasibility and stability of the proposed north-facing cut.

Based on the above information, we recommend that further investigation of this area be performed to assess its subsurface conditions, geologic structure, thickness of geologic formations, and its short and long term stability.

13 ADDITIONAL SERVICES AND LIMITATIONS

13.1 ADDITIONAL SERVICES

The review of plans and specifications and field observation and testing by Kleinfelder of earthwork related construction activities are an integral part of the conclusions and recommendations made in this report. It is recommended that Kleinfelder be present at the pre-bid meeting with the prospective grading contractors to clarify any issues and to address any questions regarding the recommendations presented in this report. If Kleinfelder is not retained for these services, the Client will be assuming Kleinfelder's responsibility for any potential claims that may arise during or after construction due to the misinterpretation of the recommendations presented herein and/or our upcoming update report. The recommended tests, observations, and consultation by Kleinfelder prior to and during construction include, but are not limited to:

- Review of plans and specifications;
- Observations of earthwork operations spanning site clearing and stripping through final grading and utility trench backfill;
- Observation of foundation excavations and foundation construction;
- Construction observation and in-place density testing of fills, backfills; and finished subgrades; and
- Regular site visits by our CEG during grading and construction to observe keyway excavations, subdrain layout, cut slopes, groundwater seepage, adverse bedding and weakness zones exposed during grading, and other geological aspects affecting the project, such as landslides.

13.2 LIMITATIONS

The recommendations contained in this report are based on our supplemental field observations and subsurface explorations, limited laboratory tests, our present knowledge of the proposed construction, and review of previous investigations. It is possible that soil/bedrock conditions could vary between or beyond the points explored. If soil/bedrock conditions are encountered during construction that differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed.

We prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by Kleinfelder during the construction phase in order to evaluate compliance with our recommendations. Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the author of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report, or if conditions at the site have changed. If this report is used beyond this period, Kleinfelder should be contacted to evaluate whether site conditions have changed since the report was issued.

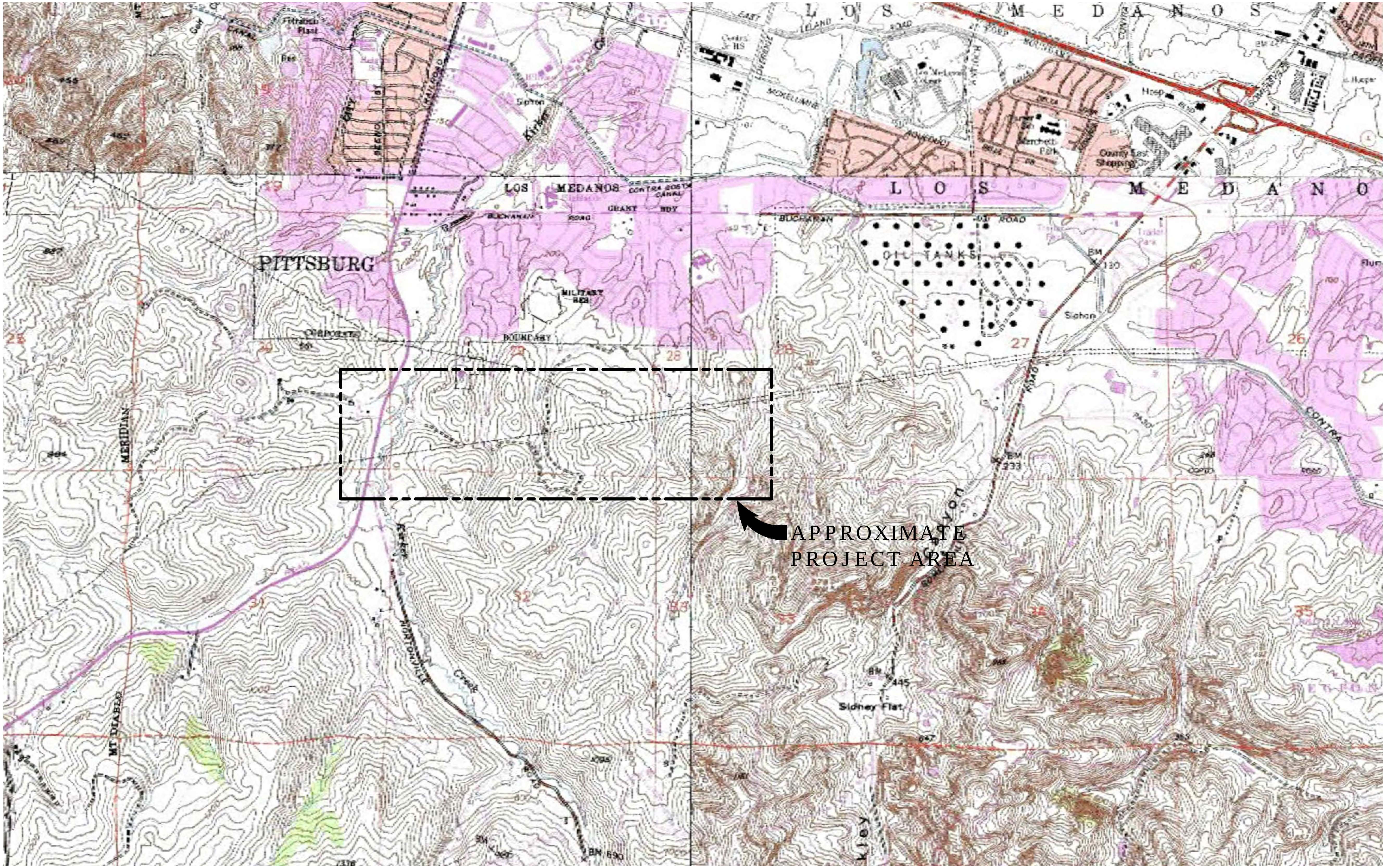
Also, land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, Kleinfelder may recommend that additional work be performed and that an updated report be issued.

The scope of work for this supplemental 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 conducted subsurface exploration and provided recommendations for this project. We understand that Kleinfelder will be given the opportunity to perform a formal geotechnical review of the final project plans and specifications. In the event Kleinfelder is not retained to review the final project plans and specifications to evaluate if our recommendations have been properly interpreted, we will assume no responsibility for misinterpretation of our recommendations.

We recommend that all earthwork during construction be monitored by a representative from Kleinfelder, including site preparation, footing excavation, placement of engineered fill, and trench backfill. The purpose of these services would be to provide Kleinfelder the opportunity to observe the actual soil conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes in design or construction procedures if conditions differ from those described herein.

PLATES



1800 900 0 1800

APPROXIMATE SCALE (feet)

N

REFERENCE:
USGS 7.5 Minute Series (Topographic)
Antioch South Quadrangle, dated 1953, photorevised 1980;
Clayton Quadrangle, dated 1953, photorevised 1980, revised 1994

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PROJECT NO.	123460
DRAWN:	SEPT 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	VIC-PLAN.dwg

SITE VICINITY MAP

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
1

PLOTTED: 31 Aug 2012, 2:45pm, JSala

ATTACHED IMAGES: Images: 20120120172112_00001.jpg Images: AREA GEOLOGY LEGEND.jpg Images: SITE-VIC.jpg Images: SITEPLAN.jpg
ATTACHED XREFS: XRef: Eng-B_11x17_L_StyleA
PLEASANTON, CA CAD FILE: C:\Users\JSala\appdata\local\temp\AcPublish_6844\ LAYOUT: SITE-VIC

LEGEND

APPROXIMATE PROJECT AREA

MIDDLE ALIGNMENT (C2-LOW)

NOTE: Locations are approximate.

500 250 0 500

APPROXIMATE SCALE (feet)

N

REFERENCE:
Google Earth Pro., Imagery date 10-29-2011

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PROJECT NO.	123460
DRAWN:	SEPT 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	VIC-PLAN.dwg

AERIAL SITE PLAN

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
2

PLOTTED: 31 Aug 2012, 2:45pm, JSala
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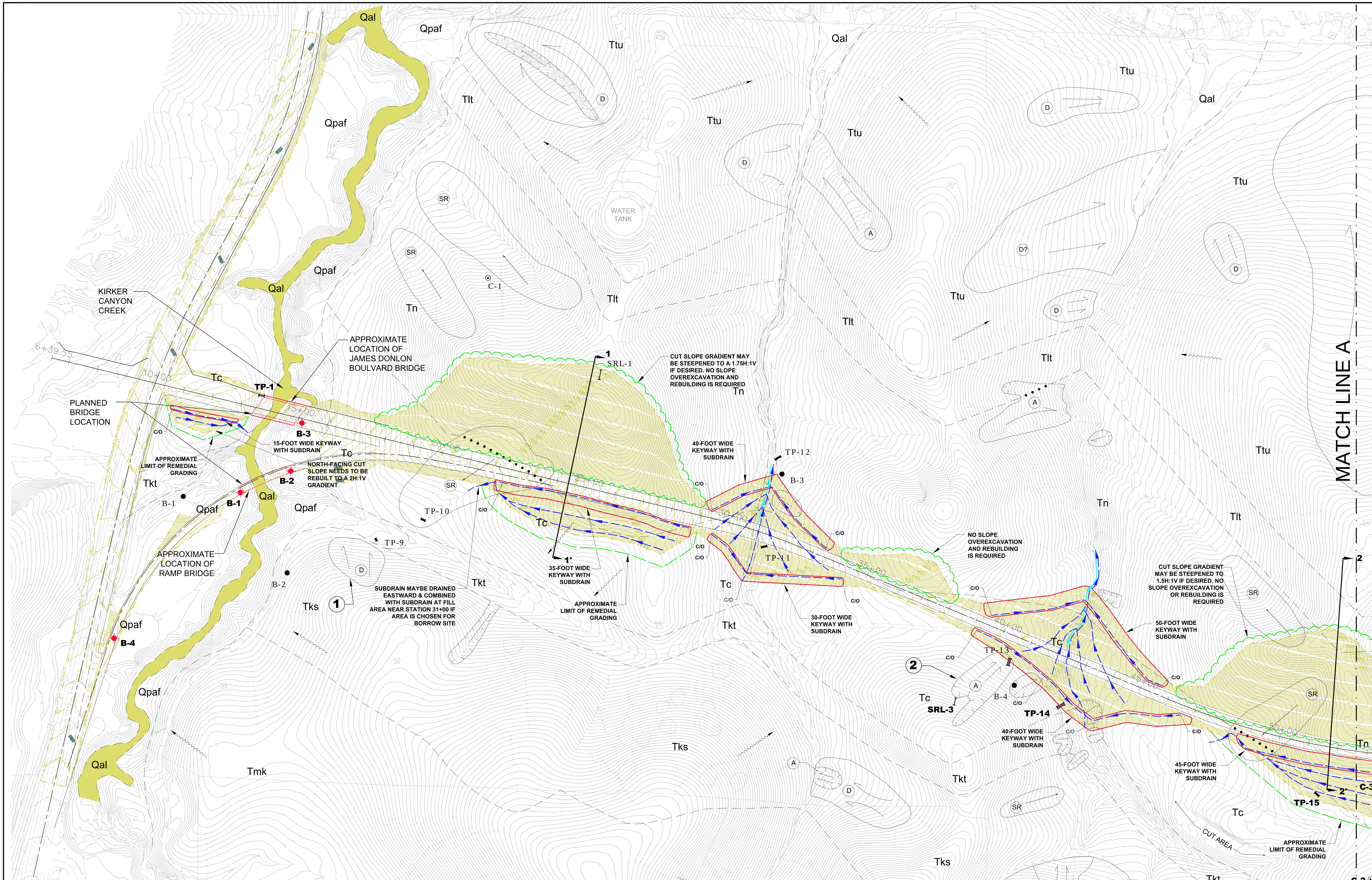


EXPLANATION

APPROXIMATE GEOLOGIC CONTACT – dotted where buried	Qal	UNDIFFERENTIATED RECENT ALLUVIUM	
CREEPING COLLUVIUM	Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT	
INCISED EROSION RILLING OR SHALLOW SLUMP	Ttu	TERTIARY TULARE FORMATION (Undifferentiated)	
SLOPE WASH (arrows indicate direction of movement)	Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)	
ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)	Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)	
DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)	Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)	
	Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)	

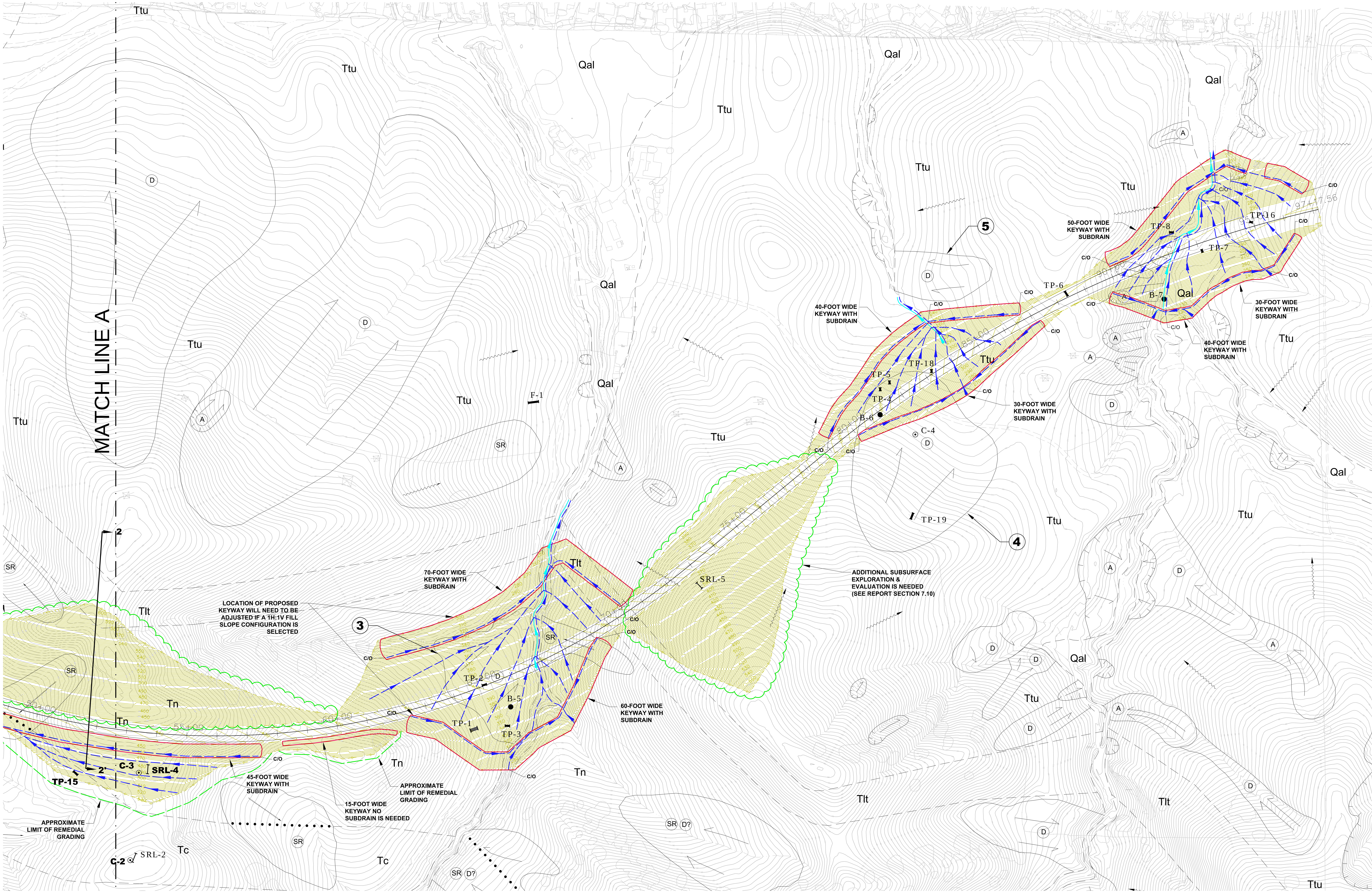
Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)
B-7	SOIL BORING (By Kleinfelder, 2008)
C-4	ROCK CORE (By Kleinfelder, 2008)
TP-19	TEST PIT OR EXPLORATORY FAULT TRENCH (By Kleinfelder, 2008)
SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)
F-1	FAULT TRENCH (By Kleinfelder, 2008)
B-4	SOIL BORING (By Kleinfelder, 2011)
TP-1	TEST PIT (By Kleinfelder, 2011)
	PERFORATED SUBDRAIN PIPE WITH FLOW ARROWS
	KEYWAY
C/O	CLEAN OUT
	12-INCH DIAMETER, NON-PERFORATED SOLID COLLECTOR SUBDRAIN PIPE WITH FLOW ARROWS
1 1'	LOCATION OF SLOPE STABILITY CROSS SECTION (SEE APPENDIX C)
2	LANDSLIDE IDENTIFICATION NUMBER

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11;
File Emailed By RBF Consulting, "0129-GRBS-C2-Low Profile-Full drawing" dated 2/9/12.

N

100 50 0 100
APPROXIMATE SCALE (feet)

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123460	REMEDIAL GRADING PLAN	PLATE
	DRAWN: NOV 2012		3A
	DRAWN BY: JDS		
	CHECKED BY: SD	MIDDLE ALIGNMENT (C2-LOW) JAMES DONLON BOULEVARD EXTENSION PITTSBURG, CALIFORNIA	
	FILE NAME: BASE_LOW profile-11-2012.dwg		



EXPLANATION

	APPROXIMATE GEOLOGIC CONTACT – dotted where buried
	CREEPING COLLUVIUM
	INCISED EROSION RILLING OR SHALLOW SLUMP
SR	SLOPE WASH (arrows indicate direction of movement)
A	ACTIVE LANDSLIDE DEPOSIT (arrows indicate direction of movement)
D	DORMANT LANDSLIDE DEPOSIT (arrows indicate direction of movement)

Qal	UNDIFFERENTIATED RECENT ALLUVIUM
Qpaf	PLEISTOCENE ALLUVIAL FAN DEPOSIT
Ttu	TERTIARY TULARE FORMATION (Undifferentiated)
Tlt	TERTIARY LAWLOR TUFF (TUFFACEOUS SANDSTONE AND VOLCANIC TUFF)
Tn	TERTIATY NEROLY FORMATION (BLUE SANDSTONE)
Tc	TERTIARY CIERBO FORMATION (FOSSILIFEROUS SANDSTONE)
Tkt	TERTIARY KIRKER FORMATION (VOLCANIC TUFF)

Tks	TERTIARY KIRKER FORMATION (TUFFACEOUS SANDSTONE)
Tmk	TERTIARY MARKLEY FORMATION (SANDSTONE AND SHALE)
B-7	SOIL BORING (By Kleinfelder, 2008)
C-4	ROCK CORE (By Kleinfelder, 2008)
TP-19	TEST PIT OR EXPLORATORY FAULT TRENCH (By Kleinfelder, 2008)
SRL-5	SEISMIC REFRACTION LINES (By Kleinfelder, 2008)
F-1	FAULT TRENCH (By Kleinfelder, 2008)

B-4	SOIL BORING (By Kleinfelder, 2011)
TP-1	TEST PIT (By Kleinfelder, 2011)
	PERFORATED SUBDRAIN PIPE WITH FLOW ARROWS
	KEYWAY
C/O	CLEAN OUT
	12–INCH DIAMETER, NON–PERFORATED SOLID COLLECTOR SUBDRAIN PIPE WITH FLOW ARROWS
1 1'	LOCATION OF SLOPE STABILITY CROSS SECTION (SEE APPENDIX C)
3	LANDSLIDE IDENTIFICATION NUMBER

REFERENCE:
Emailed By RBF Consulting, "Aerial Topographic Map," dated 12/10/11 and 12/13/11;
File Emailed By RBF Consulting, "0129–GRBS–C2–Low Profile–Full drawing" dated 2/9/12.

N

100 50 0 100
APPROXIMATE SCALE (feet)

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 123460	REMEDIAL GRADING PLAN	PLATE
	DRAWN: NOV 2012		3B
	DRAWN BY: JDS		
	CHECKED BY: SD	MIDDLE ALIGNMENT (C2-LOW) JAMES DONLON BOULEVARD EXTENSION PITTSBURG, CALIFORNIA	
	FILE NAME: BASE_LOW profile-11-2012.dwg		

ATTACHED IMAGES: Images: SWALE.jpg Images: Topo with road.jpg
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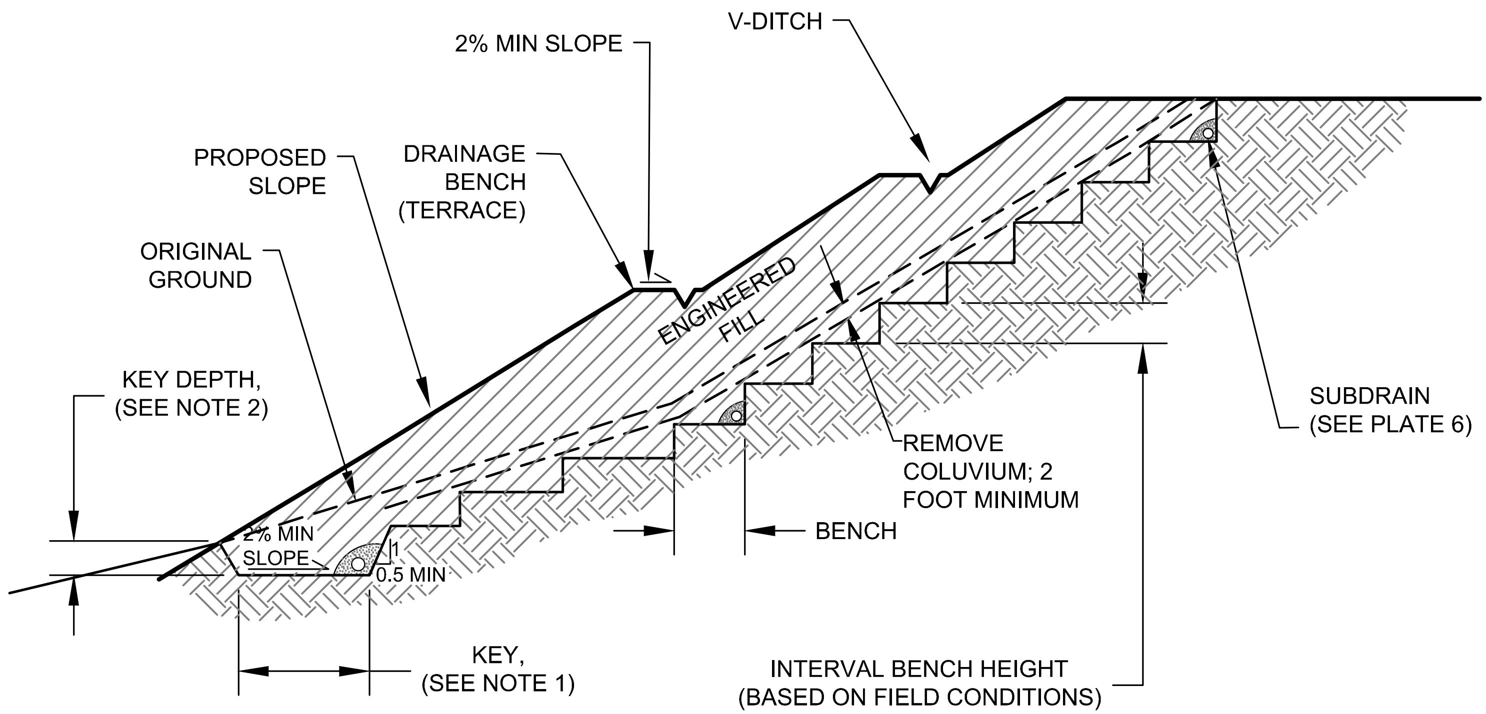
PLEASANTON, CA CAD FILE: L:\2012\12Projects\123460-james Donlon-final\GRAPHICs\Keyway\ LAYOUT: FILL SLOPE PLOTTED: 30 Nov 2012, 9:52am, JSala



NOTES:

1. KEYWAY WIDTH: 1/2 VERTICAL HEIGHT OF FILL.

2. KEYWAY DEPTH: MINIMUM 10 FEET INTO COMPETENT SOIL OR BEDROCK. FINAL DEPTH TO BE ADJUSTED BASED ON FIELD CONDITIONS.

3. SEE ATTACHED SUBDRAIN DETAIL. LOCATION OF SUBDRAIN WILL VARY DEPENDING ON FIELD CONDITIONS.

4. DRAINAGE BENCHES: WIDTH AND LOCATIONS BASED ON LOCAL CODES. STANDARD SPACING IS APPROXIMATELY 30 VERTICAL FEET; FOR SLOPES WITH ONLY ONE BENCH, THE BENCH SHALL BE AT MID-HEIGHT. SLOPES OVER 30' HIGH MUST HAVE 6'-WIDE DRAINAGE TERRACES.

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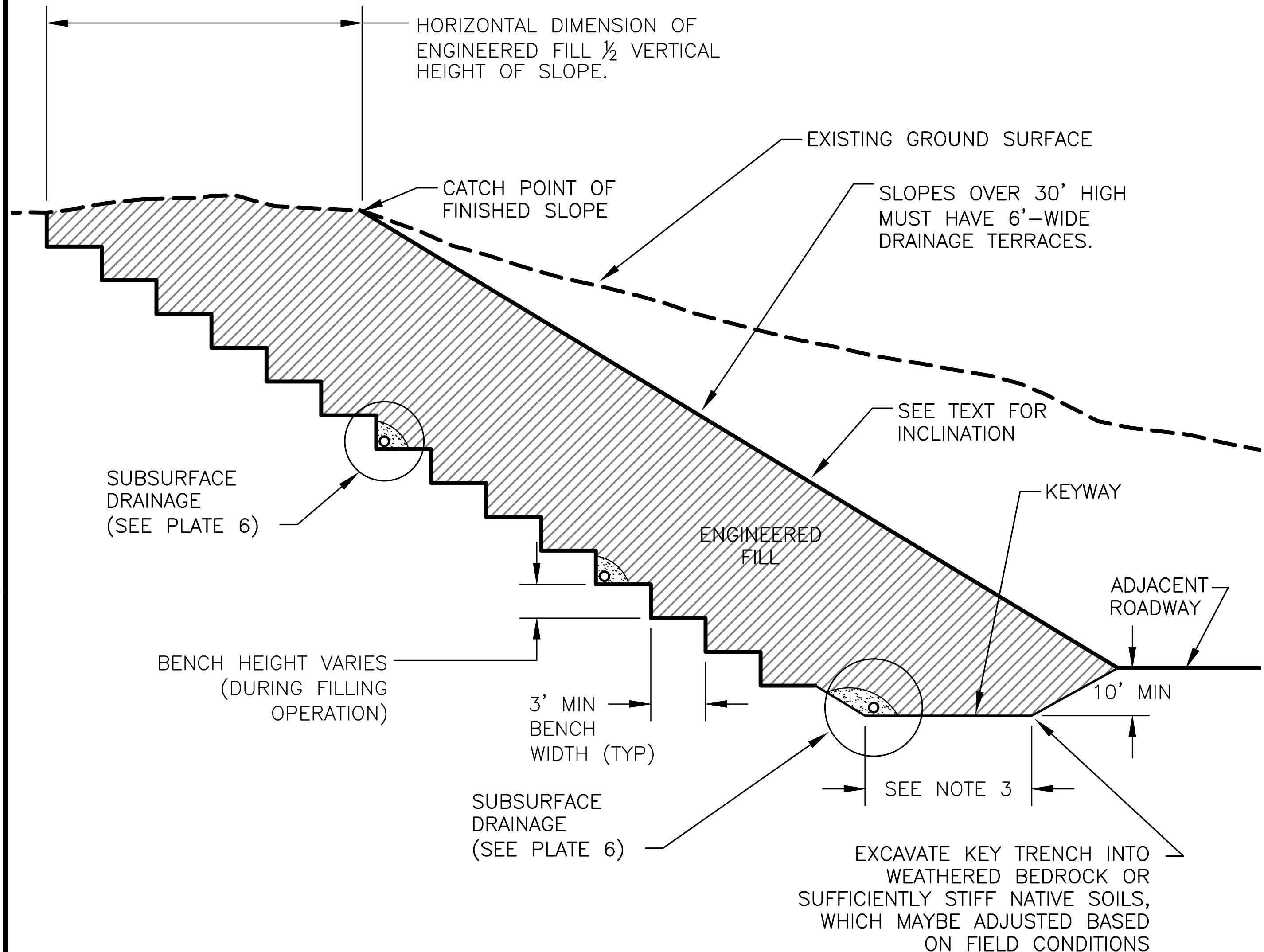
PROJECT NO.	123460
DRAWN:	NOV 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	SBDRNS.dwg

TYPICAL FILL SLOPE DETAIL

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE 4



NOTES:
1. FINAL BENCHING AND KEYWAY EXCAVATION DEPTHS AND DETAILS SUBJECT TO FIELD CONDITIONS.

2. DIMENSIONS AND LOCATIONS OF SUBDRAINAGE ARE APPROXIMATE. ACTUAL INSTALLATION TO BE BASED ON FIELD CONDITIONS.

3. KEYWAY WIDTH = ½ OF VERTICAL SLOPE HEIGHT.

NOT TO SCALE

PLOTTED: 26 Nov 2012, 5:06pm, JSala
LAYOUT: Layout1
CAD FILE: C:\Users\JSala\Documents_Archive_PROJECTS\123460 James\
XRef: Style A_08x11
ATTACHED IMAGES:
ATTACHED XREFS:
PLEASANTON, CA

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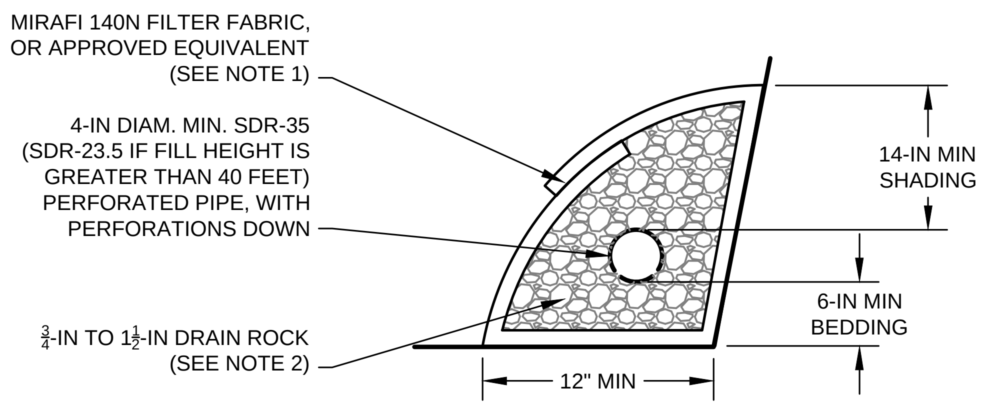
PROJECT NO.	123460
DRAWN:	NOV 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	KEYWAY.dwg

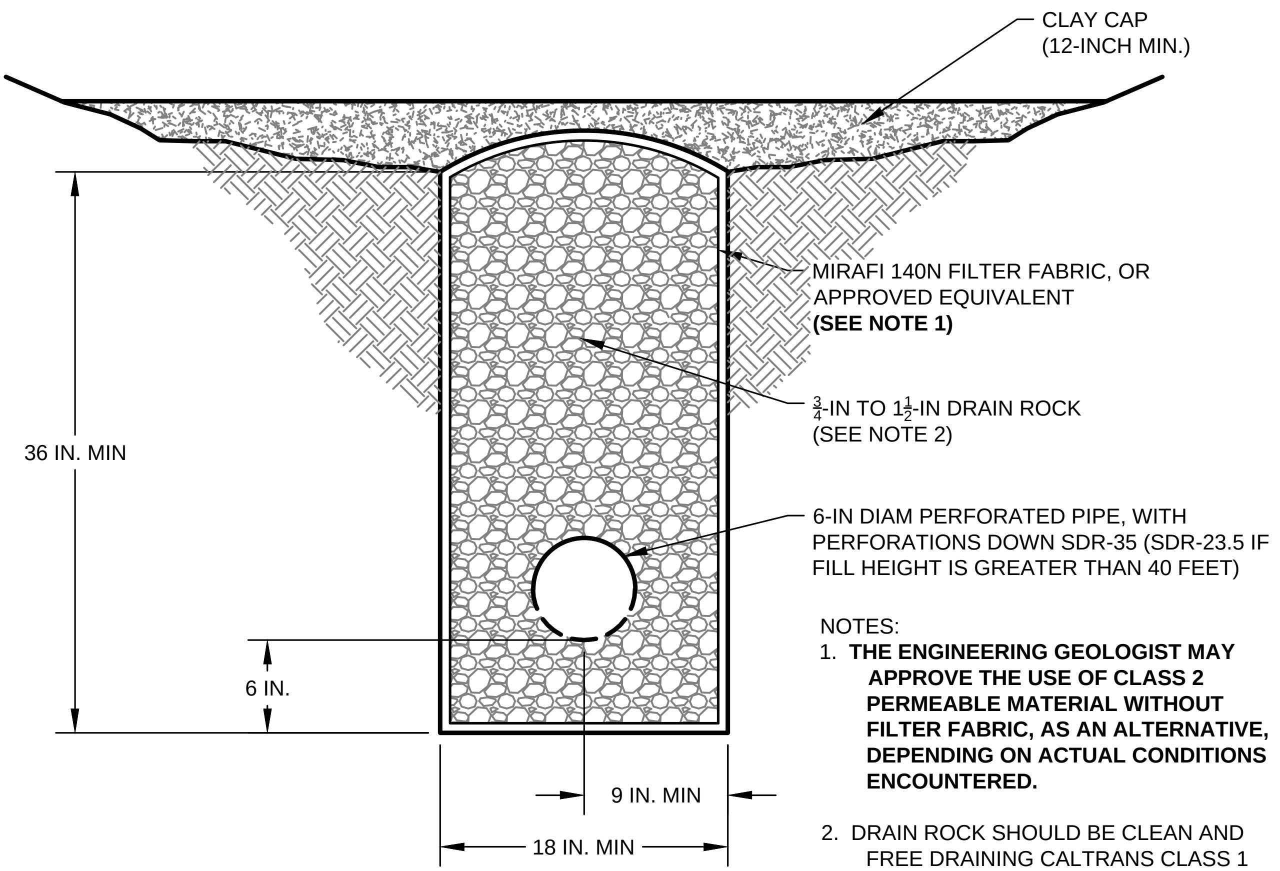
TYPICAL KEYWAY AND BUTTRESS FILL FOR CUT SLOPE RECONSTRUCTION

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
5



TYPICAL KEYWAY AND BENCH SUBDRAIN



NOTES:

1. **THE ENGINEERING GEOLOGIST MAY APPROVE THE USE OF CLASS 2 PERMEABLE MATERIAL WITHOUT FILTER FABRIC, AS AN ALTERNATIVE, DEPENDING ON ACTUAL CONDITIONS ENCOUNTERED.**
2. DRAIN ROCK SHOULD BE CLEAN AND FREE DRAINING CALTRANS CLASS 1 TYPE B.
3. DRAWING IS NOT TO SCALE.

TYPICAL CANYON SUBDRAIN

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PROJECT NO.	123460
DRAWN:	NOV 2012
DRAWN BY:	JDS
CHECKED BY:	SD
FILE NAME:	SBDRNS.dwg

TYPICAL SUBDRAIN DETAILS

JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
6

APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0745, 5/31 — Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 — Approximate water level observed in boring following drilling

PEN	Pocket Penetrometer reading, in tsf
TV:Su	Torvane shear strength, in ksf

LL	Liquid Limit	UC	Unconfined Compression
PI	Plasticity Index	TxUU	Triaxial Shear
%-#200	Sieve Analysis (#200 Screen)	CONSOL	Consolidation
DS	Direct Shear	R-Value	Resistance Value
C	Cohesion (psf)	SE	Sand Equivalent
PHI	Friction Angle	EI	Expansion Index
		FS	Free Swell (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

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PROJECT NO. *123460*

BORING LOG LEGEND

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE

A-1

8/31/2012 3:38:38 PM

Date Completed: **11/29/11**

Logged By: **O. Khan**

Total Depth: **Approximately 51.5 ft**

Drilling method: **8" Hollow Stem Auger**
Driller: Exploration Geoservices

Hammer Wt: **Automatic Hammer - 140 lbs., 30" drop**

Notes: **Grass covered ground surface**

Surface Elevation: **Estimated 279.0 feet (MSL)**

Depth, ft	Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
0								**Sandy Fat CLAY (CH)**, yellowish brown, moist, very hard, low to medium plasticity, mottled with calcium carbonate, fine to medium grained sand (Pleistocene Alluvial Fan Deposit)
	■	23				LL=51; PI=37 (see Plate B-2)	>4.5	
5	■	28	106	17.6		Corrosion (see Plate B-13) TXUU: C= 7,300 psf (see Plate B-3)	>4.5	Trace fine gravel, increase in sand content
10	■	10					3.5	Carbonate nodules Grades to clayey sand
15	■	11	99	19.0	1.97 @ 10.0%	TXUU: C= 2,000 psf (see Plate B-4)	1.5 2.3	**Lean CLAY (CL)**, light brown, moist, medium plasticity, firm (Pleistocene Alluvial Fan Deposit) **Sandy Lean CLAY (CL)**, yellowish brown, moist, firm, fine to medium grained sand, low plasticity (Pleistocene Alluvial Fan Deposit)
20	■	11						**Silty SAND/Clayey SAND (SM-SC)**, light yellowish brown, moist, loose to medium dense, fine to medium grained sand, charcoal fragments (Pleistocene Alluvial Fan Deposit)
25	■	17		19.5			2.0	Yellow, medium dense, grades to poorly graded sand **Sandy Lean CLAY (CL)**, olive gray, moist, hard, low plasticity, fine grained sand (Pleistocene Alluvial Fan Deposit)
30								

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PROJECT NO. 123460

LOG OF BORING NO. B-1

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
A-2

Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		15						**Silty Sand (SM)**, yellowish brown, moist, medium dense, fine to medium grained sand, trace fine grained gravel (Pleistocene Alluvial Fan Deposit)
								Sandy Lean CLAY (CL), dark grayish brown, moist, low to medium plasticity, fine to medium grained sand (Pleistocene Alluvial Fan Deposit)
35		18	101	23.1		TXUU: C= 2,100 psf (see Plate B-5)	2.5	Mottled greenish gray and bluish gray, firm, low plasticity, gypsum, minor amounts or carbonate
40		23					2.3	
45		15	100	26.3			1.8	Increase in sand content, greenish gray, wet
			13:30 11/29/2011					
50		16	92	31.3			1.5	Iron oxide staining
			12:30 11/29/2011					Boring terminated at a depth of approx. 51.5 feet. Boring backfilled with cement grout.
55								
60								

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PROJECT NO. 123460

LOG OF BORING NO. B-1

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE **A-2** **(cont'd)**

Date Completed:	11/30/11	Drilling method:	8" Hollow Stem Auger Driller: Exploration Geoservices
Logged By:	O. Khan	Hammer Wt:	Automatic Hammer - 140 lbs., 30" drop
Total Depth:	Approximately 51.5 ft	Notes:	Grass covered ground surface

	FIELD		LABORATORY					DESCRIPTION
Depth,ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	Surface Elevation: **Estimated 275.0 feet (MSL)**
								Sandy Lean CLAY (CL), dark yellowish brown, moist, hard, fine grained sand, medium plasticity, trace carbonate, gypsum (Pleistocene Alluvial Fan Deposit)
		15					>4.5	
5								Increase in sand content, mottled with gypsum
		21	104	18.6		TXUU: C= 4,200 psf (see Plate B-6)	4.3	
								Tuff fragments, gypsum fragments, iron oxide staining
10								Lenses of clayey sand
		12					>4.5	Low plasticity
15			92	23.5				Light yellowish brown, firm, fine to medium grained sand, low to medium plasticity
		9					2.0-3.0	
								Silty SAND (SM), yellow, moist, loose, predominantly fine grained sand (Pleistocene Alluvial Fan Deposit)
20		13						
								Medium to coarse grained sand with gravel
25								**Lean Clay with SAND (CL)**, mottled yellowish brown and olive, moist, soft to firm, low to medium plasticity, fine grained sand, mottled with gypsum (Pleistocene Alluvial Fan Deposit)
		4						
30								

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LOG OF BORING NO. B-2

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE **A-3**

Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION (Continued from previous plate)
								Higher sand content
		8	91	29.9		TXUU: C= 800 psf (see Plate B-7)	0.5	
35		10					0.8	Mottled light brownish gray and light yellowish brown, lower sand content, mottled with gypsum
40		12						Higher sand content, mottled greenish gray to olive gray, mottled with gypsum Bluish gray clayey sand lenses
45		14	96	28.5			1.3	No gypsum
								Clayey sand lenses, charcoal fragments, gypsum fragments
50		12						Top of a sample was wet. Possibly indicative of perched water Firm
								Boring terminated at a depth of approx. 51.5 feet. No groundwater encountered at time of drilling. Boring backfilled with cement grout.
55								
60								

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PROJECT NO. 123460

LOG OF BORING NO. B-2

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
A-3
(cont'd)

Date Completed:	11/30/11	*Drilling method:*	8" Hollow Stem Auger Driller: Exploration Geoservices
Logged By:	O. Khan	*Hammer Wt:*	Automatic Hammer - 140 lbs., 30" drop
Total Depth:	Approximately 51.5 ft	*Notes:*	Grass covered ground surface

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION — *Surface Elevation:* **Estimated 275.0 feet (MSL)**
0								**Sandy Lean CLAY (CL)**, yellowish brown, moist, hard, fine grained sand, medium plasticity, mottled with gypsum (Pleistocene Alluvial Fan Deposit)
		11				Corrosion (see Plate B-13)	4.0	
5		20	102	21.8		TXUU: C= 3,300 psf (see Plate B-8)	3.8	Trace angular gravel Dark yellowish brown, decrease in sand content, hard, Tuff fragments
10		15				LL=43; PI=29 (see Plate B-2)	2.5-3.5	Increase in sand content, gypsum filaments within
15		13	98	21.5		TXUU: C= 2,000 psf (see Plate B-9)	2.5-3.0	Yellowish brown
20		7						Sand lens
25		15					2.0	Dark yellowish brown mottled with olive, iron oxide staining
30								

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PROJECT NO. 123460

LOG OF BORING NO. B-3

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE **A-4**

Depth, ft	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		15	94	27.5			2.3	Trace fine subrounded gravel
35		17					2.5	Mottled bluish gray to olive gray Medium to coarse grained sand, transition to clayey sand to poorly graded sand with clay
40		15					1.3-1.5	Mottled bluish gray and yellowish brown, iron oxide staining
45		13	103	22.9		TXUU: C= 1,300 psf (see Plate B-10)	1.0-1.3	Trace coarse grained sand
50		10						Bluish gray, charcoal fragments within
								Boring terminated at a depth of approx. 51.5 feet. No groundwater encountered at time of drilling. Boring backfilled with cement grout.
55								
60								

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PROJECT NO. **123460**

LOG OF BORING NO. B-3

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE **A-4** **(cont'd)**

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Date Completed:	11/29/11	*Drilling method:*	8" Hollow Stem Auger
			Driller: Exploration Geoservices
Logged By:	O. Khan		
		Hammer Wt:	Automatic Hammer - 140 lbs., 30" drop
Total Depth:	Approximately 26.5 ft	*Notes:*	Grass covered ground surface

Surface Elevation: **Estimated 289.0 feet (MSL)**

Depth, ft	Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
								Sandy Lean CLAY (CL), yellowish brown, moist, very hard, fine to medium grained sand, low to medium plasticity, caliche, fine grained gravel (Pleistocene Alluvial Fan Deposit)
		16					>4.5	
5		17					>4.5	
10		10	102	19.4		TXUU: C= 1,900 psf (see Plate B-11)	3.0	Increase in sand content, grades to clayey sand, occasional charcoal fragments
15		2						Mottled olive yellow and light brownish gray, very soft, medium plasticity
		10	15:00 11/29/2011					**Poorly Graded Sand with CLAY (SP-SC)**, yellowish brown, moist, medium dense, fine to medium grained sand (Pleistocene Alluvial Fan Deposit) Wet sampler shoe. Possibly indicative of perched water
20		9	90	31.4		Consolidation (see Plate B-12)	0.8	**Lean Clay with SAND (CL)**, dark yellowish brown to dark grayish brown, moist, firm medium plasticity, iron oxide (Pleistocene Alluvial Fan Deposit)
			15:30 11/29/2011					
25		10						Mottled with gypsum, firm
								Boring terminated at a depth of approx. 26.5 feet. Boring backfilled with cement grout.
30								

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LOG OF BORING NO. B-4

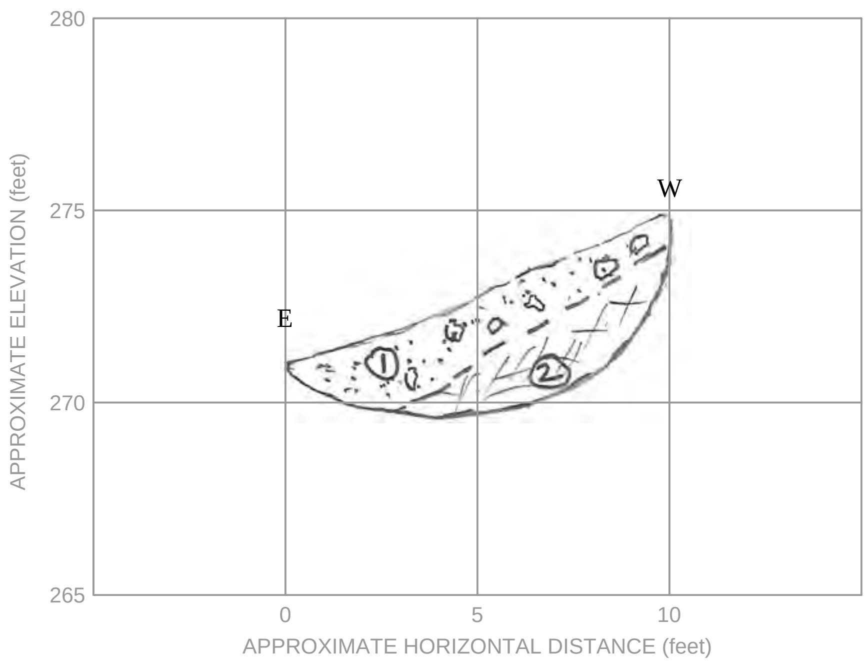
MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PROJECT NO. 123460

PLATE **A-5**

TP-1

EW TREND SOUTH FACE



UNIT NO.	MATERIALS DESCRIPTION
1	Dark Brown Sandy Lean Clay to Clayey Sand (CL/SC), dry to moist, fine to medium grained sand, sandstone fragments floating within soil matrix, low plasticity, pocket penetrometer = 1.3-4.5 tsf (Residual Soil)
2	Sandstone, white to pale olive, moderatelly to highly weathered, weak to moderately strong, highly fractured, friable, fossiliferous, fractures filled with carbonate deposits, fine to medium grained sand (Cierbo Formation)
	Out Crop: N30W, 35NE Bedding

LOGGED BY: Omar Khan DATE: 12/1/11

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PROJECT NO.	123460
DRAWN:	SEPT 2012
DRAWN BY:	JDS
CHECKED BY:	CM
FILE NAME:	TESTPIT log.dwg

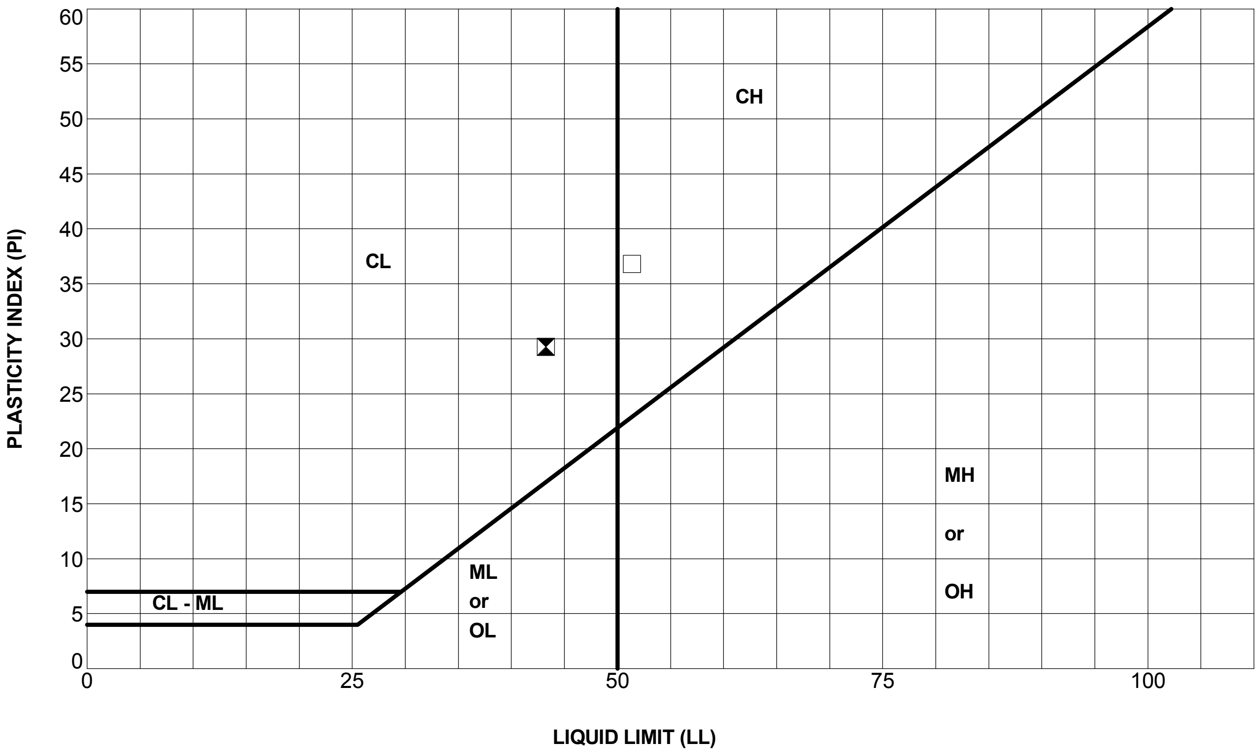
LOG OF TEST PIT
TP-1

MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

PLATE
A-6

APPENDIX B

BORING NO.	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE	WATER CONTENT (%)	DRY DENSITY (pcf)	MAXIMUM DEVIATOR STRESS (ksf)	AXIAL STRAIN (%)	CONFINING STRESS (ksf)
B-1	3.5	51	15	36						
B-1	6.0					17.6	105.7	14.6	5.0	0.5
B-1	16.0					19.0	98.8	3.9	7.5	1.0
B-1	25.5					19.5				
B-1	36.0					23.1	100.9	4.2	15.0	2.0
B-1	45.5					26.3	100.1			
B-1	50.5					31.3	92.4			
B-2	6.0					18.6	104.1	8.4	6.6	0.5
B-2	15.5					23.5	92.0			
B-2	31.0					29.9	90.7	1.5	12.2	2.0
B-2	45.5					28.5	95.5			
B-3	6.0					21.8	102.1	6.6	3.6	0.5
B-3	10.5	43	14	29						
B-3	16.0					21.5	97.6	4.0	4.6	1.0
B-3	30.5					27.5	94.0			
B-3	46.0					22.9	102.8	2.7	14.3	2.5
B-4	11.0					19.4	101.6	3.7	7.3	0.7
B-4	20.5					31.4	89.6			



SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
☐	B-1	3.5	51	15	36	Yellowish Brown Sandy Fat Clay (CH)
⧗	B-3	10.5	43	14	29	Yellowish Brown Sandy Lean Clay (CL)

Unified Soil Classification

Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (DRY PREP)
Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does not meet), if provided.

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PROJECT NO. *123460*

ATTERBERG LIMITS*

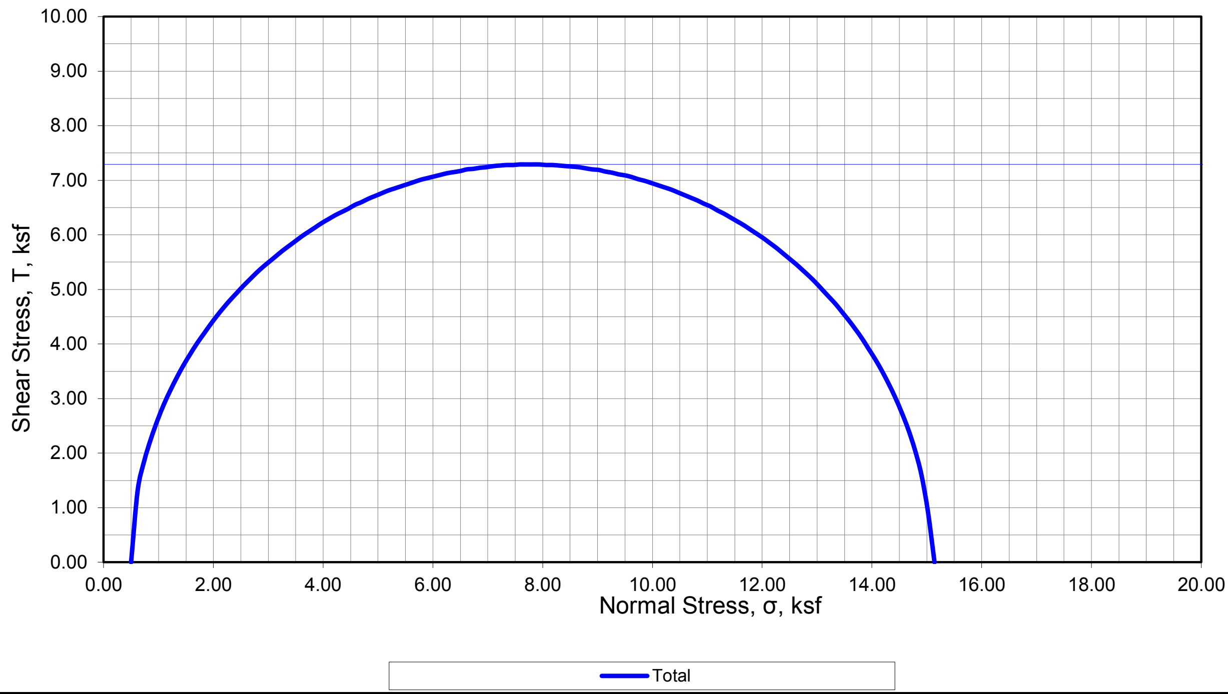
MIDDLE ALIGNMENT (C2-LOW)
JAMES DONLON BOULEVARD EXTENSION
PITTSBURG, CALIFORNIA

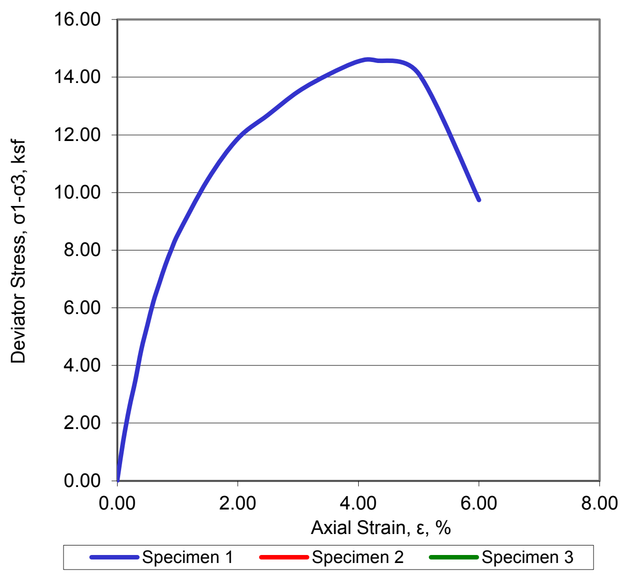
PLATE

B-2

Triaxial Compression Test Report

Total		
c =	7.3	ksf





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.80
	Water Content, %	ω_O	17.6
	Dry Density, lbs/ft^3	γ_{d_o}	105.7
	Saturation, %	S_O	82.6
	Void Ratio	e_O	0.565
Minor Principal Stress, ksf		σ_3	0.5
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	14.6
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	5.0
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	9.7
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	5.00

Description of Specimen: Yellowish Brown Fat Clay (CH)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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Hayward, CA 94545
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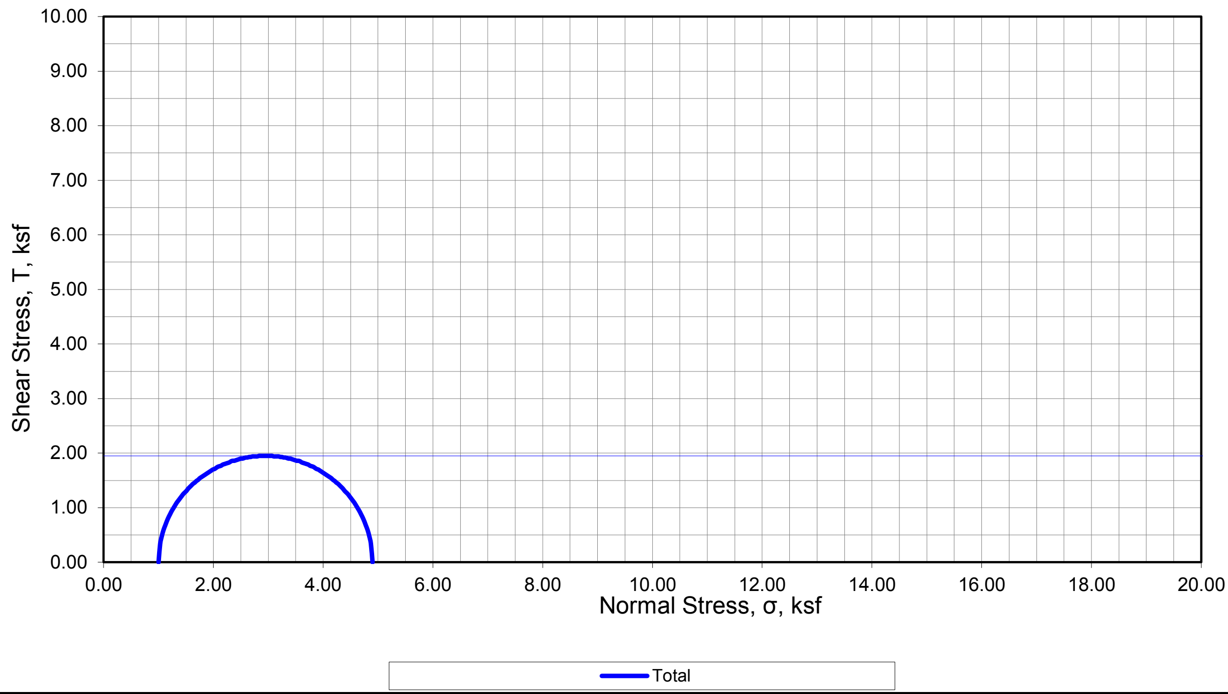
Project No.:	123460
Boring:	B-1
Sample:	2C
Depth, ft.:	6.0
Date:	January 3, 2012

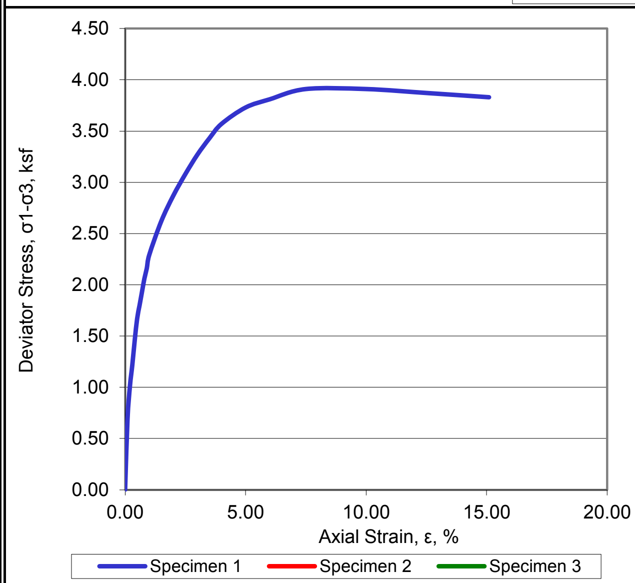
Plate

B-3

Triaxial Compression Test Report

	Total	
c =	2.0	ksf





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.67
	Water Content, %	ω_O	19.0
	Dry Density, lbs/ft^3	γ_{d_o}	98.8
	Saturation, %	S_O	74.7
	Void Ratio	e_O	0.674
Minor Principal Stress, ksf		σ_3	1.0
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	3.9
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	7.5
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.9
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	7.50

Description of Specimen: Light Brown Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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Sacramento, CA 95827
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kleinfelder.com

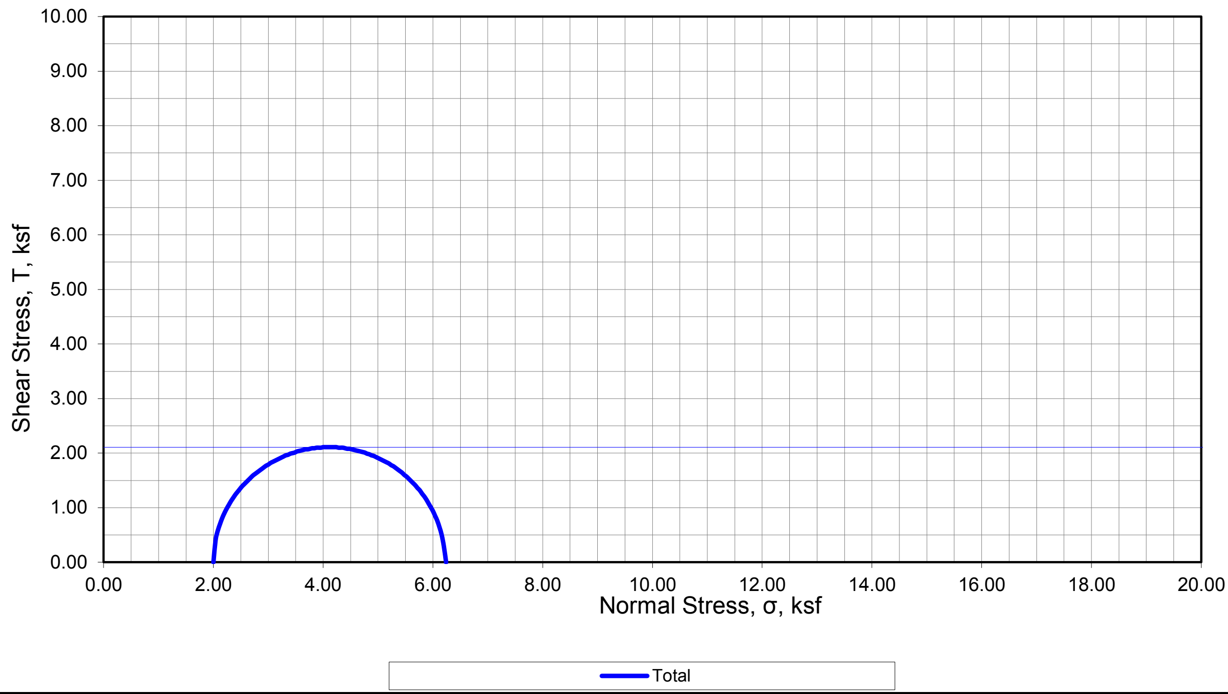
Project No.:	123460
Boring:	B-1
Sample:	4C
Depth, ft.:	16.0
Date:	January 3, 2012

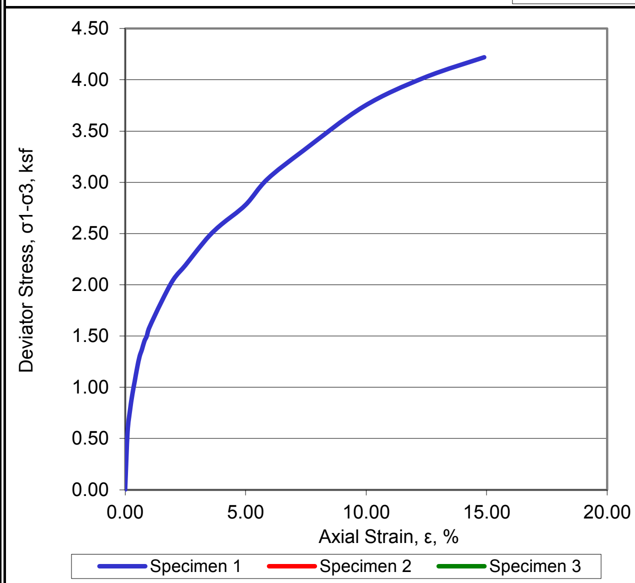
Plate

B-4

Triaxial Compression Test Report

Total		
c =	2.1	ksf





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.75
	Water Content, %	ω_O	23.1
	Dry Density, lbs/ft^3	γ_{d_o}	100.9
	Saturation, %	S_O	95.8
	Void Ratio	e_O	0.639
Minor Principal Stress, ksf		σ_3	2.0
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	4.2
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	15.0
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	4.2
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.0
Axial Strain at Failure, %		ε_f	15.00

Description of Specimen: Mottled Greenish Gray and Bluish Gray Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

KLEINFELDER
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3077 Fite Circle
Sacramento, CA 95827
p| 916.366.1701
f| 916.366.7013
kleinfelder.com

Project No.:	123460
Boring:	B-1
Sample:	8C
Depth, ft.:	36.0
Date:	January 3, 2012

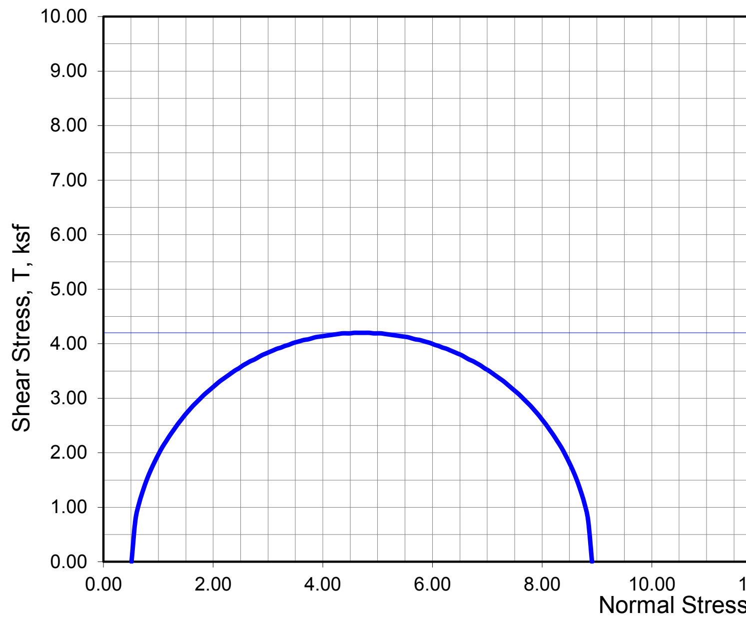
Plate

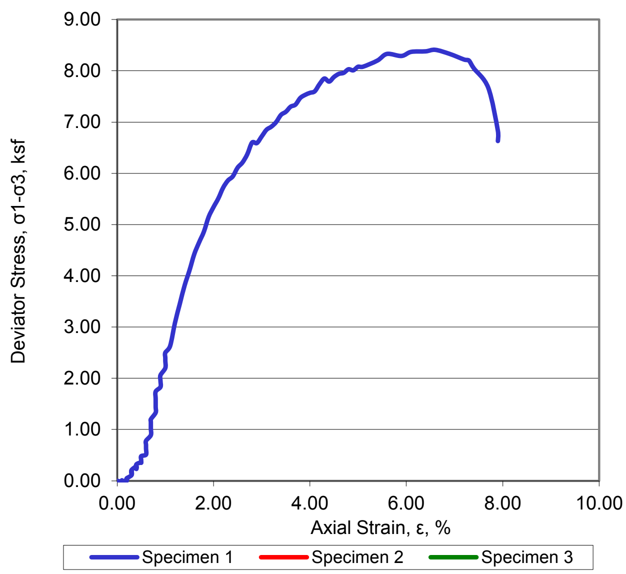
B-5

Triaxial Compression Test Report

Total		
c =	4.2	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.73
	Water Content, %	ω_O	18.6
	Dry Density, lbs/ft^3	γ_{d_o}	104.1
	Saturation, %	S_O	83.8
	Void Ratio	e_O	0.589
Minor Principal Stress, ksf		σ_3	0.5
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	8.4
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	6.6
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.8
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	6.58

Description of Specimen: Dark Yellowish Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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f| 510.887.5932
kleinfelder.com

Project No.:	123460
Boring:	B-2
Sample:	2C
Depth, ft.:	6.0
Date:	January 3, 2012

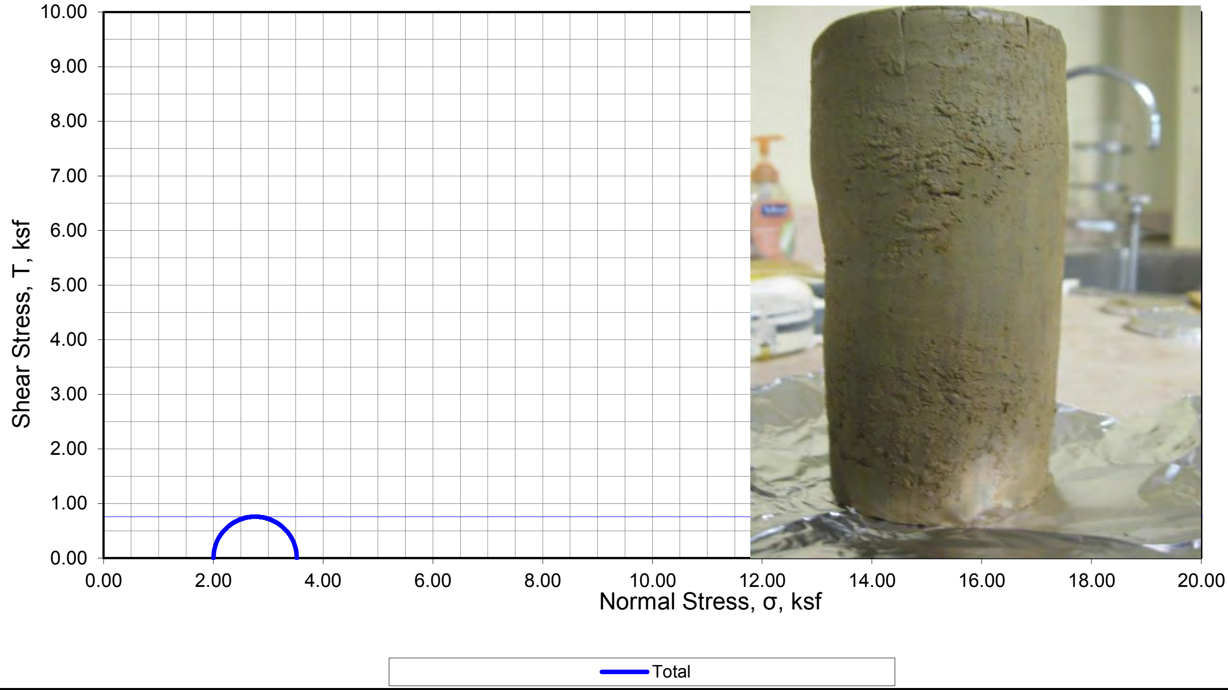
Plate

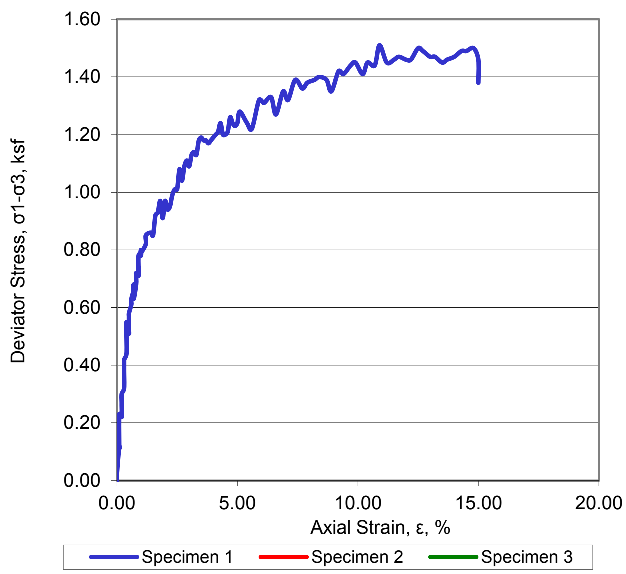
B-6

Triaxial Compression Test Report

Total		
c =	0.8	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.84
	Water Content, %	ω_O	29.9
	Dry Density, lbs/ft^3	γ_{d_o}	90.7
	Saturation, %	S_O	96.4
	Void Ratio	e_O	0.822
Minor Principal Stress, ksf		σ_3	2.0
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	1.5
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	12.2
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	1.5
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	12.22

Description of Specimen: Mottled Yellowish Brown Lean Clay with Sand (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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Project No.:	123460
Boring:	B-2
Sample:	7C
Depth, ft.:	31.0
Date:	January 3, 2012

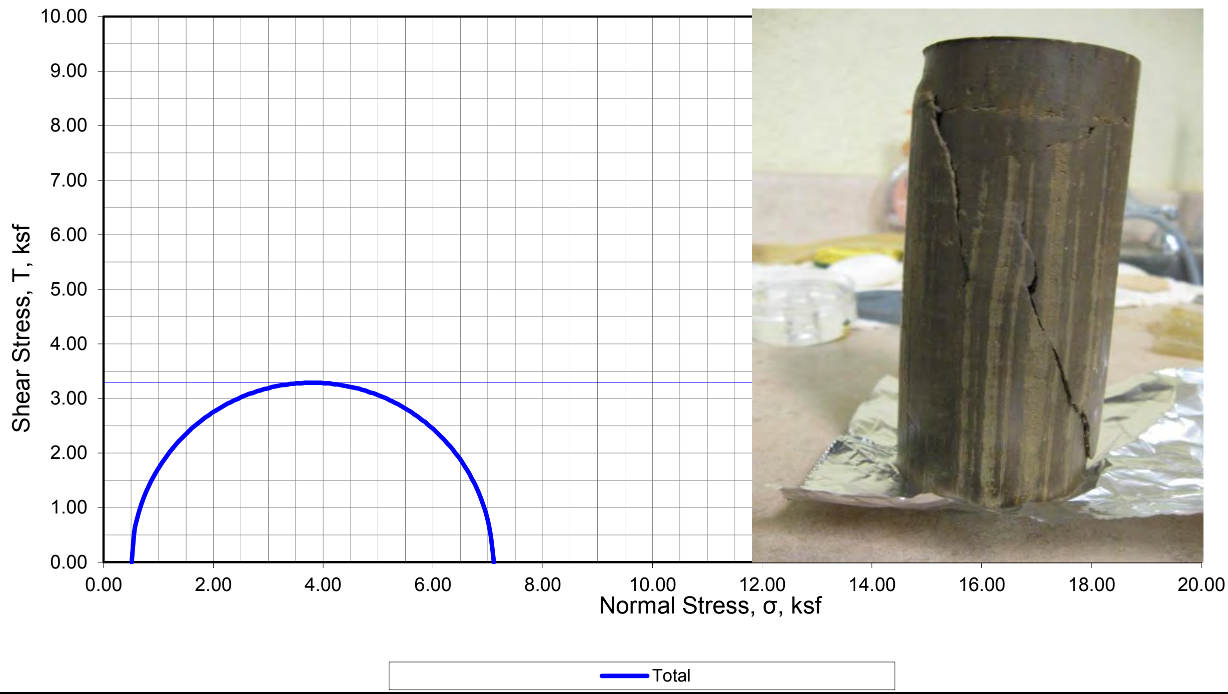
Plate

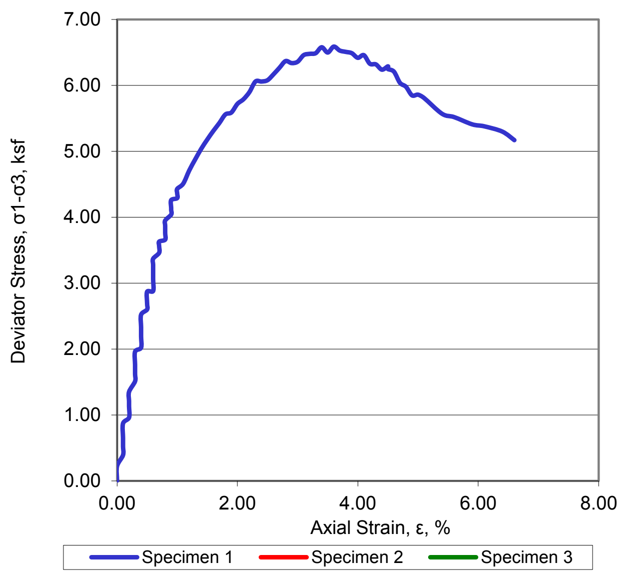
B-7

Triaxial Compression Test Report

Total		
c =	3.3	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.68
	Water Content, %	ω_O	21.8
	Dry Density, lbs/ft^3	γ_{d_o}	102.1
	Saturation, %	S_O	93.0
	Void Ratio	e_O	0.620
Minor Principal Stress, ksf		σ_3	0.5
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.6
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	3.6
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	5.2
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	3.55

Description of Specimen: Dark Yellowish Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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Project No.:	123460
Boring:	B-3
Sample:	2C
Depth, ft.:	6.0
Date:	January 3, 2012

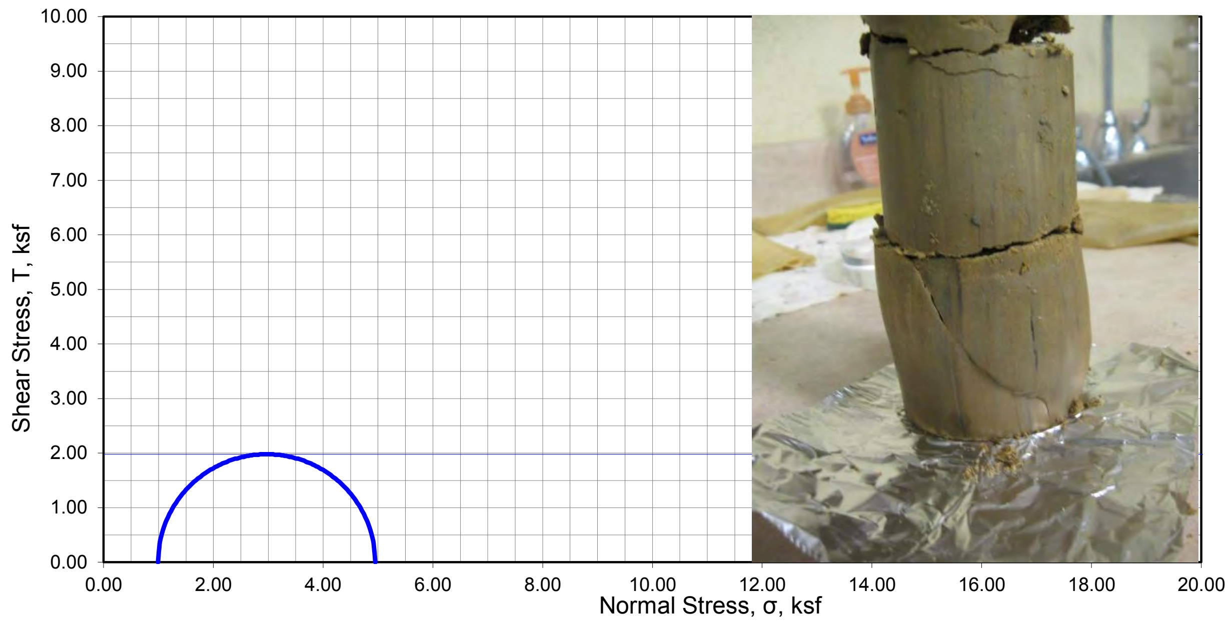
Plate

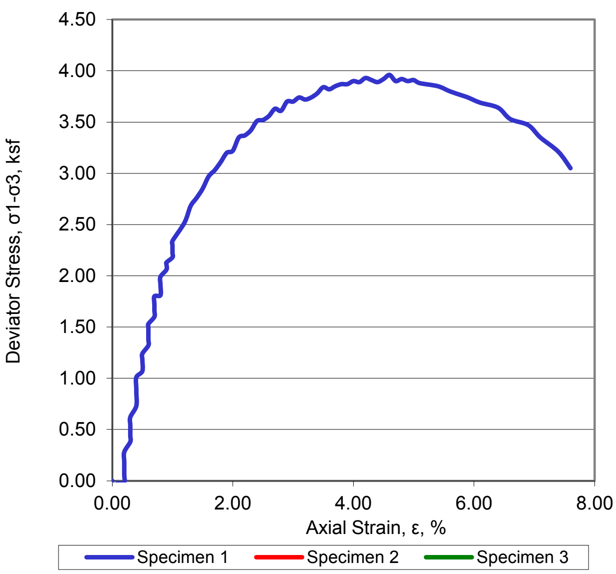
B-8

Triaxial Compression Test Report

Total		
c =	2.0	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.90
	Water Content, %	ω_O	21.5
	Dry Density, lbs/ft^3	γ_{d_o}	97.6
	Saturation, %	S_O	82.0
	Void Ratio	e_O	0.694
Minor Principal Stress, ksf		σ_3	1.0
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	4.0
Time to $(\sigma_1$-$\sigma_3)_{max}$, min		t_f	4.6
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.1
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	4.55

Description of Specimen: Yellowish Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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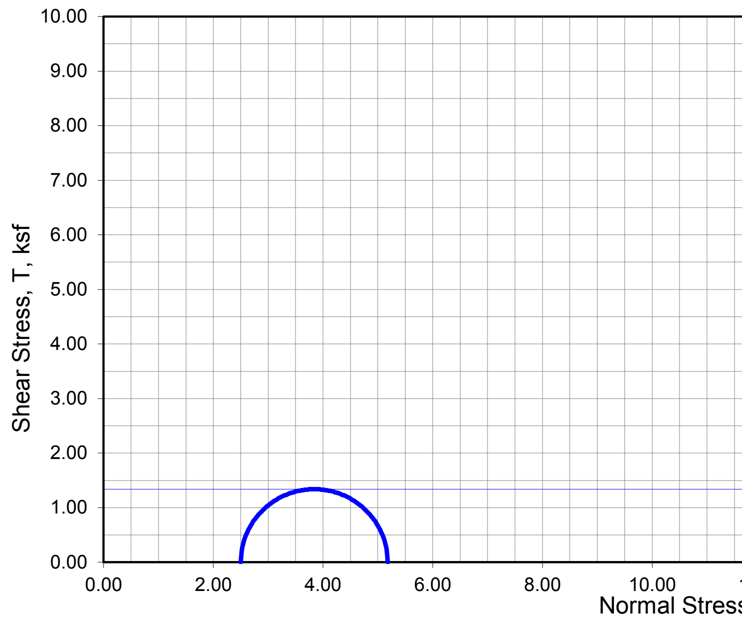
Project No.:	123460
Boring:	B-3
Sample:	4C
Depth, ft.:	16.0
Date:	January 3, 2012

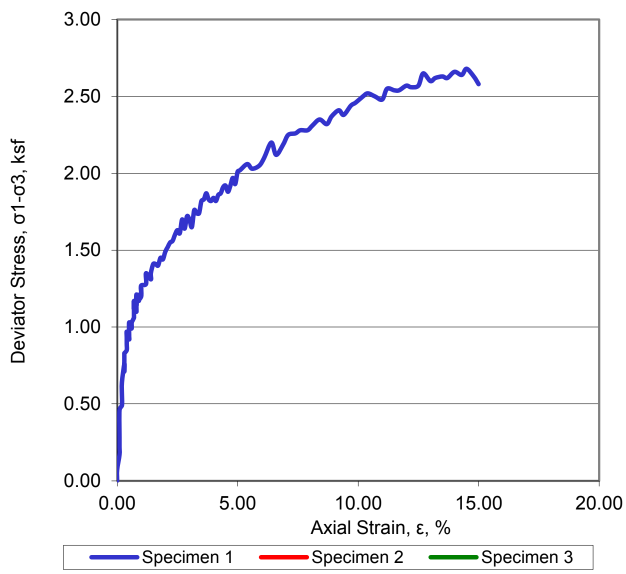
Plate

B-9

Triaxial Compression Test Report

Total		
c =	1.3	ksf





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.99
	Water Content, %	ω_O	22.9
	Dry Density, lbs/ft^3	γ_{d_o}	102.8
	Saturation, %	S_O	99.9
	Void Ratio	e_O	0.608
Minor Principal Stress, ksf		σ_3	2.5
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	2.7
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	14.3
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.6
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Mottled Bluish Gray and Yellowish Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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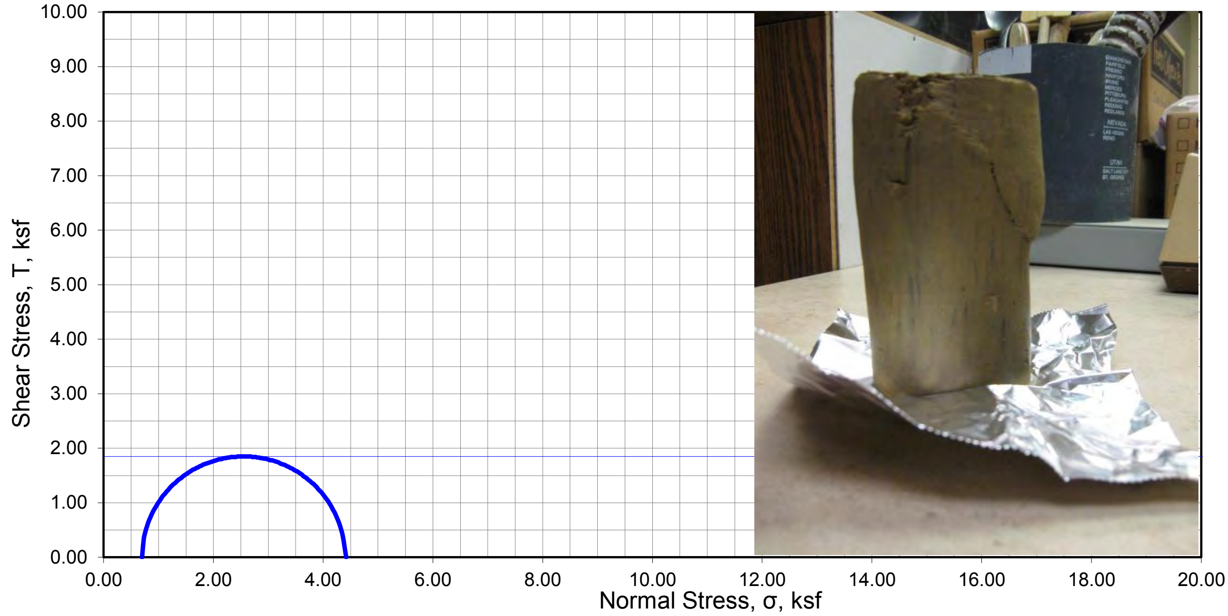
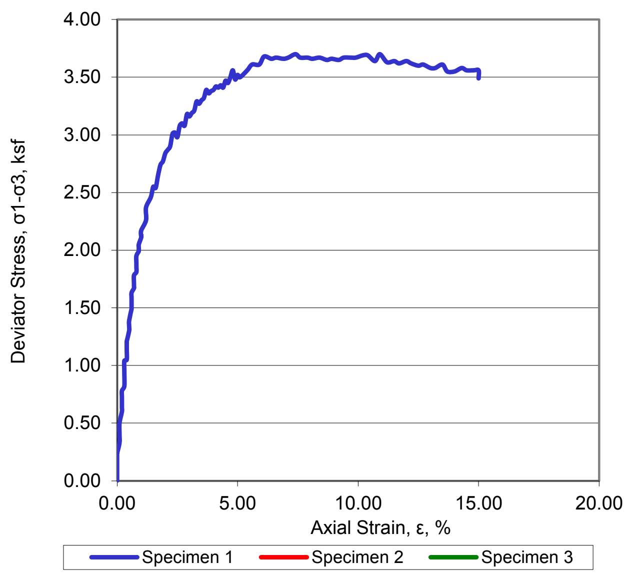
Project No.:	123460	Plate
Boring:	B-3	**B-10**
Sample:	10C	
Depth, ft.:	46.0	
Date:	January 3, 2012	

Triaxial Compression Test Report

Total		
c =	1.9	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.66
	Water Content, %	ω_O	19.4
	Dry Density, lbs/ft^3	γ_{d_o}	101.6
	Saturation, %	S_O	81.7
	Void Ratio	e_O	0.628
Minor Principal Stress, ksf		σ_3	0.7
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	3.7
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	7.3
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	3.6
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.0
Axial Strain at Failure, %		ε_f	7.32

Description of Specimen: Dark Yellowish Brown Sandy Lean Clay (CL)

Amount of Material Finer than the No. 200, %: nm

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

nm = not measured, na = not applicable

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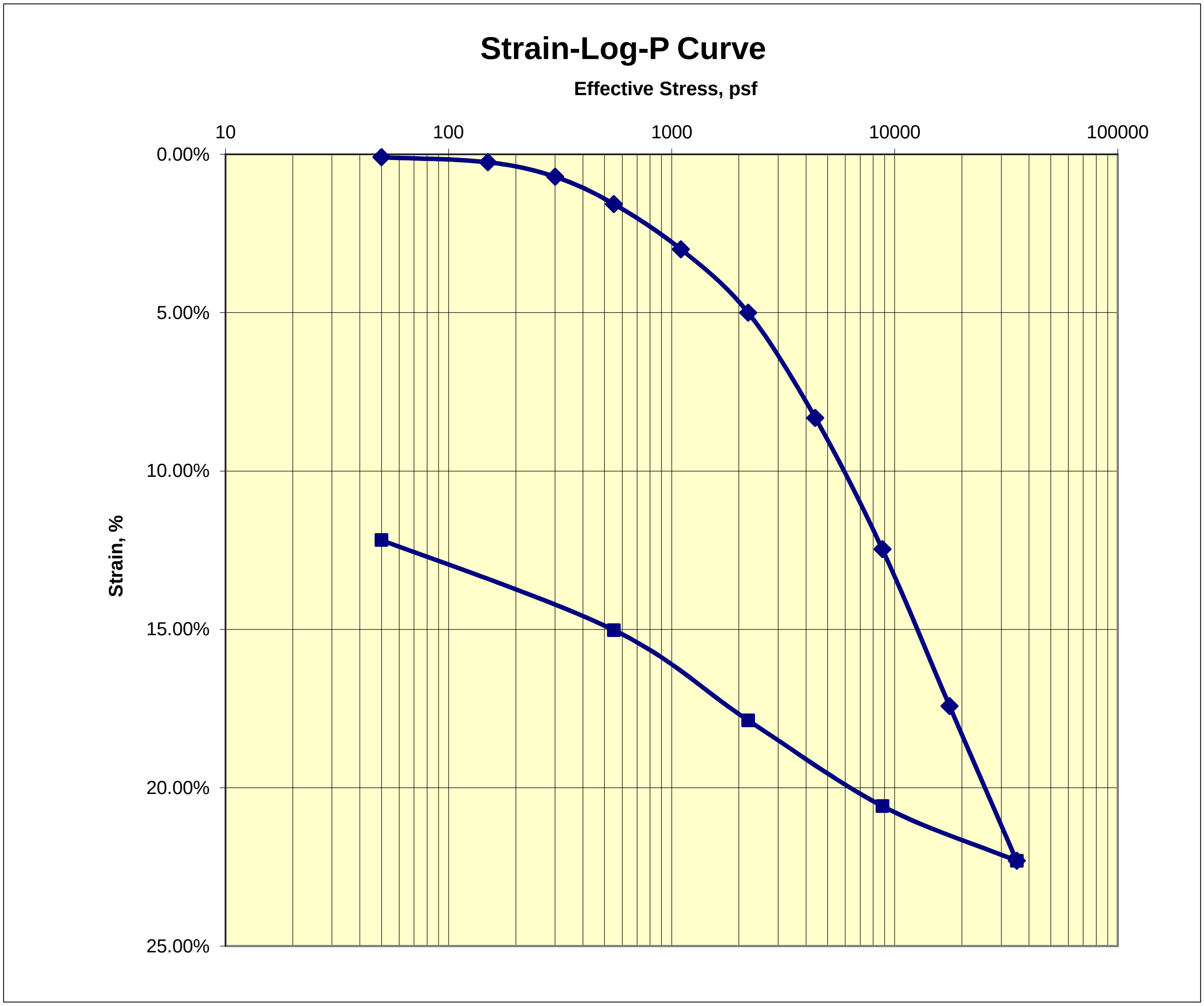
Project No.:	123460
Boring:	B-4
Sample:	3C
Depth, ft.:	11.0
Date:	January 3, 2012

Plate

B-11

Consolidation Test
ASTM D2435

Job No.:	336-219	**Boring:**	B-4	**Run By:**	MD
Client:	Kleinfelder	**Sample:**	6C	**Reduced:**	PJ
Project:	James Donlon Extension - 123066	**Depth, ft.:**	22	**Checked:**	PJ/DC
Soil Type:	Olive Brown CLAY			**Date:**	1/5/2012



Ass. Gs = 2.8	**Initial**	**Final**
Moisture %:	31.4	27.4
Dry Density, pcf:	89.6	99.1
Void Ratio:	0.952	0.765
% Saturation:	92.3	100

Remarks:

PLATE B-12

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

27 December, 2011

Job No.1112109
Cust. No.10527

Mr. Cristiano Melo
Kleinfelder
4670 Willow Road, Ste. 100
Pleasanton, CA 94566

Subject: Project No.: 123066
Project Name: James Donlon Extension
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Melo:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on December 14, 2011. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "severely corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations ranged from 260 to 480 mg/kg. Because the chloride ion concentrations are greater than 300 mg/kg, they are determined to be sufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations ranged from 720 to 790 mg/kg and are determined to be sufficient to potentially be detrimental to reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, with a maximum water-to-cement ratio of 0.55.

The pH of the soils ranged from 7.5 to 8.0, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential for both samples is 460-mV, which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

PLATE B-13

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 123066
Client's Project Name: James Donlon Extension
Date Sampled: 14-Dec-11
Date Received: 14-Dec-11
Matrix: Soil
Authorization: Signed Chain of Custody and PO#10718

Date of Report: 27-Dec-2011

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1112109-001	B-1, 2B, 5-1/2'	460	8.0	-	490	-	480	790
1112109-002	B-3, 1B, 3'	460	7.5	-	400	-	260	720

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	20-Dec-2011	20-Dec-2011	-	20-Dec-2011	-	27-Dec-2011	27-Dec-2011

* Results Reported on "As Received" Basis

Cheryl McMillen
Cheryl McMillen
Laboratory Director

PLATE B-13
(Cont'd)

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX C

Cross Section 1-1' - 1.75H:1V Slope
Static Analysis with GWL 50 feet below surface

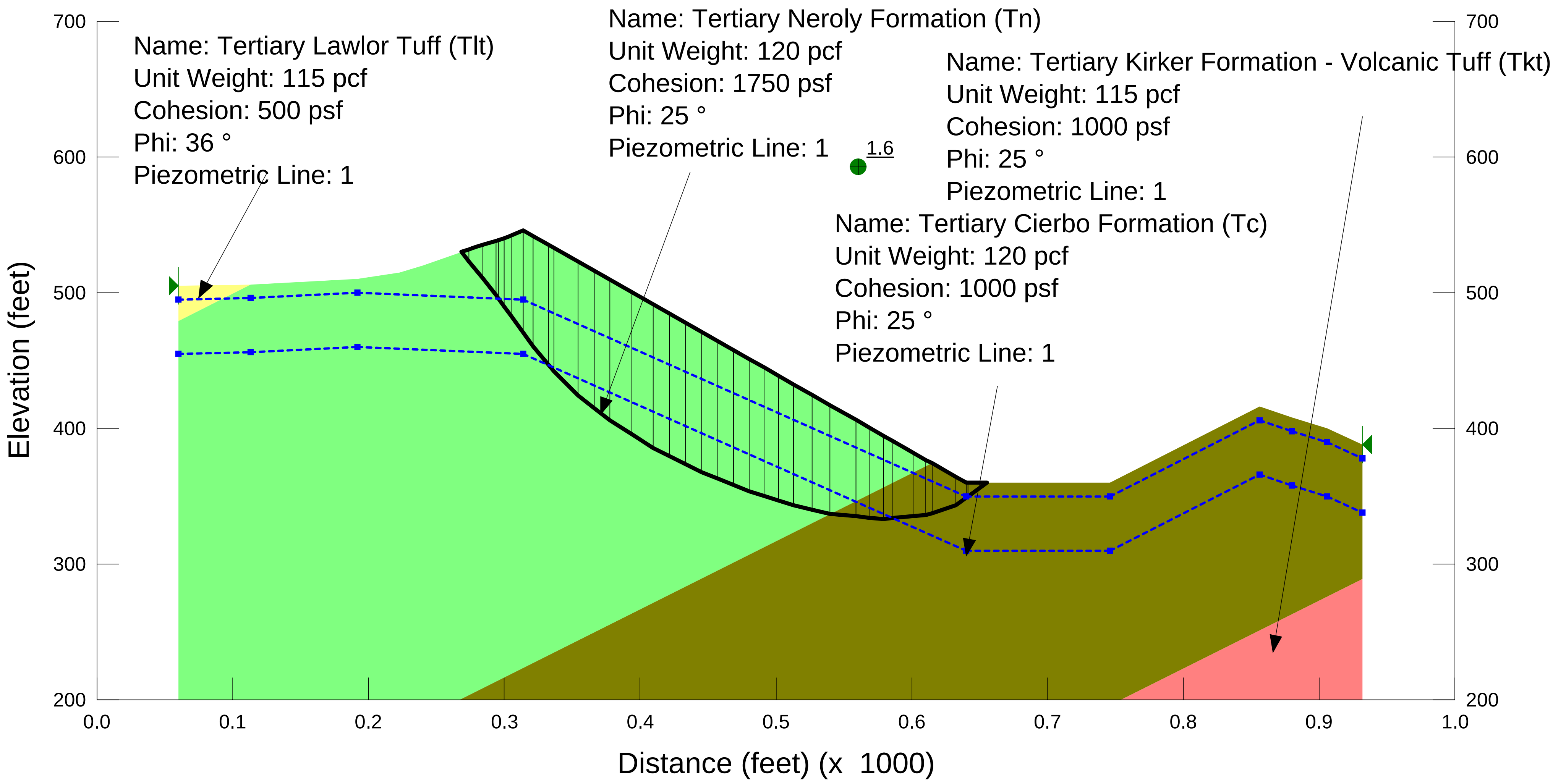


PLATE C-1

Cross Section 1-1' - 1.75H:1V Slope
Static Analysis with GWL 10 feet below surface

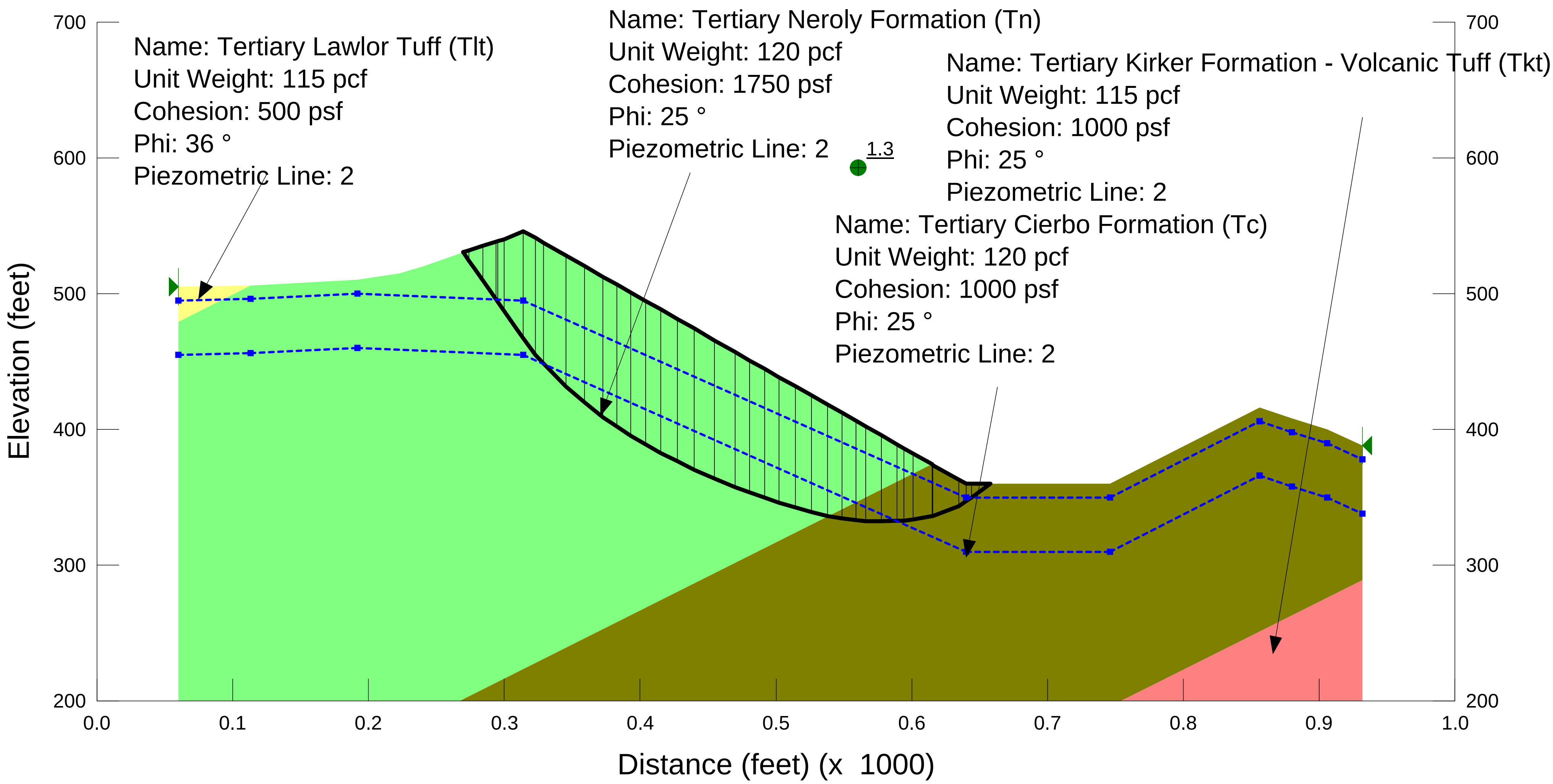


PLATE C-2

Cross Section 1-1' - 1.75H:1V Slope
Seismic Analysis (Kh=0.16) with GWL 50 feet below surface

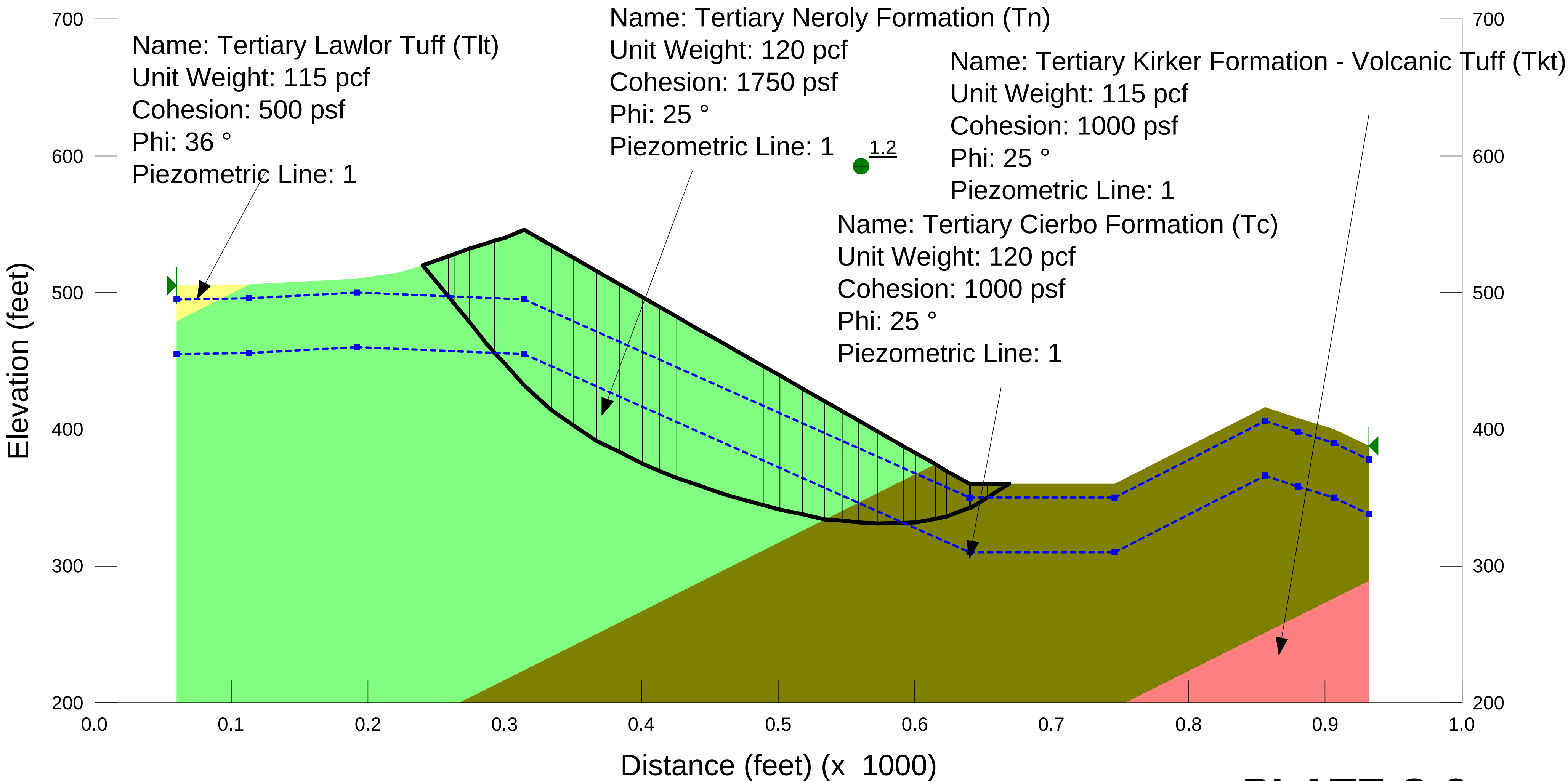


PLATE C-3

Cross Section 2-2' - 1.5H:1V Slope
Static Analysis with GWL 50 feet below surface

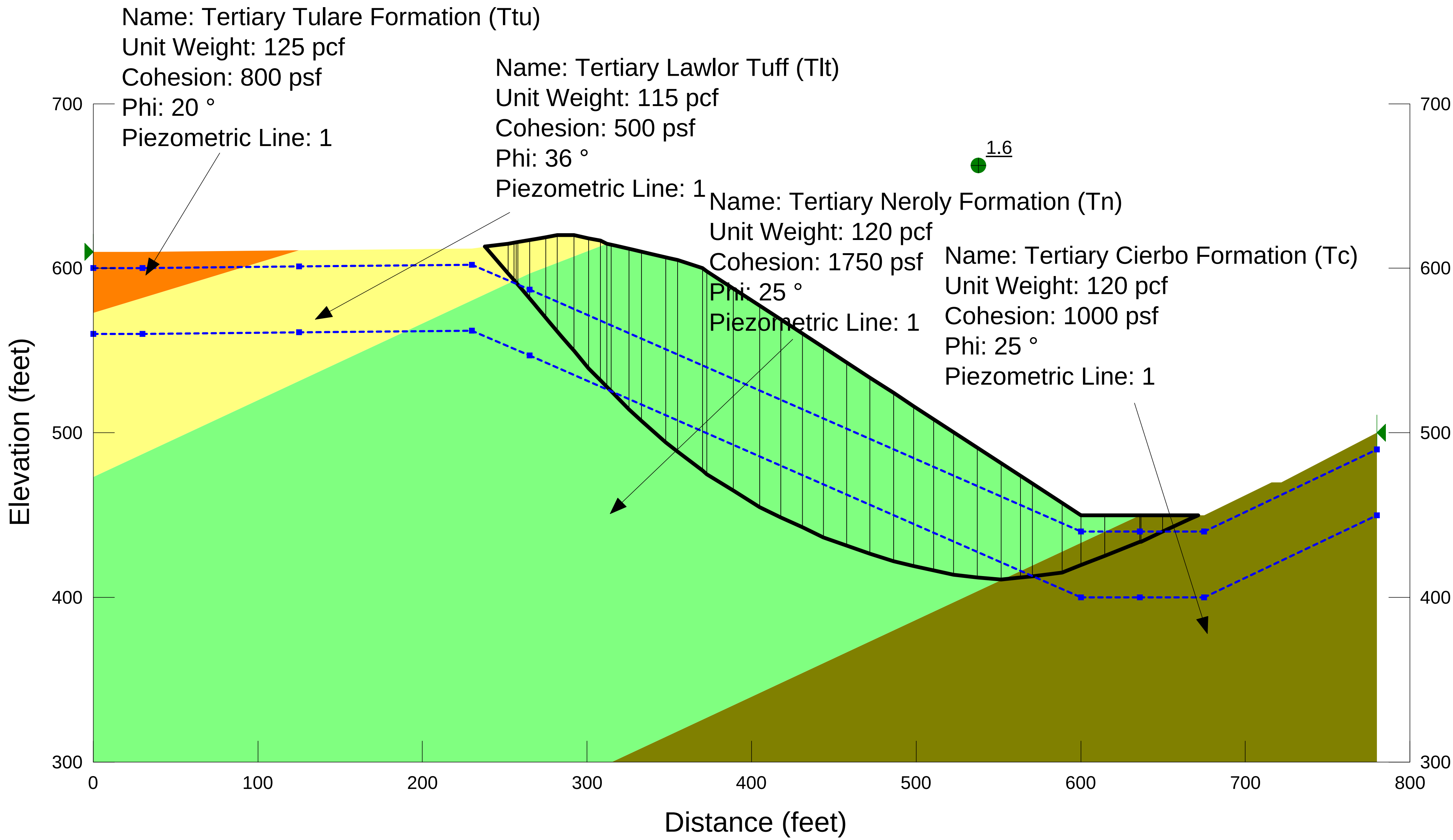


PLATE C-4

Cross Section 2-2' - 1.5H:1V Slope
Static Analysis with GWL 10 feet below surface

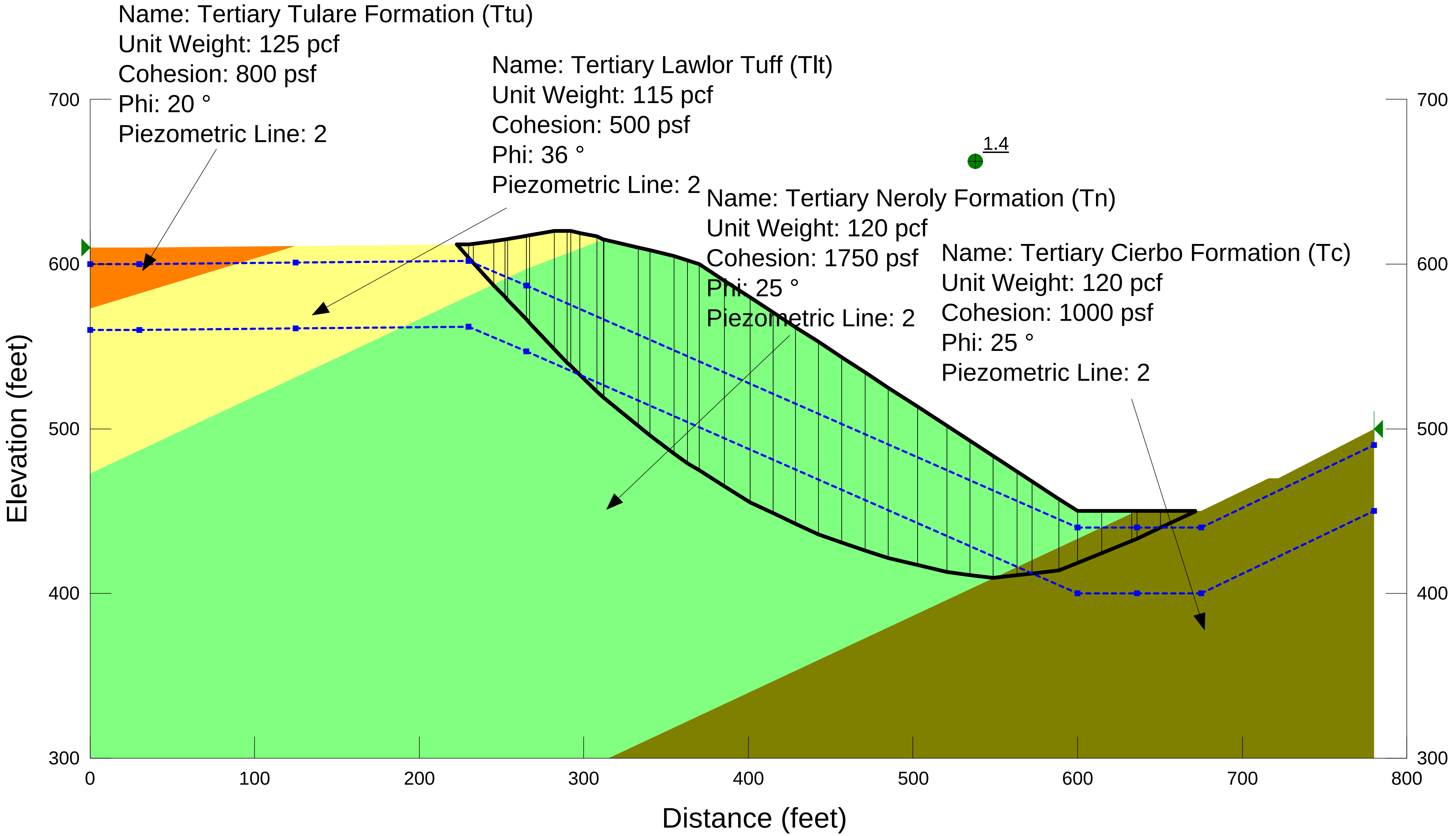


PLATE C-5

Cross Section 2-2' - 1.5H:1V Slope
Seismic Analysis (Kh=0.16) with GWL 50 feet below surface

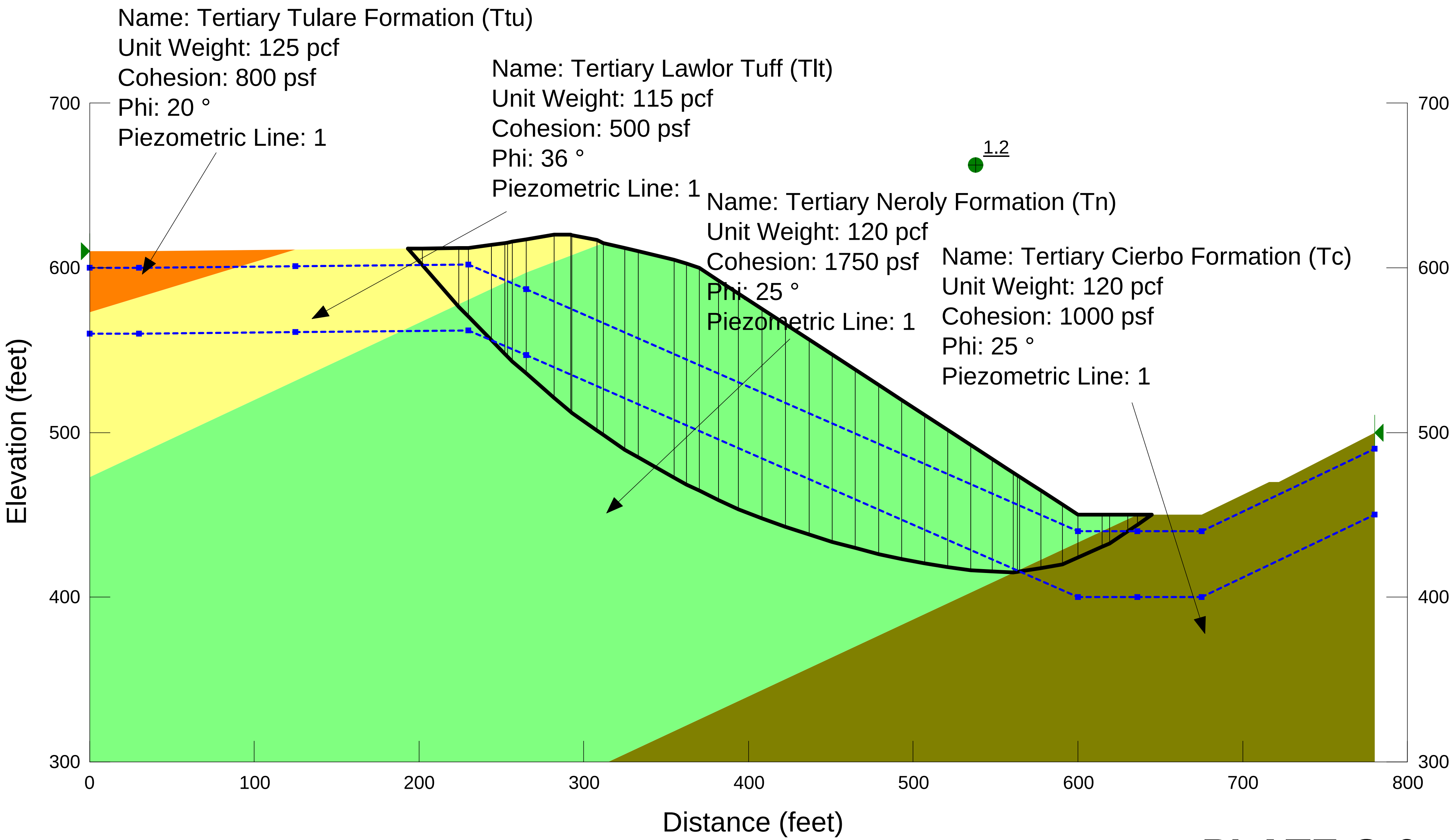
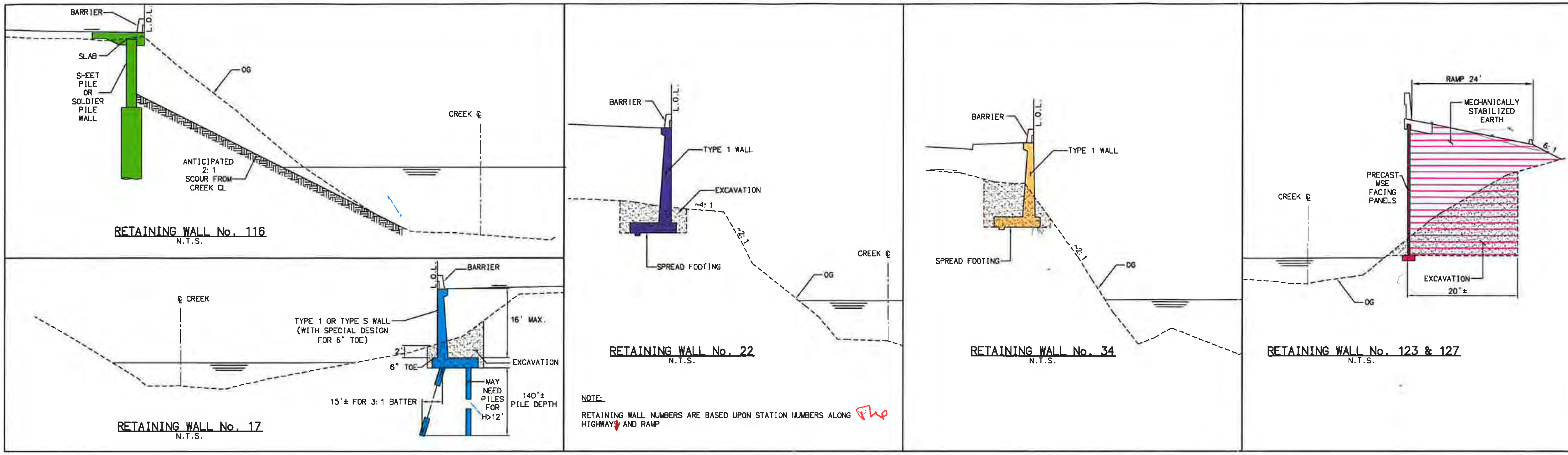
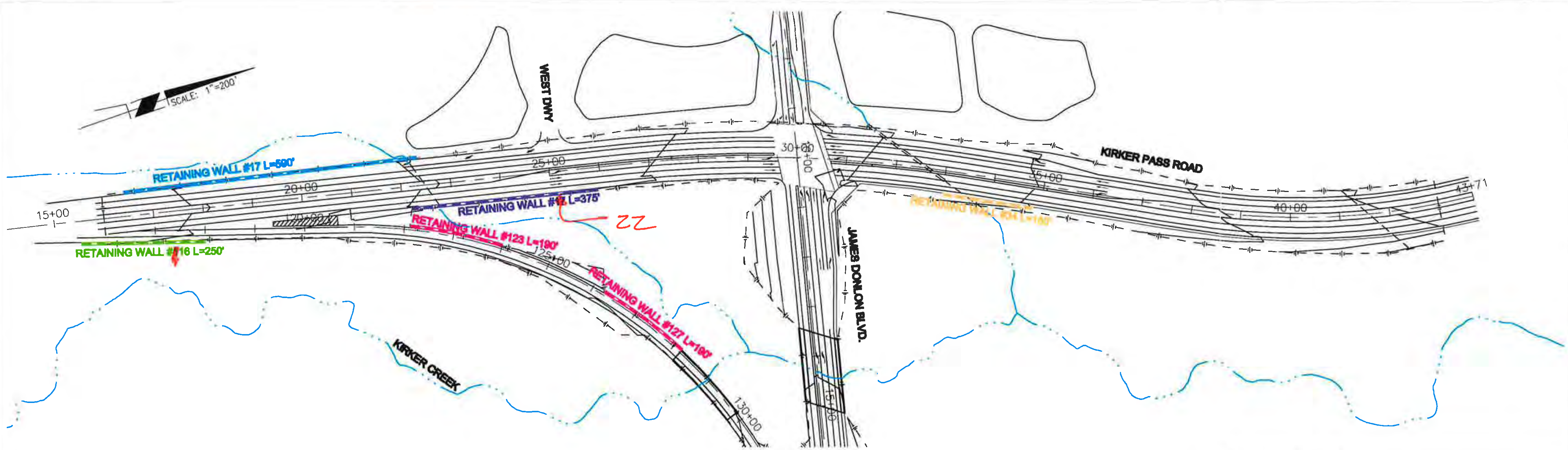


PLATE C-6

APPENDIX D





PROPOSED RETAINING WALLS

RBF

A Baker Company

PLANNING ■ DESIGN ■ CONSTRUCTION

500 YGNACIO VALLEY ROAD, SUITE 300
WALNUT CREEK, CALIFORNIA 94596-3847
PH 925.906.1460 • FAX 925.906.1465 • www.RBF.com

EXHIBIT B-4

H:\PDATA\35100129\CADD-ENGLISH\TRANSPW\EXHIBITS\MISC\REPORT EXHIBITS\0129-RETAINING WALLS.DWG ATORBICA 3/12/12 11:55 am