UPDATED PRELIMINARY GEOTECHNICAL INVESTIGATION
AND UPDATED FAULT RUPTURE HAZARD EVALUATION
TENTATIVE TRACT 34760, CORONA
RIVERSIDE COUNTY, CALIFORNIA 92882

FOR

MR. MANUEL VALENCIA
1253 ENTERPRISE ROAD
CORONA, CALIFORNIA 92882

W.O. 5166-A-SC OCTOBER 9, 2006

GeoSoils, Inc.

Geotechnical • Coastal • Geologic • Environmental

26590 Madison Avenue • Murrieta, California 92562 • (951) 677-9651 • FAX (951) 677-9301

October 9, 2006

W.O. 5166-A-SC

Mr. Manuel Valencia
1253 Enterprise Road
Corona, California 92882

Subject: Updated Preliminary Geotechnical Investigation and Fault Rupture Hazard Evaluation, Tentative Tract 34760, Corona, Riverside County, California

Dear Mr. Valencia:

In accordance with your request, GeoSoils, Inc. (GSI) is pleased to present the results of our updated preliminary geotechnical investigation and fault rupture hazard evaluation of the subject site. The purpose of our work was to update existing geologic and geotechnical site work completed by GSI (1998, 1997, 1996, 1995a, 1995b, and 1995c), including an evaluation of the geologic and geotechnical conditions of the site, and preliminary conclusions for grading and foundation design/construction pertinent to the currently proposed development. This report includes a supplemental geotechnical investigation, supplemental fault/lineament evaluation including a summary of all previous onsite fault investigations, and updated geotechnical foundation design parameters. The scope of our services has included: a review of the referenced documents (see Appendix A), including our previous reports (GSI; 1998, 1997, 1996, 1995a, 1995b, and 1995c) and other consultants' reports, as well as the 100-scale Tentative Tract Map 34760 by Armstrong & Brooks Consulting Engineers, Inc. (A&B, 2006); a field review of existing onsite conditions; supplemental field exploration, where warranted; analysis of data collected; and, preparation of this summary report. Considering that standards of practice change over time, recommendations provided in our previous reports may have been modified somewhat herein. Unless specifically superceded herein, the conclusions and recommendations contained in GSI (1998, 1997, 1996, 1995a, 1995b, and 1995c) remain pertinent and applicable, and should be appropriately implemented during planning, design, and construction.

EXECUTIVE SUMMARY

Based on our review of data (see Appendix A), additional field exploration (this study), and geologic and engineering analyses, the proposed site appears suitable for its intended use, from a geotechnical and geologic viewpoint, provided the recommendations presented in the text of this report and other applicable GSI reports (see Appendix A) are implemented. The most important elements of our review are provided below:

- Generally, as established through compilation of available data (including supplemental field work by GSI [this study]) and observation of present site conditions, the site may be characterized as being primarily underlain by Tertiary-age (Paleocene) bedrock designated as the Silverado Formation, capped by a relatively thin layer of colluvium, undocumented artificial fill associated with access roads and agricultural operations, alluvium in incised drainage areas, and localized surficial landslide deposits on the order of 5 to 10 feet thick. Removal and recompaction of the undocumented artificial fill, topsoil/colluvium, alluvium, surficial landslide deposits, and near-surface weathered bedrock materials will be required should settlement-sensitive improvements be proposed within their influence. For preliminary planning purposes, these depths are estimated to vary from ±1 to ±12 feet, with perhaps localized deeper removals, if not removed by planned excavation. Actual removal depths will be evaluated during onsite grading.

- It should be noted that the Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 1997 and 2001) indicates that removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the residential structures. Relatively deep removals may also necessitate a special zone of consideration, on perimeter, confining areas, such that this potential zone is approximately equal to the depth of removals, if removals cannot be performed offsite. Thus, any settlement-sensitive improvements (perimeter walls, curbs, flatwork, etc.) constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and related distress. This will likely occur along the property boundaries where buttress/slope restoration is currently recommended.

- Stabilization fills for slopes exposing the Silverado Formation are recommended. Based on sampling, laboratory testing, and our supplemental slope stability analyses (see Appendix F), proposed 2:1 (horizontal:vertical [h:v]) cut and fill slopes are considered grossly stable, provided our recommendations are properly implemented, including stabilization of cut slopes, where recommended. Therefore, stabilized cut and fill slopes constructed at 2:1 (h:v) gradients should not exceed ±135 feet in height. Provided our recommendations are properly implemented, cut and fill slopes would then be considered surficially stable, under conditions of normal care, maintenance, and rainfall. Based on the nature of the bedrock, depth of weathering, and the existing landslides, cut slope inclinations should be selected following a review by GSI. Cut slopes without stabilization should be allowed only after detailed evaluation and site specific data is available. Keyways for stabilization fills, fills over cut, and daylight cuts should be properly constructed, as depicted in the General Earthwork and Grading Guidelines section of this report (see Appendix G), and as shown on the map (see Plate 1) and cross-sections (see Plate 6) provided herein. Furthermore, close monitoring and observation of all cut slopes will be required during grading to further evaluate the presence or absence of adversely oriented geologic structures (i.e., bedding, fractures, etc.). While not anticipated, should such structures be identified during earthwork construction, additional remedial stabilization measures would be recommended at that time, based on the conditions exposed.

- The preliminary locations of the stabilization keyways are provided herein. The actual location of stabilization forecuts and backcuts should be shown by the design civil engineer. Subdrainage, and subdrainage outlets should be reviewed and recommendations and/or locations should also be similarly provided by the design civil engineer. These locations should be reviewed and modified as appropriate when 40-scale plans are made available.

- Based on our field reconnaissance mapping and observations, minor to relatively significant quantities of surficial slumps occur along some of the existing slopes (natural and constructed) on and/or adjoining the site. Surficial slumps, typically underlain by the Tertiary-age Silverado Formation, are located throughout the property. The adjacent property to the north is developed as residential, and slumps should occurred on the existing, constructed, descending slope to the development. Proposed residential development is planned to the east and southeast. The Cleveland National Forest exists to the south. In general, the existing surficial slumps will be removed during proposed grading, and proposed cut slope design, and/or stabilization fills should adequately mitigate the potential for future surficial slope instability, provided our recommendations are properly implemented, under normal rainfall conditions. This effort will not preclude onsite erosion-induced surficial instability nor offsite (outside grading) surficial instability that may affect the subject site. Our recommendations for debris walls, provided herein, generally mitigate this potential.

- Settlement monitoring will need to be conducted for engineered fill areas in excess of 50 feet in thickness. Settlement monitoring is estimated, at this time, to take place for a time period of approximately one to three months, with monitoring for deeper fills lasting from six to 15 months, or possibly less, based on the geometry and available data obtained. It should also be noted that generally-accepted standards of practice require that basal fill materials below, or thicker than, an engineered fill depth of 50 feet (including removals), be compacted to 95 percent of the laboratory standard. That is to say the upper 50 feet is compacted to 90 percent and below that depth to dense bedrock, the fills should be placed at 95 percent relative compaction (ASTM D-1557). Estimates of vertical deformation (settlement) and time rates should be revisited during the 40-scale plan review stage.

- Some extended waiting period for safe building should be anticipated for lots underlain by fills deeper than about 50 feet, based on the available data. Our preliminary analyses indicate that lots with fills less than 50 feet thick may generally be built on within about three to nine months from the conclusion of grading. In order to comply with accepted criteria, lots with fills exceeding 50 feet and approaching 100 feet may have suggested waiting periods potentially on the order of about six to 15 months, or longer. This may require planning/phasing of construction to avoid unnecessary delays. Following data review during and after grading is complete, lots may be released with respect to a future settlement waiting period, and within industry standards acceptable for construction.

- Further, in order to comply with current generally-accepted criteria for settlement, overexcavation of deep fills should be performed such that the maximum to minimum ratio of fill across any individual lot does not exceed 3:1 (maximum to minimum), and that the difference in fill thickness across each lot does not exceed about 25 feet.

- Based on laboratory testing, the expansion potential of the onsite soils generally ranges from very low to high. From a geotechnical viewpoint, conventional foundations may be utilized for very low expansion potentials (Expansion Index [E.I.] = 0-20) and where as-built fill thicknesses are less than 25 feet. Post-tension foundations are specifically required for low to highly expansive soils (E.I. >20) and where as-built fill thicknesses exceed 25 feet or if fill thickness variation across the site cannot be maintained at 3:1 (maximum:minimum). Preliminary foundation recommendations for conventional and post-tension designs are provided herein.

- Samples of the site materials have been previously evaluated (GSI, 1995a) for soluble sulfate content and corrosivity, with resultant negligible sulfate content, as well as the soils being mildly to moderately alkaline and moderately corrosive toward ferrous metals, etc. Additional laboratory testing should be performed during site development to further evaluate conditions encountered during GSI's feasibility-level geotechnical investigation (GSI, 1995a). Based on the preliminary laboratory test results, a corrosion engineer should also be consulted to provide specific recommendations for foundations, piping (other corrosion-sensitive improvements), etc., and where site soils will be in contact with structures or improvements.

- Perched groundwater was encountered (during this study) at depths of ±6 to 12 feet below the ground surface in localized low-lying drainage areas in the eastern portion of the site. No other occurrences of significant regional groundwater were noted during the previous explorations completed onsite (see Appendix A). Generally, regional groundwater is not expected to be a major factor in site development. However, due to the nature of the site materials, seepage may be encountered throughout the site along with seasonal perched water within existing canyon drainage areas, along zones of contrasting permeabilities (i.e. fill juxtaposed against alluvial fan deposits/sedimentary formational or bedrock materials), and also may be encountered in "daylighted" bedding within the Silverado Formation or in joints/fractures in bedrock, during or after grading. Accordingly, homeowners, and any homeowners association as well as all interested/affected parties should be notified of this potential. Should such conditions develop, GSI should be contacted for evaluation, and to provide recommendations for mitigation.

- Subdrains are recommended within drainage/canyon areas where proposed fills exceed 10 feet in thickness, as well as in some abutting areas where the as-built fill thickness exceeds 10 feet. Subdrains of buttress keyways and back cuts for stabilized slopes in accordance with Appendix F should be included in future designs. The drainage of subdrains toward suitable outlets and elevations that allow for gravity flow of subdrains should be addressed during 40-scale plan reviews. Additionally, subdrainage systems for the control of localized groundwater seepage should be anticipated following grading, due to excess irrigation or precipitation. This potential should be disclosed to all homeowners and any homeowners association, as well as all interested/affected parties.

- The potential for mass wasting, including mudflow debris, should be properly mitigated in steep portions of the site. It is recommended that debris impact walls/catchment basins or other comparable mitigative devices (GSI, 1995a) be incorporated into the project design, in accordance with the design civil engineer's recommendations, where steep slopes descend into the site, or where up-gradient drainages intercept cut slopes and/or the proposed development.

- Daylight cut lots will have some potentially compressible/erodible colluvium/topsoil exposed at the cut/natural interface adjoining slopes. This area will be more subject to erosion, and down-slope movement. Accordingly, improvements and/or foot traffic should <u>not</u> be allowed in this area, and proper drainage is imperative to the stability of this zone. This potential will be mitigated by the recommended setbacks, from a geotechnical viewpoint. These conditions will need to be disclosed to all homeowners and any homeowners association, as well as all interested/affected parties. The actual location of this zone should be further evaluated during grading.

- Stabilization for the road and planned slope near the adjacent existing subdivision development in Cross-Section B-B' (Plate 6, enclosed) has been designed for a potential for downslope movement on the adjacent property. As indicated in our analyses, geogrid reinforcement of the slope in Cross-Section B-B' is necessary to reduce the potential for an offsite surficial slide to affect the project and to retain the required factor-of-safety (FOS) for slope stability. Our current slope stabilization design has considered this potential in the design. A detailed review of this grid reinforced slope will be necessary at the 40-scale grading plan review.

- Due to the retention of the home on Cross-Section D-D', GSI has not completed the design of the buttress in this area since the current plan will likely change. Localized surficial instability near the existing residence has been mapped during our recent field work. Based on our current review and available field and laboratory data, this area, in its existing condition, may not meet current slope FOS when considering the design seismic event.

- Active faults have been previously mapped just outside the subject property, or outside of the proposed improvements, and were encountered during field exploration performed by Highland Soils Engineering, Inc. (HSE, 1987). The site is situated in an area of active faulting. The Elsinore fault zone is located on the northern portion of the property, is considered active, and is included within an Alquist-Priolo Earthquake fault zone. Fault setback zones were previously recommended by HSE (1987); and based on our additional exploration, supplemental fault trends, and available data, GSI generally concurs with the location of the faults and associated setback zone. The fault setback zone is shown on Plate 1, and has been previously approved by the governing agency.

- The site generally has a moderate to high risk of being affected by seismic hazards. The seismicity acceleration values provided herein should be considered during the design of the proposed development. Strong seismic shaking remains a concern at this site with an anticipated probabilistic upper bound acceleration (Probalistic Horizontal Site Acceleration [PHSA]) of 0.87g for 10 percent probability of exceedance in 50 years. Although liquefaction is not considered a significant seismic hazard, some seismic densification, on the order of ¼ percent (up to 3½ inches) in some deeper fill areas, should be anticipated. Some deformation of the tops of fill slopes (more onerous than the UBC [ICBO, 1997]) setback zone, should be anticipated (on the order of several inches), when subjected to the design level seismic loading.

- Construction monitoring will be needed between this project and the existing tract development, as well as homes that may remain. This includes, but not limited to, constructive vibration monitoring, slope inclinometers, photographic/video documentation, and surveys.

- Adverse geologic features that would preclude project feasibility were not encountered, although geometric constraints with respect to backcuts and slope stabilization fills may warrant reconfiguration of pads.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,

GeoSoils, Inc.

John P. Franklin
Engineering Geologist, CEG 1340

Andrew T. Guatelli
Principal Engineer, GE 2320

Daniel P. Walsh
Engineering Geologist, CEG 2413

DPW/JPF/ATG/jk/jh

Distribution: (6) Addressee

TABLE OF CONTENTS

UPDATED PRELIMINARY GEOTECHNICAL INVESTIGATION AND FAULT RUPTURE HAZARD EVALUATION TENTATIVE TRACT 34760, CORONA RIVERSIDE COUNTY, CALIFORNIA 92882

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of readily available soils and geologic data pertaining to the site, including previous onsite work by GSI and HSE (see Appendix A).

2. Geologic site reconnaissance and geologic mapping.

3. Subsurface exploration consisting of the excavation of four hollow stem auger borings, 11 backhoe test pits, excavation of six fault-locating trenches and one dozer cut on an access road, for geotechnical logging and sampling (see Appendix B). Logs of previous site exploration (GSI; 1995a, 1995b, and 1995c), including previously-approved complete reports, are included in Appendix C, on compact disc.

4. General areal seismicity evaluation (see Appendix D).

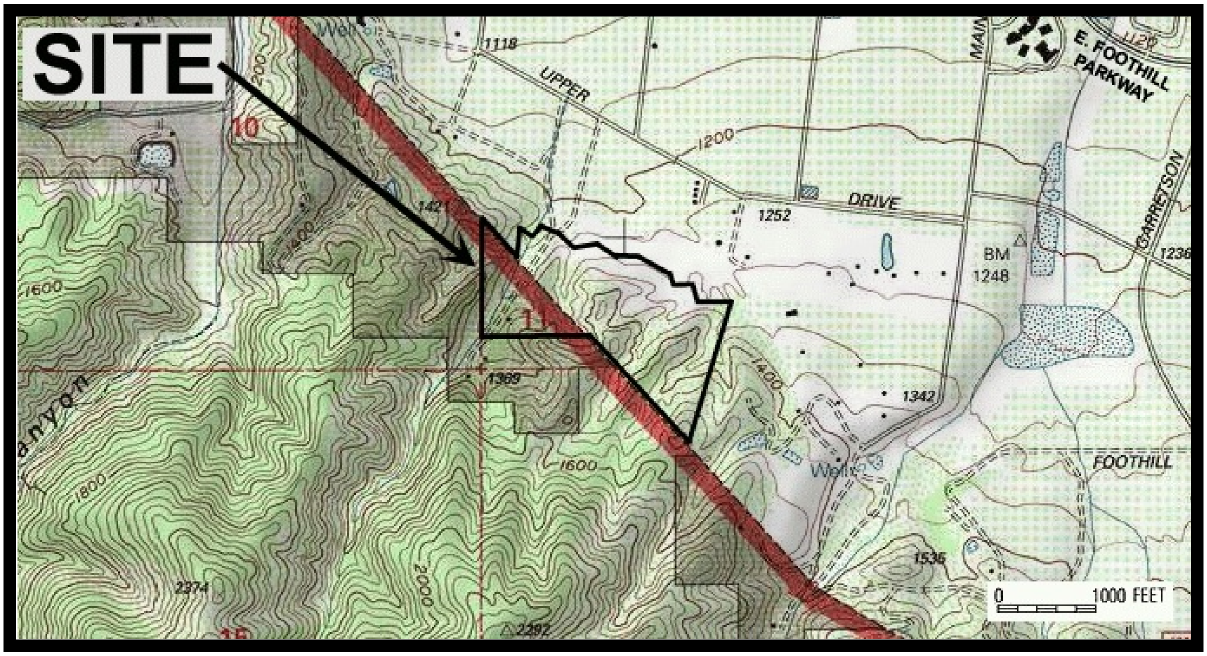
5. Laboratory testing of representative site soils (see Appendix E).

6. Slope stability analyses (see Appendix F).

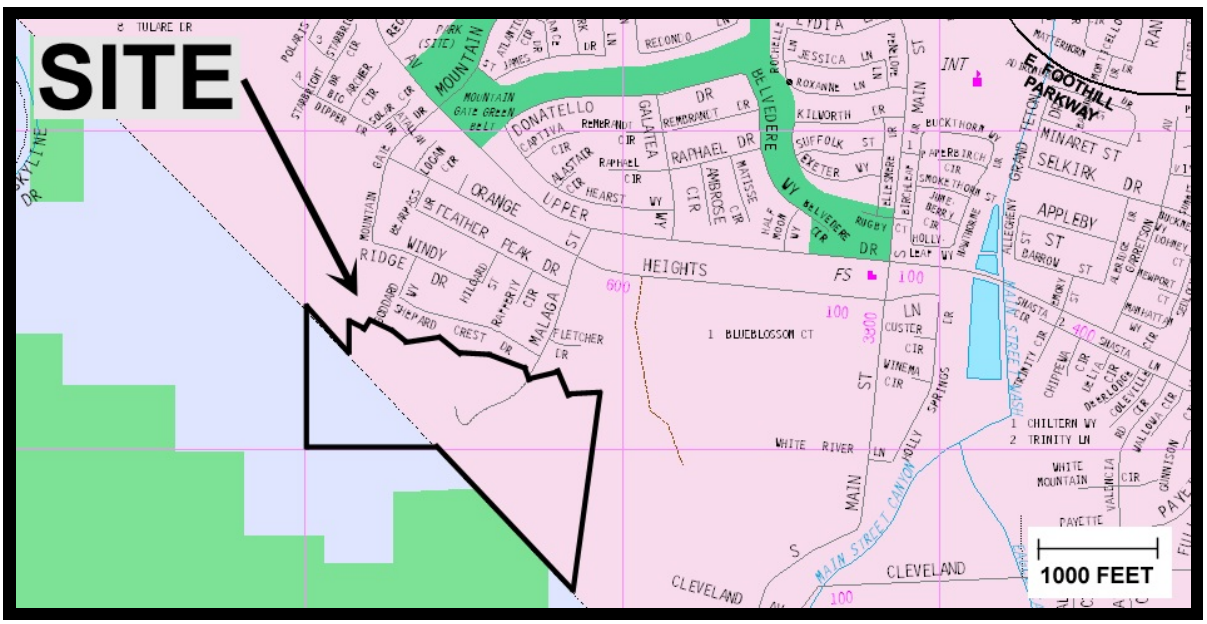
7. Appropriate engineering and geologic analysis of data collected and preparation of this report.

SITE DESCRIPTION

The project site under review consists of a hilly, irregular-shaped property, located on the south side of Shepard Crest Drive, near Malaga Street, on the flanks of the Santa Ana Mountains foothills (Cleveland National Forest) in Corona, Riverside County (see Figure 1, Site Location Map). The elevation onsite ranges from approximately 1,230 feet Mean Sea Level (MSL) to about 1,590 feet MSL. Surface drainage (sheetflow) generally follows topography to the northeast. The property is currently used for avocado and citrus agriculture. Several graded but unimproved roads, used for agricultural maintenance, provide access into nearly all areas of the site. Appurtenances associated with the active orchards are scattered throughout the project area. A residence exits on private property within the southwestern portion of the property.



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Corona South Quadrangle, California–Riverside Co., 7.5-Minute, dated 1997, current 1997.



Base Map: The Thomas Guide, Riverside Co. Street Guide and Directory, 2005 Edition, by Thomas Bros. Maps, pages 772 and 773.

LOCATION AND SCALES APPROXIMATE

N

GeoSoils, Inc.	W.O. 5166-A-SC
SITE LOCATION MAP	
Figure 1	

PROPOSED DEVELOPMENT

According to our review of the Tentative Tract Map 34760 (Armstrong & Brooks Consulting Engineers, Inc. [A&B], 2006), 28 single-family residential lots are proposed with interior roadways and utility improvements. Building loads are assumed to be typical for one- or two-story, wood-frame buildings utilizing slab-on-grade and/or post-tension foundations. Typical cut and fill grading techniques are proposed to develop the site at design grades. Fills on the order of 33 to 102 feet and cuts on the order of 13 to 73 feet are proposed. Maximum 2:1 (horizontal:vertical [h:v]) cut and fill slopes are proposed to 91 and 95 feet, respectively. Limits of a fault zone setback, located on the northern portion of the site outside the proposed residential development, is shown on the Tentative Tract Map 34760. Sewage disposal is proposed to be accommodated by tying into the regional municipal system.

BACKGROUND/PREVIOUS STUDIES

Based upon a review of our previous reports (see Appendix A), the property is flanked on the northern boundary by the state-designated Alquist-Priolo Earthquake Fault Zone (APEFZ) for the Elsinore fault. The site may be surficially characterized as generally being underlain by relatively recent alluvial deposits in active drainage bottoms, local surficial landslide deposits, and Paleocene-age sediments, known as the Silverado Formation.

A previous phase of geotechnical site work was completed in 1995 by GSI, with our findings, conclusions, and recommendations presented in a geotechnical report dated November 9, 1995 (GSI, 1995a). This phase of site work included subsurface exploration, laboratory testing, preliminary earthwork factors, engineering analysis including slope stability analysis and liquefaction analysis, and geotechnical recommendations for site development. The site boundaries for this previous study were slightly different than the current boundaries and the previous property was identified as Tentative Parcel 28201. The following is a summary of the pertinent conclusions.

- Due to the presence of surficial landslides on the site, remedial grading consisting of the removal and recompaction of side slopes would be required in order to bring the site to design grades. Owing to the fractured nature and engineering properties of the bedrock onsite, stabilization of the proposed cut slopes associated with the building pads would be required. Within the immediate areas proposed for improvements, the landslide deposits were expected to be completely removed by the planned excavation, with the exception of the area near the southern boundary. Direct mitigation (buttress or shear key, etc.) was not anticipated for this condition. Field mapping during grading was recommended to be performed to further evaluate the exposed conditions.

- Within the site, due to the presence of bedrock faults and fractures owing to the folding of these sediments, there was and is some potential for secondary movement on these discontinuities, should a large seismic event occur nearby. Accordingly, a 5-foot overexcavation, and replacement with properly compacted fill was recommended. In addition, the use of post-tension slabs or pier and grade beam foundation systems was also recommended in order to further mitigate the potential for sympathetic secondary movement along the bedrock discontinuities.

- The expansive characteristics of soil and bedrock materials encountered throughout the site are expected to generally range from low to high. The use of post-tension slabs or pier and grade beam foundation systems was recommended in order to mitigate the potential for shrink/swell and/or settlement.

- Ground rupture along a nearby segment of the Elsinore fault may provide significant deformations within portions of the site. However, no portion of the proposed development crossed the mapped splay of the Elsinore fault zone.

- The control of overland flow on the natural and man-made slopes in close proximity of the proposed building pads is essential owing to the erosive nature of the onsite formation materials. Debris catchment of deflection devices should be designed by the project civil engineer and constructed to mitigate the mudflow debris potential.

- It is anticipated that subdrainage may be necessary in canyon/swale areas; however, it will also be necessary in any stabilization backcuts. The need for subdrainage should be further evaluated when project grading plans are finalized, and during project earthwork.

FIELD STUDIES

Field work accomplished during our current supplemental site investigation for this study entailed geologic reconnaissance mapping, excavation of 11 exploratory test pits, advancement of four hollow-stem borings, excavation of six fault-locating trenches and one dozer cut on an access road. The field mapping, exploratory test pits, hollow stem borings, fault-locating trenches, and dozer cut were completed/logged by an Engineering Geologist from our firm. Logs of the test pits and hollow stem borings are presented in Appendix B. Logs of the trenches and dozer cut are provided on Plates 2 through 5. Logs of previous site exploration (GSI; 1995a, 1995b, and 1995c), are included on disc in Appendix C. The approximate locations of the exploratory test pits, hollow-stem borings, trenches, dozer cut, and previous site explorations are shown on Plate 1 (Geotechnical Map). Geologic cross-sections depicting the subsurface data are included as Plate 6.

GEOLOGY

Regional Geologic Setting

The site is located on the western margin of the Perris Block, a portion of a prominent natural geomorphic province in southwestern California known as the Peninsular Range. The Peninsular Range is characterized by steep, elongated ranges and valleys that trend northwesterly. The Santa Ana Mountains lie along the western side of the Elsinore fault zone, and the Perris Block is located along the eastern side of the fault zone. This province is typified by plutonic and metamorphic rocks (bedrock), which comprise the majority of the mountain masses with relatively thin volcanic and sedimentary deposits discontinuously overlying the bedrock, and with alluvial fan deposits filling in the valleys, and younger alluvium filling in the incised drainages. The alluvial deposits are derived from the water borne deposition of the products of weathering and erosion of the bedrock.

The Corona-Santa Ana Narrows region is comprised of three major structural blocks: the Santa Ana Mountains block, bounded on the northeast by the Elsinore and Whittier faults; the Puente (Chino) Hills block, bounded on the northeast by the Chino fault and on the southwest by the Elsinore and Whittier faults; and the Perris block, located on the northeast side of the Chino fault. The present landforms of the Santa Ana Mountains and Corona area are a result of late Quaternary faulting and uplift associated with the Elsinore fault zone. The bedrock has been eroded, and the resulting detritus has been deposited as a series of Quaternary-age alluvial fans. Younger stream alluvium associated with modern drainage is being deposited in drainages incised within the bedrock and alluvial fans.

Local Geology and Site Earth Materials

The onsite bedrock or formational units are predominantly the Paleocene-age Silverado Formation (Gray, et al., 2002). The surficial units consist of minor amounts of undocumented artificial fill associated with agriculture, colluvium (topsoil/slope wash/talus soils), young active alluvium, and landslide deposits. Please note that our previous report indicated that the onsite bedrock consisted of the Puente Formation; however, based upon paleosols encountered in our recent trenches onsite, it appears that this formation is likely the Silverado Formation (Leyva, 2002). Accordingly, our previous test pit logs (GSI, 1995a), are re-interpreted in light of the above. Leyva (2002) also points out the occurrence of homoclinal folds associated with fault blocks within the Silverado Formation. Further, in a gross sense, the onsite formational sediments, although locally tightly folded, appear to be tilted and moderately inclined, perhaps in grossly homoclinal fashion, generally in a southerly direction.

The earth materials are generally described below from youngest to oldest; and the approximate limits of the mappable units, based on the available data, are presented on Plate 1. Geologic cross-sections are provided on Plate 6. These units are described as follows, from youngest to oldest.

Artificial Fill - Undocumented (Not Mapped)

Small to moderate amounts of undocumented artificial fill are scattered throughout the site. This material is generally in the range of 5 to 10 feet thick, locally thicker, and is associated with old roads and past agricultural operations. Undocumented fill typically consists of mixed loose surficial materials. As encountered onsite, the material consists of dark brown, clayey sand with abundant branches and tree trunks, scattered cobbles and concrete debris. The undocumented fill typically has a low to high expansion potential based on visual classification. These materials are considered potentially compressible in their existing state and may settle appreciably under additional fill or foundation and improvement loadings. The distribution of significant amounts of undocumented fill is shown on Plate 1. Due to the development of portions of the site into orchards, it is our observation that some undocumented fills associated with the orchard terraces, supporting utilities, and road repairs are likely present onsite.

Quaternary-age Colluvium and/or Topsoil/Slope Wash/Talus Soils (Not Mapped)

In general, the site is mantled by a relatively thin layer of colluvium (topsoil/slope wash/talus soils). The colluvium was generally observed to be light to dark brown to reddish brown, dry to damp to mainly moist, loose to medium dense and medium stiff to very stiff, clayey to silty sands and sandy clays having locally abundant cobbles. These surficial materials were typically variably porous, moderately to highly plastic, fine to medium grained, and contained minor to locally abundant roots. In general, these materials typically have a medium to high expansion potential. The colluvium was encountered ranging from ± 1 to ± 10 feet in thickness. Due to the potentially compressible nature of these surficial soils, they are considered unsuitable for support of structures and/or improvements in their existing state. Therefore, these soils will be need to be removed and recompacted, during planned excavation, should settlement-sensitive improvements be proposed within their influence. Some colluvium may exist within orchard areas that was observed during our field work and may become obvious once these orchards are removed.

Quaternary-age Alluvium (Map Symbol - Qal)

Up to a $\pm$ 4- to 15-foot thick veneer of alluvium was encountered in the active drainage channels that dissect parts of the site. As observed, these sediments basically consist of light to medium yellowish brown to reddish brown, dry to moist, silty sands to gravelly sands with occasional to locally abundant subangular to subrounded cobbles (up to ± 1 foot in diameter). The alluvial deposits are generally porous, contain locally abundant roots, and exhibit densities ranging from loose to medium dense to locally dense. Due to the potentially liquefiable, densifiable, compressible, and/or collapsible nature of these soils, they are considered unsuitable for support of structures and/or improvements in their existing state and therefore will be need to be removed and recompacted, in areas proposed for development. Some alluvium in smaller drainages may exist within orchard areas that were obscured during our field work and may become obvious once these orchards are removed.

Quaternary-age Landslide Deposits (Map Symbol - Qls)

Locally, surficial landslide deposits occur throughout the site. The landslide deposits are derived from parental rocks that include colluvium and the underlying bedrock, as a likely result of translational or bedding plane failures, and/or rotational failures and slumps. As such, these materials have similar engineering properties. The landslide deposits are anticipated to be relatively thin, on the order of 5 to 10 feet in thickness. These materials are considered potentially compressible and subject to lateral movement. Within the immediate areas proposed for improvements, the landslide deposits are expected to be completely removed by the planned excavation, with the exception of the area along the south portion of the property, or during remedial removals, and as such, direct mitigation (buttress or shear key, etc.) is not anticipated for this condition. However, this will have to be further evaluated during grading, based on exposed conditions. Stabilization of the proposed cut slopes associated with the building pads near the south portion of the site will be required.

Tertiary (Paleocene-age) Silverado Formation (Map Symbol - Tsi)

The bedrock on the site is the Silverado Formation. It varies in composition throughout the site from yellow to gray to brown to reddish brown to olive green siltstone and minor claystone, and fine- to coarse-grained, silty sandstone to a conglomerate with a sandstone matrix. The bedrock has shears/fractures and bedrock faults oriented in varying directions (Highland Soils Engineering, Inc. [HSE], 1987). These sediments are Paleocene in age. The upper 1 to 2 feet of the bedrock is highly weathered. The unweathered bedrock is considered suitable for the support of settlement-sensitive improvements and/or engineered fill in its present condition. However, these sediments were encountered to be are generally medium to highly expansive, and therefore, care should be taken not to allow these expansive sediments to be located in the zone of influence of foundations (i.e., less than 7 feet below finish grade). Structurally, site bedrock observed generally strikes to the northwest and dips in a southwesterly direction. The bedrock observed was generally disturbed (i.e., moderately to highly fractured and locally folded); however, no subsurface signs of deep-seated landslides were observed. Some instability of the unweathered Silverado in cuts (or backcuts) should be anticipated owing to the folded/mudstone layers within the formation. Competency of site bedrock should be further evaluated during the 40-scale grading plan review as well, as during grading by GSI.

It is anticipated that most of the bedrock within the Silverado Formation onsite should excavate readily with conventional heavy-duty grading equipment. However, grading could become more difficult where unweathered and/or well-indurated bedrock or bedrock containing concretions (well-cemented zones) are encountered. This should be considered during project planning. Bulking of the unweathered claystone and sandstone layers within the Silverado should be expected as indicated in the grading section of this report. Some seeps may manifest during grading of cut slopes in the Silverado.

Geologic Structure

Based on our field mapping and subsurface exploration, bedding within the Silverado Formation is highly disrupted owing to the presence of the Elsinore fault. Because features possibly indicative of faulting were generally coincidental with the trace of the Elsinore fault zone to the north, fault setback zones were previously recommended by HSE (1987); and based on additional exploration, GSI generally concurs with the location of the faults and associated setback zone. This setback zone has been approved by the County. The fault setback zone is shown on Plate 1. Bedding attitudes taken within the Silverado Formation indicate that this formation is generally moderately to highly inclined, generally to the southwest, although local folding allows for variable orientations. A folded series of interbedded sandstones, with lesser amounts of siltstone and claystone, that dip moderately to steeply southwesterly, have been inferred also. As a result of this folding, numerous fractures, bedrock and bedding plane faults are apparently contributing to the occurrence of numerous surficial landslides noted on the flanks of the hills. The depth of this bedrock characteristic should be re-evaluated during the 40-scale grading plan review.

Bedding within the alluvial deposits is generally thick and horizontally to gently inclined, dipping northerly to northeasterly, grossly paralleling the existing topography. Bedding here is not obviously affected by local faulting.

MASS WASTING

Mass wasting refers to the various processes by which earth materials are moved downslope in response to the forces of gravity. Most of the slopes within the sites are subject to downslope creep of surface or near surface materials. The materials most subject to creep of surface are artificial fill, topsoil, slopewash, and highly weathered/fractured bedrock. Creep is the slowest form of mass wasting, and generally involves the outer 5 to 10 feet of the slope surface. During heavy rains, such as those in 1969, 1978, 1980, 1983, 1993, 1995, and 2004, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides and/or surficial failures). Examples of these types of slope instabilities exist locally on portions of the subject sites. Creep effects were observed in onsite improvements, including existing roads, utilities, and structures.

Surficial failures are most apparent along canyon areas and/or along the steeper slopes, and typically involve the outer 1 to 4 feet of the slope surface. Erosion of these areas may also produce local gullying. Proposed graded areas adjoining natural slopes subject to creep, surficial failures, and gullying, etc., may require structural setbacks and/or appropriate protective devices, in accordance with the recommendations of the design engineer.

Slope failures appear associated with unsupported bedding planes that are adversely oriented relative to existing natural slopes and highly-erosive formational materials. They are also due to local, highly-fractured bedrock affected by past folding and faulting. Some of the slides are better defined than others. Although site drilling and trenching (GSI, 1995b and 1995c) have been performed at the adjacent tract where topographic expressions could be interpreted to suggest a possibility of deep-seated sliding, no deep-seated slides have been identified on the subject site by this firm, based on field and air photo review to date.

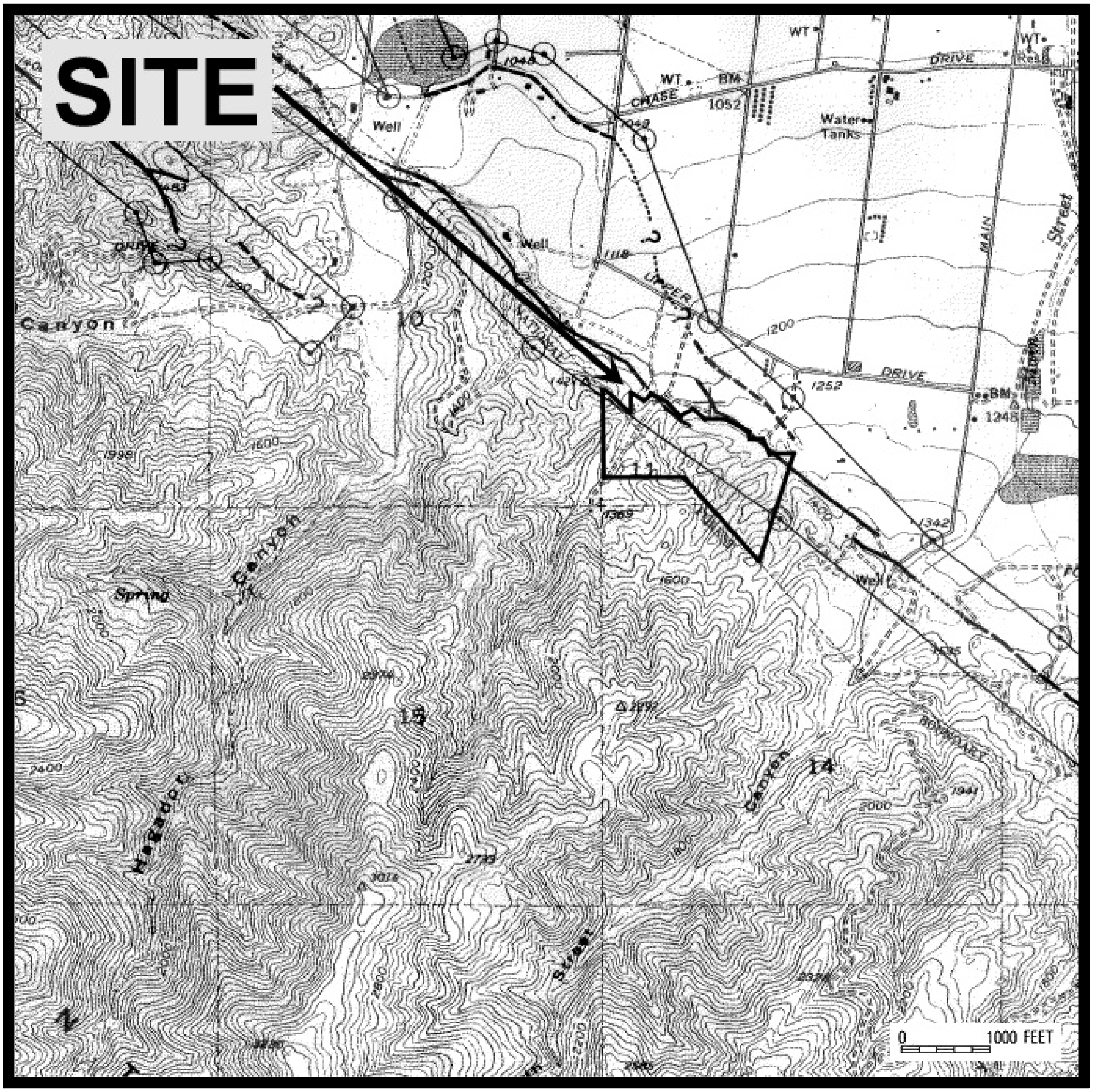
Soils of the Silverado Formation tend to deteriorate or break down as a result of weathering. Geotechnical observations of exposed conditions should be performed by a representative of this office during site grading to further assess the stability of site bedrock material(s).

The hillside alluvial and formational materials are erosive in nature. The mechanism by which the silty sand or clayey sand material deteriorated with sufficient water (during irrigation of hillside orchards or during winter storms as previously discussed) may be explained, in part, by the natural cementation of the soil/formation in hillside test pits around the proposed development. Hand samples (drive tubes, and chunks) were subjected to inundation with water. Although the field samples of these earth materials are not anticipated to exhibit significant collapse potential as a response to wetting (previous geotechnical studies, Appendix A), they did exhibit slaking. This deterioration in water develops from the outside or surface of the chunk and inward. This combined with formation structure, jointing that is near vertical, etc., and oversteepened cut slopes associated with agricultural roads, may be the likely explanation for the numerous surficial landslide deposits observed on the site.

FAULTING

Regional Faulting

Active faults have been previously mapped on the subject property, and were encountered during field explorations performed by HSE (1987). The site is situated in an area of active faulting. The Elsinore fault zone is located on the northern portion of the property and is considered active and is included within an APEFZ (see Figure 2). Major active fault zones that may have a significant affect on the site should they experience activity are listed in the following table (modified from Blake, 2000a):



Base Map: State of California, Special Studies Zones, Corona South Quadrangle, dated 2003.

LOCATION AND SCALES APPROXIMATE

GeoSoils, Inc.	W.O. 5166-A-SC
EARTHQUAKE FAULT ZONE MAP	
Figure 2	

N

ABBREVIATED FAULT NAME	APPROX. DISTANCE MILES (KM)	ABBREVIATED FAULT NAME	APPROX. DISTANCE MILES (KM)
Chino-Central Ave. (Elsinore)	0.0 (0.0)	Cleghorn	33.3 (53.6)
Elsinore (Glen Ivy)	0.9 (1.4)	Upper Elysian Park Blind Thrust	33.9 (54.6)
Whittier	3.5 (5.6)	North Frontal Fault Zone (west)	36.6 (58.9)
San Joaquin Hills	15.5 (25.0)	Palos Verdes	36.7 (59.0)
Puente Hills Blind Thrust	17.9 (28.8)	San Jacinto - Anza	38.7 (62.3)
Elsinore (Temecula)	18.7 (30.1)	Verdugo	38.8 (62.4)
San Jose	19.4 (31.3)	Hollywood	42.1 (67.8)
Cucamonga	22.0 (35.4)	Coronado Bank	43.6 (70.2)
Sierra Madre	22.1 (35.6)	Elsinore (Julian)	45.3 (72.9)
San Jacinto - San Bernardino	23.1 (37.2)	Rose Canyon	49.8 (80.2)
San Jacinto - San Jacinto Valley	23.8 (38.3)	Santa Monica	50.6 (81.4)
Newport-Inglewood (offshore)	25.0 (40.3)	Sierra Madre (San Fernando)	51.2 (82.4)
Newport-Inglewood (L.A. Basin)	25.0 (40.3)	Pinto Mountain	51.8 (83.4)
San Andreas - Coachella	29.4 (47.2)	San Gabriel	52.3 (84.1)
San Andreas - San Bernardino	29.4 (47.3)	North Frontal Fault Zone (East)	52.4 (84.4)
San Andreas - 1857 Rupture	32.0 (51.5)	Helendale - S. Lockhardt	55.4 (89.1)
San Andreas - Mojave	32.0 (51.5)	Malibu Coast	56.2 (90.5)
Clamshell - Sawpit	32.0 (51.5)	Northridge (E. Oak Ridge)	56.6 (91.1)
Raymond	32.7 (52.7)	Santa Susana	62.5 (100.6)

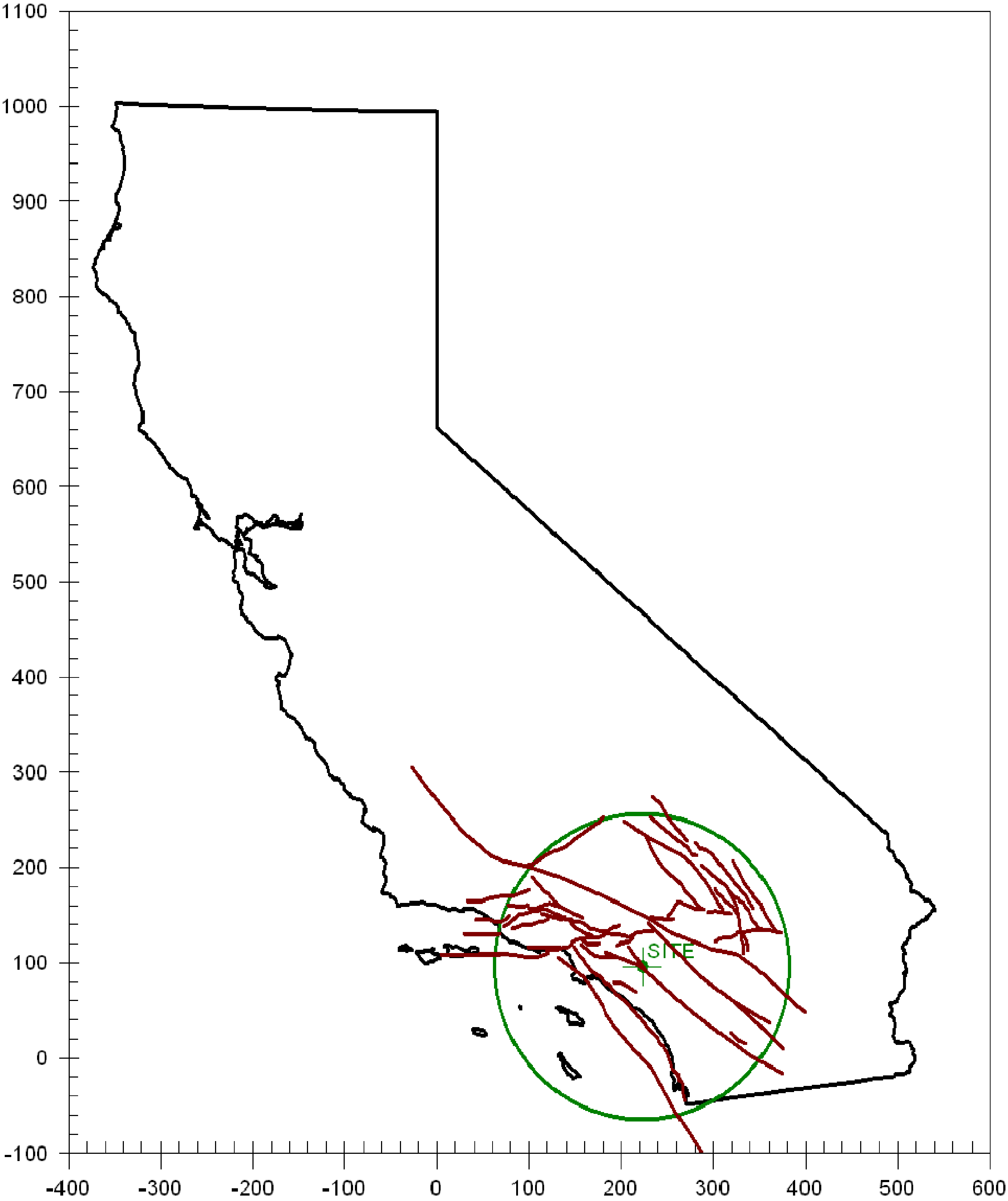
The relationship of the site to these major mapped faults is indicated on Figure 3 (California Fault Map). Other faults have been mapped in the vicinity; however, these faults are shorter, and hence, are generally considered less likely to produce significant seismic events.

Local Faulting

The Elsinore fault zone is the dominant structural feature in the general area. The Elsinore fault zone forms a boundary that separates the Perris block on its eastern side from the Santa Ana Mountains on its western side. The northwest-trending Elsinore fault zone extends about 125 miles from directly north of the Mexican border to the northern end of the Santa Ana Mountains. Significant lateral offset has been reported (Lamar and Rockwell, 1986). At the northern end of the Santa Ana Mountains, the Elsinore fault appears to split into the Chino fault, which continues north-northwest near the eastern margin of the Puente Hills, and the Whittier fault, which continues west-northwest along the southwest side of the Puente Hills.

CALIFORNIA FAULT MAP

Valencia



W.O. 5166-A-SC

GeoSoils, Inc.

Figure 3

The Chino fault has been mapped as a 13-mile-long, steeply west-dipping, northwest-trending, reverse fault that lies along the northeastern margin of the Puente (Chino) Hills, crosses the Prado Basin, and converges with the Whittier and Elsinore faults (Heath, et al., 1982). The Elsinore fault, Whittier fault, and Chino fault have been recognized as active faults. The Puente Hills area has been shown to be seismically active (Lamar, 1972). The Chino fault has been suggested (Dibblee, 1982) to ultimately connect to the left-lateral San Antonio fault, well to the northeast of the site, which is related to the interaction of the Cucamonga and San Jacinto faults. Maher (1982) suggests that the bifurcation of the Chino, and Whittier and Elsinore faults are a result of transpression along the Whittier-Elsinore trend. This transpression has resulted in the uplift, tilting and folding of the sedimentary rocks of the Miocene-age Puente Formation in the Puente (Chino) Hills area. Conspicuously, this large-scale uplift, tilting and folding is not apparent along the previously mapped trace(s) of the Chino fault in the site vicinity.

The Chino fault primarily exhibits dip-slip displacement (Heath, et al., 1982). The Elsinore and Whittier fault zones in this area are a wide zone consisting of parallel or subparallel, en-echelon, fault strands or branches. Both dip-slip and strike-slip movement have been documented along the Whittier and Elsinore fault zones (Lamar and Rockwell, 1986).

Heath, et al. (1982) indicated that the vertical deformation slip rate on the Chino fault is about 0.06 mm/year. However, Blake (2000a) indicates that the vertical slip rate is about 1.0 mm/year. Heath, et al. (1982) pointed out that the lateral deformation rate is probably not greater than the vertical rate. They further concluded that the dominate displacement on the Chino fault is likely reverse, with the Puente Hills block uplifted with respect to the Perris block to the east. They also suggested that a small amount of counter-clockwise rotation is occurring on the Puente block. Thus, the Chino fault does not appear to be a primary extension of the active Elsinore fault. However, it does appear to be tectonically related to the Elsinore and Whittier faults and the presence of continuing north-south compression in the region makes future movement on the Chino fault appear likely. Recent mapping of the Chino fault by Jennings (1994), shows several northwesterly to northeasterly trending faults well north of the sites, that appear to be the manifestations of the transfer of slip (transtension) from the Elsinore fault to the Chino fault. Blake (2000a) indicates a recurrence interval of about 2,000 to 19,000 years for M6.0 to 7.0 earthquakes, and about 200 to 650 years for M5.0 to M5.5 earthquakes, respectively, on the Chino fault. Blake (2000a) estimated slip rates of 4.0 to 6.0 mm/year for the Elsinore fault, which equates to a recurrence interval of about 500 to 3,000 years for M6.0 to 7.0 earthquakes, and about 10 to 325 years for M5.0 to M5.5 earthquakes, respectively, on the Elsinore fault. However, it should also be noted that trenching done at Glen Ivy Marsh on the Glen Ivy segment of the Elsinore fault, (Rockwell, et al., 1986) demonstrated a 150- to 200-year recurrence interval for surface ruptures, with the most recent event in 1910. Current publications on the Elsinore fault by the USGS and CGS indicate a preferred recurrence interval of 250 and 340 years, respectively.

The local strand of the Elsinore fault north of the site, has been given the name of the "Main Street fault" segment, as found in the vicinity of the mouth of Main Street Canyon, south of Corona (Weber, 1977). This segment is expressed principally by steep and prominent scarps, and these features extend discontinuously along the entire length of the Main Street fault. Youthful activity is also expressed by apparently faceted spurs, offset drainage, and linear swales and gullies. Based on the above discussion, and review of HSE's report (1987), GSI concurs with the recommended structural setbacks. It is GSI's opinion, however, that the setback zone associated with the Elsinore fault zone probably does not bend as indicated by HSE (1987), as the location of the fault is constrained at only one location (HSE, 1987). The net result, however, is that the setback zone is wider and thus, more conservative. Ground rupture within the site would most likely only occur with an earthquake along the Elsinore fault zone.

FAULT RUPTURE HAZARD EVALUATION

Background

As previously stated, the active Elsinore fault zone is located on the northern boundary of the property. The northeastern half of the property is included within an Alquist-Priolo Earthquake Fault Zone. The portion of the site that lies within the Alquist-Priolo Earthquake Fault Zone was investigated in the late 80's (HSE, 1987), and again in the mid 90's (GSI, 1995). The fault zone was located, setbacks were recommended, and those reports were submitted to and approved by the County of Riverside. In 2003, the State amended the northern boundary of the Alquist-Priolo Earthquake Fault Zone, extending it well offsite. This current study has been performed to further evaluate the Elsinore fault zone to the south, and does not duplicate the previous approved work within the Alquist-Priolo Earthquake Fault Zone.

Since GSI's initial work, the site has been incorporated into the City, and an additional 40 acres has been acquired by the Client for development and annexation into the City. The overall purpose of this report is to evaluate the mapped faults and lineaments that traverse the portions of the site that were not previously investigated. However, all previous reports for the site are included herein on compact disc (Appendix C [in pocket]), and this report is intended to be a stand-alone document regarding conclusions and recommendations in light of the proposed development and the potential presence of faulting onsite.

During this updated fault rupture hazard evaluation, we independently performed a photo-lineament analysis, and evaluated the structure, nature, location, occurrence, and recency of activity of the previously mapped faults on the site, based on exposures in newly excavated exploratory trenches. This report summarizes both the previous and current work, and provides reasonably conservative interpretations of the geology and recency of fault movement. The scope of our work has included a review of the referenced documents (see Appendix A), analysis of data, and preparation of this report. Unless specifically superceded herein, the conclusions and recommendations contained in the referenced reports by GSI (see Appendix A), remain pertinent and applicable, and should be appropriately implemented during planning, design, and construction.

Definitions

Inasmuch as portions of the site lie in an Alquist-Priolo Earthquake Fault Zone (Special Publication 42), some definitions of terms used herein are appropriate. Special Publication 42 (Hart and Bryant, 1997) states: "A *fault* is defined as a fracture or zone of closely associated fractures along which rocks on one side have been displaced with respect to those on the other side." Hart and Bryant (1997) indicate that: "an *active fault* is defined by the State Mining and Geology Board as one which has 'had surface displacement within Holocene time (about the last 11,000 years).'" Similarly, Neuendorf, et al. (2005) defines a fault as: "A fracture or a zone of fractures along which there has been displacement of the sides relative to one another parallel to the fracture." Accordingly, the key criteria for determining whether a feature is a fault, is the existence of displacement. Fractures (including joints) with no displacement, do not meet the criteria to be classified as faults.

Further, for the purposes of the State, faults should be "sufficiently active" and "well-defined" (Hart and Bryant, 1997), generally displaying geomorphic features indicative of active faulting. As summarized by Hart and Bryant (1997), "the more recent the faulting, the greater the probability for future faulting." The State also notes that, "A fault is deemed sufficiently active if there is evidence of Holocene surface displacement along one or more of its segments or branches....." and "A fault is considered well-defined if its trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface." Surface is defined by Neuendorf, et al. (2005) as: "... top of the ground...." The critical consideration by the State is that the fault, or some part of it, can be located in the field with sufficient precision and confidence to indicate that the required site-specific investigations would meet with some success.

Literature Research and Photolineament Analysis

In order to identify possible unmapped faults and to evaluate topographic expressions of published or previously reported fault traces, an independent photolineament analysis was performed by GSI. Stereoscopic "false-color" infrared aerial photographs at a scale of approximately 1:40,000, and black and white aerial photographs at a scale of 1:24,000 and 1:19,200, were utilized in the lineament analysis. The moderate photo-lineaments observed generally correlate with faults with moderate photolineaments suggested by Weber (1977). We understand the map by Weber (1977) is the general basis for the inclusion of the site in the County of Riverside's "County Fault Zone."

Lineaments were generally classified as strong, moderate, or weak. A strong lineament is a well-defined features which can be continuously traced several hundred feet to a few thousand feet. A moderate lineament is less well-defined, somewhat discontinuous, and can be traced for only a few hundred feet. A weak lineament is discontinuous, poorly defined, and can be traced for a few hundred feet or less. Our lineament analysis was performed independently of the previously mapped faults, and concluded that moderately-developed northwest to westerly trending lineaments generally coincided with the previously mapped faults/lineaments by Weber (1977), as indicated on Figure 4.

In summary, our analysis of aerial photographs of the site revealed six moderately-aligned linear valleys traversing the site, and one possibly offset ridge. Thus, our fault-finding trenches were emplaced to intercept these features.

General Site Geology

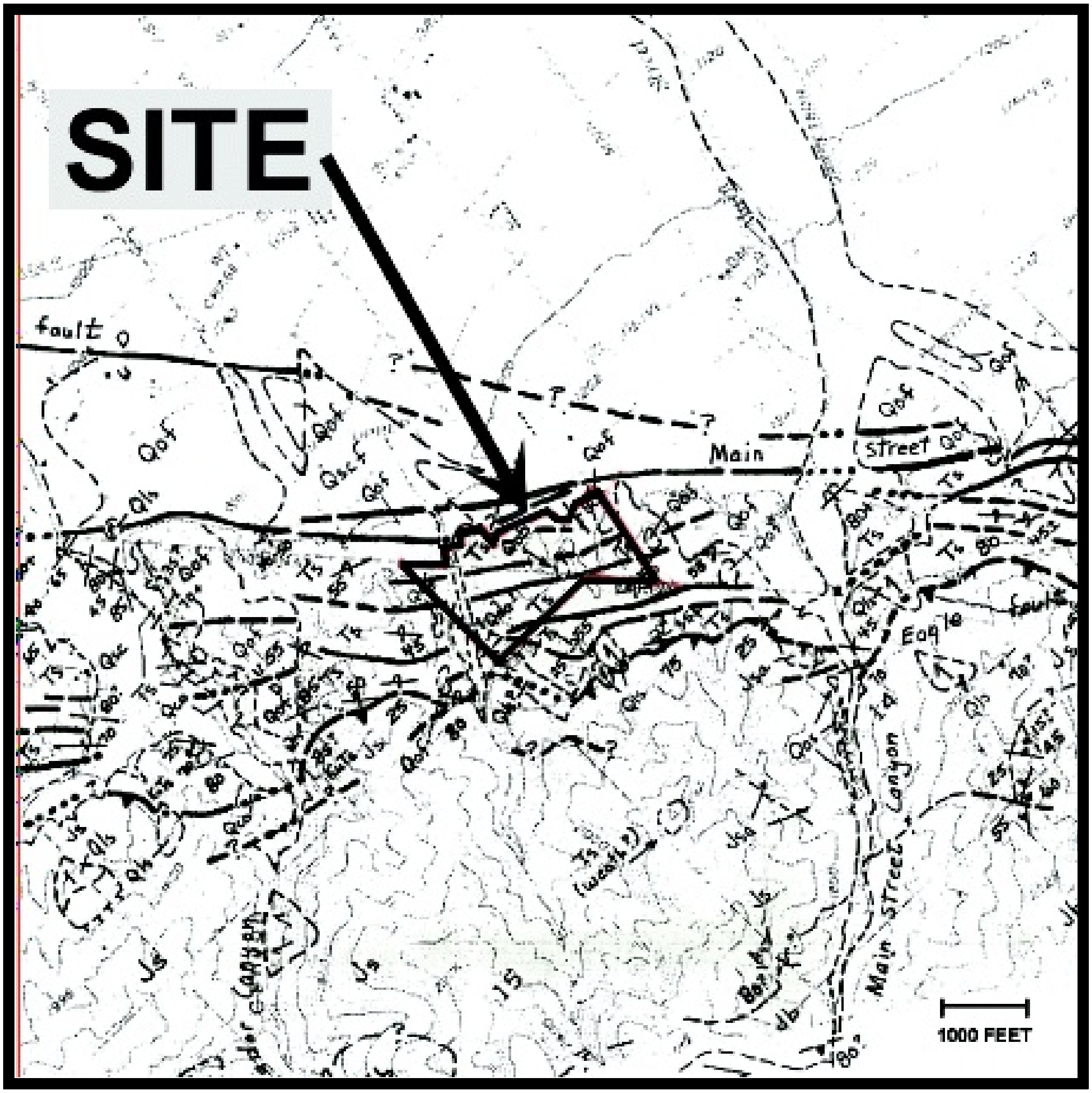
Three basic deposits were identified in our fault-locating trenches: 1) Quaternary (Holocene)-age alluvium; 2) Quaternary-age colluvial deposits (not mapped owing to their generally thin nature); and 3) the Tertiary-age (reported as [Paleocene]), Silverado Formation. The limits of mappable units encountered in GSI (this study) are shown on Plate 1, which is modified as appropriate, from GSI (1995), based on the recently obtained data. The major geologic units encountered in the fault trenches are discussed below, while complete descriptions of all units encountered onsite have been discussed previously in this report. Supplemental descriptions for the trench units are shown on Plates 2 through 5.

Quaternary-age Alluvium (Map Symbol - Qal)

Alluvium was encountered overlying the Silverado Formation in FT-6. The alluvium generally consisted of unbroken, fine- to coarse-grained silty sands, with gravel and cobbles. Three, and locally four, distinct, climatically-controlled fining-upward sedimentary sequences were observed in FT-6. Each of these fining-upward sequences was capped by pedogenic A/C soil profiles. Excluding the time of sediment deposition, each such A/C profile typically takes 250 to 500 years to form in semi-arid climates, such as the subject site (R. Shlemon, personal communication). Thus, the entire alluvial section in this area is minimally 1,000 to 2,000 years old, and perhaps much older. Accordingly, the age of this alluvial unit is estimated to be late Holocene. The late Holocene alluvium was not offset or displaced by faulting in this area.

Quaternary-age Colluvium (Not Mapped)

Colluvium was encountered overlying the Silverado Formation in FT-2. The colluvium consisted of fine- to medium-grained silty sand with trace gravel. A buried pedogenic Bt soil horizon exhibiting a blocky, columnar structure with weakly-developed peds was observed within a portion of the colluvium. This buried Bt soil horizon is similar in



Base Map From: Weber, H.F., Jr., 1977, Seismic hazards related to geologic factors, Elsinore and Chino fault zones, northwestern Riverside County, California, California Division of Mines and Geology Open-File Report 77-4.

LOCATION AND SCALES APPROXIMATE

GeoSoils, Inc.	W.O. 5166-A-SC
SITE LOCATION MAP	
Figure 4	

N

description to a Bt horizon (Q5) found in Temescal Valley by Millman and Rockwell (1986) which, given the climate, average annual rainfall, parent rock material, topography, etc., would minimally indicate an age of deposition of as much as 15,000 to 20,000 years. The paleosol was unbroken and unfaulted.

Tertiary (Paleocene)-age Silverado Formation (Not Mapped)

Regionally, the Silverado Formation consists of a sequence of both marine and non-marine sediments, with a known total stratigraphic thickness of about 1,750 to perhaps 4,000 feet, with the lower non-marine portion approaching 500 feet thick (Jahns, 1954; Gray, 1961; Leyva, 2002). Onsite, the formation consists of siltstone, sandstone, shale, and conglomerate, with some isolated paleosol development within the parental rocks, all with localized crenulations, folding, and small intra-formational ancient faults (with little offset), associated with the uplift and folding of this formation. The sediments exposed onsite are apparently near the lower portion of the stratigraphic section, marking the transition from the Paleocene to Cretaceous. This correlates well with the basal part of the Silverado Formation, which is primarily non-marine in the lower portion, and would have a thickness capable of encompassing the entire relief of the site, especially considering apparent thickness as a result of tilting or folding.

The lower member of the Silverado Formation is characterized by reddish-brown to greenish-buff clayey sandstone, siltstone, or claystone, and cobble conglomerate (Gray, 1961). The larger clasts known to occur in the non-marine section of the Silverado Formation include generally sub-rounded metamorphic, volcanic, and granitic rocks, sometimes set in a reddish-buff to gray arkosic grit, that is well indurated, and may contain concretions (Gray, 1961). Gray also indicates that the basal part of the lower member of the Silverado Formation is cobble conglomerate. Schoellhamer, et al. (1981) indicates that the base of the Silverado Formation is comprised of pebbles, cobbles, and boulders, in an arkosic sandstone matrix. Colburn and Ramierz (1995), indicate portions of the base of the Silverado Formation are structureless. Further, Maher (1982) points out that the base of the Silverado Formation is a sub-angular non-marine conglomerate that contains abundant detritus derived from both upper Cretaceous strata and rocks of the basement complex. Such clasts with those properties and concretions occur in the onsite sediments. Also of particular interest (discussed later), Gray (1961), additionally notes that reddish brown discontinuous clayey deposits tend to be near the base of the Silverado Formation, and that the clayey beds or deposits generally have little continuity and are not necessarily connected to one another, opining that this may be a result of the clayey beds/deposits being repeated by folding and faulting, or are intermittent, on both horizontal and vertical scales, such as that observed onsite.

Leyva (2002), documents the occurrence of several paleosols developed within the Silverado Formation, some of which are nearby commercial sources of clay products (Pacific Clay Products, Inc., in the Alberhill mining district). These paleosols, which are both discontinuous and relatively continuous, occur throughout the Silverado Formation, both in the lower and upper portions of the stratigraphic section. As indicated in Maher (1982), southward from the Corona area, the Silverado Formation rests on progressively older upper Cretaceous rocks from southwest to northeast. This onlap relationship suggest tilting and exposure of late Cretaceous rocks prior to the transgression and regression of Paleocene seas (Maher, 1982). The paleosols are interpreted as residual soils that are a result of tropical to subtropical climate (50 to 60 inches of annual rain, with short dry seasons), with high precipitation rates, and developed through extensive chemical weathering of the parental material. Such tropical to subtropical climate was not present in the Quaternary in this area. Leyva (2002), also points out the occurrence of homoclinal folds associated with fault blocks within the Silverado Formation.

Geologic/Seismic Evaluation

As recommended by the State (Hart and Bryant, 1997), GSI has evaluated the site on scales ranging from regional to site-specific, a procedure commonly used in assessments of fault activity (McCalpin, 1996). Without such analysis, this property would be very difficult to evaluate for active (Holocene) faulting, due to the general lack of stable geomorphic surfaces for soil-stratigraphy age-dating.

Strike-slip faults tend to concentrate deformation along a single linear strand, that may extend for tens or hundreds of kilometers with only minor changes in strike (Weldon, et al.,1996). Weldon, et al. (1996) also point out that active strike slip faulting produces a characteristic assemblage of landforms, including linear valleys, offset or deflected streams, shutter ridges, sag pond, pressure ridges, benches, scarps, and small horsts and grabens. Strike slip faults also transport nontectonic landforms laterally (i.e., fluvial terraces, stream channels, and alluvial fans), while the erosional and depositional processes forming them continue. Of the above features, only short linear valleys were observed outside of the approved setback zone. Other such typical features expected to be associated with active strike slip faulting were not observed outside of the setback zone.

Lamar and Rockwell (1986) documents more or less continuous, or at least episodic, displacement on the Elsinore fault zone during late Miocene, Pliocene, and Quaternary time, with most of the right-lateral movement indicated in Quaternary time. Thus, if the observed faults (discussed further below) were part of the current tectonic environment, a predominance of the landforms associated with active right-lateral strike slip faults should be well developed. In fact, on the subject site, outside of the previously approved setback zone, there is a distinct lack of landforms suggestive of right-lateral movement. As pointed out by Weldon, McCalpin, and Rockwell (1996), paleoearthquakes are typically recognized in exposures of strike slip faults from several general types of evidence: 1) upward termination of fault displacement; 2) abrupt changes in vertical separation of strata as faults are traced upsection or downsection; 3) abrupt changes in thickness of strata or of facies across the fault; 4) fissures and sand blows in the stratigraphic sequence; 5) angular unconformities produced by folding and tilting; and 6) colluvial wedges shed from small scarps. Such features were generally not observed by GSI in our supplemental trenching, with the exception of older bedrock faults in Trenches FT-1, FT-5, and FT-6, where angular unconformities were observed, and are discussed further herein.

Field Studies

Field studies conducted during our supplemental faulting investigation consisted of the following:

1. Supplemental geologic reconnaissance mapping and geomorphic assessment.

2. Excavation of six fault-locating trenches and one dozer cut generally on the previously mapped faults and/or their projection, coincident with GSI's lineaments, discussed earlier. These trenches and dozer cut were excavated to evaluate near surface soil and geologic conditions with respect to faulting and geologic structure.

The fault-locating trenches and dozer cut were logged by geologists from our firm. These trenches and dozer cut were excavated by a trackhoe, and in total, were about 1,100 feet in length. Although the County geologist was invited to view the trenches, the invitation was declined. The trenches have now been backfilled with native materials, and track walked. The location of previous and current explorations are shown on the Geologic Map, Plate 1. The logs of the GSI trenches are included as Plate 2 (Trench Logs FT-1 and FT-2), Plate 3 (Trench Log FT-3), Plate 4 (Trench Logs FT-4 and FT-5), and Plate 5 (FT-6 and Dozer Cut Log DC-1).

Summary of GSI Trench Data

FT-1

Trench FT-1 was placed in a saddle generally coincident with the projection along strike of a fault/lineament mapped by Weber (1977), and with Photolineament No. 1 from GSI's independent lineament analysis, along the southerly margin of the APEFZ boundary. This trench exposed sandstones, siltstones, claystones, and conglomerate of the Silverado Formation, replete with discontinuous argillic paleosols within the formation. The formational sediments were broadly to tightly folded, and comprise the ridgeline. Here, the bedding was generally well developed, but was also locally folded. Two features were noted within this trench, one near Station 80, and one near Station 110.

The feature exposed near Station 80 appeared to be a normal fault as exhibited by an approximately 2½ feet of offset in a clay bed, indicating the sense of relative movement (down to the southwest), marking an extensional tectonic environment (unlike the modern right-lateral tectonic environment), when faulting occurred. The fault was observed as trending east-west on strike, which is not consistent with the current tectonic regime of faults trending at around N30° to N45°W, and indicates that this fault is not a part of the active fault zone. Additionally, with the fault exhibiting a large amount of vertical displacement (about 2½ feet), a residual fault scarp should be readily apparent if the fault were to be active in the Holocene. However, no fault scarp was observed at the surface. Furthermore, coincident with the fault, the bedding appeared locally to be more tightly folded, and folded shear planes within the bedding were also observed, which indicates that faulting occurred in conjunction with the folding deformation of the Silverado Formation, and is not related to Holocene strike-slip faulting.

The feature exposed near Station 110 appeared to exhibited movement along the bedding plane between a paleosol and sandstone unit, although the direction of movement could not be discerned due to the pulverized nature of the paleosol. The paleosol consisted of an argillic (Bt) horizon, on which a Stage III calcic horizon had also been superimposed, attesting to formation under a vastly different paleoclimate. A small tight fold immediately adjacent to the fault is positive evidence that the movement is related to the folding and deformation of the Silverado Formation, and is not part of the current tectonic environment. The proximity of the other argillic paleosols, and their association with folding within the formation again indicate that the movement occurred during the deformation of the Silverado Formation, and is hence, pre-Holocene. Further, the complete mis-match of sediments on both sides of the feature indicate a large amount of throw, which should be readily apparent in a residual fault scarp, if the fault were to be active in the Holocene. Such was not the case.

FT-2

Trench FT-2 was also placed coincident with a fault/lineament mapped by Weber (1977), and GSI Photolineament No. 2 associated with a short linear valley/drainage, along the southern margin of the site. This trench generally exposed conglomerate and sandstone of the Silverado Formation partially overlain by undisturbed colluvium. The Silverado Formation exhibited intensely-deformed and folded bedding. The conglomerate was poorly developed to massive/structureless, with sub-angular to sub-rounded pebble- to boulder-size clasts, in a sandy arkosic matrix, that was also moderate to strongly indurated, and locally contained concretions (perhaps as rip-up clasts). The sandstone consisted of a sandy arkose and was poorly to well developed. Discontinuous paleosols, conforming with the folding, were again observed. It is also possible that the indurated condition is due to ancient ferricrete or petroferric paleosol development, and partial paleo-weathering/erosion, prior to tilting/uplift. Faults were not apparent within this trench, and the modern colluvium was not offset.

FT-3

Trench FT-3 was placed coincident with another moderately-aligned GSI Photolineament No. 3, associated with a short linear valley and a possible deflected ridge on the western margin of the property, as well as a fault mapped by Weber (1977). However, as was observed during site mapping, the possibly deflected ridge is apparently a landslide with an arcuate head scarp covered by denser vegetation. Sediments exposed included folded sandstones, siltstones, claystones, and conglomerate of the Silverado Formation overlain by unbroken, relatively flat-lying colluvium. Discontinuous paleosols, conforming with the folding, were also present. Faulting was not observed within this trench.

FT-4

Trench FT-4 was again coincident with a moderate GSI lineament (Photolineament No. 4) and a fault/lineament mapped by Weber (1977). This trench again exposed sandstones and siltstones of the Silverado Formation, with discontinuous argillic paleosols within the formation. The formational sediments were tilted and folded, and folding (peaks and valleys) did not correspond to the current topography. No faults were observed within the trench.

FT-5

Trench FT-5 was sited to intersect a moderately-developed Photolineament No. 5, as well as two faults/lineaments mapped by Weber (1977) that converge in this area. Formational materials exposed included sandstones, siltstones, and discontinuous argillic paleosols. Two possible older fault-related features were observed at the western end of the trench, at approximately Stations 5 and 25.

The feature exposed at Station 5 trended N50°W and was dipping to the northeast at 45 degrees. It was characterized by a prominent linear fracture, with severely-fractured sandstone on the east side of the feature, and relatively unfractured sandstone and an associated paleosol on the west side. No displacement or slickensides were observed on the fracture. The fracture could be traced to the top of the Silverado Formation, which was not capped by any other sediments in this area. There was no evidence of a fault scarp, nor any other evidence of recent activity on the ground surface. Since the age of last activity, nor a sense of movement could not be determined, the feature was projected on trend to the northwest to an area where younger sediments were known to exist, and FT-6 was thus sited to intercept this fault.

The feature at Station 25 trended N10°W, dipped 39 degrees to the northeast, and was characterized by an angular unconformity that exhibited severely-fractured sandstone with arcuate chunks of unfractured sandstone on the east side, which was apparently thrusted over a pulverized siltstone on the west side. This fault could be traced to within approximately 2 feet of the surface, but not to the top of the exposed Silverado Formation. Again, there was no evidence of a fault scarp at the surface. Therefore, it was concluded that this was a bedrock fault, and not active in Holocene time.

FT-6

As stated above, FT-6 was sited to intercept a feature that was uncovered at the western end of FT-5, in order to expose the fault in the vicinity of younger soils to evaluate the recency of activity. FT-6 was extended to the southwest in order to also intercept other possible features traversing the southwestern corner of the property. The trench exposed Silverado Formation overlain by a thin layer of Holocene-age alluvium, locally overlain by a 1- to 2-foot layer of undocumented fill and surficial colluvium.

At approximately Station 23, a fracture was exposed along strike and with the same orientation as the feature found in FT-5 at Station 5, as well as the same orientation of the adjacent bedding. Again, no displacement or slickensides were observed on the fracture. The fracture appeared to be along the contact between a sandstone unit on the east side and highly-fractured siltstone and a paleosol on the west side. As in FT-5, the fracture propagated to the top of the Silverado Formation, but in this case was overlain by approximately 6 feet of unfaulted alluvium, and about a foot of surficial colluvium. Again, there was no evidence of a fault scarp, nor any other evidence of recent activity at the Silverado Formation/alluvium contact. The fact that the orientation of the fracture was the same as the adjacent bedding is probably not coincidental. Most likely, any movement that occurred along this fracture was the result of bedding plane failure during the deformation of the Silverado Formation, and thus pre-Holocene.

At around Station 92, a bedrock fault was observed trending N60°W and was vertical. It appeared to offset a bed containing dark ferrous minerals by approximately one foot. However, when traced across to the opposite wall, no offset was observed. The fault did not extend into the alluvial soils above, nor was a fault scarp noted at the top of the bedrock.

At approximately Station 100, another bedrock fault was observed which again offset beds in the Silverado Formation. Trending at N54°W, with a vertical dip, this fault displaced a highly-fractured siltstone bed within the Silverado Formation by at least 6 inches. This fault was confined to within the bedrock unit, and no fault scarp was present.

Furthermore, the upper terminus of each of these bedrock faults/fractures was overlain by a minimum 1,000 to 2,000 years of alluvial sedimentation, as estimated from the three to four fining-upward sequences and corresponding pedogenic A/C profiles observed within the alluvium. Each one of the A/C profiles typically takes 250 to 500 years to form (R. Shlemon, personal communication). Considering that the Elsinore fault zone has a preferred ground-breaking earthquake recurrence interval of about 250 years, there should be evidence of at least four, and as many as eight or more earthquake events within the alluvium, if these faults were active during Holocene time. No such displacement of alluvial soils was observed. Accordingly, it is reasonably judged that any faulting is pre-Holocene.

DC-1

Dozer Cut DC-1 was placed coincident to a moderately-aligned GSI lineament (Photolineament No. 6) and a deeply-incised short valley immediately north of FT-5. The lithology exposed in the cut consisted of tilted, weathered, and fractured sandstone of the

Silverado Formation, with a thin cap of colluvium/topsoil. No faults or fault related features were observed in the dozer cut.

Discussion of Findings related to Faulting

As indicated previously, paleoearthquakes are typically recognized in exposures of strike slip faults from several general types of evidence: 1) upward termination of fault displacement; 2) abrupt changes in vertical separation of strata as faults are traced upsection or downsection; 3) abrupt changes in thickness of strata or of facies across the fault; 4) fissures and sand blows in the stratigraphic sequence; 5) angular unconformities produced by folding and tilting; and 6) colluvial wedges shed from small scarps. With three exceptions, these features were not observed, across the entire site. The bedrock faults in Trenches FT-1, FT-5, and FT-6 mark an angular unconformity; however, these faults do not flower near the surface, attesting to faulting at depth, and then subsequent uplift/erosion to its present geomorphic environment, also indicating antiquity. Additionally, utilizing the inclination of the slope developed above the fault as an indication of age (Wallace, 1977), the local slope inclination along ridgelines/divides above the fault was on the order of about 5 degrees, or less, indicating an age well in excess of 100,000 years. Further, the encountered bedrock faults were observed along folded bedding planes, and it is GSI's opinion that these faults represent flexural slip, or failures in the bedding planes as a result of deformation and folding in the Silverado Formation. Finally, the bedrock faults/fractures observed in FT-6 were overlain by an estimated minimum of 1,000 to 2,000 years of alluvial deposition. With a preferred recurrence interval of about 250 years on the Elsinore fault zone, evidence of at least four to eight earthquake events should be present within the alluvium if these faults/fractures were active during Holocene time. However, the alluvium was observed to be unfaulted and undisturbed. Thus, using these lines of evidence, the faults exposed in FT-1, FT-5 and FT-6 are all likely pre-Holocene.

Faulting Conclusions

Faults and lineaments located during the course of this study are concluded to be coincident with ancient deformation and folding of the Silverado Formation, and therefore, pre-Holocene in age. Several lines of evidence supporting this conclusion are presented herein, including; a lack of geomorphic features and landforms indicating active faulting in the current right-lateral tectonic environment; generally opposite sense of relative movement on the faults with respect to current relief; lack of shallow flower structures; localized divide/ridgeline or residual slopes over faults with gradients of less than about 5 degrees; presence of discontinuous paleosols similar to that indicated by Gray (1961), as well as degree of induration (burial and diagenesis and/or ferricrete/petroferric paleosol development), and hence Paleocene-age; age and complex geologic history (tilting, folding, and associated faulting) of the Silverado Formation under vastly different tectonic regimes; fault plane orientation coincident with bedding plane orientation, indicating flexural slip during deformation; and, absence of fault activity in alluvial deposits estimated to be at least 1,000 to 2,000 years old, with a preferred recurrence interval of about 250 years for the Elsinore fault zone. These lines of evidence all attest to the reasonably conservative interpretation of the pre-Holocene nature of faulting in the portion of the site under the purview of this report. Such ancient faulting would be expected for sediments with the long geologic history of the Silverado Formation.

Further, based on all of the above, and in conjunction with the regional geologic setting, GSI reasonably concludes that the presence of active faults on the site, outside of the previously approved setback zone, is not likely, and does not fit the extensive geologic/geomorphic evidence gleaned from the subsurface investigations of the site. It also bears repeating that the portion of the site under the purview of this investigation is not currently included in the state-designated fault zone. In contrast, the approved setback zone along the northern boundary of the subject site *does* concurrently exhibit youthful Quaternary features characteristic of active strike slip faults, and *does* lie within such a state-designated zone.

UPDATED SITE SEISMICITY

General

During a 50-year span, a structure on the site will likely be subjected to an earthquake of at least Richter magnitude of 6.0. Horizontal acceleration induced by an earthquake may affect earth structures and/or embankments. Experience has shown that wood-frame structures designed in accordance with the Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 1997 and 2001), generally survive earthquake effects. Earthquake effects may include lurching and/or localized ground cracking. This would be expected over other portions of southern California as a whole.

Ground lurching or shallow ground rupture due to shaking could occur within the site, as well as most of the Corona area, from an earthquake either originating on or along the Elsinore or Chino faults (to the north of the site) or the other nearby faults. Such lurching could possibly cause cracking of paved areas, with anticipated limited damage to foundations if design recommendations provided herein are followed.

Earthquake-induced slope stability problems may also occur within the site It is our professional opinion that these instability problems (e.g., landslides) would most likely occur where unsupported bedding planes or bedrock is highly fractured or contorted (e.g., near the strands of the Elsinore fault). Therefore, all proposed cut or natural slopes containing unsupported bedding planes within the proposed development area (or which could affect the development area) may likely require stabilization fills. This should be further evaluated based on the conditions disclosed during grading.

Seismicity

The acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999), Campbell and Bozorgnia (1997 Revised), and Sadigh, et al. (1997) have been incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources. For this study, peak horizontal ground accelerations anticipated at the site were determined based on the random mean plus 1 sigma attenuation curve developed by those authors.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is computed by one of many user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, peak horizontal ground accelerations from an upper bound event at the site may be on the order of 0.967g to 1.30g. The computer printouts of portions of the EQFAULT program are included within Appendix D.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, Campbell, Niazi (1999) and the computer program EQSEARCH (Blake, 2000b). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-mile radius, between the years 1800 to December 2005. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 to December 2005 was 0.32g. In addition, a seismic recurrence curve is also estimated/generated from the historical data (see Appendix D).

A probabilistic seismic hazards analyses was performed using FRISKSP (Blake, 2000c) which models earthquake sources as 3-D planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of this data, and considering the relative seismic activity of the southern California region, a probalistic peak horizontal site acceleration (PHSA) on the order of 0.87g was determined for the site. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period). Computer printouts of the FRISKSP program are included in Appendix D.

Seismic Deformation and Densification

As previously discussed (GSI, 1995a), the site earth materials do not appear to be susceptible to liquefaction and as such, this phenomenon is not considered significant. Future fills are planned to be densified at 90 percent, or more, relative compaction (per ASTM D-1557), subdrained, keyed, and benched into approved bedrock contacts.

Furthermore, loose surficial soils are to be remediated prior to fill placement and subdrained as necessary. No significant liquefaction potential should remain following that planned earthwork and remedial grading. However, due to the granular nature of the site earth materials, fills in the upper 50 feet of the soil profile (excluding dense formational materials) may be susceptible to seismic volumetric strain or seismic densification when subjected to the design level seismic loading. These artificial fills are anticipated to exhibit vertical deformation (settle) on the order of ¼ percent, or approximately up to 1.5 to 3 inches for planned fills overlying dense formational materials. Fills compacted to 95 percent (ASTM D-1557) relative compaction , contain fines in excess of 35 percent, are not anticipated to be susceptible nor contribute to this seismic induced vertical deformation. Lots that are constructed with 25 feet of differential fill thickness or less (minimum/maximum fill thickness), a seismic differential settlement across the lot should be on the order of ¾ inch or less. Lateral deformation at the top of fill slopes subjected to the design seismic event, may be on the order of several inches within the UBC/CBC (ICBO, 1997 and 2001) setback zone.

Seismic Shaking Parameters

Based on the site conditions, Chapter 16 of the UBC/CBC (ICBO, 1997 and 2001), the following seismic parameters are provided. The Chino-Central Avenue (Elsinore) fault is the design earthquake fault for the subject tract, located about 0.5 miles (0.8 km) north of the site.

CHAPTER 16 OF UBC	SEISMIC PARAMETERS
Seismic zone (per Figure 16-2)	4
Seismic zone factor (per Table 16-I)	0.40
Soil Profile Types (per Table 16-J)	S_D
Seismic Coefficient C_a (per Table 16-Q)	$0.44N_a$
Seismic Coefficient C_v (per Table 16-R)	$0.64N_v$
Near Source Factor N_a (per Table 16-S)	1.46
Near Source Factor N_v (per Table 16-T)	1.9
Seismic Source Type (per Table 16-U)	B
Distance to Seismic Source	0.0 mi. (0.0 km)
Upper Bound Earthquake (Chino-Central Ave)	M_W 6.7

GROUNDWATER

Groundwater was encountered as seepage during our recent subsurface investigation on the site in the lower elevations at depths ranging from 6 to 12 feet below existing ground surface. This seepage is likely "perched" groundwater, where the water is contained in more permeable sediments overlying relatively impermeable sediments. No groundwater was encountered in our four recent borings onsite (this report). In addition, HSE (1988) did not encounter groundwater to a depth of at least 30½ feet below the existing grade. Further, Envicom (1976) indicates that the depth to the regional water table is greater than 50 feet from existing grade. HSE (1988) indicated that the subsurface water bearing alluvial deposits ranging in thickness from 200 to 1,000 feet, and are underlain by sedimentary bedrock. They also reported that the older sedimentary rock is essentially non-water bearing and may act as a partial retarder to groundwater flow within the aquifer system.

Wells in the vicinity are apparently producing from various perched aquifers throughout the alluvium. A review of available data indicates that the regional groundwater is at an elevation of about 700 feet MSL; and the gradient is generally down toward the north-northeast, and generally parallels the regional topography (California Division of Mines and Geology, 1960).

It is anticipated that subdrainage may be necessary in canyon/swale areas; however, it will be necessary in any stabilization backcuts. The need for subdrainage should be further evaluated when project grading plans are finalized, and during project earthwork.

These observations reflect site conditions at the time of our investigation and do not preclude changes in local groundwater conditions in the future from heavy irrigation, precipitation, or other factors not obvious at the time of our field work. It should be noted; however, that groundwater may occur in the alluvium and fan deposits, formational sediments, bedrock, or along fractures and joints, due to migration from adjacent developments and/or during and after periods of above normal or heavy precipitation, and should be anticipated during and after the site is developed. This potential will need to be disclosed to all homeowners, any homeowners association, and all interested/affected parties. Should perched groundwater conditions occur, GSI should be contacted to provide recommendations for mitigation. Groundwater conditions will also be further evaluated during site grading. Additional discussions of groundwater are presented within the conclusions section of this report.

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System. The soil classifications are shown on the Test Pit and Hollow Stem Boring Logs (see Appendix B), and the Laboratory Test Results are presented in Appendix E.

Moisture Density

The field moisture contents and dry unit weights were determined for undisturbed ring samples for the soils encountered in the exploratory test pits and borings. The dry unit weight was determined in pounds per cubic foot (pcf) and the field moisture content was determined as a percentage of the dry unit weight. The results of these tests are shown on the Test Pit and Hollow Stem Boring Logs (Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was determined for the major soil types encountered in the exploratory test pits and borings. The laboratory standard used was ASTM D-1557. The moisture-density relationship obtained for the site soils are shown below:

SOIL TYPE	LOCATION AND DEPTH (FT)	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
SANDY SILTSTONE, yellowish brown (Silverado Formation)	GTP-8 @ 1-2 (GSI, 1995a)	120.0	14.0
SILTY SANDSTONE, yellowish brown (Silverado Formation)	GTP-8 @ 2 (GSI, 1995a)	126.0	11.5
Interbedded SILTY CLAYSTONE and SANDY SILTSTONE, olive brown to reddish brown (Silverado Formation)	GTP-15 @ 2 (GSI, 1995a)	116.0	17.0
SILTY SANDSTONE, pale yellow (Silverado Formation)	GTP-16 @ 2 (GSI, 1995a)	123.0	11.0
Interbedded SILTSTONE and CLAYSTONE (Silverado Formation)	B-1 @ 30-35	118.0	14.0
CLAYEY SAND, dark brown (Alluvium)	TP-1 @ 0-5	124.0	11.5
CLAYEY SAND, light yellow brown (Landslide Deposits)	TP-4 @ 0-5	124.0	11.5
SILTY SANDSTONE, light yellow brown (Silverado Formation)	TP-8 @ 0-5	120.0	13.0
SILTY SANDSTONE, light yellow brown (Silverado Formation)	TP-10 @ 0-5	126.0	10.0
SILTY SANDTONE, reddish brown (Silverado Formation)	TP-11 @ 0-5	119.0	14.0

Expansion Potential

Expansion Index (E.I.) testing was performed on a representative sample of site earth materials in general accordance with Table 18-I-B of the UBC (ICBO, 1997). The results of expansion testing are presented in the following table.

Multi-Stage Unconsolidated Undrained Triaxial Test

Testing was performed on an undisturbed soil samples of Silverado Formation in Borings B-3 and B-5 to evaluate the undrained shear strength of various bedrock components (i.e., claystone/siltstone) in general accordance with ASTM D-2850. The test results are presented in Appendix E. GSI used a staged triaxial program to evaluate the multiple undrained strengths from the same samples. The samples were repeatedly subjected to increasing confining pressure and an axial re-load was applied three times for each of the two samples tested.

SLOPE STABILITY DISCUSSION

As indicated by GSI (1995a), and this study (see Appendix F for Supplemental Slope Stability Analyses, using the cross-sections shown on Plate 6), on a preliminary basis from a geotechnical perspective, all proposed slopes should be grossly and surficially stable to heights proposed, provided our recommendations are properly implemented, and under normal rainfall conditions. Final evaluations will be made in the field during grading operations. Due to the naturally steep inclination (generally deeply incised with some oversteepening near the base) of the existing canyon walls, the out-of-slope bedding in the folded Silverado Formation, and the relatively cohesionless nature of some surficial materials mantling existing slopes, along with their susceptibility to erosion (our estimated average surficial erosion rate of less than about 1.25 inches a year), some slopes should be stabilized. In addition, GSI has noted on the site plans other stabilization that will need to be incorporated into the project grading. GSI has indicated a stabilization fill with keyways (buttress stabilization).

Slope stabilization buttress designs are depicted in the Appendix F. Given the 100-scale level of design and the alternatives that would affect the depth or width of the selected buttress designs, GSI has not indicated these on Plate 6. The civil consultant should review the depth of the buttress selected and evaluate the elevation that the buttress subdrains may gravity flow to a suitable outlet.

In evaluating Cross-Section A-A', GSI used a select fill soil with a cohesion and internal friction angle of 250 psf and 30 degrees, respectively. This was necessary to achieve the minimum acceptable factor-of-safety (FOS). The use of this higher friction material may necessitate: 1) selective grading onsite; and 2) the use of surficial slope retention methods (biotechnical or geogrid) to reduce the potential for erosion and surficial instability.

GSI has used a series of grid layers on the buttress for Cross-Section B-B'. The purpose of these grids (long-term design strength [LTDS] of 3,500 pounds similar to a Mirafi 10XT, or equivalent) is to stabilize the outer most area of the buttress should an offsite slide occur. GSI has utilized a reasonably conservative assumption of an 8-foot high scarp/slump in the area in front of the buttress to model a potential downslope surficial slope failure. Given this level of surficial slide the global stability within and around the buttress was evaluated. Based on these analyses, a slump may occur and the planned slope will remain within the guidelines for FOS both static and seismic until a slope restoration project below this project (downslope) may be performed. As designed, these grids are from an elevation 10 feet from the bottom of the keyway, and extend from the forecut (key front cut) to approximately 30 feet into the key (see Appendix F).

Based on the unresolved nature of the existing home site on Cross-Section D-D', GSI did not design a buttress in this area. Rather, we elected to evaluate the existing condition of the slope from a gross stability standpoint. Although the static stability appears to meet the minimum FOS, the seismic stability did not (see Appendix F). GSI recommends that as this area be redesigned for either the retention of this property or its removal and regrade in accordance with the current plan. At that time, a buttress design solution will be provided. The surficial stability of the slope at the location of Cross-Section D-D' was not performed and is not implied from the surficial stability, as there is observable slope creep effects in this area of the project site.

Backcut analyses in the Silverado Formation indicated some potential difficulties and instabilities in Cross-Section A-A' where GSI selected an approximately 1.5:1 (h:v) backcut inclination. It was necessary to use an inclination of 33 degrees for this backcut in order to stabilize the hillside and remain within the property line(s) on either side of this section. This backcut would require a "slot" cutting program, leaving approximately 20 feet of in-place material in a maximum 50-foot wide "slots" during grading of the key/buttress and achieve a minimum acceptable FOS of 1.2, as with other backcuts on this project. Other backcut inclinations of 2:1 and 1.8:1 (h:v) were used for Cross-Sections B-B' and C-C' respectively. In both of these temporary backcut conditions, a minimum FOS of 1.2 was maintained.

The potential for mass wasting and mudflow debris should be properly mitigated in portions of the site. It is recommended that debris impact walls/catchment basins or other comparable mitigative devices (GSI, 1995a) be incorporated into the project design, at the discretion of the design civil engineer.

Subdrains are recommended within drainage/canyon areas where proposed fills exceed 10 feet in height, as well as in some abutting areas where the as-built fill thickness exceeds 10 feet. Additionally, subdrainage systems for the control of localized groundwater seepage should be anticipated following grading due to excess irrigation or precipitation.

SLOPE STABILITY

Conventional slope stability analyses were performed utilizing version number two of the computer program GSTABL7. The program performs a two-dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the simplified Bishop or

Janbu (Block) methods. Due to the nature of the bedrock, block-Janbu analyses were used. Representative geologic cross-sections were prepared for analysis, utilizing 100-scale rough grading plans, as well as field and laboratory data from our current study and referenced report(s). Maximum 2:1 (h:v) cut and fill slopes, or daylight natural slopes, as indicated on Cross-Sections A-A', B-B', C-C', D-D', E-E', and F-F' (see Plate 6). GSI did not perform slope stability analyses on Sections E-E' and F-F'. The results of the analyses for Cross-Sections A-A' though D-D' are included in Appendix F.

Gross Stability Analysis

The FOS for the slope buttress designs assume a water level that will need to be evaluated by the project civil designer. Should the buttress drains need to be raised, the saturated buttress designs will need to be re-evaluated.

Provided our recommendations are properly implemented, a calculated factor-of-safety greater than 1.5, or 1.1, has been obtained for the proposed, maximum proposed 2:1 (h:v) fill and cut slopes, when analyzed from a static or seismic viewpoint, respectively. The results of the analyses are included in Appendix F. Proposed cut slopes higher than 30 feet or into unfavorable geology may need to be laid back to a flatter angle (increased bench widths), or reduced in overall height based on the available data. Fill slopes have been analyzed and reviewed up to approximately 135 feet over stabilized bedrock and are acceptable from a geotechnical standpoint; however, if higher than 135 feet are proposed, they can be laid back (flatter angle).

Surficial Slope Stability

The surficial stability of the proposed slopes have been analyzed. Our evaluation indicates a calculated surficial safety factor greater than 1.5 for the maximum proposed fill, cut and stabilized slopes, provided our recommendations are properly implemented, under normal rainfall conditions.

Summary of Slope Stability

Based on our analyses, cut and fill slopes constructed at 2:1 (h:v) gradients should not exceed ±134 feet in height. Furthermore, close monitoring and inspection of all cut slopes will be required during grading to assess individual cut slopes to evaluate the presence or absence of adversely oriented geologic structures (i.e., bedding, fractures, etc.). While not anticipated, should such structures be identified during earthwork construction, remedial measures would be recommended at that time based on the conditions exposed.

Due to the size and construction limitations on the Cross-Section A-A' buttress, GSI recommends that the project civil designer and developer consider a pad elevation change (increase) of approximately 15 feet above the current pad (s). This will surcharge the toe of the planned slope in this area and significantly reduce the depth of the buttress. This alternative was presented in Appendix F as an alternative to the Cross-Section A-A' analyses. This may necessitate the use of an earth retention structure to achieve the same number of buildable pads.

The actual location of stabilization forecuts and backcuts should be provided by the design civil engineer. Subdrainage, and subdrainage outlets, should be reviewed and recommendations and/or locations should also be similarly provided by the design civil engineer.

GSI recommends a re-evaluation of the slopes around Cross-Sections D-D', E-E', and F-F; should the existing residential pad remain. This area is likely surficially unstable and may not be constructed to the minimum seismic slope stability FOS when considering the design seismic event unless unique slope stabilization engineering solutions are used.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our review and site reconnaissance, it is our opinion that the proposed site development as shown on the plans by A&B (2006) is generally feasible, from a geotechnical viewpoint. The approved setback zone has not changed. The recommendations presented herein by GSI should be incorporated into project planning, design, and construction. Recommendations for geotechnical foundation design, construction parameters, and development criteria are reiterated herein, and modified as appropriate.

RECOMMENDATIONS-EARTHWORK CONSTRUCTION

General

1. Prior to the start of the grading operation, the site should be cleaned of all vegetation (including roots), trash, construction and other deleterious materials.

2. Geologic observations should be performed during grading to verify and/or further evaluate geologic conditions. Although unlikely, if adverse geologic structures (i.e., ancient faults, existing fill, raveling sands, claystones exposed in slopes, etc.) are encountered, supplemental recommendations and earthwork may be warranted.

3. Within areas proposed for settlement-sensitive improvements, earthwork recommendations are contained in this report for removals of disturbed and/or unsuitable original natural ground.

4. Soil engineering observations and compaction testing services should be provided during any grading to aid the contractor in removing unsuitable soils and in his effort to moisture condition and compact the near-surface loose fill.

5. Our slope stability analyses indicate that, provided our recommendations are properly implemented, the maximum proposed fill, cut, and natural (daylight cut) slopes are grossly stable, to the heights specified previously, under normal rainfall conditions. However, close monitoring and observation of all cut slopes will be required during grading to assess and evaluate the presence or absence of adversely oriented formational or bedrock structures (i.e., bedding planes, joints, fractures, etc.). While not anticipated, should such structures be identified during earthwork construction, remedial measures would be recommended at that time based on the conditions exposed. Between planned earthwork and existing improvements to remain, GSI recommends slope monitoring (surface inclinometers) to reduce the potential for excessive temporary slope movement.

6. As per standards of practice, settlement monitoring will need to be conducted for engineered fill areas in excess of 50 feet in thickness. Settlement monitoring is estimated, at this time, to take place for a time period of approximately one to 15 months (for deeper fills), or possibly less, based on the data obtained. It should also be noted that current industry standards require basal fill materials below an engineered fill depth of 50 feet to be compacted to 95 percent of the laboratory standard.

7. Considering the noncohesive nature of some of the onsite material, some caving and sloughing may be expected to be a factor in subsurface excavations and trenching. This would be primarily associated with trenches excavated for utilities and foundation systems. Additional shoring or laying back excavations may be necessary to mitigate caving or sloughing. All trench excavations should conform to OSHA and local safety ordinances.

8. Onsite materials may be reused as compacted fill provided that major concentrations of vegetation and debris are removed prior to fill placement.

9. In fill areas where cavities or loose soils remain after surficial processing, the loose areas should be cleaned out, observed by the soil engineer, processed, and replaced with fill which has been moisture conditioned to <u>at least</u> optimum moisture content. The soils should be compacted to at least 90 percent of the laboratory standard.

Demolition/Grubbing

1. Any existing surficial/subsurface structures, major vegetation, and any miscellaneous debris should be removed from the areas of proposed grading.

2. The project soils engineer should be notified of any previous foundation, irrigation lines, cesspools, septic tanks, leach fields, or other subsurface structures that are uncovered during the recommended removals, so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils (including <u>all</u> previous exploratory test pits) remaining after demolition and site clearance should be cleaned out, inspected by the soils engineer, processed, and replaced with fill that has been moisture conditioned to <u>at least</u> optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D-1557).

Treatment of Existing Ground

1. Removal of all undocumented artificial fill, colluvium, alluvium, surficial landslide deposits, and generally near surface weathered Tertiary Silverado Formation materials will be necessary prior to fill placement, in areas proposed for development. GSI believe that most of the alluvium, and all of the colluvium and undocumented fill will be removed during remedial grading. However, for preliminary planning purposes, removal depths are estimated to be on the order of ±1 to ±12 feet, with locally deeper removals, in areas proposed for development. Generally, removals should extend to non-porous, competent materials (dry density of 105 pcf and/or 85 percent saturation [which has been previously demonstrated as acceptable mitigation]), be moisture conditioned, and recompacted if not removed by proposed excavation within areas proposed for settlement-sensitive improvements.

2. Where planned cuts are equal to or greater than the recommended removal depth, the area should be cut to grade, subgrade observed and tested by the geotechnical consultant, then the upper 12 inches below finish grade should be scarified, brought to at least optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard.

3. Where the planned cuts are less than the recommended removal depth, the additional removals to attain the recommended removal should be accomplished. The exposed removal surface should be scarified to a depth of 12 inches, moisture conditioned (if necessary), and then compacted prior to fill placement to finish pad grade.

4. Removed colluvium, alluvium, landslide deposits, and Tertiary Silverado Formation materials, may be reused as compacted fill <u>provided</u> that major concentrations of organic material (roots and tree remains), and miscellaneous trash and debris are removed prior to fill placement. Rock or earth particles of greater than 12 inches may be cleared from these soils or placed a depth as indicated in Appendix G. Due to the expansive nature of some of the Tertiary Silverado Formation materials, fill soils derived from this unit should <u>not</u> be placed closer than 7 feet from finish grade, on a preliminary basis.

5. Localized deeper removal may be necessary due to buried drainage channel meanders or dry porous materials. The project soils engineer/geologist should observe all removal areas during the grading.

Fill Placement

1. Fill materials should be brought to at least optimum moisture, placed in thin 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard.

2. Fill materials should be cleansed of major vegetation and debris prior to placement.

3. Any oversized rock materials greater than 8 inches in diameter should be stockpiled and placed under the observation of the soils engineer. As per UBC (ICBO, 1997) requirements, no rock materials greater than 12 inches in diameter should be placed within 10 feet of finish grade, unless prior approval has been granted by the governing agency and geotechnical engineer. Refer to Appendix G for additional recommendations regarding rock handling during grading.

4. "Deep fills" in excess of 50 feet in depth require settlement monitoring. Based on proposed finish grades and anticipated fill depths, settlement monitoring will be required.

   Settlement monitoring is estimated, at this time, to take place for a time period of approximately six to 15 months, or possibly less, based on the settlement data obtained. It should also be noted that basal fill materials below a fill depth of 50 feet should be compacted to 95 percent of the laboratory standard.

5. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Foundation designs may be altered if import materials have greater sulfate/expansion values than the onsite materials encountered in this investigation.

6. Note that some of the claystone layers in the Silverado Formation have high plasticity and could result in high expansion (E.I. >90) if used as fill. Highly expansive soils should be placed deeper than 7 feet from finish grade. Non-plastic, very low expansive granular soils, such as poorly graded sands, should be blended with silts, clays, and gravels, prior to use in the outer portions of slopes.

Subdrains

Subdrains are recommended within drainage/canyon areas where proposed fills exceed 10 feet in height, as well as in some abutting areas where the as-built fill thickness exceeds 10 feet. Additionally, subdrainage systems for the control of localized groundwater seepage should be anticipated following grading due to excess irrigation or precipitation. Subdrains in stabilization fills are also recommended.

Subdrains should be constructed of a minimum 6-inch perforated pipe (SDR 35, or equivalent, with perforations oriented downward) encased in clean, crushed gravel, and wrapped in filter fabric (Mirafi 140 or equivalent). Subdrains greater than 500 feet in linear feet should be constructed per the recommendations stated above. However, the diameter of the perforated pipe should be increased to 8 inches. Subdrains should be constructed to flow at a 1 percent gradient to a suitable outlet, in accordance with the recommendations of the design civil engineer. For subdrain details in keyways/buttress designs, refer to Appendix G.

Slope Considerations and Slope Design

Based on our slope stability analyses and experience on nearby projects, proposed cut and fill slopes constructed using onsite materials, to the heights proposed, should be grossly and surficially stable provided the recommendations contained herein are implemented during site development. Slope stability analyses for the proposed cut and fill slopes are provided in Appendix F.

All slopes should be designed and constructed in accordance with the minimum requirements of the UBC (ICBO, 1997) and/or the County, and the recommendations in the General Earthwork and Grading Guidelines section of this report (see Appendix G), and the following:

1. Fill or stabilized fill over cut slopes should be designed and constructed at a 2:1 (h:v) gradient, or flatter, and should not exceed about 135 feet in height, otherwise, further evaluation will be necessary. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Fill slopes may be properly overbuilt by ±3 to ±5 feet and trimmed/cut back to proposed finish grades. Guidelines for slope construction are presented in Appendix G.

2. Cut slopes with favorable geology should be designed at gradients of 2:1 (h:v), or flatter, and should not exceed about 30 feet in height at a 2:1 inclination. Otherwise, further evaluation will be necessary. Stabilization of most cut slopes is anticipated, as in the southern and middle portions of the tentative tract. Locally adverse geologic conditions (i.e., daylighted joints/fractures, severely weathered fan deposits, or sandy lenses) may be encountered which may require remedial grading, stabilization, or laying back of the slope to an angle flatter than the adverse geologic condition.

3. Daylight cut lots will have some potentially compressible/erodible colluvium/topsoil exposed at the cut/natural interface adjoining slopes. This area will be more subject to erosion, and down-slope movement. Accordingly, improvements and/or foot traffic should not be allowed in this area, and proper drainage is imperative to the stability of this zone. This potential will be mitigated by the recommended setbacks, from a geotechnical viewpoint. These conditions will need to be disclosed to all homeowners and any homeowners association as well as all interested/affected parties. The actual location of this zone should be evaluated during grading.

4. Local areas of highly to severely weathered Tertiary Silverado Formation materials may be present. Should these materials be exposed in cut slopes, the potential for long term maintenance or possible slope failure exists. Evaluation of cut slopes during grading would be necessary in order to identify any areas of severely weathered materials or cohesionless sands. Should any of these materials be exposed during construction, the soils engineer/geologist, would assess the magnitude and extent of the materials and their potential affect on long-term maintenance or possible slope failures. Recommendations would then be made at the time of the field inspection.

5. Landslides have been mapped onsite. Surficial localized earth failures (i.e., slumps, slopewash, etc.) were noted on some existing natural slopes/cliffs associated with the incised canyon drainage courses onsite. In general, these surficial slumps will be completely removed by the proposed grading, and as such, should not pose a major constraint to development, providing our recommendations are properly implemented. This discussion does not include the existing slopes boundary at the residence that may remain as depicted in Cross-Section D-D'.

   The potential for mass wasting, mudflow debris and rock fall, should be properly mitigated in site locations as indicated on plans (Plate 1). Additional walls or mitigation may be recommended elsewhere. It is recommended that debris impact walls or other comparable mitigative devices (GSI, 1995a) be incorporated into the project design, in accordance with the recommendations of the design civil engineer. Should other mass wasting features be encountered in natural or cut slopes above the proposed residential development, and not be removed by the proposed grading, then appropriate mitigation should be considered by the design engineer, where these features intercept the proposed development and/or cut slopes.

6. Loose rock debris and fines remaining on the face of the cut slopes should be removed during grading. This can be accomplished by high pressure water washing or by hand scaling, as warranted.

7. Where loose materials are exposed on the cut slopes, the project's engineering geologist would require that the slope be cleaned as described above prior to making their final inspection. Final approval of the cut slope can only be made subsequent to the slope being fully cut and cleaned.

Transition and Overexcavation Areas

To reduce the potential for differential settlements between cut and fill materials, and/or materials of differing expansion potentials, the entire cut portion of cut/fill transitions should be overexcavated to a minimum depth of 3 feet below finish grade, or to a maximum ratio of fill thickness of 3:1 (maximum to minimum), and replaced with compacted fill. A maximum/minimum fill thickness ratio should be constructed such that 25 feet maximum fill differential is maintained within a lot, in order to keep differential settlements within tolerance. Overexcavation may also be necessary in deep cuts for heave mitigation. In these deep cut areas (more than 50 feet of Silverado Formation is removed), a 10-foot overexcavation and replacement with compacted fill is recommended.

Based on our rock hardness evaluation, trenching for foundations and underground utility improvements will likely encounter difficulty and/or refusal at depths generally greater than ±25 feet below the existing grade. Therefore, overexcavation, during grading, of cut lots to provide a 3-foot compacted fill blanket and street right-of-ways to 1 foot below the lowest utility invert elevation in areas where finish grade/finish surface is generally greater than ±25 feet below the existing grade may be considered to better facilitate trenching. A minimum of 2 feet of fill is recommended below all shallow foundation elements. Drilled pier supported improvements may penetrate cut fill transitions with adequate design/capacity.

Additionally, due to the high expansion potential of portions of the Tertiary Silverado Formation, lots where these sediments are observed to be less than 7 feet below finish grade (after removals), should be overexcavated to provide a 7-foot low or medium expansive compacted fill cap. The purpose of overexcavating this highly expansive formation is to minimize its shrinking/swelling effects on the proposed foundations.

Temporary Construction Slopes

Due to the conditions expected to be encountered during rough grading, it is anticipated that temporary construction slopes, backcuts, false slopes, haul roads, and other temporary conditions will be constructed at a maximum slope ratio of 1:1 (h:v), or flatter. Excavations for removals, drainage devices, debris basins, and other localized conditions should be evaluated on an individual basis by the soils engineer and engineering geologist for variance from this recommendation. Due to the nature of the materials anticipated, the engineering geologist should observe all excavations and fill conditions. The geotechnical engineer should be notified of all proposed temporary construction cuts, and upon review, appropriate recommendations should be presented.

Front cuts may also be cut at a ½:1 to 1:1 slope gradient. In order to decrease the risk of backcut failure, cut slopes and shear keys should be off-loaded prior to excavation of the backcut and key. The actual location of stabilization forecuts and backcuts should be provided by the design civil engineer. Subdrainage, and subdrainage outlets, should be reviewed and recommendations and/or locations should also be similarly provided by the design civil engineer.

"Slot cuts" will need to be excavated for Cross-Section A-A' buttress backcut as previously discussed. The possible instability of temporary cut slopes during stabilization and shear key excavation, or canyon clean-out, cannot be precluded, and should be emphasized to the grading contractor. The temporary stability depends on many factors, including the slope angle, structural features in the bedrock, shearing strength along planes of weakness, height of the slope, groundwater conditions, and the length of time the cut remains unsupported and exposed to equipment vibrations and rainfall. The possibility of temporary cut slopes failing during canyon clean-outs, stabilization key excavations, etc., may be reduced by:

1. Minimizing the operations extent, in both duration and physical dimensions.

2. Limiting the length of a cut exposed to destabilizing forces at any one time.

3. Cutting no steeper than those backcut inclinations specified by the geotechnical consultant.

4. Avoiding operation of heavy equipment or stockpiling materials on or near the top of the backcut or trench. All OSHA requirements with regard to excavation safety should be implemented by the grading contractor and subcontractors, especially concrete pump trucks.

5. Provide temporary drainage and diversion retarders for the grading work to reduce the potential for ponding and erosion.

SHRINKAGE AND BULKING FACTORS

The volume change of excavated onsite materials upon recompaction is expected to vary with materials, density, insitu moisture content, location, and compaction effort. The in-place and compacted densities of soil materials vary and accurate overall determination of shrinkage and bulking cannot be made. Therefore, we recommend site grading include, if possible, a balance area or ability to adjust grades, slightly to accommodate some variation. Based on our experience with similar materials, the following values are provided as guidelines:

EARTHWORK SHRINKAGE AND BULKING ESTIMATES	
GEOLOGIC UNIT	**ESTIMATED SHRINKAGE/BULKING**
Colluvium/Slopewash/Topsoil/Younger Alluvium/Landslide Deposits	10 to 25 percent shrinkage
Silverado Formation	-5 percent shrinkage to 15 percent bulking

These values should be considered estimates only and will be dependent upon the average relative compaction obtained during grading, which id determined by the grading contractor. If possible, we suggest that provisions be made to allow for final adjustment of grades to balance the earthwork operations. Contractors should review available insitu densities, relative compaction curves, and evaluate shrinkage and bulking based on local experience. If deemed necessary, contractors may wish to provide independent boring programs to evaluate shrinkage and bulking. Subsidence in bedrock areas is estimated to be nil.

SETTLEMENT

Ground settlement should be anticipated due to primary consolidation and secondary compression of the proposed engineered fills. The total amount of settlement and time over which it occurs is dependent upon various factors, including material type, depth of fill, depth of removals, initial and final moisture content, and in-place density of subsurface materials. Planned generally granular fills (up to about $\pm$100 feet in thickness), are not generally prone to excessive differential settlement (on the order of 2 to 3 inches). However, some post-construction settlement is expected and the majority of this settlement is anticipated to occur within approximately three to 12 months following grading. Limited areas of the site with fills $\geq$100 feet in thickness, may need as much as 15 months. The differential settlement that occurs after this time is anticipated to be within acceptable limits (up to 1 inch of differential settlement in 40 lateral feet). This settlement will be monitored and design recommendations revised, as necessary, based on actual field and settlement monitoring data obtained.

Dynamic densification may increase the post-construction settlement effects and was estimated as ¼ percent within artificial fills. The differential settlement of ¾ to 1½ inches over 40 lateral feet onsite is possible given fill thickness of up to approximately 100 feet. GSI should re-evaluate these estimates of dynamic densification at the 40-scale plan review. The estimated of dynamic densification do not include the effects of lateral slope deformation on foundations. Mitigation of grading settlements may include a combination of:

1. Decreasing the slope of the cut/fill transition under building areas;

2. Using either post-tensioned slabs, or mat foundations; and/or,

3. Monitoring of engineered fill settlements, with settlement monuments installed in accordance with Appendix H.

Preliminary Settlement Evaluation

Any settlement-sensitive structures should be evaluated and designed for the combination of site-specific soil parameters and the estimated settlements and angular distortion values provided below. The 1997 UBC setbacks should be adhered to when planning improvements on the deeper fill lots. Time estimates of settlements as well as settlement magnitudes should be revisited during grading when fill materials are being placed. Where not already specified in fill (fill slopes) the use of drains within the upper 50 feet of fills may be considered to reduce wait times for settlements.

DEPTH OF FILL (FT)	ULTIMATE DIFFERENTIAL SETTLEMENT (IN)	ULTIMATE ANGULAR DISTORTION (BUILD AT COMPLETION OF GRADING)	SUGGESTED BUILDING WAIT PERIOD UNTIL 50% PRIMARY CONSOLIDATION (MONTHS)	ESTIMATED ANGULAR DISTORTION AFTER WAITING PERIOD**
0-25	<1	1/480	0 to 3	1/480
25-50	1½	1/400*	1 to 4	1/480
50-110	3	1/275*	3 to 15	1/480

* Non-buildable immediately after grading.

** After the waiting period differential settlement is approximately 1/480, or 1 inch in 40 feet. Does not include the effects of seismic deformation or lateral slope deformation.

PRELIMINARY FOUNDATION DESIGN

General

The foundation design and construction recommendations are based on laboratory testing and engineering analysis of onsite earth materials. Recommendations for conventional foundation systems as well as post-tensioned systems are provided in the following sections. The foundation systems may be used to support the proposed structures, provided they are founded in competent bearing material. The proposed foundation systems should be designed and constructed in accordance with the guidelines contained in the UBC (ICBO, 1997) and the and the differential settlement and angular distortion discussed previously and herein. Conventional foundations may be utilized for soils with an E.I. of less than 90 (i.e., very low to medium classification) and fill depths under 25 feet in thickness. Where expansive soils are exposed at finish grade and/or compacted fills in excess of 25 feet in thickness exist, post-tensioned slabs will likely be required. Recommendations for post-tensioned design are included in the following sections.

Conventional Foundation Design

1. Conventional spread and continuous footings may be used to support the proposed residential structures provided they are founded entirely in properly compacted fill or other suitable bearing material (excluding the highly expansive Tertiary Silverado Formation).

2. Analyses indicate that an allowable bearing value of 1,500 pounds per square foot (psf) may be used for design of footings which maintain a minimum width of 12 inches (continuous) and 24 inches square (isolated), and a minimum depth of at least 12 inches into the properly compacted fill or competent fan deposits, or the Tertiary Silverado Formation bedrock unit. The bearing value may be increased by one-third for seismic or other temporary loads. This value may be increased by 200 psf for each additional 12 inches in depth, to a maximum of 2,500 psf.

3. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

4. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf with a maximum earth pressure of 2,500 psf.

5. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

6. All footings should maintain a minimum 7-foot horizontal distance between the base of the footing and any adjacent descending slope, and minimally comply with the guidelines depicted on Figure No. 18-I-1 of the UBC (ICBO, 1997).

Lateral Pressure

1. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 225 pcf with a maximum earth pressure of 2,500 psf.

3. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

FOUNDATION CONSTRUCTION

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering standpoint. Onsite soils will likely vary from very low to high (E.I. 0 to 130). Final foundation design will be based upon which earth material is exposed at finished grades, as verified by testing, during or shortly after site grading.

Accordingly, the following preliminary conventional foundation construction recommendations are for soils in the top 7 feet of finish grade, which will have a very low to medium expansion potential, for planning and design considerations. Recommendations by the project's design-structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. Final foundation design will be provided based on the actual depth of fill underlying the lot and the expansion potential of the near surface soils encountered during grading.

Expansion Classification - Very Low (E.I. ≤20[1], and Fills ≤25 Feet)

1. Conventional continuous footings should be founded at a minimum depth of 12 inches below the lowest adjacent ground surface for one-story floor loads and 18 inches below the lowest adjacent ground surface for two-story floor loads. Interior footings may be founded at a depth of 12 inches below the lowest adjacent ground surface.

   Footings for one-story floor loads should have a minimum width of 12 inches, and footings for two-story floor loads should have a minimum width of 15 inches. All footings should have one No. 4 reinforcing bar placed at the top and one No. 4 reinforcing bar placed at the bottom of the footing. Isolated interior or exterior footings should be founded at a minimum depth of 24 inches below the lowest adjacent ground surface.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across the garage entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs in residential and garage areas should be a minimum of 5 inches thick, and underlain with a vapor retarder consisting of a minimum of 10-mil, polyvinyl-chloride membrane with all laps sealed. This membrane should be covered, above and below with a minimum of 2 inches of sand (total of 4 inches) to aid in uniform curing of the concrete and to prevent puncture of the vapor retarder.

4. Concrete slabs, including garage slabs, should be reinforced with No. 3 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

5. Garage slabs should be poured separately from the residence footings and be quartered with expansion joints or saw cuts. A positive separation from the footings should be maintained with expansion joint material to permit relative movement.

6. The residential and garage slabs should have a minimum thickness of 5 inches, and the slab subgrade should be free of loose and uncompacted material prior to placing concrete.

7. Presaturation is not necessary for these soil conditions; however, the moisture content of the subgrade soils should be equal to or greater than optimum moisture

---

1 In accordance with 1997 UBC Conventional Foundations may be used on very low expansive soil, with PI of less than 15, provided that the differential settlement has been mitigated in slab/foundation design.

to a depth of 12 inches below the adjacent ground grade in the slab areas, and verified by this office within 72 hours of the vapor retarder placement.

8. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard, whether it is to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

9. Foundations near the top of slope should be deepened to conform to the latest edition of the UBC (ICBO, 1997) and provide a minimum 7-foot horizontal distance from the slope face. Rigid block wall designs located along the top of slope should be reviewed by a soils engineer.

10. Based on post-construction settlement analyses, areas where compacted fill materials in excess of 25 feet exist, an engineered post-tension foundation system will likely be required.

11. Post-tension foundations will likely be required if medium to highly expansive soils are exposed at finish grade, minimum to maximum fill thickness variation does not comply with recommendations herein, or if fills exceed about 25 feet in thickness.

12. As an alternative to conventional foundation systems, an engineered post-tension foundation system may be used. Recommendations for post-tensioned slab design are provided in following sections.

PRELIMINARY POST-TENSIONED SLAB DESIGN

It is GSI's opinion that conventional slab design may not accommodate potential foundation movement that the underlying soils would impart from fill depths in excess of 25 feet in thickness and/or some low and all medium to highly expansive soils. Foundations should be designed to accommodate the differential settlement and angular distortion values provided previously and herein. The recommendations presented below should be followed in addition to those contained in the previous sections. The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer familiar with post-tensioned slab design or corrosion engineering consultant. Upon request, GSI could provide additional data/consultation regarding soil parameters as related to post-tensioned slab design.

From a soil expansion/shrinkage standpoint, a fairly common contributing factor to distress of structures using post-tensioned slabs is a significant fluctuation in the moisture content of soils underlying the perimeter of the slab, compared to the center, causing a "dishing" or "arching" of the slabs. To mitigate this possible phenomenon, a combination of soil presaturation and construction of a perimeter "cut-off" wall grade beam should be employed.

Perimeter foundations should be a minimum of 12, 18, and 24 inches deep for very low to low, medium, and highly expansive soils, respectively. Slab thickness should be a minimum of 5 inches and may need to be creased by the slab design based on steel reinforcement/cable requirements. The walls should be a minimum of 12 inches in thickness. In moisture sensitive slab areas, a vapor retarder should be utilized and be of sufficient thickness to provide a durable separation of foundation from soils (10-mils thick). The vapor retarder should be sealed to provide a continuous water-proof retarder under the entire slab. The vapor retarder should be sandwiched by two 2-inch thick layers of sand (SE>30). Specific soil presaturation is not required for very low to low expansive soils; however, the moisture content of the subgrade soils should be at or above the soils' optimum moisture content to a depth of 12 inches below grade. On a preliminary basis, specific soil presaturation is required for medium to highly expansive soils. For medium expansive soils, the slab subgrade moisture content should be at or slightly above 120 percent of the soil's optimum moisture content to a depth of 18 inches below grade. For highly expansive soils, the slab subgrade moisture content should be at or slightly above 130 percent of the soil's optimum moisture content to a depth of 24 inches below grade.

Post-tensioned slabs should be designed. Based on review of laboratory data for the onsite materials, the average soil modulus subgrade reaction K, to be used for design, is 100 pounds per cubic inch (pci). This is equivalent to a <u>surface</u> bearing value of 1,000 psf.

Post-tensioned slabs should be designed using sound engineering practice and be in accordance with the recommendations of the Post-Tensioning Institute Method, as well as local and/or national code requirements. Soil related parameters for post-tensioned slab design are presented below:

Allowable surface bearing value	1,000 psf
Modulus of subgrade reaction	75 psi per inch
Coefficient of friction	0.35
Passive pressure	250 pcf

<u>Post-Tensioning Institute Method</u>: Post-tensioned slabs should have sufficient stiffness to resist excessive bending due to non-uniform swell and shrinkage of subgrade soils. The differential movement can occur at the corner, edge, or center of slab. The potential for differential uplift can be evaluated using the 1997 UBC Section 1816, based on design specifications of the Post-Tensioning Institute. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet
Constant soil Suction (pf)	3.6
Modulus of Subgrade Reaction (pci)	75
Moisture Velocity	0.7 inches/month

The coefficients are considered minimums and may not be adequate to represent worst case conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have positive drainage that is maintained away from structures. Therefore, it is important that information regarding drainage, site maintenance, settlements, and effects of expansive soils be passed on to future owners.

Based on the above parameters, the following values were obtained from figures or tables of the 1997 UBC Section, 1816. The values may not be appropriate to account for possible differential settlement of the slab due to other factors. If a stiffer slab is desired, higher values of y_m may be warranted.

CATEGORY*	I	II	III	IV
FILL THICKNESS/ EXPANSION INDEX OF SOIL SUBGRADE	FILLS ≤25'/ OR VERY LOW EXPANSION (E.I. = 0-20)	FILLS 25-50'/ OR LOW EXPANSION (E.I. = 21-50)	FILLS 25-50'/ OR MEDIUM EXPANSION (E.I. = 51-90)	FILLS >50'/ OR HIGH EXPANSION (E.I. = 91-130)
e_m center lift	5.0 feet	5.0 feet	5.5 feet	5.5 feet
e_m edge lift	2.5 feet	3.5 feet	4.0 feet	4.5 feet
y_m center lift	1.0 inch	1.7 inches	2.7 inches	3.5 inches
y_m edge lift	0.3 inch	0.75 inch	0.75 inch	1.2 inches

* Categories are based on fill criteria or expansion characteristics. If either criteria is met, then that category for that lot should be used.

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of 12 inches should be considered a minimum. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement ("passive" steel reinforcement bars) per the structural engineer. Other applicable recommendations presented under conventional foundation and the California Foundation Slab Method should be adhered to during the design and construction phase of the project.

Slope Setback Considerations for Footings

Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. For top of slope, the horizontal distance, X, may be calculated by using X = h/3, where h is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings. For bottom (toes) of slopes, setbacks should be X/2, but need not exceed 15 feet (see UBC [ICBO, 1997], Figure 18-I-1).

SOIL MOISTURE CONSIDERATIONS

It should be noted that the foundation construction recommendations provided in GSI (1995a) were not intended to preclude the transmission of water or vapor through the slab, as indicated in current code. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component, or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2006). Therefore, the following should be considered by the structural engineer/foundation/slab designer to mitigate the transmission of water or water vapor through the slab.

- Concrete slabs should be a minimum of 5 inches thick for very low expansive soil conditions, and be minimally reinforced as previously discussed. All slab reinforcement should be supported to provide proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning. Increase of concrete slab thickness would tend to reduce moisture vapor transmission though slabs.

- Concrete slab underlayment should consist of a 10-mil to 15-mil vapor retarder, or equivalent, with all laps sealed per the UBC/CBC (ICBO, 1997 and 2001) and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E-1745 Class A or B criteria and be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E-1745). In order to break the capillary rise of soil moisture, the vapor retarder should be underlain by 2 inches of fine or coarse, washed, clean gravel (80 to 100 percent greater than #4 sieve) and be overlain by at least 2 inches of clean, washed sand (SE $\geq$30) to aid in concrete curing.

- Concrete should have a maximum water/cement ratio of 0.50.

- Where slab concrete compressive strength is increased, addmixtures used, and water/cement ratios are adjusted herein, the structural consultant should also make changes to the concrete in the grade beams and footings in kind so that the concrete used in the foundation and slabs are designed and/or treated for more uniform moisture protection.

- The use of a penetrating slab surface sealer may be considered in rooms where permeable floor tile or wood will be used. In all planned floorings, the waterproofing specialist should review the manufacturer's recommendations and adjust installation as needed. Homeowner(s) should be advised which areas are suitable for tile or wood floors.

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer.

Please be aware that the above should be implemented if the transmission of water or water vapor through the slab is undesirable. Should these recommendations not be implemented, then full disclosure of the potential for water or vapor to pass through the foundations and slabs and resultant distress should be provided to all interested parties, in writing. Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab should be anticipated.

WALL DESIGN PARAMETERS CONSIDERING EXPANSIVE SOILS

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (Class 2 permeable filter material or Class 3 aggregate base) or native materials are used to backfill any retaining walls. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. The foundation system for the proposed retaining walls should be designed in accordance with the recommendations presented in this and preceding sections of this report, as appropriate. Footings should be embedded a minimum of 18 inches below adjacent grade (excluding landscape layer, 6 inches) and should be 24 inches in width. There should be no increase in bearing for footing width. Preliminary recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) are provided in a later section.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 65 pcf, plus any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

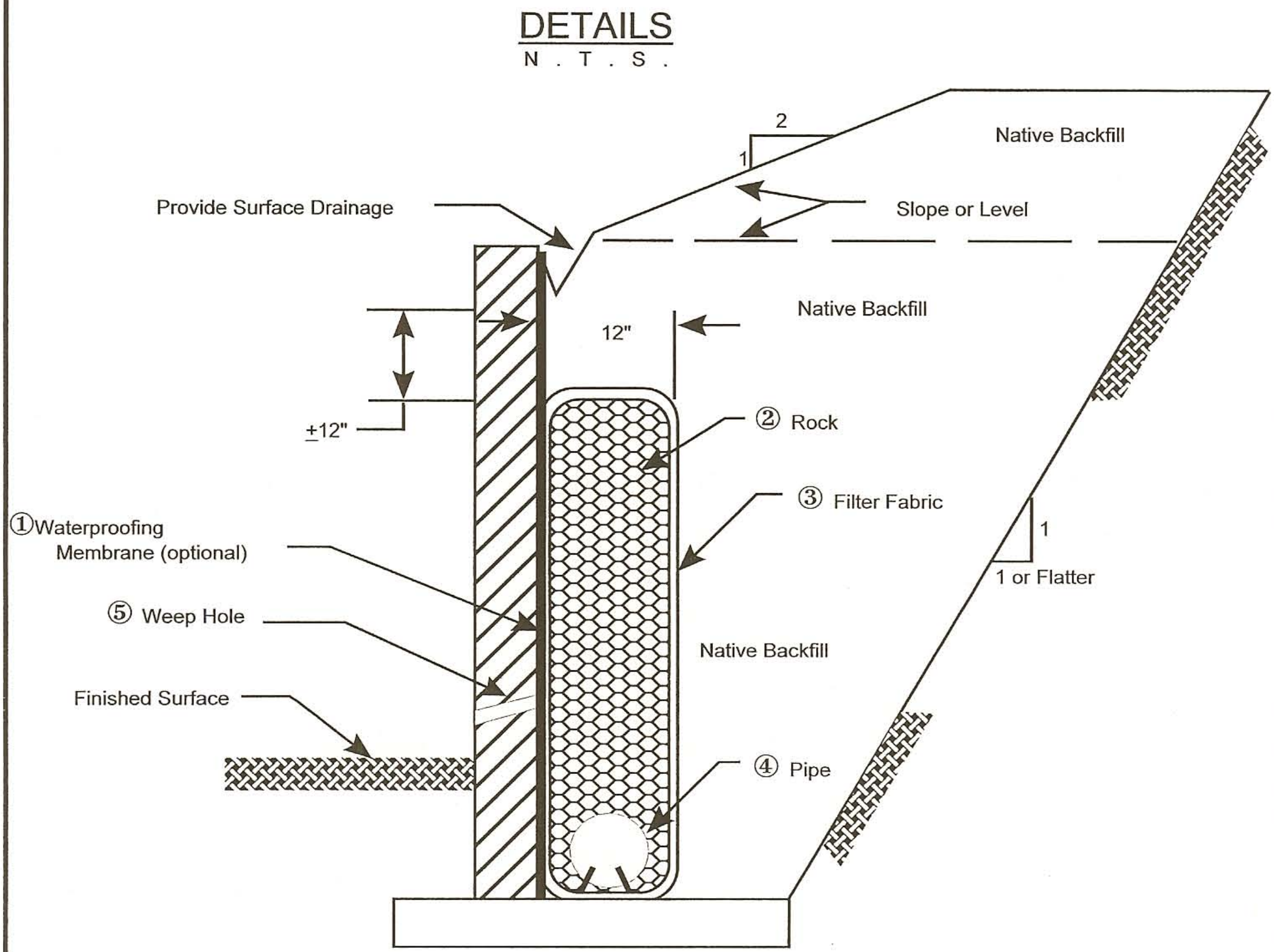
The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by City and/or County standard design. Active earth pressure (Equivalent Fluid Pressure or Weight, EFW) may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. These EFWs do not include the effects of expansive soils. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request. Considering the level of PHSA (10 percent probability of exceedance in 50 years), GSI recommends that, for walls over 6 feet in height and in close proximity to residences or main access roads, the designer consider using a seismic increment of 15H be used for a surcharge, to model seismic loadings. The pressure should be added as a uniform pressure where H is the height of the wall from footing bottom (excluding keys) to top of backfill.

SURFACE SLOPE OF RETAINED MATERIAL HORIZONTAL:VERTICAL	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)**
Level*	40	45
2 to 1	60	65

* Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall.
** Native backfill with EFW shown are for very low to low expansive soils (E.I. = 0-50).

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ½-inch to ¾-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **ROCK:**
3/4 to 1-1/2" (inches) rock.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent; place fabric flap behind core.

④ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface (No weep holes for basement walls.).

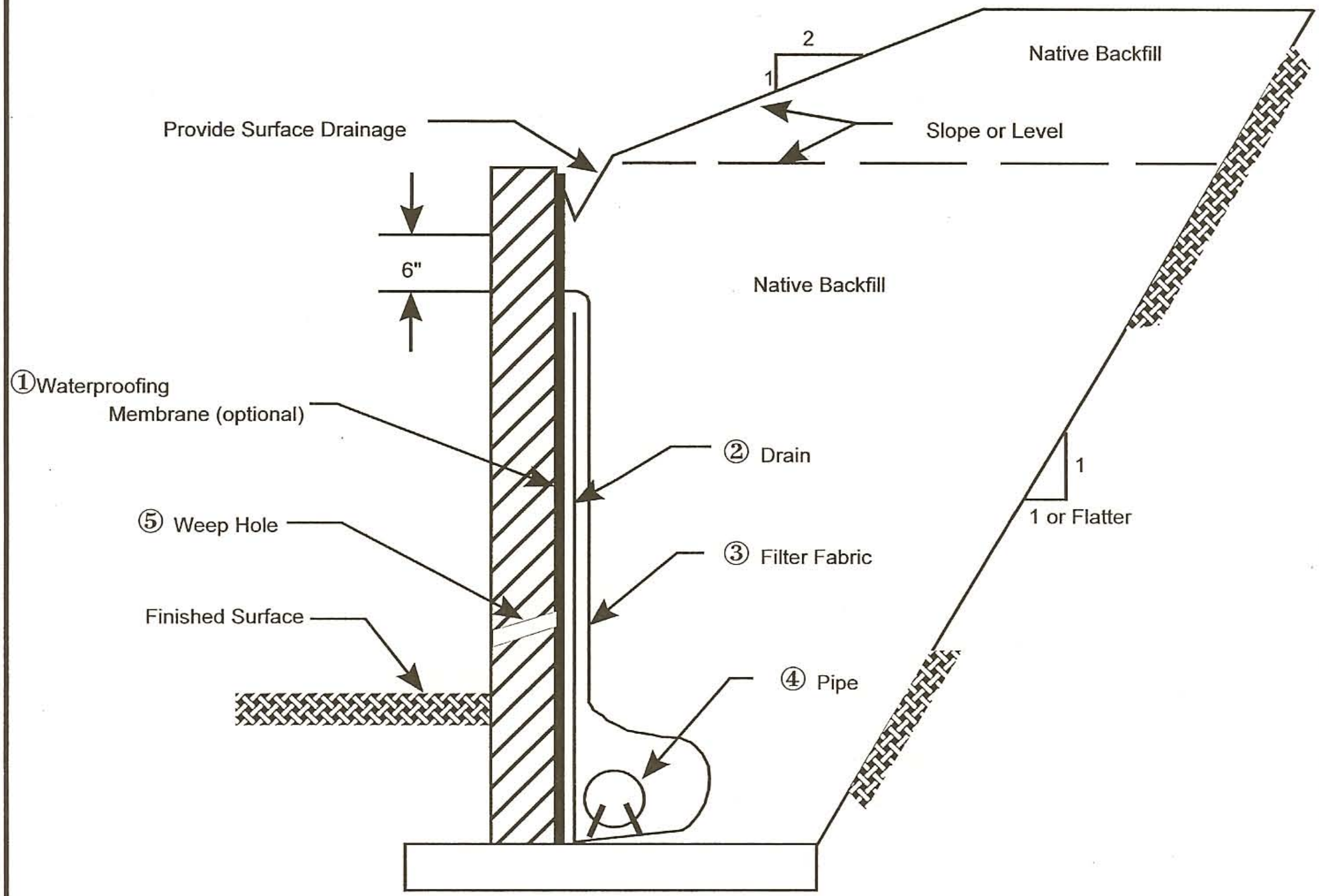
GeoSoils, Inc.

TYPICAL RETAINING WALL BACKFILL AND DRAINAGE DETAIL

DETAIL 1

Geotechnical • Coastal • Geologic • Environmental

DETAILS
N.T.S.



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **DRAIN:**
Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls.
Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (All Perforations down).

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent; place fabric flap behind core.

④ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point.

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

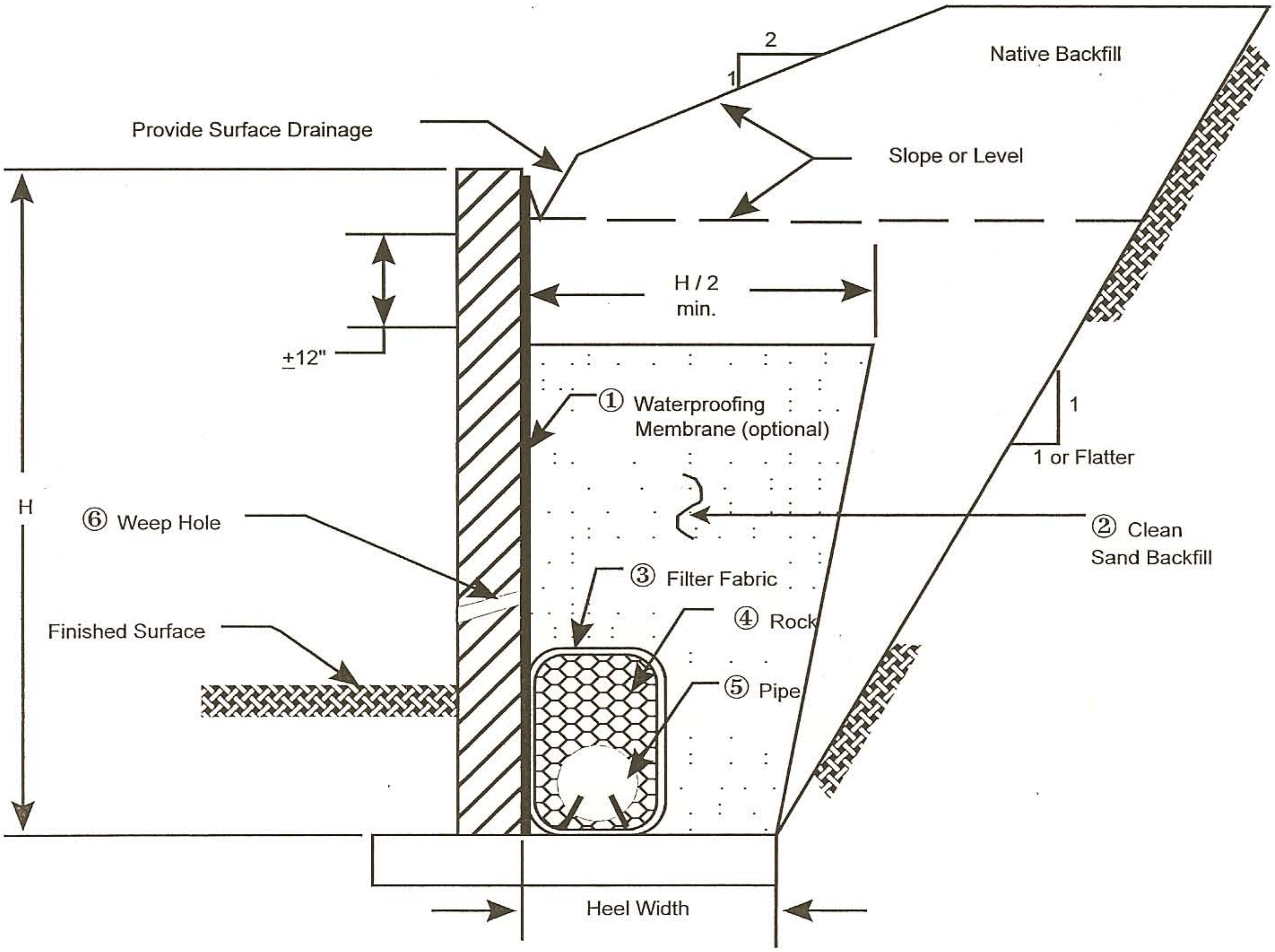
GeoSoils, Inc.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL GEOTEXTILE DRAIN

DETAIL 2

Geotechnical • Coastal • Geologic • Environmental

DETAILS
N . T . S .



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **CLEAN SAND BACKFILL:**
Must have sand equivalent value of 30 or greater; can be densified by water jetting.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent.

④ **ROCK:**
1 cubic foot per linear feet of pipe or 3/4 to 1-1/2" (inches) rock.

⑤ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑥ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

GeoSoils, Inc.

RETAINING WALL AND SUBDRAIN DETAIL
CLEAN SAND BACKFILL

DETAIL 3

Geotechnical • Coastal • Geologic • Environmental

Detail Geotextile Drain). Materials with an E.I. potential of greater than 90 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall and Subdrain Detail Clean Sand Backfill).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes in walls higher than 2 feet should not be considered. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. ≤90). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the civil designer may specify either:

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into native formational material (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

SEGMENTAL RETAINING WALLS

The geotechnical design parameters provided below are for the proposed ±17-foot high segmental retaining wall to be located along approximately 870 feet of the eastern site boundary. These design parameters assume that either non-expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials (up to and including an E.I. of 30, P.I. ≤10) are used to backfill any segmental retaining walls. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed or damp-proofed, depending on the degree of moisture protection desired.

Guidelines for Segmented Wall Construction

It is our understanding that segmented walls will be utilized on the project. These walls are, by nature, a flexible system and, as such, not suited for every slope support condition, as determined by the project design civil engineer. Slope and structural setbacks from the heel of walls will likely be necessary. The necessary setbacks should be defined by the various project consultants and approved by the governing agencies prior to final design. At a minimum, the building setback should be up at a 1:1 (h:v) projection from the heel of the segmented wall foundation, and should be shown on the precise grading plans by the design civil engineer. Building setback mitigation may be accomplished by deepening any adjoining foundations through this zone of projection, provided this does not disturb any geofabric.

In addition to the previous recommendations, the following are specific recommendations for segmented wall design and construction. These recommendations have been provided in an effort to achieve the most desirable and efficient means of construction. Some of these do not deal specifically with geotechnical aspects, but do have significant effects on the quality of the end product. As project geotechnical consultants, we feel that strong consideration should be given to these recommendations. If more onerous project specifications are required by the manufacturer or governing agency, then those guidelines should be followed.

Compared to conventional retaining walls, segmented walls require significantly more geotechnical observation and testing. The costs for these services depend on wall size, conscientiousness of the contractor, and other factors. GSI should evaluate the geotechnical aspects of the wall layouts (offsets, cross section, alignments) prior to construction.

Foundation

1. Prior to excavation for the wall base, the alignment and grade for the wall should be established in the field by the project civil engineer or project surveyor.

2. The contractor should have a qualified grade checker onsite to continually verify the gradient (or batter) and alignment of the base excavation and wall during construction.

3. The project surveyor should spot-check wall gradient (face of wall slope) and alignment at least every 10 feet vertically and 50 feet horizontally.

4. When locating the base of the wall, structural setbacks established by the governing agency, and/or geotechnical engineer should be followed.

5. Walls should be founded on compacted fill, bedrock, or other suitable materials, as described in our referenced reports.

6. The recommended equivalent fluid pressure for design of the segmented walls should be 45 pcf for level backfill and 65 pcf for 2:1 backfill, assuming a select very low to low expansive granular backfill material (E.I. $\leq$30, P.I. $\leq$10, $\phi = 28$ degrees, c = 200). These equivalent fluid pressures are based solely on static soil conditions and do not include seismic, footing surcharge, earthwork surcharge, or traffic loading which will need to be included, as necessary.

7. Utilize a seismic increment of 10 to 15H when evaluating internal gridwall stability in accordance with the Retaining Wall section of this report. For global stability of gridwalls, a seismic factor (pseudo-static) of 0.15 *i*, should be used.

8. A bearing value of 1,500 psf may be utilized for a 1 foot deep footing. A friction coefficient of 0.35 may be used for a concrete to soil contact. A friction angle of 25 degrees and a soil unit weight of 115 to 130 pcf may be utilized for the compacted fill, dense competent Silverado Formation, as verified by observation and/or testing. In addition, a cohesion value of 0 psf, for reinforced fill, 100 psf for retained fill, and 100 psf for foundation fill may be utilized.

9. Prior to placement of the segmented members, the base excavation should be observed by representatives of this firm.

10. A concrete/crushed stone leveling pad may be used to provide a uniform surface for the wall base. It is recommended that a concrete slab base be provided.

11. If it is necessary to locally deepen the wall base to obtain suitable bearing materials, the contractor should consult the project design engineer to determine if the wall location or design of the wall is affected.

12. Segmented wall height at the terminal ends of the wall should not exceed 4 feet unless lateral support is provided.

Backfill

1. Backfill within, behind, and in front of the segmented walls, which do not utilize geogrid fabric, should be compacted to a minimum of 90 percent relative compaction unless otherwise specified by the manufacturer. Backfill behind segmented walls, which utilize geogrid fabric, should be compacted to a minimum of 95 percent relative compaction. Any backfill other than the "unit core fill (¾-inch crushed rock or stone)" should be placed in controlled lifts not to exceed 6 inches in thickness, and moisture-conditioned as necessary to achieve at least optimum moisture content. Backfill within and immediately behind the walls should also be as indicated on the (precise and rough) grading plans.

2. Backfill materials should be free draining, and free from organic materials, with a maximum of 15 percent fines passing the No. 200 sieve. Lifts should be placed

horizontally and compaction equipment should not be allowed to damage the geogrid fabric, if utilized.

3. If gravel or other select granular material is used as backfill within or behind the segmented wall, it should be capped with a minimum 18 inches compacted fill composed of relatively impervious material.

4. During construction, the unfilled section of wall should not be stacked more than 2 feet above the fill behind the wall. If gravel is used to fill the wall, the wall may be stacked 3 feet above adjacent grades. The maximum gravel size should be less than ¾ inches.

5. Adequate space should be provided both behind and in front of the wall so that sufficient compaction can be obtained for all backfill. The slope of the geogrid walls and beaching (in cross section and alignment) should be in accordance with the manufacturers recommendations and as approved by the geotechnical consultant.

Wall Backdrains

A drainage system should be installed behind segmented walls in excess of 3 feet. The design of the system will depend on specific conditions. For most cases, a schedule 40 perforated collector pipe, wrapped in Mirafi 140 or equivalent, may be placed at the heel of the wall with a full height gravel drain, separated from the native backfill materials by Mirafi 140 or equivalent. In areas where native bedrock and/or terrace deposits are retained, a secondary backdrain system, as indicated previously, should also be placed at the rear of the backcut. If necessary, outlets may pass below the base of the wall at a minimum 2 percent gradient. Outlets should be tight-lined to an approved outlet area. The trenches for the outlets may be filled with either compacted material or gravel. If gravel is used, a concrete cut-off wall should be provided at the soil/gravel interface. Seepage should be anticipated below all segmented walls, and this should be disclosed to all homeowners and any homeowners association, and all interested/affected parties.

Materials and Wall Construction

Only sound segmented wall members that meet all required specifications should be used for construction of walls. Members should be free of honeycombing, cracks, broken lugs, or slumped bearing surfaces. All geogrid fabric utilized should comply with the required technical specifications. Geogrid fabric should be placed horizontally to the required length/width behind the wall.

Footing Setbacks for Segmented Walls

It is recommended that settlement-sensitive structures be built behind a 1:1 (h:v) projection above the heel of the foundation for the segmented wall. In addition, all footings should be setback behind a 1:1 projection from the heel of the geogrid reinforced excavation. If structures are located between the two 1:1 projections, the segmented wall should be designed to accommodate the additional surcharge loading from the structure, and deepened building footings may be required depending on the height of the segmented wall. All appurtenant structures (i.e., A/C pads, screen walls, light standards, pools, spas, etc.) should be placed outside a 1:1 (h:v) projection upward from the heel of the wall. Alternately, footings may be constructed such that bearing surfaces are below the 1:1 projection. Appurtenant structures, including pools, utilities, and landscaping, should not disrupt the geogrid behind the walls. All structures proposed within the setback zone will be subject to both horizontal and vertical deflections. All construction proposed within the setback area should be reviewed by the design civil engineer and GSI.

Review of Gridwalls

A qualified geotechnical consultant should review all proposed gridwalls for global stability. Gridwalls mush meet Riverside County slope stability factors-of-safety of 1.5 and 1.1 for static and seismic, respectively. Criteria for residential use (limitations of land use) within grad areas should be provided by the wall designer and reviewed by both the builder and the geotechnical consultant. These limitations should be disclosed to all future homeowners, and any homeowners association.

DEBRIS IMPACT WALLS

Containment of Mudflow Debris and Rock Fall

A potential for mudflow and possible rock fall exists for lots located below significant proposed cut slopes or below re-entrant canyons. Consequently, these lots should be protected with reinforced concrete-deflector walls designed to intercept and contain mudflow debris and rock fall. The deflector walls should be constructed along the tops of uphill-graded slopes bordering the lots located below these cut slopes. Locations of walls will vary depending on as-graded conditions upon completion of rough grading. GSI has depicted the proposed locations on Plate 1. Design parameters for walls should also be based on as-graded site conditions and on a determination of probable quantities of mudflow debris that may accumulate behind the walls, as evaluated by the design engineer.

In lieu of concrete-deflector walls, suitable alternates may possibly consist of debris basins, or raising pad grades, so that there is an ascending minimum ±5-foot slope at the toe of the descending proposed significant cut slopes. However, locations, capacities, and other design considerations should be based on as-graded site conditions. Figure 5 (Debris Device Control Methods) may be used for alternative methods to contain potential debris or mud.

For design purposes, the active earth pressures should utilize an EPF of 125 pcf. Impact and debris walls should be designed in a similar manner. The debris walls and impact walls should be supported by footings with a minimum embedment of 18 inches into competent bedrock. Consideration should be given to supporting debris and impact walls on 12-inch diameter drilled piers embedded a minimum 6 feet into engineered fill or competent bedrock. The actual design for the piers or footings should be performed by the structural consultant using the foundation parameters in this report.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS AND EXPANSIVE SOILS

Expansive Soils and Slope Creep

Soils at the site are likely to be expansive and therefore, become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots.

When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as swimming pools, concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to any homeowners and homeowners association.

Top of Slope Walls/Fences

Due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations, for slopes comprised of expansive soils with an E.I. greater than 50. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate sulfate corrosion, as warranted. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

<u>**Creep Zone:**</u> 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

<u>**Creep Load:**</u> The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

<u>**Point of Fixity:**</u> Located a distance of 1.5 times the caisson's diameter, below the creep zone.

<u>**Passive Resistance**</u>: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

<u>**Allowable Axial Capacity**</u>:

Shaft capacity : 350 psf applied below the point of fixity over the surface area of the shaft.

Tip capacity: 4,500 psf.

EXPANSIVE SOILS, DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site are likely to be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify any homeowners or homeowners association of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. The moisture content of the subgrade should be verified within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a relatively non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. The layer should wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. Driveways, sidewalks, and patio slabs adjacent to the house should be separated from the house with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the house.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by alternate wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations and/or cracking), placed near the top-of-slope, up to a maximum distance of approximately 15 feet from the top-of-slope, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes comprised of expansive materials. Although some movement should be expected, long-term movement from this source may be minimized, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per the 1997 UBC and/or adopted CBC), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to each homeowner and/or any homeowners association.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to each homeowner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Lot Surface Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Toe of Slope Drains/Toe Drains

Where significant slopes intersect pad areas, surface drainage down the slope allows for some seepage into the subsurface materials, sometimes creating conditions causing or contributing to perched and/or ponded water. Toe of slope/toe drains may be beneficial in the mitigation of this condition due to surface drainage. The general criteria to be utilized by the design engineer for evaluating the need for this type of drain is as follows:

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope?

- Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?

- Are there cut-fill transitions (i.e., fill over bedrock), within the slope?

- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.

- Are the slopes north facing? North facing slopes tend to receive less sunlight (less evaporation) relative to south facing slopes and are more exposed to the currently prevailing seasonal storm tracks.

- What is the slope height? It has been our experience that slopes with heights in excess of approximately 10 feet tend to have more problems due to storm runoff and irrigation than slopes of a lesser height.

- Do the slopes "toe out" into a residential lot or a lot where perched or ponded water may adversely impact its proposed use?

Based on these general criteria, the construction of toe drains may be considered by the design engineer along the toe of slopes, or at retaining walls in slopes, descending to the rear of such lots. Following are Detail 4 (Schematic Toe Drain Detail) and Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, homeowner irrigation, or other circumstances. Where drains are constructed during grading, including subdrains, the locations/elevations of such drains should be surveyed, and recorded on the final as-built grading plans by the design engineer. It is recommended that the above be disclosed to all interested parties, including homeowners and any homeowners association.

Erosion Control

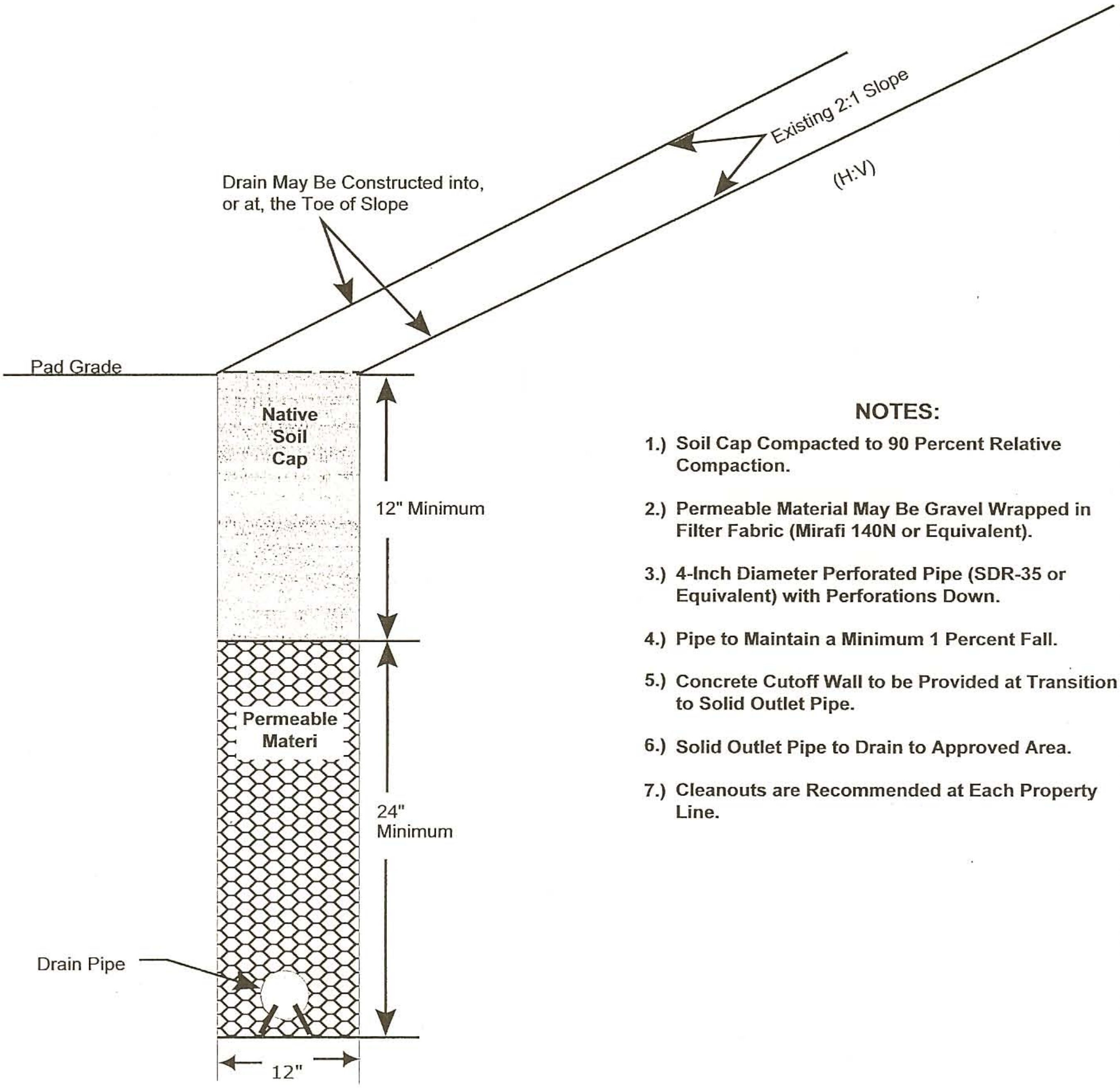
Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture retarder to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water

DETAILS
N . T . S .

SCHEMATIC TOE DRAIN DETAIL



NOTES:

1.) Soil Cap Compacted to 90 Percent Relative Compaction.

2.) Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).

3.) 4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.

4.) Pipe to Maintain a Minimum 1 Percent Fall.

5.) Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.

6.) Solid Outlet Pipe to Drain to Approved Area.

7.) Cleanouts are Recommended at Each Property Line.

GeoSoils, Inc.

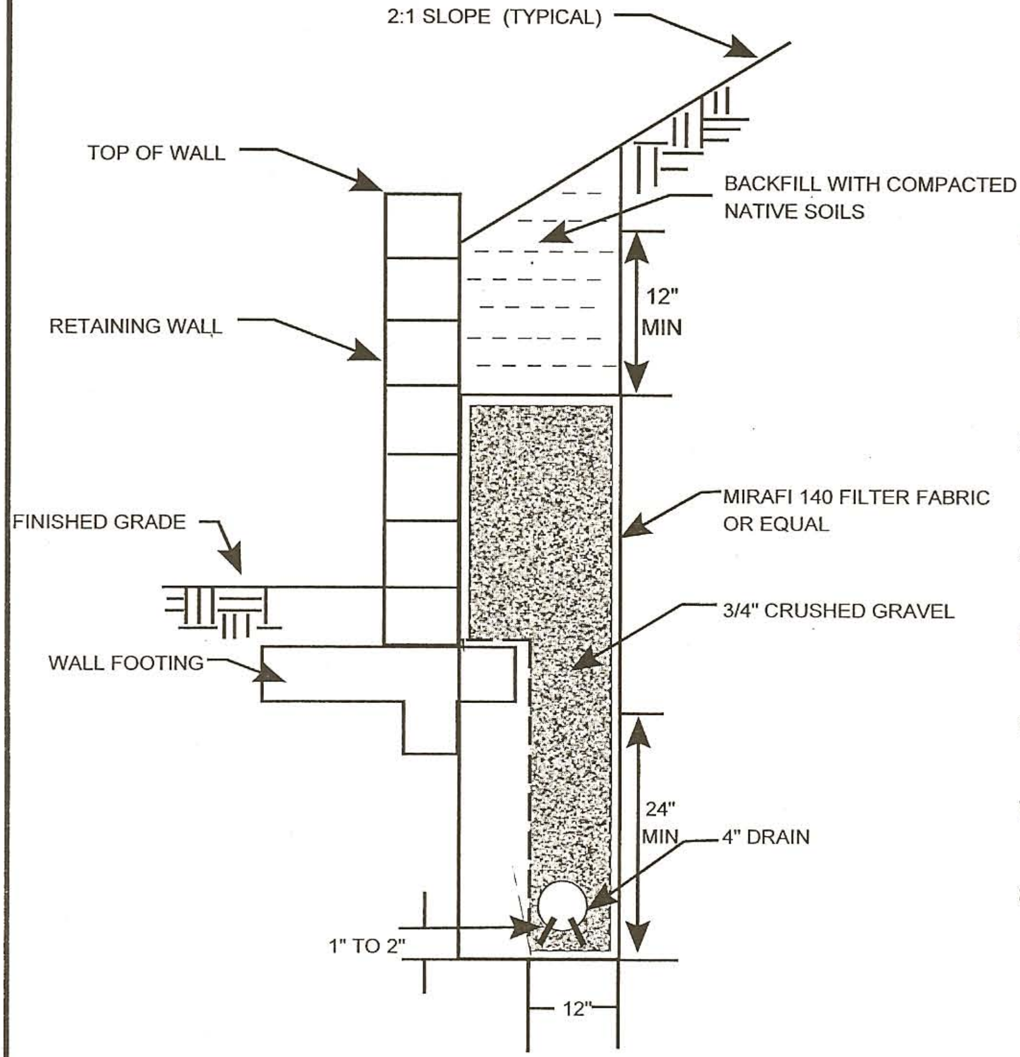
SCHEMATIC TOE DRAIN DETAIL

DETAIL 4

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N.T.S.



NOTES:

1.) Soil Cap Compacted to 90 Percent Relative Compaction.

2.) Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).

3.) 4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.

4.) Pipe to Maintain a Minimum 1 Percent Fall.

5.) Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.

6.) Solid Outlet Pipe to Drain to Approved Area.

7.) Cleanouts are Recommended at Each Property Line.

8.) Compacted Effort Should Be Applied to Drain Rock.

SUBDRAIN ALONG RETAINING WALL DETAIL

NOT TO SCALE

GeoSoils, Inc.

SUBDRAIN ALONG RETAINING WALL DETAIL

DETAIL 5

Geotechnical • Coastal • Geologic • Environmental

from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the house, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement, however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

Subsurface and Surface Water

Regional subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are properly incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors. The above potentials should be disclosed to all homeowners and any homeowners association.

Site Improvements

If in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to the homeowners, any homeowners association, and/or other interested parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and/or testing be performed by GSI at each of the following construction stages:

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).

- During retaining wall subdrain installation, prior to backfill placement.

- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During slope construction/repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any developer or homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction. GSI should review and approve such plans prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

- GSI should review project sales documents to homeowners/homeowners associations for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide homeowners maintenance guidelines which should be incorporated into such documents.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. Please note that the recommendations contained herein are <u>not</u> intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application. This may require thickened slabs, increased visqueen thickness, increased slab underlayment, etc.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the Client, in writing.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

Applied Geotechnical Engineering, Inc., 1988, Soil and foundation investigation, proposed residential subdivision, vesting tentative Tract 23245, county of Riverside, a portion of Section 15, T5S, R4W, and including part of the former Good Hope Mine, project No. 88-118, dated April 14.

Armstrong & Brooks Consulting Engineers, Inc., 2006, City of Corona, Tentative tract map 34760, 100-scale, j.n. 105-588, dated February 21.

Becker, A.M., 1994, Directivity in strong ground motion, Abstract in large earthquakes in metropolitan areas: preparedness, consequences, and response: Earthquake Engineering Research Institute annual meeting, April 7-9, 1994, Pasadena, California

Blake, T.F., 2000a, EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version.

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated to December, 2005, Windows 95/98 version.

_____, 2000c, FRISKSP, A computer program for the probabilistic estimation of peak acceleration and uniform hazard spectra using 3-D faults as earthquake sources; Windows 95/98 version.

Bozorgnia, Y., Campbell, K.W., and Niazi, M., 1999, Vertical ground motion: Characteristics, relationship with horizontal component, and building-code implications; Proceedings of the SMIP99 seminar on utilization of strong-motion data, September, 15, Oakland, pp. 23-49.

Campbell, K.W. and Bozorgnia, Y., 1997, Attenuation relations for soft rock conditions; in EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version, Blake, 2000a.

_____, 1994, Near-source attenuation of peak horizontal acceleration from worldwide accelerograms recorded from 1957 to 1993; Proceedings, Fifth U.S. National Conference on Earthquake Engineering, Vol. III, Earthquake Engineering Research Institute, pp. 283-292.

California Division of Mines and Geology, 1987, Mineral Land Classification of the greater Los Angeles Area, Special Report 143, Part VII, San Bernardino Production-Consumption Region.

_____, 1982, Mineral land classification of Pacific Clay Products, Inc., Clay deposits in the Alberhill area, Riverside County, California, DMG open-file report 81-16.

_____, 1980, State of California earthquake fault zones, Corona South, and Lake Mathews Quadrangles, January 1.

_____, 1979, Aggregates in the greater Los Angeles area, California: Special Report 139.

_____, 1978, Fault evaluation report FER-72, dated September 6.

_____, 1960, Lines of equal elevation of ground water in wells, fall, in Department of Water Resources, southern district, Geohydrology of the Chino-Riverside Area.

Campion, K.M., Sullivan, M.D., May, J.A., and Warme, J.E., 1996, Sequence stratigraphy along a tectonically active margin, Paleogene of southern California, *in* Abbott, P.L., and Cooper, J.D., eds., field conference guidebook and volume for the American Association of Petroleum Geologists annual convention, San Diego, California, May.

Carver, G.A., and McCalpin, J.P., 1996, chapter 5, Paleoseismology of compressional tectonic environments, in Paleoseismology, McCalpin, J.P., ed., Academic Press, Inc., San Diego, California.

Christenson, G.E., and Purcel, C., 1985, Correlation and age of Quaternary alluvial-fan sequences, Basin and Range province, southwestern United States; Weide, D.L. ed., *in* Soils and Quaternary geology of the southwestern United States, Geological Society of America, Special Paper 203.

Colburn, I.P., and Ramirez, P.C., 1995, The Paleocene stratigrtaphic successions in the northen Peninsular Ranges, Orange and Riverside Counties, California, The Pacific Section for Sedimentary Geology (SEPM), field trip guidebook, October.

County of Riverside, Transportation and Land Management Agency, Building and Safety Department, 2004, Review comments, County geologic report no. 1269 (geologic), geotechnical feasibility investigation, tentative tract map no. 30760, Horsethief Canyon Area, Riverside County, California, dated April 7.

Cox, B.F. and Morton, D.M., 1978a, Preliminary map of surficial materials in northwestern Riverside and southwestern San Bernardino Counties, California: U. S. Geological Survey Open File Map 78-977.

_____, 1978b, Generalized map of surficial materials in northwestern Riverside and southwestern San Bernardino counties: U.S. Geological Survey Open File Report 78-978.

CTL Tompson, 2005, Controlling moisture-related problems associated with basement slabs-on-grade in new residential construction.

Dibblee, T.W., Jr., 1982, Geology of the San Bernardino Mountains, southern California in Fife, D.L., and Minch, J.A., eds., Geology and mineral wealth of the California Transverse Ranges.

Dietrich, W.E., and Dorn, R., 1984, Significance of thick deposits of colluvium on hillslopes: A case study in the coastal mountains of northern California, Journal of Geology, v. 92, p. 133-146.

Dunne, G., and Cooper, J., 2001, Geologic excursions in southwestern California, field trip guidebook and volume prepared for the joint meeting of the Cordilleran section –GSA and Pacific Section–AAPG, April 9-11, Universal City, California.

Engle, R., Gay, T.E., and Rogers, B.L., 1959, Geology of the Lake Elsinore quadrangle, California mineral deposits of Lake Elsinore, in Bulletin 146, California Division of Mines and Geology.

Envicom Corporation, 1976, Seismic safety and safety elements technical report for the County of Riverside and the cities of Beaumont, Blythe, Coachella, Corona, Desert Hot Springs, Hemet, Indian Wells, Indio, Norco, Perris, Riverside, and San Jacinto, dated September.

Eppes, M.C., McDonald, E.V., and McFadden, L.D., 2003, Soil geomorphological studies in the Mojave desert: impacts of Quaternary tectonics, climate, and rock type on soils, landscapes, and plant-community ecology; *in* Easterbrook, D.J., ed., Quaternary geology of the United States, INQUA 2003 field guide volume.

GeoSoils, Inc. 2004a, Response to County review, letter dated April 7, 2004, tentative tract map no. 30760, Horsethief Canyon area, Riverside County, California, W.O. 4237-A1-SC, dated April 16.

_____, 2004b, Geotechnical feasibility investigation, tentative tract map no. 30760, Horsethief Canyon Area, Riverside County, California, W.O. 4237-A-SC, dated March 19.

_____, 1998, Geotechnical review of "Rough grading plan for parcel map no. 28201," City of Corona, 3 sheets, 80- and 30-scale, undated, No project number, by PGS&J, W.O. 1829-A.2-SC, dated July 22.

_____, 1997, Response no. 2 to County of Riverside of GeoSoils, Inc. reports, County geologic report no. 909, tentative parcel map no. 28201, Corona, Riverside County, California, W.O. 1829-A.2-SC, dated April 28.

_____, 1996a, Response to County of Riverside review, 'Alquist-Priolo earthquake fault zone, Tentative parcel map no. 28201, GeoSoils, Inc. W.O. 1829-A.2-SC, County geologic report no. 909, City of Corona,' dated February 28, 1996, W.O. 1829-A2-SC, dated October 14.

_____, 1996b, Supplemental geotechnical investigation and slope stability analysis, proposed debris Basins "3" and "3D," tentative tract 28153, Mountain Gate, Corona, Riverside County, California, W.O. 1829-A.3-SC, dated August 6.

_____, 1995a, Geotechnical investigation and limited investigation of the Elsinore fault zone, proposed estate lots, tentative parcel no. 28201, Corona, Riverside County, W.O. 1829-A.2-SC, dated November 9.

_____, 1995b, Additional geotechnical investigation and slope stability analyses, proposed debris basins (Lots T and U), ±43-feet high 2:1 cut slope (Lots 16 through 30), and limited investigation of the Elsinore fault, tentative tract 28153, Mountain Gate, Corona, Riverside County, California, W.O. 1829-A1-SC, October 23.

_____, 1995c, Supplemental evaluation of faulting and geotechnical investigation, Chino fault, Planning areas 21, 27, and 28, Mountain Gate, Corona, Riverside County, California, W.O. 1829-A-SC, dated May 30.

Gray, C. H., Jr., Morton, D.M., and Weber, F.H., 2002, Geologic map of the Corona South 7.5' quadrangle, Riverside and Orange Counties, California; U.S. Geological Survey Open-file report 02-21

Gray, C.H., Jr., 1961, Geology of the Corona south quadrangle and San Bernardino Counties, California and mines and mineral deposits of the Corona south quadrangle, Riverside and Orange Counties, California; California Division of Mines and Geology Bulletin 178, 120 p.

Greenwood, R. B., 1992, Geologic map of the Alberhill 7½' quadrangle, California Department of Conservation, Division of Mines and Geology, DMG Open-File Report 92-10.

Greenwood, R. B., and Morton, D. M., 1991, Geologic map of the Santa Ana 1:100,000 quadrangle, California: California Department of Conservation, Division of Mines and Geology, DMG Open-File Report 91-17.

Hart, E.W., and Bryant, W.A., 1997, Fault-rupture hazard zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.

Harvey, A.M., and Wells, S.G., 2003, Late Quaternary variation in alluvial fan sedimentologic and geomorphic processes, Soda Lake basin, eastern Mojave Desert, California; *in* Enzel, Y., Wells, S.G., and Lancaster, N., eds., Paleoenvironments and paleohydrology of the Mojave and Southern Great Basin Deserts, Geological Society of America, Special Paper 368.

Heath, E.G., Jensen, D.E., and Lukesh, D.W., 1982, Style and age of deformation on the Chino fault, <u>in</u> Cooper, J.D., ed., Neotectonics in Southern California, Guidebook for the 78th Annual Meeting of the Cordilleran Section of the Geological Society of America.

Highland Soils Engineering, Inc., 1988, Preliminary geotechnical investigation, 877± acre Foothill Ranch, Corona, California, Job no. 07-7242-002-00-00, Log no. 8-2704, dated September 16.

_____, 1987, Geologic fault investigation, Main Ranch, Corona, California, Job no. 07-6750-003-00-01, log no. 7-1901, dated July 8.

Hutton, L.K., Jones, L.M., Hauksson, E., and Given, D.D., 1991, Seismotectonics of southern California, *in* Slemmons, D.B., Engdahl, E.R., Zoback, M.D., and Blackwell, D.D. eds., Neotectonics of North America: Boulder, Colorado, Geological Society of America, Decade Map Volume 1.

International Conference of Building Officials (ICBO), 2001, California building code, California code of regulations title 24, part 2, volume 1 and 2.

_____, 1997, Uniform building code: Whittier, California, vol. 1, 2, and 3, dated April.

Jahns, R.H., 1954, Geology of the Peninsular Range Province, southern California and Baja California, *in* Jahns, R.H., ed., Geology of southern California, Bulletin 170, Division of Mines, State of California.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas: California Division of Mines and Geology, Map sheet no. 6, scale 1:750,000.

Keller, E.A., 1986, Investigation of active tectonics; use of surficial earth processes, in Active Tectonics: Studies in geophysics, Wallace, R.E., ed., National Academic Press, Washington, D.C.

Kennedy, M.P., 1977, Recency and character of faulting along the Elsinore fault zone in southern Riverside County, California: California Division of Mines and Geology, Special Report 131, 12p.

Lamar, D.L., and Rockwell, T.K., 1986, An overview of the tectonics of the Elsinore fault zone, in Ehlig, P.L., ed, Neotectonics and Faulting in Southern California, Guidebook for the 82nd Annual Meeting of the Cordilleran Section of the Geological Society of America.

Lamar, D.L., and Swanson, S.C., 1981, Technical report 81-4, study of seismic activity by selective trenching along the Elsinore fault zone, southern California.

Lamar, D.L., 1972, Microseismicity and recent tectonic activity in Whittier fault area, California: Final technical report U.S. Geol. Survey, Contract No. 14-08-001-12288, Earth Science Research Corp., Santa Monica, Calif.

Larsen, E.S.,Jr., 1948, Batholith and associated rocks of Corona, Elsinore, and San Luis Rey quadrangles, Southern California, Geological Society of America Memoir 29.

Leyva, S.M., 2002, Paleosols of the Paleocene Silverado Formation, San Joaquin Hills, California; development, distribution and paleoclimatic implications; a thesis presented to the faculty of the Department of Geological Sciences, California State University, Los Angeles, December.

Maher, A.M., 1982, The Whittier, Chino, and Elsinore faults, a study of the relationship: an unpublished masters thesis, California State University, Long Beach.

Mann, J.F., 1955, Geology of a portion of the Elsinore fault zone, California: California Division of Mines and Geology, Special Report 43, 22p.

Matti, J.C., and Morton, D.M., 1993, Paleogeographic evolution of the San Andreas fault in southern California: A reconstruction based on a new cross-fault correlation, in Powell, R.E., Weldon, R.J. II, and Matti, J. C., eds., The San Andreas Fault System: Displacement, Palinspastic Reconstruction, and Geologic Evolution: Geological Society of America Memoir 178.

McCalpin, J.P., 1996, Field techniques in paleoseismology, chapter 2, in McCalpin, J.P., ed., Paleoseismology, Academic Press, Inc., San Diego, California.

McDonald, E.V., McFadden, L.D., and Wells, S.G., 2003, Regional response of alluvial fans to the Pleistocene-Holocene climatic transition, Mojave Desert, California; *in* Enzel, Y., Wells, S.G., and Lancaster, N., eds., Paleoenvironments and paleohydrology of the Mojave and southern Great Basin deserts, Geological Society of America, Special Paper 368.

Millman, D.E., and Rockwell, T.K., 1986, Neotectonics of the Elsinore fault in Temescal Valley, California, *in* Ehlig, P.L., ed, Neotectonics and faulting in southern California, guidebook for the 82nd annual meeting of the Cordilleran section of the Geological Society of America, 159-166.

Morton, D.M., 1999, Preliminary digital geologic map of the Santa Ana 30' X 60' quadrangle, Riverside County, California, United States Geological Survey, open-file report 99-172.

Morton, D.M. and Cox, B.F.,1994, Geologic map of the Riverside West quadrangle, Riverside County, California; U.S. Geological Survey Open-File Report 88-753.

Morton, D.M. and Weber, F.H., 2003, Preliminary geologic map of the Elsinore 7.5' quadrangle, Riverside County, California, United States Geological Survey, open-file report 03-281.

Muhs, D.R., 1987, Geomorphic processes in the Pacific coast and mountain system of central and southern California, *in* Graf, W.L., ed., Geomorphic systems of North America: Geological Society of America, Centennial Special Volume 2.

National Center for Earthquake Engineering Research, 1997, Proceedings of the NCEER workshop on evaluation of liquefaction resistance of soils, Technical Report NCEER-97-0022, dated December 31.

Neuendorf, K.K.E., Mehl, J.P. Jr, and Jackson, J.A., eds, 2005, Glossary of geology, fifth edition, American Geological Institute, Alexandria, Virginia.

Norris, R.M., and Webb, R.W., 1990, Geology of California, John Wiley & Sons, Inc.

Ponti, D.J., 1985, The Quaternary alluvial sequence of the Antelope Valley, California; Soils and Quaternary geology of the southwestern United States, Weide, D.L., ed., *in* Geological Society of America, Special Paper 203.

Riverside County Flood Control and Water Conservation District, 1932, Aerial photographs, nos. A and B.

_____, 1957, Aerial photographs, nos. 5-39 and 5-40, dated January 15 & 16.

Rockwell, T.K., and Lamar, D.L., 1986, An overview of the tectonics of the Elsinore fault zone, *in* Ehlig, P.L., ed, Neotectonics and faulting in southern California, guidebook for the 82nd annual meeting of the Cordilleran section of the Geological Society of America, 149-158.

Rockwell, T.K., McElwain, R.S., Millman, D.E., and Lamar, D.L., 1986, Recurrent late Holocene faulting on the Glen Ivy North strand of the Elsinore fault at Glen Ivy Marsh, *in* Ehlig, P.L., ed, Neotectonics and faulting in southern California, guidebook for the 82nd annual meeting of the Cordilleran section of the Geological Society of America, 167-176.

Sadigh, K., Chang, C.-Y., Egan, J.A., Makdisi, F., and Youngs, R.R., 1997, Attenuation relations for shallow crustal earthquakes based on California strong motion data, Seismological Research Letters, Vol. 68, No. 1, pp. 180-189.

Schoellhamer, J.E., Vedder, J.G., Yerkes, R.F., and Kinney, D.M., 1981, Geology of the northern Santa Ana Mountains, California, *in* Geology of the eastern Los Angeles basin, southern California, Geological Survey professional paper 420-D.

Shlemon, R.J., 1978, Quaternary soil-geomorphic relationships, southeastern Mojave Desert, California and Arizona *in* Mahaney, W.C. (ed.), *Quaternary Soils*. GeoAbstracts Ltd., Univ. East Anglia, Norwich, England. pp. 187-207.

Shlemon, R.J., Wright, R.H., and Montgomery, D.R., 1987, Anatomy of a debris flow, Pacifica, California; Geological Society of America, in Reviews in engineering geology, volume VII, Debris flows/avalanches: process, recognition, and mitigation, p. 181-199.

State of California, 2006, Civil Code, Sections 896-897.

State of California, Department of Conservation, Division of Mines and Geology, 1991, letter regarding placing the reviewed and approved Petra (1991) report on open file, letter dated May 21, 1991.

Suprenant, Bruce, A., 1992, Vapor retarders under concrete slabs, Publication #C920292, The Aberdeen Group.

Sylvester, A.G., 1988, Strike slip faults in Geological Society of America Bulletin, v. 100, p. 1666-1703.

Thornbury, W.D., 1969, Principles of geomorphology, John Wiley & Sons, Inc.

Treiman, J.A., 2002, Tin Mine, Main Street, Eagle and Glen Ivy (North and South) fault strands of the Elsinore fault zone, Riverside County, California; *in* California Geological Survey fault evaluation report FER-248.

United States Department of Agriculture, 1980, Aerial photographs, Project no. 615020, Flight date October 30, flight line 1380, photo numbers 133-136, and flight date August 21, flight line 680, photo nos. 100-102, scale 1"=3333'±.

Wallace, R.E., 1977, Profiles and ages of young fault scarps, North-central Nevada, *in* Geological Society of America, v. 88, p. 1267-1282, September.

Weber, H.F., Jr., 1977, Seismic hazards related to geologic factors, Elsinore and Chino fault zones, northwestern Riverside County, California, California Division of Mines and Geology Open-File Report 77-4, 96p.

Weldon, R.J., McCalpin, J.P., and Rockwell, T.K., 1996, Chapter 6, Paleoseismology in strike-slip tectonic, *in* Paleoseismology, McCalpin, J.P., ed., Academic Press, Inc., San Diego, California.

Working Group on California Earthquake Probabilities, 1995, Seismic hazards in southern California: probable earthquakes, 1994 to 2024, *in* Bulletin of the Seismological society of America, Vol. 85, No. 2, pp. 379-439, April.

Yeats, R.S., Sieh, K., and Allen, C.R., eds, 1997, The geology of earthquakes, Oxford University Press, New York.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbols	Typical Names
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

	3"		3/4"	#4		#10	#40	#200 U.S. Standard Sieve
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
▼	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1		0-3	ML	0-5 (Bulk) 3 (Chunk)			**<u>ALLUVIUM:</u>** CLAYEY SILT, light yellowish brown, moist, soft; some fine sand.
		3-6	ML				SANDY SILT, dark brown to reddish brown, moist to wet, soft; fine sand and trace clay.
		6-7	SC				CLAYEY SAND, dark reddish brown, saturated, loose; fine to coarse sand w/gravel and cobbles to 1'.
							Total Depth = 7' Groundwater @ 6'/Some Caving below Groundwater Backfilled 5-11-2006
TP-2		0-3	ML				**<u>ALLUVIUM:</u>** CLAYEY SILT, light yellowish brown, moist, soft; some fine sand.
		3-6	ML				SANDY SILT, dark brown to reddish brown, moist to wet, soft; fine sand and trace clay.
		6-8	SC				CLAYEY SAND, dark reddish brown, saturated, loose; fine to coarse sand w/gravel and cobbles to 1'.
							Total Depth = 8' Groundwater @ 6'/Caving below Groundwater Backfilled 5-11-2006

GeoSoils, Inc.

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3		0-3	ML				**ALLUVIUM:** CLAYEY SILT, light yellowish brown, moist, soft; some fine sand.
		3-6	ML				SANDY SILT, dark brown to reddish brown, moist to wet, soft; fine sand and trace clay.
		6-9	SC				CLAYEY SAND, dark reddish brown, saturated, loose; fine to coarse sand w/gravel and cobbles to 1'.
		9-10	SM	9 (Chunk)			**SILVERADO FORMATION:** SILTY SANDSTONE, light yellowish brown, moist to wet, medium dense.
							Total Depth = 10' Groundwater @ 9'/No Caving Encountered Backfilled 5-11-2006
TP-4		0-5	SM	0-5 (Bulk)			**LANDSLIDE DEPOSITS:** SILTY SAND, light yellowish tan, moist to wet; loose.
		5-8	CL				**BURIED TOPSOIL:** SANDY CLAY, black, wet, soft; rootlets.
		8-11	SM				**SILVERADO FORMATION:** SILTY SANDSTONE, light yellowish tan, moist, medium dense; some rootlets.
							Total Depth: 11' No Groundwater/Caving Encountered Backfilled 5-11-2006
TP-5		0-12	SC				**UNDOCUMENTED ARTIFICIAL FILL:** CLAYEY SAND, light brown, moist, loose; trace cobbles, abundant branches and tree trunks, black color @ 10', concrete debris.
							Total Depth = 12½' Groundwater @ 12'/Caving @ Groundwater Backfilled 5-11-2006

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-6		0-2	SM				**TOPSOIL/COLLUVIUM:** SILTY SAND, reddish brown, dry to damp, medium dense; some clay.
		2-8	SM				**SILVERADO FORMATION:** SILTY SANDSTONE, reddish brown, dry to damp, medium dense to dense; some clay, abundant gravel and angular cobbles.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 5-11-2006
TP-7		0-10	SM				**LANDSLIDE DEPOSITS:** SILTY SAND, mottled light brown/dark brown, moist, loose; fractured w/secondary mineralization.
		10-14	CL				**BURIED TOPSOIL/ALLUVIUM:** SANDY CLAY, black, wet to saturated, soft; roots and branches.
		14-16	SM				**SILVERADO FORMATION:** SILTY SANDSTONE, yellowish gray, moist to saturated, loose to medium dense; fractured.
							Total Depth = 16' Groundwater @ 14'/Caving below Groundwater Backfilled 5-11-2006
TP-8		0-8	SM	0-5 (Bulk) 3 (Chunk) 5 (Chunk)			**SILVERADO FORMATION:** SILTY SANDSTONE, light yellowish tan, dry to damp, medium dense; dense @ 6'.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 5-11-2006

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TD-9		0-3	CL				**LANDSLIDE DEPOSITS:** SANDY CLAY, light brownish gray to dark gray, moist to wet, soft.
		3-9	CL				**BURIED ALLUVIUM:** SANDY CLAY, dark gray, wet, soft; trace cobbles to 18".
		9-11	SC				**SILVERADO FORMATION:** CLAYEY SANDSTONE, light yellowish brown, wet to saturated, medium dense.
							Total Depth = 11' Groundwater @ 9'/Caving below Groundwater Backfilled 5-11-2006
TP-10		0-5	SM	0-5 (Bulk) 4 (Chunk)			**SILVERADO FORMATION:** SILTY SANDSTONE, light yellowish tan, moist, dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 5-11-2006
TP-11		0-5	SM	0-5 (Bulk) 4 (Chunk)			**SILVERADO FORMATION:** SILTY SANDSTONE some CLAY, reddish brown, dry to damp, medium dense; gravel.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 5-11-2006

GeoSoils, Inc.

BORING LOG

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-1 SHEET 1 OF 2

DATE EXCAVATED 7-7-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,430' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				ML				**SILVERADO FORMATION:** @ 0' SANDY SILTSTONE, light brown, dry to damp, stiff; fine sand, trace clay.
5			37		111.5	9.6	53	@ 5' As per 0', mottled reddish brown and yellowish brown, very stiff.
10			46	SM	112.3	4.1	23	@ 10' SILTY SANDSTONE, light yellowish white, dry, medium dense to dense; sand, some gravel.
15			50-5"			6.0		@ 15' Rock in sample.
20			65		109.5	4.8	25	@ 20' As per 10', dense.
21½				CL				@ 21½' SILTY CLAYSTONE, olive green, damp, very stiff.
25			9/ 50-3"	SM	120.2	6.3	44	@ 25' SILTY SANDSTONE, light gray to white, dry, dense to very dense; fine sand.
26				CL				@ 26' SILTY CLAYSTONE, dark olive green, damp, very stiff.

BORING LOG

GeoSoils, Inc.

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-1 SHEET 2 OF 2

DATE EXCAVATED 7-7-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,430' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Description of Material**
30			36/ 50-3"	ML/CL	106.3	14.8	71	@ 30' Interbedded SILTSTONE and CLAYSTONE, damp, very stiff to hard.
35			31/ 50-2"	ML	103.4	10.4	46	@ 35' SILTSTONE, grayish blue, dry, very stiff. @ 36' Very hard drilling.
			50-3"		112.9	8.5	48	
40								Total Depth = 39' (due to refusal on hard bedrock) No Groundwater Encountered Backfilled 7-7-2006
45								
50								
55								

BORING LOG

GeoSoils, Inc.

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-2 *SHEET* 1 *OF* 2

DATE EXCAVATED 7-7-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,460' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**SILVERADO FORMATION:** @ 0' SILTY SANDSTONE, dark yellowish brown, dry, medium dense; some gravel and cobbles.
5		X	28		118.7	11.5	78	@ 5' As per 0', light yellowish white, trace clay.
10		X	39	SM/CL	121.8	8.7	64	@ 10' Interbedded SILTY SANDSTONE and SILTY CLAYSTONE layers, light gray to dark brown, dry, medium dense to very stiff.
15		X	42/ 50-4"	SM	126.2	3.9	33	@ 15' SILTY SANDSTONE, light yellowish white, dry, very dense; fine sand.
20		X	20/ 50-3"		128.9	6.4	60	@ 20' As per 15'.
25		X	50-6"		112.4	7.5	42	@ 25' As per 20'.

BORING LOG

GeoSoils, Inc.

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-2 SHEET 2 OF 2

DATE EXCAVATED 7-7-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,460' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
			14/ 50-4"	SM	106.2	4.9	24	@ 30' As per 25'.
35			22/ 50-4"		122.9	8.5	65	@ 35' As per 30', light gray.
40			50-4"		109.5	5.5	28	@ 40' As per 35'.
								@ 42' Very hard drilling.
								Total Depth = 43' (Refusal on hard bedrock) No Groundwater Encountered Backfilled 7-7-2006
45								
50								
55								

GeoSoils, Inc.

BORING LOG

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-3 *SHEET* 1 *OF* 2

DATE EXCAVATED 7-10-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,430' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**SILVERADO FORMATION:** @ 0' SILTY SANDSTONE, yellowish brown, dry, medium dense; fine sand, trace gravel.
5		SPT	23	CL	116.4	15.9	100	@ 5' SILTY CLAYSTONE, olive green, damp, very stiff.
10		SPT	51		126.6	9.4	81	@ 10' As per 5', some fine sand, decreased clay content, very stiff.
15		SPT	63	SM	125.6	10.2	85	@ 15' SILTY SANDSTONE, light gray, dry, dense; fine sand, trace clay.
20		SPT	61	SP/ML/CL				@ 20' Interbedded SANDSTONE, SILTSTONE, and CLAYSTONE layers, vertical bedding.
25		SPT	22/ 50-5"	SM	123.8	9.7	77	@ 25' SILTY SANDSTONE, light yellowish white, dry, very dense.

GeoSoils, Inc.

BORING LOG

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-3 *SHEET* 2 *OF* 2

DATE EXCAVATED 7-10-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,430' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
30			83	CL	125.6	10.0	83	@ 30' SILTY CLAYSTONE, light grayish blue, damp, hard.
35			19/ 50-4"	SM	120.4	9.8	69	@ 35' SILTY SANDSTONE, light yellowish gray, damp, very dense; some clay.
40			27/ 50-4"	SM	126.5	9.2	79	@ 40' As per 35', light grayish blue.
45			44/ 50-4"		121.1	7.2	52	@ 45' As per 40'.
50			34/ 60-4"		115.8	11.5	71	@ 50' As per 45'.
55								

Total Depth = 51½'
No Groundwater Encountered
Backfilled 7-10-2006

BORING LOG

GeoSoils, Inc.

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-4 *SHEET* 1 *OF* 2

DATE EXCAVATED 7-10-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,405' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**SILVERADO FORMATION:** @ 0' SILTY SANDSTONE, yellowish brown, dry, medium dense; fine sand, trace clay.
5		SPT	22		117.6	10.4	68	@ 5' As per 0', SILTY SANDSTONE.
10		SPT	33		118.9	9.6	65	@ 10' As per 5', increased silt content.
15		SPT	39	CL	96.8	16.8	63	@ 15' SILTY CLAYSTONE, dark olive green, damp, very stiff.
20		Ring	36/ 50-3"	SM	125.1	5.6	46	@ 20' SILTY SANDSTONE, light yellowish brown, damp, very dense.
25		SPT	36/ 50-4"		126.4	7.2	61	@ 25' As per 20'.

BORING LOG

GeoSoils, Inc.

W.O. 5166-A-SC

PROJECT: VALENCIA
Tentative Tract 34760

BORING B-4 *SHEET* 2 *OF* 2

DATE EXCAVATED 7-10-06

SAMPLE METHOD: Modified California Sampler

Approx. Elevation: 1,405' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
30			56	ML	115.6	13.9	86	@ 30' SANDY SILTSTONE, dark gray, damp, hard; fine sand, trace clay.
35			41	CL	110.1	16.6	87	@ 35' SANDY CLAYSTONE, olive green, damp, hard; fine sand.
40			65	ML	109.7	17.1	89	@ 40' CLAYEY SILTSTONE, olive green, damp, hard.
45			51		112.8	15.8	90	@ 45' As per 40', increased clay content.
50			31/ 50-4"		116.6	15.6	99	@ 50' As per 45', grayish blue.
55			79	CL				@ 55' SILTY CLAYSTONE, dark grayish blue, damp, hard.
								Total Depth = 56½' No Groundwater Encountered Backfilled 7-10-2006

APPENDIX C

PREVIOUS REPORTS
(On Compact Disc)

APPENDIX D

EQFAULT, EQSEARCH, AND FRISKSP

```
*************************
*                       *
*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
*                       *
*************************
```

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 5166-A-SC

DATE: 10-10-2006

JOB NAME: Valencia

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.8313
SITE LONGITUDE: 117.5826

SEARCH DATES:
START DATE: 1800
END DATE: 2006

SEARCH RADIUS:
100.0 mi
160.9 km

ATTENUATION RELATION: 25) Campbell & Bozorgnia (1997 Rev.) - Soft Rock
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 1 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 1 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.244	IX	2.4(3.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.149	VIII	10.0(16.1)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.324	IX	12.6(20.2)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.064	VI	13.9(22.3)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.153	VIII	13.9(22.3)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.064	VI	13.9(22.3)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.102	VII	22.3(35.9)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.040	V	22.3(36.0)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.082	VII	22.4(36.1)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.038	V	24.6(39.6)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.083	VII	26.6(42.8)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.026	V	26.6(42.8)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.039	V	27.5(44.3)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.036	V	28.7(46.2)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.025	V	29.0(46.7)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.027	V	29.0(46.7)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.025	V	29.2(47.1)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.025	V	29.2(47.1)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.029	V	29.3(47.1)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.025	V	29.3(47.1)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.022	IV	29.3(47.1)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.022	IV	29.3(47.1)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.025	V	29.3(47.1)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.031	V	30.4(48.9)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.032	V	31.3(50.3)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.029	V	31.7(51.1)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.048	VI	32.4(52.1)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.043	VI	32.6(52.4)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.073	VII	32.7(52.6)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.065	VI	33.5(53.9)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.018	IV	33.9(54.5)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.090	VII	33.9(54.5)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.024	IV	33.9(54.6)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.056	VI	35.0(56.2)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.093	VII	37.4(60.2)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.038	V	37.6(60.4)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.022	IV	38.2(61.5)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.022	IV	38.4(61.8)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.015	IV	38.7(62.2)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.015	IV	39.3(63.2)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.014	IV	40.0(64.3)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.014	IV	40.0(64.3)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.014	IV	40.0(64.3)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.013	III	42.4(68.3)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.017	IV	42.7(68.7)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.013	III	42.8(68.8)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.013	III	44.9(72.3)

DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.014	IV	44.9(72.3)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.018	IV	46.3(74.5)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.014	IV	47.5(76.5)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.012	III	48.3(77.7)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.014	IV	48.3(77.7)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.012	III	48.5(78.0)

EARTHQUAKE SEARCH RESULTS

Page 2

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.047	VI	50.3(80.9)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.011	III	50.4(81.1)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.013	III	51.1(82.2)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.013	III	51.1(82.3)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.012	III	52.5(84.5)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.011	III	53.5(86.0)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.012	III	53.8(86.6)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.009	III	53.8(86.6)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.009	III	53.8(86.6)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.012	III	54.0(86.9)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.014	IV	57.1(91.9)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.011	III	58.0(93.4)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.008	III	59.1(95.1)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.030	V	59.7(96.0)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.009	III	59.7(96.1)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.008	II	60.2(96.8)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.035	V	60.6(97.6)
DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.009	III	60.7(97.7)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.015	IV	61.6(99.1)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.015	IV	61.6(99.1)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.010	III	61.6(99.1)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.026	V	61.6(99.1)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.010	III	61.7(99.2)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.009	III	61.7(99.3)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.011	III	63.3(101.9)
DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.008	II	63.3(101.9)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.007	II	63.3(101.9)
DMG	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.009	III	63.3(101.9)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.007	II	63.4(102.1)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.008	III	64.9(104.4)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.007	II	65.3(105.1)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.011	III	65.6(105.5)
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.007	II	65.7(105.7)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.007	II	66.3(106.7)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.007	II	66.5(107.0)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.016	IV	66.6(107.1)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.007	II	66.9(107.7)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.024	V	69.1(111.2)
GSP	34.1390	116.4310	06/28/1992	123640.6	10.0	5.10	0.007	II	69.3(111.5)
GSP	34.3410	116.5290	06/28/1992	124053.5	6.0	5.20	0.007	II	69.8(112.3)
GSP	34.1080	116.4040	06/29/1992	141338.8	9.0	5.40	0.009	III	70.1(112.9)
GSP	34.3780	118.6180	01/19/1994	211144.9	11.0	5.10	0.007	II	70.2(113.0)
GSN	34.2010	116.4360	06/28/1992	115734.1	1.0	7.60	0.063	VI	70.4(113.3)

MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.008	II	71.4(114.9)
GSP	34.0640	116.3610	09/15/1992	084711.3	9.0	5.20	0.007	II	71.8(115.5)
GSP	34.3260	118.6980	01/17/1994	233330.7	9.0	5.60	0.010	III	72.4(116.4)
GSP	34.3690	118.6720	04/26/1997	103730.7	16.0	5.10	0.006	II	72.5(116.7)
GSP	34.3320	116.4620	07/01/1992	074029.9	9.0	5.40	0.008	III	72.8(117.2)
GSP	33.9610	116.3180	04/23/1992	045023.0	12.0	6.10	0.016	IV	73.0(117.5)
GSP	34.3940	118.6690	06/26/1995	084028.9	13.0	5.00	0.006	II	73.3(117.9)
DMG	34.0670	116.3330	05/18/1940	72132.7	0.0	5.00	0.006	II	73.4(118.1)
DMG	34.0670	116.3330	05/18/1940	55120.2	0.0	5.20	0.007	II	73.4(118.1)
PAS	34.3270	116.4450	03/15/1979	21 716.5	2.5	5.20	0.007	II	73.5(118.3)

EARTHQUAKE SEARCH RESULTS

Page 3

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSP	34.0290	116.3210	08/21/1993	014638.4	9.0	5.00	0.006	II	73.5(118.4)
GSP	34.2680	116.4020	06/16/1994	162427.5	3.0	5.00	0.006	II	74.0(119.0)
GSP	34.3770	118.6980	01/18/1994	004308.9	11.0	5.20	0.007	II	74.1(119.2)
GSP	33.9020	116.2840	07/24/1992	181436.2	9.0	5.00	0.006	II	74.6(120.1)
GSB	34.3790	118.7110	01/19/1994	210928.6	14.0	5.50	0.009	III	74.8(120.3)
DMG	34.0830	116.3000	05/18/1940	5 358.5	0.0	5.40	0.008	III	75.5(121.5)
GSP	33.8760	116.2670	06/29/1992	160142.8	1.0	5.20	0.007	II	75.5(121.5)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.005	II	76.5(123.0)
PAS	34.5160	116.4950	06/01/1975	13849.2	4.5	5.20	0.006	II	78.1(125.6)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.011	III	78.7(126.7)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.017	IV	79.5(128.0)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.011	III	81.2(130.6)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.012	III	81.4(131.0)
MGI	34.0000	119.0000	12/14/1912	0 0 0.0	0.0	5.70	0.009	III	82.0(132.0)
DMG	34.0000	119.0000	09/24/1827	4 0 0.0	0.0	7.00	0.030	V	82.0(132.0)
DMG	32.8170	118.3500	12/26/1951	04654.0	0.0	5.90	0.011	III	82.8(133.3)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.005	II	83.6(134.6)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.005	II	83.6(134.6)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.005	II	83.6(134.6)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.009	III	84.3(135.7)
DMG	34.0650	119.0350	02/21/1973	144557.3	8.0	5.90	0.010	III	84.7(136.4)
DMG	34.2500	116.1670	03/20/1945	2155 7.0	0.0	5.00	0.005	I	86.0(138.4)
DMG	34.7120	116.5030	09/25/1965	174344.1	10.6	5.20	0.005	II	86.6(139.3)
GSP	34.4420	116.2480	10/16/1999	125721.0	1.0	5.70	0.008	III	87.1(140.2)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.005	II	87.7(141.1)
PAS	33.6710	119.1110	09/04/1981	155050.3	5.0	5.30	0.006	II	88.4(142.3)
GSP	34.5830	116.3190	07/05/1992	211827.1	0.0	5.40	0.006	II	88.9(143.0)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.007	II	89.0(143.2)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.013	III	89.0(143.2)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.004	I	89.0(143.2)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.005	II	89.0(143.2)
GSP	34.9700	116.8190	03/18/1997	152447.7	1.0	5.10	0.005	II	89.9(144.6)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.014	III	90.7(146.0)
DMG	34.0000	116.0000	09/05/1928	1442 0.0	0.0	5.00	0.004	I	91.4(147.1)
DMG	34.0000	116.0000	04/03/1926	20 8 0.0	0.0	5.50	0.007	II	91.4(147.1)
GSG	34.5940	116.2710	10/16/1999	094644.1	0.0	7.10	0.027	V	91.5(147.3)
DMG	34.8300	116.5200	09/26/1929	20 022.7	0.0	5.10	0.005	I	91.8(147.7)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.008	II	93.6(150.6)
GSP	34.8000	116.4100	10/21/1999	015435.0	1.0	5.00	0.004	I	94.6(152.2)

GSP	34.6800	116.2800	10/16/1999	095935.0	8.0	5.80	0.008	III	94.7(152.3)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.014	IV	94.7(152.3)
T-A	34.8300	118.7500	11/27/1852	0 0 0.0	0.0	7.00	0.023	IV	95.8(154.2)
GSP	34.8600	116.4100	10/22/1999	160848.0	1.0	5.00	0.004	I	97.5(157.0)
DMG	34.9500	116.5330	04/10/1947	171822.0	0.0	5.00	0.004	I	97.7(157.2)
DMG	34.9670	116.5500	04/11/1947	747 0.0	0.0	5.00	0.004	I	98.0(157.7)
DMG	34.9670	116.5500	04/10/1947	16 3 0.0	0.0	5.10	0.004	I	98.0(157.7)
GSP	34.8600	116.3900	10/21/1999	015738.0	4.0	5.00	0.004	I	98.3(158.2)
DMG	34.9830	116.5500	04/10/1947	1558 6.0	0.0	6.20	0.011	III	98.9(159.2)
GSP	35.2100	118.0660	07/11/1992	181416.2	10.0	5.70	0.007	II	99.1(159.4)
DMG	33.2310	116.0040	05/26/1957	155933.6	15.1	5.00	0.004	I	99.9(160.7)
DMG	33.2910	119.1930	10/24/1969	82912.1	10.0	5.10	0.004	I	99.9(160.7)

****

-END OF SEARCH- 157 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2006

LENGTH OF SEARCH TIME: 207 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 2.4 MILES (3.8 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.324 g

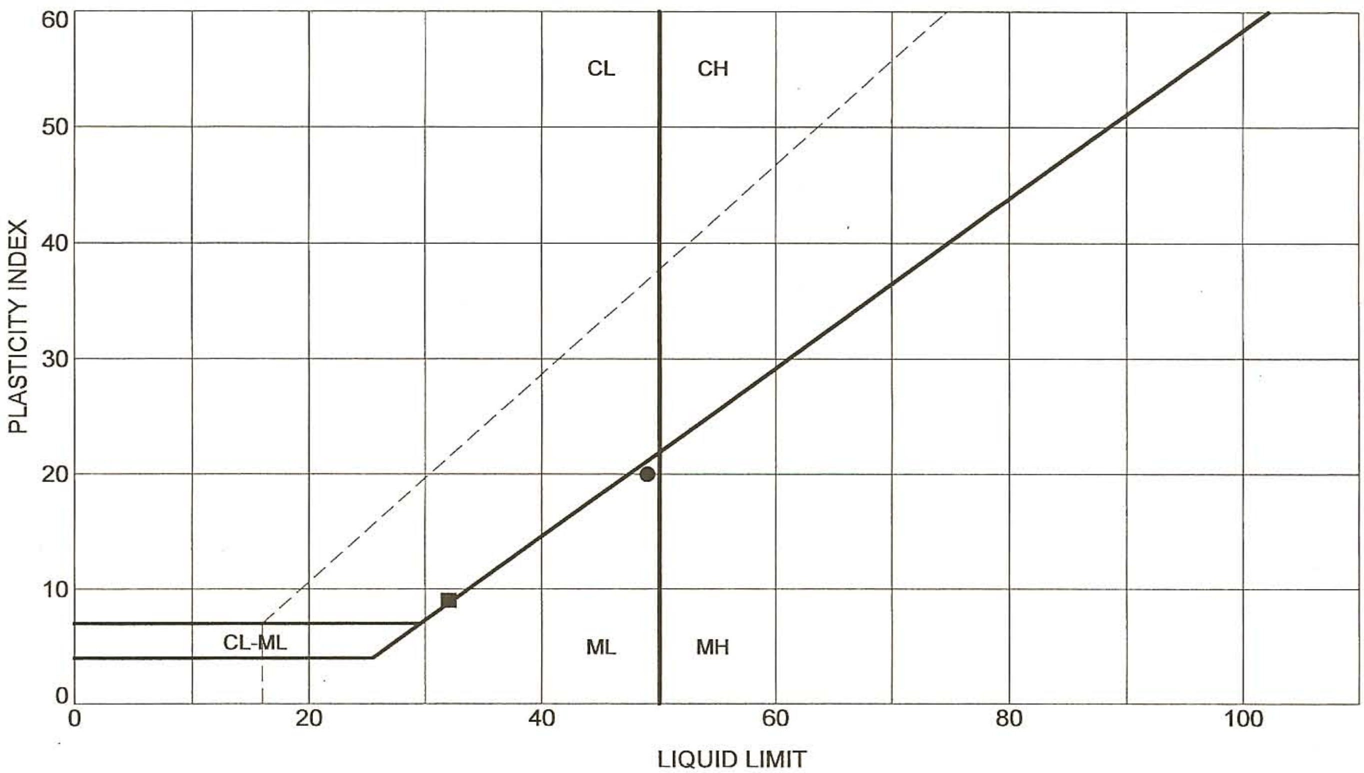
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.571
b-value= 0.386
beta-value= 0.889

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	157	0.76214
4.5	157	0.76214
5.0	157	0.76214
5.5	54	0.26214
6.0	29	0.14078
6.5	12	0.05825
7.0	6	0.02913
7.5	1	0.00485

APPENDIX E

LABORATORY DATA



	Sample	Depth/El.	LL	PL	PI	Fines	USCS CLASSIFICATION
●	B-1	30.0	49	29	20		Sandy Clay
■	TP- 1	0.0	32	23	9		Clayey Sand

US ATTERBERG LIMITS 5166.GPJ US LAB.GDT 10/5/06

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

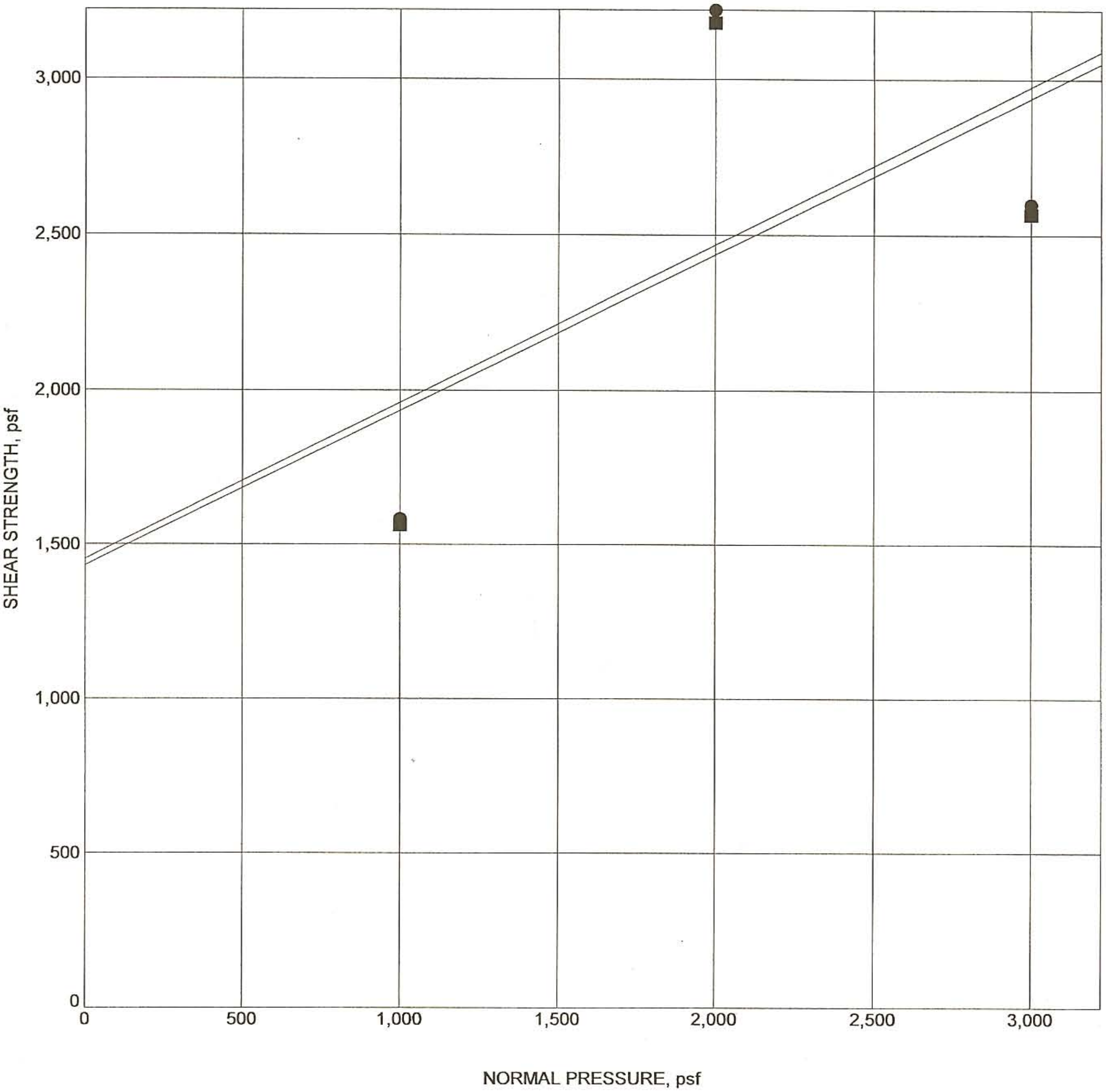
ATTERBERG LIMITS' RESULTS

Project: VALENCIA

Number: 5166-A-SC

Date: October 2006

Plate: E–1



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	B-4	35.0		Sandy Clay	Primary Shear	Undisturbed	107.0	18.6	1452	27
■	B-4	35.0			Residual Shear	Undisturbed	107.0	18.6	1432	27

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

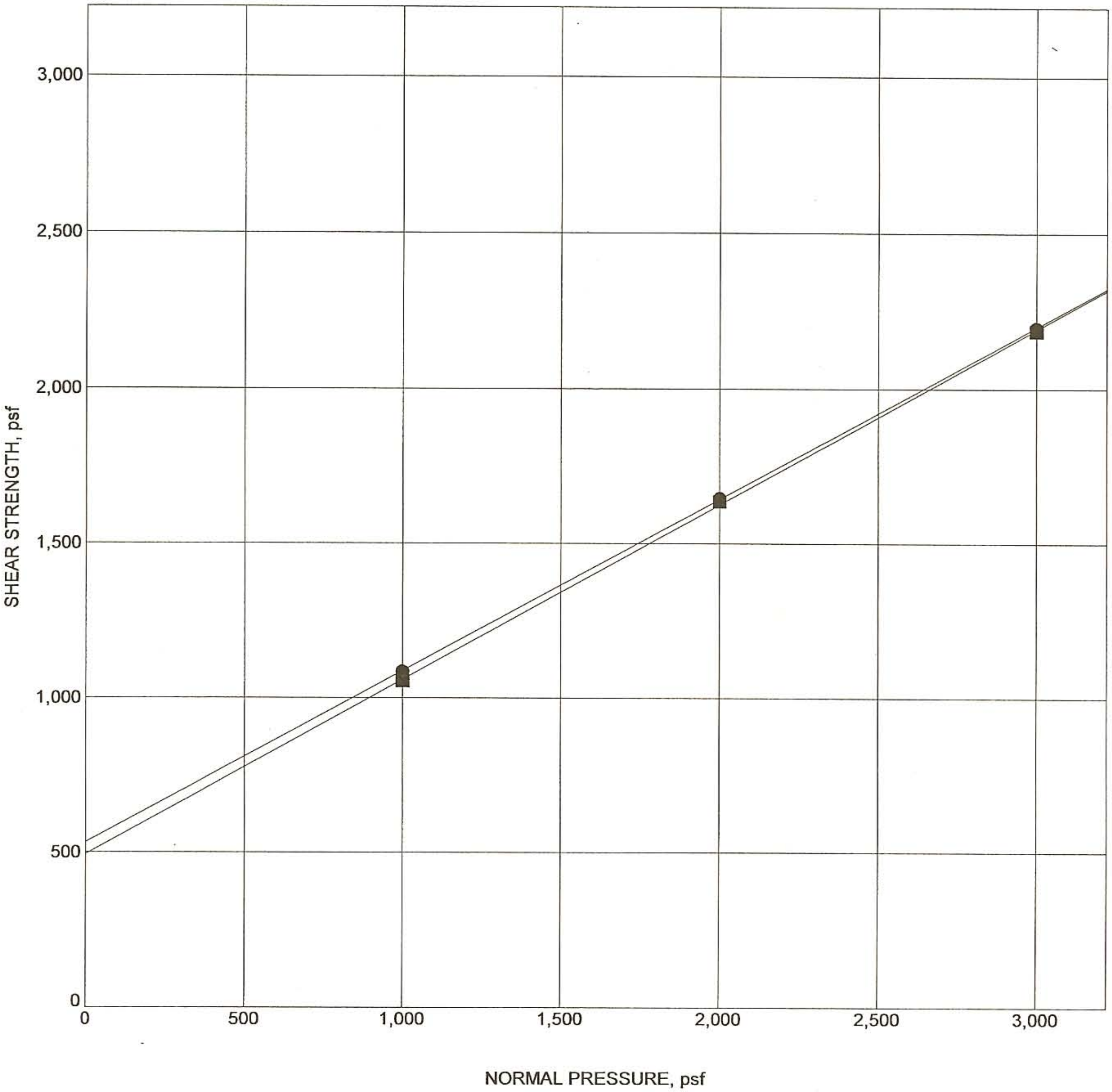
Project: VALENCIA

Number: 5166-A-SC

Date: October 2006

Plate: E–2

US DIRECT SHEAR 5166.GPJ US LAB.GDT 10/5/06



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	TP- 1	3.0		Clayey Sand	Primary Shear	Remolded	111.6	12.0	532	29
■	TP- 1	3.0			Residual Shear	Remolded	111.6	12.0	493	30

Note: Sample Innundated prior to testing

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

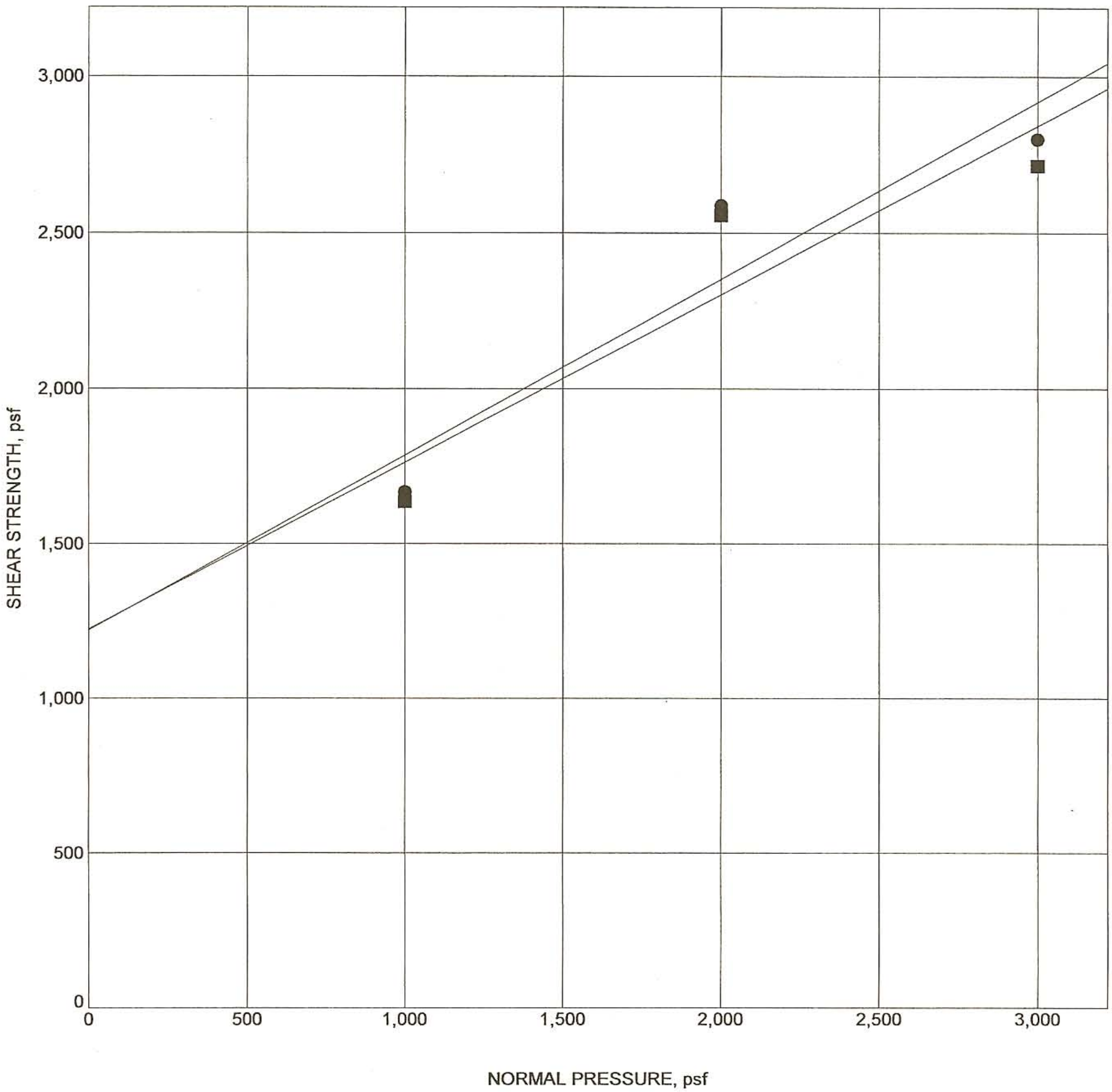
DIRECT SHEAR TEST

Project: VALENCIA

Number: 5166-A-SC

Date: October 2006

Plate: E–3



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	B-3	30.0		Sandy Clay	Primary Shear	Undisturbed	120.8	10.0	1220	30
■	B-3	30.0			Residual Shear	Undisturbed	120.8	10.0	1224	28

Note: Sample Innundated prior to testing

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

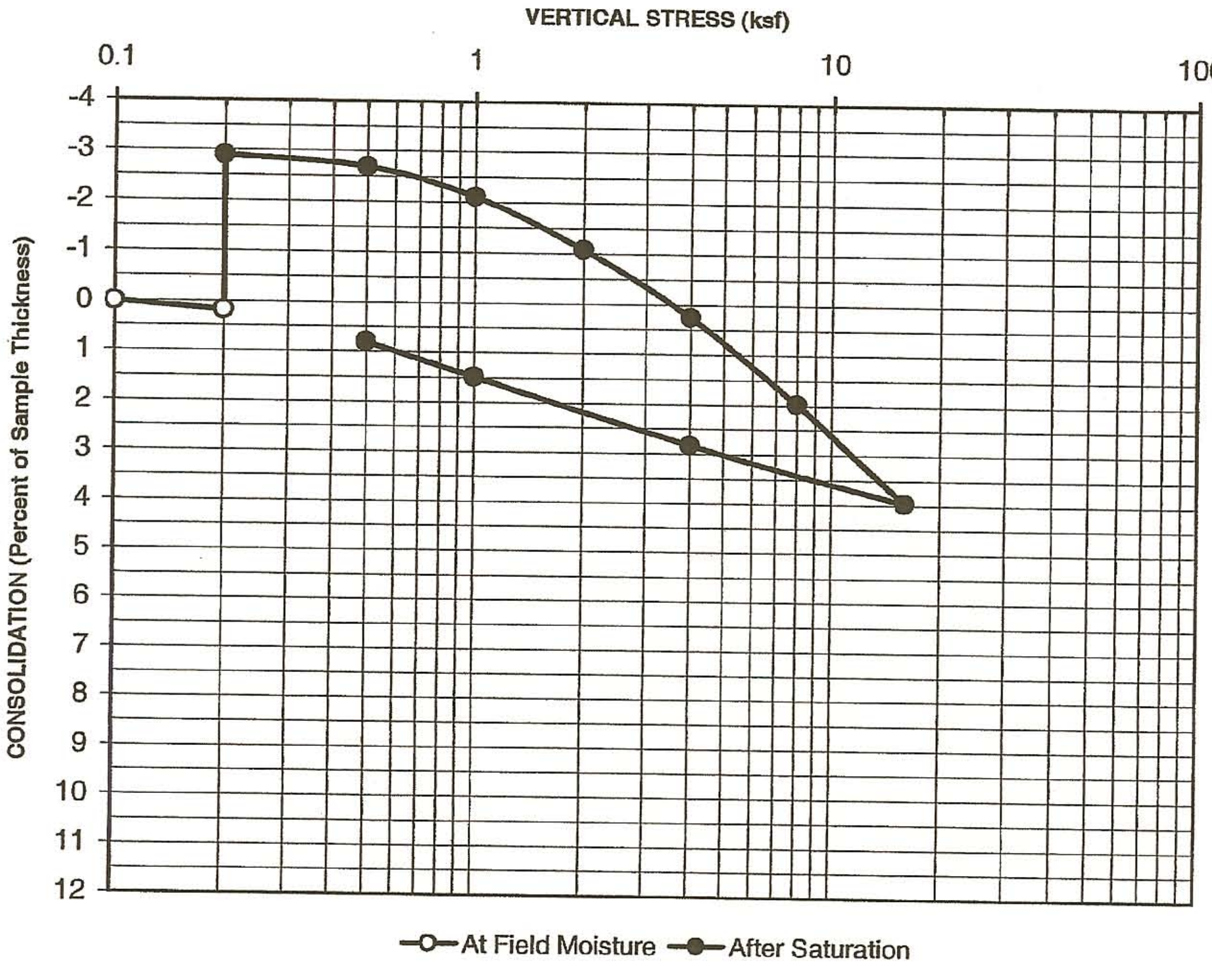
DIRECT SHEAR TEST

Project: VALENCIA

Number: 5166-A-SC

Date: October 2006

Plate: E–4

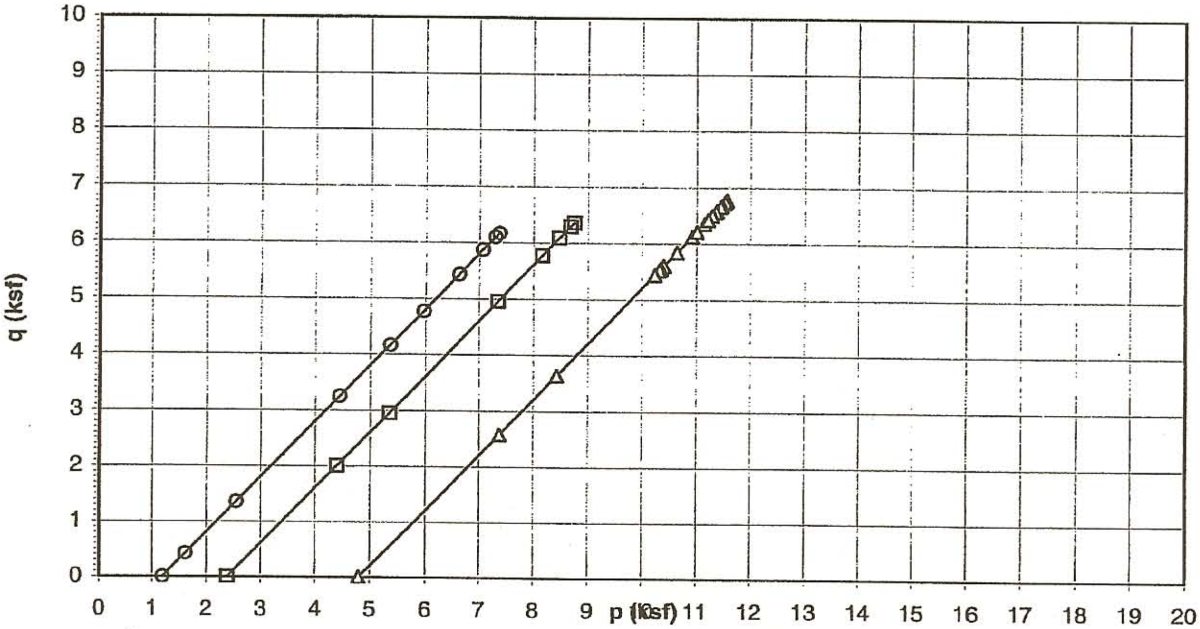


Boring No. :	B-3	Initial Dry Unit Weight (pcf):	117.9
Sample No.:	-	Initial Moisture Content (%):	11.1
Depth (feet):	20	Final Moisture Content (%):	16.6
Sample Type:	Undisturbed	Assumed Specific Gravity:	2.7
Soil Description:	Yellowish Brown Claystone	Initial Void Ratio:	0.43

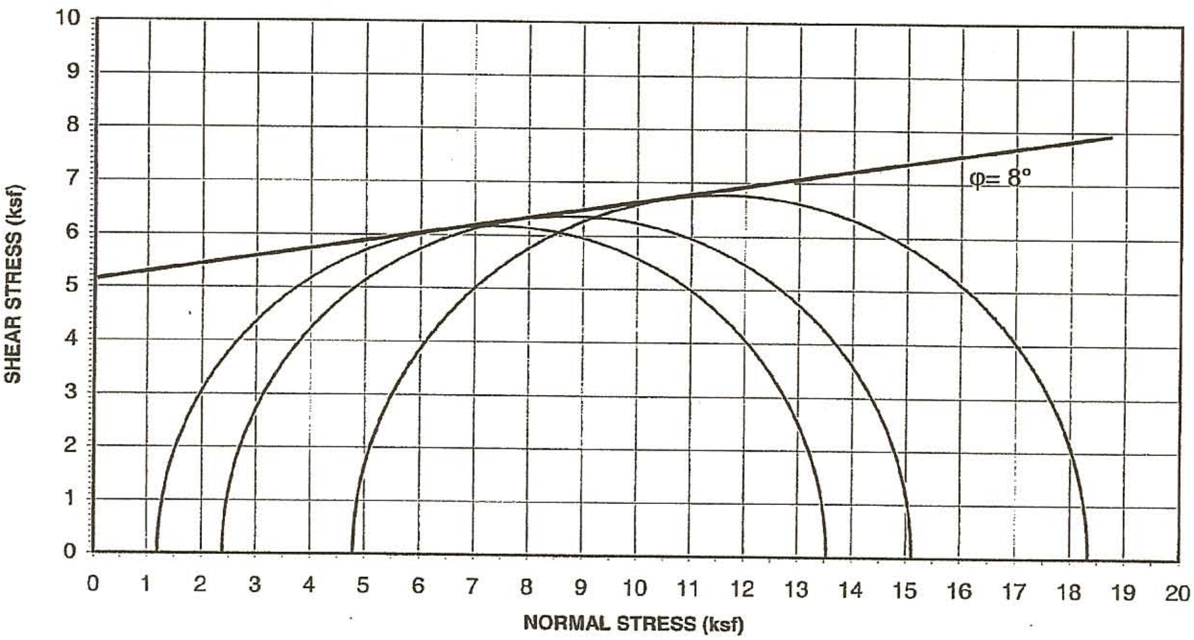
CONSOLIDATION CURVE
ASTM D 2435

Project Name: Valencia Estates
Project No.: 5166-A-SC
Date: 8/1/2006
AP No: 26-07111 **Figure No:**

AP Engineering and Testing, Inc.
Geotechnical Testing Laboratory



LEGEND: CONFINING PRESSURES= ○ 1.2 KSF □ 2.4 KSF △ 4.8 KSF



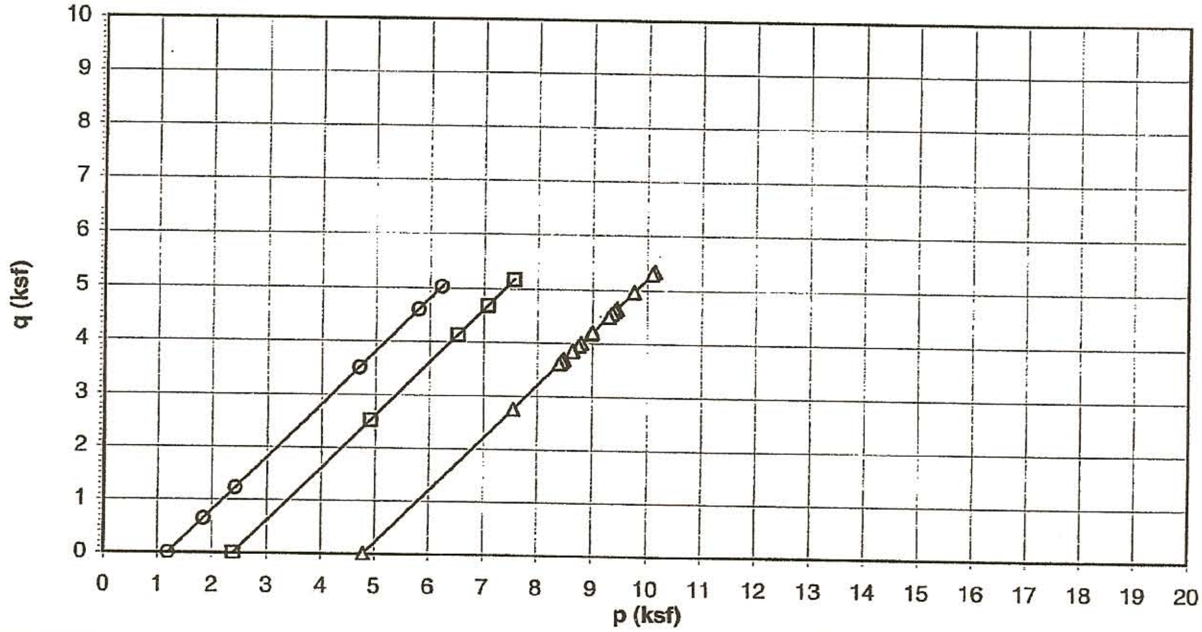
STRENGTH PARAMETERS: TOTAL STRESS: C= 5.1 ksf, φ= 8°

Project Name:	Valencia Estates	Test Condition:	As-received Mositure and Density
Project No.:	W.O.# 5166-A-SC	Sample Description:	Dk Gray Silty Claystone
Boring No.:	B-4	Avg Dry Unit Weight (pcf):	120.7
Sample No.:	-	Avg Initial Moisture Content (%):	13.0
Depth (ft):	55.0		

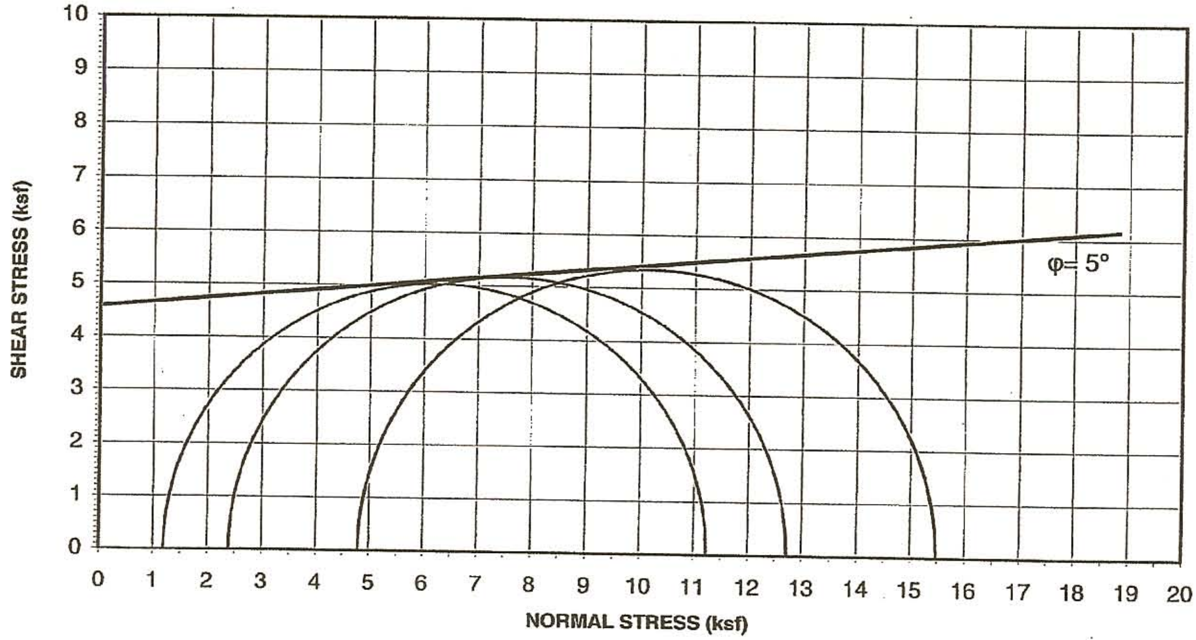
MULTI-STAGE UU TRIAXIAL TESTS

AP ENGINEERING AND TESTING, INC.

Geotechnical Testing Laboratory



LEGEND: CONFINING PRESSURES= ○ 1.2 KSF □ 2.4 KSF △ 4.8 KSF



STRENGTH PARAMETERS: TOTAL STRESS: C= 4.6 ksf, φ= 5°

Project Name:	Valencia Estates	Test Condition:	As-received Mositure and Density
Project No.:	W.O.# 5166-A-SC	Sample Description:	Yellowish Brown Silty Claystone
Boring No.:	B-3	Avg Dry Unit Weight (pcf):	121.9
Sample No.:	-	Avg Initial Moisture Content (%):	11.1
Depth (ft):	20.0		

MULTI-STAGE UU TRIAXIAL TESTS

AP ENGINEERING AND TESTING, INC.

Geotechnical Testing Laboratory

APPENDIX F

SLOPE STABILITY ANALYSIS

APPENDIX F

SLOPE STABILITY ANALYSIS
INTRODUCTION OF GSTABL7 v.2 COMPUTER PROGRAM

Introduction

GSTABL7 v.2 is a fully integrated slope stability analysis program. It permits the engineer to develop the slope geometry interactively and perform slope analysis from within a single program. The slope analysis portion of GSTABL7 v.2 uses a modified version of the popular STABL program, originally developed at Purdue University.

GSTABL7 v.2 performs a two-dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the simplified Bishop or Janbu methods. This program can be used to search for the most critical surface or the factor of safety may be determined for specific surfaces. GSTABL7, Version 2, is programmed to handle:

1. Heterogenous soil systems
2. Anisotropic soil strength properties
3. Reinforced slopes
4. Nonlinear Mohr-Coulomb strength envelope
5. Pore water pressures for effective stress analysis using:
    a. Phreatic and piezometric surfaces
    b. Pore pressure grid
    c. R factor
    d. Constant pore water pressure
6. Pseudo-static earthquake loading
7. Surcharge boundary loads
8. Automatic generation and analysis of an unlimited number of circular, noncircular and block-shaped failure surfaces
9. Analysis of right-facing slopes
10. Both SI and Imperial units

General Information

If the reviewer wishes to obtain more information concerning slope stability analysis, the following publications may be consulted initially:

1. The Stability of Slopes, by E.N. Bromhead, Surrey University Press, Chapman and Hall, N.Y., 411 pages, ISBN 412 01061 5, 1992.

2. Rock Slope Engineering, by E. Hoek and J.W. Bray, Inst. of Mining and Metallurgy, London, England, Third Edition, 358 pages, ISNB 0 900488 573, 1981.

3. Landslides: Analysis and Control, by R.L. Schuster and R.J. Krizek (editors), Special Report 176, Transportation Research Board, National Academy of Sciences, 234 pages, ISBN 0 309 02804 3, 1978.

GSTABL7 v.2 Features

The present version of GSTABL7 v.2 contains the following features:

1. Allows user to calculate factors of safety for static stability and dynamic stability situations.

2. Allows user to analyze stability situations with different failure modes.

3. Allows user to edit input for slope geometry and calculate corresponding factor of safety.

4. Allows user to readily review on-screen the input slope geometry.

5. Allows user to automatically generate and analyze unlimited number of circular, non-circular and block-shaped failure surfaces (i.e., bedding plane, slide plane, etc.).

Input Data

Input data includes the following items:

1. Unit weight, residual cohesion, residual friction angle, peak cohesion, and peak friction angle of fill material, bedding plane, and bedrock, respectively. Residual cohesion and friction angle is used for static stability analysis, where as peak cohesion and friction angle is for dynamic stability analysis.

2. Slope geometry and surcharge boundary loads.

3. Apparent dip of bedding plane can be specified in angular range (i.e., from 0 to 90 degrees.

4. Pseudo-static earthquake loading (an earthquake horizontal loading coefficient of 0.15 *i* was used in the analyses). A vertical pseudo static coefficient of 0.1 *i* was used in the analyses.

Seismic Discussion

Seismic stability analyses were approximated using a pseudo-static approach. The major difficulty in the pseudo-static approach arises from the appropriate selection of the seismic coefficient used in the analysis. The use of a static inertia force equal to this acceleration during an earthquake (rigid-body response) would be extremely conservative for several reasons including: (1) only low height, stiff/dense embankments or embankments in confined areas may respond essentially as rigid structures; (2) an earthquake's inertia force is enacted on a mass for a short time period. Therefore, replacing a transient force by a pseudo-static force representing the maximum acceleration is considered unrealistic;

(3) assuming that total pseudo-static loading is applied evenly throughout the embankment for an extended period of time is an incorrect assumption, as the length of the failure surface analyzed is usually much greater than the wave length of seismic waves generated by earthquakes; and (4) the seismic waves would place portions of the mass in compression and some in tension, resulting in only a limited portion of the failure surface analyzed moving in a downslope direction, at any one instant of time.

The coefficients usually suggested by regulating agencies, counties and municipalities are in the range of 0.05g to 0.25g. For example, past regulatory guidelines within the city and county of Los Angeles indicated that the slope stability pseudostatic coefficient = 0.15 *i*.

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki (T&S, 1986), was referenced. This method is based on empirical data and the performance of existing earth embankments during seismic loading. Our review of "Guidelines for Evaluating and Mitigating Seismic Hazards in California (Davis, 1997) indicates the State of California recommends using pseudo-static coefficient of 0.15 for design earthquakes of M 8.25 or greater and using 0.1 for earthquake parameter M 6.5. Therefore, for conservatism a horizontal seismic coefficient of 0.15 *i* was used in our analysis. A vertical seismic coefficient of two-thirds the horizontal coefficient was also used in our analyses. This is due to the higher PHSA of 0.87 g evaluated for the site. Note that although it is standard-of-practice to increase the earth material shear strengths by 20 percent to model seismic (transient) soil shear strengths, no such increase was used.

Output Information

Output information includes:

1. All input data.

2. Factors of safety for the ten most critical surfaces for static and pseudo-static stability situation.

3. High quality plots can be generated. The plots include the slope geometry, the critical surfaces and the factor of safety.

4. Note, that in the analysis, a minimum of 100 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Calculation

Table F-1 shows parameters used in slope stability calculations. Summaries of the slope stability analysis are presented in Table F-2. Surficial aslope stability calculations and detailed output information is presented in Plates F-1 to F-19. The locations of the geologic cross-sections are presented on Plate 1. The geologic cross-sections are presented on Plate 6.

Temporary Slope Stability

TABLE F-1

SOIL PARAMETERS USED

EARTH MATERIALS	CROSS BED		PARALLEL BED	
	c (psf)	Φ (degrees)	c (psf)	Φ (degrees)
Silverado, Tsl Upper	500	25	800	15
Silverado, Tsl Lower	500	25	800	15

EARTH MATERIALS	c (psf)	Φ (degrees)
Artificial Compacted Fill	500	25
Undocumented Fill	200	20
Landslide Deposit	100	20
Alluvium, Qal	250	17

TABLE F-2

SUMMARY OF SLOPE ANALYSIS

STABILITY	SLOPE CONFIGURATION	SLOPE GRADIENT	FOS		REMARKS
			STATIC	SEISMIC	
Gross A-A'**	100'w x 75'd buttress 100'w x 62'd buttress	2:1 2:1	1.83 1.74	1.15 1.16	Janbu, block original design Janbu, block modified pad elevation increase by 15 feet
Gross B-B'	50' x 40'd buttress 50' x 40'd buttress	2:1 2:1	2.26 1.97	1.38 1.35	Janbu, block w/8' surficial slump. Janbu, block mid slope and with 8' surficial slump and geogrid
Gross C-C'	50' x 35'd buttress	2:1	1.98	1.37	Janbu Block
Gross D-D'	Existing w/Residence	Existing	1.5	0.98	Janbu Block
Surficial	Fill Slopes	2:1	2.89	—	Non-Select fill c, Φ Value
Surficial	Cut Slopes, Silverado Cut Slopes, Silverado	2:1 Cross Bedding 2:1 Parallel Bedding	2.89 4.12	— ---	Composite Strength of Tsu/Tsl

*Artificial fill strength and Silverado Formation values for static and seismic utilize the same strengths for modeling of static transient seismic loads.

**Cross-Section A-A' buttress design utilizes a select soil onsite to achieve the minimum factors of safety, $c = 250$ psf and a $\phi = 30$ degrees was used in these evaluations.

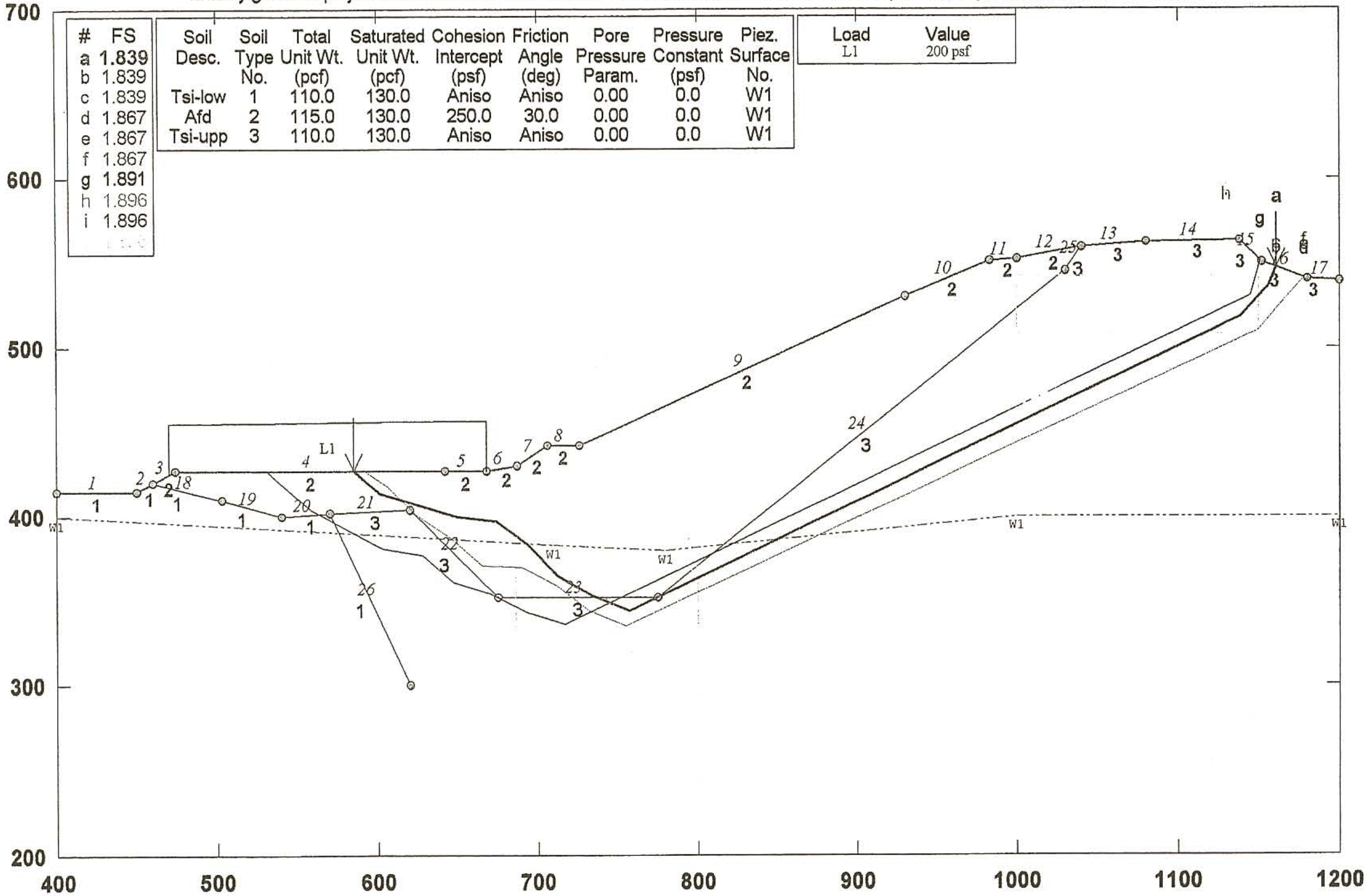
TABLE F-3

TEMPORARY SLOPE STABILITY - BACKCUTS

FORMATION A	FORMATION	TEMP FOS	REMARKS
Horizontal to Vertical 1.5:1	Silverado	1.06	Without slot cutting, Cross-Section A-A'
Horizontal to Vertical 1.5:1	Silverado	1.19	With slot cutting, Cross-Section A-A'
Horizontal to Vertical 2:1	Silverado	1.77	Without slot cutting, Cross-Section B-B'
Horizontal to Vertical 1.8:1	Silverado	1.41	Without slot cutting, Cross-Section C-C'

Valencia Estates / WO 5166 Section A-A' Fill Slope 100' w x 75'd buttress static

n:\andy\gsi 2006 project files\valencia\section aa\static\seciton aa statiic 100 x 75 buttress.pl2 Run By: AG 10/4/2006 11:29AM



GSTABL7 v.2 FSmin=1.839

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

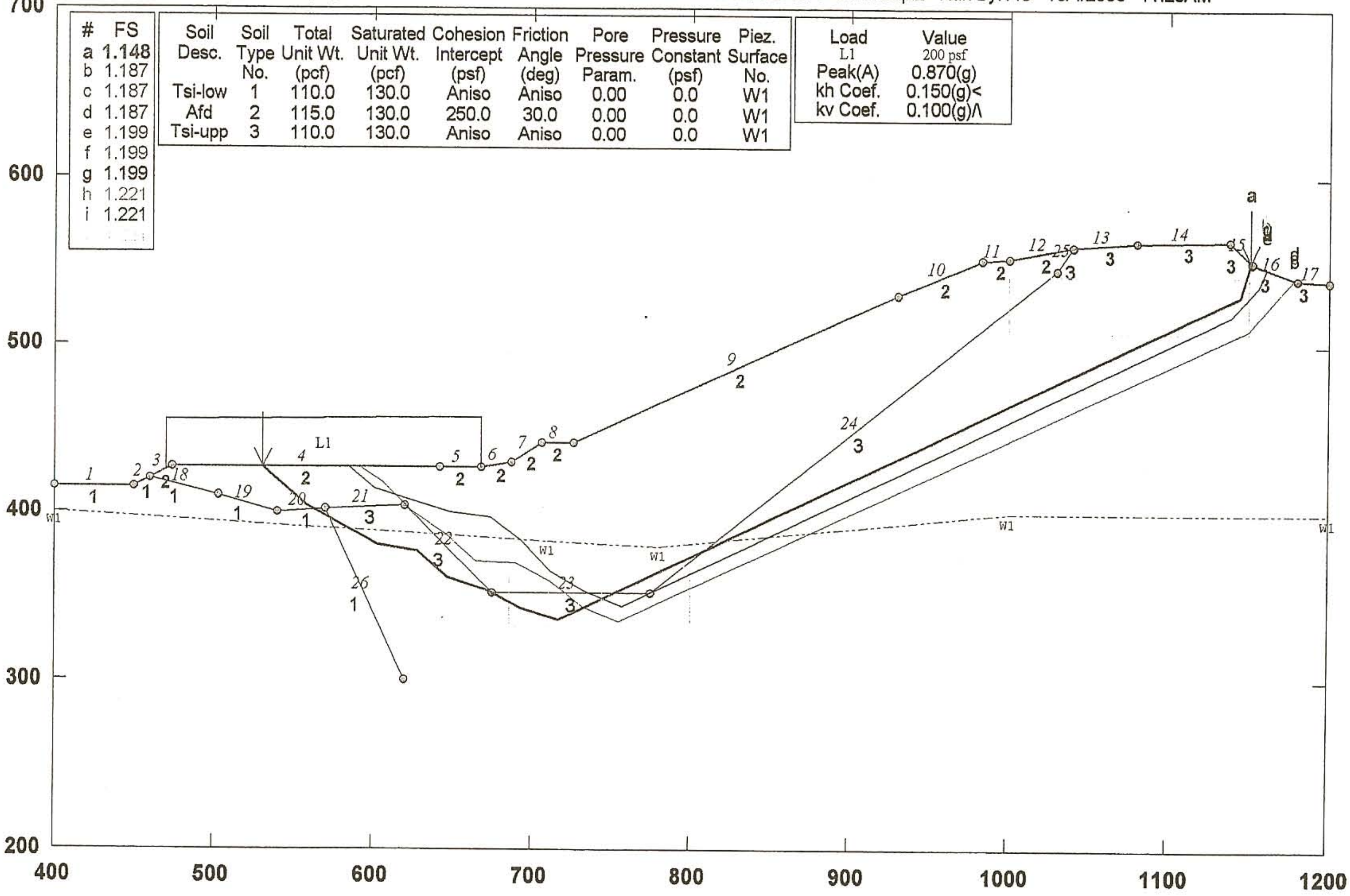
Valencia Estates / WO 5166 Section A-A' Fill Slope w/100' w x 75'd buttress seis

n:\andy\gsi 2006 project files\valencia\section aa\seismic\seciton aa seismic 100 x 75 buttress.pl2 Run By: AG 10/4/2006 11:28AM

#	FS
a	1.148
b	1.187
c	1.187
d	1.187
e	1.199
f	1.199
g	1.199
h	1.221
i	1.221

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tsi-low	1	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	2	115.0	130.0	250.0	30.0	0.00	0.0	W1
Tsi-upp	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
Peak(A)	0.870(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)Λ



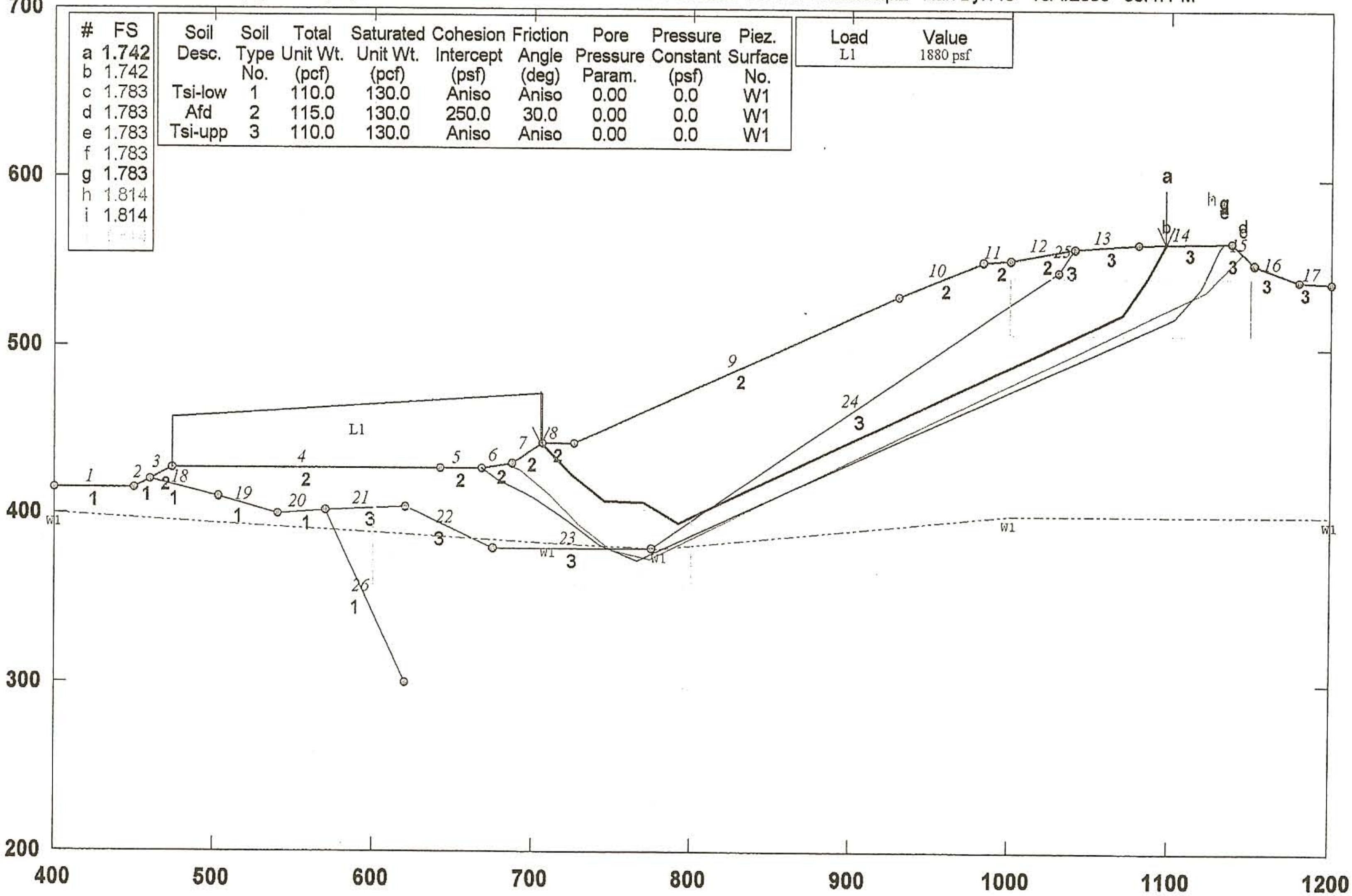
GSTABL7 v.2 FSmin=1.148

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates / WO 5166 Section A-A' Fill Slope 100' w x 62'd Btr Alt 1 stat

n:\andy\gsi 2006 project files\valencia\section aa\static\seciton aa statci 100 x 62 buttress.pl2 Run By: AG 10/4/2006 03:47PM



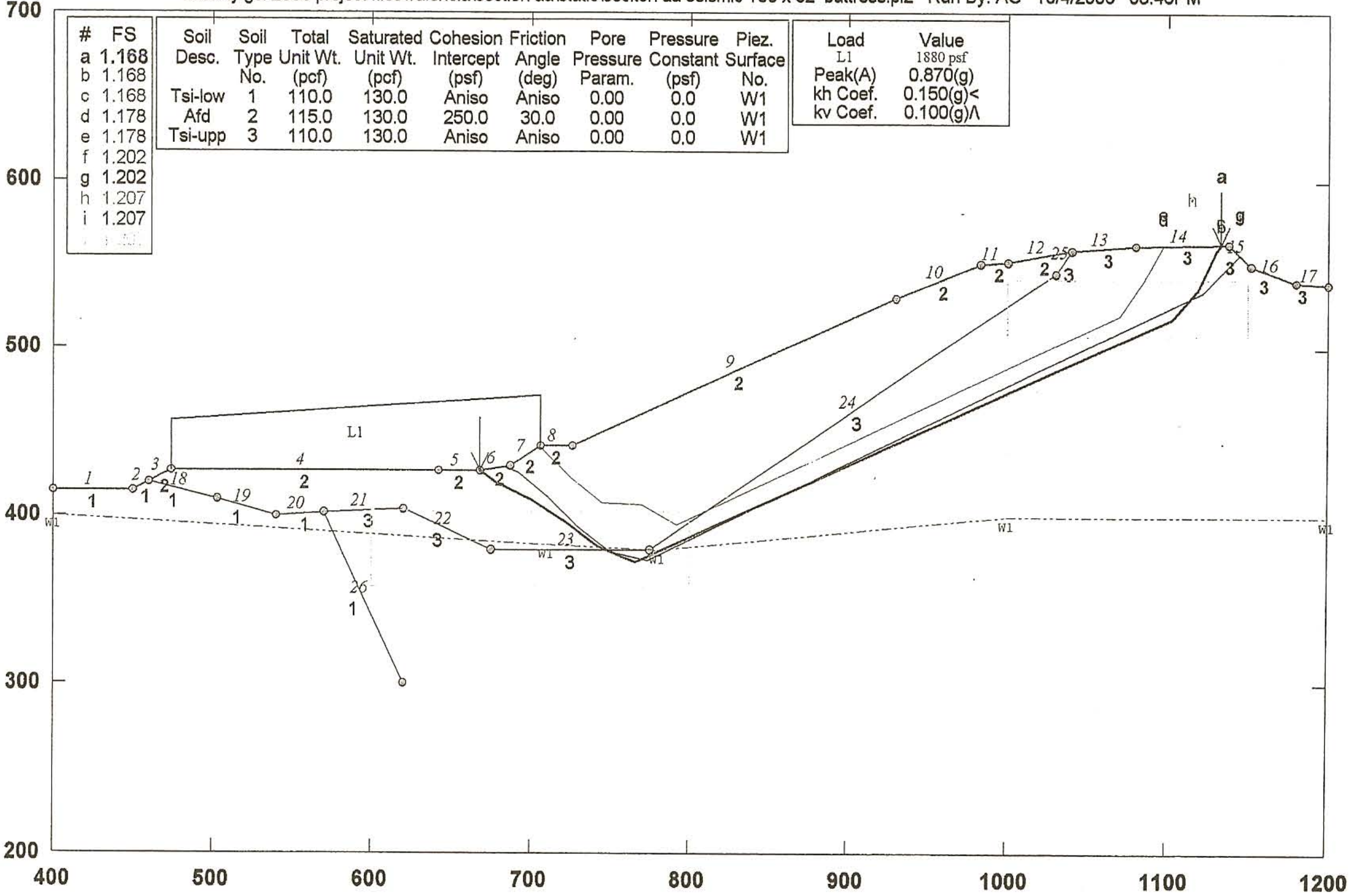
GSTABL7 v.2 FSmin=1.742

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates / WO 5166 Section A-A' Fill Slope 100' w x 62'd Btr Alt 1 seis

n:\andy\gsi 2006 project files\valencia\section aa\static\seciton aa seismic 100 x 62 buttress.pl2 Run By: AG 10/4/2006 03:46PM



#	FS
a	1.168
b	1.168
c	1.168
d	1.178
e	1.178
f	1.202
g	1.202
h	1.207
i	1.207

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tsi-low	1	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	2	115.0	130.0	250.0	30.0	0.00	0.0	W1
Tsi-upp	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	1880 psf
Peak(A)	0.870(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)Λ

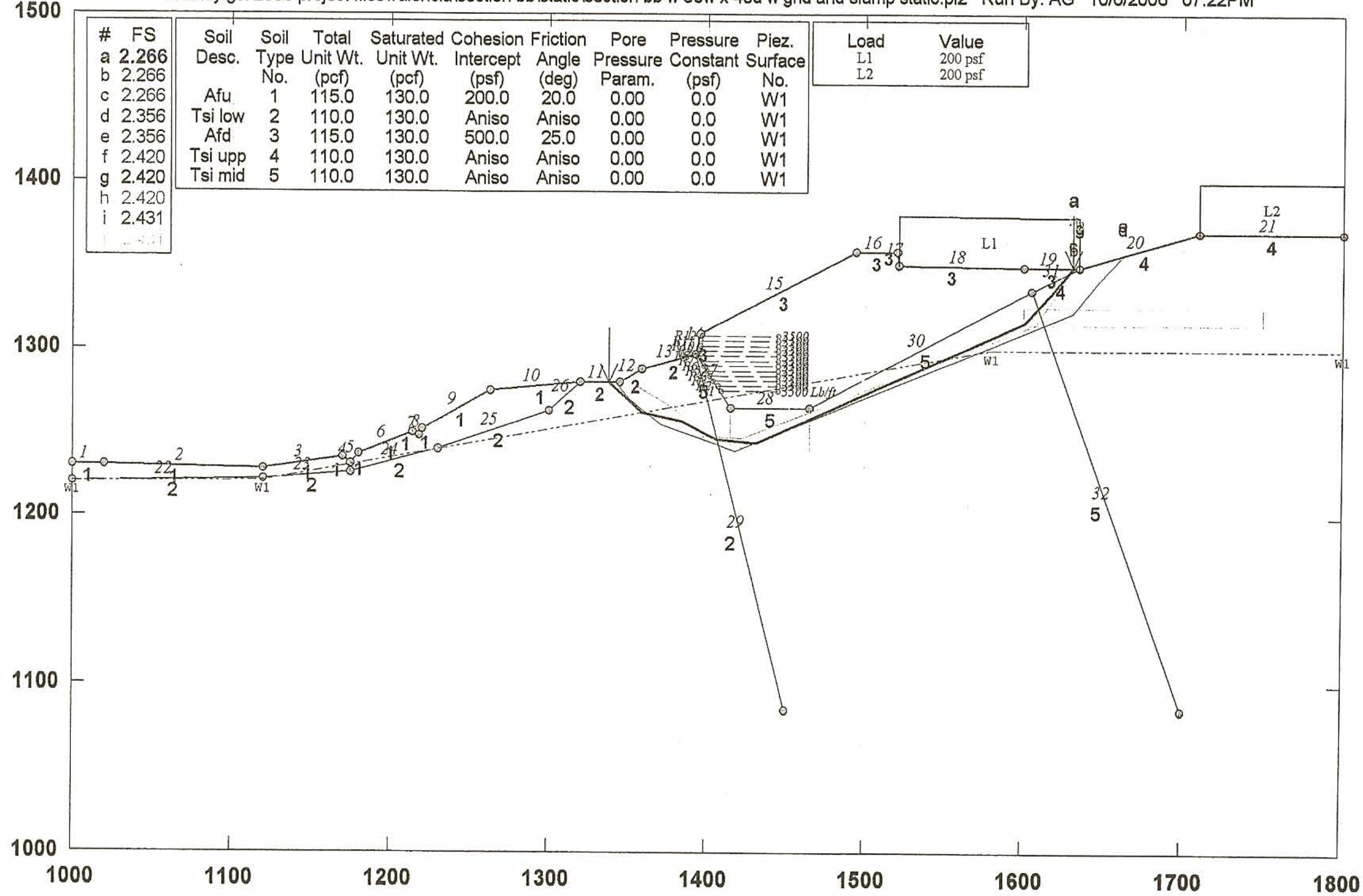
GSTABL7 v.2 FSmin=1.168

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ WO 5166 Section B-B' Planned Slpe w/50' w x 40' d key static

n:\andy\gsi 2006 project files\valencia\section bb\static\section bb w 50w x 40d w grid and slump static.pl2 Run By: AG 10/6/2006 07:22PM



#	FS
a	2.266
b	2.266
c	2.266
d	2.356
e	2.356
f	2.420
g	2.420
h	2.420
i	2.431

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afu	1	115.0	130.0	200.0	20.0	0.00	0.0	W1
Tsi low	2	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	3	115.0	130.0	500.0	25.0	0.00	0.0	W1
Tsi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi mid	5	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
L2	200 psf

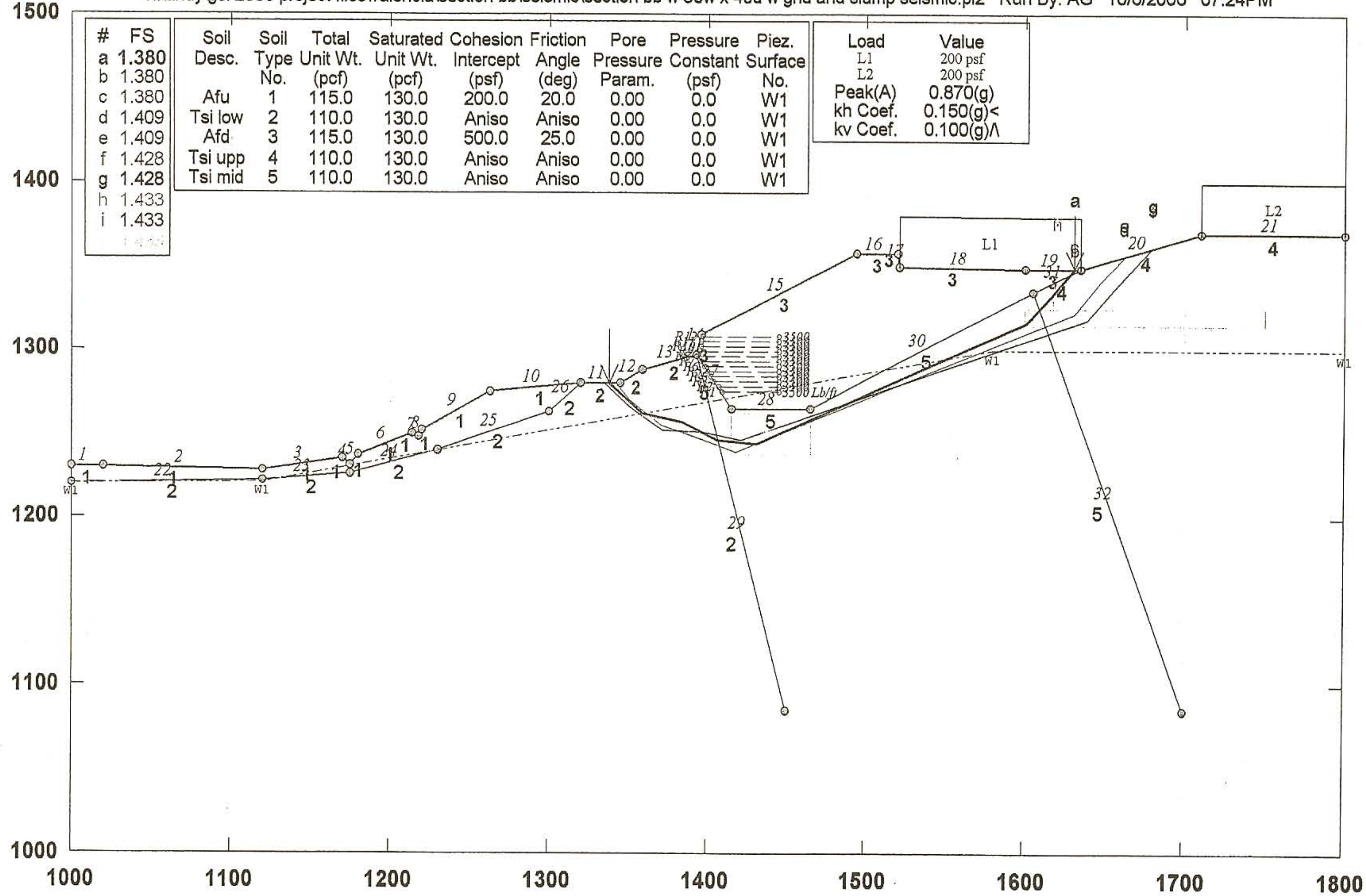
GSTABL7 v.2 FSmin=2.266

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ WO 5166 Section B-B' Planned Slpe w/50' w x 40' d key seis

n:\andy\gsi 2006 project files\valencia\section bb\seismic\section bb w 50w x 40d w grid and slump seismic.pl2 Run By: AG 10/6/2006 07:24PM



#	FS
a	1.380
b	1.380
c	1.380
d	1.409
e	1.409
f	1.428
g	1.428
h	1.433
i	1.433

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afu	1	115.0	130.0	200.0	20.0	0.00	0.0	W1
Tsi low	2	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	3	115.0	130.0	500.0	25.0	0.00	0.0	W1
Tsi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi mid	5	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
L2	200 psf
Peak(A)	0.870(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)^

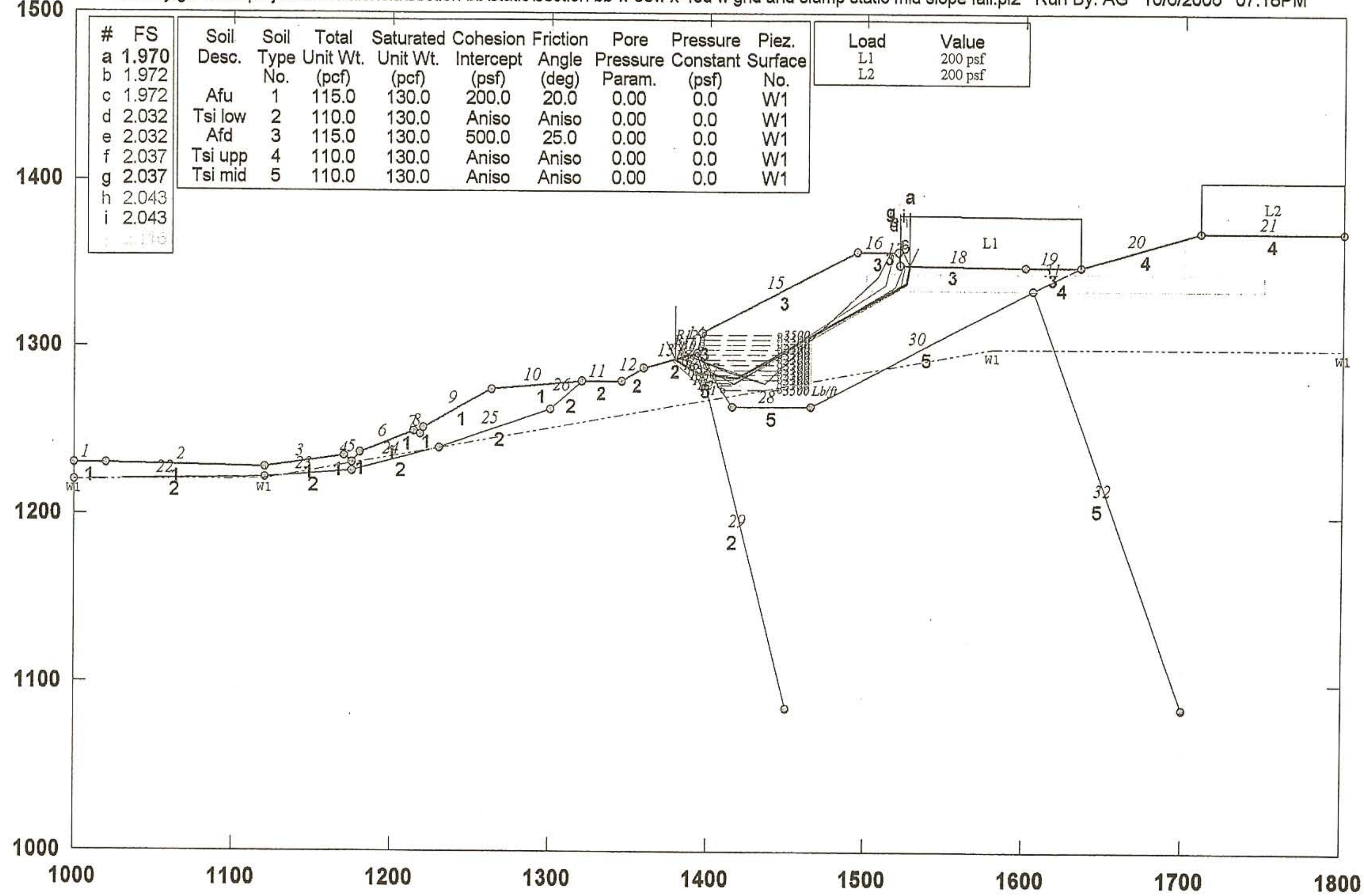
GSTABL7 v.2 FSmin=1.380

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ WO 5166 Section B-B' Planned Slpe w/50' w x 40' d key static

n:\andy\gsi 2006 project files\valencia\section bb\static\section bb w 50w x 40d w grid and slump static mid slope fail.pl2 Run By: AG 10/6/2006 07:18PM



#	FS
a	1.970
b	1.972
c	1.972
d	2.032
e	2.032
f	2.037
g	2.037
h	2.043
i	2.043

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afu	1	115.0	130.0	200.0	20.0	0.00	0.0	W1
Tsi low	2	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	3	115.0	130.0	500.0	25.0	0.00	0.0	W1
Tsi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi mid	5	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
L2	200 psf

GSTABL7 v.2 FSmin=1.970

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

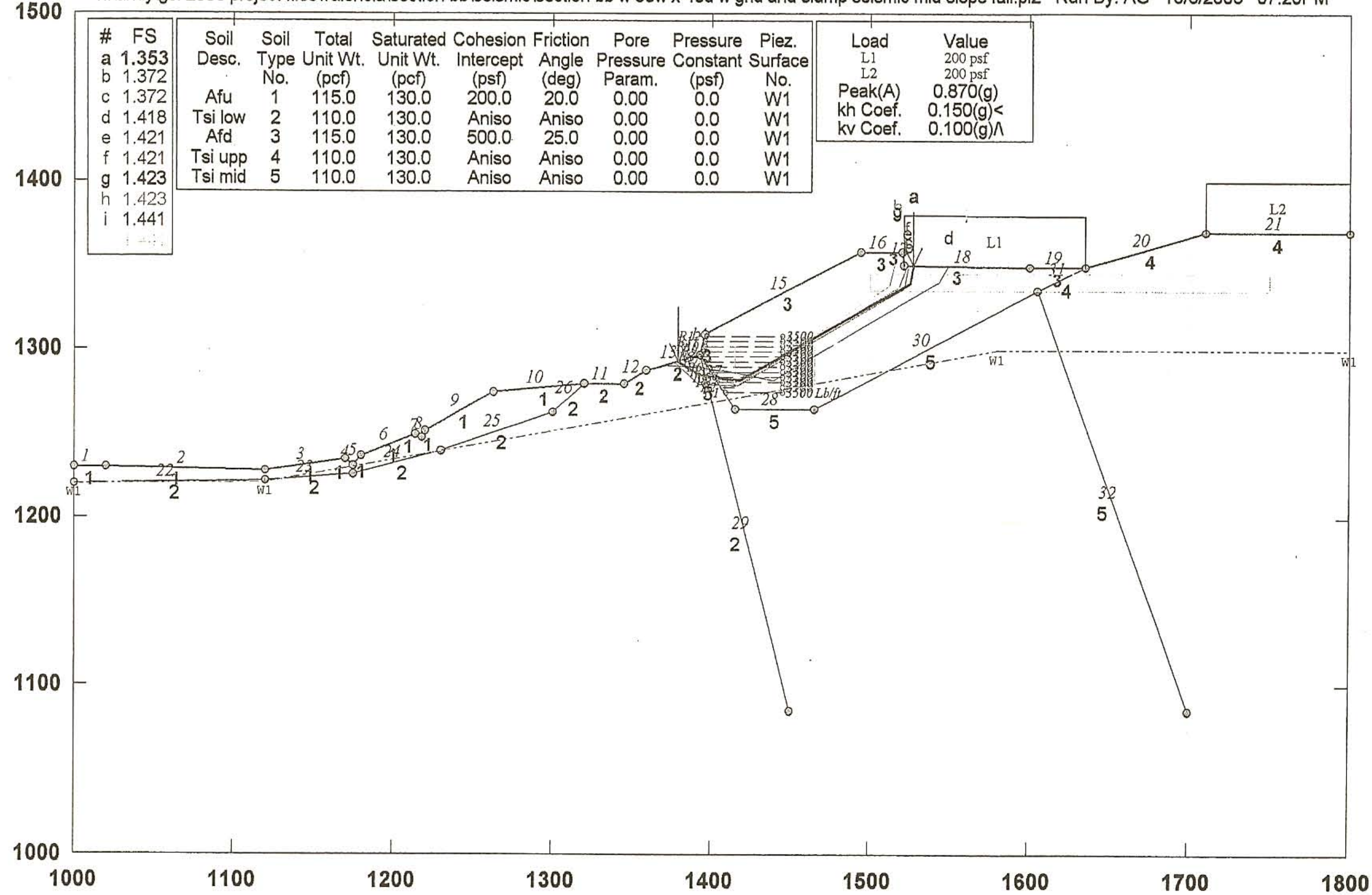
Valencia Estates/ WO 5166 Section B-B' Planned Slpe w/50' w x 40'd key seismic

n:\andy\gsi 2006 project files\valencia\section bb\seismic\section bb w 50w x 40d w grid and slump seismic mid slope fail.pl2 Run By: AG 10/6/2006 07:20PM

#	FS
a	1.353
b	1.372
c	1.372
d	1.418
e	1.421
f	1.421
g	1.423
h	1.423
i	1.441

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afu	1	115.0	130.0	200.0	20.0	0.00	0.0	W1
Tsi low	2	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	3	115.0	130.0	500.0	25.0	0.00	0.0	W1
Tsi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi mid	5	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
L2	200 psf
Peak(A)	0.870(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)^



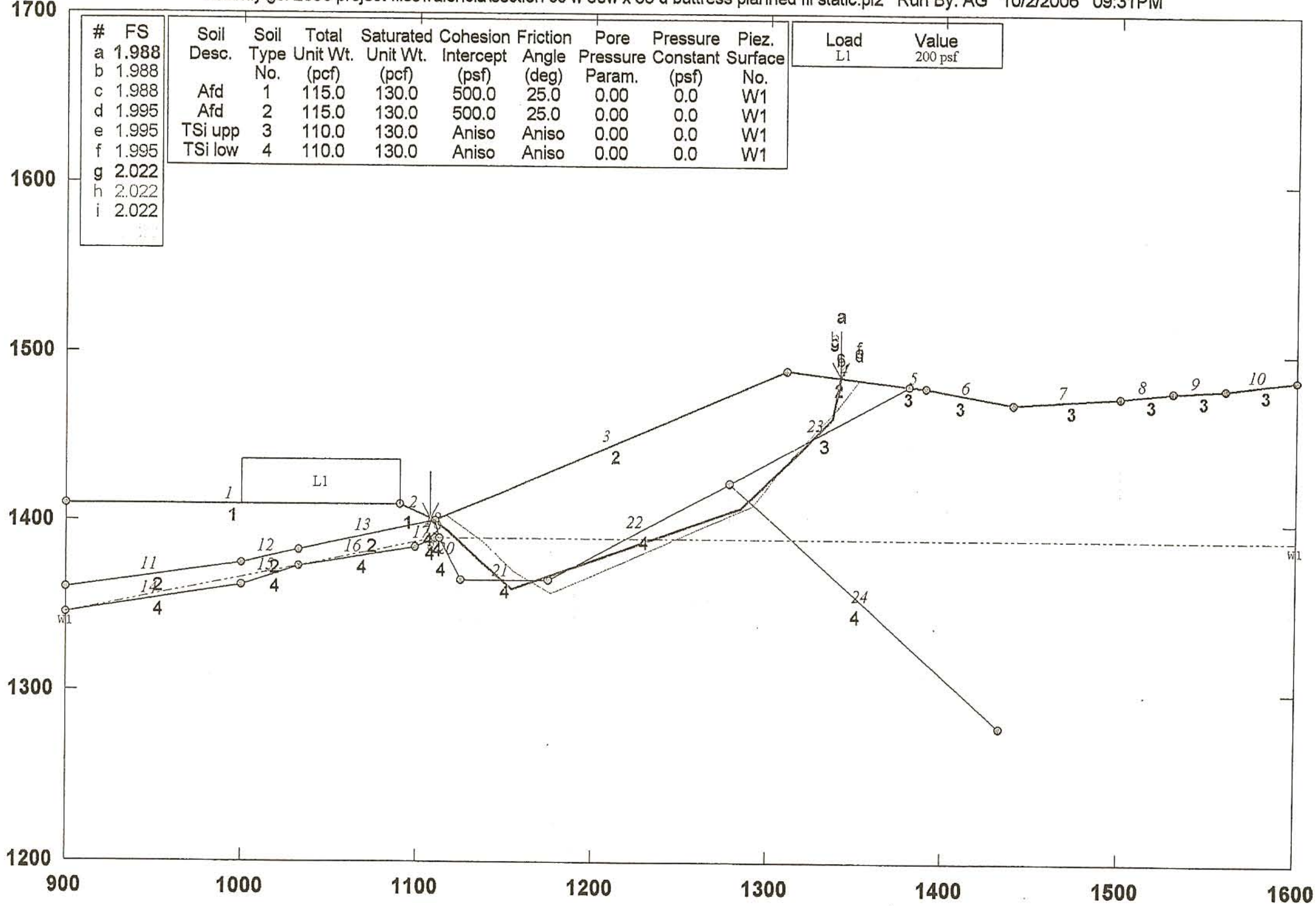
GSTABL7 v.2 FSmin=1.353

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ W.0.5166 Section C-C' Plan Slpe w/50'w x 35'd buttress static

n:\andy\gsi 2006 project files\valencia\section cc w 50w x 35 d buttress planned fil static.pl2 Run By: AG 10/2/2006 09:31PM



#	FS
a	1.988
b	1.988
c	1.988
d	1.995
e	1.995
f	1.995
g	2.022
h	2.022
i	2.022

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afd	1	115.0	130.0	500.0	25.0	0.00	0.0	W1
Afd	2	115.0	130.0	500.0	25.0	0.00	0.0	W1
TSi upp	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
TSi low	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf

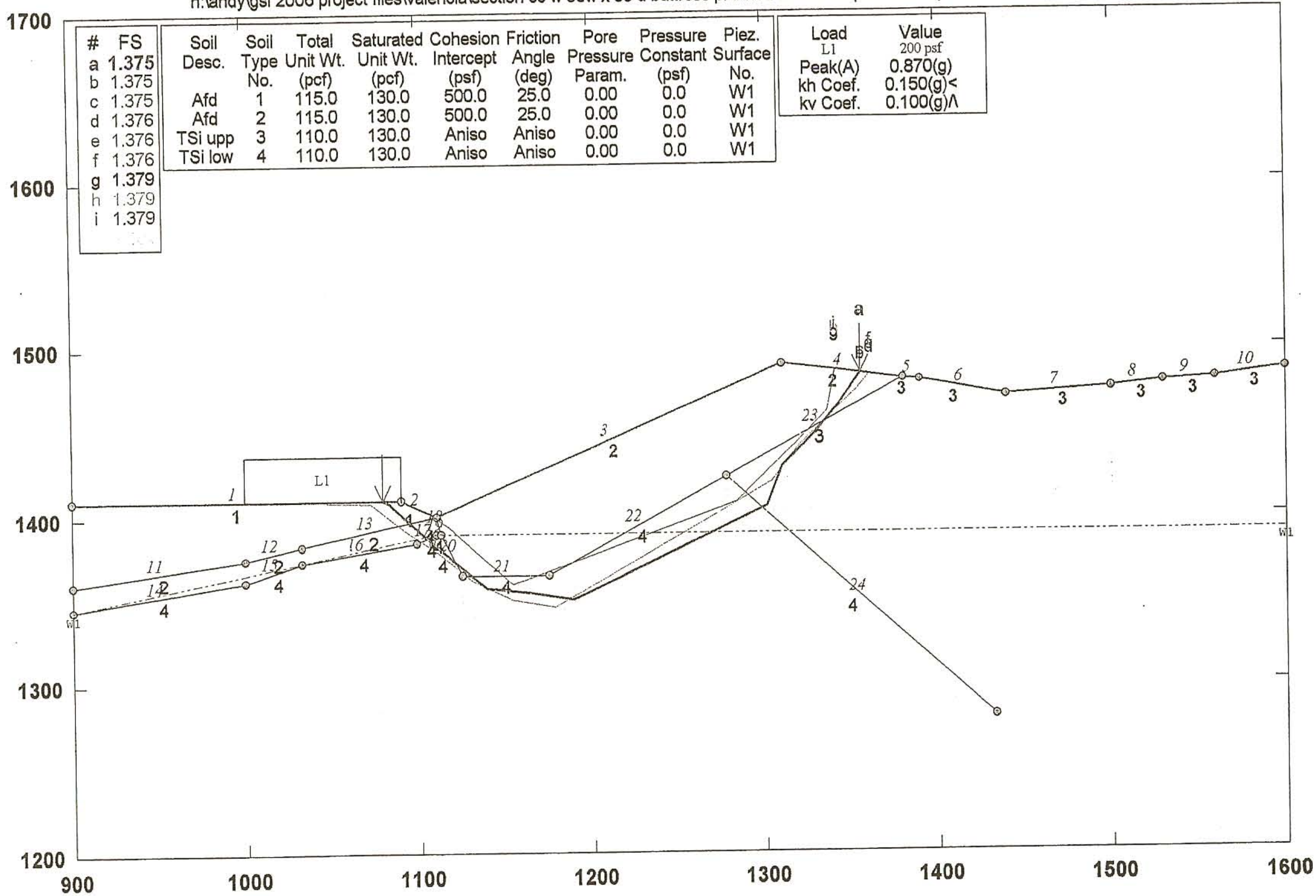
GSTABL7 v.2 FSmin=1.988

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ W.0.5166 Section C-C' Plan Slpe w/50'w x 35'd buttress seis.

n:\andy\gsi 2006 project files\valencia\section cc w 50w x 35 d buttress planned fil seismic.pl2 Run By: AG 10/2/2006 09:30PM



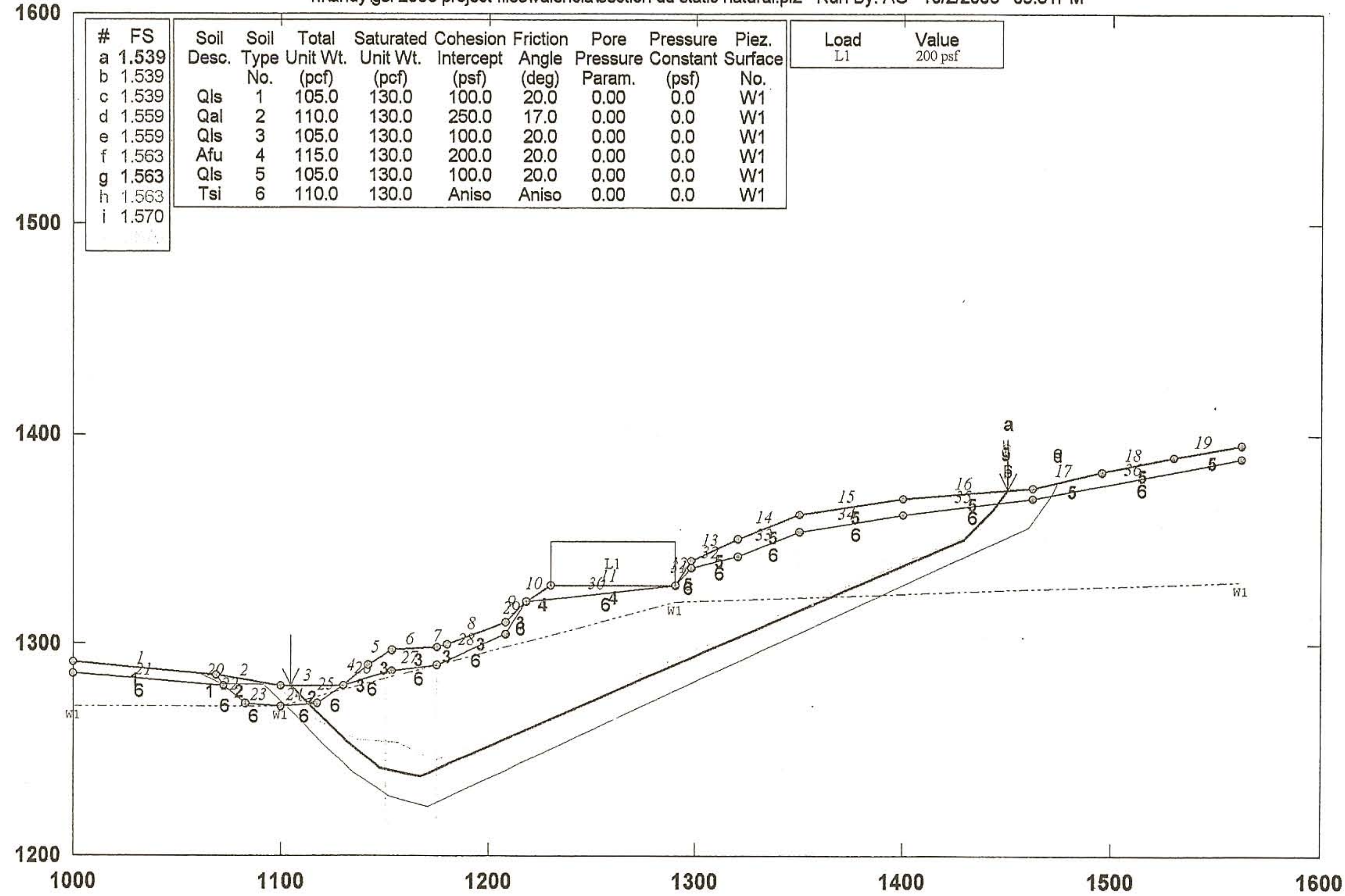
GSTABL7 v.2 FSmin=1.375

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/W.O. 5166 Section D-D' Exist. condition w/ (E) Hme/no mit. stat

n:\andy\gsi 2006 project files\valencia\section dd static natural.pl2 Run By: AG 10/2/2006 09:51PM



#	FS
a	1.539
b	1.539
c	1.539
d	1.559
e	1.559
f	1.563
g	1.563
h	1.563
i	1.570

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qls	1	105.0	130.0	100.0	20.0	0.00	0.0	W1
Qal	2	110.0	130.0	250.0	17.0	0.00	0.0	W1
Qls	3	105.0	130.0	100.0	20.0	0.00	0.0	W1
Afu	4	115.0	130.0	200.0	20.0	0.00	0.0	W1
Qls	5	105.0	130.0	100.0	20.0	0.00	0.0	W1
Tsi	6	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf

GSTABL7 v.2 FSmin=1.539

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

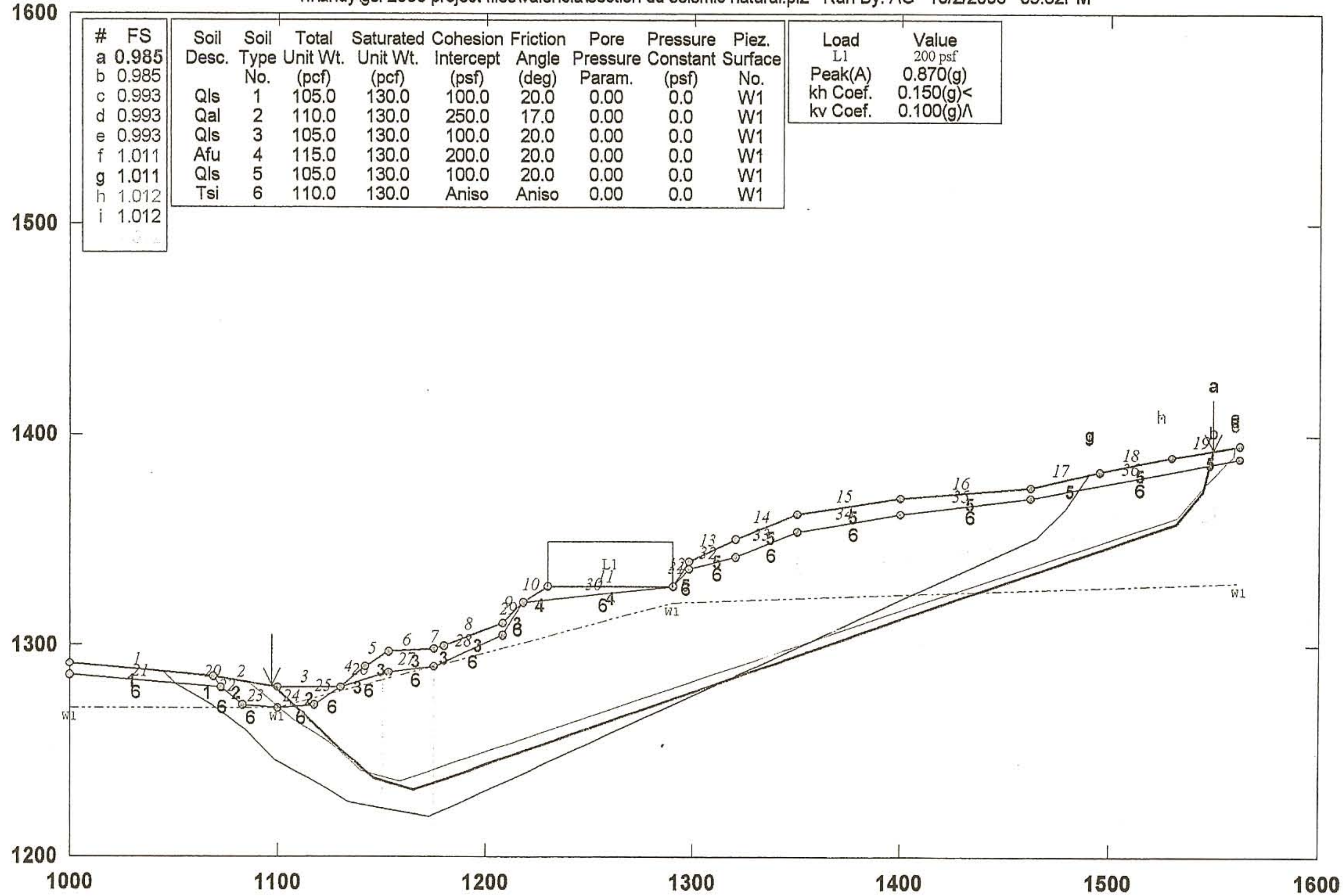
Valencia Estates/W.O. 5166 Section D-D' Exist. condition w/ (E) Hme/no mit. seis

n:\andy\gsi 2006 project files\valencia\section dd seismic natural.pl2 Run By: AG 10/2/2006 09:52PM

#	FS
a	0.985
b	0.985
c	0.993
d	0.993
e	0.993
f	1.011
g	1.011
h	1.012
i	1.012

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qls	1	105.0	130.0	100.0	20.0	0.00	0.0	W1
Qal	2	110.0	130.0	250.0	17.0	0.00	0.0	W1
Qls	3	105.0	130.0	100.0	20.0	0.00	0.0	W1
Afu	4	115.0	130.0	200.0	20.0	0.00	0.0	W1
Qls	5	105.0	130.0	100.0	20.0	0.00	0.0	W1
Tsi	6	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	200 psf
Peak(A)	0.870(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)Λ



GSTABL7 v.2 FSmin=0.985

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

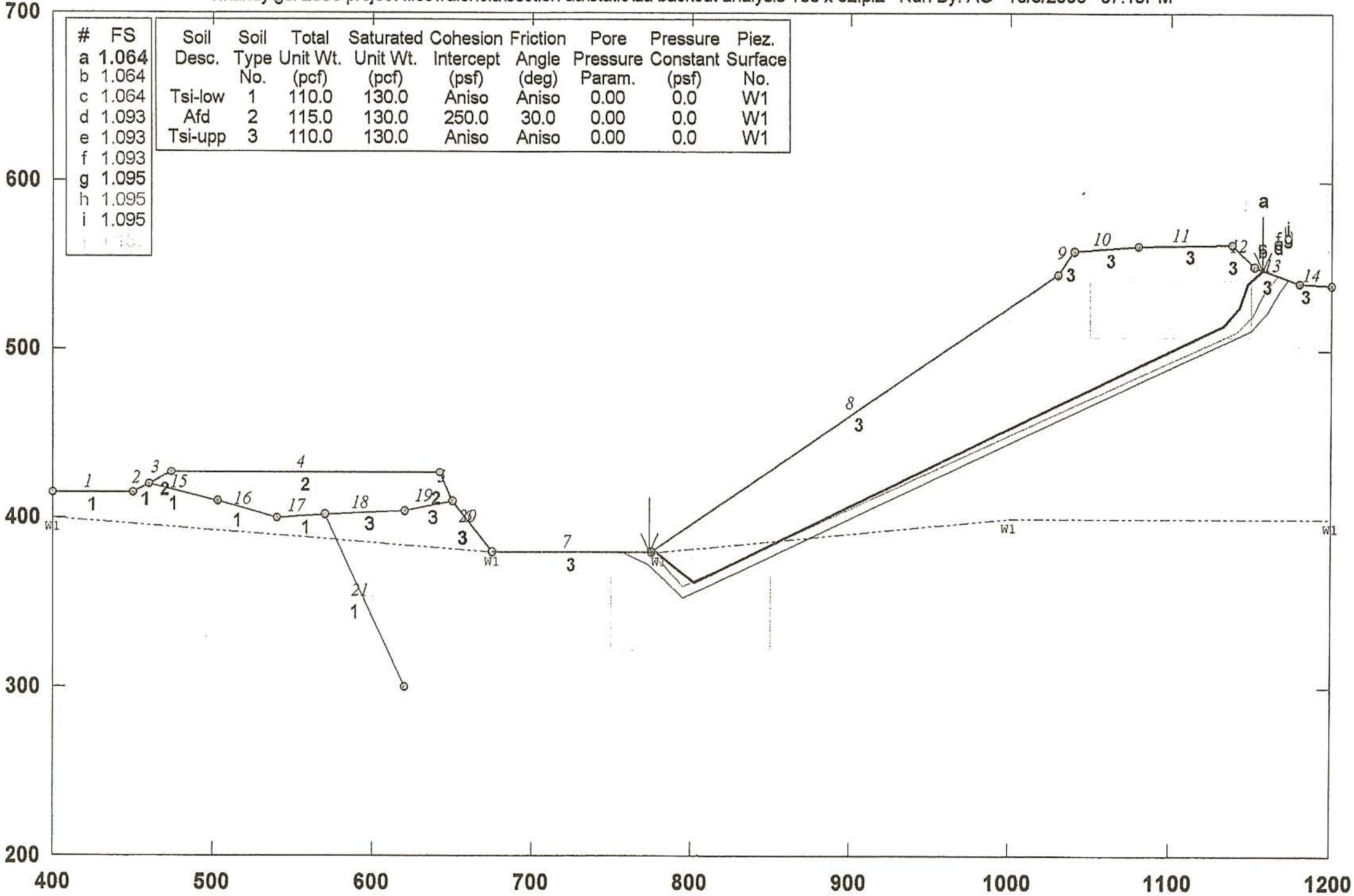
GSTABL7

GeoSoils, Inc.

Without Slot Cutting

Valencia Estates / WO 5166 Section A-A' Buttress 100' w x 62'd Backcut to PL

n:\andy\gsi 2006 project files\valencia\section aa\static\aa backcut analysis 100 x 62.pl2 Run By: AG 10/5/2006 07:19PM



#	FS
a	1.064
b	1.064
c	1.064
d	1.093
e	1.093
f	1.093
g	1.095
h	1.095
i	1.095

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tsi-low	1	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	2	115.0	130.0	250.0	30.0	0.00	0.0	W1
Tsi-upp	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.064

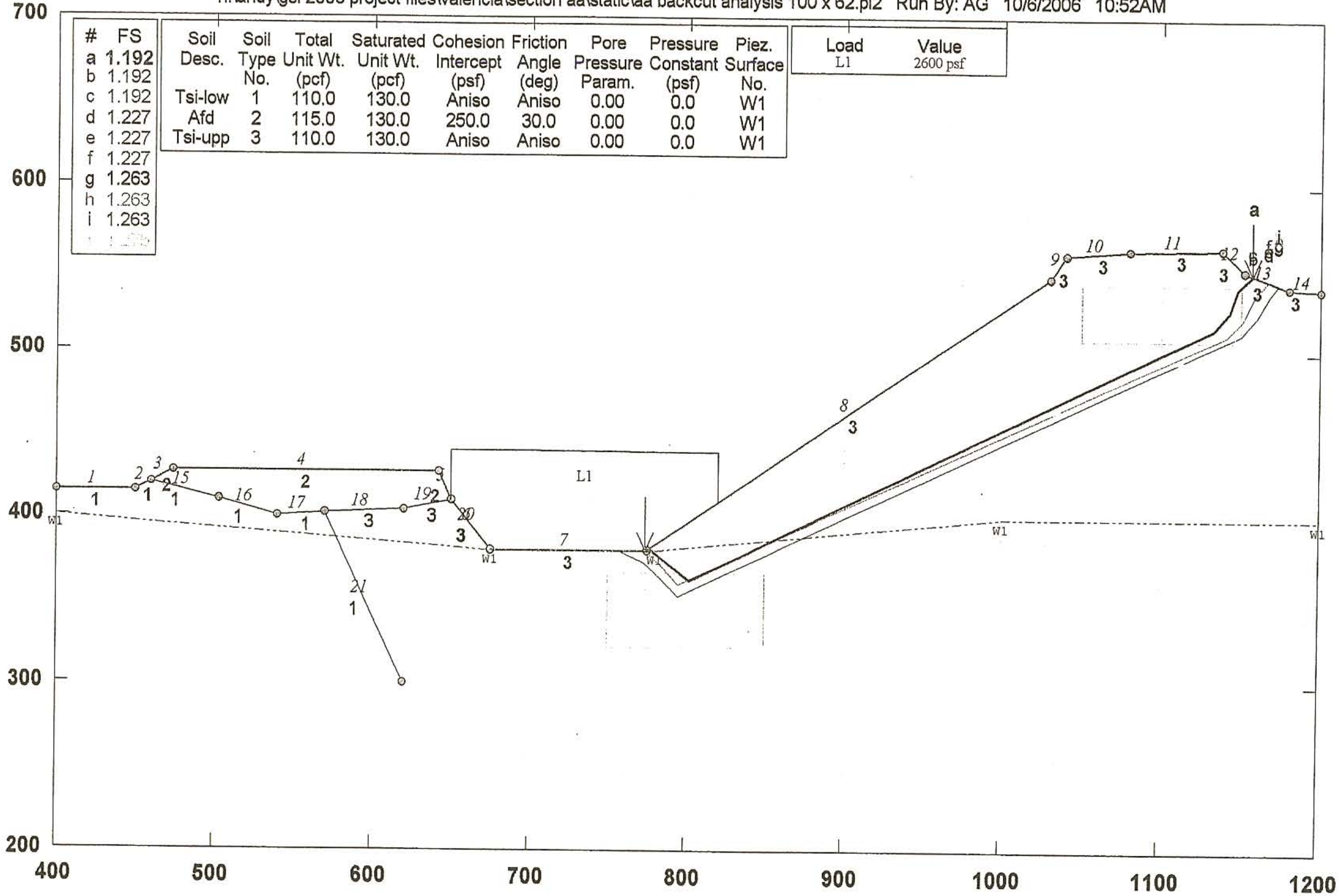
Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

With Slot Cutting

Valencia Estates / WO 5166 Section A-A' Buttress 100' w x 62'd Backcut to PL

n:\andy\gsi 2006 project files\valencia\section aa\static\aa backcut analysis 100 x 62.pl2 Run By: AG 10/6/2006 10:52AM



#	FS
a	1.192
b	1.192
c	1.192
d	1.227
e	1.227
f	1.227
g	1.263
h	1.263
i	1.263

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tsi-low	1	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Afd	2	115.0	130.0	250.0	30.0	0.00	0.0	W1
Tsi-upp	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

Load	Value
L1	2600 psf

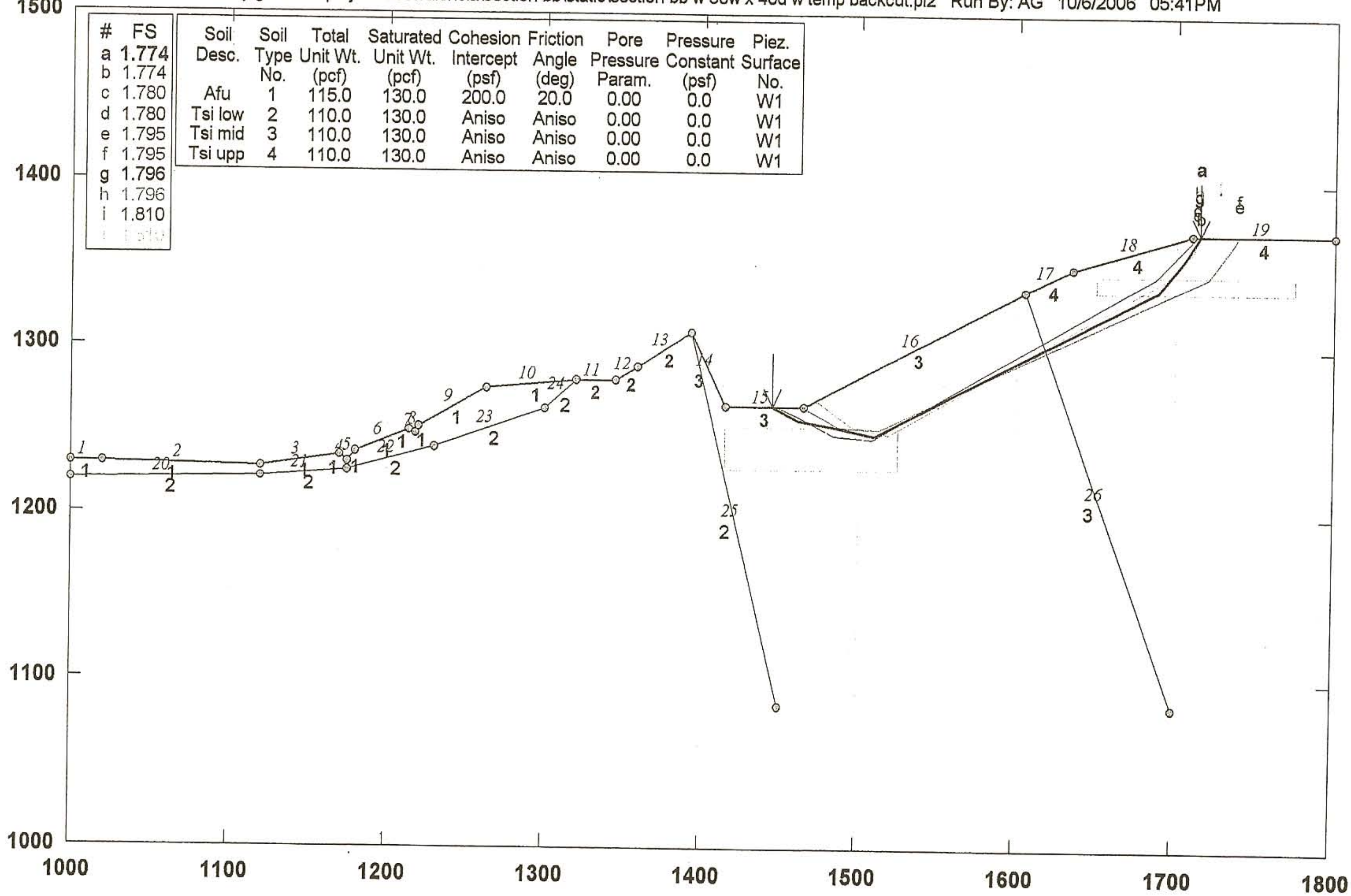
GSTABL7 v.2 FSmin=1.192

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7

Valencia Estates/ WO 5166 Section B-B' Planned Slpe temp key backcut

n:\andy\gsi 2006 project files\valencia\section bb\static\section bb w 50w x 40d w temp backcut.pl2 Run By: AG 10/6/2006 05:41PM



#	FS
a	1.774
b	1.774
c	1.780
d	1.780
e	1.795
f	1.795
g	1.796
h	1.796
i	1.810

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afu	1	115.0	130.0	200.0	20.0	0.00	0.0	W1
Tsi low	2	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi mid	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
Tsi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.774

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

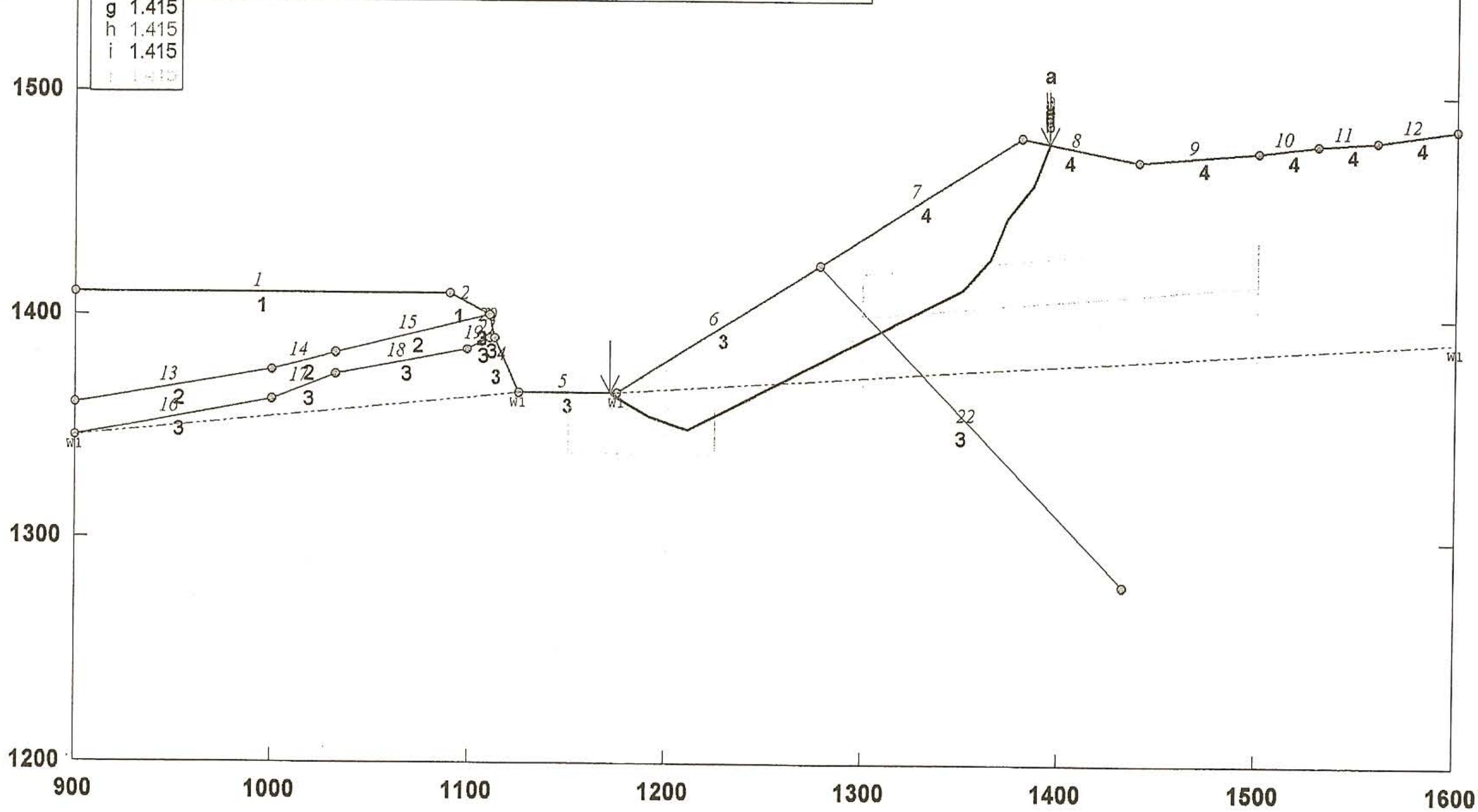
GSTABL7

Valencia Estates/ W.0.5166 Section C-C' Plan Slpe w/50'w x 35'd backcut static

n:\andy\gsi 2006 project files\valencia\section cc\static\section cc w 50w x 35 d temp backcut.pl2 Run By: AG 10/6/2006 03:41PM

#	FS
a	1.415
b	1.415
c	1.415
d	1.415
e	1.415
f	1.415
g	1.415
h	1.415
i	1.415

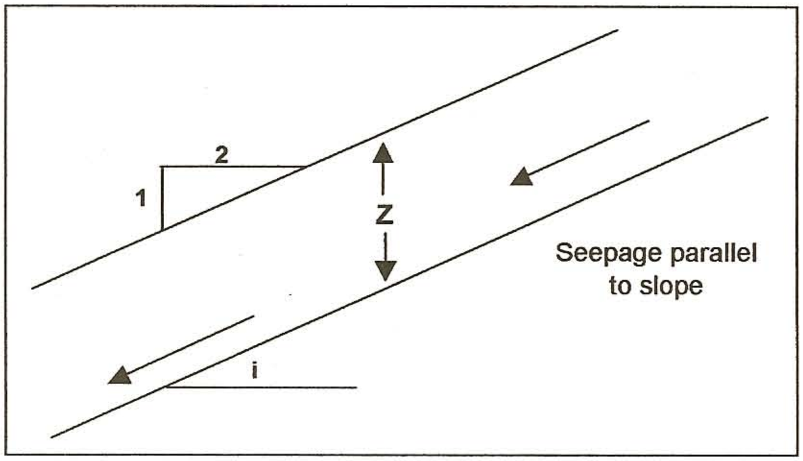
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Afd	1	115.0	130.0	500.0	25.0	0.00	0.0	W1
Afd	2	115.0	130.0	500.0	25.0	0.00	0.0	W1
TSi low	3	110.0	130.0	Aniso	Aniso	0.00	0.0	W1
TSi upp	4	110.0	130.0	Aniso	Aniso	0.00	0.0	W1



GSTABL7 v.2 FSmin=1.415
Safety Factors Are Calculated By The Simplified Janbu Method for the case of phi=0

GSTABL7

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Valencia Estates
Material Type:	Artificial Fill Soil

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	130	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	25	degrees
Apparent Cohesion (C)	500	lb/ft^2

Fs = Static Safety Factor = $$\frac{z\,(\gamma_{sat}-\gamma_w)\,Cos^2(i)\,Tan\,(\phi) + C}{z\,(\gamma_{sat})\,Sin\,(i)\,Cos\,(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	2.89

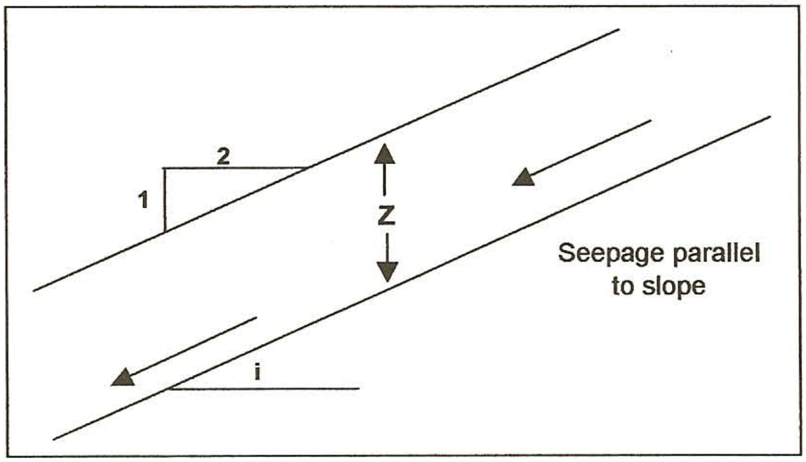
GeoSoils, Inc. W.O. 5166-A-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

Figure F-17

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Valencia Estates
Material Type:	Silverado Formation upper/ lower
	Cross Bed Exposed in Cut/ surficial

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	130	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	25	degrees
Apparent Cohesion (C)	500	lb/ft^2

Fs = Static Safety Factor = $$\frac{z\,(\gamma_{sat}-\gamma_w)\,\text{Cos}^2(i)\,\text{Tan}\,(\phi) + C}{z\,(\gamma_{sat})\,\text{Sin}\,(i)\,\text{Cos}\,(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	2.89

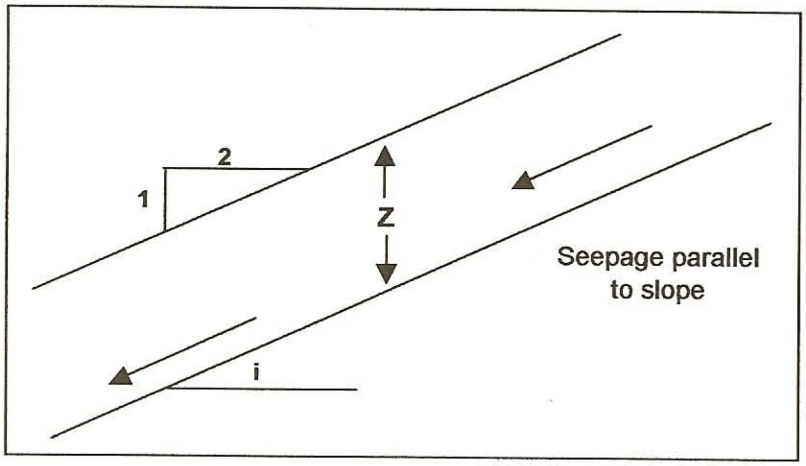
GeoSoils, Inc. W.O. 5166-A-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

Figure F-18

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Valencia Estates
Material Type:	Silverado Formation upper/ lower
	Parallel Bed Exposed in Cut/ surficial

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	130	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	15	degrees
Apparent Cohesion (C)	800	lb/ft^2

Fs = Static Safety Factor = $$\frac{z\,(\gamma_{sat}-\gamma_w)\,\mathrm{Cos}^2(i)\,\mathrm{Tan}\,(\phi) + C}{z\,(\gamma_{sat})\,\mathrm{Sin}\,(i)\,\mathrm{Cos}\,(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	4.12

GeoSoils, Inc. W.O. 5166-A-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

Figure F-19

APPENDIX G

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the geotechnical report are part of the earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications. The project soil engineer and engineering geologist (geotechnical consultant), or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report, the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that determination may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the project engineering geologist and/or soil engineer prior to placing and fill. It is the contractor's responsibility to notify the engineering geologist and soil engineer when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D-1556, D-2937 or D-2922, and D-3017, at intervals of approximately ±2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the soil engineer, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the soil engineer. The contractor should also remove all non-earth material considered unsatisfactory by the soil engineer.

It is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading guidelines, codes or agency ordinances, and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, determined by the soil engineer or engineering geologist as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the soil engineer.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the soil engineer. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the soil engineer before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified to a minimum depth of 6 to 8 inches, or as directed by the soil engineer. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site soils engineer and/or engineering geologist. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the soil engineer and/or engineering geologist. In fill over cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the soil engineer, the minimum width of fill keys should be approximately equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the soil engineer and/or engineering geologist prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been determined to be suitable by the soil engineer. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the soil engineer. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the soil engineer. Oversized material should be taken offsite, or placed in accordance with recommendations of the soil engineer in areas designated as suitable for rock disposal. Per the UBC/CBC, oversized material should not be placed within 10 feet vertically of finish grade (elevation) or within 20 feet horizontally of slope faces (any variation will require prior approval from the governing agency).

To facilitate future trenching, rock (or oversized material) should not be placed within 10 feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the soil engineer and/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the soil engineer to determine it's physical properties and suitability for use onsite. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the soil engineer as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The soil engineer may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as determined by ASTM test designation D-1557, or as otherwise recommended by the soil engineer. Compaction equipment should be adequately sized and should be specifically designed for soil compaction or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and/or moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the soil engineer.

In general, per the UBC/CBC, fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final determination of fill slope compaction should be based on observation and/or testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ±2 to ±8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to

achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

6. Erosion control and drainage devices should be designed by the project civil engineer in compliance with ordinances of the controlling governmental agencies, and/or in accordance with the recommendation of the soil engineer or engineering geologist.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The soil engineer and/or engineering geologist may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the engineering geologist. If directed by the engineering geologist, further excavations or overexcavation and refilling of cut areas should be performed, and/or remedial grading of cut slopes should be performed. When fill over cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the engineering geologist prior to placement of materials for construction of the fill portion of the slope. The engineering geologist should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the engineering geologist and soil engineer should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the engineering geologist, whether anticipated or not.

Unless otherwise specified in soil and geological reports, no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the soil engineer and engineering geologist have finished their observations of the work, final reports should be submitted subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the soil engineer and/or engineering geologist.

All finished cut and fill slopes should be protected from erosion and/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as practical after completion of grading.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation, and Clearance

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

Trench and Vertical Excavation

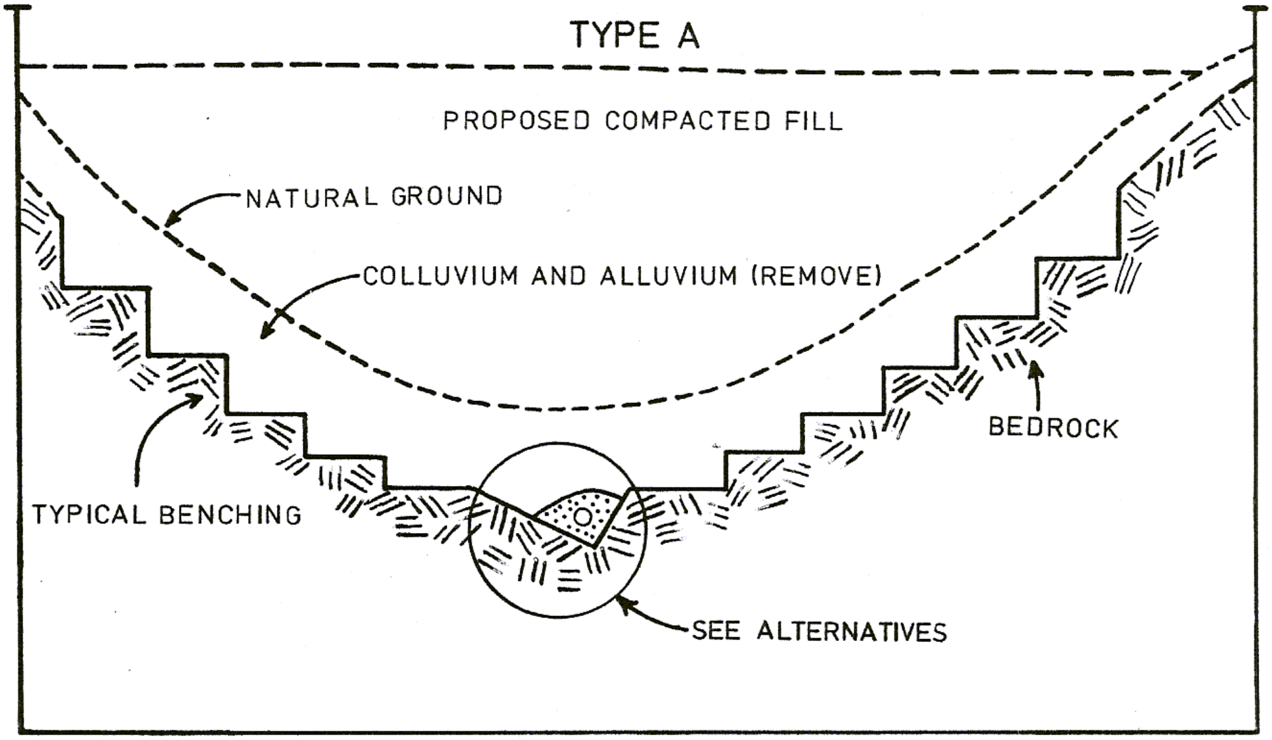
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

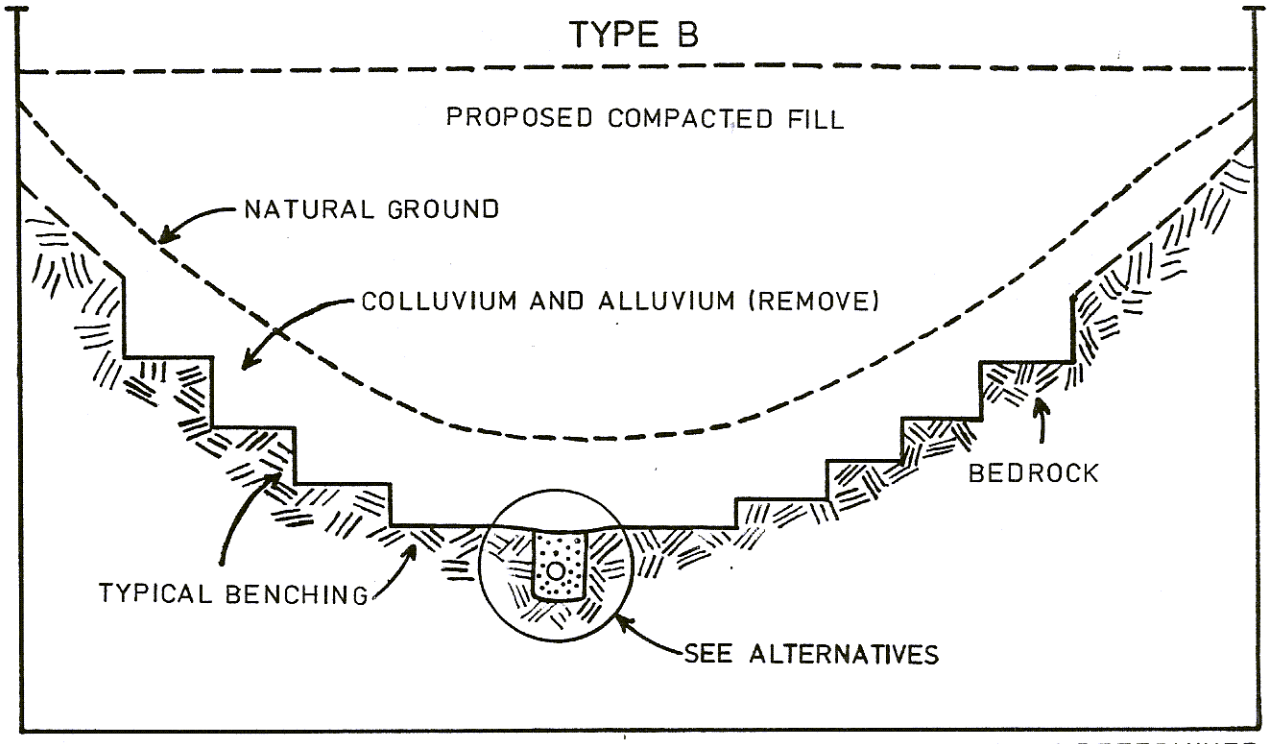
All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with CAL-OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing and/or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify CAL-OSHA and/or the proper controlling authorities.

CANYON SUBDRAIN DETAIL



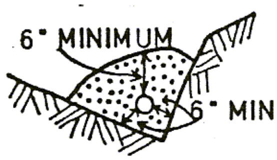


NOTE: ALTERNATIVES, LOCATION AND EXTENT OF SUBDRAINS SHOULD BE DETERMINED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST DURING GRADING.

PLATE EG—1

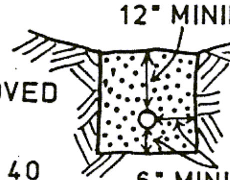
CANYON SUBDRAIN ALTERNATE DETAILS

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A—1

FILTER MATERIAL: MINIMUM VOLUME OF 9 $FT.^3$ /LINEAR FT. 6" Ø ABS OR PVC PIPE OR APPROVED SUBSTITUTE WITH MINIMUM 8 (1/4" Ø) PERFS. LINEAR FT. IN BOTTOM HALF OF PIPE.
ASTM D2751, SDR 35 OR ASTM D1527, SCHD. 40
ASTM D3034, SDR 35 OR ASTM D1785, SCHD. 40
FOR CONTINUOUS RUN IN EXCESS OF 500 FT. USE 8" Ø PIPE

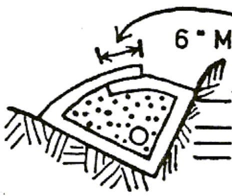


B—1

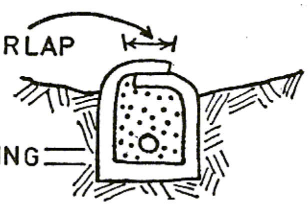
FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1 INCH	100
3/4 INCH	90—100
3/8 INCH	40—100
NO. 4	25—40
NO. 8	18—33
NO. 30	5—15
NO. 50	0—7
NO. 200	0—3

ALTERNATE 2: PERFORATED PIPE, GRAVEL AND FILTER FABRIC



A—2



B—2

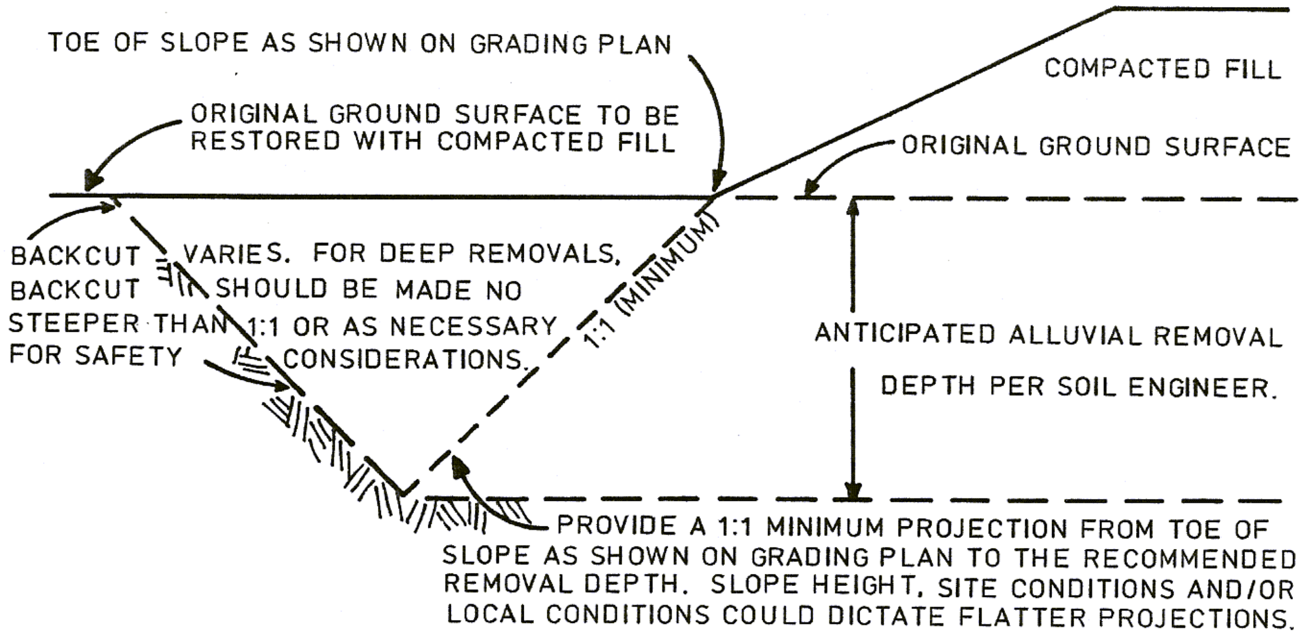
GRAVEL MATERIAL 9 FT^3/LINEAR FT.

PERFORATED PIPE: SEE ALTERNATE 1

GRAVEL: CLEAN 3/4 INCH ROCK OR APPROVED SUBSTITUTE

FILTER FABRIC: MIRAFI 140 OR APPROVED SUBSTITUTE

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON



REMOVAL ADJACENT TO EXISTING FILL

ADJOINING CANYON FILL

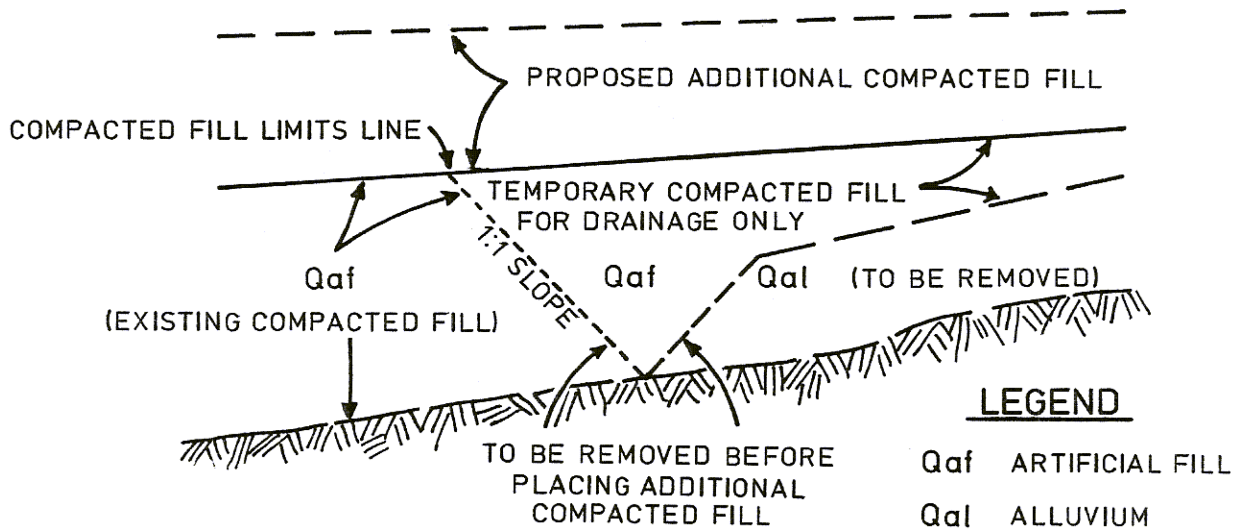


PLATE EG—3

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

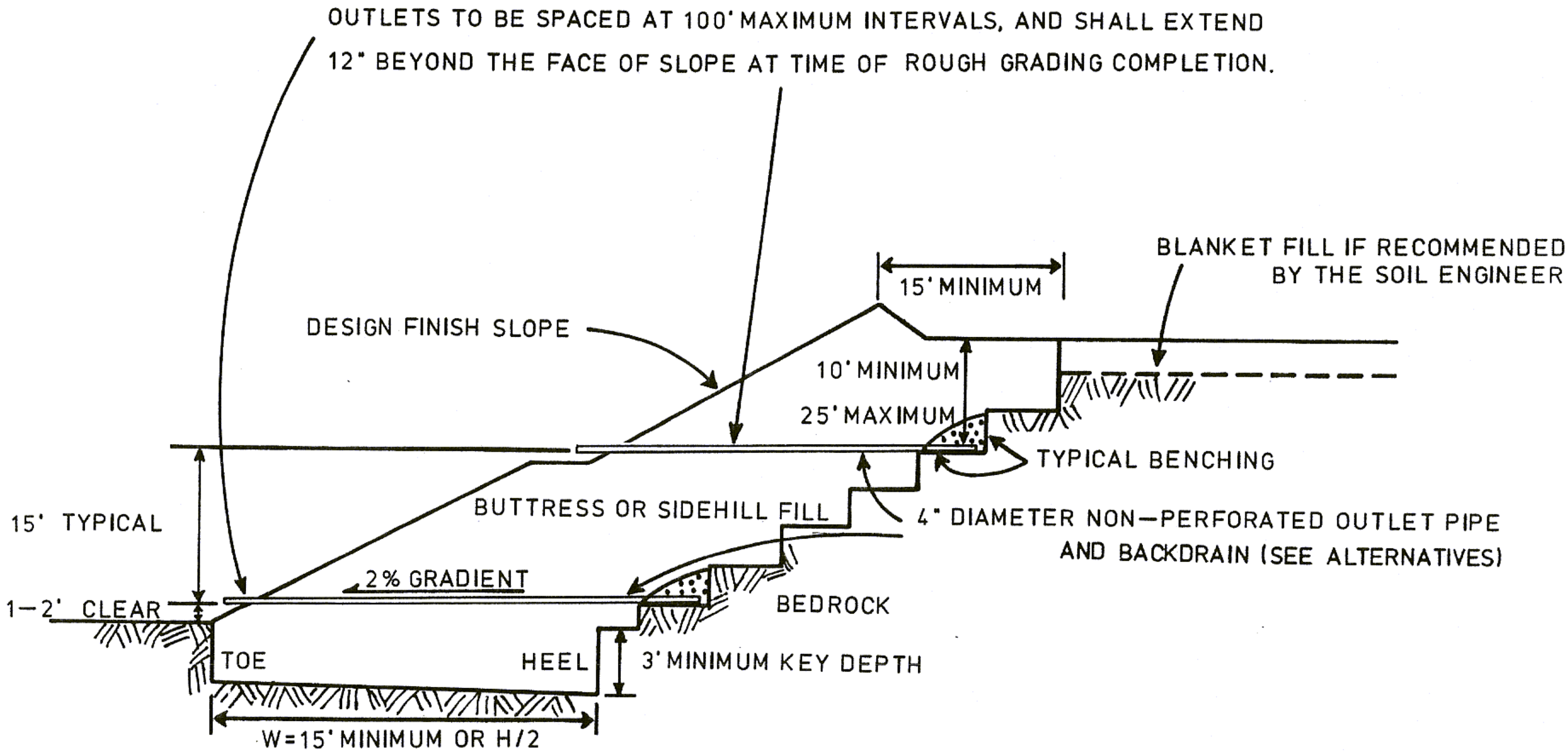
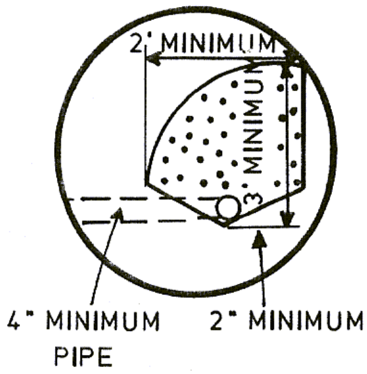
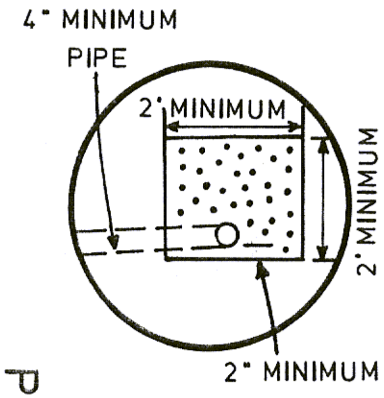


PLATE EG-4

TYPICAL STABILIZATION / BUTTRESS SUBDRAIN DETAIL





<u>FILTER MATERIAL:</u> MINIMUM OF FIVE Ft^3/LINEAR Ft OF PIPE OR FOUR Ft^3/LINEAR Ft OF PIPE WHEN PLACED IN SQUARE CUT TRENCH.

<u>ALTERNATIVE IN LIEU OF FILTER MATERIAL</u>: GRAVEL MAY BE ENCASED IN APPROVED FILTER FABRIC. FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12" ON ALL JOINTS.

<u>MINIMUM 4" DIAMETER PIPE:</u> ABS—ASTM D—2751, SDR 35 OR ASTM D—1527 SCHEDULE 40 PVC—ASTM D—3034, SDR 35 OR ASTM D—1785 SCHEDULE 40 WITH A CRUSHING STRENGTH OF 1,000 POUNDS MINIMUM, AND A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS OF BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2% TO OUTLET PIPE. OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW.

NOTE:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON—SITE SOIL.
2. BACKDRAINS AND LATERAL DRAINS SHALL BE LOCATED AT ELEVATION OF EVERY BENCH DRAIN. FIRST DRAIN LOCATED AT ELEVATION JUST ABOVE LOWER LOT GRADE. ADDITIONAL DRAINS MAY BE REQUIRED AT THE DISCRETION OF THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST.

<u>FILTER MATERIAL</u> SHALL BE OF THE FOLLOWING SPECIFICATION OR AN APPROVED EQUIVALENT:

SIEVE SIZE	PERCENT PASSING
1 INCH	100
3/4 INCH	90—100
3/8 INCH	40—100
NO. 4	25—40
NO. 8	18—33
NO. 30	5—15
NO. 50	0—7
NO. 200	0—3

<u>GRAVEL</u> SHALL BE OF THE FOLLOWING SPECIFICATION OR AN APPROVED EQUIVALENT:

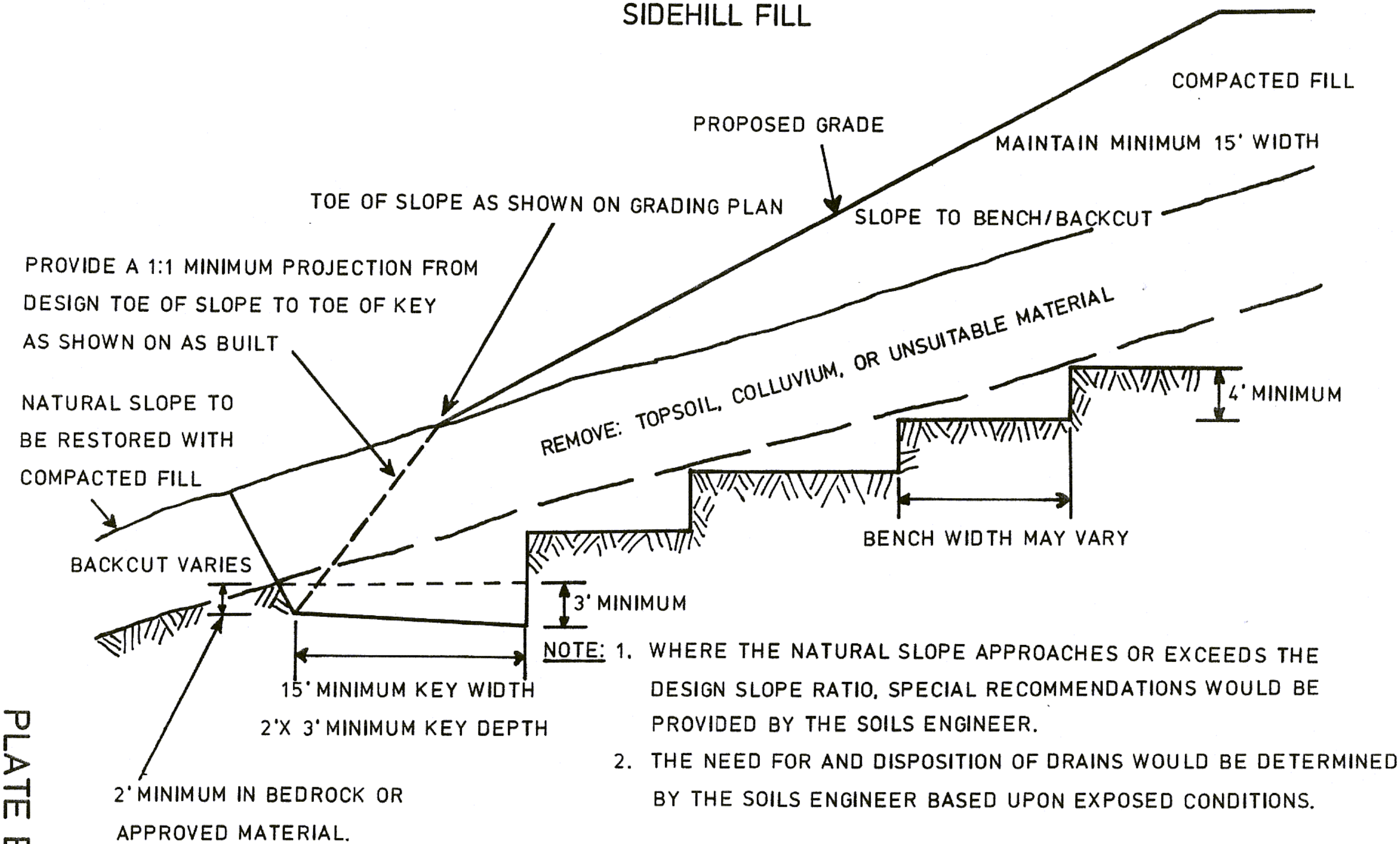
SIEVE SIZE	PERCENT PASSING
1 1/2 INCH	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT: MINIMUM OF 50

PLATE EG—5

FILL OVER NATURAL DETAIL

SIDEHILL FILL

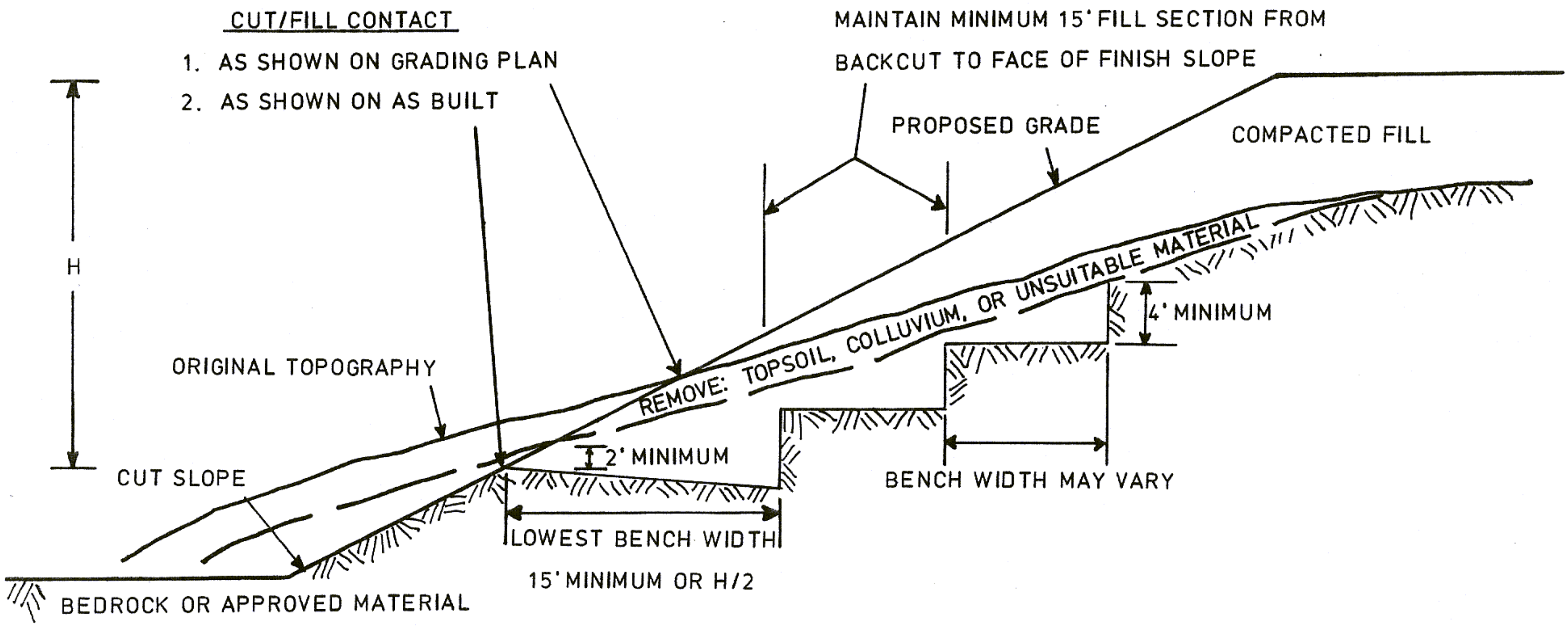


NOTE: 1. WHERE THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN SLOPE RATIO, SPECIAL RECOMMENDATIONS WOULD BE PROVIDED BY THE SOILS ENGINEER.

2. THE NEED FOR AND DISPOSITION OF DRAINS WOULD BE DETERMINED BY THE SOILS ENGINEER BASED UPON EXPOSED CONDITIONS.

PLATE EG—6

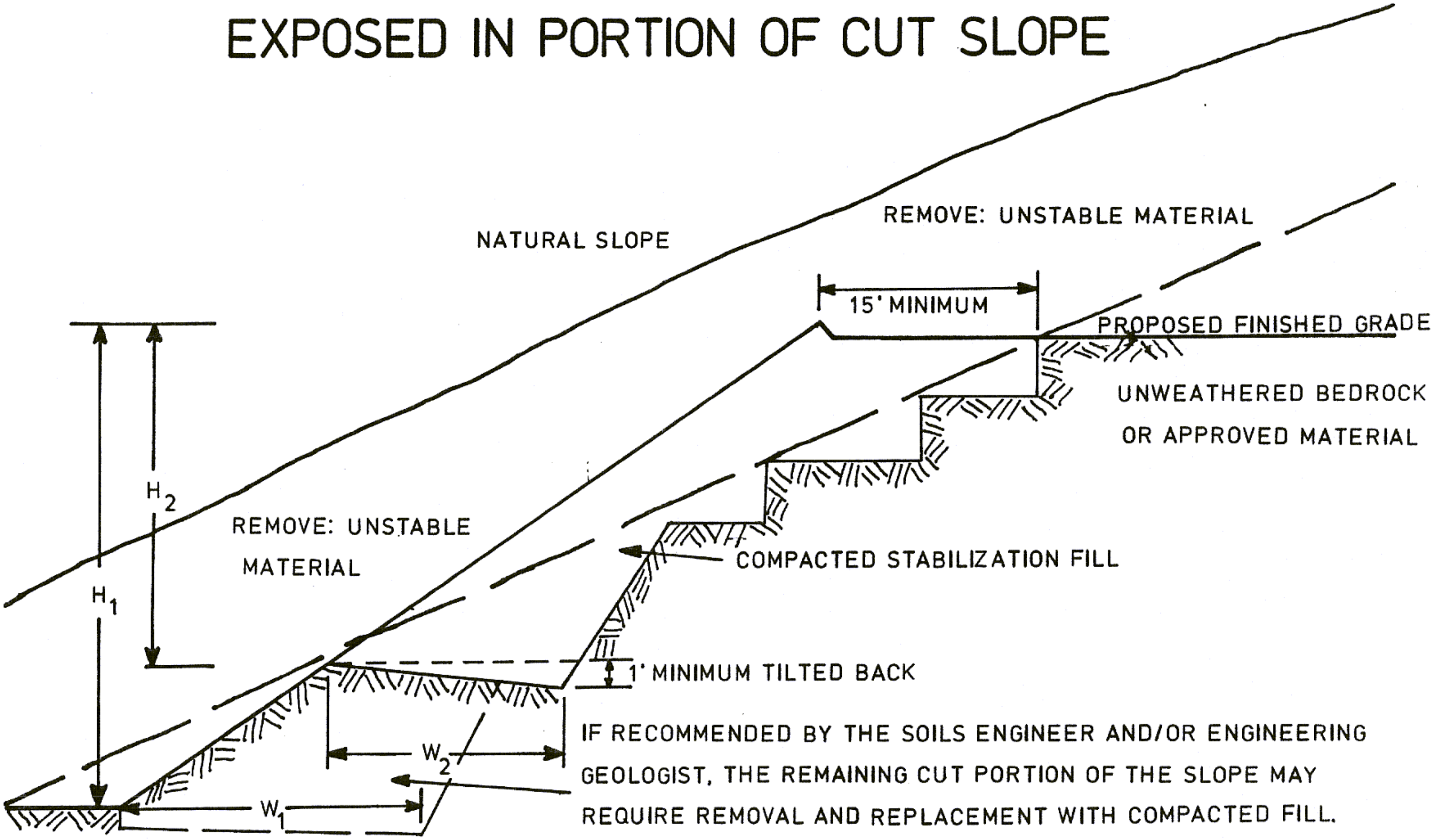
FILL OVER CUT DETAIL



NOTE: THE CUT PORTION OF THE SLOPE SHOULD BE EXCAVATED AND EVALUATED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST PRIOR TO CONSTRUCTING THE FILL PORTION.

PLATE EG–7

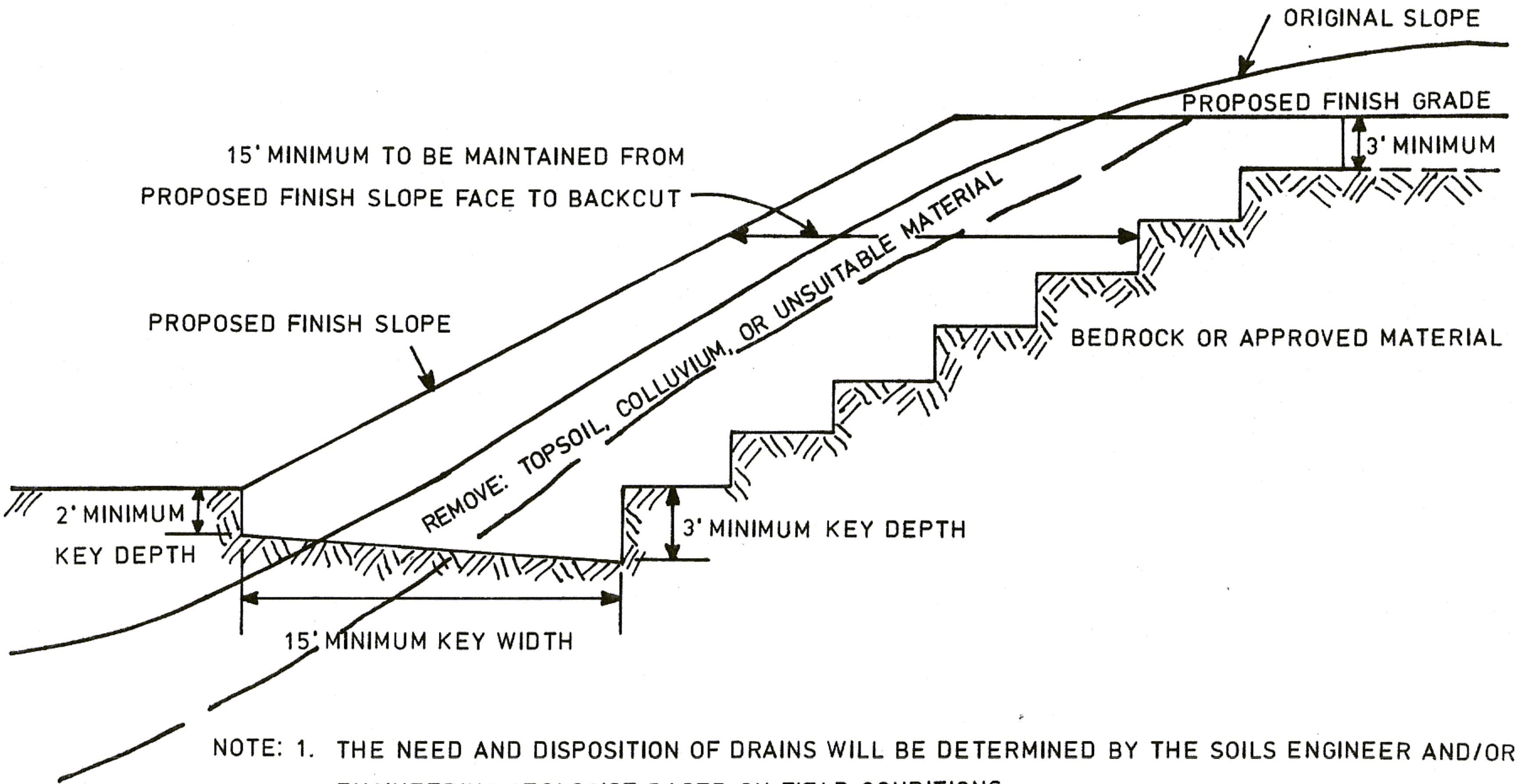
STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



NOTE: 1. SUBDRAINS ARE NOT REQUIRED UNLESS SPECIFIED BY SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST.

2. "W" SHALL BE EQUIPMENT WIDTH (15') FOR SLOPE HEIGHTS LESS THAN 25 FEET. FOR SLOPES GREATER THAN 25 FEET "W" SHALL BE DETERMINED BY THE PROJECT SOILS ENGINEER AND /OR ENGINEERING GEOLOGIST. AT NO TIME SHALL "W" BE LESS THAN H/2.

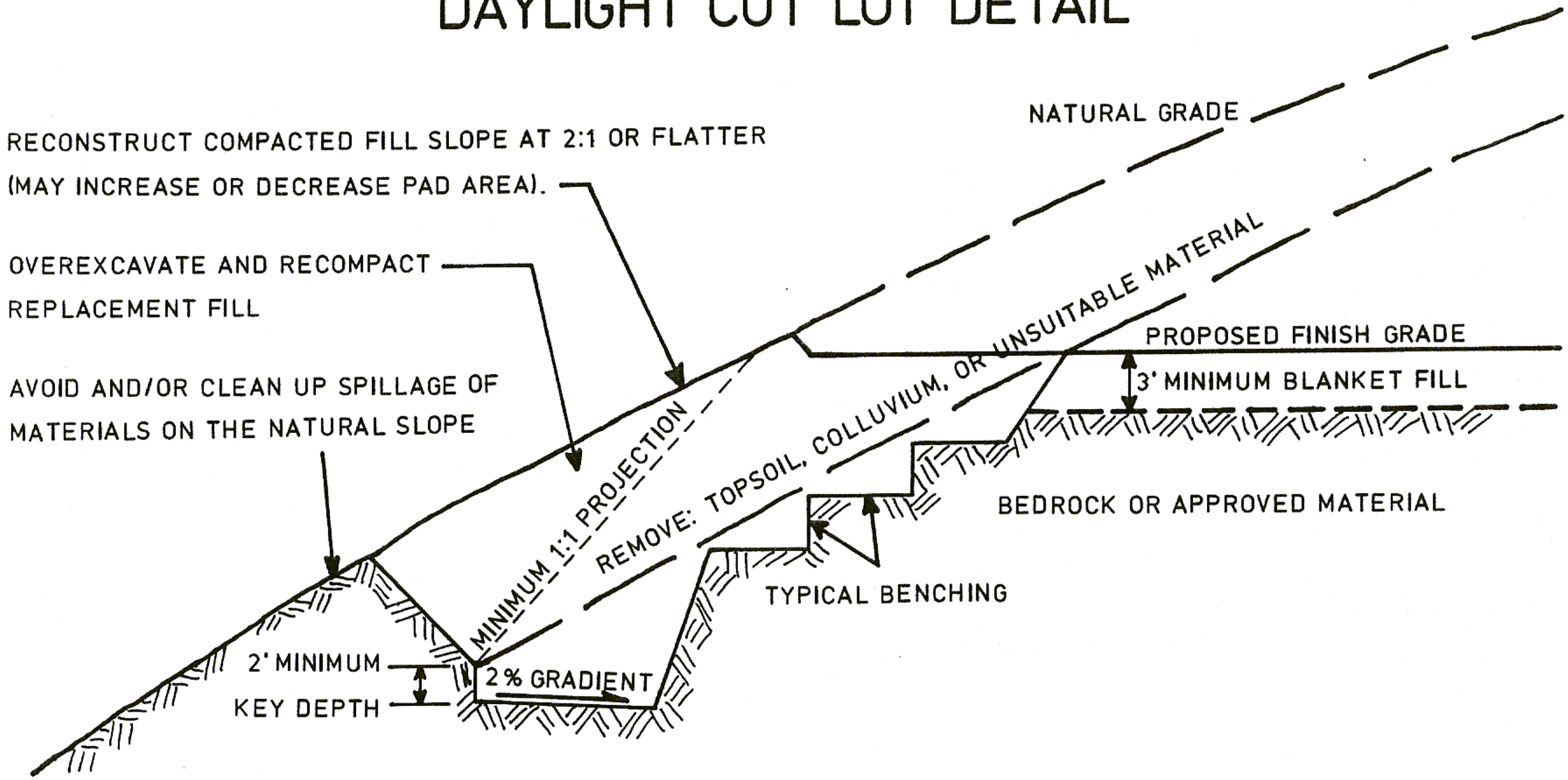
SKIN FILL OF NATURAL GROUND



NOTE: 1. THE NEED AND DISPOSITION OF DRAINS WILL BE DETERMINED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST BASED ON FIELD CONDITIONS.

2. PAD OVEREXCAVATION AND RECOMPACTION SHOULD BE PERFORMED IF DETERMINED TO BE NECESSARY BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST.

DAYLIGHT CUT LOT DETAIL

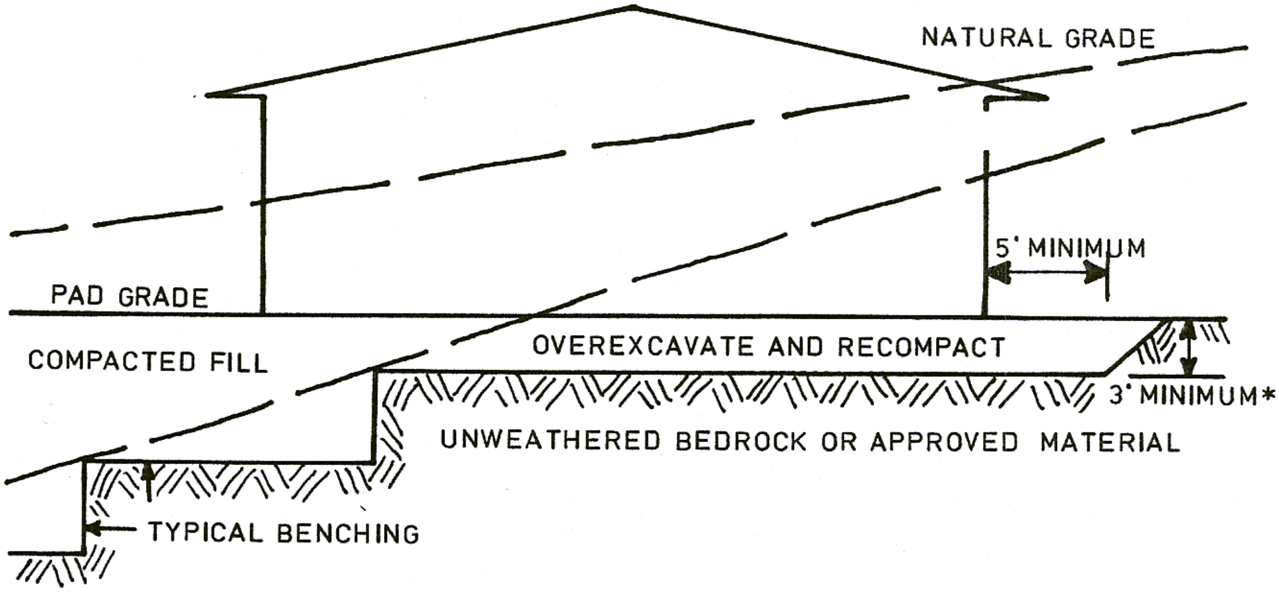


NOTE: 1. SUBDRAIN AND KEY WIDTH REQUIREMENTS WILL BE DETERMINED BASED ON EXPOSED SUBSURFACE CONDITIONS AND THICKNESS OF OVERBURDEN.

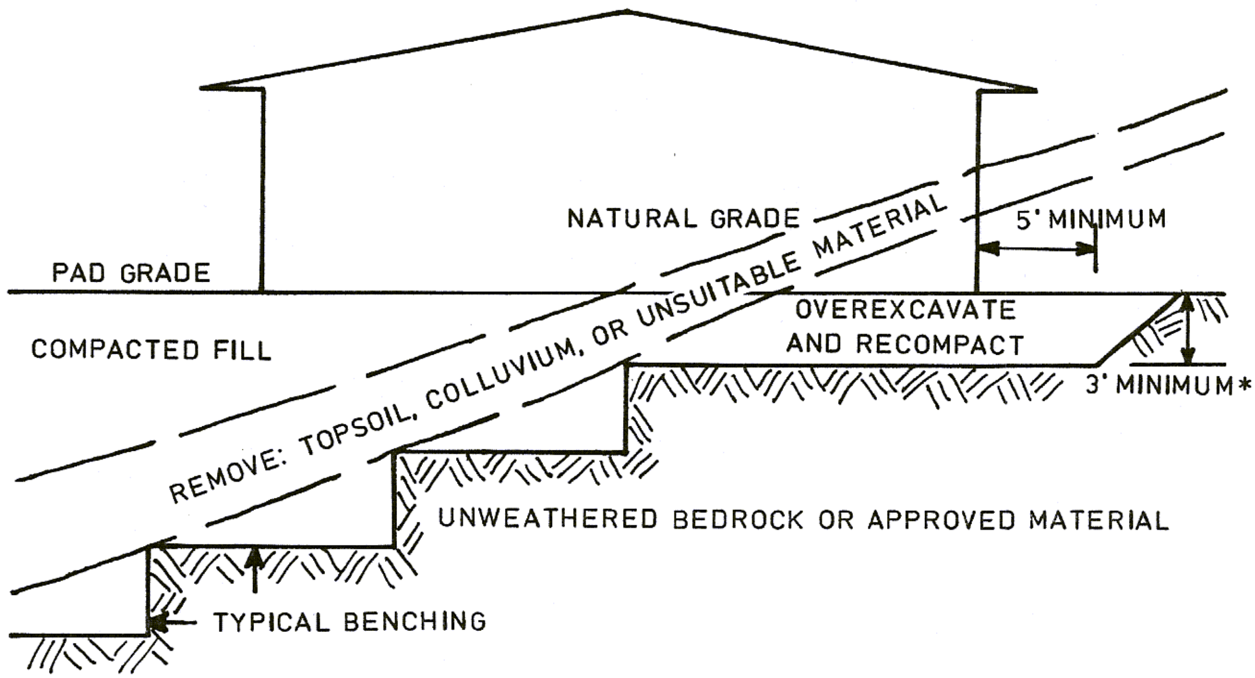
2. PAD OVER EXCAVATION AND RECOMPACTION SHOULD BE PERFORMED IF DETERMINED NECESSARY BY THE SOILS ENGINEER AND/OR THE ENGINEERING GEOLOGIST.

TRANSITION LOT DETAIL

CUT LOT (MATERIAL TYPE TRANSITION)

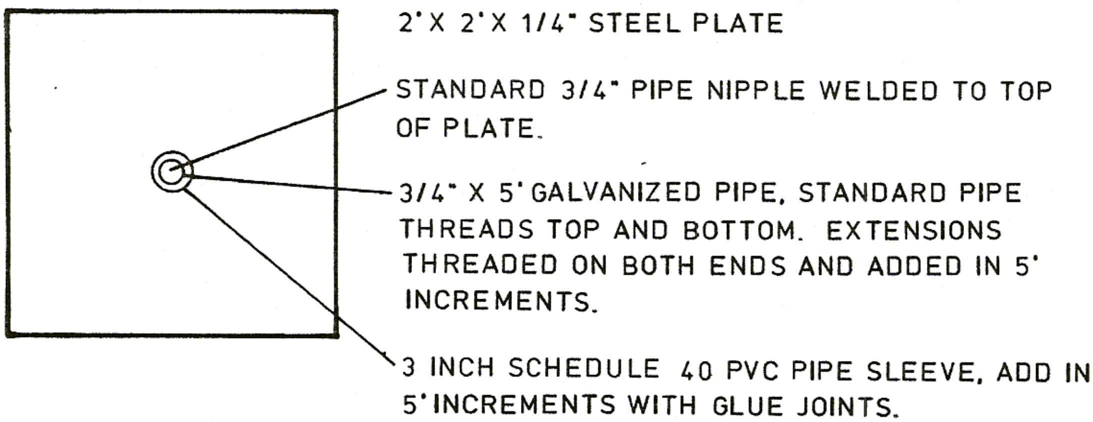


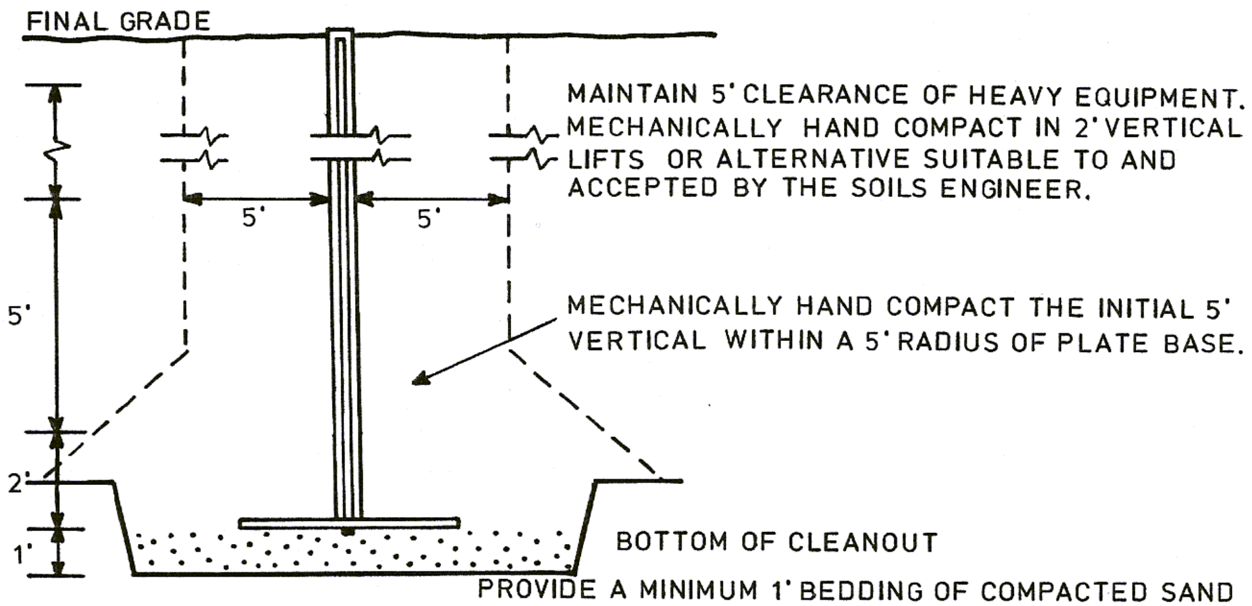
CUT–FILL LOT (DAYLIGHT TRANSITION)



NOTE: * DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST IN STEEP CUT–FILL TRANSITION AREAS.

SETTLEMENT PLATE AND RISER DETAIL





NOTE:

1. LOCATIONS OF SETTLEMENT PLATES SHOULD BE CLEARLY MARKED AND READILY VISIBLE (RED FLAGGED) TO EQUIPMENT OPERATORS.
2. CONTRACTOR SHOULD MAINTAIN CLEARANCE OF A 5' RADIUS OF PLATE BASE AND WITHIN 5' (VERTICAL) FOR HEAVY EQUIPMENT. FILL WITHIN CLEARANCE AREA SHOULD BE HAND COMPACTED TO PROJECT SPECIFICATIONS OR COMPACTED BY ALTERNATIVE APPROVED BY THE SOILS ENGINEER.
3. AFTER 5' (VERTICAL) OF FILL IS IN PLACE, CONTRACTOR SHOULD MAINTAIN A 5' RADIUS EQUIPMENT CLEARANCE FROM RISER.
4. PLACE AND MECHANICALLY HAND COMPACT INITIAL 2' OF FILL PRIOR TO ESTABLISHING THE INITIAL READING.
5. IN THE EVENT OF DAMAGE TO THE SETTLEMENT PLATE OR EXTENSION RESULTING FROM EQUIPMENT OPERATING WITHIN THE SPECIFIED CLEARANCE AREA, CONTRACTOR SHOULD IMMEDIATELY NOTIFY THE SOILS ENGINEER AND SHOULD BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
6. AN ALTERNATE DESIGN AND METHOD OF INSTALLATION MAY BE PROVIDED AT THE DISCRETION OF THE SOILS ENGINEER.

PLATE EG—14

TYPICAL SURFACE SETTLEMENT MONUMENT

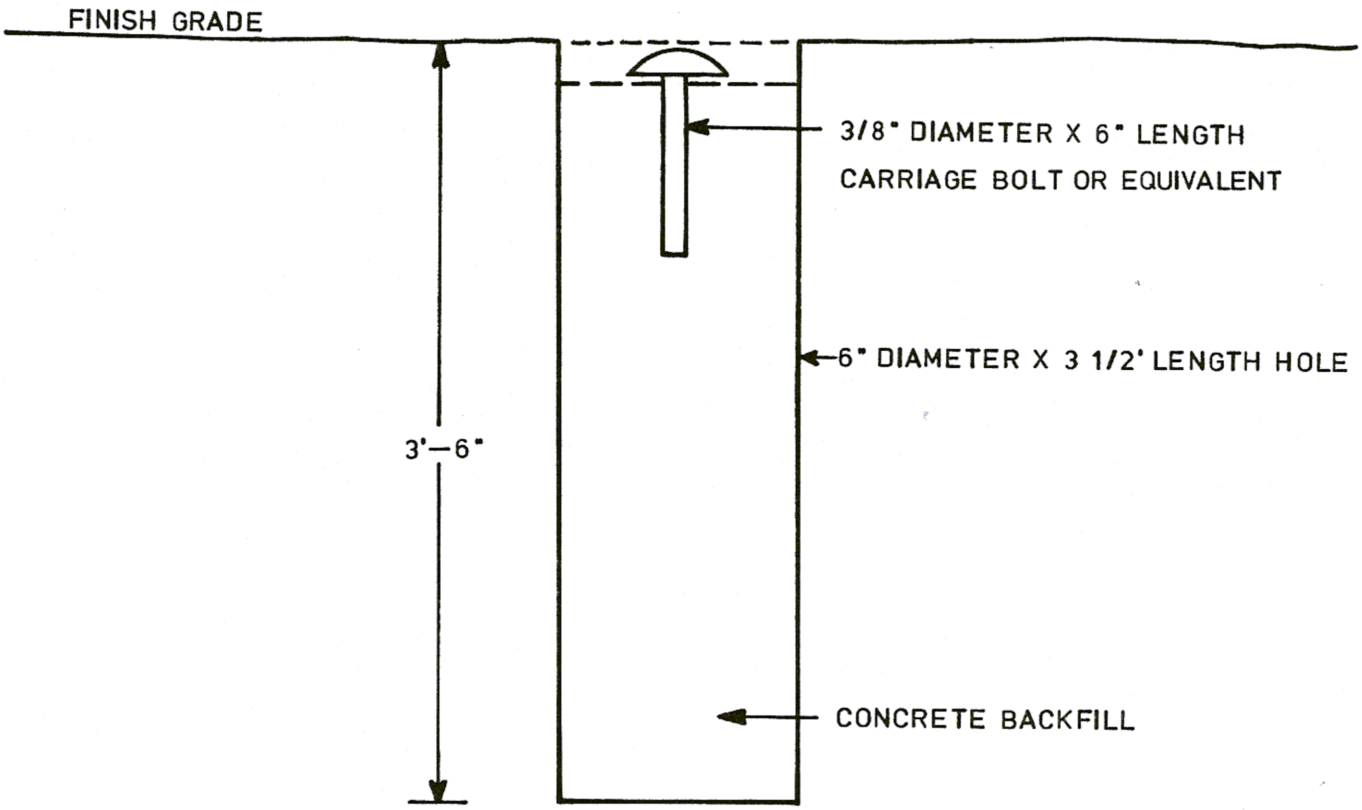
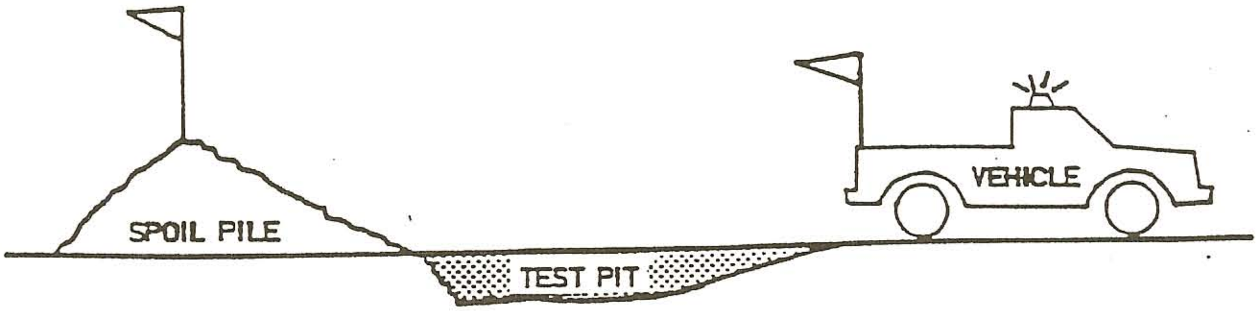


PLATE EG—15

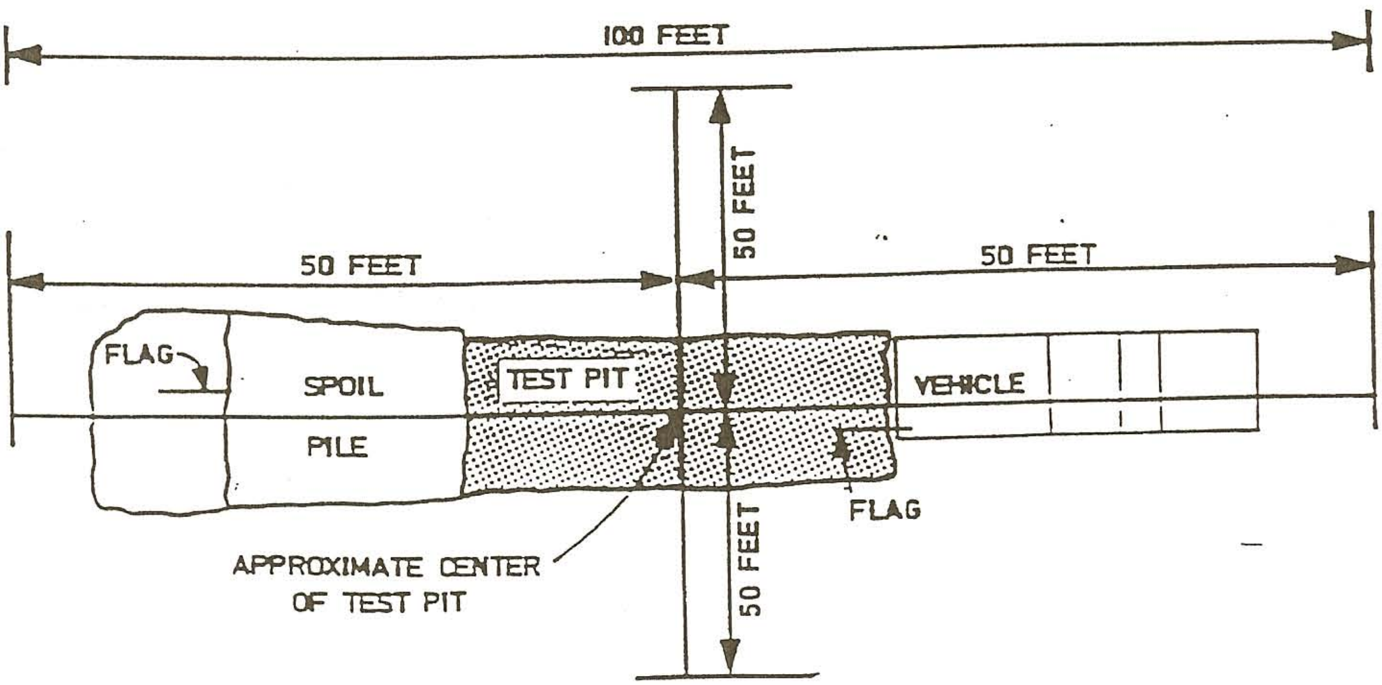
TEST PIT SAFETY DIAGRAM

SIDE VIEW



(NOT TO SCALE)

TOP VIEW

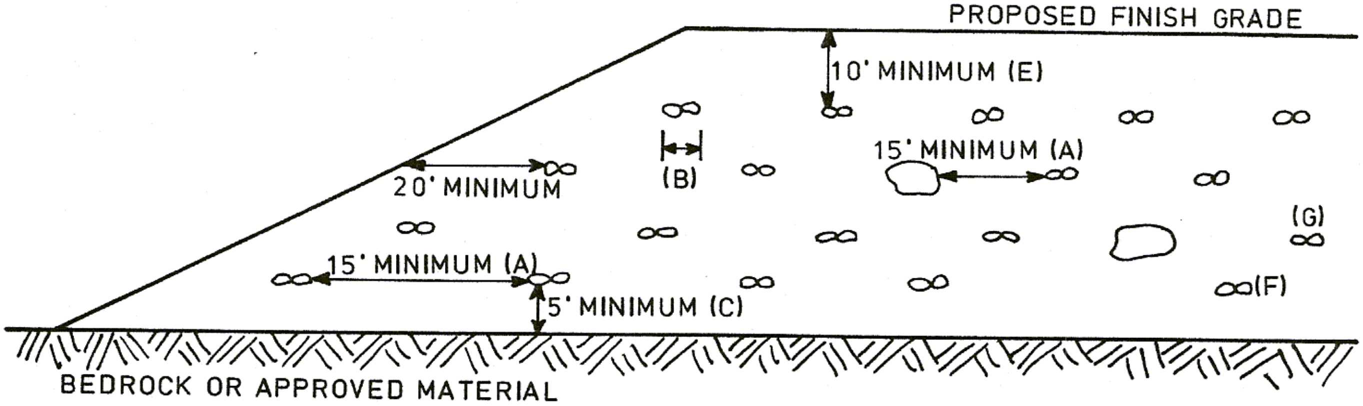


(NOT TO SCALE)

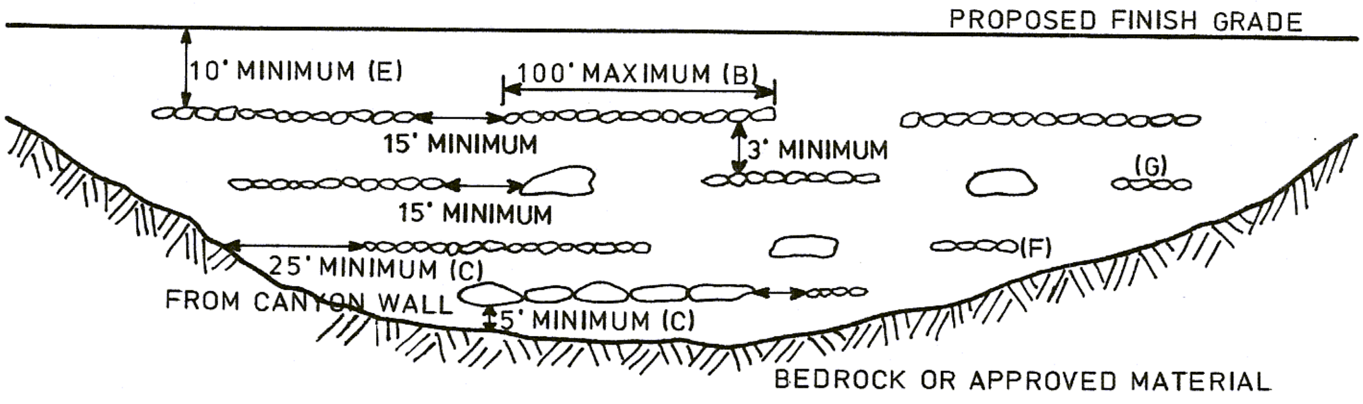
PLATE EG—16

OVERSIZE ROCK DISPOSAL

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



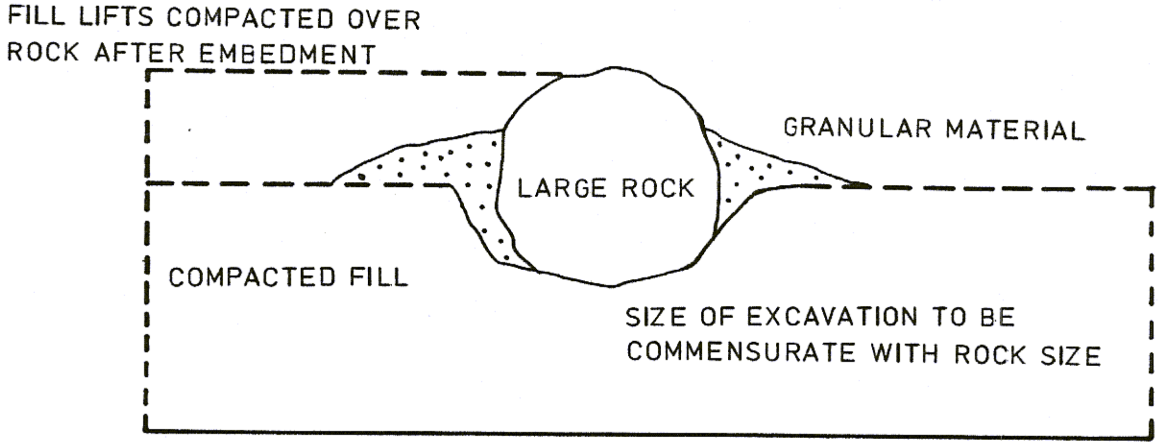
NOTE: (A) ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.
(B) HEIGHT AND WIDTH MAY VARY DEPENDING ON ROCK SIZE AND TYPE OF EQUIPMENT. LENGTH OF WINDROW SHALL BE NO GREATER THAN 100' MAXIMUM.
(C) IF APPROVED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST, WINDROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIAL OR BEDROCK PROVIDED ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.
(D) ORIENTATION OF WINDROWS MAY VARY BUT SHOULD BE AS RECOMMENDED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST. STAGGERING OF WINDROWS IS NOT NECESSARY UNLESS RECOMMENDED.
(E) CLEAR AREA FOR UTILITY TRENCHES, FOUNDATIONS AND SWIMMING POOLS.
(F) ALL FILL OVER AND AROUND ROCK WINDROW SHALL BE COMPACTED TO 90% RELATIVE COMPACTION OR AS RECOMMENDED.
(G) AFTER FILL BETWEEN WINDROWS IS PLACED AND COMPACTED WITH THE LIFT OF FILL COVERING WINDROW, WINDROW SHOULD BE PROOF ROLLED WITH A D–9 DOZER OR EQUIVALENT.

VIEWS ARE DIAGRAMMATIC ONLY. ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN.

PLATE RD–1

ROCK DISPOSAL PITS

VIEWS ARE DIAGRAMMATIC ONLY. ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN.



ROCK DISPOSAL LAYERS

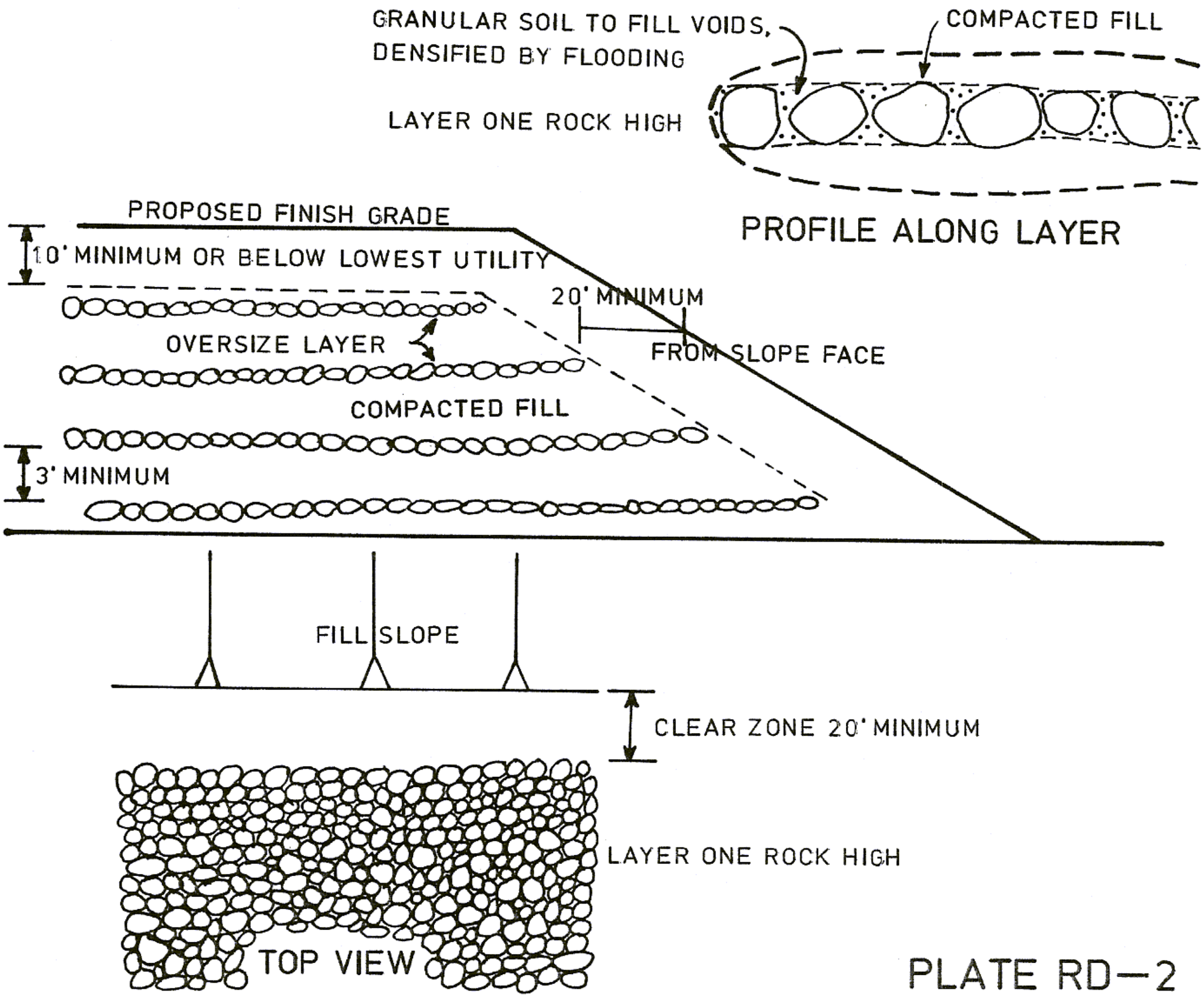


PLATE RD–2

TENTATIVE TRACT MAP 34760

GeoSoils, Inc.
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOTECHNICAL MAP

Plate 1 of 6

W.O. 5166-A-SC	DATE 10/06	SCALE 1"=100'

ALL LOCATIONS ARE APPROXIMATE

DPR 06-010

ASSESSOR'S PARCEL NO.

114-040-019,114-040-020, 275-100-003, AND 275-100-004

LEGAL DESCRIPTION:

PARCEL 1 OF PARCEL MAP 28201, AS SHOWN BY MAP RECORDED IN BOOK 187, PAGES 40-42, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, TOGETHER WITH A PORTION OF PARCEL 1 OF PARCEL MAP 28154, AS SHOWN BY MAP RECORDED IN BOOK 188, PAGES 77-83, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, ALSO TOGETHER WITH A PORTION OF GOVERNMENT LOT 1, A PART OF SECTION 11, TOWNSHIP 4 SOUTH, RANGE 7 WEST, SAN BERNARDINO BASE AND MERIDIAN.

ZONING:

EXISTING:
CITY OF CORONA: (39.9 ACRES)
HILLSIDE DEVELOPMENT OVERLAY ZONE (CMC SECTION 17.62.110) - MOUNTAIN GATE SPECIFIC PLAN (SP-89-01)
MINIMUM LOT AREA IS ONE-ACRE (43,560 S.F.)
COUNTY OF RIVERSIDE: (25.5 ACRES)
RURAL RESIDENTIAL (R-R) MINIMUM LOT AREA IS ONE-HALF ACRE (20,000 S.F.)
PROPOSED:
ANNEX INTO CITY OF CORONA
HILLSIDE DEVELOPEMENT OVERLAY ZONE (SP-89-01)

SURROUNDING ZONING:

NORTH: (ER) ESTATE RESIDENTIAL
SOUTH: CLEVELAND NATIONAL FOREST
EAST: (ER) ESTATE RESIDENTIAL
WEST: (OS) OPEN SPACE

SPECIFIC PLAN:

SP - 89-1
MOUNTAIN GATE SPECIFIC PLAN

GENERAL PLAN:

(E) ESTATE (1-3 DU/AC)

SURROUNDING GEN. PLAN:

NORTH: (E) ESTATE
SOUTH: CLEVELAND NATIONAL FOREST
EAST: (E) ESTATE
WEST: CITY OF CORONA: (LMDR) LOW-MEDIUM DENSITY RESIDENTIAL
COUNTY OF RIVERSIDE: (R-R) RURAL RESIDENTIAL

LAND USE:

AVOCADO GROVES

SURROUNDING LAND USE:

NORTH: SINGLE FAMILY RESIDENTIAL
SOUTH: CLEVELAND NATIONAL FOREST
EAST: VACANT
WEST: VACANT

STREET LINEAR FOOTAGE:

PROPOSED MALAGA STREET	489 L.F.
PROPOSED MALAGA CIRCLE	749 L.F.
PROPOSED "A" CIRCLE	1,663 L.F.
PROPOSED "B" CIRCLE	822 L.F.
TOTAL	3,723 L.F.

LANDSCAPE AREA

PROPOSED MALAGA STREET	7,824 S.F.
PROPOSED MALAGA CIRCLE	11,984 S.F.
PROPOSED "A" CIRCLE	26,608 S.F.
PROPOSED "B" CIRCLE	13,152 S.F.
TOTAL	59,568 S.F.

PHASING:

PROJECT HAS ONE PHASE, ONE FINAL MAP.

OWNER/DEVELOPER

RANCHO PASO DE VALENCIA
1253 ENTERPRISE COURT
CORONA, CA. 92882
PH. (951) 279-4877
FAX (951) 279-4889
ATTN: MANUEL & JOSE VALENCIA

ENGINEER

ARMSTRONG & BROOKS CONSULTING ENGINEERS
1530 CONSUMER CIRCLE, UNIT B
CORONA, CA. 92880
PH. (951) 372-8400
FAX (951) 372-8430
ATTN: DENNIS ARMSTRONG

VILLAGE NO. AND CFP AREA:

VILLAGE NUMBER - 3
COMMUNITY FACILITIES PLANNING AREA - (332)

DWELLING UNITS

28 DU - 65.4 AC = 0.43 DU/AC

TARGET DENSITY FACTOR:

EXISTING: (1.47 DU/ACRE) = ESTATE (0-3 DU/AC.)
PROVIDED: (0.43 DU/ACRE) = 0.43 DU X 65.4 AC = 28 DU.

MAX. NO. OF LOTS:

ALLOWABLE: 1.47 DU/ACRE X 65.4 AC = 96 PARCELS
96.1 PARCELS X 70% = 67 PARCELS
65.4 AC X 70% = 45 PARCELS
PROVIDED: 28 PARCELS

SITE ACREAGE

CITY OF CORONA = 39.9 ACRES
COUNTY OF RIVERSIDE = 25.5 ACRES
(ANNEX INTO CITY OF CORONA)
GROSS ACREAGE = 65.4 ACRES

SLOPE ANALYSIS

0-25% SLOPE	790,418 S.F.	28%
25.1%-100% SLOPE	2,056,743 S.F.	72%
TOTAL	2,847,161 S.F.	100%

DISTURBED 25.1% AND GREATER SLOPE AREA

NATURAL STATE	826,308 S.F.	40.2%	
DISTURBED SLOPE	1,230,435 S.F.	59.8%	(70% MAX. OF ALLOWABLE DENSITY)
TOTAL	2,056,743 S.F.	100.0%	

SERVICE PROVIDERS:

CITY OF CORONA UTILITY SERVICES	(951) 736-2263
CITY OF CORONA SEWER AND WATER	(951) 736-2321
SOUTHERN CALIFORNIA GAS (DISTRIBUTION)	(800) 422-4133
SOUTHERN CALIFORNIA ELECTRIC	(800) 442-4950
PACIFIC BELL	(800) 422-4133
AMERICAN TELEPHONE AND TELEGRAPH	(951) 381-7380
CORONA PUBLIC WORKS DEPARTMENT	(951) 736-2261
R.F. DICKSON STREET SWEEPING	(800) 841-7994
WESTERN WASTE	(951) 737-0343
CORONA DIAL-A-RIDE (SUPERVISOR)	(951) 734-1017
U.S. POSTAL SERVICE	(951) 737-0451
CORONA-NORCO UNIFIED SCHOOL DISTRICT	(951) 736-3340
SCHOOL DISTRICT TRANSPORTATION MANAGER	(951) 736-3305
RIVERSIDE TRANSIT AGENCY	(951) 684-0850
CORONA POLICE DEPARTMENT (FOR NOTIFICATION)	(951) 736-2330
CORONA FIRE DEPARTMENT (FOR NOTIFICATION)	(951) 736-2221
COUNTY CABLEVISION	(951) 272-8680

ARMSTRONG & BROOKS
CONSULTING ENGINEERS, INC.
PLANNING-INFRASTRUCTURE-SITE DEVELOPMENT-WATER RESOURCES
1530 CONSUMER CIRCLE, UNIT B
CORONA, CALIFORNIA 92880
(951) 372-8400 Ph. (951) 372-8430 Fax

JN 105-588 DATE 02/21/06

EASEMENT NOTES

A AN EASEMENT IN FAVOR OF REIWER J.K. MOSS AND THOMAS MOSS FOR INGRESS AND EGRESS AND INCIDENTAL PURPOSES, RECORDED JULY 5, 1940 IN BOOK 470, PAGE 100 OF OFFICIAL RECORDS; CANNOT BE LOCATED FROM THE RECORDS.

B AN EASEMENT IN FAVOR OF LEONE M. REDDING FOR INGRESS AND EGRESS AND INCIDENTIAL PURPOSES, RECORDED NOVEMBER 18, 1952 IN BOOK 1417, PAGE 17 OF OFFICIAL RECORDS; CANNOT BE LOCATED FROM THE RECORDS.

C AN EASEMENT IN FAVOR OF CALIFORNIA ELECTRIC POWER COMPANY FOR EITHER OR BOTH POLE LINES, CONDUITS OR UNDERGROUND FACILITIES AND INCIDENTIAL PURPOSES, RECORDED AUGUST 26, 1960 AS INSTRUMENT NO. 75548 OF OFFICIAL RECORDS; CANNOT BE LOCATED FROM THE RECORDS.

D AN EASEMENT IN FAVOR OF ED E. RITTER AND HELEN H. RITTER FOR ROADWAY AND INCIDENTIAL PURPOSES, RECORDED MAY 29, 1968 AS INSTRUMENT NO. 49982 OF OFFICIAL; CANNOT BE PLOTTED.

E AN EASEMENT IN FAVOR OF THE CITY OF CORONA FOR WATER PIPES, RECORDED NOVEMBER 5, 1887 IN BOOK 65, PAGE 187 OF DEEDS; CANNOT BE LOCATED FROM THE RECORDS.

F AN EASEMENT GRANTED TO CORONA CITY WATER COMPANY FOR WATER PIPES PURPOSES, RECORDED FEBRUARY 2, 1900 IN BOOK 91, PAGE 351 OF DEEDS; CANNOT BE LOCATED FROM THE RECORDS.

1 AN EASEMENT GRANTED TO CALIFORNIA ELECTRIC POWER COMPANY FOR PUBLIC UTILITIES PURPOSES, RECORDED AUGUST 19, 1960 AS INSTRUMENT NO. 73395 OF OFFICAL RECORDS; WIDTH NOT GIVEN.

2 AN EASEMENT GRANTED TO ADOBE OAKS, LLC FOR CONSTRUCTION AND MAINTENANCE OF AN ACCESS ROAD, RECORDED FEBRUARY 26, 1996 AS INSTRUMENT NO. 66474 OF OFFICIAL RECORDS.

3 AN EASEMENT GRANTED TO CORONA CITY FOR STREET AND PUBLIC UTILITY PURPOSES, RECORDED FEBRUARY 26, 1996 AS INSTRUMENT NO. 66475 OF OFFICIAL RECORDS.

4 AN EASEMENT GRANTED TO PACIFIC BELL FOR PUBLIC UTILITIES PURPOSES, RECORDED MARCH 22, 1999 AS INSTRUMENT NO. 117116 OF OFFICIAL RECORDS.

5 AN EASEMENT GRANTED TO ED E. RITTER FOR ACCESS PURPOSES, RECORDED MAY 18, 1999 AS INSTRUMENT NO. 214567 OF OFFICIAL RECORDS.

6 AN EASEMENT GRANTED TO BRUCE F. HICKS AND JEFFREY R. MORGULIS FOR ACCESS PURPOSES, RECORDED MAY 18, 1999 AS INSTRUMENT NO. 214568 OF OFFICIAL RECORDS.

7 AN EASEMENT GRANTED TO CENTEX HOMES, A NEVADA GENERAL PARTNERSHIP FOR CONSTRUCTION, PLANTING, PLACEMENT, MAINTENANCE AND USE OF HABITAT MITIGATION AND RELATED IMPROVEMENTS AS REQUIRED BY THE CALIFORNIA DEPARTMENT OF FISH AND GAME, RECORDED JUNE 23, 1999 AS INSTRUMENT NO. 278409 OF OFFICIAL RECORDS.

LOT AREAS:

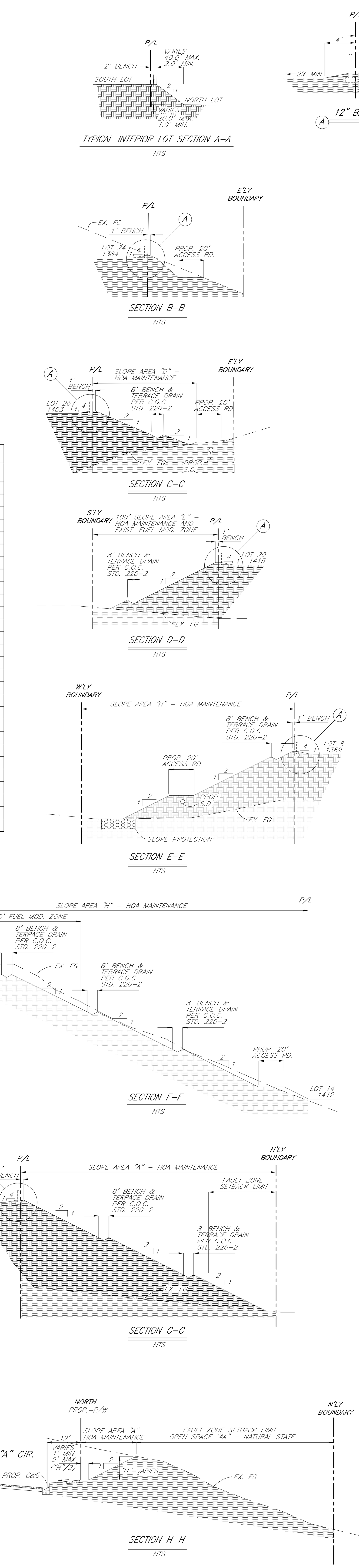
LOT DATA					PAD DATA
LOT NO.	LOT AREA (SF)	LOT AREA (ACRE)	WIDTH	DEPTH	PAD AREA (SF)
1	36,532	0.84	156'	275'	34,505
2	26,508	0.61	136'	194'	25,348
3	25,992	0.60	128'	205'	21,366
4	26,246	0.60	158'	162'	23,876
5	24,885	0.57	143'	173'	21,411
6	28,521	0.66	131'	226'	23,905
7	30,299	0.70	124'	242'	28,212
8	31,005	0.71	111'	245'	30,126
9	39,274	0.90	172'	258'	28,026
10	35,481	0.81	133'	274'	24,554
11	31,951	0.73	137'	231'	24,204
12	28,966	0.66	134'	205'	24,323
13	27,312	0.63	130'	200'	25,352
14	23,725	0.55	86'	220'	23,417
15	24,496	0.56	147'	167'	20,749
16	24,629	0.56	119'	152'	17,161
17	41,150	0.95	139'	259'	29,348
18	35,507	0.82	131'	267'	24,511
19	43,655	1.00	146'	319'	34,841
20	37,925	0.87	162'	277'	36,125
21	29,752	0.68	176'	187'	20,713
22	25,312	0.58	132'	190'	20,177
23	37,271	0.86	141'	311'	27,103
24	42,863	0.98	83'	309'	38,555
25	39,924	0.92	137'	284'	33,028
26	35,737	0.82	134'	270'	26,849
27	41,321	0.95	124'	317'	35,568
28	40,224	0.92	144'	262'	33,359

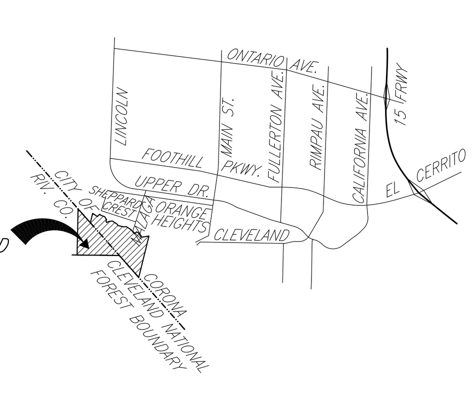
SLOPE AREAS: (H.O.A. MAINTENANCE)

LOT DATA		
LOT LTR.	LOT AREA (SF)	LOT AREA (ACRE)
"A"	78,281	1.80
"B"	14,817	0.34
"C"	31,458	0.72
"D"	14,884	0.34
"E"	148,025	3.40
"F"	17,554	0.40
"G"	13,457	0.31
"H"	385,533	8.85
TOTAL	704,009	16.16

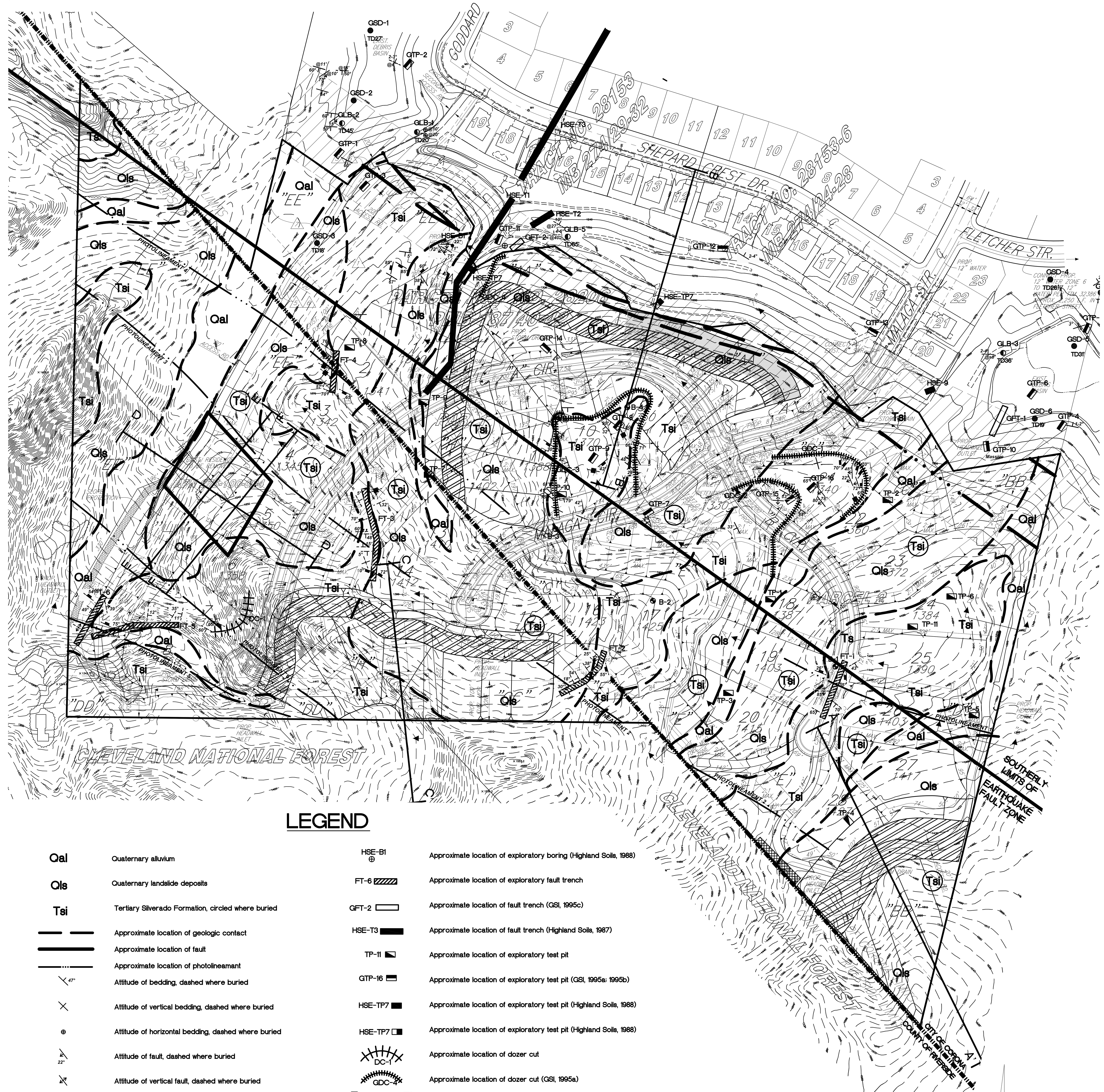
OPEN SPACE: (NATURAL STATE)

LOT DATA		
LOT LTR.	LOT AREA (SF)	LOT AREA (ACRE)
"AA"	119,086	2.73
"BB"	254,286	5.84
"CC"	75,034	1.72
"DD"	33,217	0.76
"EE"	516,877	11.87
TOTAL	998,500	22.92





VICINITY MAP
N.T.S.



LEGEND

Qal	Quaternary alluvium
Qls	Quaternary landslide deposits
Tsi	Tertiary Silverado Formation, circled where buried
	Approximate location of geologic contact
	Approximate location of fault
	Approximate location of photolineamant
47°	Attitude of bedding, dashed where buried
	Attitude of vertical bedding, dashed where buried
	Attitude of horizontal bedding, dashed where buried
22°	Attitude of fault, dashed where buried
	Attitude of vertical fault, dashed where buried
70°	Attitude of joint, dashed where buried
	Attitude of vertical joint, dashed where buried
B-4 TD56½'	Approximate location and total depth of exploratory boring
GLB-5 TD65'	Approximate location and total depth of exploratory large-diameter boring
GSD-6 TD31'	Approximate location and total depth of exploratory small-diameter boring (GSI, 1995b)
HSE-B1	Approximate location of exploratory boring (Highland Soils, 1988)
FT-6	Approximate location of exploratory fault trench
GFT-2	Approximate location of fault trench (GSI, 1995c)
HSE-T3	Approximate location of fault trench (Highland Soils, 1987)
TP-11	Approximate location of exploratory test pit
GTP-16	Approximate location of exploratory test pit (GSI, 1995a; 1995b)
HSE-TP7	Approximate location of exploratory test pit (Highland Soils, 1988)
HSE-TP7	Approximate location of exploratory test pit (Highland Soils, 1988)
DC-1	Approximate location of dozer cut
GDC-4	Approximate location of dozer cut (GSI, 1995a)
F F'	Geologic cross section
Forecut Keyway Backcut	Approximate location of proposed keyway and buttress
	Approximate location of proposed geogrid
	Approximate location of proposed debris wall

LEGEND

PROP. BOUNDARY
COUNTY OF RIVERSIDE AND CITY OF CORONA LIMITS
AMEND AREA TO CITY OF CORONA MOUNTAIN GATE SPECIFIC PLAN
FAULT ZONE SETBACK LIMITS

100 0 100 200 300
SCALE 1" = 100'

CITY OF CORONA	Drawing No.
TENTATIVE TRACT MAP 34760	Sh 1 of 1

LEGEND

Afu	Artificial fill - undocumented
Qcol	Quaternary topsoil/colluvium
Qal	Quaternary alluvium
Tsi	Tertiary Silverado Formation

0 5 10 20
Scale Feet

ALL LOCATIONS, STATION NUMBERS, AND DEPTHS ARE APPROXIMATE

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

FAULT TRENCH LOG FT-1 / FT-2

Plate 2 of 6

W.O. 5166-A-SC	DATE 10/06	SCALE 1"=5'

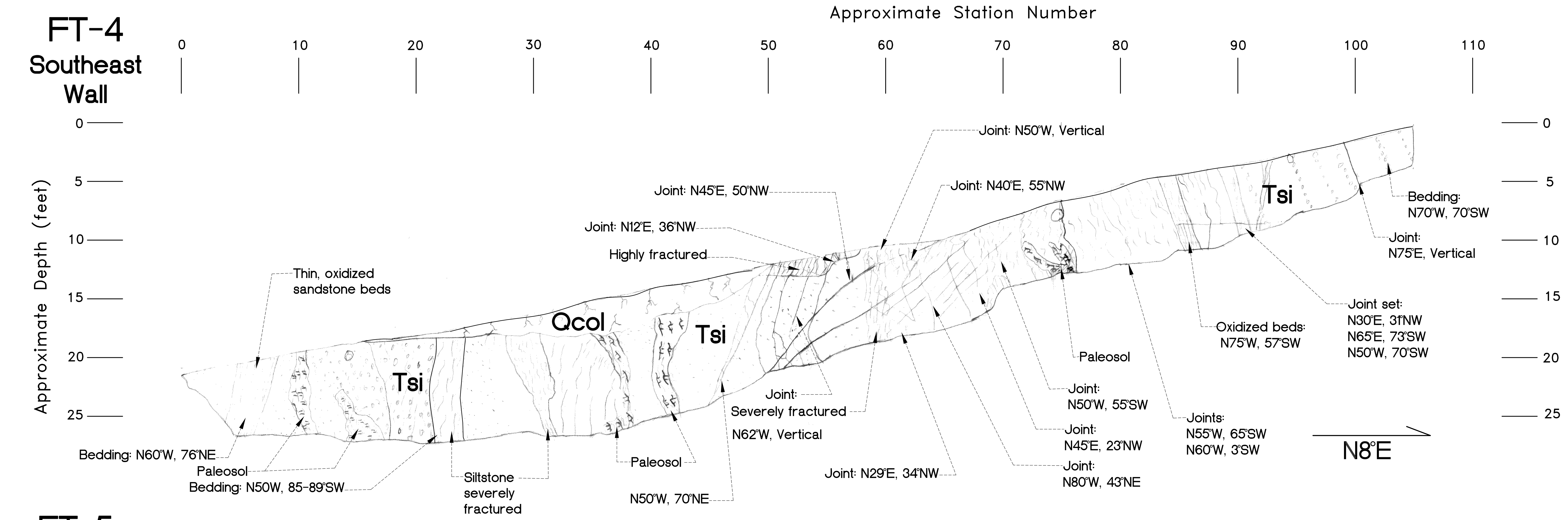
ALL LOCATIONS, STATION NUMBERS, AND DEPTHS ARE APPROXIMATE

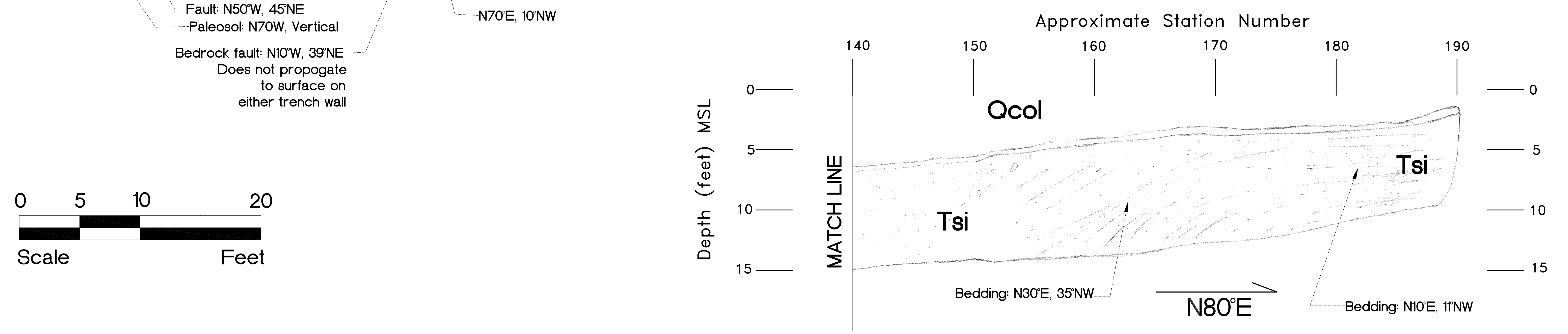
GeoSoils, Inc. RIVERSIDE CO. ORANGE CO. SAN DIEGO CO.

FAULT TRENCH LOG FT-3

Plate 3 of 6

W.O. 5166-A-SC	DATE 10/06	SCALE 1"=5'





ALL LOCATIONS, STATION NUMBERS, AND DEPTHS ARE APPROXIMATE

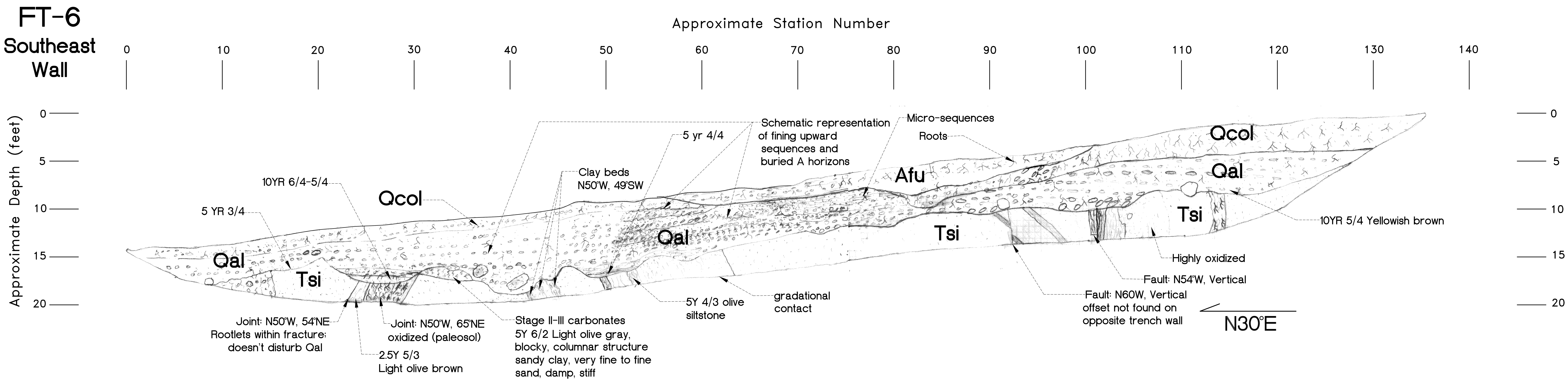
GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

FAULT TRENCH LOG FT-4 / FT-5

Plate 4 of 6

W.O. 5166-A-SC	DATE 10/06	SCALE 1"=5'



0 5 10 20
Scale Feet

ALL LOCATIONS, STATION NUMBERS, AND DEPTHS ARE APPROXIMATE

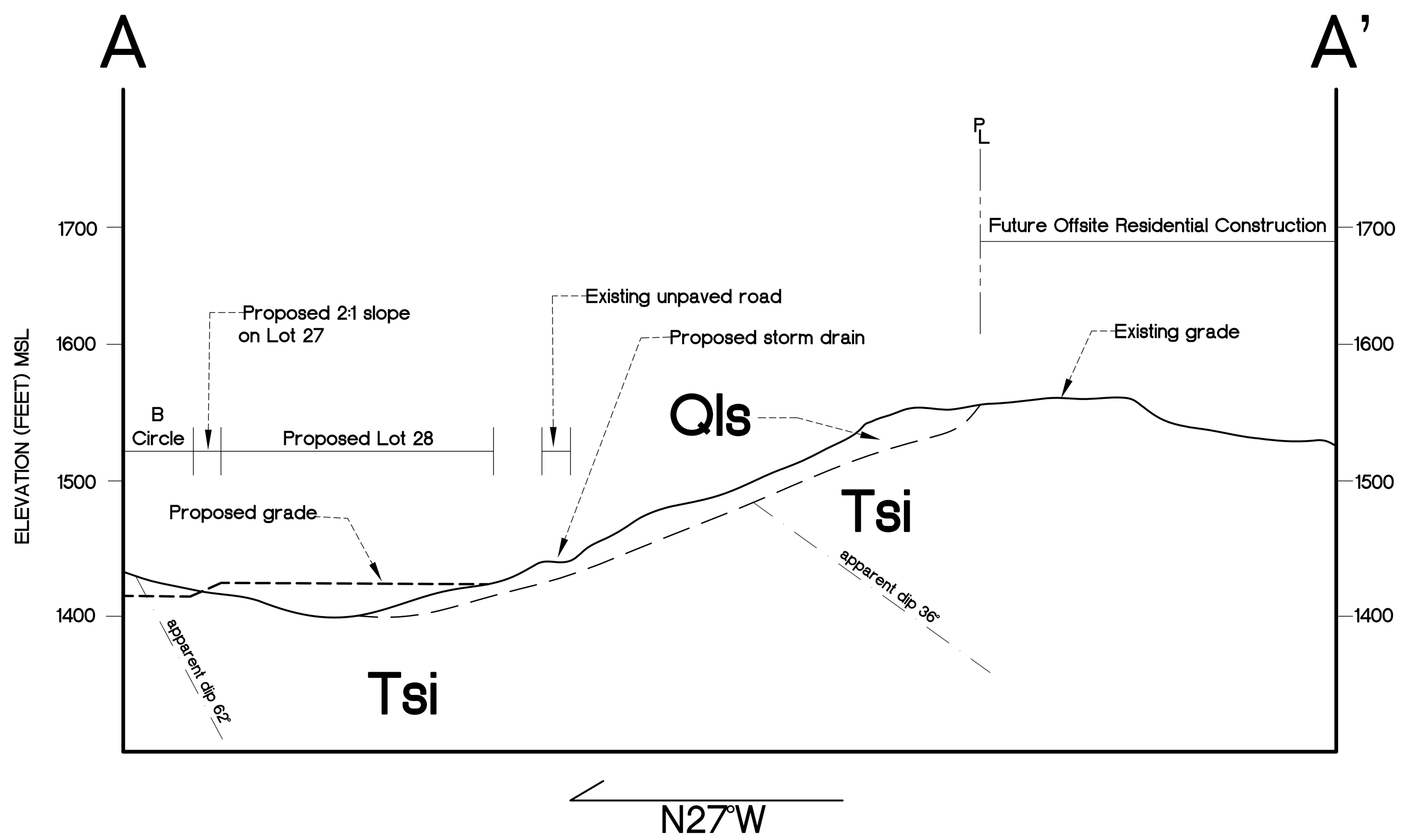
GeoSoils, Inc.

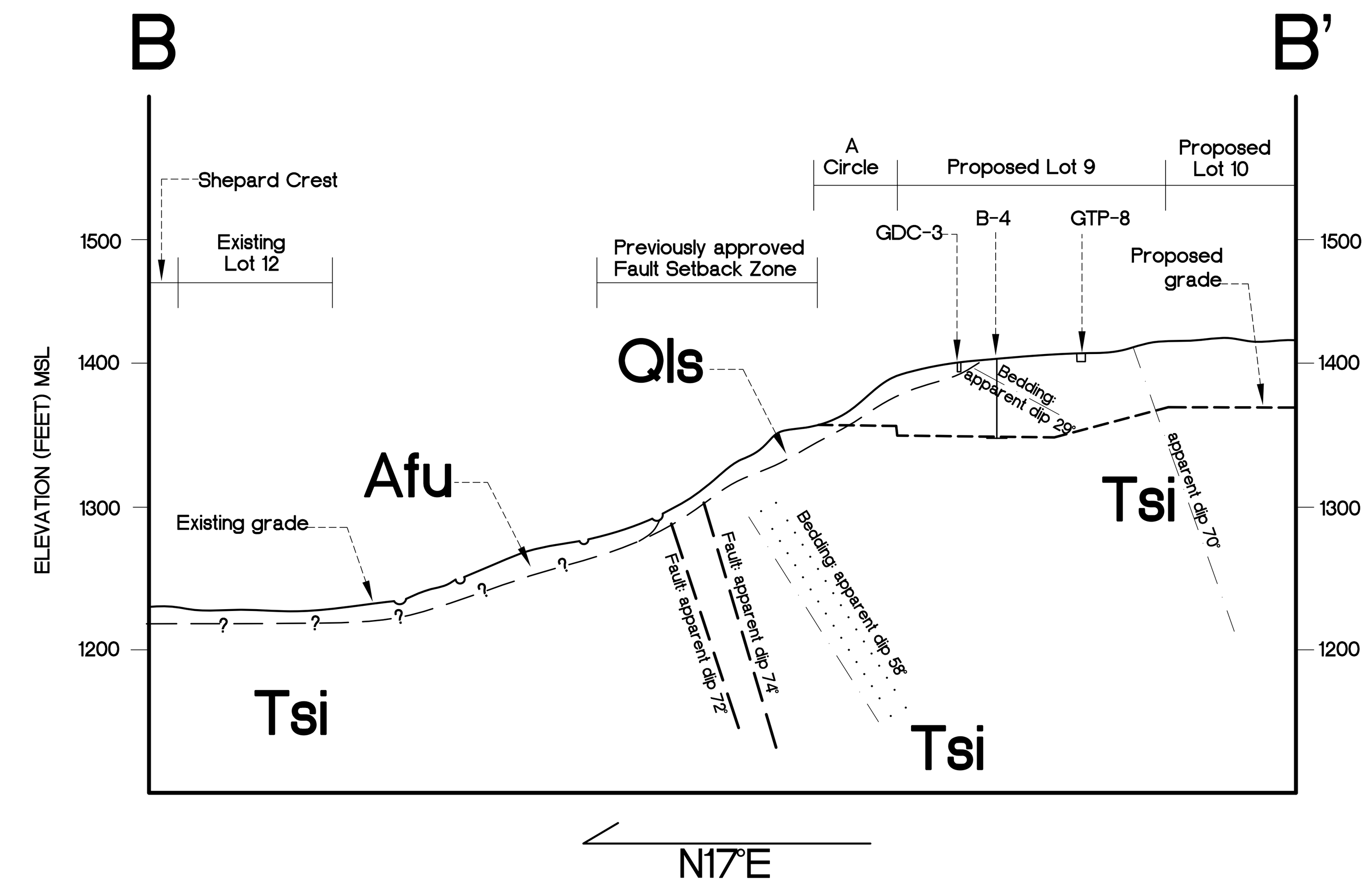
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

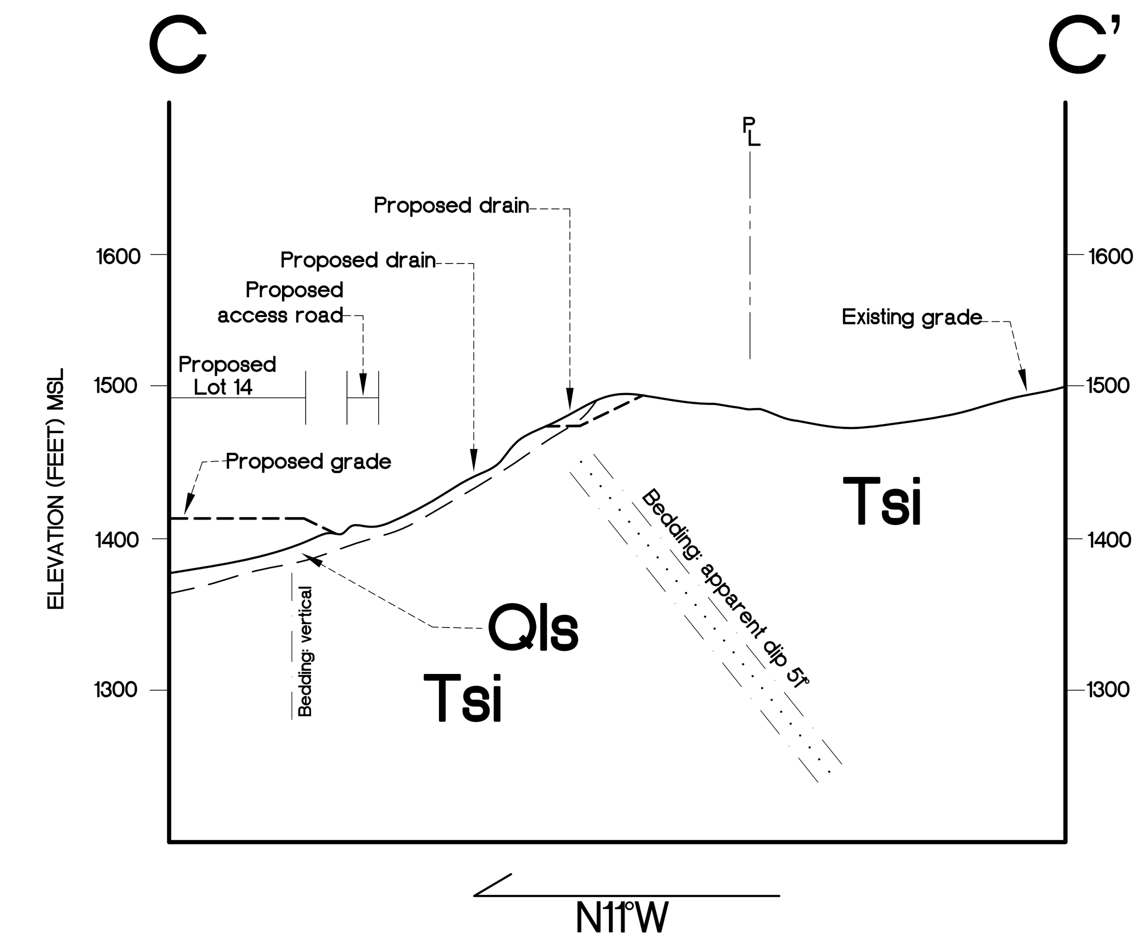
FAULT TRENCH LOG FT-6 / DOZER CUT LOG DC-1

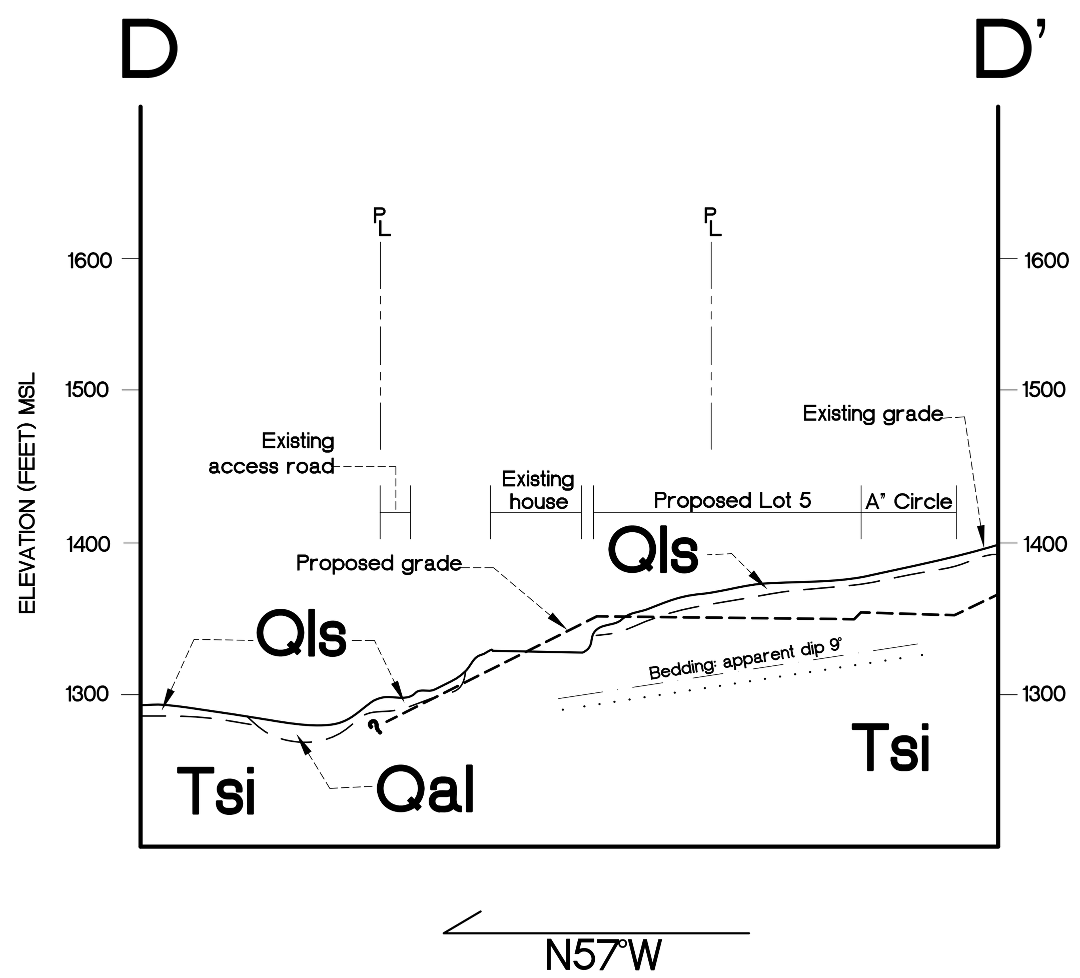
Plate 5 of 6

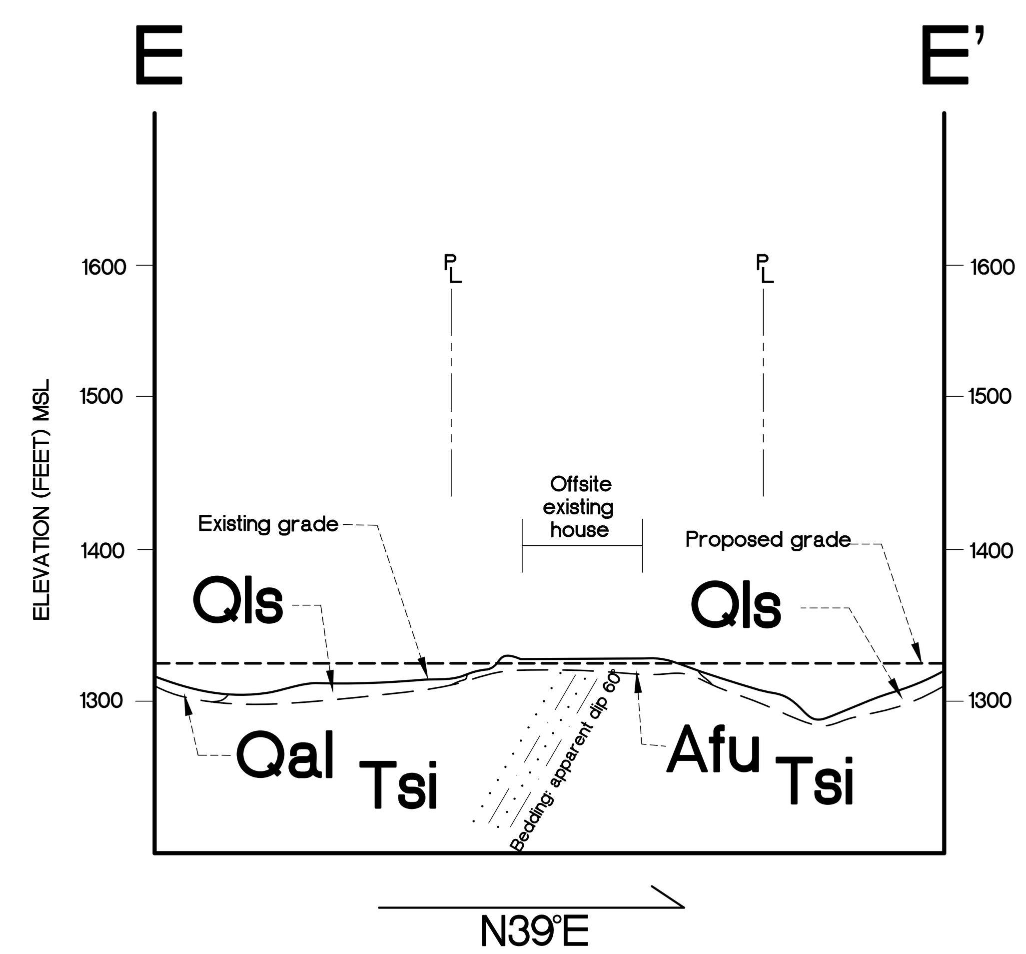
W.O. 5166-A-SC	DATE 10/06	SCALE 1"=5'

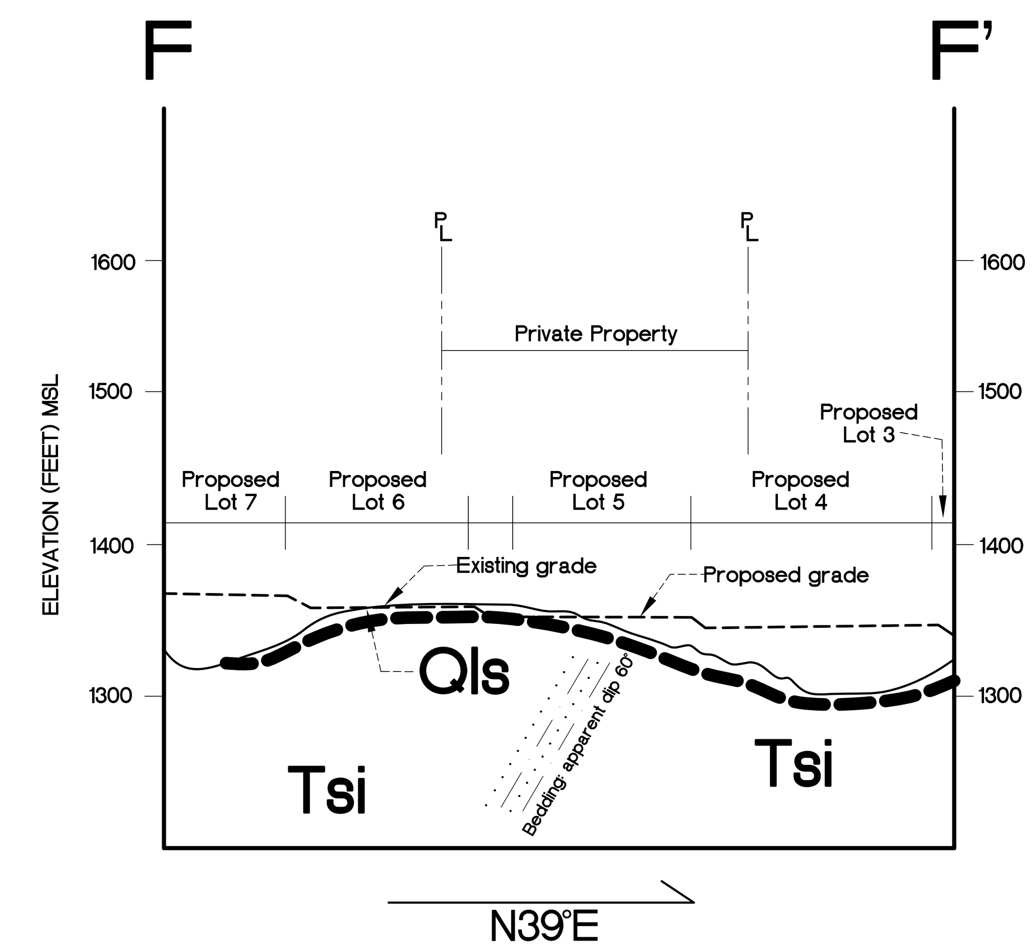












LEGEND

Afu	Artificial fill - undocumented
Qal	Quaternary alluvium
Qls	Quaternary landslide debris
Tsi	Tertiary Silverado Formation
— ?	Approximate location of geologic contact, queried whre uncertain

0 100 200 300
Scale Feet

ALL LOCATIONS ARE APPROXIMATE

NOT FOR CONSTRUCTION

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOLOGIC CROSS SECTIONS

Plate 6 of 6

W.O. 5166-A-SC | DATE 10/06 | SCALE 1"=100'