PRELIMINARY GEOTECHNICAL INVESTIGATION
IN SUPPORT OF ENVIRONMENTAL IMPACT REPORT
AND SPECIFIC PLAN SUBMITTAL, KELLER CROSSINGS
NORTHWEST CORNER OF WINCHESTER AND KELLER ROADS
RIVERSIDE COUNTY, CALIFORNIA

FOR

HANNA MARITAL TRUST
8105 IRVINE CENTER DRIVE, STE 1170
IRVINE, CALIFORNIA 92618

W.O. 5815-A-OC JULY 29, 2009

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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July 29, 2009

W.O. 5815-A-OC

Hanna Marital Trust
8105 Irvine Center Drive, Ste 1170
Irvine, California 92618

Attention: Mr. Geoff Scott

Subject: Preliminary Geotechnical Investigation in Support of Environmental Impact Report and Specific Plan Preparation, Keller Crossing, Northwest Corner of Winchester and Keller Roads, Riverside County, California

Dear Mr. Scott:

In accordance with your request and authorization, this report presents the results of GeoSoils Inc.'s (GSI's) preliminary geotechnical investigation in support of the revised Environmental Impact Report and Specific Plan submittals for the planned Keller Crossing project in Riverside County, California. The purpose of the study was to evaluate the onsite geotechnical and geologic conditions and their environmental impacts on the proposed site development, from a geotechnical viewpoint. Preliminary geotechnical design criteria along with discussions regarding the environmental impact of geologic and geotechnical site conditions, including mitigation measures, are presented herein.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), as well as field exploration (see Appendix B), seismicity analysis (see Appendix C), laboratory testing (see Appendix D), and geologic and engineering analysis, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided that mitigation measures presented in the text of this report are properly incorporated into design and construction of the project. The most significant elements of this study are summarized below:

- In general, the site may be characterized as being underlain by Mesozoic-age, meta-sedimentary phyllite bedrock, as mapped by Morton (2001), and is generally mantled by relatively thin, very old alluvial channel deposits. Additionally, unmapped young alluvial deposits intermittently occur in the natural drainage onsite.

- Due to the previous agricultural use of the property, any remaining organic debris, and other deleterious materials should be removed from the area to be graded and disposed offsite. Removals of potentially compressible colluvial/topsoil, very old alluvial channel deposits and near surface weathered bedrock materials will be necessary prior to fill placement within fill and shallow cut areas proposed for settlement-sensitive improvements. Estimated removal depths are outlined in the "Geologic and Geotechnical Impacts and Mitigation Measures" section of this report. In general, removals will be on the order of ±1 to ±2 feet (bedrock areas) and/or ±4 to ±8 feet, or greater (very old alluvial channel deposit areas), should settlement-sensitive improvements be proposed within those areas. Fill placement criteria are discussed in the text.

- Due to the nature of the underlying meta-sedimentary bedrock, oversize materials (angular boulders, etc., greater than 12 inches in diameter) should be anticipated during grading and possibly during trenching for utilities. Excavation and placement difficulties, as well as difficult ripping and blasting, and onsite disposal of these materials should be considered if improvements are proposed within these areas (see Appendix E for Seismic Refraction data). As per the Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 1997 and 2001; California Building Standards Commission [CBSC], 2007) requirements, no oversize materials (greater than 12 inches in diameter) should be placed within 10 feet of finish grade. This may necessitate the need for additional handling to non-structural areas (beyond the influence of building or settlement-sensitive improvements), onsite crushing to code compliant dimensions or construction of rock fill blankets at depths greater than 10 feet below finish grade, if the oversized materials are not removed from the area to be graded, or reduced in size, prior to site development. Additional discussion is provided herein.

- GSI's review indicates no known active faults are crossing the site, and the site is not within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007), nor is it within a liquefaction or subsidence zone established by the County of Riverside (County, 2008).

- Regional seismic shaking, ranging from moderate to severe, may occur on the property associated with nearby and/or regional faults. Accordingly, the proposed structures and foundations should be designed to resist seismic forces in accordance with the criteria contained in the CBC (ICBO, 2001; or CBSC, 2007; whichever is the most currently adopted code). Based on our site specific seismic hazard analysis, appropriate seismic design parameters are provided herein.

- Based on the relatively dense nature of the underlying bedrock materials, field sampling, laboratory testing, and our general screening evaluation per Special Publication 117 (California Department of Conservation, California Geological Survey, 2008), the potential for liquefaction within the site after development is considered very low.

- In general and based upon the available data to date, groundwater is not expected to be a major factor in development over most of the site. Groundwater was not encountered in the subsurface investigation. In conformance with the standard of practice for land development, the installation of subdrains would be required in the low-lying drainage swales. If developed during the rain season (October to May), some surface water and saturated swales should be anticipated.

- Any fill slopes proposed should be designed at a 2:1 (horizontal:vertical [h:v]) gradients or flatter and should not exceed about 30 feet in height, without a specific slope stability evaluation. Fill slopes should be properly built (see Appendix F, General Earthwork, Grading Guidelines, and Preliminary Criteria) and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Any cut slopes proposed should be designed at gradients of 2:1 and should not exceed 30 feet in height, without a specific slope stability evaluation. While stabilization of such cut slopes is not anticipated, locally adverse geologic conditions (e.g., adverse joints/fractures, daylighted laminations, etc.) may be encountered which may require remedial grading or laying back of the slope to an angle flatter than the adverse geologic condition. General guidelines for slope construction are presented in Appendix F.

- The expansion potential of onsite soils tested during this study is considered low to medium; however, higher and/or lower expansive soils may not be precluded from being encountered onsite. Based on the above, preliminary foundation designs should include categories for very low to medium expansive soils. For building pad areas with an Expansion Index (E.I.) greater than 20 and Plasticity Index (P.I.) $\geq$15, the foundations should be specifically designed in accordance with Section 1815 or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001) and/or CBC (CBSC, 2007). Mat-type foundations will likely be recommended if medium expansive soils are exposed, at or near finish pad grade elevations. Selective grading may be used to reduce the influence of expansive soils on building foundation(s). Alternately, through an appropriate blend of site soils or potential import soils may be utilized to achieve code compliance for low expansive soils. Based on the general parameters outlined above, preliminary foundation design recommendations are provided within the text of this report. As per the requirements of the controlling authorities, additional E.I. testing will need to be conducted during, or shortly after site grading to further evaluate the E.I. of the onsite soils within each building pad area. The E.I. testing frequency for as-graded conditions should be evaluated in the field based on the conditions exposed during site grading, generally one (1) to two (2) E.I. tests per building pad area is recommended.

- A typical sample of the site earth materials has been analyzed for corrosion/soluble sulfate potential. Laboratory testing indicates that soils generally appear to have negligible sulfate contents and low saturated resistivities, per the UBC/CBC (ICBO, 1997 and 2001) and/or CBC (CBSC, 2007). Accordingly, the use of sulfate resistant

concrete is currently not anticipated. The site soils were generally evaluated to be slightly alkaline (pH of 7.22 and 7.39) and are considered severely corrosive toward buried ferrous metals in a saturated state. Based on the above, consultation with a corrosion engineer is recommended to provide specific advisement for corrosion sensitive or buried improvements (i.e., foundations, piping, etc.). As per the requirements of the controlling authorities, additional sulfate/corrosion testing will need to be conducted during, or shortly after, site grading to further evaluate the corrosion potential of the onsite soils at finish pad grade. The testing frequency for as-graded conditions should be evaluated in the field based on the conditions exposed during site grading, typically the same frequency as E.I. testing.

- Adverse geologic structures that would preclude project feasibility were not encountered.

- The remediation measures presented in this report should be incorporated into the design and construction considerations of the project. All future design phases will require geotechnical review and/or investigation.

- The influence of site flooding may increase erosion of unprotected or unplanted site soils due to the relatively low plasticity of the earth materials onsite. The use of a surface drainage and landscaping will likely reduce this potential to a manageable performance level.

- The influence of the Diamond Valley lake dam innundation areas on the project was not reviewed for the hazard posed by the dam failure and potential to affect life and property. The project civil consultant should review the available County flood innundation maps. The potential for this type of hazard was presented in the original dam design and should be reviewed by the project planner and the other members of the design team. Techniques to reduce this hazard to a new yield level may be followed as determined by the project civil engineer.

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

Respectfully submitted,

GeoSoils, Inc.

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SHW/DCA/PLM/ATG/JPF/jh

Distribution: (4) Addressee

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Appendix B - Test Pit Logs by GSI ..... Rear of Text
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Appendix D - Laboratory Data by GSI ..... Rear of Text
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Plate 1 - Geotechnical Map ..... Rear of Text in Folder

PRELIMINARY GEOTECHNICAL INVESTIGATION
IN SUPPORT OF ENVIRONMENTAL IMPACT REPORT
AND SPECIFIC PLAN SUBMITTAL, KELLER CROSSINGS
NORTHWEST CORNER OF WINCHESTER AND KELLER ROADS
RIVERSIDE COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic data for the site and site area, including aerial photographs (see Appendix A).

2. Review of the current 100-scale Specific Plan, prepared by Proactive Engineering Inc. (2009).

3. Performance of a photo-lineament analysis for the site and vicinity (United States Department of Agriculture [USDA], 1980).

4. Geologic site reconnaissance and geologic mapping of significant geologic contacts and surficial deposits.

5. Subsurface exploration consisting of the excavation of 11 geotechnical exploratory test pits for logging and sampling (Appendix B).

6. Site specific seismic hazard analysis (see Appendix C).

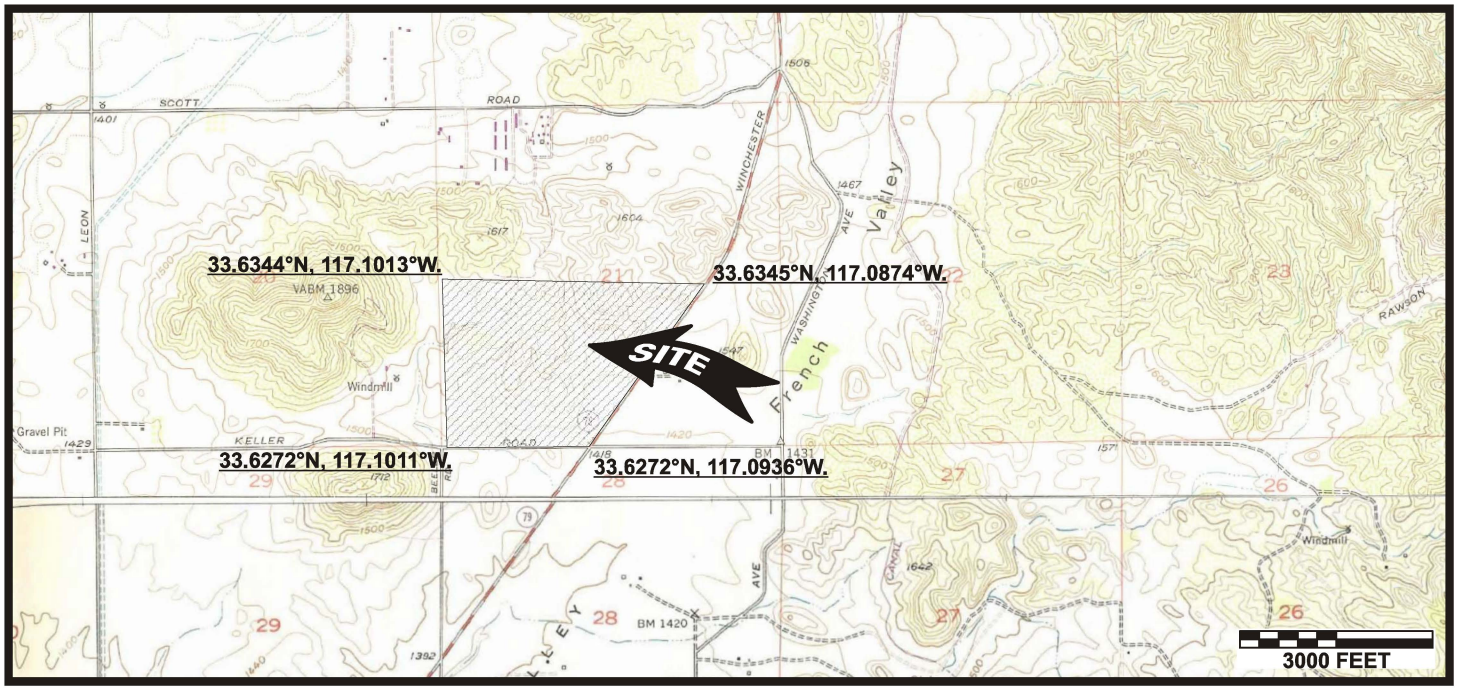
7. Pertinent laboratory testing of representative soil samples collected during our subsurface exploration program. Laboratory testing included: moisture/density testing, maximum density testing, sulfate/corrosion, Expansion Index (E.I.), Atterberg Limits, consolidation, direct shear testing, and R-value testing of the materials encountered during our field investigation (Appendix D).

8. Performance of 12 double direction, seismic refraction traverses to evaluate rock hardness for general rippability assessment (Appendix E).

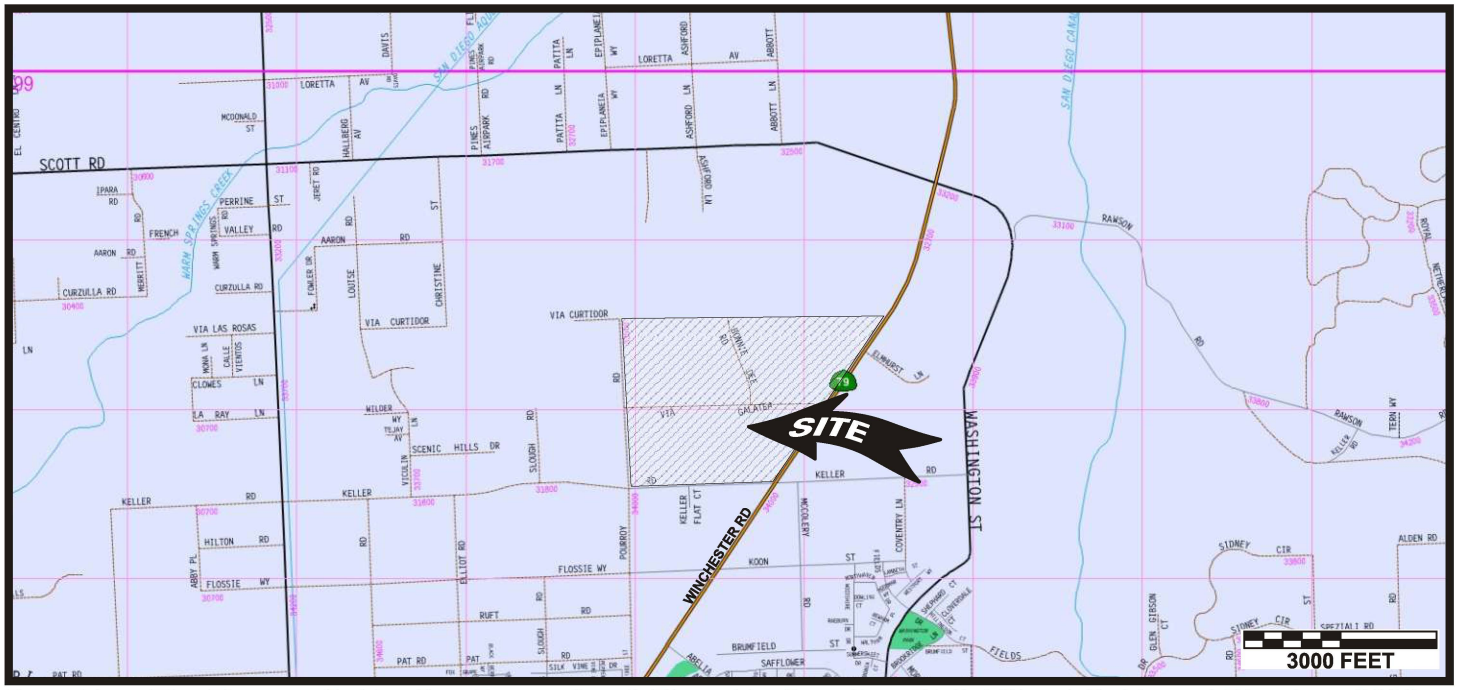
9. Appropriate engineering and geologic evaluation of data collected, and preparation of this preliminary geotechnical investigation report, including test pit logs, laboratory test results, regional seismic data, seismic refraction traverse data and remediation measures for the identified geologic and geotechnical impacts.

SITE DESCRIPTION

The proposed Keller Crossing project is located on the northwest corner of Winchester and Keller Roads in Riverside County, California (see Figure 1). The approximately 200-acre



U.S.G.S. Winchester and Bachelor Mountain Quadrangles, California, Riverside Co., 7.5 Minute, dated 1953, photorevised 1979 and 1973, respectively.



Base Map: The Thomas Guide, San Bernardino & Riverside Counties, Digital Street Guide and Directory, 2008 Edition, by Thomas Bros. Maps, pages 869, 870, 899, and 900.

ALL LOCATIONS APPROXIMATE

N

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SITE LOCATION MAP

Figure 1

Keller Crossing

Northwest Corner of Winchester and Keller Roads

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site is bound by Winchester Road to the east, Keller Road to the south, Pourroy Road to the west and vacant land to the north. Excepting the north, rural residential property surrounds the site.

The trapezoid-shaped parcel contains approximate elevations ranging from 1,420 feet Mean Sea Level (MSL) on the southeast portion of the property to 1,580 feet MSL on the northwest property boundary. Total site relief is approximately 160 vertical feet.

The site topography can be described as a series of predominantly south trending ridges with intervening valleys that are descending from a strong east-west trending ridge. The majority of the natural slopes are shallow to moderate, with steepening of the ridge to the north. Slope ratios vary from approaching 1.5:1 (horizontal:vertical [h:v]) to approximately 3:1 in the lower bedrock elevations, and nearly flat on the lower portions of the site. Drainage is by sheet flow along and down the sides of the ridges and is conveyed offsite to the south, west and southeast, along the valleys. These were dry at the time of GSI's study.

The site was recently disced for farming operations and only light vegetation has re-established. The steeper non-farmed portions are covered with native grass and scrub vegetation. No significant man-made or cultural improvements were observed onsite.

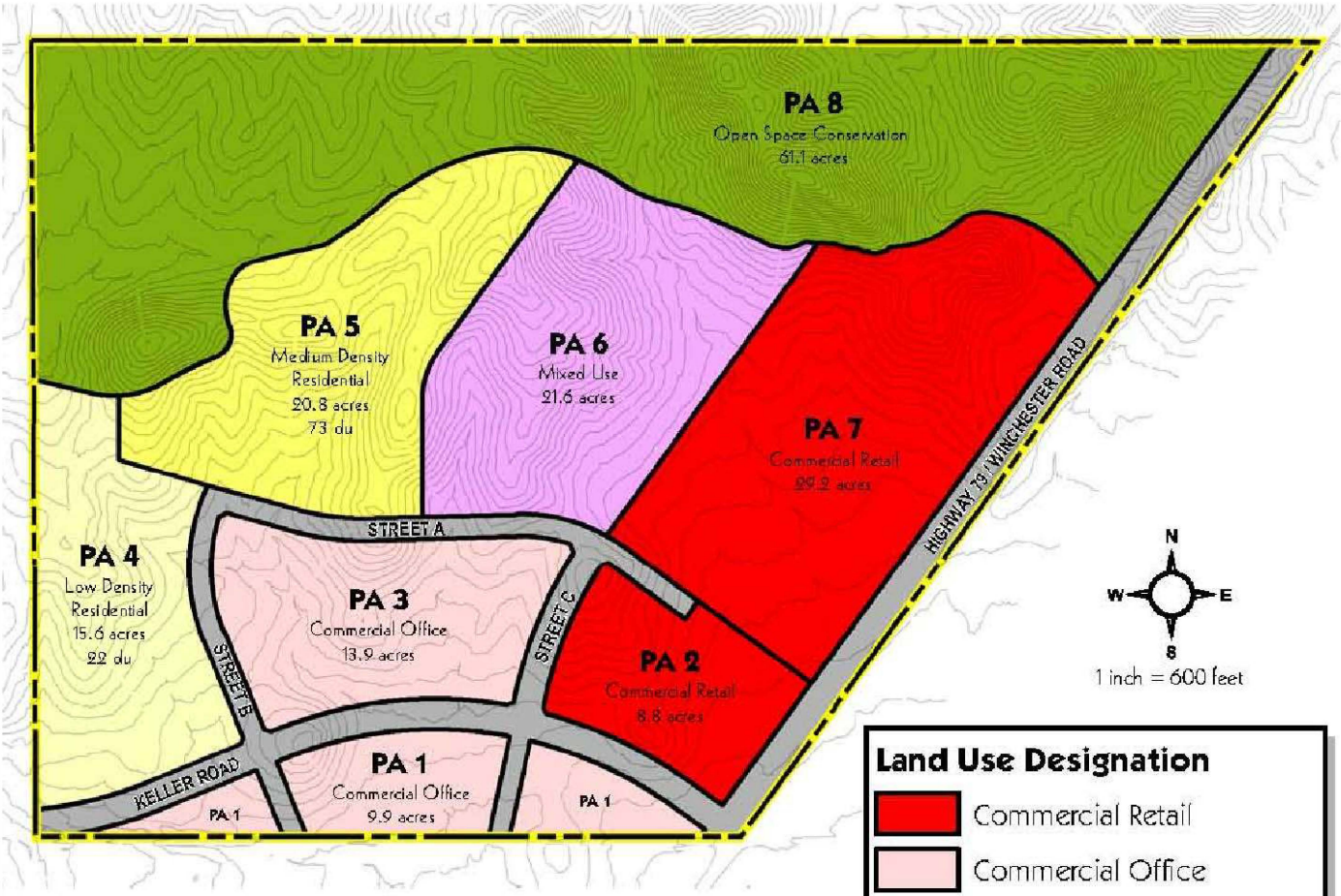
PROPOSED DEVELOPMENT

Based upon the present Specific Plan Land Use Plan (Figure 2), the development will consist of the construction of a balance of commercial retail, commercial office, low density residential, mixed use, and open space conservation. The project would also include associated roadways, parking areas, and underground utility improvements. GSI further understands that the proposed one- to five-story buildings would be designed with slabs-on-grade and continuous footings, utilizing wood-frame, masonry block, or tilt-up type of construction. Building loads are assumed to be typical for the relatively light construction (one- to three-stories). Typical cut and fill grading techniques, as well as heavy ripping and blasting, would be required to bring the site to design elevations. It is our understanding that cut and fill slopes are proposed up to about ± 30 feet in overall height, at an inclination of 2:1 (h:v), or flatter. The need for import soils is currently unknown at this time. Sewage disposal is proposed to be accommodated by tying into the regional sewer system.

FIELD STUDIES

Previous Geotechnical Investigations

A prior geological and geotechnical site characterization investigation by Eberhart/United Consultants ([EUC], 2005) has been performed. The EUC investigation was in support of



Base Map provided by:
HELIX Environmental Planning, Inc., 2011

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

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SPECIFIC PLAN LAND USE PLAN REVISED

Figure 2

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a different development plan that consisted entirely of residential development. During that study, 23 exploratory backhoe trenches were excavated, logged, sampled and laboratory testing performed. The geologic and geotechnical data collected during EUC's investigation were considered during GSI's study. The EUC trench logs and laboratory data are included in Appendix B-2 and Appendix D-2 of this document and are shown on the accompanying geologic map (Plate 1).

Present Investigation

Field studies conducted during the current GSI's evaluation of the property consisted of geologic reconnaissance mapping and excavation of 11 additional exploratory backhoe test pits to evaluate near surface soil and geologic conditions. The test pits were logged by a geologist from our firm who collected representative, bulk and relatively undisturbed ring samples for laboratory testing. Logs of the test pits are presented in Appendix B.

In addition to the test pits, 12 seismic traverses were performed to evaluate the rock hardness and general excavation characteristics of the bedrock underlying the site. Seismic refraction data is presented in Appendix E. The approximate locations of the exploratory test pits, and seismic traverses are presented on Plate 1.

REGIONAL GEOLOGY

The subject property is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. It is characterized by steep, elongated ranges and valleys that trend north-westward. 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. The mountain ranges are underlain by pre-Cretaceous meta-sedimentary and meta-volcanic rocks, and Cretaceous plutonic rocks of the southern California batholith. Tertiary sediments and volcanics, along with Quaternary sediments flank the mountain ranges. The Tertiary and Quaternary rocks are generally composed of non-marine sediments consisting of sandstones, mudstones, fanglomerates, and scattered volcanic rocks. Uplift on the regional faults, and erosion has resulted in the incision of the older rocks, and subsequent deposition of coalescing alluvial fans in the valleys.

Keller Crossing is located within the Perris block. The Perris Block is a large mass of granitic rock generally bounded by the San Jacinto Fault, the Elsinore Fault, the Santa Ana River and an undefined southeast boundary. The Perris Block has had a history of vertical land movements of several thousand feet due to shifts in the Elsinore and San Jacinto faults.

SITE GEOLOGIC UNITS

The site geologic units encountered during our subsurface investigation and site reconnaissance included colluvium/topsoil, very old alluvial fan deposits, and Mesozoic-age metasedimentary bedrock. For context, a Regional Geologic Map is provided as Figure 3. Site earth units are generally described below from youngest to oldest; and their areal limits, based on the available data, are presented on Plate 1.

Colluvium/Topsoil (Unmapped)

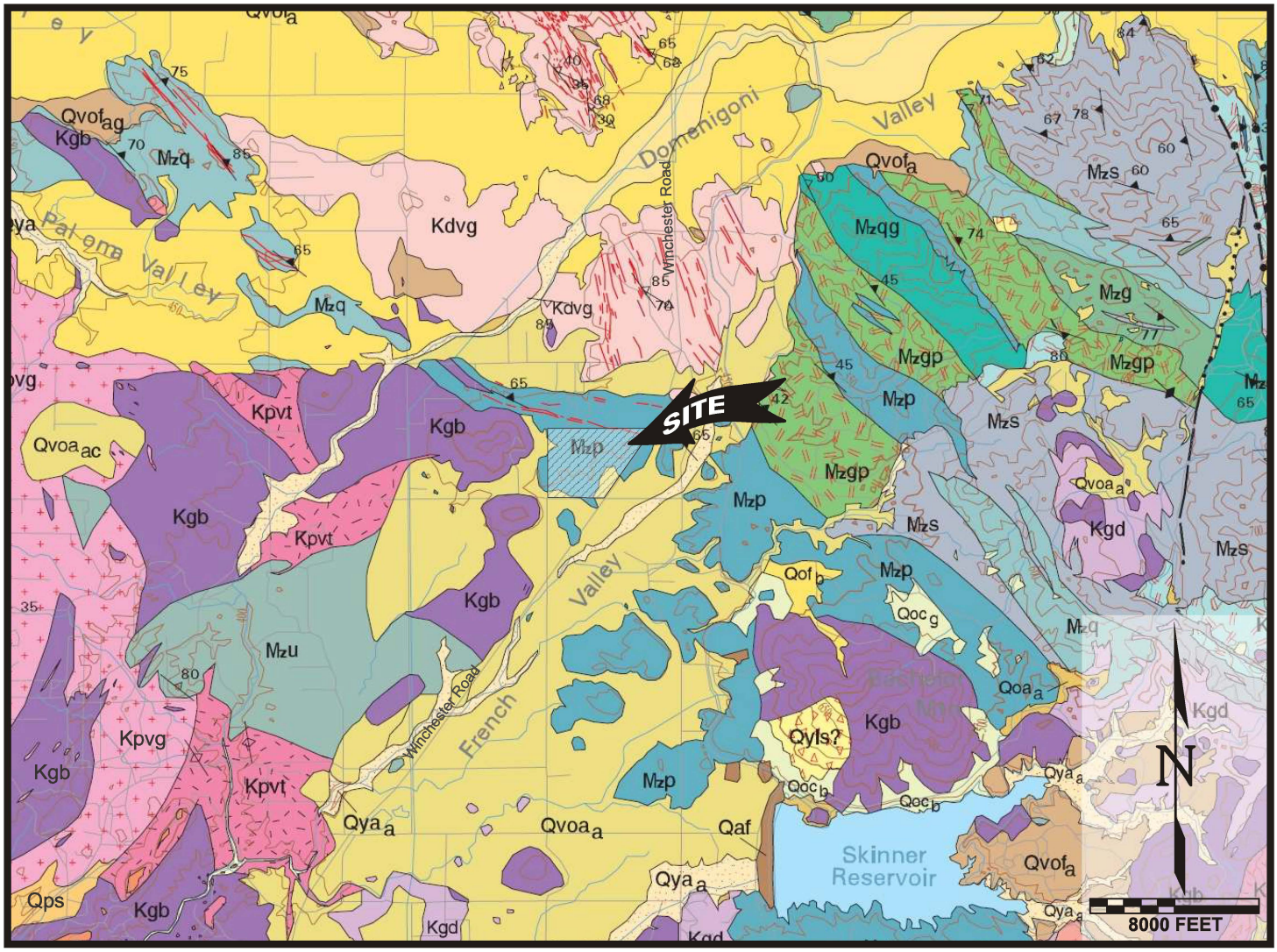
The site is generally mantled by a relatively thin layer of colluvium/topsoil. These soils are mostly residual having developed through weathering and decomposition of the underlying bedrock materials. Thickness of the colluvium/topsoil ranges from approximately ±1 to ±2 feet over the site. The colluvium/topsoil generally consists of reddish brown, sandy clay to clayey sands. These soils were generally dry to moist. All colluvium/topsoil is loose to medium dense, porous, and is considered subject to compression and hydro-collapse. Accordingly, the colluvium is considered unsuitable for support of additional fill, improvements, or structures in its' existing state and therefore will need to be removed and recompacted in areas proposed for settlement-sensitive improvements.

Very Old Axial Channel Deposits (Map Symbol - Qvoa)

Middle to late Pleistocene-age very old alluvial fan deposits, as classified by Morton (2001), blankets the lower site elevations and in-fills the valleys. The mapped surficial extent of this unit is shown on Plate 1. Locally, the very old alluvial fan deposits mask underlying bedrock highs (less then 4 feet in depth) as evidenced by Test Pit 3. These fluvial sediments were deposited on valley floors. They consists of moderately to well-indurated, reddish brown, mostly very dissected gravel, sand, silt, and clay-bearing alluvium.

Bedrock - Mesozoic-age Phyllite (Map Symbol - M_zp)

Mesozoic-age meta-sedimentary rocks, classified by Morton (2001) as phyllite underlie the site at varying depths. The Mesozoic sequence of meta-sedimentary rocks are identified as the end rocks of the Peninsular Ranges batholith. These rocks can be described as a fissile black phyllite that commonly has a sheen produced by very fine-grained white mica. The unit is highly foliated with a predominant east-west strike with steep northerly dips. The phyllite bedrock materials were generally encountered at depth underlying the very old alluvial fan deposits within the incised drainage and flatter areas onsite and outcrop at the higher elevations. Minor interbeds of quartzite and a probable xneolith or stoped block of the batholith have also been encountered onsite (EUC, 2005). These rocks were generally moderately weathered in the near surface and become hard to very hard with depth.



Base Map: Morton, D. M., (2004)

EXPLANATION

- Qaf — Artificial Fill (Recent)
- Qyls — Young Landslide Deposits (Holocene)
- Qya — Young Axial Channel Deposits (Holocene and late Pleistocene)
- Qoc — Old Colluvial Deposits (late to middle Pleistocene)
- Qvof — Very Old Alluvial Fan Deposits (middle to early Pleistocene)
- Qvoa — Very Old Axial Channel Deposits (middle to early Pleistocene)
- Kdvg — Domenigoni Valley Pluton - Granodiorite to Tonalite (Cretaceous)
- Kpvg — Paloma Valley Ring Complex (Cretaceous) Monzogranite to Granodiorite
- Kpvt — Tonalite
- Kgd — Peninsular Ranges Batholith (Cretaceous) Granodiorite, Undifferentiated
- Kgb — Gabbro
- Mzu — Metasedimentary Rocks, Undifferentiated (Mesozoic)
- Mzq — Quartz-Rich Rocks (Mesozoic)
- Mzqg — Intermixed Quartzite and Graywacke (Mesozoic)
- Mzgp — Intermixed Graywacke and Phyllite (Mesozoic)
- Mzp — Phyllite (Mesozoic)
- Mzs — Schist (Mesozoic)
- Geologic Contact, dashed where approximate
- Strike and Dip of Primary Igneous Foliation (Vertical)
- Strike and Dip of Metamorphic Foliation (Vertical)
- 70 — Bearing and Plunge of Linear Features
- 70 — Fault, dashed where approximate, dotted where concealed

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REGIONAL GEOLOGIC MAP

Figure 3

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GEOLOGIC STRUCTURE

The very old alluvial deposits identified are generally massively to horizontally bedded. Structural elements within the phyllite rocks in the study area are characterized by a predominant east-west trending foliation that steeply dips to the north. Continuous or consistent adverse foliation or joint systems were not noted during our field investigation; however, our review indicates that generally jointing parallels the regional northwest trend. Based on the available data, adverse geologic structures are not anticipated to affect the proposed development.

GROUNDWATER

Seeps, springs, mottled soils, or other indications of a high regional groundwater level were not noted during the time of our field investigation. Provisions for the installation of subdrains should be considered in the low-lying drainage swales. During periods of heavy precipitation, shallow groundwater should be anticipated at the soil/bedrock contact, and at daylight occurrences of discontinuities.

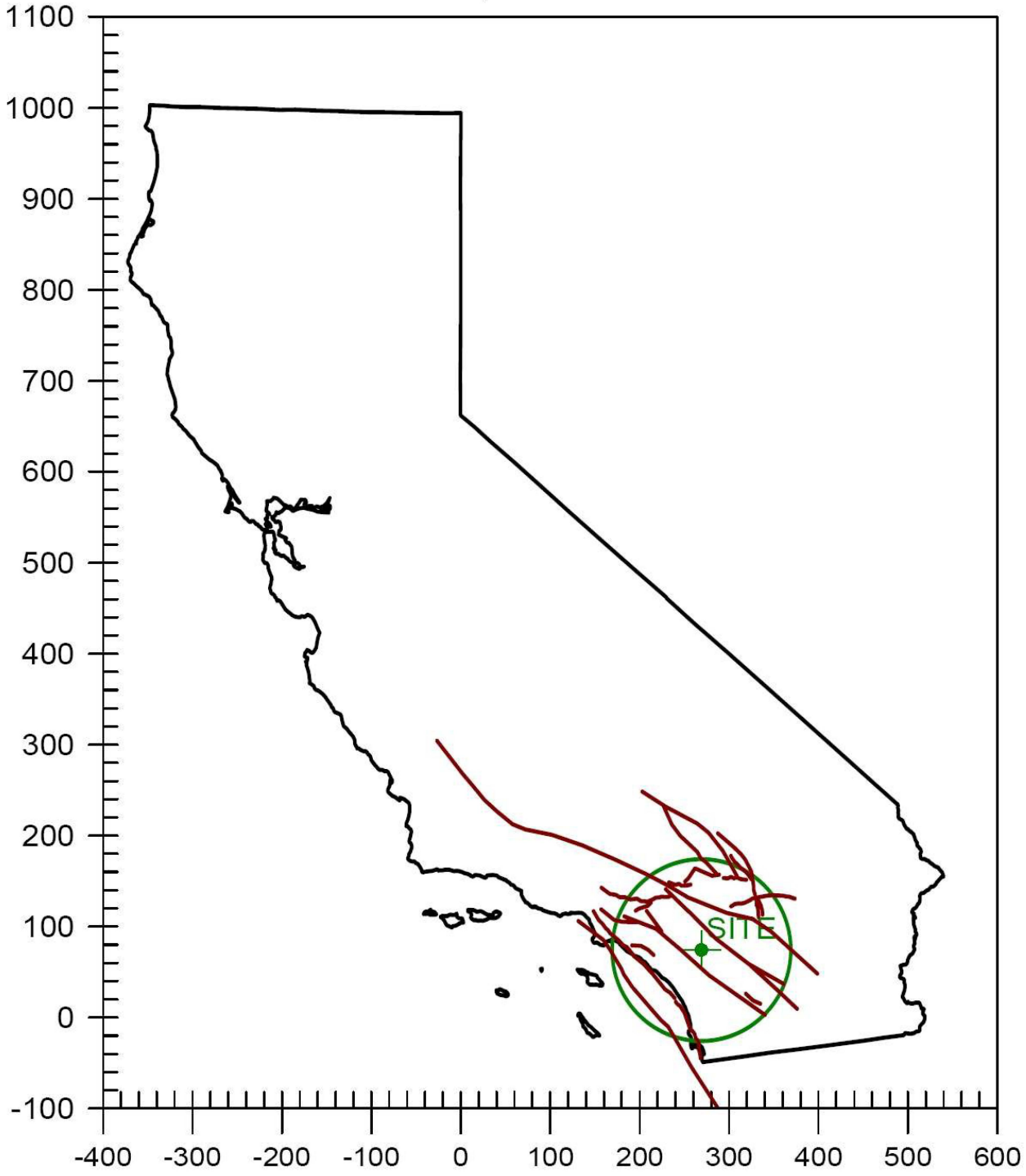
FAULTING AND REGIONAL SEISMICITY

The possibility of ground shaking at the site may be considered similar to the southern California region as a whole. The site is situated in an area of active as well as potentially-active faults. The nearby Temecula segment of the Elsinore fault zone, the design fault for the site, is considered active and is included within Alquist-Priolo Earthquake Fault Zone. Our review indicates that there are no known active faults crossing the site, and the site is not within a Fault-Rupture Hazard Zone (Hart and Bryant, 2007). During our review of aerial photographs (USDA, 1980), GSI did not observe photolineaments or other features specifically indicative of active (as defined by Hart and Bryant [2007]) faulting crossing the site. A map and list of the major faults and fault zones in southern California that could have a significant effect on the site, should they experience activity, is provided in Appendix C. The relationship of the site location to these major mapped faults is indicated on the California Fault Map (see Figure 4).

The acceleration-attenuation relations of Campbell and Bozorgnia (1997 Rev.), have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the random mean plus 1-sigma attenuation curves developed by those authors. These acceleration-attenuation relations have been incorporated in EQFAULT, a computer program by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources. The program estimates the closest distance of each fault to the selected site location. 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 the upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is

CALIFORNIA FAULT MAP

W.O. 5815-A-OC, Hanna Marital Trust



ALL LOCATIONS APPROXIMATE

N

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

CALIFORNIA FAULT MAP

Figure 4

W.O. 5815-A-OC	DATE 07/09	SCALE: Bar Scale

computed by user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the above, peak horizontal ground accelerations from an upper bound event may be on the order of 0.32g.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Campbell and Bozorgnia (1997 Rev.), and the computer program EQSEARCH (Blake, 2000b, updated to June 2008). This program performs a search of historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-mile radius, between the years 1800 to June 2008. 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 through June 2008 was 0.31g. In addition, site specific probability of exceeding various peak horizontal ground accelerations and a seismic recurrence curve are also estimated/generated from the historical data. Printouts from EQSEARCH are provided in Appendix C.

A probabilistic seismic hazards analysis was performed using FRISKSP (Blake, 2000c) which models earthquake sources as 3-dimensional planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of these data, and considering the relative seismic activity of the southern California region, a peak horizontal ground acceleration of 0.33g was calculated. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period). Printouts from FRISKSP are also included in Appendix C. The data is summarized for a 50- and 100-year design life in the table below:

DESIGN LIFE (YEARS)	EARTHQUAKE LEVEL	PROBABILITY OF EXCEEDANCE (%)	CALCULATED PEAK GROUND ACCELERATION (G)		CALCULATED RETURN PERIOD (YEARS)
			WITHOUT MAGNITUDE WEIGHTING FACTOR	WITH MAGNITUDE WEIGHTING FACTOR	
100	Operating	50	0.242	0.215	144
100	Contingency	10	0.374	0.329	949
50	Operating	50	0.20	0.167	72
50	Contingency	10	0.326	0.288	475

Seismic Design Parameters

The following seismic design parameters should be used to construct the Design Response Spectrum, per Chapter 16A of the 2007 CBC; however, the actual construction of the Design Response Spectra should be based on the actual design grades and specific plans:

2007 CBC	DESCRIPTION	FACTOR		
Table 1613.A.5.2	Site Class	A (rock)	B (unweathered rock)	C (> 10' of fill beneath foundations)
Figure 1613.5(1)	Seismic Region	1		
Figure 1613.5(3)	Mapped Acceleration Parameter, S_S	1.2	1.5	1.5
Figure 1613.5(5)	Mapped Acceleration Parameter, S_1	0.5	0.6	0.8
Table 1613A.5(1)	Site Coefficient, F_a	0.8	1.0	1.0
Table 1613.A.5.3(2)	Site Coefficient, F_v	0.8	1.0	1.3
Nearest/Design Fault	Elsinore (Temecula) Fault	15.5 km		

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large-scale earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of low order or non-design seismic events can increase the level of damage onsite if mitigation and repairs are not made after each significant seismic event.

Seismic Hazards

The following list includes seismic related hazards other then ground shaking that have been considered during our evaluation of the site.

Seismic Settlement - liquefaction and/or dry sand settlement
Lateral spreading
Surface Fault Rupture
Ground Lurching or Shallow Ground Rupture

The hazards listed are considered low to negligible and/or completely mitigated as a result of site location, soil characteristics, and typical site development procedures. Such procedures would include, but not be limited to, the removal and recompaction of near surface potentially compressible and/or liquefiable soils, appropriate foundation design, appropriate structural design, etc.:

It is important to keep in perspective that in the event of an upper bound (maximum credible) earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. This potential would be no greater than that for other existing structures and improvements in the immediate vicinity.

Liquefaction and Densification

Liquefaction of cohesionless soils can be caused by a strong, vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils with a certain grain size distribution, saturated by a relatively shallow groundwater table are most susceptible to liquefaction. In view of the relatively dense nature of the underlying bedrock materials and lack of colluvial/alluvial deposits, which will be removed and recompacted during site grading for settlement-sensitive improvements, it is our opinion that the potential for liquefaction or associated seismically-induced dynamic settlement is considered low. Densification is a phenomenon where volumetric strain, due to seismic loading of loose to medium dense granular dry to moist near-surface soils, can cause surface settlement. Some volumetric strain of the fill may occur during the design seismic event resulting in some surface settlement on the order of 0.25 to 0.5 percent of the fill thickness. However, due to the anticipated low magnitude and the relatively dense nature of the underlying earth materials, no specific mitigative measures above the recommendations provided herein are considered necessary. Densification may be reduced by increasing the compactive efforts of the fill soils on the order of 90 to 95 percent (ASTM D 1557).

In accordance with Special Publication 117 (SP 117 [California Department of Conservation, California Geological Survey, 2008]), this report may be considered a "screening investigation," as discussed in SP 117. Based on our investigations, groundwater is in excess of 50 feet. Sediments underlying the site generally do not consist of medium to fine grained, cohesionless sands, but rather very dense meta-sedimentary bedrock, with relatively high shear wave velocities. Such materials are generally not prone to liquefaction. It is therefore our opinion that our investigation demonstrates the absence of a liquefaction hazard per SP 117 and that no further investigation or analyses of this nature are warranted.

OTHER GEOLOGIC HAZARDS

Mass Wasting

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, rockfall, deep-seated landslides, etc. 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, 1998, 2004, 2005, and 2007, creep-affected materials may become saturated, resulting in a more rapid form of down slope movement (i.e., landslides, surficial failures, and/or rockfall). Since the subject site consists of very hard bedrock, indications of significant mass wasting phenomena were not observed during our review of stereoscopic photographs of the site area (USDA, 1980) or during our site investigation. The potential for mass wasting phenomena to affect the site is considered low, provided our recommendations are properly implemented. Likewise,

the potential for seismically induced landsliding is considered low. This does not preclude surficial erosion and instability of granular fill or natural soils if unplanted slopes and pads are exposed to storms of sufficient intensity and duration.

Subsidence

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., alluvium vs. bedrock) are present. In view of the relatively dense nature of the underlying bedrock materials and the generally thin overlying alluvial deposits that will be removed and recompacted in areas proposed for settlement-sensitive improvements, the potential for this phenomena to affect the site is considered low.

In addition, our review of available data, as well as stereoscopic aerial photographs (USDA, 1980), showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression[s], linearity of depressions associated with mountain fronts, etc.). Ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faulting. Our review did not reveal any information that active faulting or excessive groundwater withdrawal, ground fissures, or hydroconsolidation in the specific site vicinity, is occurring at this time. Therefore, the potential for areal subsidence or ground fissures is considered low.

LABORATORY TESTING

Classification

The soils were classified visually by GSI during our site investigation according to the Unified Soils Classification System and by supplemental index testing, as warranted. The soil classifications are shown on the Test Pit Logs (Appendix B).

Field Moisture and Density

Field moisture content and dry unit weight were determined for relatively "undisturbed" samples of earth materials obtained from GSI's exploratory excavation. The dry unit weight was determined in general accordance with ASTM D 2937, in pounds per cubic foot (pcf), and the field moisture content was determined as a percentage of the dry weight. Water contents were measured in general accordance with ASTM D 2216. Results of these tests are summarized on the Boring Logs (see Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was determined for the major soil type encountered in the exploratory test pits, in general accordance with the laboratory

standard ASTM D 1557. The moisture-density relationship obtained for the site soil is shown below:

SOIL TYPE (Map Unit)	LOCATION AND DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
CLAYEY SAND, Reddish Brown (Qvoa)	TP-2 @ 0-5'	125.0	10.0
CLAYEY SAND w/GRAVEL, Grayish Brown (Topsoil)	TP-8 @ 0-5'	124.0	12.0

Expansion Potential

Expansion Index (E.I.) testing was performed on representative samples of site earth materials in general accordance with ASTM D 4829 and classified in accordance with Table 18-I-B of the UBC/CBC (ICBO, 1997 and 2001). Test results of E.I. = 41 (TP-2 at 0-5 feet), E.I. = 17 (TP-8 at 0-5 feet) and 75 (TP-10 at 4-5 feet) indicate that the surficial site soils where tested are generally low to medium in expansive potential (E.I. from 21 to 90). However, higher expansive soils may be encountered onsite. Based on the above, and requirements of the controlling authorities, additional E.I. tests should be performed during future development to further evaluate the conditions encountered during this preliminary study.

Plasticity Index Testing/Atterberg Limits

For foundation planning purposes, plasticity index testing was performed where E.I. testing indicated an E.I. greater than 20. Testing was performed to evaluate the liquid limit, plastic limit and plasticity index in general accordance with ASTM D 4318. The test results were also utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. For building pads with an E.I. greater than 20, foundations should be specifically designed in accordance with Section 1815 and/or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001) and/or 1802A.3.2 criteria designed in accordance with 1805.A.8 (CBC [CBSC, 2007]). The plasticity index test results are presented in the following table:

LOCATION AND DEPTH (ft.)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-2 @ 0-5'	25	16	9
TP-8 @ 0-5'	29	20	9
TP-10 @ 4-5'	32	21	11

Direct Shear

A strain-controlled direct shear test was performed on a remolded sample in general conformance with the ASTM D 3080 test method. The test results are presented in Appendix D.

Consolidation Testing

Consolidation testing was performed on relatively undisturbed soil samples in general accordance with ASTM Test Method D 2435. The consolidation test results are presented in Appendix D.

Grain Size Distribution

Representative samples were dried, weighed, and soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The samples were prepared in general accordance with ASTM Test Method D 421. The portion of the material retained on the No. 200 sieve was oven-dried and then run through a standard set of sieves in general accordance with ASTM Test Method D 422. The test results are presented in Appendix D.

Soluble Sulfates/Corrosion

A typical sample of the site materials was analyzed for soluble sulfates, pH, resistivity, etc. The soluble sulfate and corrosion potential results are shown as follows:

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES PERCENTAGE BY WEIGHT	pH	RESISTIVITY (OHMS-CM)
TP-2 @ 0-5'	0.0035	7.39	1,500
TP-8 @ 0-5'	0.0020	7.22	850

For preliminary planning purposes, based upon the soluble sulfate test results and the UBC/CBC (ICBO, 1997 and 2001) or Table 4.3.1 of ACI 318.08 for CBC requirements (CBSC, 2007), the soluble sulfate content is categorized as negligible (0.00-0.10 Water-Soluble Sulfate in Soil, percentage by weight) and sulfate-resistant concrete should not be necessary. Additionally, a modified cement to water ratio and modified concrete compressive strength should not be necessary.

Based on the results of the resistivity and pH testing, the onsite soils are generally neutral to mildly alkaline (a pH of 7.4 to 7.8 is considered mildly alkaline) and are considered corrosive to severely corrosive toward ferrous metals in a saturated state (1,000 to

2,000 ohm-cm is considered corrosive, and under 1,000 ohm-cm is considered severely corrosive). Although the site soils are categorized as being corrosive to severely corrosive to buried ferrous metals, no exposure conditions stated in Table 19-A-2 of the UBC/CBC (ICBO, 1997 and 2001) were indicated by our limited laboratory testing. However, the designers should review the requirements of ACI 318-08 from CBC (CBSC, 2007) compliance. It is our understanding that ferrous metals embedded in properly placed and formed Type I, II, or V concrete should be adequately protected from these conditions. Typical development does not generally use significant amounts of exposed metal piping and/or foundation elements. Additionally, based on the conditions encountered, a consulting corrosion engineer should be retained to provide specific advisement for buried improvements including foundations, piping, etc. The laboratory test results, including chlorides, are presented in Appendix D.

R-value

Preliminary pavement design is provided in the "Preliminary Conclusions and Recommendations" section of this report. As per the requirements of the controlling authorities, additional R-value sampling should be performed during future development to further evaluate the conditions encountered during our preliminary subsurface investigation. GSI recommends that R-values following utility placement be obtained in general accordance with Riverside County and Green Book (Public Works Standards, Inc., 2006) standards, whichever is more stringent.

EARTH MATERIAL EXCAVATION CHARACTERISTICS EVALUATION

A seismic refraction survey, consisting of 12 seismic refraction lines or traverses, was conducted on the site using a Geometrics SmartSeis 12-channel exploration seismograph, with a hammer and plate energy source. The approximate locations of the seismic traverses (ST) are shown on the enclosed Plate 1, and the velocity and depth interval results are summarized in Appendix E. An example of the raw seismic data is included as Plate E-1 in Appendix E, and illustrates a forward and split spread shot from the same line.

The first arrival information, shot point locations, geophone locations, and line geometry from each survey are utilized in the computer program SIPwin (Rimrock Geophysics, 2004) which produces time-distance plots for each of the survey lines (see example Plate E-2 in Appendix E). The graphic curves reflect the actual time-distance plots generated by the program, showing the shot points and phone locations. The first curve, from left to right shows the forward spread from the first shot. The second, or split spread shot point creates two curves in opposite directions from the shot in the middle of the spread. The third curve represent the reverse shot from the distant end of the spread.

The data for the surveys performed generally show a three-layer case. The uppermost layer is generally thin as would be expected, reflecting the surficial materials

(i.e., topsoil/colluvium). Undulations in time-distance curves can be attributed to a lack of elevation corrections to the raw data, possible minor disturbances from noise (e.g., wind or traffic), decreased energy at distant geophones, and discontinuities in the subsurface.

The velocity-depth models, or cross-sections, generated are included as Plates E-3 through E-14 (see Appendix E) for ST-1 through ST-12, respectively. As can be seen on these plates, the boundaries between various seismic velocity layers appear to be somewhat undulatory, typical of fractured and weathered bedrock where there is an access to the subsurface for air and water. Fracture density/frequency also contributes to the variation in depth of weathering and therefore differences in seismic velocities.

Layer boundaries tend to mimic the surface topography, although variations are common depending upon the depth of weathering, fracturing, etc. Thickness of the surficial layer (i.e., colluvium or weathered bedrock) averages about ±3 feet, but varies from ±2 to ±6 feet. Average velocity of Layer 1 materials is about ±1,239 feet per second (fps). This is typical for such material. The average thickness of Layer 2 (generally moderately to slightly weathered bedrock) is about ±28 feet, but ranges from ±20 to ±39 feet. Average velocity of Layer 2 is about ±3,696 fps, with some variability. Average velocity in Layer 3 (relatively fresh bedrock), is about ±8,480 fps, with seismic velocities ranging from 5,319 to 13,333 fps. At depths where velocities are greater than about 5,000 to 6,000 fps, rippability is ambiguous and blasting usually is required.

An evaluation has been made of the seismic refraction line data to estimate the approximate depth to non-rippable trenching (i.e., utility excavation) and to non-rippable bedrock. Approximate cut-off velocities of ±3,800 and ±6,000 fps were used as a basis for non-rippable trenching (assuming a Cat 235 Hoe [a large trackhoe], or equivalent), and non-rippable bedrock (assuming a D9L, or equivalent), respectively.

Bedrock excavatability with respect to trenching shallower than the approximate ±3,800 fps cut-off depth is expected to vary from easy to very difficult and the necessity for localized areas requiring blasting cannot be entirely precluded. Similarly, bedrock rippability shallower than the approximate ±6,000 fps cut-off depth is expected to vary from easy to very difficult, and the necessity for localized areas requiring blasting cannot be entirely precluded.

Variations should be expected. As such, bedrock excavations from the surface downward may generate oversize rock. Isolated "floaters" or corestones may also be encountered. The bulk of the materials derived from the weathered portion of the bedrock (up to and including the ±3,800 to 6,000 fps cut-off) are anticipated to disintegrate to approximately 12 to 24 inches and smaller constituents. Any oversize materials generated would require special handling for use in fills, and should not be placed within 10 feet of finish grade or used as backfill in utility trenches (County of Riverside earthwork standards, current and adopted). It is possible that the governing agency may have more stringent requirements with respect to rock size and/or hold-down distances.

Based upon our experience in this area, and the seismic refraction data obtained, the following table reflects our estimates of the rippability at the various seismic refraction survey lines; other interpretations are possible:

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-1	Rippable with to a depth of ±29 feet on the northwest, and ±30 feet on the southeast. Trenchable to a depth of ±29 feet on the northwest, and ±30 feet on the southeast. Marginally rippable below those depths, blasting may be necessary.
ST-2	Rippable to a depth of ±32 feet on the southeast, and ±29 feet on the northwest. Blasting likely below those depths. Non-trenchable below ±4 to ±5 feet on the southeast and ±3 to ±4 feet on the northwest.
ST-3	Rippable to a depth of ±30 feet on the southeast, and ±23 feet on the northwest. Blasting likely below those depths. Trenchable with difficulty to a depth of ±30 feet on the southeast, and ±23 feet on the northwest.
ST-4	Rippable to a depth of ±28 feet on the northwest, and ±41 feet on the southeast. Blasting likely below those depths. Non-trenchable below ±2 to ±3 feet on the northwest, and ±2 to ±3 feet on the southeast.
ST-5	Rippable to a depth of ±37 feet on the north, and ±33 feet on the south. Blasting likely below those depths. Non-trenchable below ±2 to ±3 feet on the north and south.
ST-6	Rippable to a depth of ±36 feet on the north, and ±30 feet on the south. Blasting likely below those depths. Trenchable with difficulty to a depth of ±36 feet on the north, and ±30 feet on the south.
ST-7	Rippable to a depth of ±25 feet on the northwest, and ±38 feet on the southeast. Blasting likely below those depths. Trenchable to a depth of ±25 feet on the northwest, and ±38 feet on the southeast.
ST-8	Rippable to a depth of±23 feet on the south, and ±30 feet on the north. Blasting likely below those depths. Trenchable with difficulty to a depth of ±23 feet on the south, and ±30 feet on the north.
ST-9	Rippable to a depth of ±30 feet on the west, and ±27 feet on the east. Blasting likely below those depths. Non-trenchable below a depth of ±3 feet on the west, and ±4 feet on the east.
ST-10	Rippable to a depth of ±32 feet on the southeast, and ±41 feet on the northwest. Blasting likely below those depths. Very difficult trenching below ±2 to ±3 feet on the southeast, and ±2 to ±3 feet on the northwest.
ST-11	Rippable to a depth of ±24 feet on the north, and ±38 feet on the south. Blasting likely below those depths. Trenchable with difficulty to a depth of ±24 feet on the north, and ±38 feet on the south.
ST-12	Rippable to a depth of ±27 feet on the east, and ±32 feet on the west. Blasting likely below those depths. Non-trenchable below a depth of ±2 to ±3 feet on the east, and ±3 to ±4 feet on the west.

Due to the variable nature of the metasedimentary bedrock terrain, blasting should be anticipated to achieve proposed cut depths and/or street/roadway undercuts for utility

construction where indicated above. Locally very difficult ripping and trenching should also be anticipated.

Using the 3,800 fps cut-off for non-rippable trenching (in lieu of street overexcavation), very difficult ripping or blasting (e.g., "line-shooting") may be necessary for trenching of utilities along the proposed roadway alignments, depending upon the final design grades. However, in order to ease the effort of excavating for utilities, overexcavation of street areas during mass grading to 1 foot below the depth of the lowest utility should be considered. This is not a geotechnical requirement in streets; however, it may be mandated by the governing agency. The use of particles larger than 4 inches in utility trench backfill may be restricted by the individual utility company/agency and may be more stringent than Riverside County standards.

Summary

In general, utilizing the seismic data, it appears that the site may be characterized as being underlain by a thin layer of surficial soils (topsoil/colluvium) and/or weathered bedrock to depths ranging from about ±1 up to about ±6 feet in thickness. Below that horizon, basically moderately to slightly weathered bedrock occurs, ranging from approximately ±20 to ±39 feet in thickness. Below the surficial soils and differentially weathered bedrock horizons, fresh and very dense meta-sedimentary rock exists. Based on all of the above, the need for overexcavation, blasting and/or line shooting would be anticipated on the site, should proposed cut grades exceed the depths indicated herein. In order to ease the effort of excavating for utilities, overexcavation of street areas during mass grading to 1 foot below the depth of the lowest utility should be considered. It should be noted that a conventional rubber-tired backhoe will experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps. The seismic refraction data presented herein should be further reviewed in conjunction with final grading plans (when available). It should be noted that due to the variability of bedrock weathering, and the potential for local boulders/corestones, or less weathered bedrock, very difficult ripping and/or blasting cannot be entirely precluded, even locally at the surface.

PRELIMINARY EARTHWORK FACTORS

Preliminary earthwork factors (shrinkage and bulking) for the subject property have been estimated based upon our field and laboratory testing, visual site observations, and experience. It is apparent that shrinking would vary with depth and with areal extent over the site based on previous site use. Variables include vegetation, weed control, discing, and previous filling or exploring. However, all these factors are difficult to define in a three-dimensional fashion.

Therefore, the information presented below represents average shrinkage/bulking values, using certain earthwork assumptions as follows:

Maximum Density	=	125.0 pcf
Relative Compaction	=	92%

Colluvium/Topsoil (Silty to Clayey Sands) . . . . . 5-10% shrinkage
Older Alluvium (Silty Sands) . . . . . 0-10% shrinkage
Bedrock - Phyllite . . . . . 15-25% bulking

Subsidence due to dynamic compaction from equipment routes, etc., is estimated to be 0.10 feet in non-bedrock areas. It should be noted that the grading contractor also controls shrinkage by the degree of relative compaction attained in the field (i.e., 92 vs. 94 percent compaction, etc.). Accordingly, a shrinkage factor for relative compaction variances may be on the order of 2 or 3 percent additional. The above facts indicate that earthwork balance for the site would be difficult to define and flexibility in design is essential to achieve a balanced end product.

GEOLOGIC AND GEOTECHNICAL IMPACTS AND MITIGATION MEASURES

This discussion addresses potential geologic and geotechnical impacts of the site and mitigation measures relative to the impacts. This discussion assumes that the design conclusions and recommendations presented herein are adhered to during the design and construction phases of the project.

Geologic Impacts

Impact: Mass Wasting/Landslides

Mass wasting, also known as slope movement or mass movement, is the geomorphic process by which soil, regolith, and rock move downslope under the force of gravity. Types of mass wasting include creep, slides, flows, topples, and falls, each with their own characteristic features, and take place over timescales from seconds to years. No significant mass wasting features were observed at this site nor have been shown on the published maps.

Mitigation:

No mitigation measures are required for mass wasting. Mass wasting is considered to be at a level of insignificance.

Impact: Historic or Pre-existing Landslides:

Historic and pre-historic landsliding does not exist onsite nor do any off-site landslides impact the site.

Mitigation:

No mitigation measures are required for historic or pre-existing landslides. The impact of historic or pre-existing landslides is considered to be at a level of insignificance. Surficial erosion/slump features will be mitigated by the recommended cut/fill grading techniques.

Impact: Rock Fall Potential

Typically, rockfall on slopes on slopes at the slope ratios onsite only occurs during a triggering event such as ground shaking and seismic acceleration. Minor amounts of isolated boulders exist on the natural slopes. Due to the angular nature of these large rocks, and slope inclination, the potential for rock fall is considered very low.

Mitigation:

No mitigation measures are required. The rock fall potential is considered to be at a level of insignificance.

Impact: Earthquake Surface Fault-Rupture Hazards

No active faults have been identified on-site. The site does not include any portion of an Alquist-Priolo Earthquake Fault Surface Rupture zone. Hence, the potential for surface rupture due to earthquake faulting is extremely low.

Mitigation:

No mitigation measures are required. The potential for surface rupture is considered to be at a level of insignificance.

Impact: Earthquake Ground Shaking

Ground shaking should be expected at the project site during it's design life.

Mitigation:

Building code design criteria utilizing the design earthquake seismic shaking parameters are required to mitigate this hazard. Properly designed and constructed structures should protect public welfare and safety, and the impact of seismic shaking is reduced to a level of acceptance.

Impact: Earthquake Induced Ground Failure

Earthquake induced landsliding and rock fall is considered remote. Ancient and historic landsliding does not exist on site.

Mitigation:

No mitigation is required for earthquake induced ground failure. Earthquake induced ground failure is considered to be at a level of insignificance.

Impact: Tsunami, Seiche, and Flooding (Earthquake Induced Dam Failure)

The site is well removed from the Pacific Ocean and the tsunami hazard is considered non-existent. Seiche is overtopping of a confined water body and flooding can be due to dam failure or climatic causes. Lake Skinner is down gradient of the site hence flooding due to dam failure at Lake Skinner should not approach the site. Up gradient of the site is the Diamond Valley (Domenigoni) Reservoir. The site is removed from the natural drainage courses that would accomodate most of the flood waters, should the dam fail. Additionally, a ridgeline isolates the site from the potential flood zone. However, the dam innundation flood zone map from Diamond Valley should be reviewed and compared to the planning development.

Mitigation:

The design civil engineer will further address flooding potential at the site. This includes review of the Diamond Valley Dam failure innundation maps.

Geotechnical Impacts

Impact: Slope Stability

There are commonly five (5) cases for slope stability evaluation, and they are: fill slopes; cut slopes; fill over cut slopes; ascending natural slopes; descending natural slopes. Based on the site topography, proposed maximum slope heights (± 30 feet), their proposed inclinations (2:1 [h:v] or flatter), geologic characteristics of site bedrock and sediments, engineering properties of site geologic units and soils, all design and natural slopes are considered stable and exceed the minimum factor of safety requirements.

Mitigation:

No mitigation is required to reduce this potential impact with the exception of proper grading techniques and maintenance. For surficial and erosion features/slumps, see comment under pre-existing landslides.

Impact: Seismically Induced Ground Failure

Seismically induced ground failure can include liquefaction, dry sand settlement and lateral spreading. The conditions for these potential impacts are not existent at the project site.

Mitigation:

The recommended grading techniques will reduce this potential impact to less than significant.

Impact: Hydroconsolidation

Hydroconsolidation or hydro-collapse occurs when the susceptible soils are saturated and exist under a new load. This can result in differential settlements in excess of accepted industry standards. Hydroconsolidation susceptible soils are not existent onsite.

Mitigation:

The recommended grading techniques will reduce this potential impact to insignificant.

Impact: Expansive Soils:

The soils tested onsite have an expansion potential that typically ranges from very low to medium.

Mitigation:

The recommended grading techniques and foundation design in accordance with either 2001 CBC or 2007 CBC will reduce this potential impact to less than significant.

Impact: Settlement

Ground settlement of compressible soils that have been left in-place can result in differential and gross ground movement.

Mitigation:

The recommended grading techniques, including removal of all compressible soils will render this impact to a level of insignificance

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

Based on our field exploration, laboratory testing, and our experience on similar sites in the vicinity, it is GSI's opinion that the project site appears suitable for its intended use, from a soils engineering, geologic, and rock hardness/rippability viewpoint. This conclusion is contingent on the conclusions and recommendations provided herein are properly

implemented during planning, construction, maintenance, and operation of the proposed development. The primary geotechnical concerns with respect to the proposed development are:

- Depth to competent bearing material
- Expansion and corrosion potential of onsite soils
- Ongoing potential for perched groundwater/seepage
- Rock hardness (difficult ripping to blasting) of phyllite bedrock onsite.
- Potential for perched water to occur after development
- Regional seismic activity
- Seepage and perched water

The recommendations presented below should be incorporated in the design, grading, and construction phases of the project.

1. Soil engineering, observation, and testing services should be provided during grading to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

2. In general and based upon the available data, groundwater is not expected to be a factor in the grading and development of the site. However, subdrains should be placed in all canyon bottoms or along selected cut/fill contacts and outletted to an approved location to accomodate future post-development subsurface (seepage) water.

3. Cut and fill slopes should be constructed at gradients of 2:1 (h:v), or flatter, and should not exceed a maximum height of about ±30 feet without a specific slope stability evaluation. Based on our experience, field mapping, subsurface sampling, laboratory testing, 2:1 (h:v), or flatter, cut and fill slopes should be grossly and surficially stable, under normal conditions of maintenance and rainfall. However, the need for stabilization fills due to weathered and/or cohesionless materials, adverse bedding, or jointing/fractures within the bedrock may not be precluded. All cut slopes should be observed by the geotechnical consultant during grading to provide any additional recommendations if necessitated by exposed adverse geological conditions. Cut and fill slopes should be properly constructed, as discussed in the "General Earthwork, Grading Guidelines, and Preliminary Criteria" section of this report (see Appendix F).

4. Due to the low to moderate cohesive nature of some onsite materials, caving and sloughing may be anticipated to be a factor in all subsurface excavations and trenching. General Earthwork, Grading Guidelines, and Preliminary Criteria are provided at the end of this report as Appendix F. Specific recommendations are provided in the following sections. The use of shoring in accordance with Cal-OSHA standards should be followed. Foundation trenches may experience sloughing due to granular nature of the fill soils.

Demolition/Grubbing

1. Due to the past and current agricultural use of the property (i.e., row crop farming), all organic debris, including stumps, and other deleterious materials should be removed from the areas to be graded and disposed offsite. Removals of potentially compressible undocumented artificial fill (if encountered), colluvium/topsoil, and near surface weathered bedrock materials will be necessary prior to fill placement within areas proposed for settlement-sensitive improvements.

2. Any previous foundations, irrigation lines, cesspools, septic tanks, leach fields, wells or other subsurface structures uncovered during the recommended removals should be observed by GSI so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils (including all exploratory test pits) remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. The cavities should be replaced with fill materials that have been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

Treatment of Existing Ground

1. Removals of potentially compressible undocumented artificial fill (if encountered), colluvium/topsoil, alluvium, and near surface weathered bedrock materials will be necessary prior to fill placement within areas proposed for settlement-sensitive improvements.

2. For preliminary planning purposes, remedial removals will be on the order of ±1 to ±2 feet (bedrock areas) and/or ±3 to ±8 feet, or greater (alluvial drainage areas), should settlement-sensitive improvements be proposed within the area with the possibility of localized deeper removals. Remedial removals should be observed by a qualified geotechnical consultant prior to placement of planned fill soils.

3. Following remedial removals, the upper 6 inches of the underlying earth materials should be scarified moisture conditioned, and compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). The moisture content should be evaluated by the geotechnical consultant prior to any fill placement. To reduce the effects of seismic densification or proposed settlement-sensitive improvements as increased compactive efforts may be used on this project (i.e., 90 to 95 percent relative compaction).

4. Due to the nature of the underlying phyllite bedrock, oversize materials (boulders, etc., greater than 12 inches in diameter) should be anticipated during grading and likely during trenching for utilities. Excavation and placement difficulties, difficult ripping, as well as onsite disposal of these materials should be considered if

improvements are proposed within these localized areas. More stringent standards for utility trench backfill (i.e., 4 to 12 inches), oversized materials may be used by local utility entities.

5. Existing removed natural ground materials may be reused as compacted fill provided that major concentrations of organic debris, vegetation, and miscellaneous debris are removed prior to fill placement.

6. Localized deeper removals, in addition to those previously mentioned, may also be necessary due to buried drainage channel meanders or dry porous materials. The project geotechnical consultant should determine the depth of the potentially compressible materials within the proposed fill areas that will need to be removed, based on the exposed conditions during grading.

Fill Placement

1. Fill materials should be brought to at least optimum moisture content, placed in thin 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. This standard may be increased from groundwater fill supporting settlement-sensitive improvements to reduce the effects of dynamic densification (i.e., 90 to 95 percent).

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

3. Any oversized rock materials greater than 12 inches in diameter should be placed under the recommendations and supervision of the geotechnical consultant. As per the UBC/CBC (ICBO, 1997 and 2001; or CBSC, 2007) requirements, no oversize materials greater than 12 inches in diameter should be placed within 10 feet of finish grade. This may necessitate the need for additional handling to non-structural areas or construction of rock fill blankets, if the oversized materials are not removed from the area to be graded, or reduced in size, prior to site development. Such procedures are outlined in the "Rock Placement Guidelines" and "General Earthwork, Grading Guidelines, and Preliminary Criteria" sections of this report. Current and adopted County of Riverside standards should be followed and may not be more stringent than indicated herein.

4. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Foundation designs may be considerably altered if import materials have a greater expansion value and/or corrosivity characteristics than the onsite materials encountered in this investigation.

Transition Areas/Overexcavation

In order to reduce the potential for differential settlements between cut and fill materials, materials of differing expansion potentials, or different consistencies, 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. The bottom of the overexcavation area should be sloped (minimum of 1 to 2 percent) to reduce the potential for perched groundwater. This slope should be toward the thickening fill and/or to a planned subdrain area.

Subdrains

Local seepage along the contact between the bedrock and overburden materials (cut/fill contacts) or along jointing patterns of the bedrock may require subdrain systems. If required, the locations of the subdrains should be determined during grading and/or during the 40-scale grading plan review based on depth of fill, finish grade elevations, and potential flowline outlet areas. Where removals are below the subdrain flowline, the removal materials may be reused as compacted fill provided they are granular, and at a moisture content of at least 2 percent over optimum moisture content (or 1.2 times optimum moisture content, whichever is greater). If required, subdrainage for slopes and embankments should adhere to the specifications in Appendix F, which should be incorporated into the project plans and construction documents.

ROCK PLACEMENT GUIDELINES

Rock Disposal

GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rock fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet.

General

Generally for the purpose of this report, the materials may be described as either 12 inches or less, greater than 12 and less than 36 inches, and greater than 36 inches. These three categories set the basic dimensions for where and how the materials are to be placed.

Materials 12 Inches in Diameter or Less

Since rock fragments along with the overburden materials are anticipated to be a part of the materials used in the grading of the site, a criteria is needed to facilitate the placement of these materials within guidelines which would be workable during the rough grading, post-grading improvements, and serve as acceptable compacted fill.

1. Fines and rock fragments 12 inches or less in diameter may be placed as compacted fill cap materials within the building pads, slopes, and street areas as described below. The rock fragments and fines should be brought to at least optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.

2. The purpose for the 12-inch diameter limit is to allow reasonable sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 12-inch diameter size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility line trenches.

3. Fill materials 12 inches or less in diameter should be placed (but not limited to) within the hold-down depth on proposed fill pads, the upper 3 feet of overexcavated cut areas of cut/fill transition pads, and the entire street right-of-way width, including the proposed overexcavated areas and replacement fill areas, from the depth of the lowest utility (within the street and lot), to subgrade, or to the hold-down depth below finish grade. Overexcavation is discussed later in this report.

4. Utility trench backfill may restrict the use of oversize materials to 4 to 12 inches, or less.

Materials Greater Than 12 inches and Less Than 36 Inches in Diameter

1. During the process of bedrock excavation, a significant amount of rock fragments or constituents larger than 12 inches in diameter may be generated. These significant amounts of oversized materials, greater than 12 and less than 36 inches in diameter, may be incorporated into the fills utilizing a series of rock blankets.

2. Each rock blanket should consist of rock fragments of approximately greater than 12 and less than 36 inches in diameter along with fines generated from the

proposed cuts and overburden materials from removal areas. The blankets should be limited to 24 to 36 inches in thickness and should be placed with granular fines which are flooded into and around the rock fragments.

3. Rock blankets should be restricted to areas which are at least 1 foot below the lowest utility invert within the street right-of-way, at least the hold-down depth below finish grade on the proposed fill lots, and a minimum of 20 horizontal feet from the face of fill slopes, and outside of any utility laterals.

4. Compaction may be achieved by utilizing wheel rolling methods with scrapers and water trucks, track-walking by bulldozers, and sheepsfoot tampers.

5. Each rock blanket should be completed with its surface compacted prior to placement of any subsequent rock blanket or rock windrow.

6. Minor amounts of rock material in this size range may also be placed a rock windrows (see below).

Materials Greater Than 36 Inches in Diameter

1. Oversize rock greater than 36 inches in diameter should be placed in single rock windrows. The windrows should be at least 15 feet or an equipment width apart, whichever is greatest.

2. The void spaces between rocks in windrows should be filled with the more granular soils by flooding them into place.

3. A minimum vertical distance of 3 feet between soil fill and rock lift should be maintained. Also, the windrows should be staggered from lift to lift. Rock windrows should not be placed closer than 20 feet from the face of fill slopes, and outside of any utility laterals.

4. Larger rocks too difficult to be placed into windrows may be individually placed into a dozer trench. Each trench should be excavated into the compacted fill or dense natural ground a minimum of 1 foot deeper than the size of the rock to be buried. After the rocks are placed in the trench (not immediately adjacent to each other), granular fill material should be flooded into the trench to fill the voids.

5. The oversize rock trenches should be no closer together than 15 feet at a particular elevation and at least 20 feet from any fill slope face. Trenches at higher elevations should be staggered and there should be 4 feet of compacted fill between the top of one trench and the bottom of the next higher trench. Placement of rock into these trenches should be under the full-time observation of the geotechnical consultant.

6. Consideration should be given to using oversize materials in open space "green belt" areas that would be designated as non-structural fills.

SUBSTRUCTURES PLACED IN THE HOLD-DOWN DEPTH ZONE

If rock disposal becomes necessary during grading, disclosure to all interested/affected parties regarding the proximity of oversize materials, excavation difficulties, etc., that may potentially impact future improvements is recommended. The cap above the hold-down distance is only intended to support shallow foundations. Utility poles or similar improvements that penetrate or nearly penetrate the fill cap should have a site-specific subsurface investigation, and review by the geotechnical consultant, prior to planning, design, and construction. The rock hold down should be increased in pad areas to facilitate pool/spa construction or other below grade improvements. Utility entities may further restrict the use of oversized materials within the influence of pipes/pump stations, sewer manholes, or within trench backfill.

SLOPE CONSIDERATIONS AND SLOPE DESIGN

Based on our evaluation and experience on nearby projects, proposed cut and fill slopes at inclinations of 2:1 (h:v), or flatter, constructed utilizing onsite materials to the heights not exceeding 30 feet, should be grossly and surficially stable provided the recommendations contained herein are properly implemented during site development, under normal conditions of care, maintenance, and rainfall.

All cut slopes should be observed by the geotechnical consultant during grading to provide any additional recommendations if necessitated by exposed adverse geologic conditions. Slopes constructed with, or exposing, granular earth materials may be subject to erosion if left unplanted or unprotected. These potentially erosive materials may be overexcavated and blended until fines are placed in the outer 20 feet of slopes as materials are available.

All slopes should be designed and constructed in accordance with the recommendations contained within this report and applicable requirements of the UBC/CBC (ICBO, 1997 and 2001) and CBC (CBSC, 2007), or latest adopted code, and/or County of Riverside recommendations, Greenbook (Public Works Standards, Inc., 2006), and the recommendations in the "General Earthwork, Grading Guidelines, and Preliminary Criteria" section of this report (Appendix F), and the following:

1. Fill slopes should be designed and constructed at a 2:1 (h:v) gradient or flatter, and should not exceed about ±30 feet in height. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Compaction above this minimum standard may be desired if dynamic strains are not compatible with post-earthquake performance expectations.

Guidelines for slope construction are presented in Appendix F. The outer portion of fill slopes should be either overbuilt by 2 feet (minimum) and trimmed back to the finished slope or compacted in increments of 5 feet (maximum) by a sheepsfoot roller as the fill is placed and then track walked on the finished slope to achieve the final configuration.

2. Cut slopes should be designed at gradients of 2:1 (h:v) or flatter and should not exceed about ±30 feet in vertical height. While stabilization of such slopes is not anticipated, locally adverse geologic conditions (i.e., daylighted joints/fractures or severely weathered bedrock) may be encountered which may require remedial grading or laying back of the slope to an angle flatter than the adverse geologic condition.

3. Local areas of weathered bedrock (i.e., decomposed phyllite) 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 rock or cohesionless sands. Should any of these materials be exposed during construction, the geotechnical consultant, 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 observations.

4. Loose rock debris and fines remaining on the face of the cut slopes should be removed during grading. This can be accomplished by utilizing a slope board or by hand scaling, as warranted. Alternatively, replacement slopes (fill and cut) as indicated in Appendix F may be planned.

5. 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 evaluation. Final approval of the cut slope can only be made subsequent to the slope being fully cut and cleaned.

6. Cut slopes should be mapped by the project engineering geologist during grading to allow amendments to the recommendations should exposed conditions warrant alteration of the design or stabilization.

7. Fill slopes that toe into alluvium (map symbol Qal) may require additional grading and benching to support fill in acceptable bedrock earth materials.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

In the event that the information concerning the proposed development concept is not correct or any changes in the design, location, or loading conditions of the proposed structure(s) are made, the conclusions and recommendations contained in this report are for the subject site only and shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are considered minimums and are not meant to supercede design(s) by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. They are considered preliminary recommendations for proposed construction, in consideration of our field investigation, laboratory testing, and engineering analysis.

We anticipate for the planned residential and commercial construction that the average and maximum static column loads of 50 and 250 kips, respectively. Maximum wall loads are anticipated to be on the order of 2 to 6 kips per lineal foot. Based on the above, we have considered the following design alternatives:

- Isolated spread/continuous footings - very low to low expansive soils, P.I. < 15.
- Mat-type foundation (UBC/CBC [ICBO, 1997 and 2001], Section 1815 and/or1816 or 1805A.8 of the CBC [CBSC, 2007]) - medium expansive soils, P.I. ≥15.

The preliminary foundation design and construction recommendations provided herein are based on laboratory testing and engineering analysis of onsite earth materials by GSI. Recommendations for footings/foundation systems and associated design parameters are provided in the following sections. The foundation systems may be used to support the proposed structures, provided they are founded in competent bearing materials. Mat-type foundations (or other alternative foundation system such as post-tensioned slabs) will likely be recommended for parcels where the E.I. exceeds, and a P.I. at or above 15, as per the Section 1815 and/or Section 1816 of the UBC/CBC (ICBO, 1997 and 2001) or 1802A.3.2 criteria of the 2007 CBC. The site structural engineer should be informed of this to aid in preliminary foundation designs. The proposed foundation systems should also be designed and constructed in accordance with other applicable guidelines contained in the UBC/CBC (ICBO, 1997 and 2001; or CBSC, 2007).

Isolated Spread and Continuous Footings (E.I. 0 to 50 and P.I. <15)

1. Based on the anticipated foundation loads and preliminary design information provided by the Client, it is our opinion that the proposed structure(s) can favorably be supported on recompacted fill soils. Building loads may be supported on

continuous or isolated spread footings (typically 18 to 30 inches below planned grades) designed in accordance with the following recommendations.

ALLOWABLE BEARING VALUES FOR FOOTINGS ON COMPACTED FILL		
DEPTH BELOW LOWEST ADJACENT FINISHED GRADE (INCHES)	ALLOWABLE BEARING CAPACITY FOR INTERIOR SPREAD FOOTINGS (MINIMUM WIDTH = 4 FEET)	ALLOWABLE BEARING CAPACITY FOR CONTINUOUS WALL FOOTINGS (MINIMUM WIDTH = 2 FEET)
18	2.0 ksf	2.0 ksf
24	2.5 ksf	2.5 ksf
30	3.0 ksf	3.0 ksf

The above values are for dead plus live loads and may be increased by one-third for short-term wind or seismic loads. Where column or wall spacings are less than twice the width of the footing, some reduction in bearing capacity may be necessary to compensate for the effects of "group" or overlapping bearing zone action. Reinforcement should be designed in accordance with local codes and structural considerations. For footings bearing into approved bedrock, the above values may be increased by 50 percent.

The recommended allowable bearing capacity is generally based on maximum total and differential settlements indicated in a later section of this report for building areas. Actual settlement can be estimated on the basis that settlement is roughly proportional to the net contact bearing pressure. The majority of the settlement should occur during construction as the building loads are applied. Since settlement is a function of footing size and contact bearing pressure, some differential settlement can be expected between adjacent columns or walls after construction where a large differential loading condition exists. However, for most cases, static differential settlements are considered unlikely to exceed those indicated herein. With increased footing depth/width ratios, differential settlement should be less, provided a minimum fill cap is maintained beneath all footings. Alternatively, the heavily loaded foundation may bear directly into approved bedrock. GSI should review foundation plans and evaluate foundation specific load patterns.

2. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to fill soil contact when multiplied by the dead load. This may be increased to 0.4 for concrete to bedrock contacts.

3. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf) with a maximum lateral earth pressure of 2,500 pounds per square foot (psf) for foundation embedded into fill soils. These

values may be increased to 400 and 4,000 psf, respectively, if foundations are extended into approved bedrock.

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

5. Due to the anticipated granular nature of the site fill, i.e., native soils, as well as the potential for seismic loading, all footings should maintain a minimum 7-foot horizontal distance from the base of the footing to any adjacent descending slope, and minimally comply with the guidelines depicted on Figure No. 18-I-1 of the UBC/CBC (ICBO, 1997 and 2001) or Figure 1805A.3.1 of the 2007 CBC. Increases in setback for purely granular fill soil slopes should be considered.

6. Soil generated from footing excavations to be used onsite should be moisture conditioned to at least optimum moisture content and compacted to at least 90 percent minimum relative compaction, whether it is to be placed inside the foundation perimeter or in landscape/right-of-away areas. This material must not alter positive drainage patterns that direct drainage away from the structural area and toward the street or other approved area.

Construction

The following foundation construction recommendations are presented as a minimum criteria from a geotechnical engineering viewpoint. The preliminary expansion potentials of the area under the purview of this report have been tested to be generally low to medium, however, very low to medium expansive soils are likely to be encountered onsite. Therefore, foundation design should include categories for very low to medium expansive soils. Accordingly, the following preliminary foundation construction recommendations are for soils in the upper 7 feet from finish grade, which may have a very low to medium expansion potential. Selective grading techniques may be used to reduce the effects on expansive soils. For foundation design, the project's design-structural engineer or architect, may exceed the geotechnical consultant's recommendations and should take precedence over the following minimum requirements. Code compliance with either Chapter 18 of the 2001 or 2007 CBC should be followed on all foundation designs on expansive soils.

1. Conventional continuous footings should be founded at a minimum depth of 18 to 30 inches (depending on the allowable bearing value from the previous section) below the lowest adjacent ground surface for typical tilt-up loads. Interior footings may be founded at a minimum depth of 18 to 30 inches below the lowest adjacent ground surface. The entire foundation should be supported by at least 5 feet of compacted fill. Footings should have a minimum width of 24 inches. All continuous footings should be reinforced with a minimum of four No. 5 reinforcing bars, two at the top and two No. 5 reinforcing bars at the bottom.

2. Isolated exterior pier and column footings may be constructed 24 inches square by 24 inches deep, and tied to the main foundation in at least one direction with a grade beam. Isolated footing reinforcement should be designed by the project structural engineer.

3. A grade beam, reinforced as above and at least 18 inches deep (minimum of 12 inches square in cross), should be provided across garages, large, or wide entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings. For commercial structures, grade beams should be increased to 18 inches square (in cross sections) across industrial garage openings or between heavily loaded columns.

4. Concrete slabs should be constructed and underlain with a vapor retarder and under slab treatment as indicated in the "Soil Moisture Transmission Considerations" section of this report.

5. A minimum slab thickness of 5 inches is recommended, and the slab subgrade should be free of loose and uncompacted material prior to placing concrete. The design engineer should determine the actual thickness of concrete slabs based upon proposed loading and use.

6. Concrete slabs should be minimally reinforced with No. 3 reinforcement bars (per the UBC/CBC [ICBO, 1997 and 2001; or CBSC, 2001]), placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis).

7. 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.

8. Specific presaturation is not required for very low to low expansive soil conditions; however, the moisture content of the subgrade soils should be equal to, or greater than, optimum moisture to a depth of 18 to 30 inches below the adjacent ground grade in the slab areas for very low to low expansive soils and/or the depth of the foundation. This should be evaluated by the geotechnical consultant within 72 hours of the vapor retarder and steel reenforcement placement. Soils with the upper 7 feet that exhibit medium expansive soil properties after grading and/or blending and have a P.I. >15 will require additional recommendations for foundations and presaturation.

9. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard (ASTM D 1557), whether it is to be placed inside the foundation perimeter or within surrounding areas. This material must not alter positive drainage patterns that direct drainage away from structural areas and/or toward approved drainage areas.

10. Foundations near the top of slope should be deepened to conform to the latest edition of the UBC/CBC (ICBO, 1997 and 2001) or Figure 1805A.3.1 of 2007 CBC, and provide a minimum of 7 feet horizontal distance from the slope face. Rigid block wall designs, located along the top of slope, should be reviewed by a geotechnical consultant.

FLOOR SLAB DESIGN RECOMMENDATIONS - ISOLATED SPREAD AND CONTINUOUS FOOTING FOUNDATION SYSTEMS

General

Concrete slab-on-grade floor construction is anticipated. The following are presented as minimum design parameters for the slab, but they are in no way intended to supercede design by the structural engineer. Design parameters do not account for concentrated loads (e.g., fork lifts, heavy rack loads, other machinery, etc.) and/or the use of freezers or heating boxes.

These recommendations are meant as minimums. The project architect and/or structural engineer should review and verify that the minimum recommendations presented herein are considered adequate with respect to anticipated uses.

Light Load Floor Slabs

The slabs in areas that will receive relatively light live loads (i.e., residential or office space, less than 50 psf) should be a minimum of 5 inches thick and be reinforced with No. 3 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel.

The project structural engineer should consider the use of transverse and longitudinal control joints to help control slab cracking due to concrete shrinkage or expansion. Two of the best ways to control this movement are: 1) add a sufficient amount of reinforcing steel to increase the tensile strength of the slab; and 2) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion. Transverse and longitudinal crack control joints should be spaced no more than 12 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. Either ACI (American Concrete Institute) or PCA (Portland Concrete Association) guidelines should be followed in the layout of crack and exploration jacket design.

Heavy Load Floor Slabs

The project structural engineer should design floor slabs in areas subject to high industrial/commercial loads (machinery, forklifts, storage racks, etc. exerting >50 psf). The Modulus of subgrade reaction (K_S-value) may be used in the design of the floor slab supporting heavy truck traffic, fork lifts, machine foundations, and heavy storage areas. A K_S-value of 100 pounds per square inch per inch (pci) would be prudent to utilize for preliminary slab design. An R-value test and/or plate load test may be used to verify the K_S-value on near-surface fill soils.

Concrete slabs should be at least 6 inches thick and reinforced with No. 4 reinforcing bars placed 12 inches on center in two horizontally perpendicular directions and near the vertical midpoint of the slab. Selection of slab thickness compatibility with anticipated loads should be provided by the structural engineer.

Transverse and longitudinal crack control joints should be spaced no more than 14 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. The use of expansion joints in the slab should be considered. Concrete used in slab construction should have a maximum water/cement ratio of 0.5. Spacing of expansion or crack control joints should be modified based on the footprint of the area to be heavily loaded. PCA guidelines for detailing industrial slabs should be utilized.

Subgrade Preparation

Subgrade material should be compacted to a minimum of 90 to 95 percent of the maximum laboratory dry density for light and heavily loaded floor slabs, respectively. Prior to placement of concrete, the subgrade soils should be presaturated to 18 to 30 inches below grade (depending on the footing depth utilized) to at least the soils' optimum moisture content, for very low to low expansive soils. This should be evaluated by a field representative of GSI prior to vapor retarder placement and prior to and within 72 hours of concrete placement. Alternative methods, including sealing the subgrade surface with select sand/base and periodic moisture conditioning, may also be considered, as long as the minimum recommended soil moisture contents are achieved.

MAT-TYPE FOUNDATIONS

As an alternative to reinforced conventional foundation systems, mat-type foundations and slabs may be utilized to mitigate the effects of expansive soils and will likely be recommended for this site for foundations and slabs when the expansion potential is greater than 50 and P.I. $\geq$15 and to improve the seismic performance of foundations. The structural mat should have a double mat of steel (minimum No. 5 reinforcing bars located at 12 inches on center each way, top and bottom) and a minimum thickness of 10 inches. Mat edges should have a minimum edge footing of 18 inches wide and 18 inches deep

below the lowest adjacent grade and be embedded into properly compacted fill. Concrete mix design and slab underlayment recommendations are provided in the "Soil Moisture Transmission Considerations" section of this report. Mat-type foundations may be designed by 1997 UBC Section 1815 (Div. III) methods using an effective P.I. of 15 to 20 (laboratory tests during construction will evaluate P.I.) or Chapter 18 of the 2007 CBC.

Mat-type foundations and slabs may be designed for a modulus of subgrade reaction (K_S) of 100 pci when placed on very low to medium expansive soils with a P.I. equal to or greater than 15. This value is a unit value for a one-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations.

$$K_R = K_S \left[\frac{B+1}{2B} \right]^2$$

where: K_S = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width (in feet)

Please note that if the designers of the building foundations require an alternative to the above foundation system, post-tension slabs or other foundation recommendations in compliance with either 2001 or 2007 CBC can be provided upon request.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

Near-surface site soils are generally low in expansion potential, based on ASTM D 4829. Accordingly, GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical industrial/commercial floor coverings and improvements. Please note that typical slab moisture emission rates, range from about 2 to 27 lbs/ 24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost v. benefit analysis (owner complaints and repairs/replacement), along with disclosure to all interested/affected parties. These recommendations do not consider the requirements for a clean-room, laboratory, sensitive equipment, or other specialized equipment that may be utilized within the building. If required, GSI can provide supplemental recommendations to address specialized equipment and/or building uses.

Considering the E.I. test results of low expansion potential, anticipated typical water vapor transmission rates, floor coverings and improvements (to be chosen by the client) that can tolerate those rates without distress, the following alternatives are provided:

- Non-structural concrete slabs should be a minimum of 5 inches thick. See discussions for light and heavily loaded floor slabs for additional slab thickness recommendations.

- Concrete slab underlayment should consist of a 10-mil to 15-mil vapor retarder, or equivalent, with all laps sealed (ICBO, 2001; or CBSC, 2007) and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A or B criteria, and be installed in accordance with ACI 302.1R-04.

- The 10- to 15-mil vapor retarder (ASTM E 1745 - Class A or B) shall be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.).

- The vapor retarder may be placed directly on properly compacted subgrade soils with very low to low expansion potential, and should be overlain by a 2-inch thick layer of washed sand (SE>30).

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 19-A-4 of the UBC/CBC (ICBO, 1997 and 2001) for corrosion or other corrosive requirements or Table 4.3.1 of ACI-318-08 for 2007 CBC requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or "waterproofing" specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

- Where slab water/cement ratios are as indicated above, and/or admixtures used, 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 owner(s) should be specifically advised which areas are suitable for tile flooring, vinyl flooring, or other types of water/vapor-sensitive flooring and which are not suitable. In all planned floor areas, flooring shall be installed per the manufactures recommendations.

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and water-proofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the commercial foundations or improvements. The vapor retarder contractor should have representatives onsite during the initial installation.

PRELIMINARY FOUNDATION AND FILL SETTLEMENTS

In addition to designing foundations (conventional, mat-type, or other) for the soil conditions described herein, the estimated vertical deformation and angular distortion values that an individual structure (including walls, footings, and/or other settlement-sensitive improvements, etc.), could be subjected to should be evaluated by a structural engineer. The levels of angular distortion were evaluated on a 50-foot length assumed as the minimum dimension of building spans; if, from a structural standpoint, a decreased or increased length over which the differential settlement is assumed to occur is justified, this change should be incorporated into the design. Localized foundation settlements are anticipated to be ¾-inch and ½-inch for total and differential vertical static and dynamic settlements, respectively. Differential vertical settlements should be applied between the heaviest and lightest foundation elements (i.e., between heaviest column or wall footing). These localized settlements are anticipated to be mostly complete at the conclusion of construction. Dynamic settlement of 0.25 to 0.5 percent of the fill thickness may occur in a differential measurement. This dynamic differential settlement will likely occur. Cross a horizontal distance equal to the fill thickness (i.e., 200 foot thick fill, 20 foot horizontal at 0.25 to 0.5 percent.

A preliminary evaluation of primary vertical deformation and secondary compression within the fill for the area under the purview of this report has been conducted (see Appendix E). On a preliminary basis, the footings and/or slabs should be designed to accommodate a total vertical static and seismic deformation of up to 1½ inches and a differential settlement of 1 inch (i.e., 1 inch in a 50-foot distance). Any post-construction settlement of the fill should be mitigated by proper foundation design, provided the design parameters, provided in this report, are properly utilized in final design of foundation system. In addition to the above, the structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the County and/or the UBC/CBC (ICBO, 1997 and 2001; or CBSC, 2007).

The dynamic component of anticipated settlement may be reduced if compaction standard requirements for fill soils are increased to above the minimum to 90 to 95 percent of ASTM D 1557. This will likely reduce the potential settlements below the settlement values indicated in this section. The County typically requires fills thicker than 50 feet (including

remedial removal areas), to be placed at a minimum of 95 percent of the laboratory standard.

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. 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.

WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below 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 50) 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. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

Restrained Walls/Loading Dock 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. GSI recommends that loading dock walls be designed for a restrained condition. Temporary loads of stockpiled goods/supplies adjacent to the wall, should be applied to wall pressures. GSI recommends that 40 percent of these temporary loads should be added to the lateral pressures in order to account for these transient loading conditions.

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

Riverside County standard design. Active earth pressure 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. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

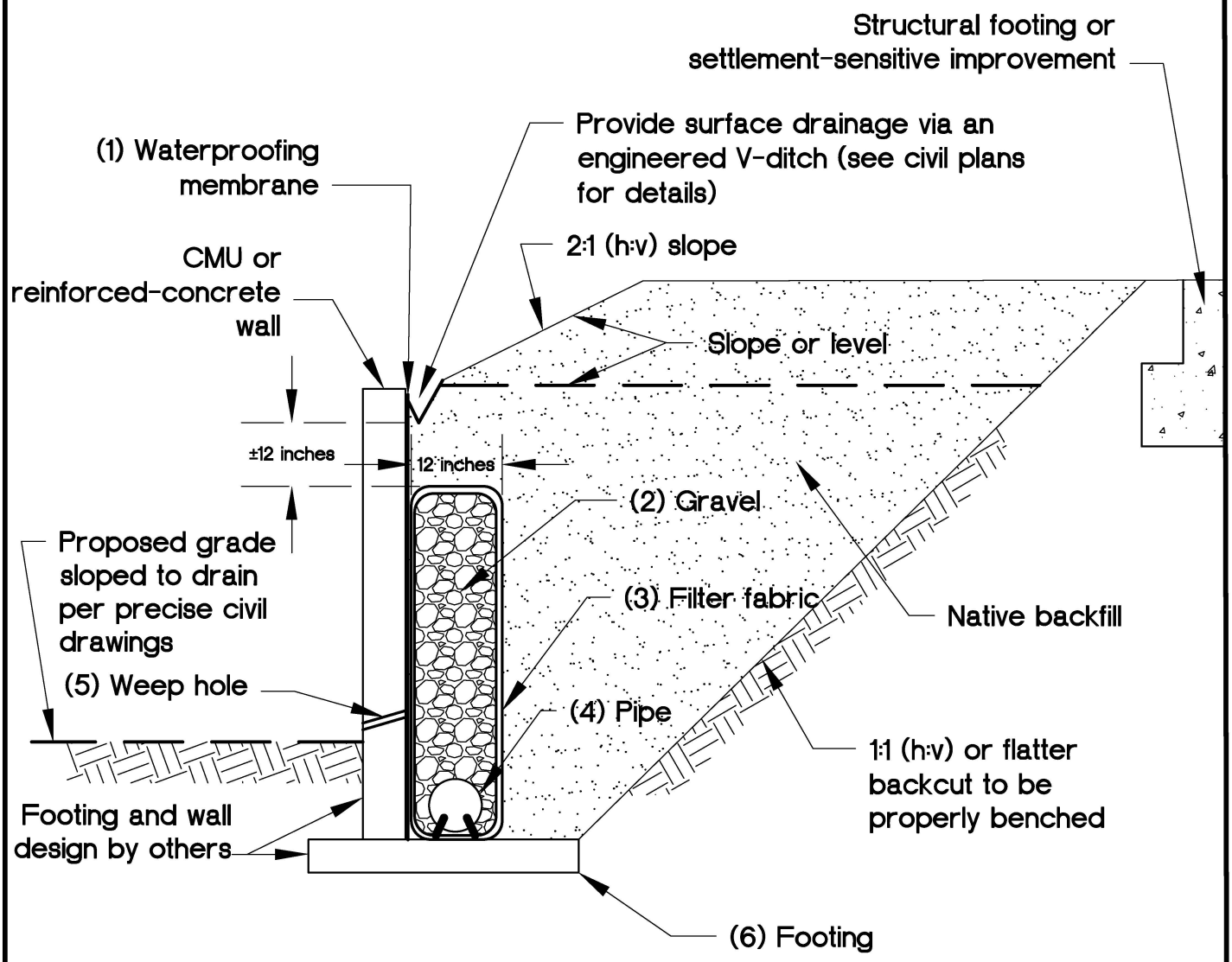
SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)
Level*	35	45
2 to 1	55	65
* Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall.		

Retaining Wall Seismic Surcharge

If any retaining walls for the planned development have the potential (during a seismic event) to fail and reduce/eliminate entry or exit to the property, a seismic surcharge should be added to the wall pressures. This seismic surcharge (seismic increment) should be added as 15H, where "H" is the height of the wall from the bottom of footing to the top of backfill (excluding shear key). This is a uniform pressure that surcharges the wall similar to a traffic load. GSI recommends this load be added to walls that are used in below grade storage areas or utility vaults. The factor-of-safety in wall overturning and stem design should maintain a 1.2 when this seismic surcharge is applied. This seismic design is for local wall stability and Section 1802.2 of the 2007 CBC, as well as CGS Special Publication 117A (CGS, 2008), should also be followed, whichever is more stringent.

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. Details 1, 2, and 3, present the back drainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-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



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, ¾ to 1½ inch.

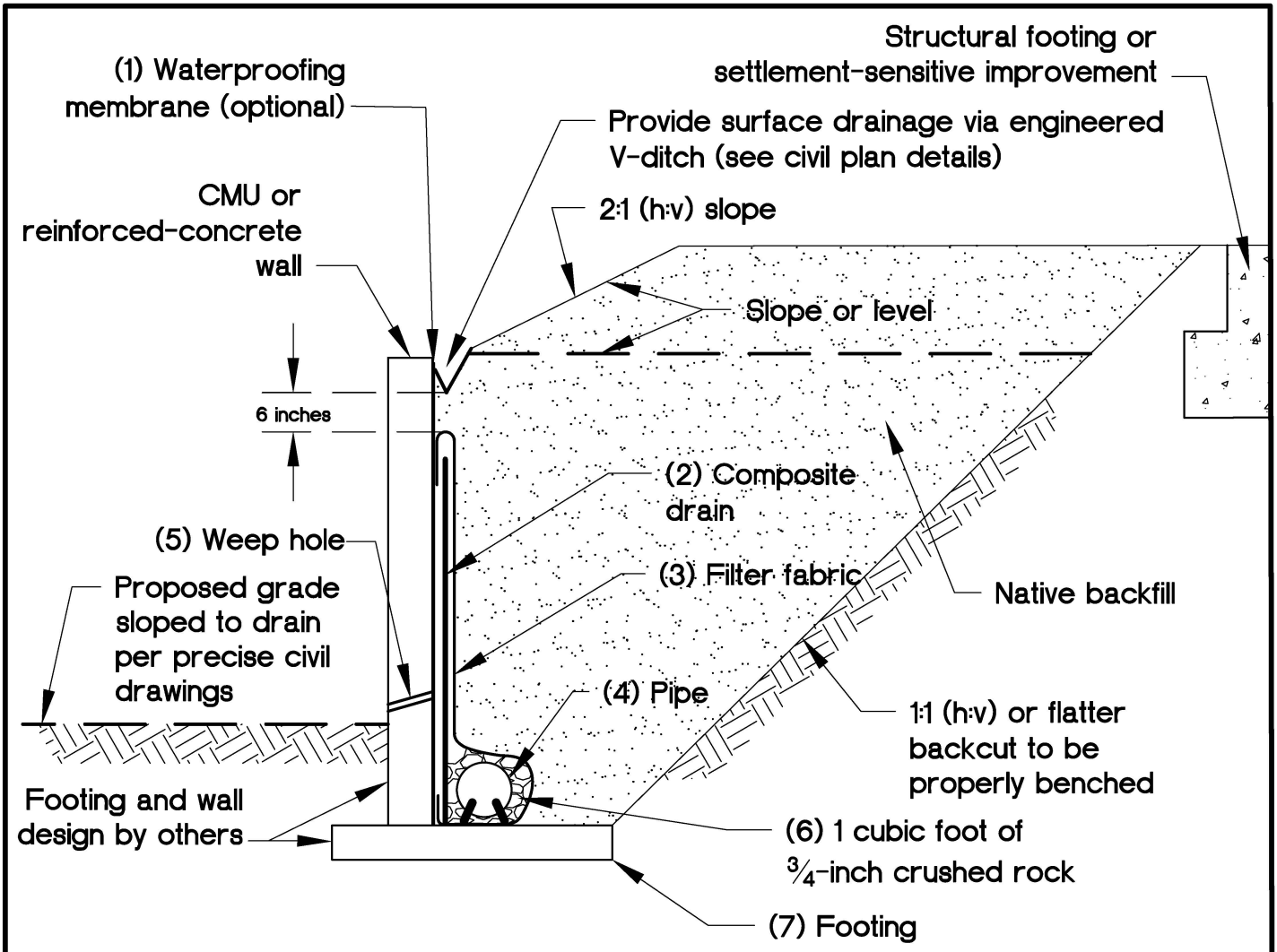
(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL – ALTERNATIVE A	Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic 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.

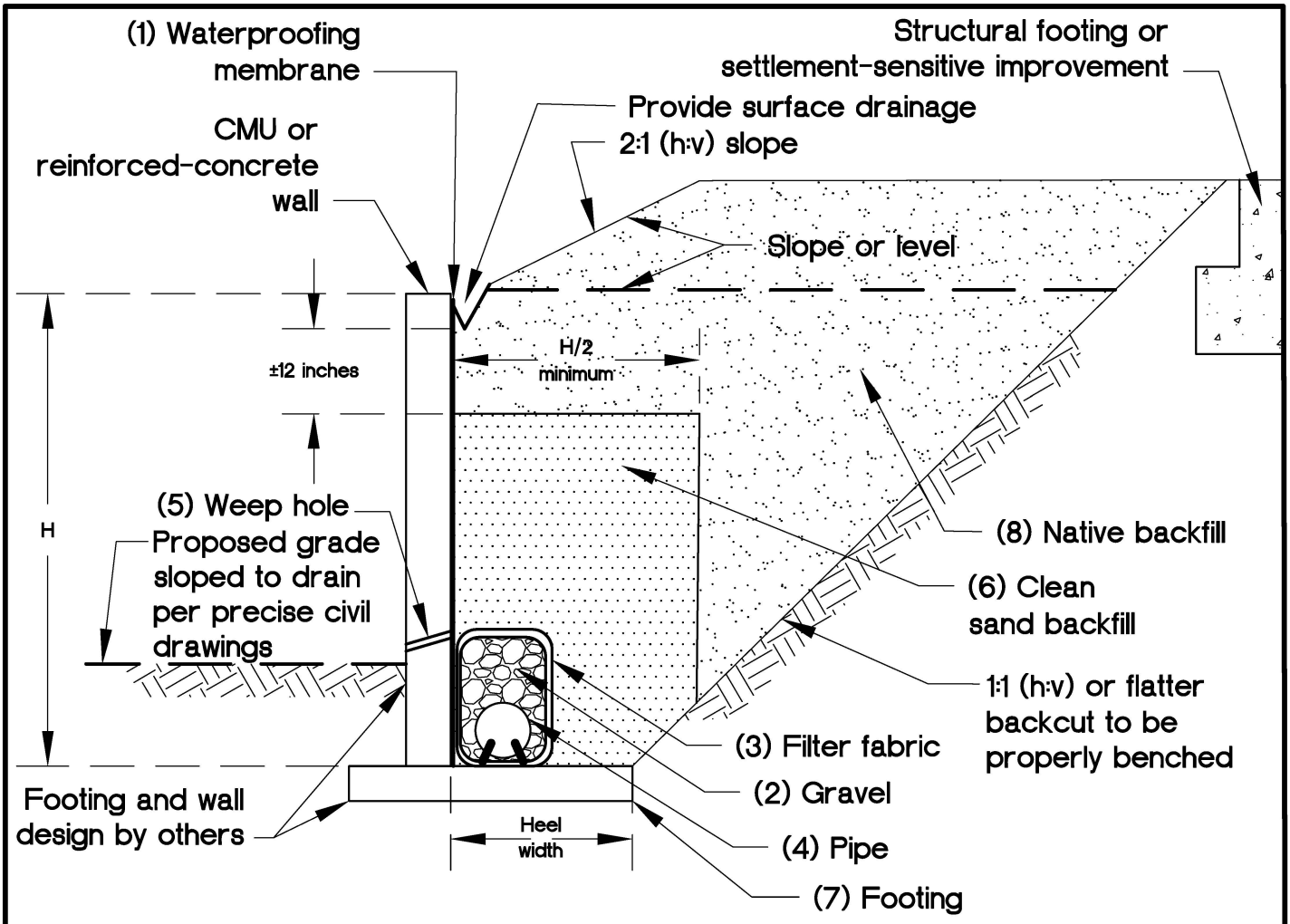
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, ¾ to 1½ inch.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL – ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. <21 and S.E. ≥ 35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL – ALTERNATIVE C	Detail 3

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 Detail Geotextile Drain). Materials with an E.I. potential of greater than 50 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, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$50). 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 placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. 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.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS

Slope Creep

Soils at the site may be expansive and therefore, may 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 interested/affected parties.

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 deepened foundations without any consideration for creep forces, where the E.I. of the materials comprising the outer 15 feet of the slope is less than 50, or a combination of grade beam and caisson foundations, for expansion indices greater than 50 comprising the slope, with creep forces taken into account. 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:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

Creep Load: 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.

Point of Fixity: Located a distance of 1.5 times the caisson's diameter, below the creep zone.

Passive Resistance: 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.

Allowable Axial Capacity:

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

Tip capacity: 4,500 psf.

PRELIMINARY PAVEMENT DESIGN

Pavement sections presented are based on an assumed R-value data (to be verified by specific R-value testing at completion of grading) from a representative sample taken from nearby projects, the anticipated design classification, and the minimum requirements of the County of Riverside. For planning purposes, pavement sections consisting of asphaltic concrete over base are provided. Anticipated asphaltic concrete (AC) pavement sections are presented on the following table.

ASPHALTIC CONCRETE PAVEMENT				
TRAFFIC AREA	TRAFFIC INDEX[1] (TI, Assumed)	SUBGRADE R-VALUE	A.C. THICKNESS (inches)	CLASS 2 AGGREGATE BASE THICKNESS[2] (inches)
Parking Lot	5.0	22	3.0 [3]	7.0
Access/Local Road	5.5	22	3.0 [3]	8.5
Collector	7.0	22	4.0 [3]	11.5

[1] TI values have been assumed for planning purposes herein and should be confirmed by the design team during future plan development.
[2] Denotes standard Caltrans Class 2 aggregate base R $\geq$78, SE $\geq$22).
[3] County of Riverside minimum (T.I. = 5 and 5.5 is 3" AC over 6" base. T.I. 7 is 4" AC over 6" base)

All pavement installations, including preparation and compaction of subgrade and placement and rolling of asphaltic concrete should be done in accordance with the County standards and under the observation and testing of the project geotechnical engineer and/or governing authority.

The recommended pavement sections provided above are meant as minimums. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the TI used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

Subgrade preparation and aggregate base preparation should be performed in accordance with the recommendations presented below, and the minimum subgrade (upper 12 inches) and Class 2 aggregate base compaction should be 95 percent of the maximum dry density (ASTM D 1557). If adverse conditions (i.e., saturated ground, etc.) are encountered during preparation of subgrade, special construction methods may need to be employed. These recommendations should be considered preliminary. Further R-value testing and pavement design analysis should be performed upon completion of grading for the site.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed.

Subgrade

Within street areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of 12 inches, moisture conditioned as necessary and compacted to 95 percent of maximum laboratory density, as determined by ASTM test method D 1557. Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed.

The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to ensure a uniformly firm and unyielding surface. All grading and fill placement should be observed by the project geotechnical consultant and/or his representative.

Base

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the maximum laboratory density as determined by ASTM test method D 1557. Base aggregate should be in accordance to the "Standard Specifications for Public Works Construction" (green book) current edition.

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base and/or subbase course.

2. Traffic is not routed over completed base before paving.

3. Construction is completed during the dry season of May through October.

4. The base is free of dirt and debris.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving and the time between completion of base and paving is reduced to three days, provided the base is free of dirt and debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, subbase course, and/or subgrade to conditions that will meet specifications as directed by the geotechnical consultant. Best management construction practices should be followed at all times. Water should not be allowed to pond behind curbs and saturate trench backfill, subgrade, or base rock; otherwise distress in streets may occur.

ALTERNATIVE PAVEMENTS

If Portland Cement Concrete Pavement (PCCP) or paver stones are requested for the private drive areas, GSI can assist the civil designer and architect in providing additional recommendations for these alternative pavement types, upon request. PCA guidelines for industrial concrete pavements in truck loading dock areas.

DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site may 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 interested/affected parties 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 percentage points above (or 120 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a 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. If very low expansive soils are present, the rock or gravel or sand may be deleted. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials. 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.

3. 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. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. $\leq$20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. 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.

4. 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.

5. Driveways, sidewalks, and patio slabs adjacent to the structure should be separated from the structure 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.

6. Planters and walls should not be tied to the structure.

7. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions. If very low expansion soils are present, footings need only be tied in one direction.

8. Any masonry landscape walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

10. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.

11. Positive site drainage should be maintained at all times. Finish grade on the lots should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the owner(s).

12. Air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety (FOS) 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 California Building Code), 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 interested/affected parties.

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 owner(s) and/or other interested/affected parties. Over-steepening of slopes should be avoided during building construction activities and landscaping.

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. Post earthquake settlements and/or static fill settlements may cause drainage reversal(s) if drainage areas are relatively flay lying. 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?

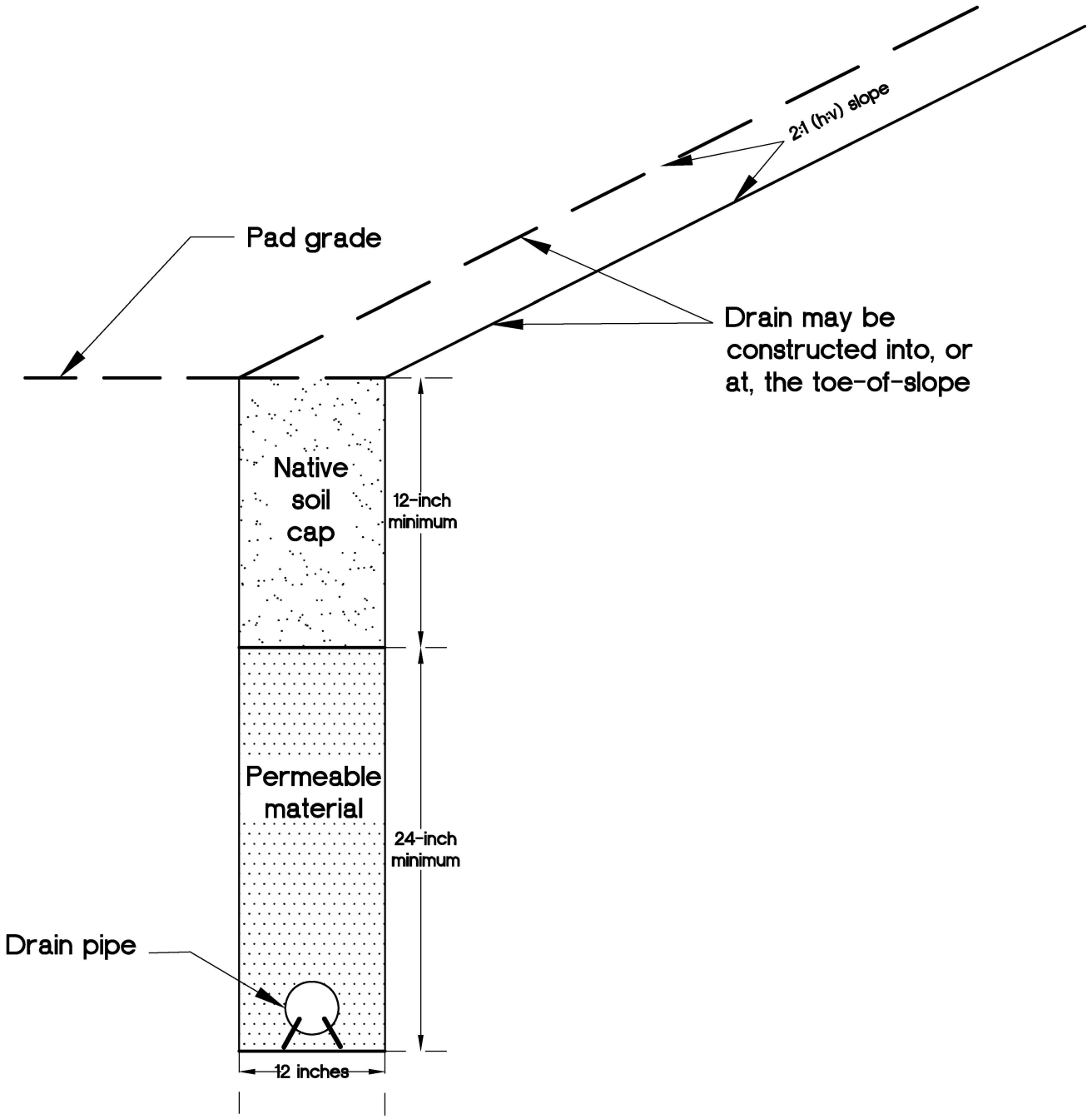
- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.

- 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.

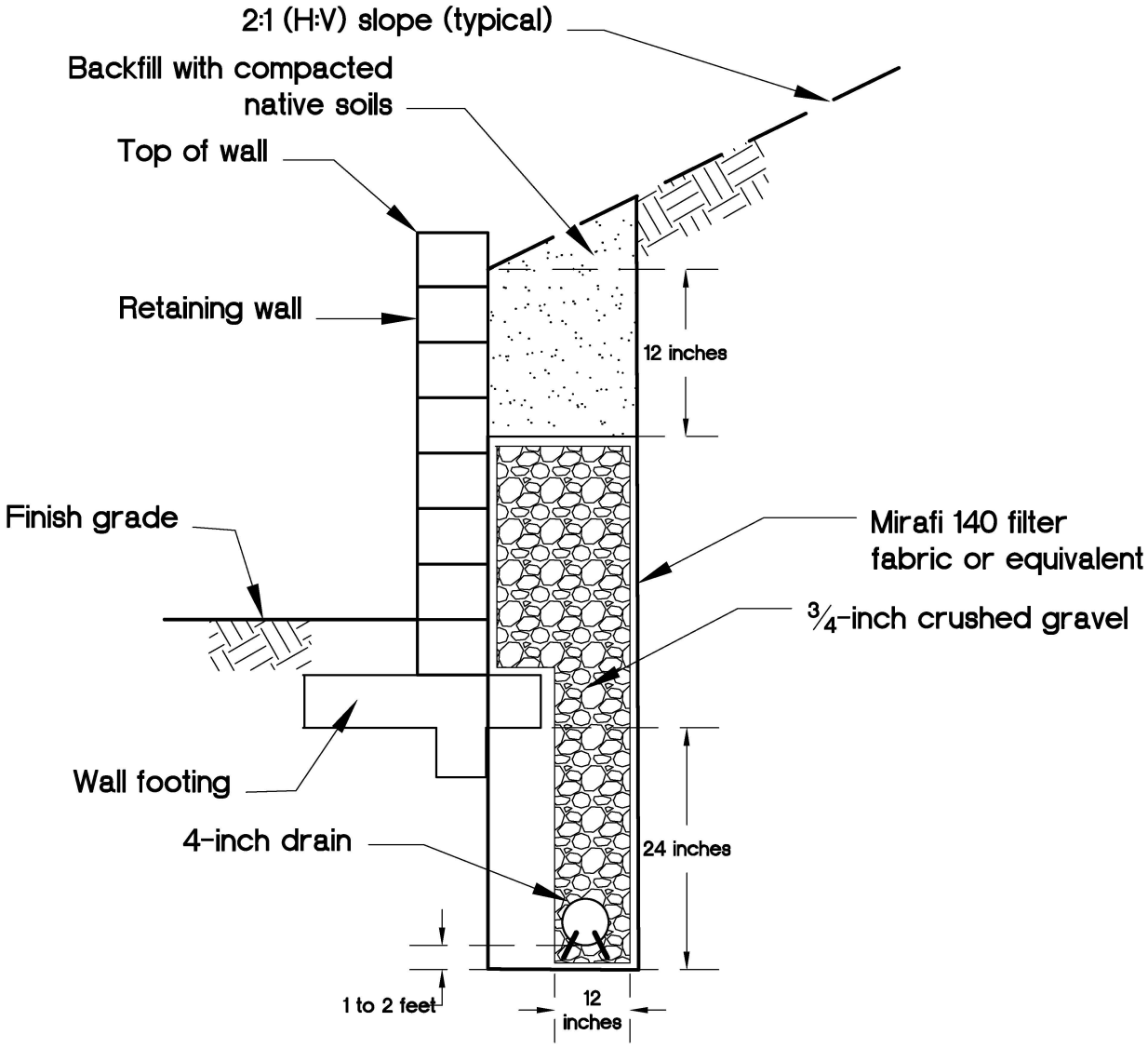
- Do the slopes "toe out" into another lot or a lot where perched or ponded water may adversely impact its proposed use?

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, owner(s) irrigation, or other circumstances. Where drains are constructed



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 cut-off 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



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 cut-off 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. Effort to compact should be applied to drain rock.

GeoSoils, Inc.	SUBDRAIN ALONG RETAINING WALL DETAIL	Detail 5

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/affected parties.

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 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 structure, 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

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are 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.

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 any interested/affected 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 owner(s), 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.

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 owner 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 owner(s) for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide owner(s) 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 not 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.

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.

Typical review by the geotechnical consultant occurs for the 40-scale grading plan, precise grading plan, and foundation drawings. Due to the dense bedrock within the zone anticipated for deep utilities, GSI should also review utilized sheet improvement plans.

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

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APPENDIX B

TEST PIT LOGS BY GSI

LOG OF TEST PIT TP-01

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1460

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to medium grained, brownish gray, slightly moist, very soft, with rootlets and few, angular gravel					
5		**BEDROCK (Mzp)** @ 1.5' Biotite and alkali feldspar mylonite, light gray to dark gray, strongly and thinly foliated, with manganese staining, weathered to 3' F @ 4': N45W/52NE, lineation: S86E, 45°					
10		Total Depth = 5 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15		**EXPLANATION** MAX = Maximum Dry Density and Optimum Moisture Content EI = Expansion Index, CR = Corrosivity, HY = Hydrometer AL = Atterberg Limits, RDS = Remolded Direct Shear CN = Consolidation, PA = Particle-Size Analysis					
20		F = Metamorphic Foliation Attitude J = Joint Attitude					

LOG OF TEST PIT TP-02

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1430

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Clayey SAND (SC), fine grained, grayish brown, slightly moist to moist, very soft, with rootlets and few, angular gravel			5.7	106.2	MAX, EI, CR, AL, RDS, PA, HY CN
5		**VERY OLD AXIAL CHANNEL DEPOSITS (Qvoa)** @ 7": Clayey SAND (SC), fine to medium grained, brown, dry to slightly moist, medium stiff, with trace gravel and pore holes			12.6	111.4	
		BEDROCK (Mzp) @ 6.5' Biotite phyllite, black, strongly foliated, soapy texture, with limonite staining					
10		Total Depth = 8 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

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W.O. 5815-A-OC

TESTPIT 5815A.GPJ LAGNNN07.GDT 2/25/09

LOG OF TEST PIT TP-03

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1425

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to medium grained, brown, moist, very soft, with abundant rootlets and angular gravel					
5		**VERY OLD AXIAL CHANNEL DEPOSITS (Qvoa)** @ 3": Sandy CLAY (CL), fine to medium grained, dark reddish brown, slightly moist, stiff to very stiff, with few, angular gravel, Munsell 5YR 3/3					
10		**BEDROCK (Mzp)** @ 1.5' Quartz and syenite (alkali feldspar) schist to phyllite, light gray and white, moderately thick foliation, very hard, weathered to 2.5', with limonite staining F @ 2.5': N55W/70NE					
15		Total Depth = 3 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
20							

LOG OF TEST PIT TP-04

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1448

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to coarse grained, dark gray, slightly moist, stiff, with rootlets and pore holes					
5		**BEDROCK (Mzp)** @ 1.25' Biotite phyllite, black, thinly foliated, with abundant limonite staining F @ 4': N76W/90 J @ 4': N21E/80NW @ 6': Biotite Schist, F: N80E/90					
10		Total Depth = 6 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

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LOG OF TEST PIT TP-05

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1548

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine grained, grayish brown, moist, soft, with few gravel to boulders up to 2'					
5		**BEDROCK (Mzp)** @ 1' Quartz and syenite (alkali feldspar) schist, light gray and white, thinly foliation, very hard, weathered to 2.5', with limonite staining F @ 2': N61W/55NE					
10		Total Depth = 2.25 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

LOG OF TEST PIT TP-06

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1520

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to medium grained, brownish gray, moist, soft, with rootlets, pore holes, and trace gravel					
5		**BEDROCK (Mzp)** @ 0.5' Mica Schist, gray and white, thinly foliated, weakly lineated, with limonite and manganese staining J @ 2': N8E/86NW, N56E/90 F @ 2.5': N71W/56NE					
10		Total Depth = 3.5 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

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TESTPIT 5815A.GPJ LAGNNN07.GDT 2/25/09

LOG OF TEST PIT TP-07

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1462

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
0		**TOPSOIL** @ Surface: Sandy CLAY (CL), fine grained, brown, slightly moist to moist, soft, with rootlets, pore holes, and trace gravel **BEDROCK (Mzp)** @ 1.5' Quartz and syenite (alkali feldspar) schist, gray, moderately thick foliation, very hard, with abundant limonite and manganese staining F: N55W/35NE Total Depth = 2 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
5							
10							
15							
20							

LOG OF TEST PIT TP-08

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1444

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
0		**TOPSOIL** @ Surface: Clayey SAND with Gravel (SC), fine to medium grained, grayish brown, slightly moist, dense, with pore holes @ 0.5': dry to slightly moist, light gray @ 2': increasingly rocky **BEDROCK (Mzp)** @ 3' Biotite phyllite, black, thinly foliated, heavily weathered to 6', with limonite staining F @ 5': N24W/82NE, wavy J @ 6': N15W/10SW, N25W/85NE, manganese and sulfur lined, undulatory Total Depth = 7 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					MAX, EI, CR, AL, PA, HY
5							
10							
15							
20							

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Phone: (714)632-0151 Fax: (714)632-0745

Hanna Property
Keller and Winchester Roads, Riverside Co.
W.O. 5815-A-OC

TESTPIT 5815A.GPJ LAGNNN07.GDT 2/25/09

LOG OF TEST PIT TP-09

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1438

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to coarse grained, brown, slightly moist, soft, with pore holes, rootlets, and trace gravel					
5		**BEDROCK (Mzp)** @ 2.75' Biotite phyllite, black, thinly foliated, with limonite and manganese staining, weathered to Clay in places AND Quartz syenite (alkali feldspar) schist, light gray and white, moderately thick foliation, very hard Contact between trending similar to foliation F @ 3': N85E/90, N60E/88SE J @ 3': N42W/82SW, N5W/90					
10		Total Depth = 4.5 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

LOG OF TEST PIT TP-10

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1426

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Clayey SAND (SC), fine to medium grained, brownish gray, dry to slightly moist, loose			6.2	98.0	
5		**VERY OLD AXIAL CHANNEL DEPOSITS (Qvoa)** @ 1.5": Sandy CLAY (CL), fine to medium grained, brown, slightly moist to moist, medium stiff to stiff, with pore holes and trace, angular gravel			17.8	106.9	EI, AL CN
		@ 3.5': with increasing Clay			13.1	103.0	
10		@ 4': CLAY (CL), reddish brown, moist, stiff, with trace, coarse graines Sand and gravel @ 5': CLAY with Sand (CL), fine grained, dark gray, slightly moist, stiff, with limonite staining and limonite-filled root holes and trace gravel, Munsell 2.5YR 4/1					
15		**BEDROCK (Mzp)** @ 7.5' Biotite phyllite, black, thinly foliated, with limonite, manganese, and trace sulfur					
20		Total Depth = 9 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					

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W.O. 5815-A-OC

TESTPIT 5815A.GPJ LAGNNN07.GDT 2/25/09

LOG OF TEST PIT TP-11

Date Excavated: 12/4/08 Logged by: SHW

Equipment: Backhoe Surface Elevation(ft): +/- 1502

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
		TOPSOIL @ Surface: Sandy CLAY (CL), fine to medium grained, brownish gray, slightly moist, soft, with rootlets and pore holes					
5		**BEDROCK (Mzp)** @ 0.5' Biotite phyllite, black, thinly foliated, joints are limonite and sulfur filled F @ 3': N66W/72NE J @ 3': N37E/81SE, N13E/87SE					
10		Total Depth = 4 Feet, Refusal on Hard Bedrock No Groundwater Encountered, No Caving Hole Backfilled with Cuttings					
15							
20							

TESTPIT 5815A.GPJ LAGNNN07.GDT 2/25/09

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Hanna Property
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W.O. 5815-A-OC

APPENDIX B2

TRENCH LOGS BY EUC

APPENDIX A

Trench Logs

TRENCH LOG No. 1

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	15-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1436	**Total Depth:**	5.5	**Subcontractor:**	Al-Roy Drilling
Orientation:	<---N40W	**Length:**	12	**Equip. type:**	JD330

SUBSURFACE CONDITIONS

Surficial Soil

0-2.5 Silty Clay, dark brown, dry, hard.
At 12" becomes moist, stiff

Bedrock (Mzp)

2.5-5.5 Phyllite, dark gray, damp, fissile.
Excavates with effort.

REMARKS **Logged By:** jd

Foliation: N40W,42NE

GRAPHIC LOG **SCALE:** 1"=5'

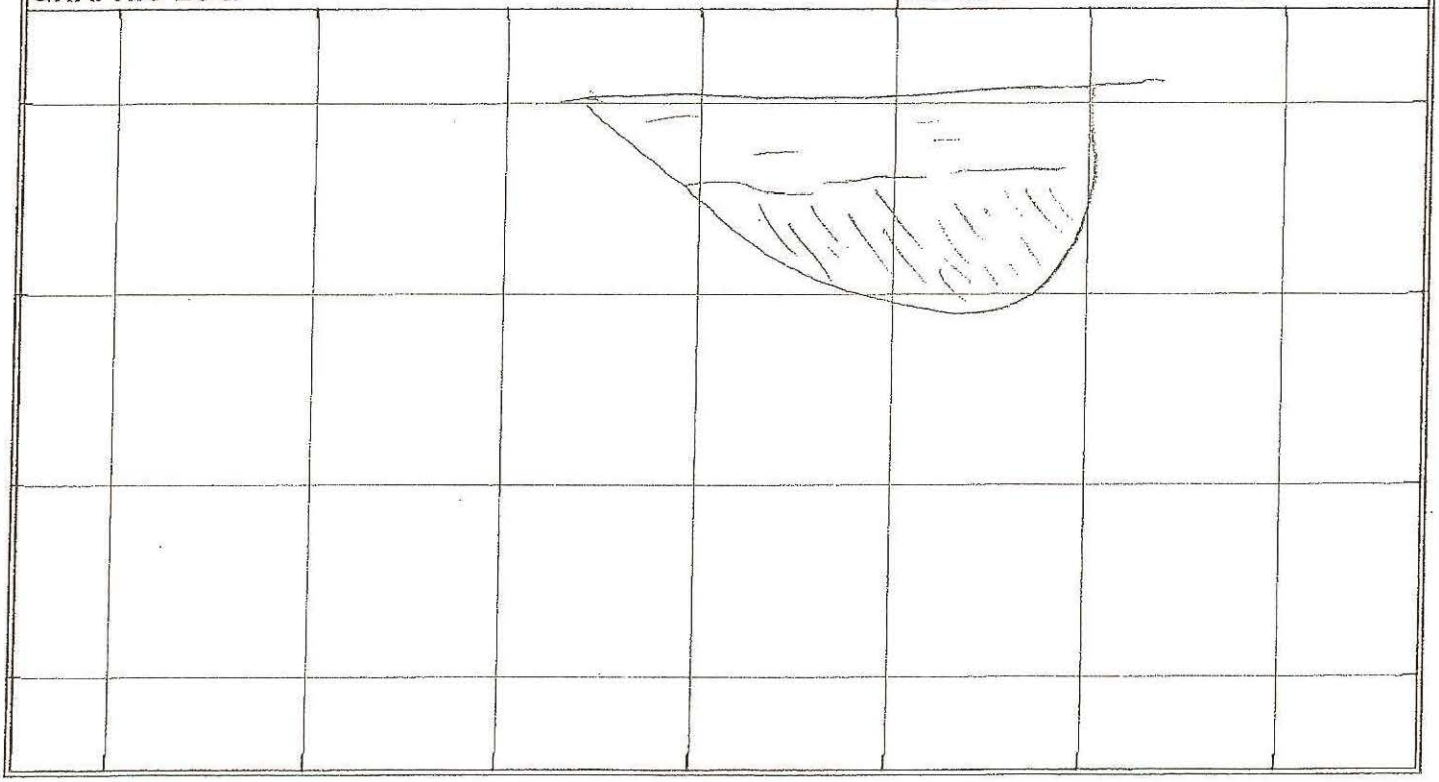


Plate: A - 1

TRENCH LOG No. 2

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1432	**Total Depth:** 5	**Subcontractor:** Al-Roy Drilling	
Orientation: <--N30E	**Length:** 12	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Surficial Soil

0-3 Silty Clay, dark gray, moist, firm, porous.
Upper 12" dry and loose.
Grades to bedrock.

Bedrock (Mzp)

3-5 Phyllite, dark gray, damp to moist, fissile.
Excavates with ease 3-4
Excavates with effort 4-5.

REMARKS

Logged By: jd

Bag Sample
1-2
4-5

Drive Sample	Blows
3	12

Foliation N15E,60E
Joint E-W, 58N

GRAPHIC LOG

SCALE: 1"=5'

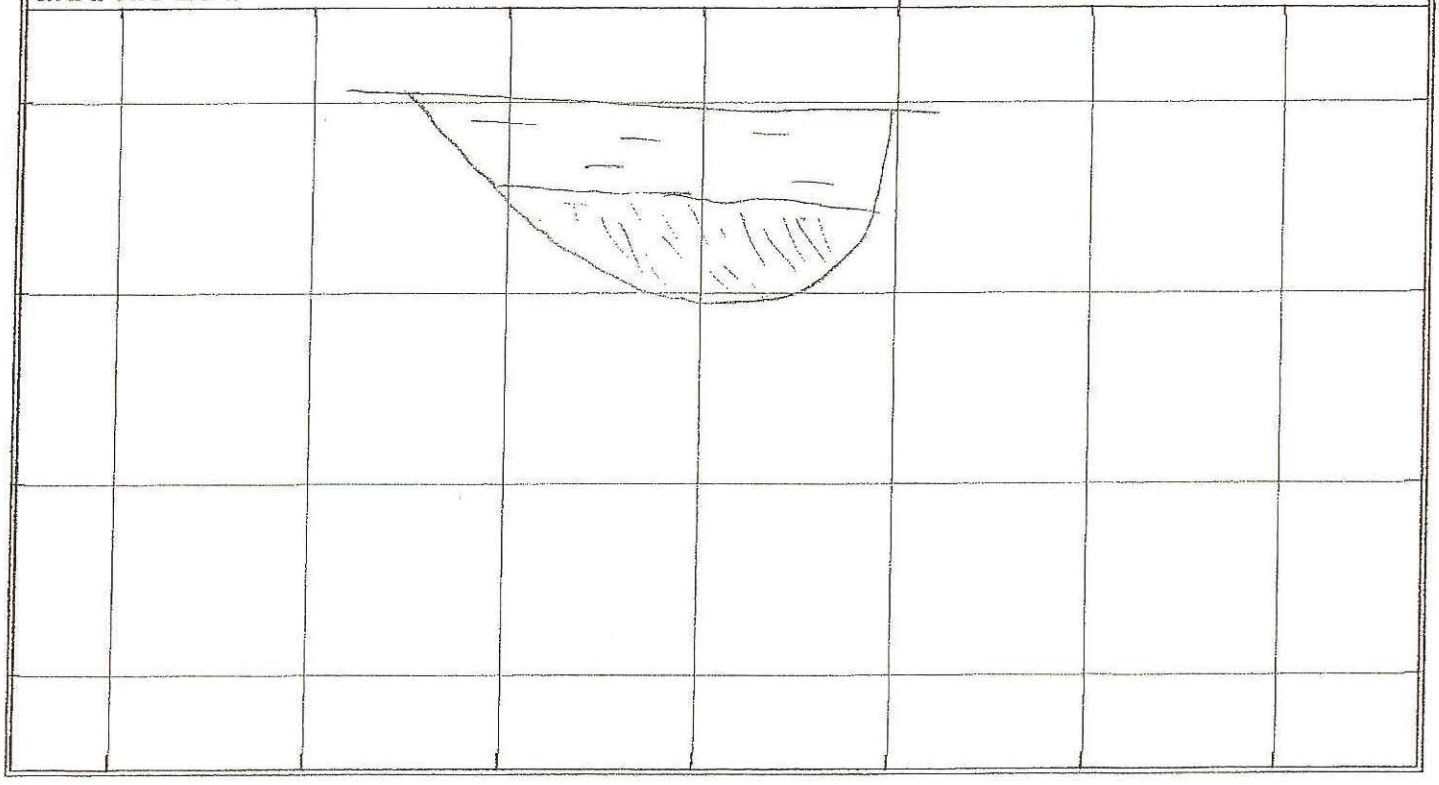


Plate: A - 2

TRENCH LOG No. 3

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	15-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1435	**Total Depth:**	5	**Subcontractor:**	Al-Roy Drilling
Orientation:	W--->	**Length:**	11	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	**REMARKS**	**Logged By:** jd
Very old Channel Deposits (Qvoa) 0-4 Clayey silt, dark gray, moist, firm, porous. Upper 12" dry and loose. Bedrock (Mzp) 4-5 Weathered Quartzite, orange-brown, moist, soft rock, Massive. Excavates with ease	Bag Sample 1-2 4-5 Drive Sample Blows 1' 12	

GRAPHIC LOG — SCALE: 1"=5'

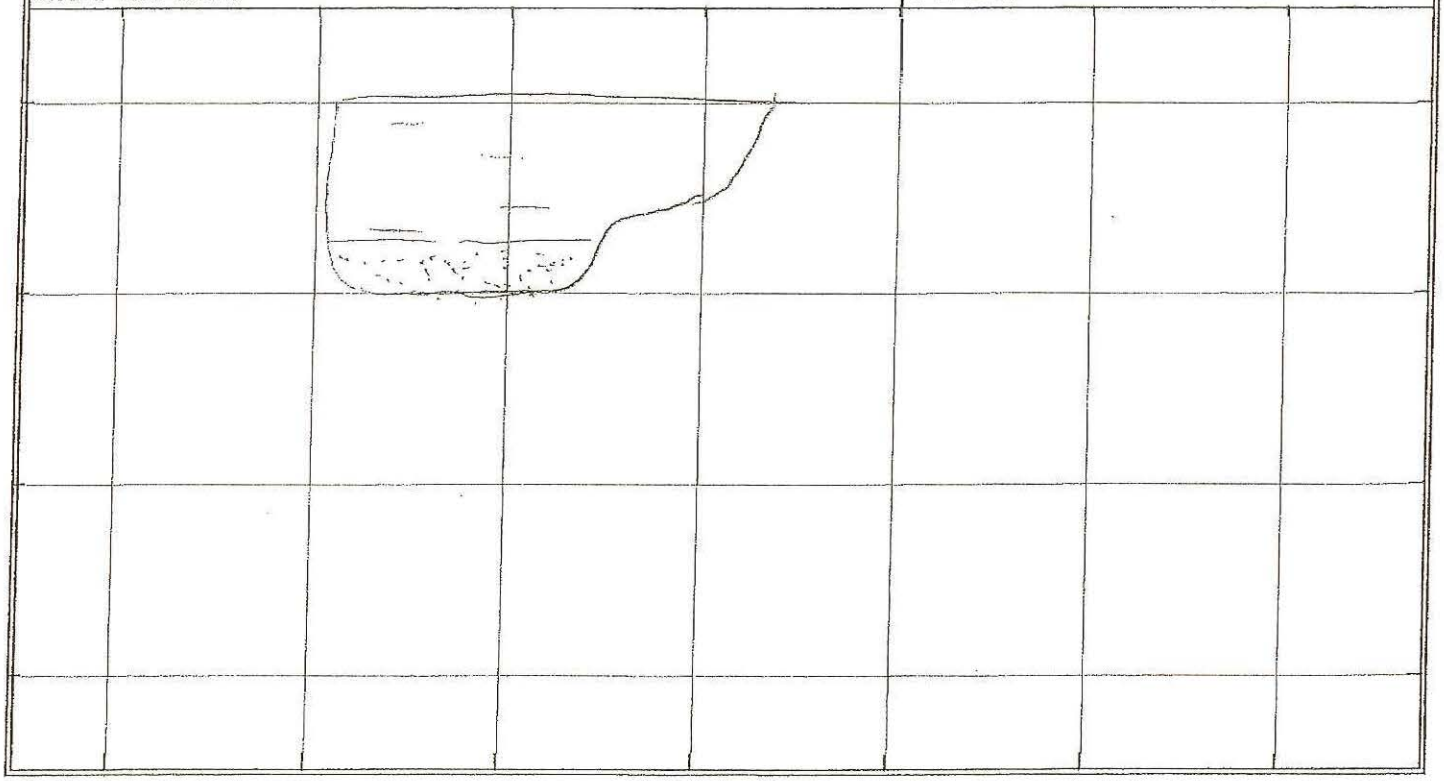


Plate: A - 3

TRENCH LOG No. 4

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1430	**Total Depth:** 6	**Subcontractor:** Al-Roy Drilling	
Orientation: N20W--->	**Length:** 15	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-6 Clayey silt, dark gray, moist, firm, porous. Upper 12" dry and loose.

Bedrock (Mzp)

@6' Gray aphanitic mafic rock, very hard. Refusal at 6 feet.

REMARKS **Logged By:** jd

Bag Sample

2-3

Drive Sample	Blows
2'	10

GRAPHIC LOG **SCALE:** 1"=5'

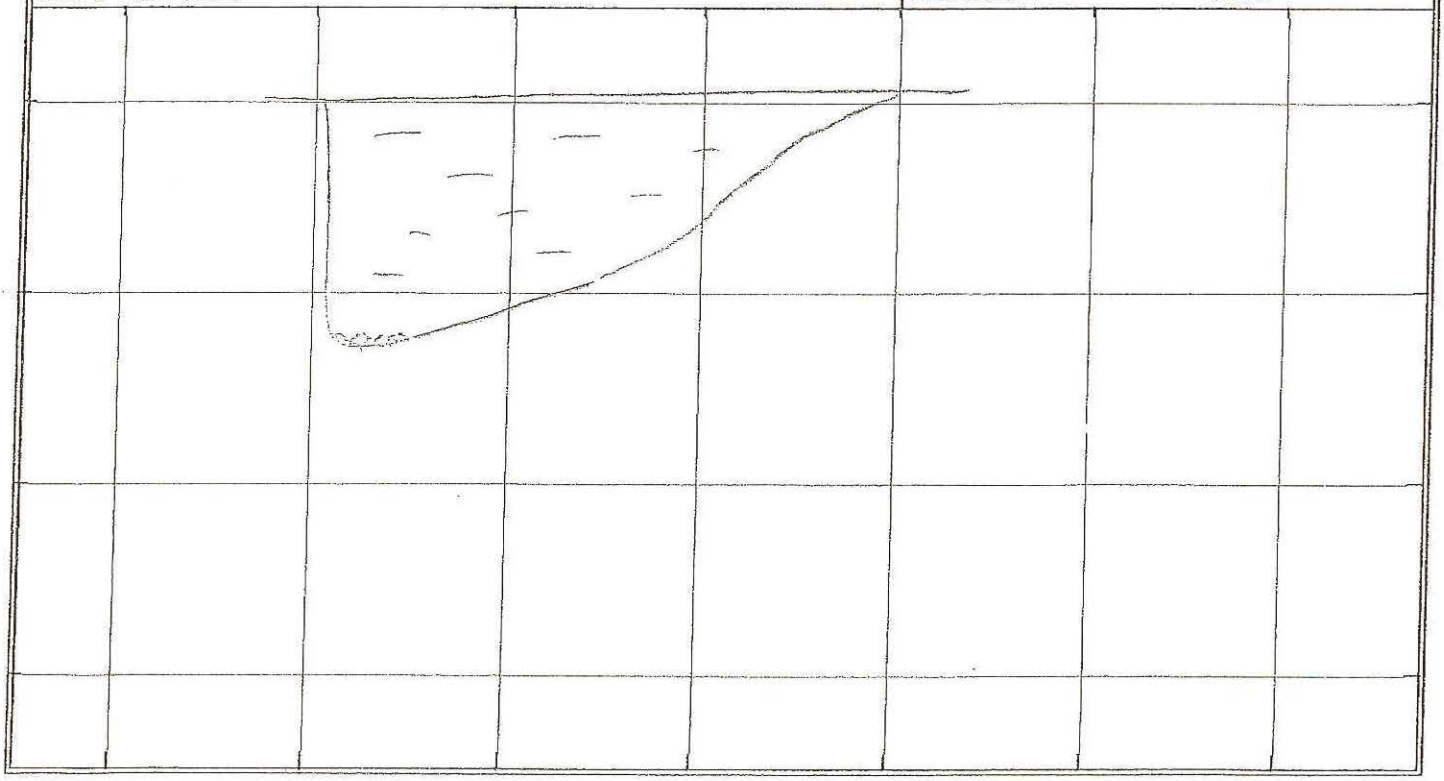


Plate: A - 4

TRENCH LOG No. 5

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1440	**Total Depth:** 6	**Subcontractor:** Al-Roy Drilling	
Orientation: N-->	**Length:** 12	**Equip. type:** JD330	

SUBSURFACE CONDITIONS	**REMARKS** **Logged By:** jd
Surficial Soil 0-3 Silty Clay, dark brownish gray, moist, firm, porous. Upper 12" dry and loose. Bedrock (Mzp) 3-6 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort 3-4 Excavates with difficulty 4-6.	Foliation N80E,79N Joint N65E,90

GRAPHIC LOG — SCALE: 1"=5'

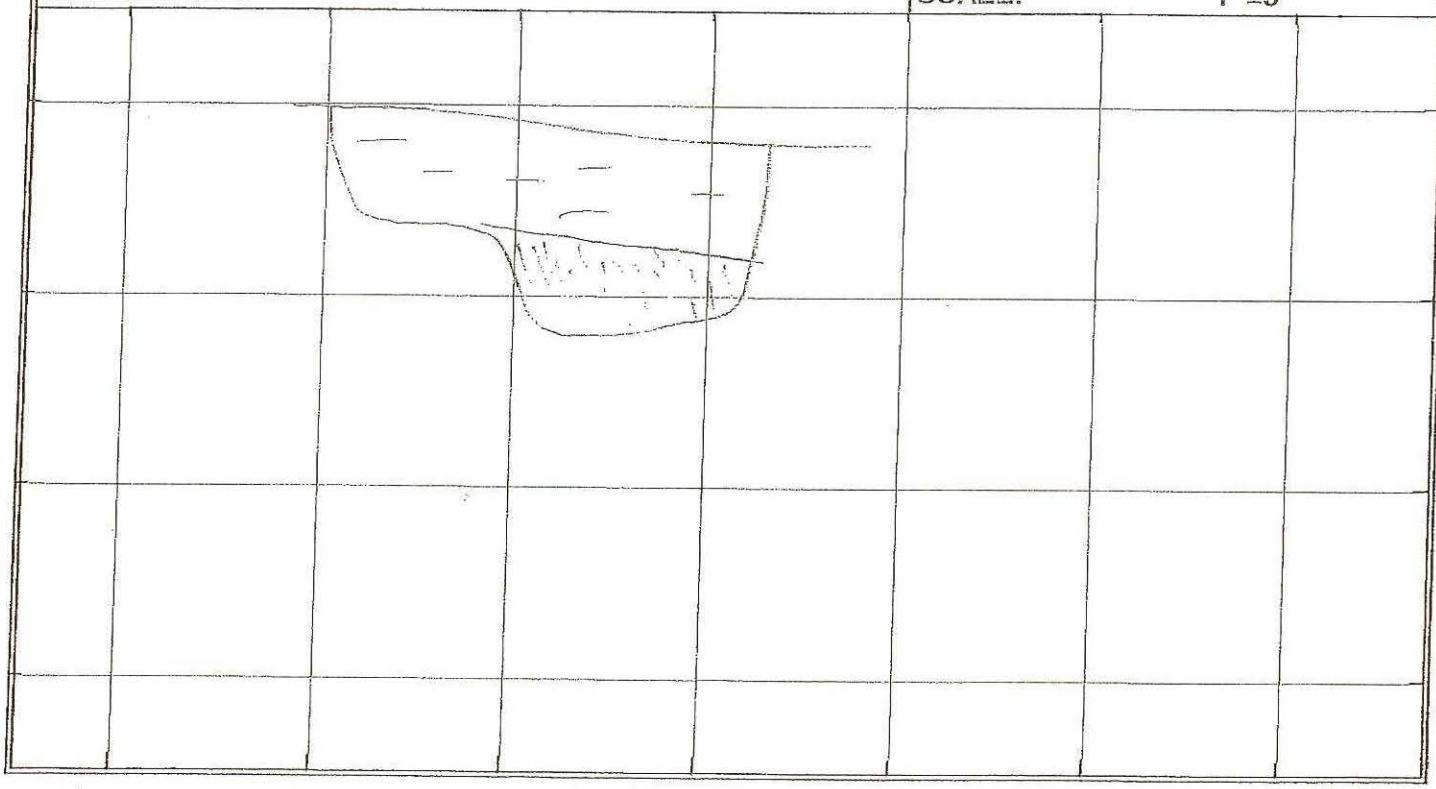


Plate: A - 5

TRENCH LOG No. 6

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1465	**Total Depth:** 4	**Subcontractor:** Al-Roy Drilling	
Orientation: N30W--->	**Length:** 12	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Surficial Soil

0-1 Clayey Silt, gray, dry to damp, loose.

Bedrock (Mzp)

1-4 Phyllite, dark gray, damp, fissile, soft rock.
Excavates with difficulty 3-4
Refusal at 4

REMARKS **Logged By:** jd

Foliation N80W,85S

GRAPHIC LOG **SCALE:** 1"=5'

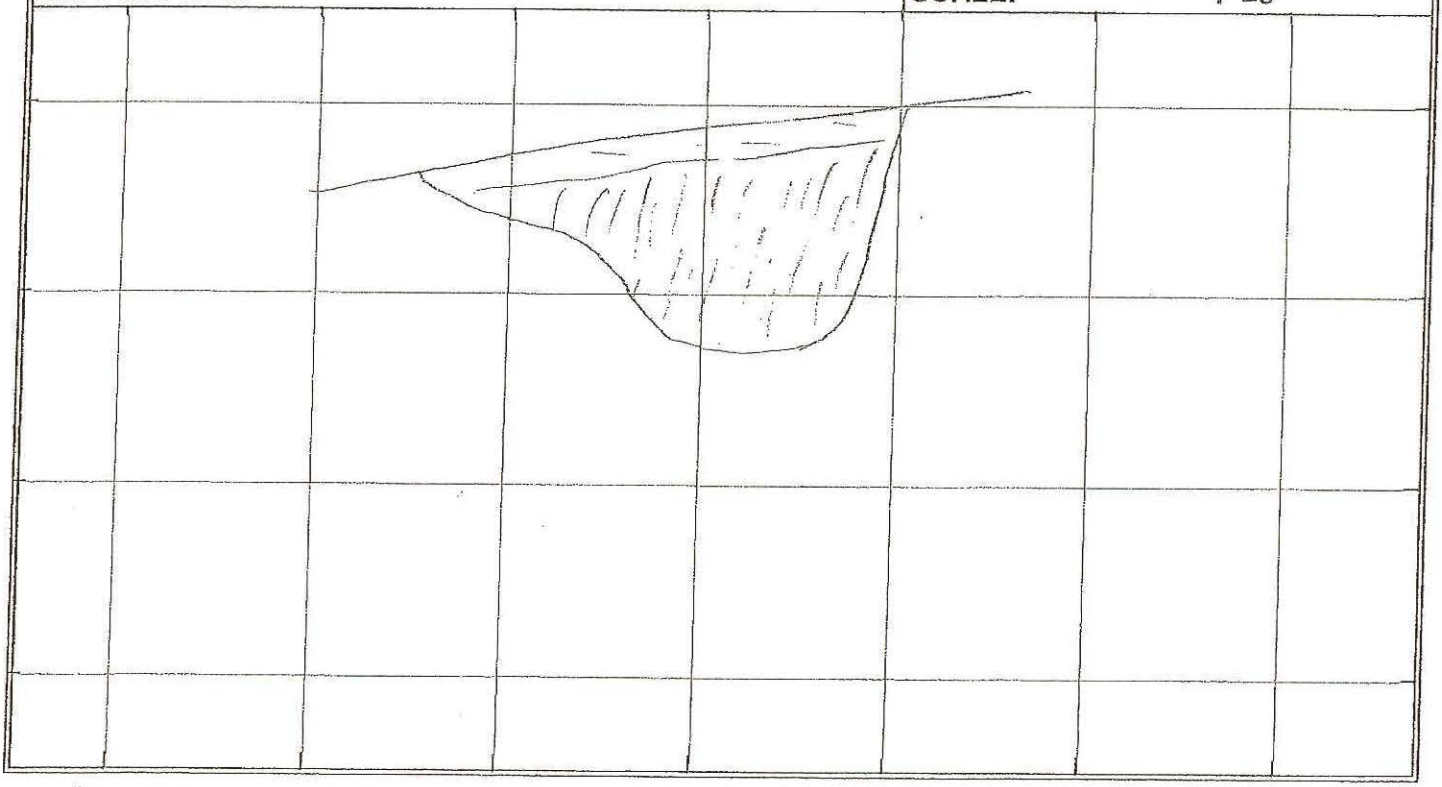


Plate: A - 6

TRENCH LOG No. 7

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1461	**Total Depth:** 7	**Subcontractor:** Al-Roy Drilling	
Orientation: N---->	**Length:** 13	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-4 Silt, dark gray, moist, firm, porous, with phyllite and occasional quartzite rock fragments to 6 inch diameter. Upper 12" dry and loose.

Bedrock (Mzp)

4-7 Phyllite, dark gray, damp, fissile, soft rock. Orange stain on joints. Excavates with ease

REMARKS **Logged By:** jd

Bag Sample
1-3

Drive Sample	Blows	
3	20	(nr)
7	50	

Foliation N70W,77SW
Joint N17E,84E

GRAPHIC LOG **SCALE:** 1"=5'

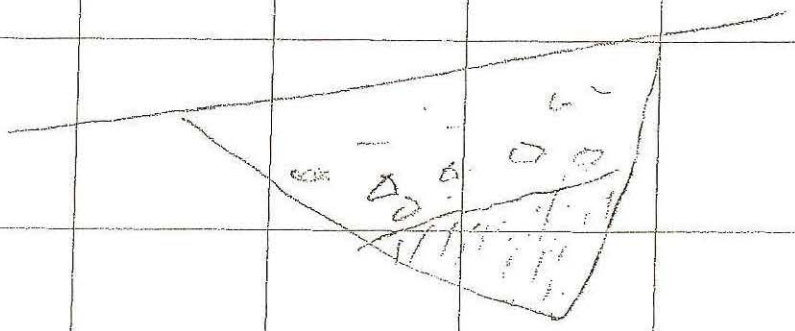


Plate: A - 7

TRENCH LOG No. 8

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1512	**Total Depth:** 5.5	**Subcontractor:** Al-Roy Drilling	
Orientation: <---W	**Length:** 14	**Equip. type:** JD330	

SUBSURFACE CONDITIONS	**REMARKS** Logged By: jd
Very old Channel Deposits (Qvoa) 0-2 Silt, with trace of clay, brown, damp, firm, porous 2-5 Silt with sub angular chunks of phyllite and quartzite difficult excavation Bedrock (Mzp) 5-5.5 Phyllite, dark gray, damp, fissile, soft rock. Orange stain on joints. Excavates with difficulty	Bag Sample 1-2 Foliation NN88W,56N Joint N40E,90

GRAPHIC LOG — SCALE: 1"=5'

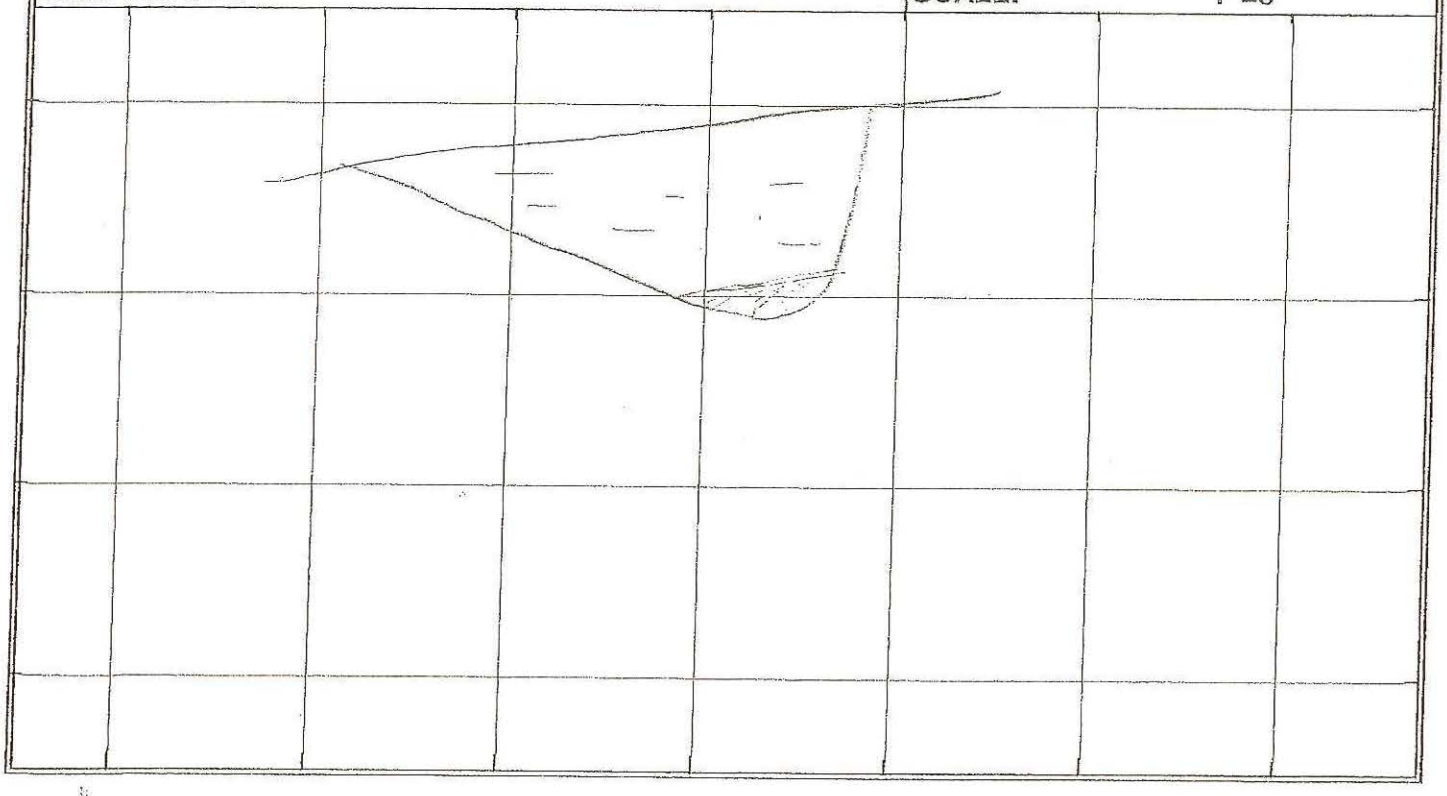


Plate: A - 8

TRENCH LOG No. 9

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1507	**Total Depth:** 7	**Subcontractor:** Al-Roy Drilling	
Orientation: <---S50E	**Length:** 14	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-4 Silt, with trace of clay, brown, damp, firm, porous
Upper 12" dry and loose.

4-5 Silt with sub angular chunks of phyllite and quartzite
moist, dense, difficult excavation

Bedrock (Mzp)

5-5.5 Phyllite, dark gray, damp, fissile, soft rock.
Orange stain on joints.
Excavates with difficulty

REMARKS **Logged By:** jd

Drive Sample	Blows
2	6

Foliation N10W, 42NE
Joint N-S,35E

GRAPHIC LOG **SCALE:** 1"=5'

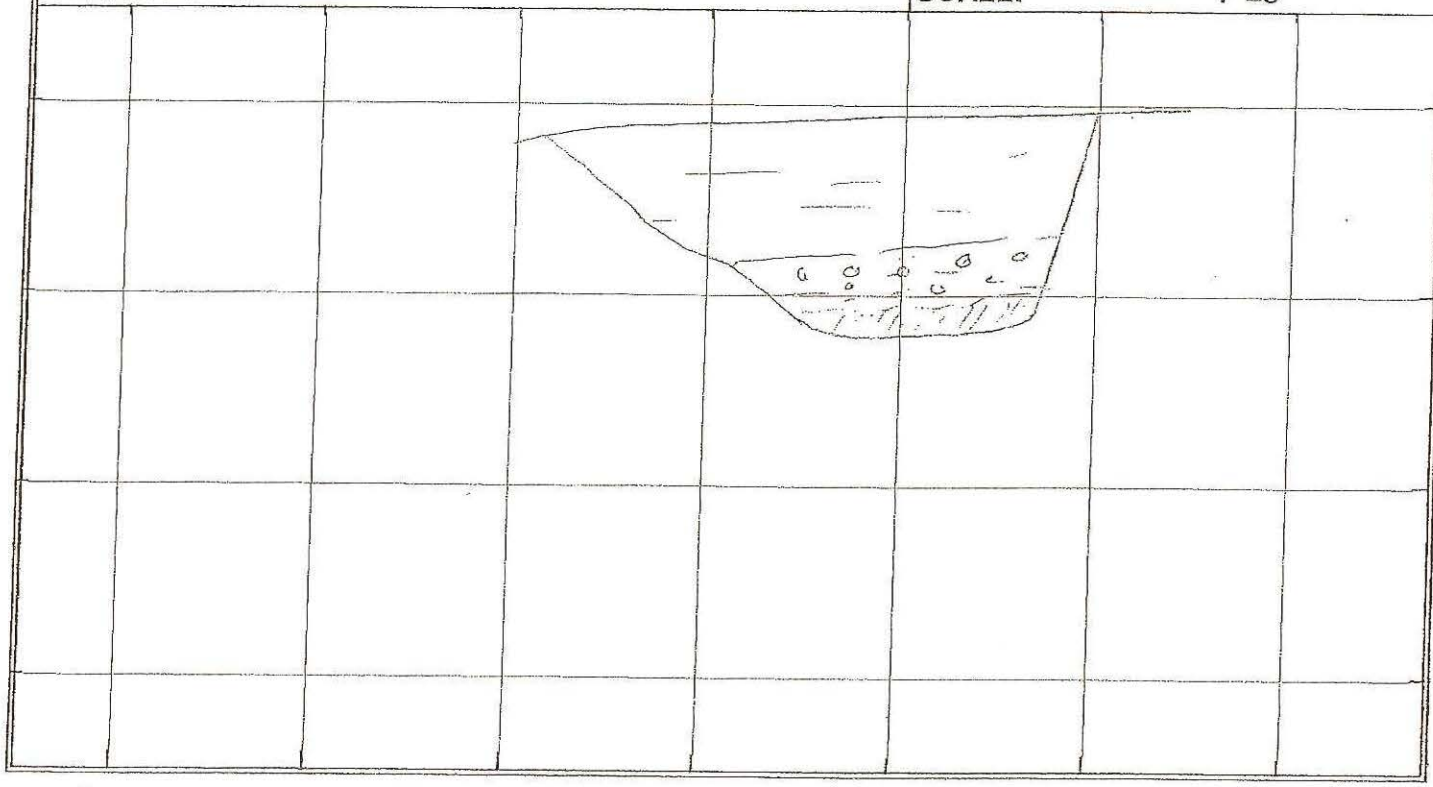


Plate: A - 9

TRENCH LOG No. 10

Client:	**Brookfield Land**		**W.O.** 81-02081-0011
Project:	Hanna Property		**Date:** 15-Aug-05
Location:	Winchester		**Sheet:** 1 of 1
Est. Surface Elev: 1419	**Total Depth:** 4	**Subcontractor:**	Al-Roy Drilling
Orientation: N30E-->	**Length:** 10	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-3 Silt, dark gray, damp, porous. Upper 12 inches dry, loose. Bedrock (Mzp) 3-4 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N54W,56NE	

GRAPHIC LOG SCALE: 1"=5'

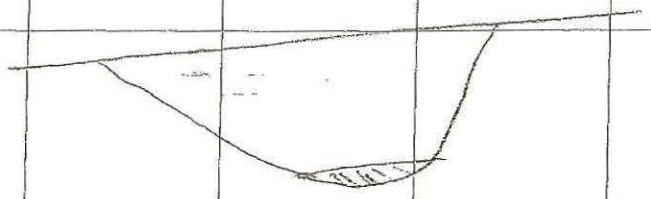


Plate: A - 10

TRENCH LOG No. 11

Client:	Brookfield Land			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	15-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1511	**Total Depth:**	4	**Subcontractor:**	Al-Roy Drilling
Orientation:	N20E--->	**Length:**	10	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-2 Silt, dark gray, damp, porous. Upper 12 inches dry, loose. Bedrock (Mzp) 2-4 Phyllite, dark gray, damp, fissile, soft rock. Orange stain on joints. Excavates with effort.	Foliation N74E,67NW Joint N17E,84E 5W,60W	

GRAPHIC LOG — SCALE: 1"=5'

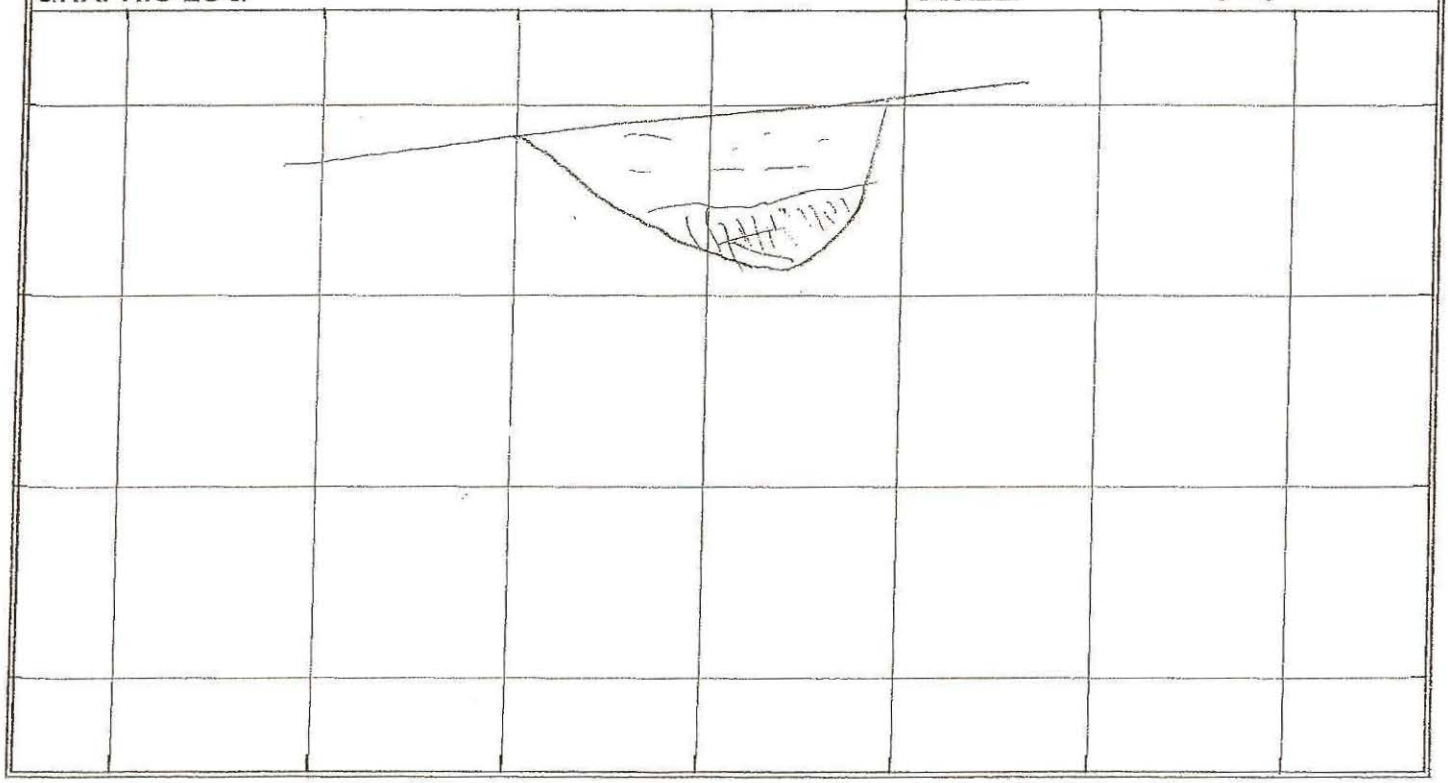


Plate: A - 11

TRENCH LOG No. 12

Client:	**Brookfield Land**		**W.O.** 81-02081-0011
Project:	Hanna Property		**Date:** 15-Aug-05
Location:	Winchester		**Sheet:** 1 of 1
Est. Surface Elev: 1536	**Total Depth:** 4	**Subcontractor:**	Al-Roy Drilling
Orientation: N30E--->	**Length:** 11	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-1 Silt, trace of clay, gray, dry, loose. Bedrock (Mzp) 1-4 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort to 3'. Excavates with difficulty 3-4.	Foliation N70W,54NE	

GRAPHIC LOG SCALE: 1"=5'

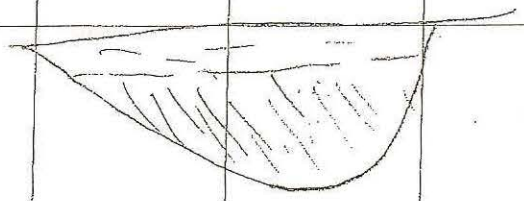


Plate: A - 12

TRENCH LOG No. 13

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1465	**Total Depth:** 5	**Subcontractor:** Al-Roy Drilling	
Orientation: N20E--->	**Length:** 10	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Surficial Soil

0-3.5 Sandy silt, gray, dry, loose. Becomes moist, medium dense at 12"

@3 Cobble layer.

Bedrock (Mzp)

3.5-5 Phyllite, dark gray, damp, fissile, soft rock. Excavates with difficulty.

REMARKS **Logged By:** jd

Bag Sample

1-3

Drive Sample Blows

3 25

Foliation N67W,85N

Joint N27E,47NW

GRAPHIC LOG **SCALE:** 1"=5'

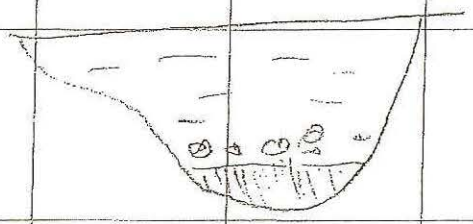


Plate: A - 13

TRENCH LOG No. 14

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 15-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1449	**Total Depth:** 8.5	**Subcontractor:** Al-Roy Drilling	
Orientation: N35W--->	**Length:** 13	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-8 Silt with fine sand, brown, dry, loose.
Becomes damp at 12 inches. Porous.

@3 Cobble layer

@7 Cobble layer

Bedrock (Mzp)

8-8.5 Phyllite, dark gray, damp, fissile, soft rock.
Thin quartzite interbeds .
Excavates with ease

REMARKS **Logged By:** jd

Bag Sample
1-3

Drive Sample	Blows
3	18
5	33

GRAPHIC LOG **SCALE:** 1"=5'

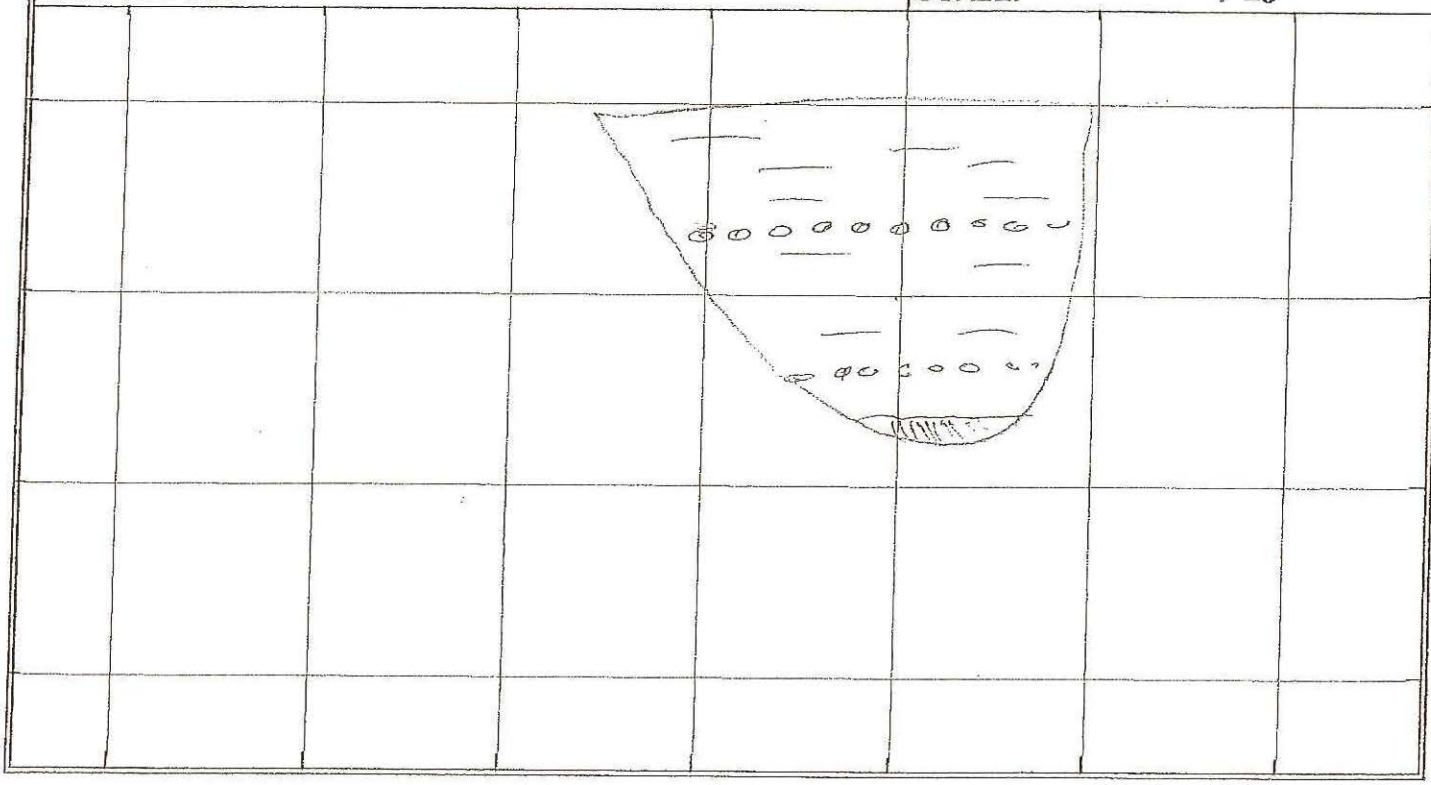


Plate: A - 14

TRENCH LOG No.15

Client:	**Brookfield Land**		**W.O.** 81-02081-0011
Project:	Hanna Property		**Date:** 16-Aug-05
Location:	Winchester		**Sheet:** 1 of 1
Est. Surface Elev: 1458	**Total Depth:** 7	**Subcontractor:**	Al-Roy Drilling
Orientation: N25W--->	**Length:** 13	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-3 Silt with cobble size sub angular rock fragments, gray-brown, dry, loose. Bedrock (Mzp) 3-4 Phyllite, dark gray, damp, fissile. Excavates with effort.	Foliation N67W,63NE	

GRAPHIC LOG — SCALE: 1"=5'

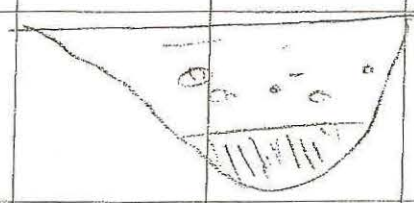


Plate: A - 15

TRENCH LOG No. 16

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 16-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1485	**Total Depth:** 2	**Subcontractor:** Al-Roy Drilling	
Orientation: N--->	**Length:** 5	**Equip. type:** JD330	

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-0.5 Silty sand, gray, dry, loose Bedrock (Mzp) 0.5-2 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N15W,33NE Joint N-S,62E	

GRAPHIC LOG — SCALE: 1"=5'



Plate: A - 16

TRENCH LOG No. 17

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1445	**Total Depth:**	4	**Subcontractor:**	Al-Roy Drilling
Orientation:	N20W--->	**Length:**	9	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	**REMARKS**	**Logged By:** jd
Surficial Soil 0-0.5 Silty sand, gray, dry, loose Bedrock (Mzp) 0.5-2 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N58W,72NE	

GRAPHIC LOG **SCALE:** 1"=5'

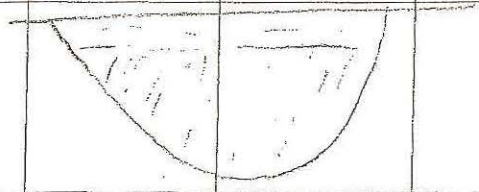


Plate: A - 17

TRENCH LOG No. 18

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1433	**Total Depth:**	7	**Subcontractor:**	Al-Roy Drilling
Orientation:	N45W--->	**Length:**	10	**Equip. type:**	JD330

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-6 Sandy Silt, dark brown, dry, loose
At 1' becomes moist, soft, porous
Grades to bedrock.

Bedrock (Mzp)

6-7 Phyllite, dark gray, damp, fissile, soft rock.
Excavates with ease
Excavates with effort at 7.

REMARKS **Logged By:** jd

Drive Sample	Blows
3	6

GRAPHIC LOG **SCALE:** 1"=5'

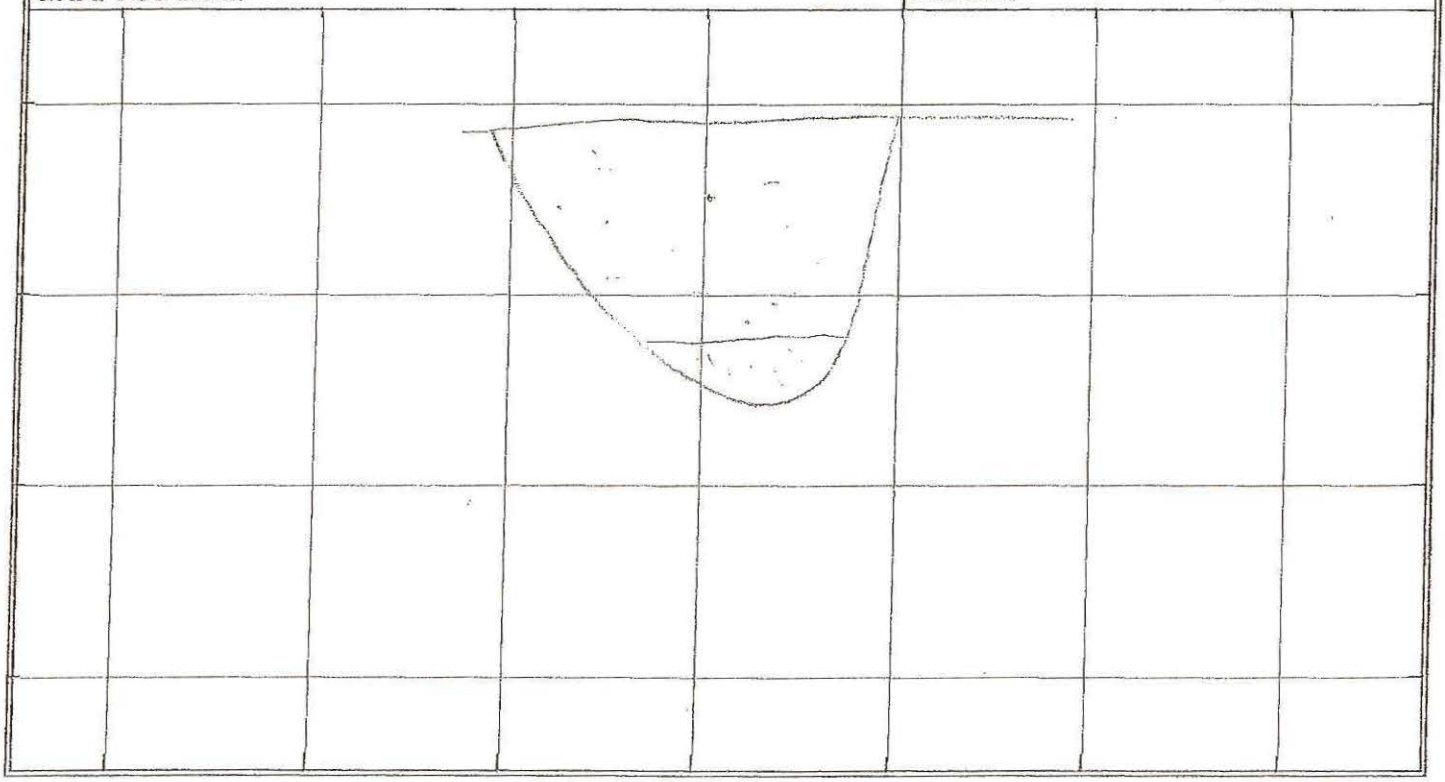


Plate: A - 18

TRENCH LOG No. 19

Client: Brookfield Land			**W.O.** 81-02081-0011
Project: Hanna Property			**Date:** 16-Aug-05
Location: Winchester			**Sheet:** 1 of 1
Est. Surface Elev: 1438	**Total Depth:** 9.5	**Subcontractor:** Al-Roy Drilling	
Orientation: N10E—>	**Length:** 9	**Equip. type:** JD330	

SUBSURFACE CONDITIONS

Surficial Soil

0-6.5 Sandy silt, brown, dry, soft/loose

@1 Becomes moist, soft to firm and porous

Bedrock (Mzp)

6.5-9.5 Phyllite, dark gray, damp, fissile, soft rock.
Excavates with ease to 8.5
Excavates with effort 8.5-9.5

REMARKS **Logged By:** jd

Bag Sample
3-5

Drive Sample	Blows
3	9

GRAPHIC LOG **SCALE:** 1"=5'

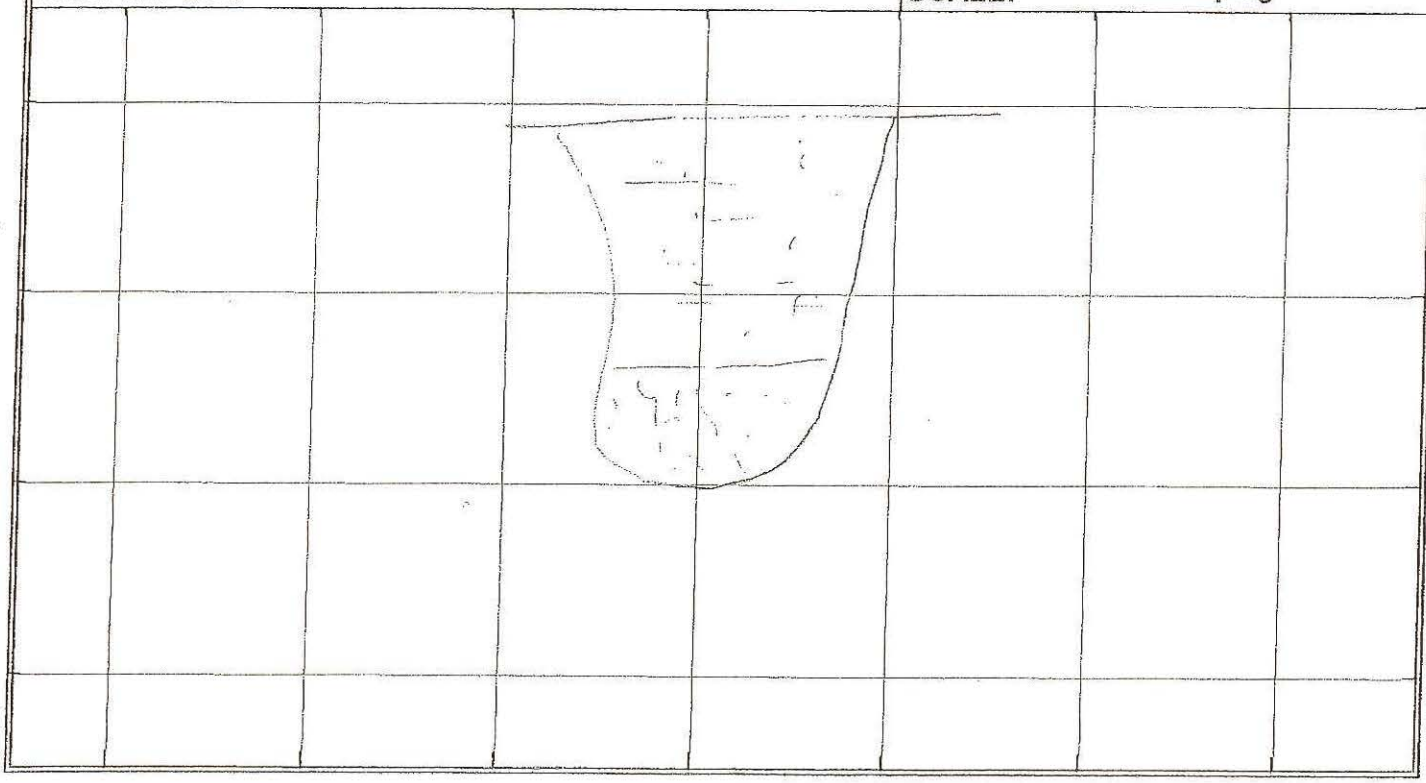


Plate: A - 19

TRENCH LOG No. 20

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1450	**Total Depth:**	6	**Subcontractor:**	Al-Roy Drilling
Orientation:	N40W--->	**Length:**	11	**Equip. type:**	JD330

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-4.5 Sandy Silt, brown, dry, loose
At 1' becomes moist, soft, porous
Grades to bedrock.

Bedrock (Mzp)

4.5-6 Phyllite, dark gray, damp, fissile, soft rock.
Excavates with effort.

REMARKS **Logged By:** jd

Foliation E-W,74N

GRAPHIC LOG **SCALE:** 1"=5'

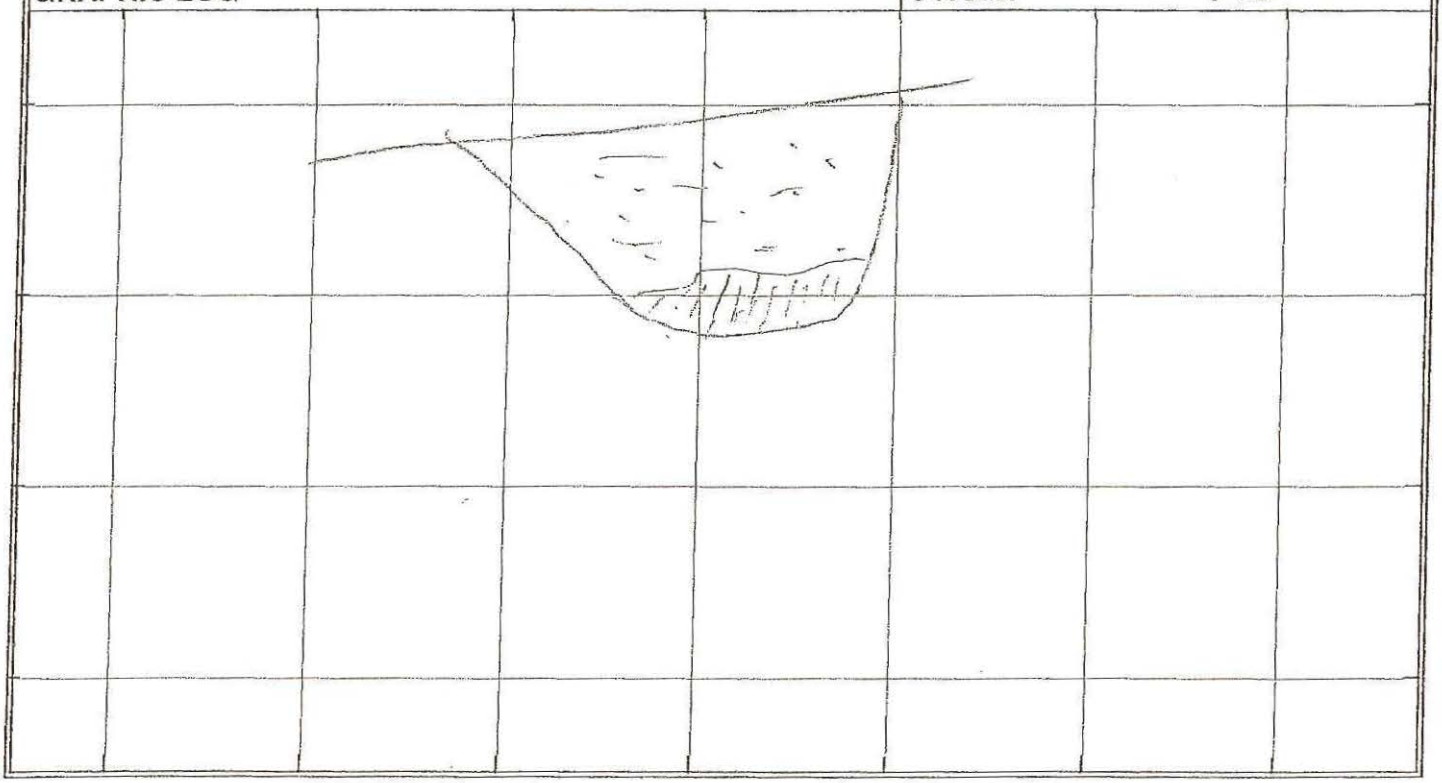


Plate: A - 20

TRENCH LOG No. 21

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1464	**Total Depth:**	4	**Subcontractor:**	Al-Roy Drilling
Orientation:	N15E--->	**Length:**	9	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	**REMARKS**	**Logged By:** jd
Surficial Soil 0-3 Clayey silt, dark gray, moist, soft to firm, porous Bedrock (Mzp) 3-5 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N80W, 62NE	

GRAPHIC LOG **SCALE:** 1"=5'

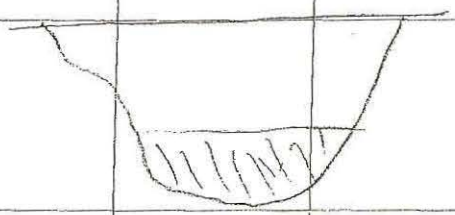


Plate: A - 21

TRENCH LOG No. 22

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1500	**Total Depth:**	4.5	**Subcontractor:**	Al-Roy Drilling
Orientation:	<---N45W	**Length:**	9	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-3.5 Sandy silt, dark brown, damp, soft to firm, porous Upper 12 inches dry, loose. Bedrock (Mzp) 3.5-4.5 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N62W,59N	

GRAPHIC LOG — SCALE: 1"=5'

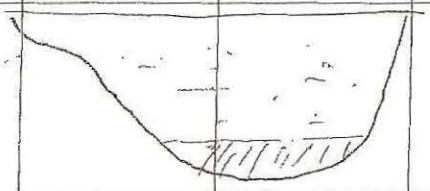


Plate: A - 22

TRENCH LOG No. 23

Client:	**Brookfield Land**			**W.O.**	81-02081-0011
Project:	Hanna Property			**Date:**	16-Aug-05
Location:	Winchester			**Sheet:**	1 of 1
Est. Surface Elev:	1516	**Total Depth:**	3.5	**Subcontractor:**	Al-Roy Drilling
Orientation:	N--->	**Length:**	6	**Equip. type:**	JD330

SUBSURFACE CONDITIONS	REMARKS	Logged By: jd
Surficial Soil 0-2 Sandy silt, gray, dry, loose. Becomes damp at 1'. Bedrock (Mzp) 2-3.5 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.	Foliation N72W,43NE	

GRAPHIC LOG — SCALE: 1"=5'

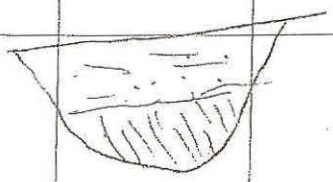


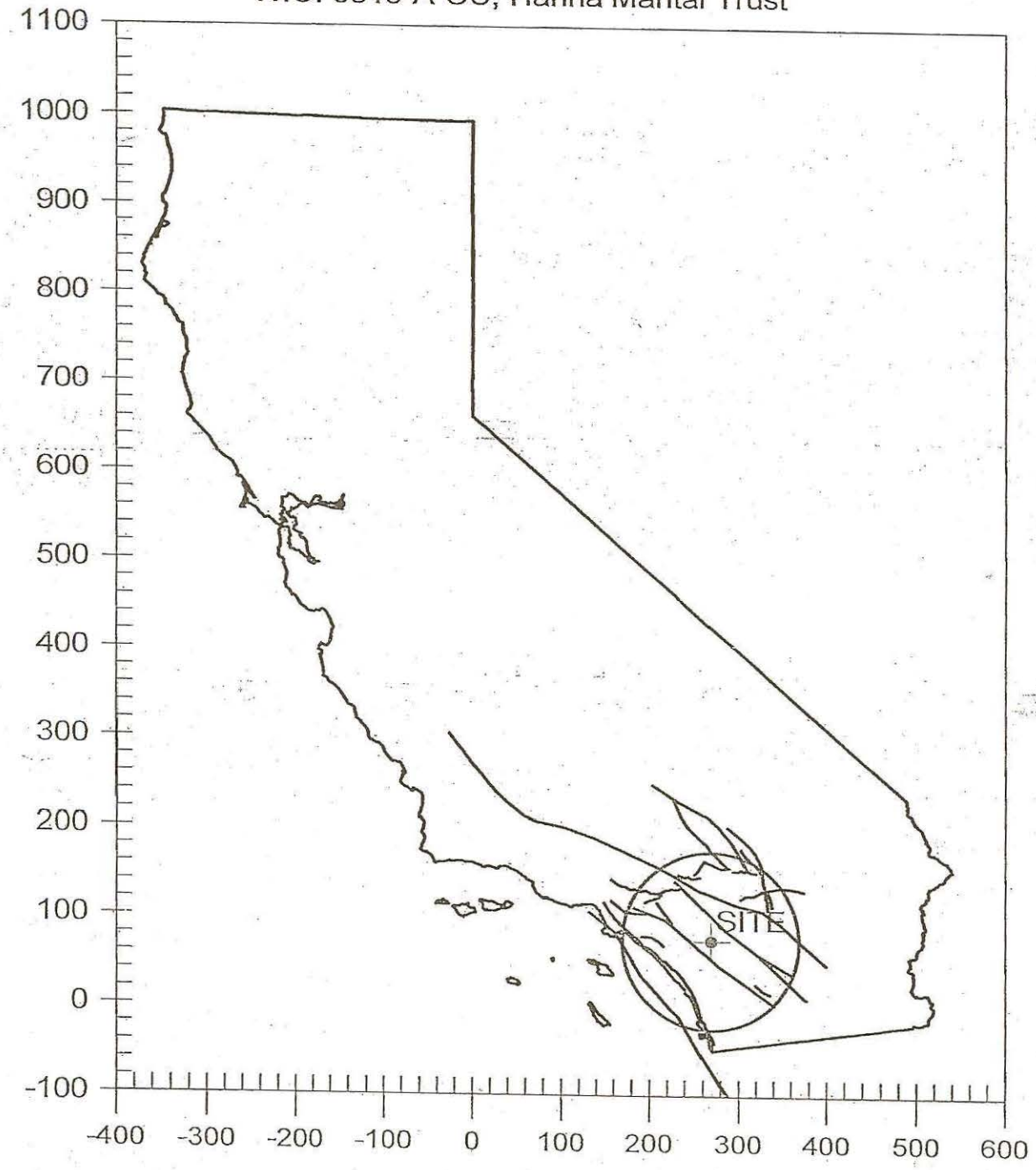
Plate: A - 23

APPENDIX C

EQFAULT, EQSEARCH, AND FRISKSP

CALIFORNIA FAULT MAP - EQFAULT

W.O. 5815-A-OC, Hanna Marital Trust



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*   E Q F A U L T    *
*                    *
*    Version 3.00    *
*                    *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: W.O. 5815-A-OC

DATE: 01-05-2009

JOB NAME: Hanna Marital Trust

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.6303
SITE LONGITUDE: 117.0948

SEARCH RADIUS: 62.14 mi

ATTENUATION RELATION: 16) Campbell & Bozorgnia (1997 Rev.) - Hard Rock
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 1
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 1
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

W.O. 5815-A-OC, Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG. (Mw)	ESTIMATED MAX. EARTHQUAKE EVENT: PEAK SITE ACCEL. g	ESTIMATED MAX. EARTHQUAKE EVENT: EST. SITE INTENSITY MOD. MERC.
ELSINORE (TEMECULA)	9.6(15.5)	6.8	0.321	IX
SAN JACINTO-SAN JACINTO VALLEY	12.8(20.6)	6.9	0.257	IX
SAN JACINTO-ANZA	12.9(20.7)	7.2	0.301	IX
ELSINORE (GLEN IVY)	14.7(23.7)	6.8	0.209	VIII
ELSINORE (JULIAN)	18.2(29.3)	7.1	0.200	VIII
SAN JACINTO-SAN BERNARDINO	28.0(45.1)	6.7	0.087	VII
SAN ANDREAS - Whole M-1a	30.0(48.3)	8.0	0.202	VIII
SAN ANDREAS - San Bernardino M-1	30.0(48.3)	7.5	0.147	VIII
SAN ANDREAS - SB-Coach. M-1b-2	30.0(48.3)	7.7	0.168	VIII
SAN ANDREAS - SB-Coach. M-2b	30.0(48.3)	7.7	0.168	VIII
CHINO-CENTRAL AVE. (Elsinore)	30.9(49.8)	6.7	0.078	VII
SAN JOAQUIN HILLS	34.5(55.6)	6.6	0.061	VI
WHITTIER	34.9(56.2)	6.8	0.068	VI
SAN JACINTO-COYOTE CREEK	35.9(57.7)	6.6	0.055	VI
PINTO MOUNTAIN	36.5(58.7)	7.2	0.090	VII
NEWPORT-INGLEWOOD (Offshore)	36.5(58.7)	7.1	0.083	VII
ROSE CANYON	39.9(64.2)	7.2	0.079	VII
SAN ANDREAS - Coachella M-1c-5	41.3(66.4)	7.2	0.075	VII
NORTH FRONTAL FAULT ZONE (West)	42.2(67.9)	7.2	0.070	VI
EARTHQUAKE VALLEY	42.9(69.0)	6.5	0.038	V
NORTH FRONTAL FAULT ZONE (East)	44.7(71.9)	6.7	0.043	VI
CUCAMONGA	44.8(72.1)	6.9	0.050	VI
CLEGHORN	45.5(73.2)	6.5	0.035	V
BURNT MTN.	45.7(73.5)	6.5	0.035	V
NEWPORT-INGLEWOOD (L.A.Basin)	47.8(76.9)	7.1	0.055	VI
SAN JOSE	48.4(77.9)	6.4	0.030	V
EUREKA PEAK	49.0(78.8)	6.4	0.029	V
PUENTE HILLS BLIND THRUST	49.0(78.9)	7.1	0.051	VI
SIERRA MADRE	51.1(82.3)	7.2	0.052	VI
SAN ANDREAS - Cho-Moj M-1b-1	51.1(82.3)	7.8	0.091	VII
SAN ANDREAS - Mojave M-1c-3	51.1(82.3)	7.4	0.065	VI
SAN ANDREAS - 1857 Rupture M-2a	51.1(82.3)	7.8	0.091	VII
CORONADO BANK	52.8(85.0)	7.6	0.074	VII
HELENDALE - S. LOCKHARDT	52.8(85.0)	7.3	0.057	VI
LANDERS	53.2(85.6)	7.3	0.056	VI
PALOS VERDES	54.2(87.3)	7.3	0.055	VI
LENWOOD-LOCKHART-OLD WOMAN SPRGS	56.6(91.1)	7.5	0.061	VI
SAN JACINTO - BORREGO	59.8(96.2)	6.6	0.025	V
JOHNSON VALLEY (Northern)	59.8(96.2)	6.7	0.028	V

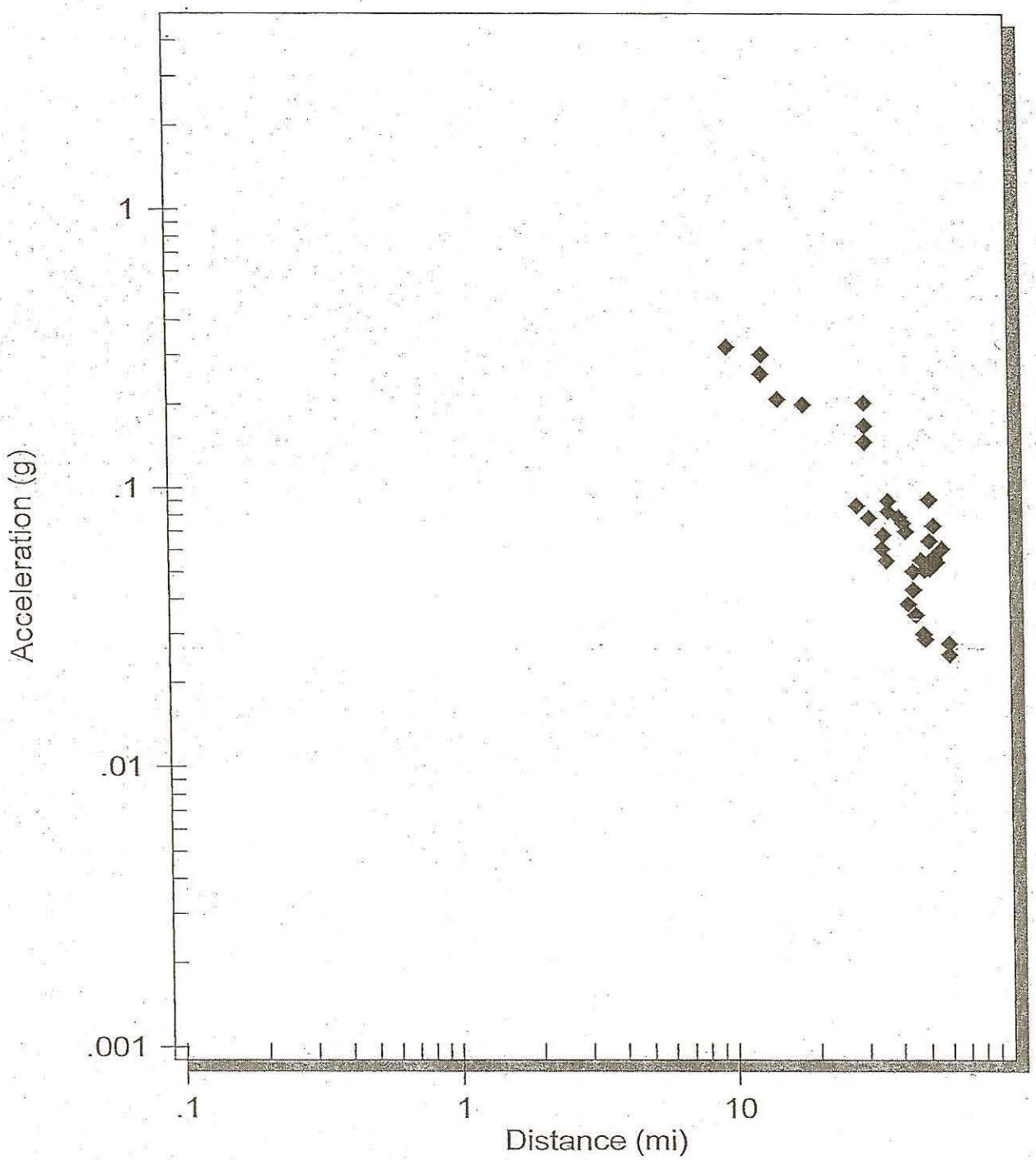
-END OF SEARCH- 39 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 9.6 MILES (15.5 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3213 g

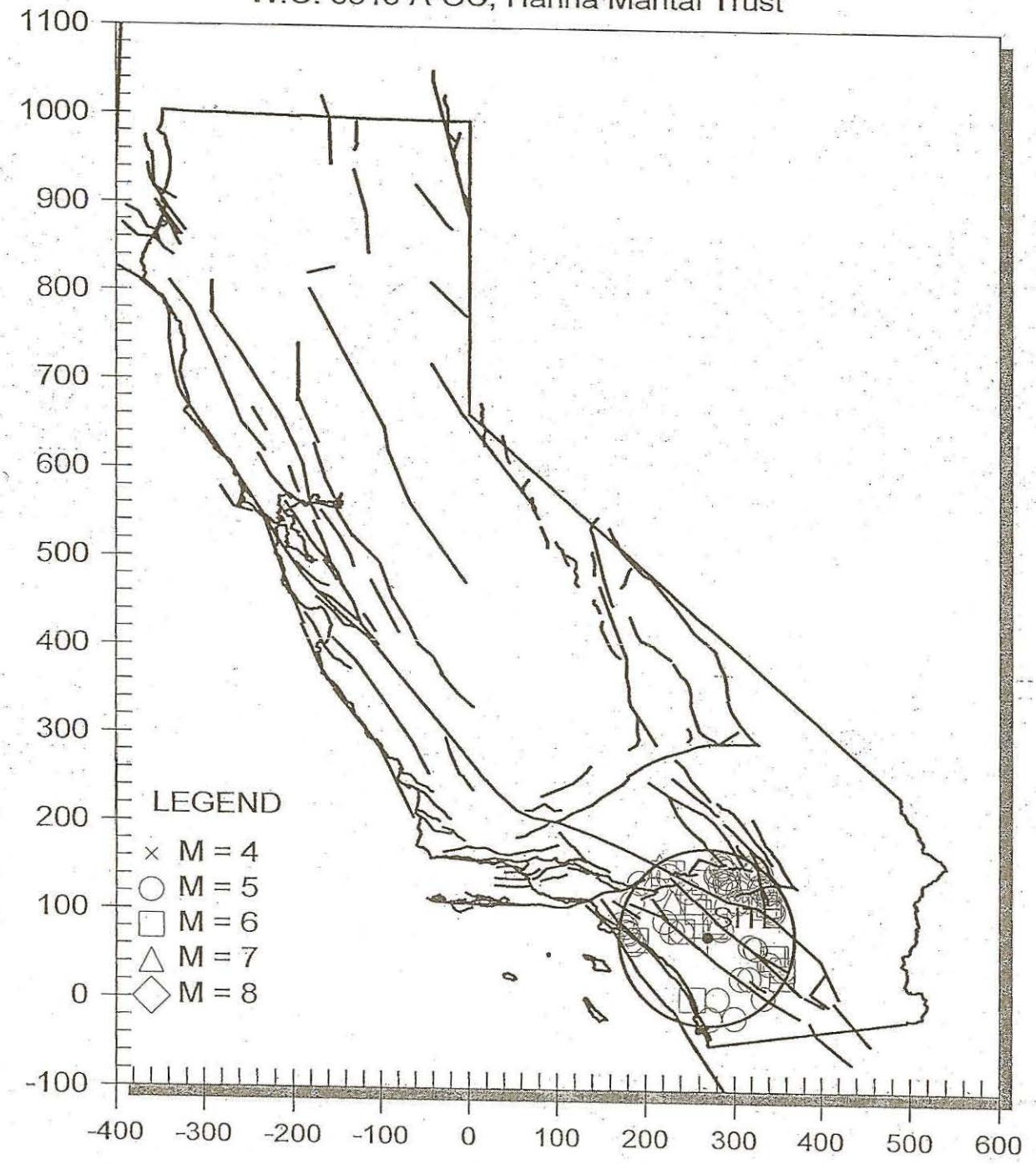
MAXIMUM EARTHQUAKES

W.O. 5815-A-OC, Hanna Marital Trust



EARTHQUAKE EPICENTER MAP

W.O. 5815-A-OC, Hanna Marital Trust



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*    E Q S E A R C H    *
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*     Version 3.00      *
*                       *
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          ESTIMATION OF
      PEAK ACCELERATION FROM
  CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: W.O. 5815-A-OC
                                        DATE: 01-05-2009

JOB NAME: Hanna Marital Trust

EARTHQUAKE-CATALOG-FILE NAME: C:\Program Files\EQSEARCH\ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.6303
   SITE LONGITUDE:  117.0948

SEARCH DATES:
          START DATE:   1800
          END DATE:   2008

SEARCH RADIUS:
          62.1 mi
          100.0 km

ATTENUATION RELATION:  16) Campbell & Bozorgnia (1997 Rev.) - Hard 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:  0     Campbell SHR:  1
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
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EARTHQUAKE SEARCH RESULTS

W.O. 5815-A-OC, 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]
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.308	IX	9.9(15.9)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.082	VII	9.9(15.9)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.070	VI	11.2(18.0)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.185	VIII	12.9(20.8)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.089	VII	18.2(29.3)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.036	V	18.2(29.3)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.036	V	18.2(29.3)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.080	VII	19.6(31.5)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.037	V	24.4(39.2)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.021	IV	26.2(42.1)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.062	VI	27.0(43.5)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.020	IV	30.9(49.7)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.016	IV	31.3(50.3)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.017	IV	32.1(51.6)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.019	IV	33.4(53.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.015	IV	34.5(55.5)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.018	IV	34.5(55.5)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.082	VII	34.5(55.6)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.022	IV	34.6(55.7)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.013	III	35.4(56.9)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.014	IV	36.6(58.8)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.012	III	37.4(60.2)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.021	IV	37.8(60.9)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.014	III	39.2(63.2)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.012	III	39.2(63.2)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.015	IV	39.3(63.2)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.028	V	39.3(63.3)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.015	IV	39.5(63.6)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.014	IV	41.2(66.3)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.014	IV	41.2(66.3)
GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.047	VI	42.4(68.2)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.016	IV	43.0(69.3)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.010	III	43.3(69.7)
DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.011	III	43.3(69.7)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.015	IV	43.3(69.7)
DMG	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.012	III	43.3(69.7)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.010	III	43.9(70.6)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.012	III	44.5(71.7)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.015	IV	44.6(71.7)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.036	V	45.1(72.6)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.035	V	45.9(73.8)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.010	III	46.3(74.6)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.018	IV	47.5(76.4)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.027	V	48.4(77.9)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.010	III	49.0(78.9)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.010	III	49.4(79.5)
GSP	33.9610	116.3180	04/23/1992	045023.0	12.0	6.10	0.021	IV	50.1(80.6)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.025	V	50.1(80.7)
GSP	33.9020	116.2840	07/24/1992	181436.2	9.0	5.00	0.008	II	50.2(80.7)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.010	III	50.3(80.9)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.019	IV	50.4(81.1)
GSP	33.8760	116.2670	06/29/1992	160142.8	1.0	5.20	0.009	III	50.5(81.2)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.011	III	51.0(82.1)

EARTHQUAKE SEARCH RESULTS

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]
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.009	III	51.2(82.4)
GSP	34.1080	116.4040	06/29/1992	141338.8	9.0	5.40	0.011	III	51.5(82.9)
GSP	34.0640	116.3610	09/15/1992	084711.3	9.0	5.20	0.009	III	51.6(83.1)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.029	V	51.7(83.2)
GSP	34.1390	116.4310	06/28/1992	123640.6	10.0	5.10	0.008	III	51.8(83.3)
GSP	34.0290	116.3210	08/21/1993	014638.4	9.0	5.00	0.007	II	52.2(84.0)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.010	III	52.2(84.1)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.008	II	53.0(85.3)
DMG	34.0670	116.3330	05/18/1940	72132.7	0.0	5.00	0.007	II	53.1(85.4)
DMG	34.0670	116.3330	05/18/1940	55120.2	0.0	5.20	0.009	III	53.1(85.4)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.017	IV	54.5(87.8)
GSN	34.2010	116.4360	06/28/1992	115734.1	1.0	7.60	0.070	VI	54.6(87.8)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.011	III	55.0(88.5)
DMG	34.0830	116.3000	05/18/1940	5 358.5	0.0	5.40	0.010	III	55.3(88.9)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.007	II	56.1(90.2)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.007	II	56.1(90.2)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.006	II	57.3(92.3)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.007	II	57.4(92.3)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.008	III	57.4(92.3)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.006	II	57.4(92.3)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.007	II	57.4(92.3)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.006	II	57.4(92.3)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.006	II	57.7(92.9)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.010	III	57.7(92.9)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.007	II	57.7(92.9)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.019	IV	57.7(92.9)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.006	II	57.9(93.1)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.007	II	57.9(93.2)
GSP	34.3410	116.5290	06/28/1992	124053.5	6.0	5.20	0.007	II	58.8(94.6)
GSP	34.2680	116.4020	06/16/1994	162427.5	3.0	5.00	0.006	II	59.3(95.4)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.019	IV	59.5(95.8)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.011	III	59.8(96.2)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.036	V	60.1(96.8)
GSP	34.3320	116.4620	07/01/1992	074029.9	9.0	5.40	0.008	III	60.5(97.3)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.008	III	60.6(97.4)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.009	III	60.6(97.6)
PAS	34.3270	116.4450	03/15/1979	21 716.5	2.5	5.20	0.007	II	60.8(97.9)

****

-END OF SEARCH- 90 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2008

LENGTH OF SEARCH TIME: 209 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 9.9 MILES (15.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.308 g

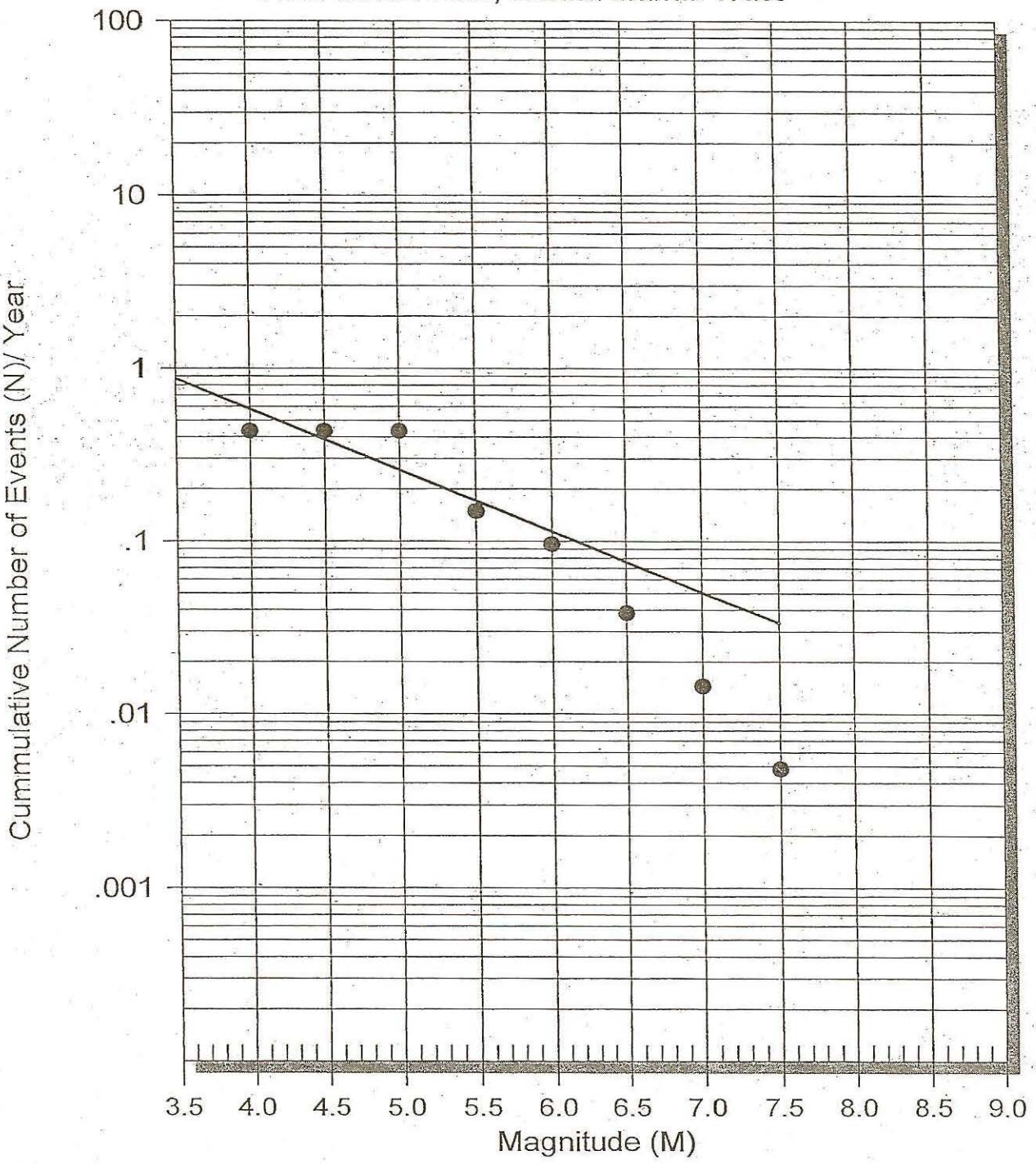
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.180
b-value= 0.354
beta-value= 0.815

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	90	0.43062
4.5	90	0.43062
5.0	90	0.43062
5.5	31	0.14833
6.0	20	0.09569
6.5	8	0.03828
7.0	3	0.01435
7.5	1	0.00478

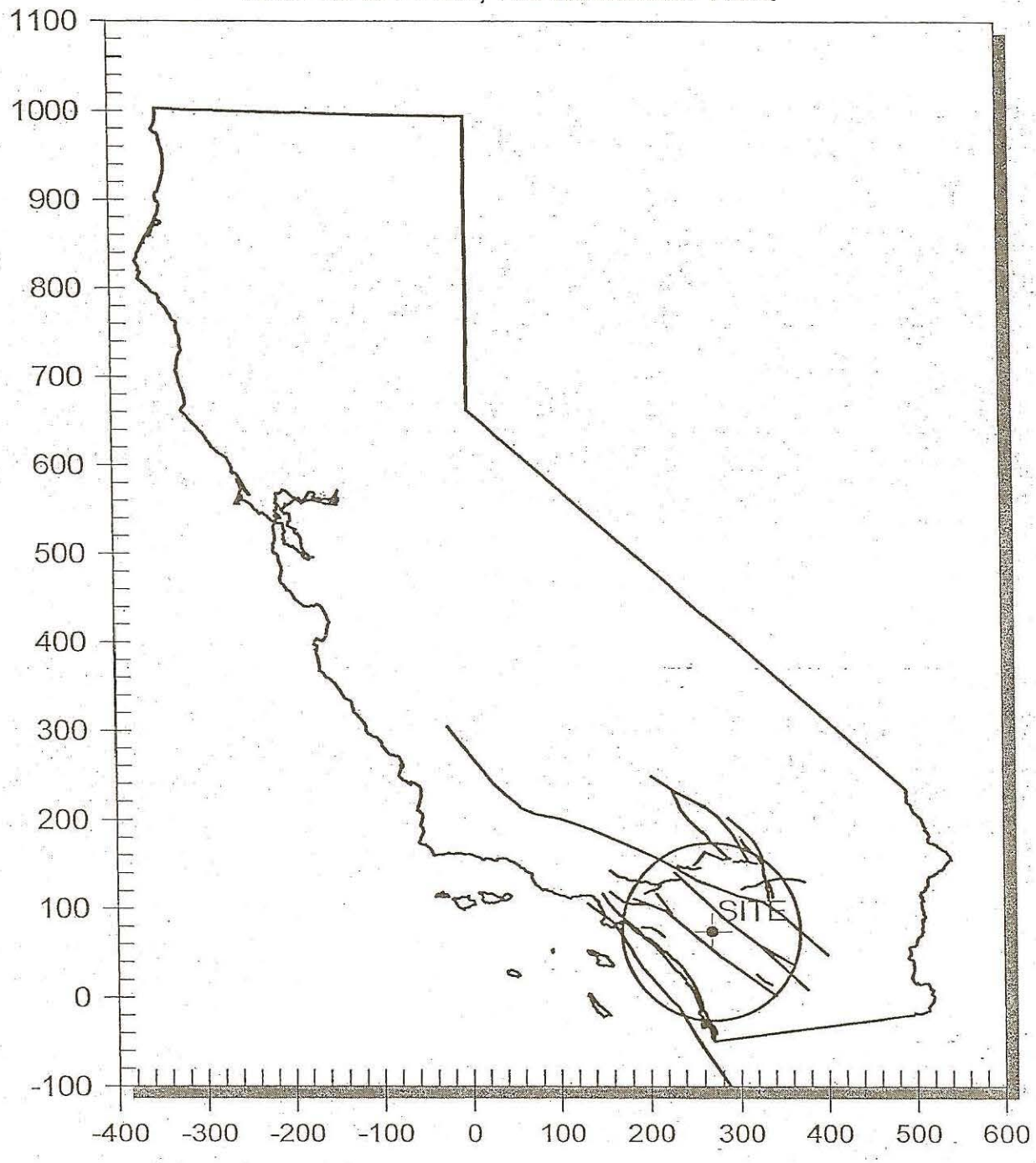
EARTHQUAKE RECURRENCE CURVE

W.O. 5815-A-OC, Hanna Marital Trust



CALIFORNIA FAULT MAP - FRISKSP

W.O. 5815-A-OC, Hanna Marital Trust



```
*****************************************
*                                       *
*        FRISKSP - IBM-PC VERSION       *
*                                       *
*  Modified from *FRISK* (McGuire 1978) *
*   To Perform Probabilistic Earthquake *
*  Hazard Analyses Using Multiple Forms *
* of Ground-Motion-Attenuation Relations *
*                                       *
*   Modifications by:  Thomas F. Blake  *
*              - 1988-2000 -            *
*                                       *
*              VERSION 4.00             *
*            (Visual Fortran)           *
*****************************************
```

TITLE: Hanna Marital Trust

IPR_FILE
0

IPLOT
0

SITE CONDITION
1.00

BASEMENT DEPTH (km)
1.00

RHGA FACTOR	RHGA DIST (km)
1.000	0.000

NFLT	NSITE	NPROB	NATT	LCD
40	1	2	6	1

CLOSEST DISTANCES BETWEEN SITE AND FAULT RUPTURES

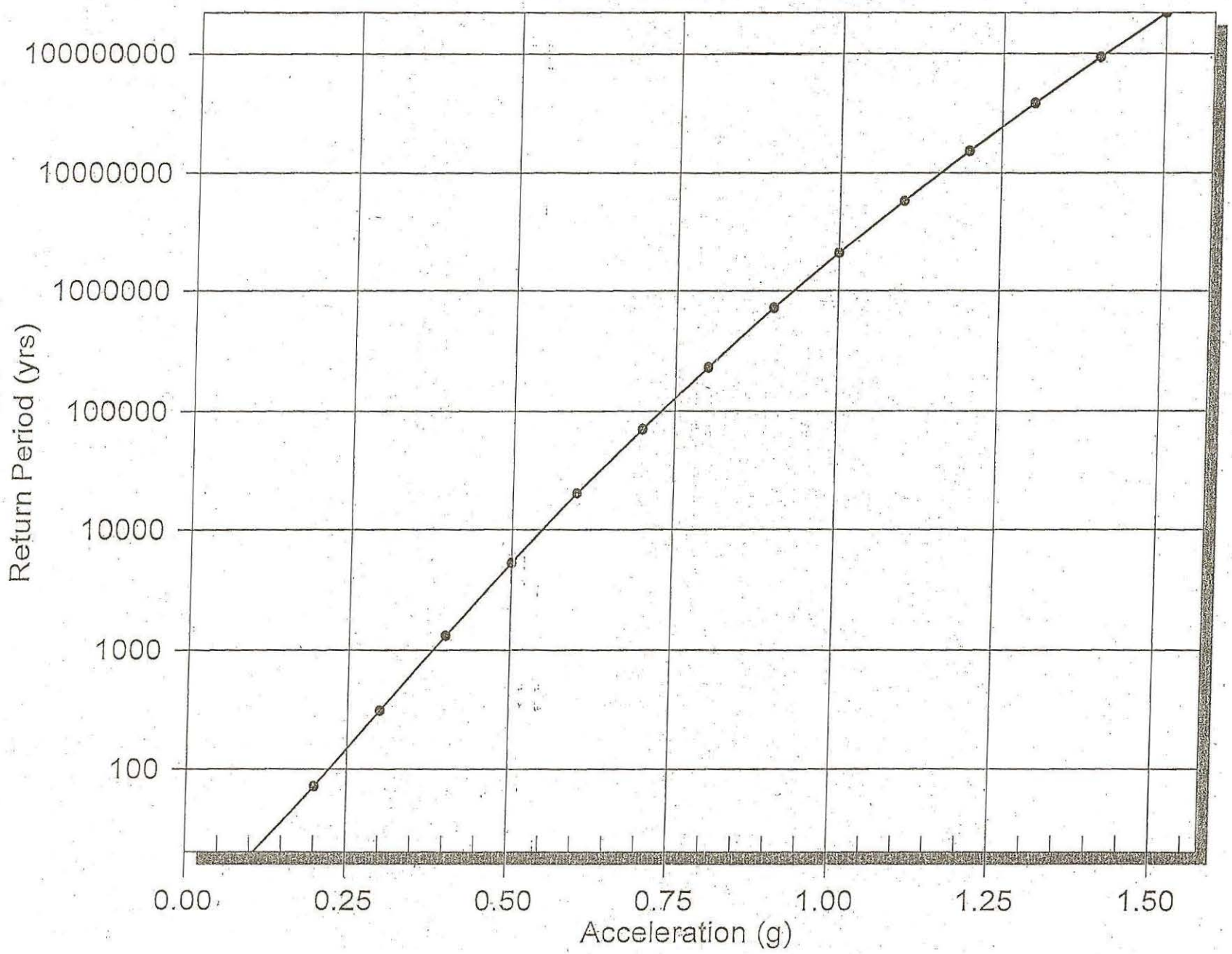
NO.	FAULT NAME	CD_1DRP	CD_2DRP	CDIST	CLODIS	CD_EPI	CD_HYPO
1	ELSINORE (TEMECULA)	15.2	15.2	15.5	15.2	15.2	15.2 km
2	SAN JACINTO-SAN JACINTO VALLEY	20.3	20.3	20.6	20.3	20.3	20.4 km
3	SAN JACINTO-ANZA	20.5	20.5	20.7	20.5	20.8	20.8 km
4	ELSINORE (GLEN IVY)	23.5	23.5	23.7	23.5	24.4	24.4 km
5	ELSINORE (JULIAN)	29.2	29.2	29.3	29.2	30.0	30.1 km
6	SAN JACINTO-SAN BERNARDINO	45.0	45.0	45.1	45.0	45.9	45.9 km
7	SAN ANDREAS - SB-Coach. M-1b-2	48.2	48.2	48.3	48.2	48.3	48.4 km
8	SAN ANDREAS - Whole M-1a	48.2	48.2	48.3	48.2	48.2	48.6 km
9	SAN ANDREAS - SB-Coach. M-2b	48.2	48.2	48.3	48.2	48.3	48.4 km
10	SAN ANDREAS - San Bernardino M-1	48.2	48.2	48.3	48.2	48.2	48.3 km
11	CHINO-CENTRAL AVE. (Elsinore)	49.0	49.0	49.8	49.0	50.2	50.2 km
12	SAN JOAQUIN HILLS	53.8	53.8	55.6	53.8	55.2	55.3 km
13	WHITTIER	56.0	56.0	56.2	56.0	57.1	57.1 km
14	SAN JACINTO-COYOTE CREEK	57.6	57.6	57.7	57.6	58.6	58.6 km
15	PINTO MOUNTAIN	58.6	58.6	58.7	58.6	59.5	59.5 km
16	NEWPORT-INGLEWOOD (Offshore)	58.6	58.6	58.7	58.6	58.6	58.7 km
17	ROSE CANYON	64.1	64.1	64.2	64.1	64.6	64.6 km
18	SAN ANDREAS - Coachella M-1c-5	66.4	66.4	66.4	66.4	66.8	66.8 km
19	CUCAMONGA	69.1	69.1	72.1	69.1	69.9	69.9 km
20	EARTHQUAKE VALLEY	68.9	68.9	69.0	68.9	70.0	70.0 km
21	CLEGHORN	73.2	73.2	73.2	73.2	73.2	73.2 km
22	BURNT MTN.	73.5	73.5	73.5	73.5	73.9	73.9 km
23	SAN JOSE	77.0	77.0	77.9	77.0	77.4	77.4 km
24	NEWPORT-INGLEWOOD (L.A.Basin)	76.9	76.9	76.9	76.9	77.5	77.5 km
25	NORTH FRONTAL FAULT ZONE (West)	78.0	66.7	67.9	67.9	67.9	69.0 km
26	EUREKA PEAK	78.8	78.8	78.8	78.8	79.3	79.3 km
27	PUENTE HILLS BLIND THRUST	78.8	78.8	78.9	78.9	80.1	80.3 km
28	SIERRA MADRE	81.1	81.1	82.3	81.1	82.4	82.4 km
29	SAN ANDREAS - 1857 Rupture M-2a	82.3	82.3	82.3	82.3	83.2	83.2 km
30	SAN ANDREAS - Mojave M-1c-3	82.3	82.3	82.3	82.3	83.2	83.2 km
31	SAN ANDREAS - Cho-Moj M-1b-1	82.3	82.3	82.3	82.3	83.2	83.2 km
32	NORTH FRONTAL FAULT ZONE (East)	83.2	70.7	71.9	71.9	71.8	72.8 km
33	HELENDALE - S. LOCKHARDT	84.9	84.9	85.0	84.9	85.5	85.5 km
34	CORONADO BANK	85.0	85.0	85.0	85.0	85.0	85.0 km
35	LANDERS	85.5	85.5	85.6	85.5	86.1	86.1 km
36	PALOS VERDES	87.2	87.2	87.3	87.2	87.4	87.4 km
37	LENWOOD-LOCKHART-OLD WOMAN SPRGS	91.0	91.0	91.1	91.0	91.6	91.6 km
38	JOHNSON VALLEY (Northern)	96.1	96.1	96.2	96.1	96.5	96.5 km
39	SAN JACINTO - BORREGO	96.2	96.2	96.2	96.2	97.2	97.2 km
40	CLAMSHELL-SAWPIT	97.1	97.1	100.1	97.1	98.2	98.2 km

EXPLANATION

CD_1DRP = Closest distance to projection of rupture area along fault trace.
CD_2DRP = Closest distance to surface projection of the rupture area.
CDIST = Closest distance to seismogenic rupture.
CLODIS = Closest distance to subsurface rupture.
CD_EPI = Closest epicentral distance.
CD_HYPO = Closest hypocentral distance.

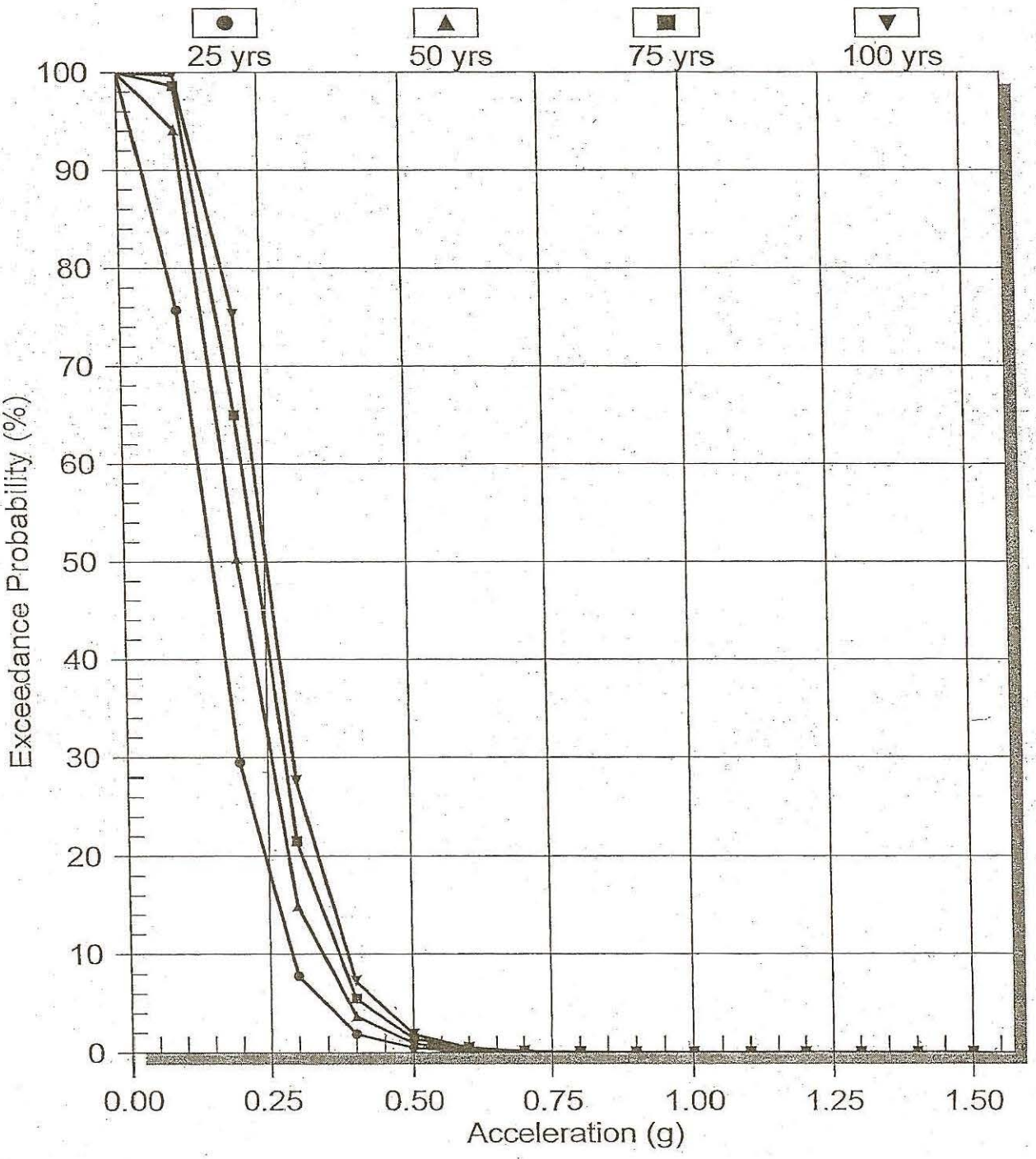
RETURN PERIOD vs. ACCELERATION

W.O. 5815-A-OC, CAMP. & BOZ. (1997 Rev.) HR 1



PROBABILITY OF EXCEEDANCE

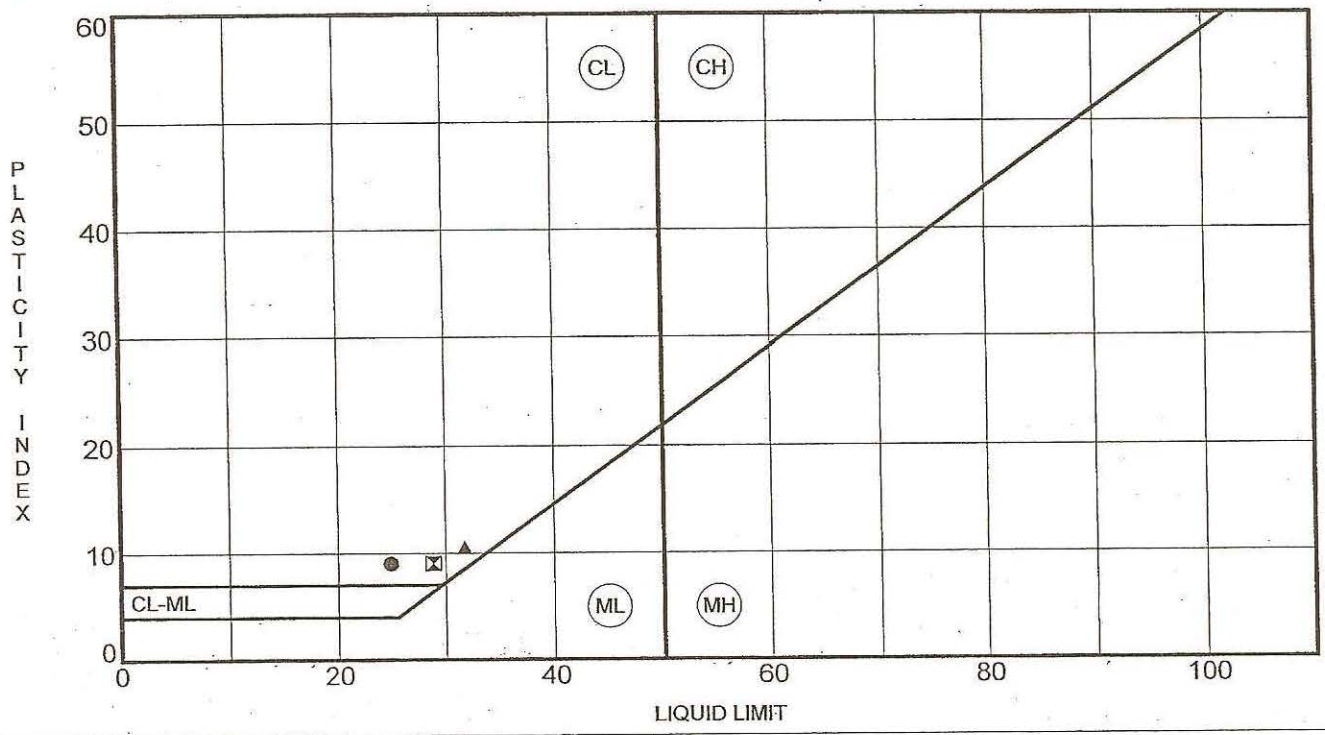
W.O. 5815-A-OC, CAMP. & BOZ. (1997 Rev.) HR 1



	Problem Name	.013889 72	.01 100	.006944 144	.002105 475	.001054 949	Risk 6	Risk 7	Risk 8	Risk 9	Risk 10
1	CAMP. & BOZ. (1997 Rev.) HR 1	0.19987	0.21904	0.24239	0.32642	0.37437	0.00000	0.00000	0.00000	0.00000	0.00000
2	CAMP. & BOZ. (1997 Rev.) HR 2	0.16669	0.19299	0.21451	0.28769	0.32930	0.00000	0.00000	0.00000	0.00000	0.00000
3		0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4							0.00000	0.00000	0.00000	0.00000	0.00000
5		0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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7		0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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13		0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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APPENDIX D

LABORATORY DATA BY GSI



	Specimen Identification	LL	PL	PI	Fines	Classification
●	TP-02 @ 0-5'	25	16	9		Clayey SAND (SC), Grayish Brown
⊠	TP-08 @ 0-5'	29	20	9		Clayey SAND with Gravel (SC), Grayish Brown
▲	TP-10 @ 4-5'	32	21	11		CLAY (CL), Reddish Brown

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Telephone: (714)632-0151
Fax: (714)632-0745

ATTERBERG LIMITS' RESULTS

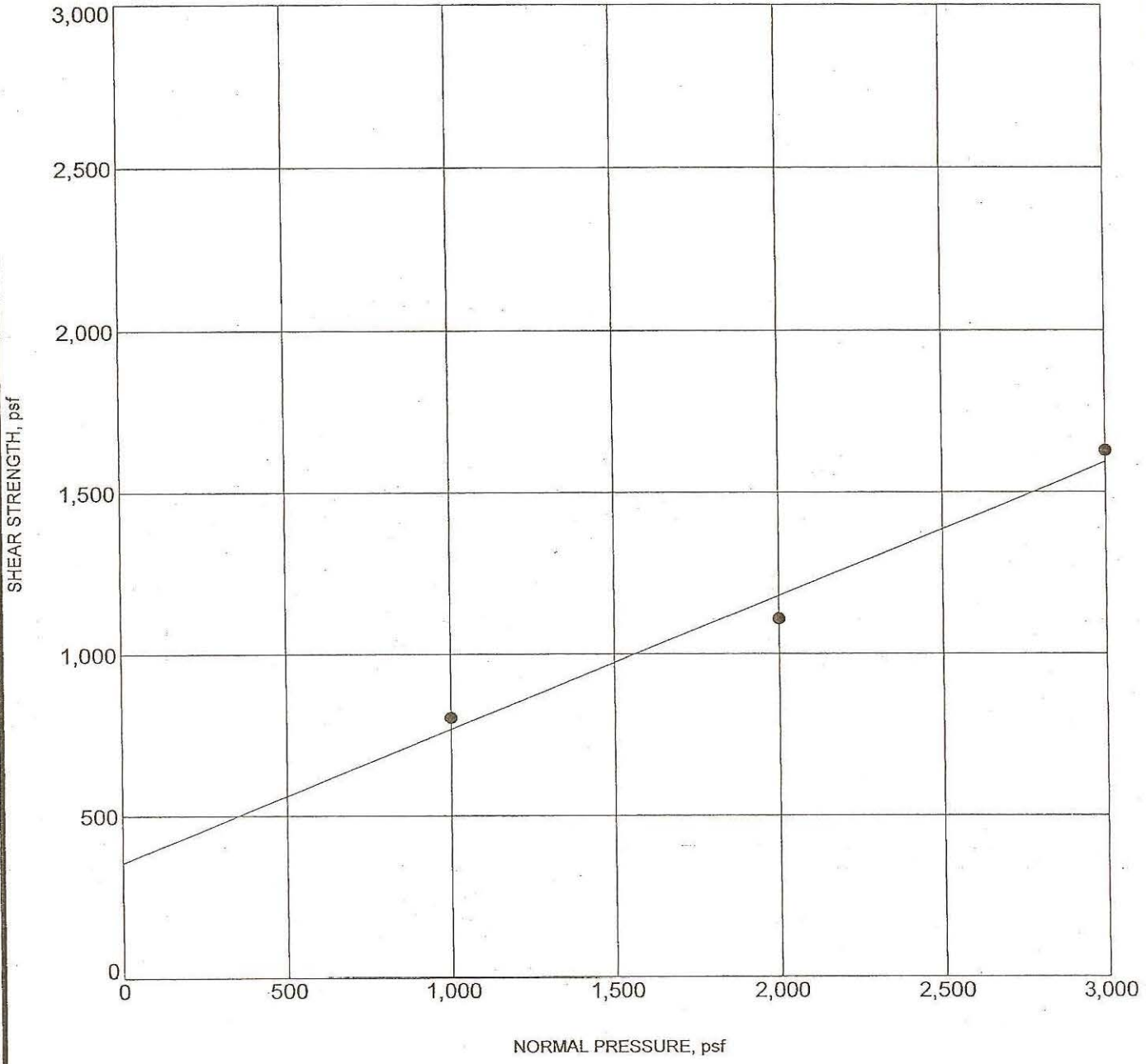
Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co

Number: W.O. 5815-A-OC

PLATE D - 1

US ATTERBERG LIMITS 5815A.GPJ US LAB.GDT 2/25/09



Specimen Identification	Classification	γ_d	MC%	c	ϕ
● TP-02 @ 0-5'	Clayey SAND (SC), Grayish Brown - Remolded	110	18	355	22

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Telephone: (714)632-0151
Fax: (714)632-0745

DIRECT SHEAR TEST

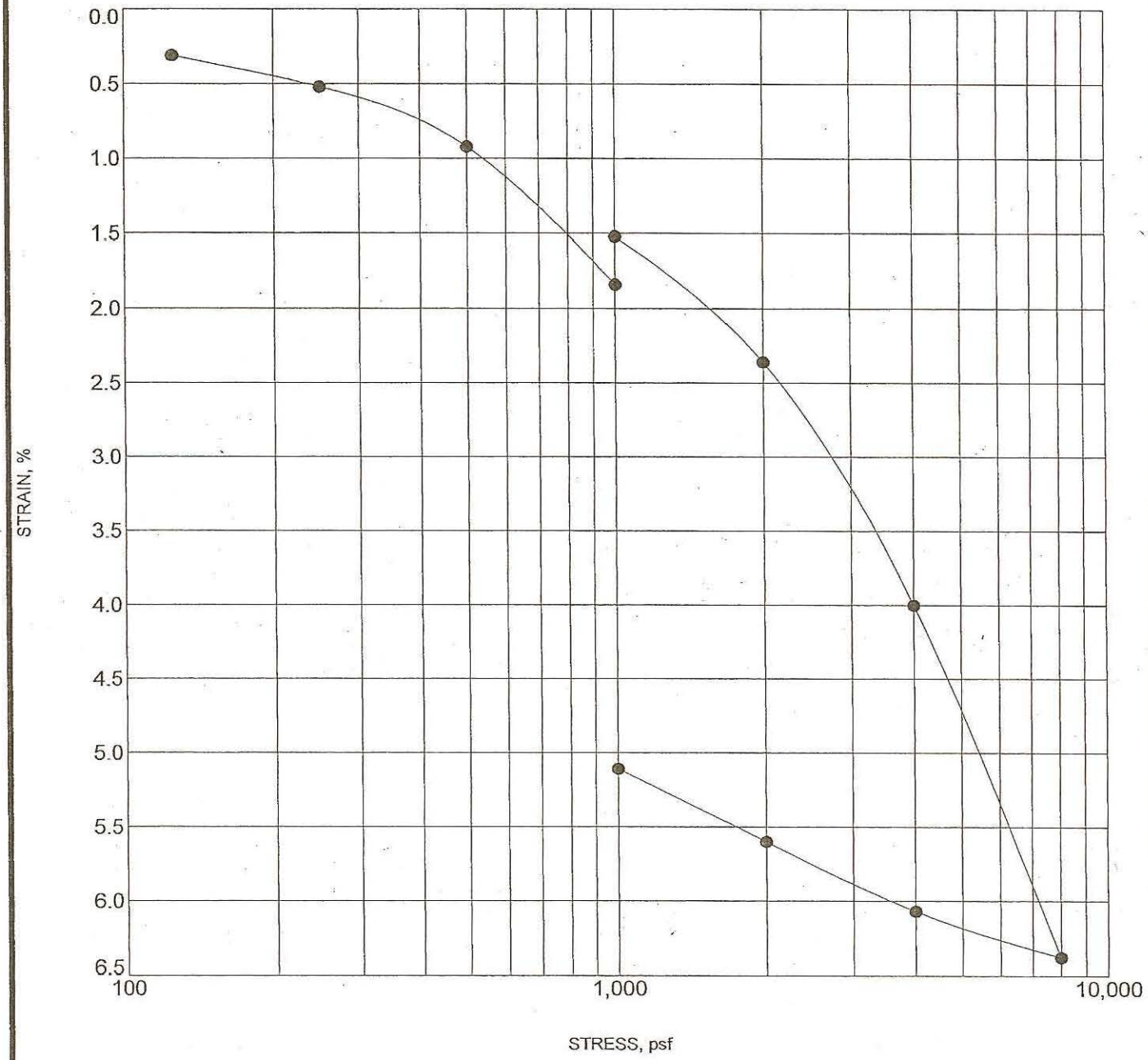
Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE D - 2

US_DIRECT_SHEAR_5815A.GPJ US_LAB.GDT 2/25/09



Specimen Identification	Classification	γ_d	MC%
● TP-02 @ 5'	Clayey SAND (SC), Gray - (Qvoa)	107	18

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Telephone: (714)632-0151
Fax: (714)632-0745

CONSOLIDATION TEST

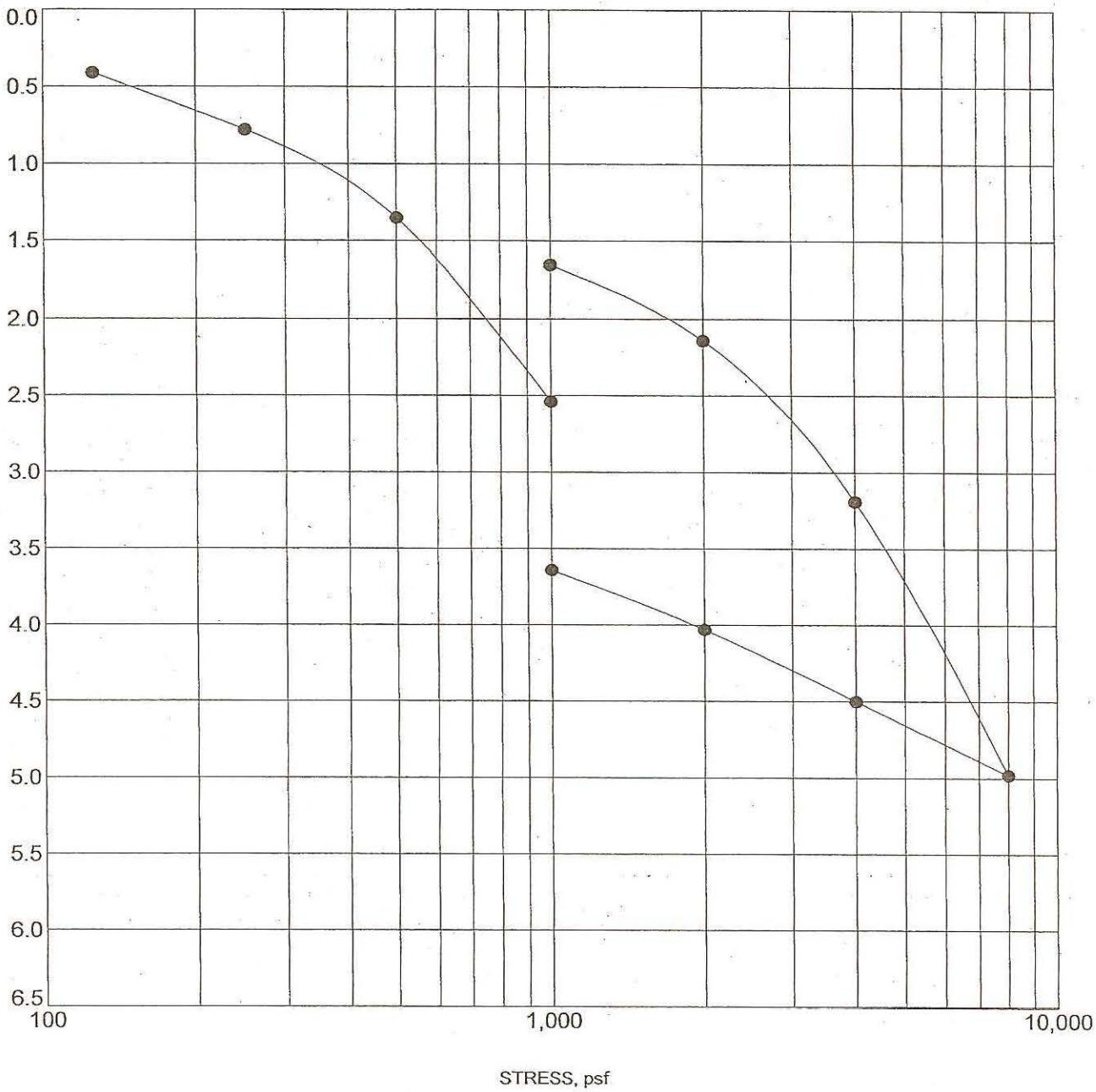
Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE D - 3

US_CONSOL_STRAIN_5815A.GPJ US_LAB.GDT 2/25/09



Specimen Identification	Classification	γ_d	MC%
● TP-10 @ 5'	CLAY with Sand (CL), Dark Gray - (Qvoa)	107	20

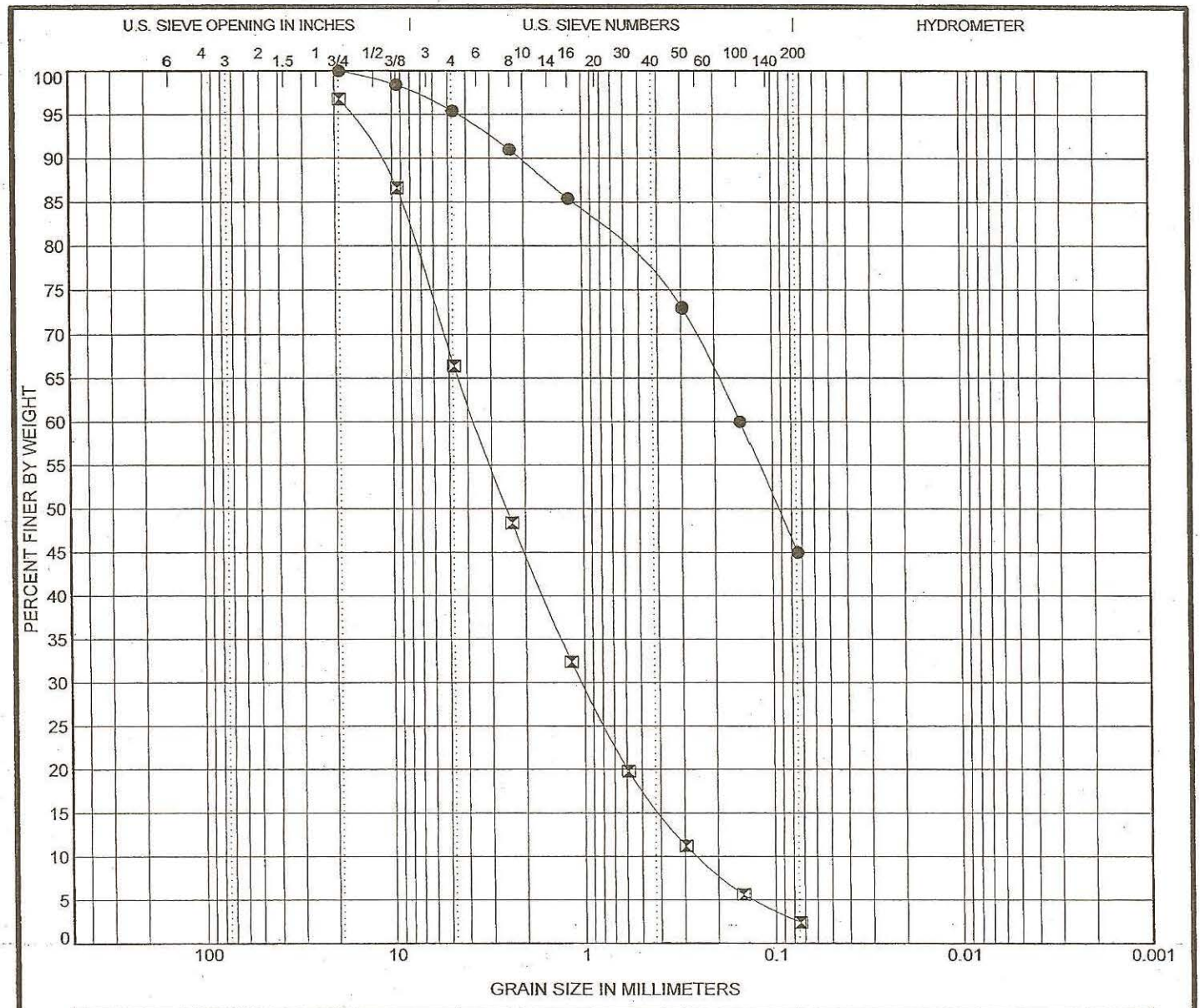
US_CONSOL_STRAIN 5815A.GPJ US_LAB.GDT 2/25/09

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Telephone: (714)632-0151
Fax: (714)632-0745

CONSOLIDATION TEST

Project: Hanna Property
Location: Keller and Winchester Rds, Riv. Co.
Number: W.O. 5815-A-OC

PLATE
D - 4



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Classification	LL	PL	PI	Cc	Cu
● TP-02 @ 0-5'	Clayey SAND (SC), Grayish Brown	25	16	9		
⊠ TP-08 @ 0-5'	Clayey SAND with Gravel (SC), Grayish Brown	29	20	9	1.12	14.48

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP-02 @ 0-5'	18.85	0.147			4.6			
⊠ TP-08 @ 0-5'	18.85	3.68	1.025	0.254	30.1			

GSI
GEOSOILS, INC.
3812 E. La Palma Ave.
Anaheim, California
Telephone: (714)632-0151
Fax: (714)632-0745

GRAIN SIZE DISTRIBUTION

Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE D - 5

US GRAIN SIZE 5815A.GPJ US LAB.GDT 2/25/09

SUMMARY OF LABORATORY TEST DATA

GeoSoils, Inc.
3812 E. La Palma Avenue
Anaheim, CA 92807

QCI Project No.: 08-029-012b
Date: December 12, 2008
Summarized by: ABK

Client: T & B Planning
Geo Soils W.O. 5815-A-OC

Sample ID	Sample Depth (Feet)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 (% By Weight)	Resistivity CT-532 (643) (ohm-cm)
TP-2	0-5	7.39	73	0.0035	1,500
TP-8	0-5	7.22	50	0.0020	850

APPENDIX D2

LABORATORY DATA BY EUC

MOISTURE-DENSITY AND POROSITY

TEST PROCEDURE: ASTM D 2937

Project Name: Brookfield Land
Project No. : 81-02081-0011

Tested By: MY Date: 08/25/05
Data Input By: MY Date: 08/25/05
Checked By: MS Date:

Sample Type	Ring	Ring	Ring	Ring	Ring		
Boring No.	TP-2	TP-3	TP-4	TP-9	TP-18		
Sample No.							
Depth (ft.)	3	1	2	2	3		
Wet Density (pcf)	123.9	122.8	124.8	111.0	119.5		
Moisture Content (%)	10.8	12.4	12.1	8.2	8.3		
Dry Density (pcf)	111.9	109.3	111.3	102.6	110.4		
Void Ratio							
Total Porosity							
Pore Volume (cc)							
Degree of Saturation (%)							
Visual Sample Description:	Fine sandy silt	Fine sandy silt	Fine sandy silt	Fine sandy silt	Fine sandy silt		
Group Symbol	ML	ML	ML	ML	ML		
Color	Gray	Brown	Brown	Brown	Brown		
Maximum Particle Size							
Grain Size Dist. (GR:SA:FI)							
Coarse-Grained Angularity							
Dilitancy (None, Slow, Rapid)							
Cementation							
Moisture Condition							
Consistency / Rel. Density							
Plasticity							
Reaction to HCL							
Dry Strength							

Container Number	24	25	21	2	29		
Wt. Wet Soil + Ring/Tube (gm.)	1221	1519	1532	1443	1498		
Wt. of Ring/Tube (gm.)	146	728	728	728	728		
Average Length (in.)	4	5	5	5	5		
Average Diameter (in.)	2.5	2.5	2.5	2.5	2.5		
Wt. Wet Soil + Container (gm.)	289.50	244.00	258.50	265.00	288		
Wt. Dry Soil + Container (gm.)	276.40	234.10	247.30	256.30	275		
Weight Container (gm.)	156.20	154.00	155.00	150.00	155		
Specific Gravity (assumed):	2.70	2.70	2.70	2.70	2.70		

ATTERBERG LIMITS

Test Procedure ASTM D 4318-93

Project Name: Brookfield Land
Project No. : 81-02081-0011
Location: TP-3 @ 1-2'
Sample No. : 1
Soil Description: Fine sandly silt; brown (SM)

Tested By : MY Date: 08/24/05
Calculated By : MY Date: 08/24/05
Checked By : MS Date:
Depth (ft) :

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			40	27	21	15
Container No.	109	118	117	110	115	114
Wet Wt. Of Soil + Cont. (gm.)	116.11	116.82	116.35	113.77	113.61	112.6
Dry Wt. Of Soil + Cont. (gm.)	114.22	114.84	113.64	111.57	111.15	110.26
Wt. Of Container (gm.)	102.30	102.36	102.82	103.14	102.13	102.26
Moisture Content (%) [Wn]	15.70	15.70	25.00	26.10	27.80	29.30

LIQUID LIMIT	27
PLASTIC LIMIT	16
PLASTICITY INDEX	9

Classification of fine-grained & fine-grained fraction of soils

PI at "A" - Line = 0.73(LL - 20) = 5

One - Point Liquid Limit Calculation LL = Wn(N/25)

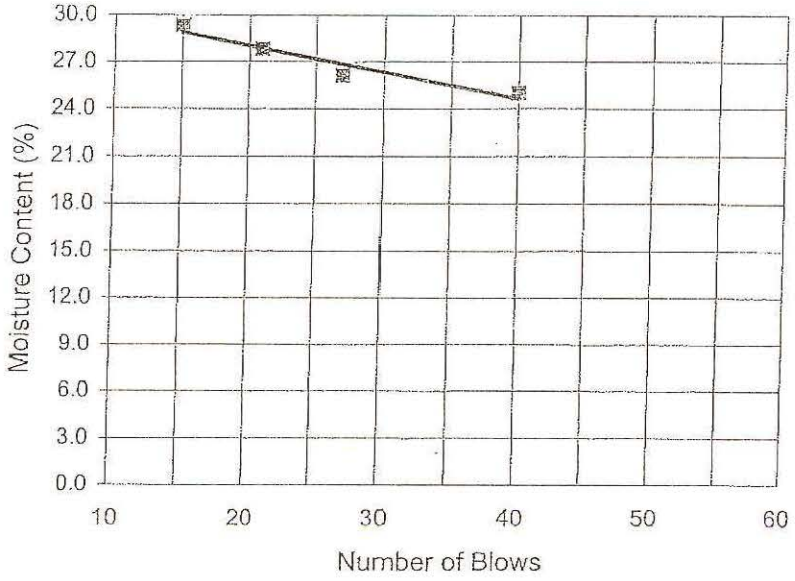
PROCEDURES USED

[x] Wet Preparation
Multipoint Wet Preparation

[ ] Dry Preparation
Multipoint Dry Preparation

[ ] Procedure A
Multipoint Test

[ ] Procedure B
One-point Test



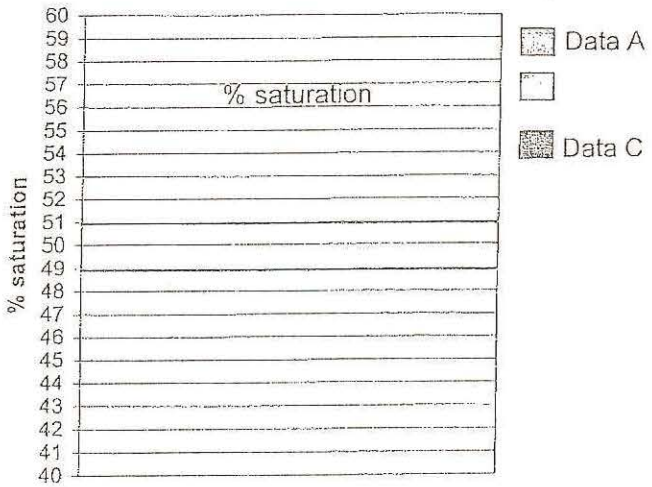
EXPANSION INDEX TEST

Test Procedure ASTM D 4829

Project Name:	Brookfield Land	Tested By :	MY	Date:	08/24/05
Project No. :	81-02081-0011	Calculated By :	MY	Date:	08/24/05
Location:	1-2'	Checked By :	MS	Date:	
Sample No. :	TP-2	Depth (ft) :			
Soil Description:	Fine sandy silt (ML)				

Moisture Content of prepared sample		Final Moisture		F - Fraction passing No. 4		Dry Density of molded sample	
Tare No.		soil + ring wt.	6.38	soil + No. 4 wt.		soil + ring wt.	608.8
wet wt. + tare	200.0	ring wt.	200.3	soil - No. 4 wt.		ring wt.	200.3
dry wt. + tare	183.3	wet wt. of soil	437.7	total wt.		wet wt. of soil	408.5
tare wt.	0	dry wt. of soil	374.4			dry wt. of soil	374.4
water loss	16.7	water wt.	63.3			vol. of ring	0.0073
dry wt.	183.3					dry density	113.07
% moisture	9.1	% moisture	16.90	% - No. 4		% saturation	50.06

Date	Time	Reading
08/24/05	2:40 PM	0.588
	2:50 PM	0.591
	3:10 PM	0.589
08/25/05	8:20 AM	0.57

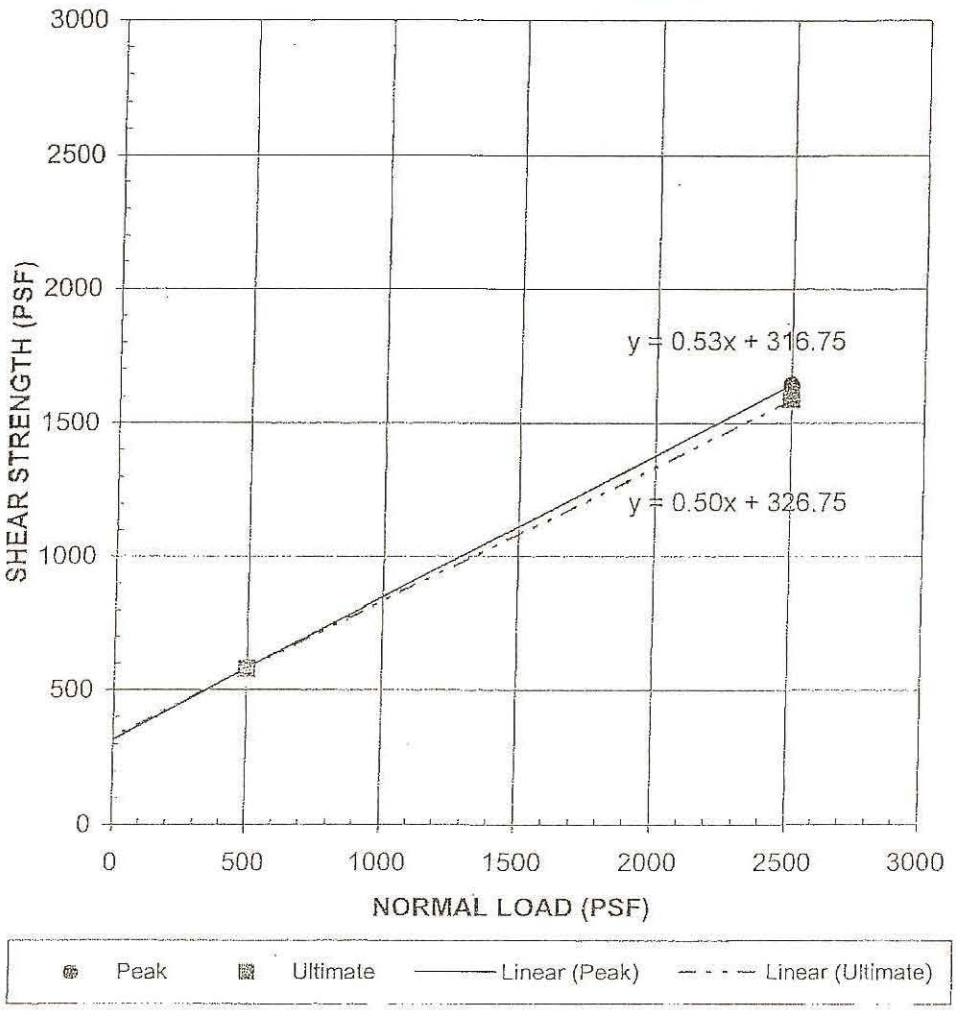


Expansion Index	Potential Expansion
0 - 20	very low
21 - 50	low
51 - 90	medium
91 - 130	high
above 130	very high

Initial reading	0.591
Final reading	0.57
Changing H.	0.021

E.I. = **21**

DIRECT SHEAR RESULTS



	PEAK	ULTIMATE
Cohesion (psf):	317	327
Phi (degrees):	28	27

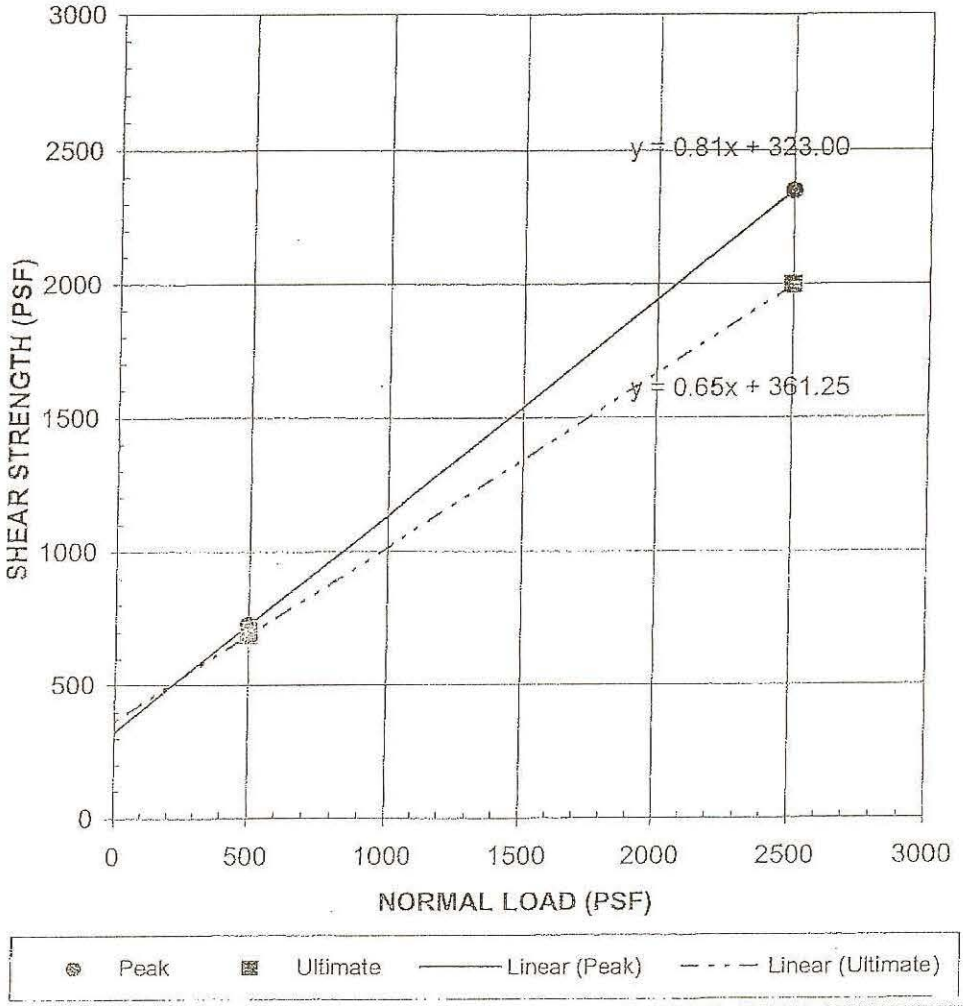
	INITIAL	FINAL
Dry Density (pcf):	105.6	
Moisture (%):	9.0	

NORMAL LOAD (psf): 500, 2500
RATE (in/min): 0.003
SOIL DESCRIPTION: Fine Sandy Silt
U.S.C.S.: ML

W.O. 81-02081-0011 LOCATION: TP-19 @ 3-5'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS

DIRECT SHEAR RESULTS



	PEAK	ULTIMATE
Cohesion (psf):	323	361
Phi (degrees):	39	33

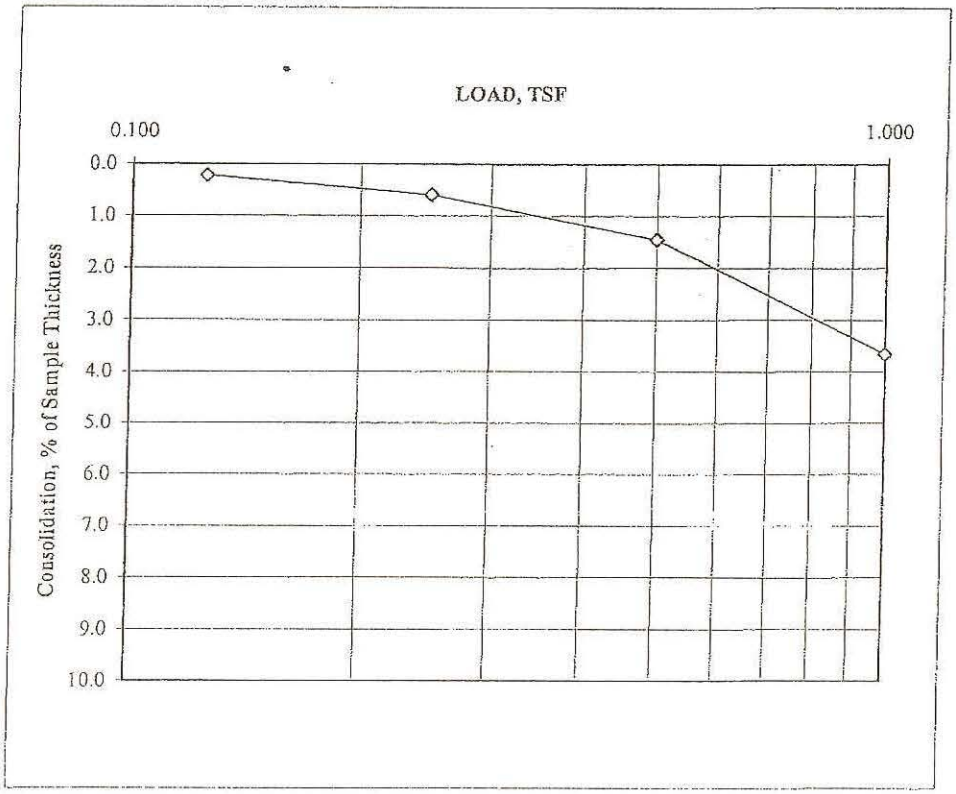
	INITIAL	FINAL
Dry Density (pcf):	105.6	
Moisture (%):	8.5	

NORMAL LOAD (psf): 500, 2500 (Undisturbed)
RATE (in/min): 0.003
SOIL DESCRIPTION: Sandy Silt/Silty Sand
U.S.C.S.: SM/ML

W.O. 81-02081-0011 LOCATION: TP-19 @ 3'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS

CONSOLIDATION TEST RESULTS



Note: Filled rectangle denotes readings after saturation with water

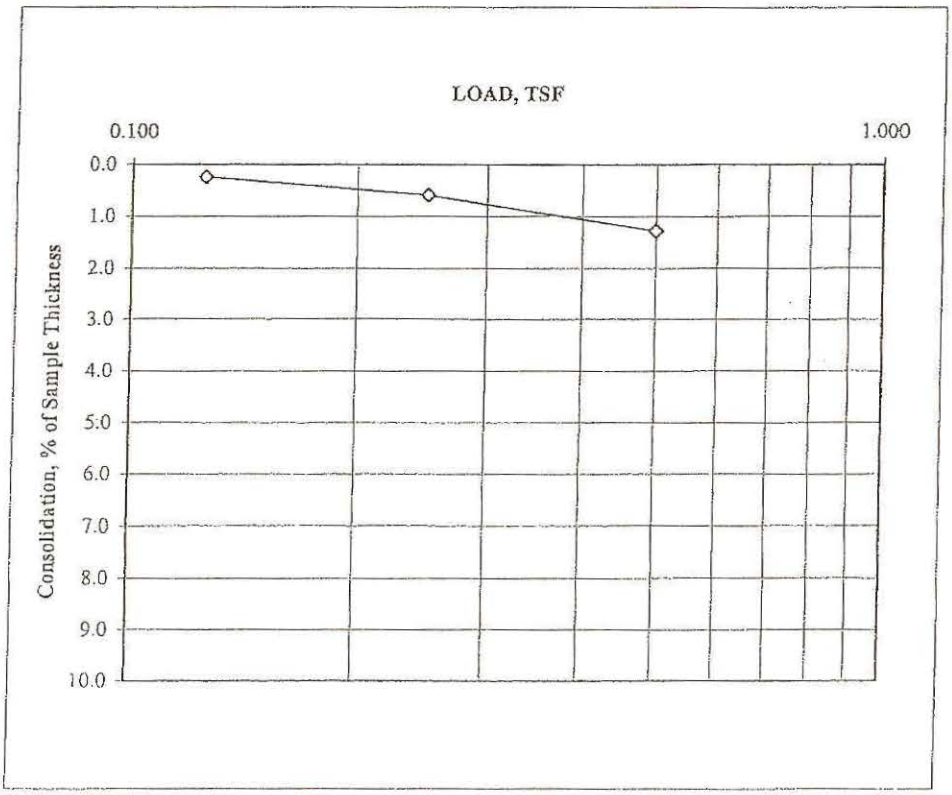
	In-place	Remolded:
Dry Density, (pcf):	109.3	
Moisture (%):	12.4	

WATER ADDED @:	1 TSF
MAXIMUM LOAD, TSF:	1 TSF
SOIL DESCRIPTION:	Fine Sandy Silt; brown
U.S.C.S.	ML

W.O. 81-02081-0011 LOCATION: TP-3 @ 1'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS, INC.

CONSOLIDATION TEST RESULTS



Note: Filled rectangle denotes readings after saturation with water

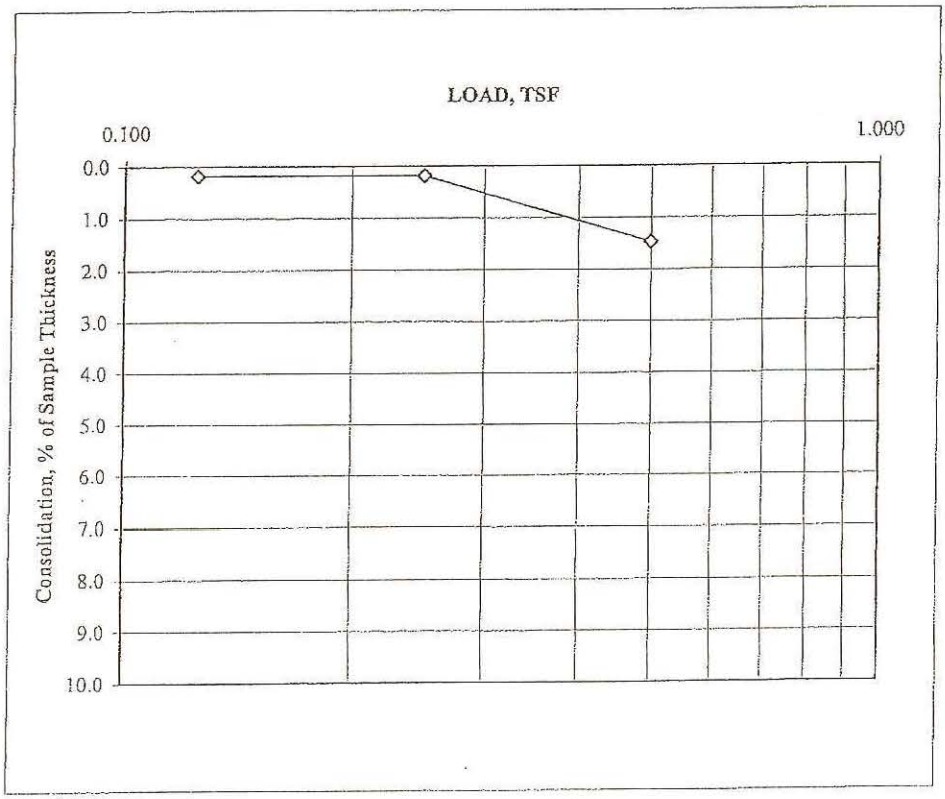
	In-place	Remolded:
Dry Density, (pcf):	111.3	
Moisture (%):	12.1	

WATER ADDED @:	1 TSF
MAXIMUM LOAD, TSF:	1 TSF
SOIL DESCRIPTION:	Fine Sandy Silt; brown
U.S.C.S.	ML

W.O. 81-02081-0011 LOCATION: TP-4 @ 2'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS, INC.

CONSOLIDATION TEST RESULTS



Note: Filled rectangle denotes readings after saturation with water

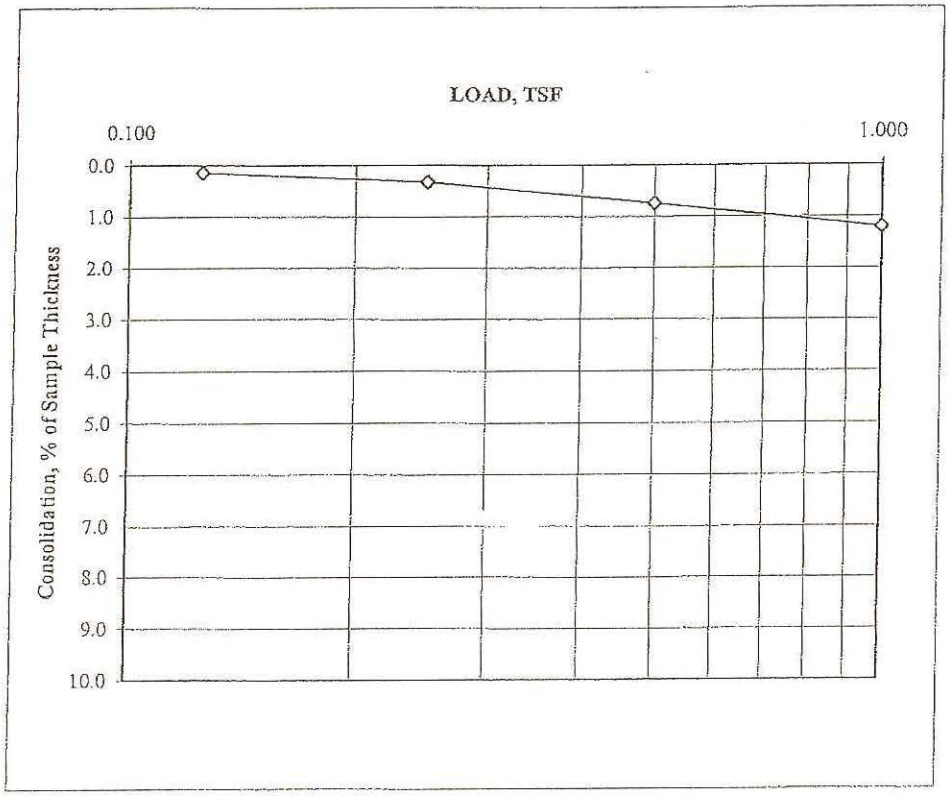
	In-place	Remolded:
Dry Density, (pcf):	102.6	
Moisture (%):	8.2	

WATER ADDED @:	0.5 TSF
MAXIMUM LOAD, TSF:	0.5 TSF
SOIL DESCRIPTION:	Fine Sandy Silt; brown
U.S.C.S.	ML

W.O. 81-02081-0011 LOCATION: TP-9 @ 2'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS, INC.

CONSOLIDATION TEST RESULTS



Note: Filled rectangle denotes readings after saturation with water

	In-place	Remolded:
Dry Density, (pcf):	110.4	
Moisture (%):	8.3	

WATER ADDED @:	1 TSF
MAXIMUM LOAD, TSF:	1 TSF
SOIL DESCRIPTION:	Fine Sandy Silt; brown
U.S.C.S.	ML

W.O. 81-02081-0011 LOCATION: TP-18 @ 3'
CLIENT: Hanna Properties

EBERHART / UNITED CONSULTANTS, INC.

R-VALUE TEST

Test Procedure ASTM D 2844

Project Name: Brookfield Land Tested By : MY Date: 08/26/05

Project No. : 81-02081-0011

Boring No. : Checked By : MS Date:

Sample No. : TP-3 @ 1-2'

Location:

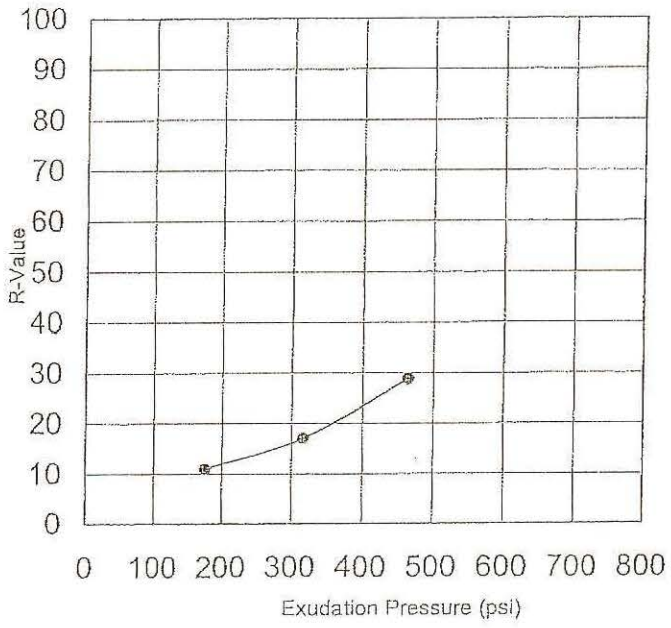
Soil Description: Fine Sandy Silt (ML)

Mold Number	13	5	6		
Water Added (grams)	85	75	70		
Compaction Gage Pressure (psi)	75	110	130		
Gross Weight Mold (grams)	3043	3158	3172		
Tare Weight (grams)	2060	2083	2076		
Net Sample Weight (grams)	983	1075	1096		
Exudation Pressure (lbs)	2220	3980	5830		
Exudation Pressure (psi)	177	317	464		
Sample Height (inches)	2.57	2.5	2.5		
Expansion (inches X .0001)	0	0	0		
Stability 2,000 (160 psi)	130	120	100		
Turns Displacement	4.65	4.06	3.7		
R-Value Uncorrected	11	17	29		
R-Value Corrected	11	17	29		
Actual Test, Water (%)	12.08	11.2	10.8		
Dry Density (pcf)					
Traffic Index					
G.E. by Stability					
G.E. by Expansion					

R-Value by Exudation = 16

R-Value by Expansion =

Equilibrium R-Value (by Exudation) = 16



R-VALUE GRAPHICAL PRESENTATION

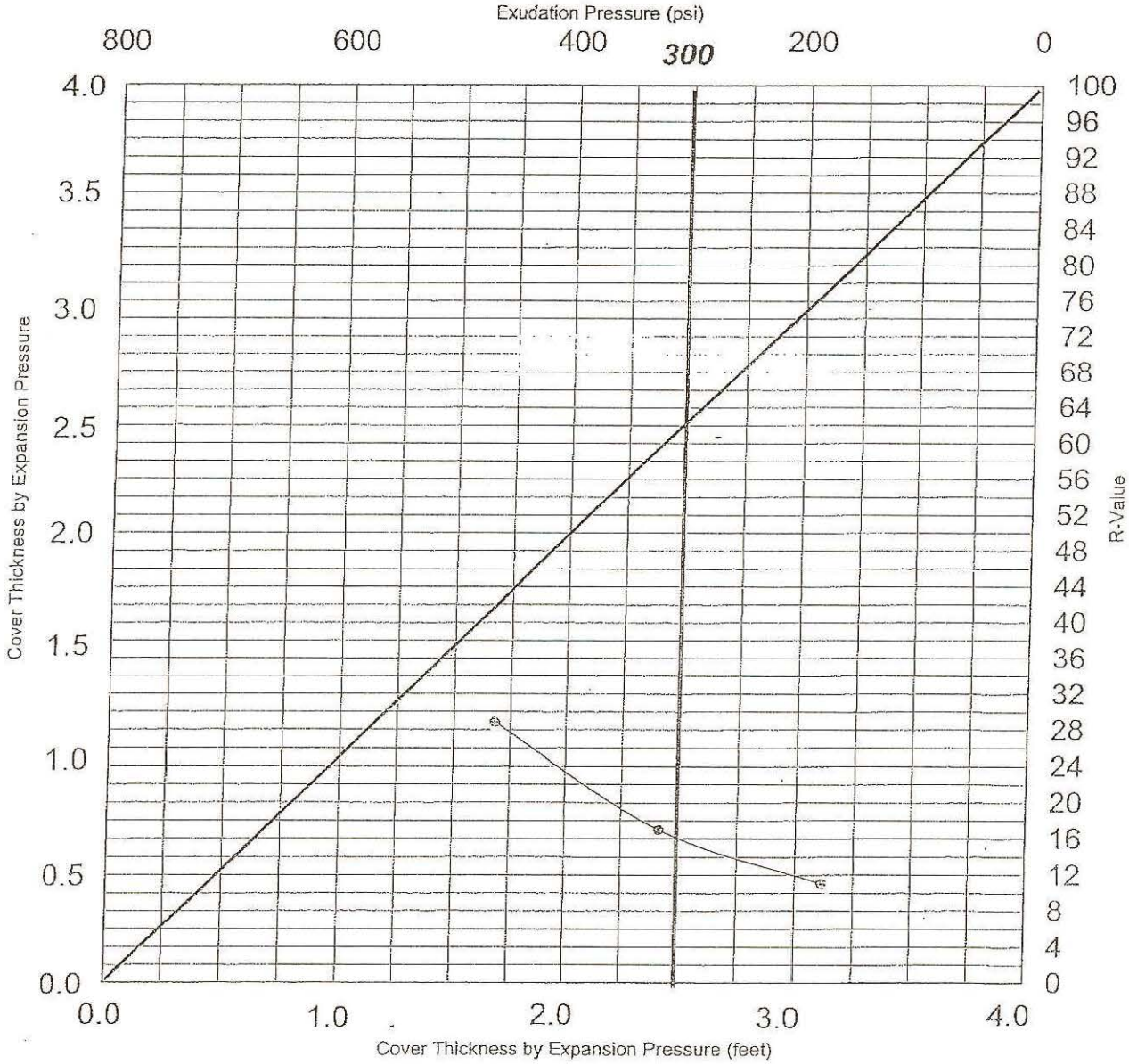
Project Name: _____ Traffic Index: _____

Project No. : _____ R-Value by Exudation: _____

Boring No. : _____ R-Value by Expansion: _____

Sample No. : _____

Location: _____



AP Engineering and Testing, Inc.

Geotechnical Testing Laboratory

RECEIVED SEP 0 8 2005

CORROSION TEST RESULTS

Client Name: Eberhart/United Consultants
Project Name: Brookfield Homes – HANNA
Project No.: ~~81-0204-0011~~ 81-0208-0011

AP Job No.: 25-0880
Date 08/27/05

Boring No.	Sample No.	Depth (ft)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
TP-3	-	1-2	SM	NR	NR	22	NR
TP-3	-	4-5	ML	NR	NR	40	NR
TP-19	-	3-5	ML	NR	NR	23	NR

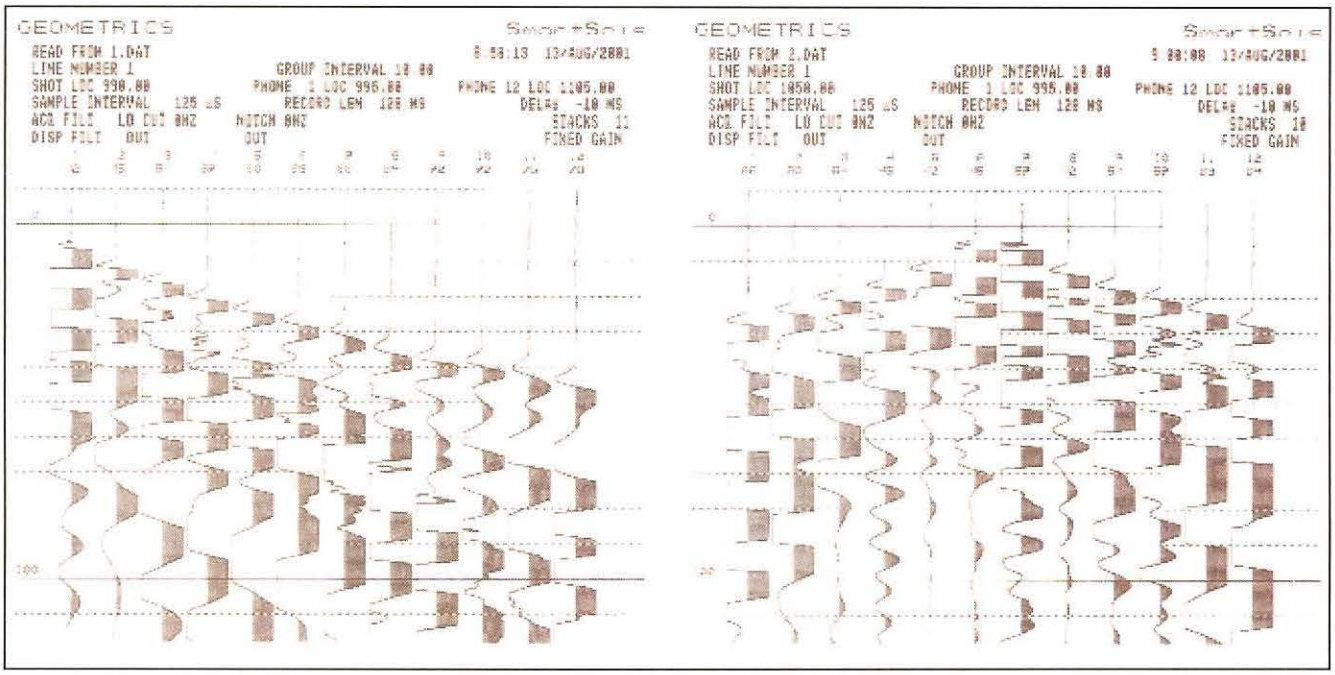
NOTES: Resistivity Test and pH: California Test Methods 532 and 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

2607 Pomona Boulevard, Pomona, CA 91768
Tel. (909) 869-6316 Fax. (909)869-6318

APPENDIX E

SEISMIC REFRACTION DATA

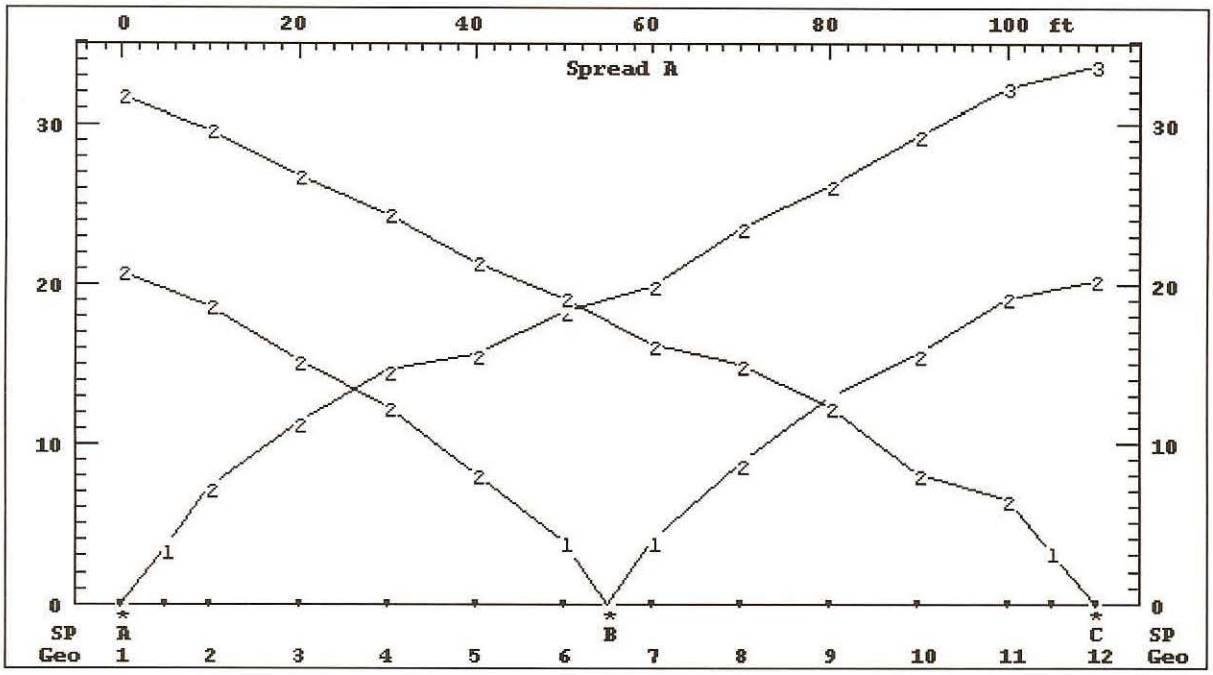
Example
Raw Seismic Data



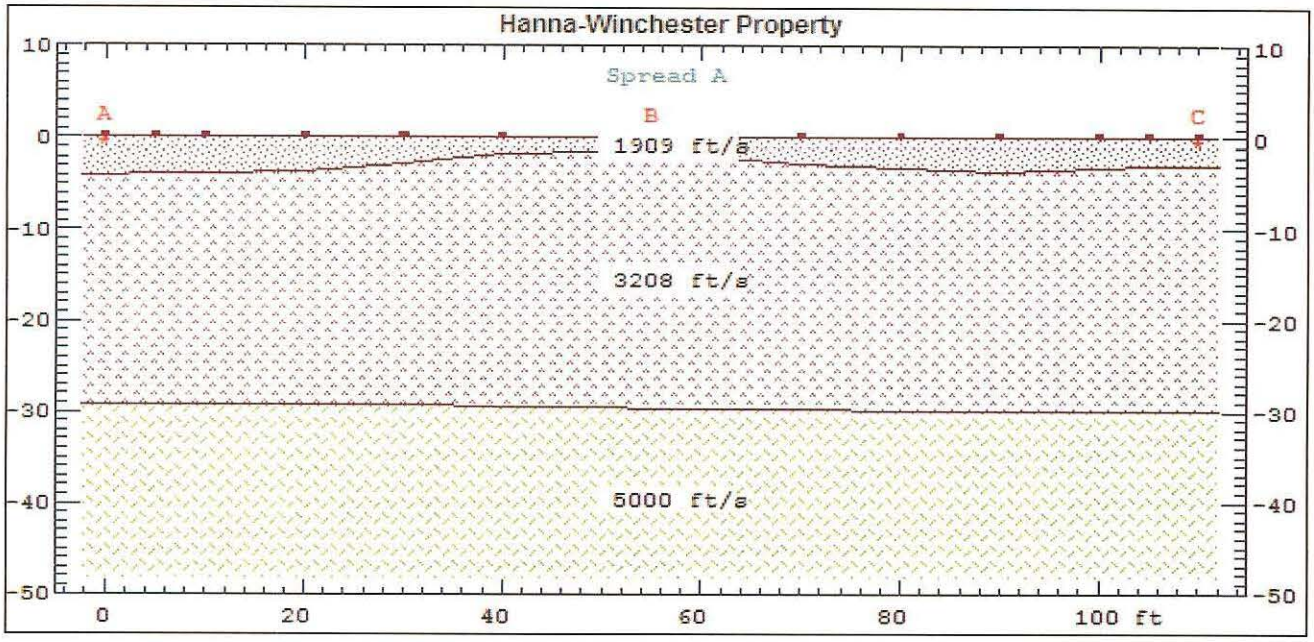
Example

Seismic Line

Time Distance Curves



Seismic Line
ST-1



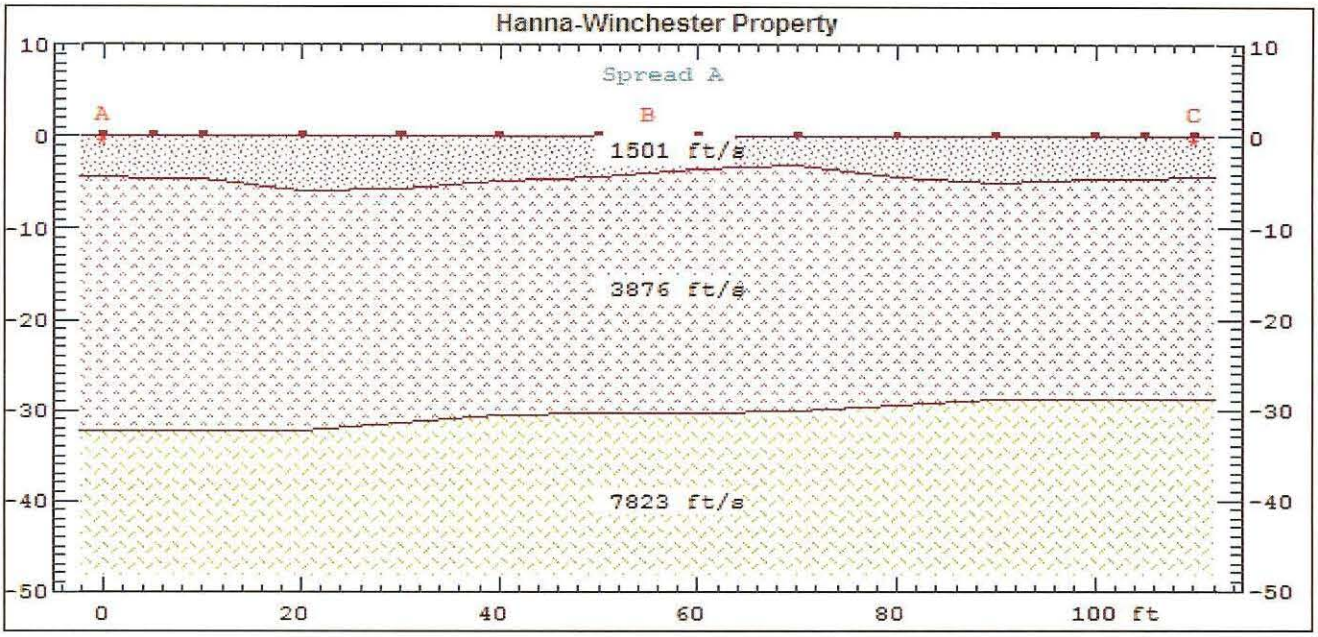
Northwest

Southeast

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 3

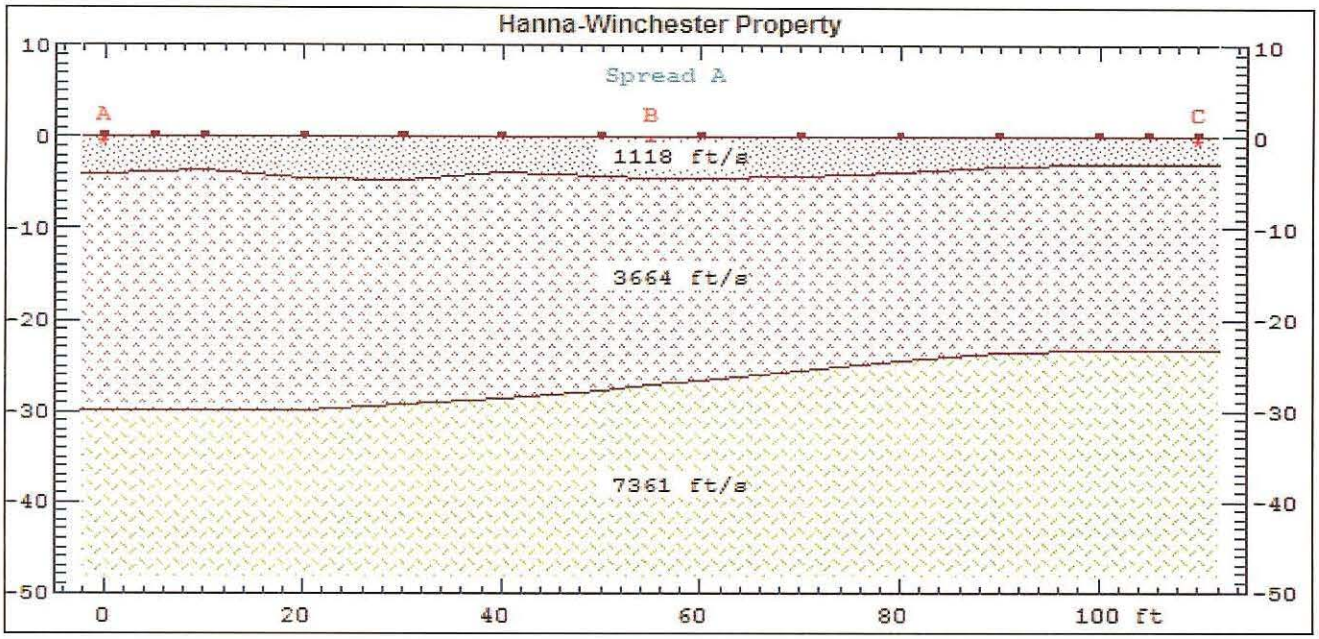
Seismic Line
ST-2



Southeast

Northwest

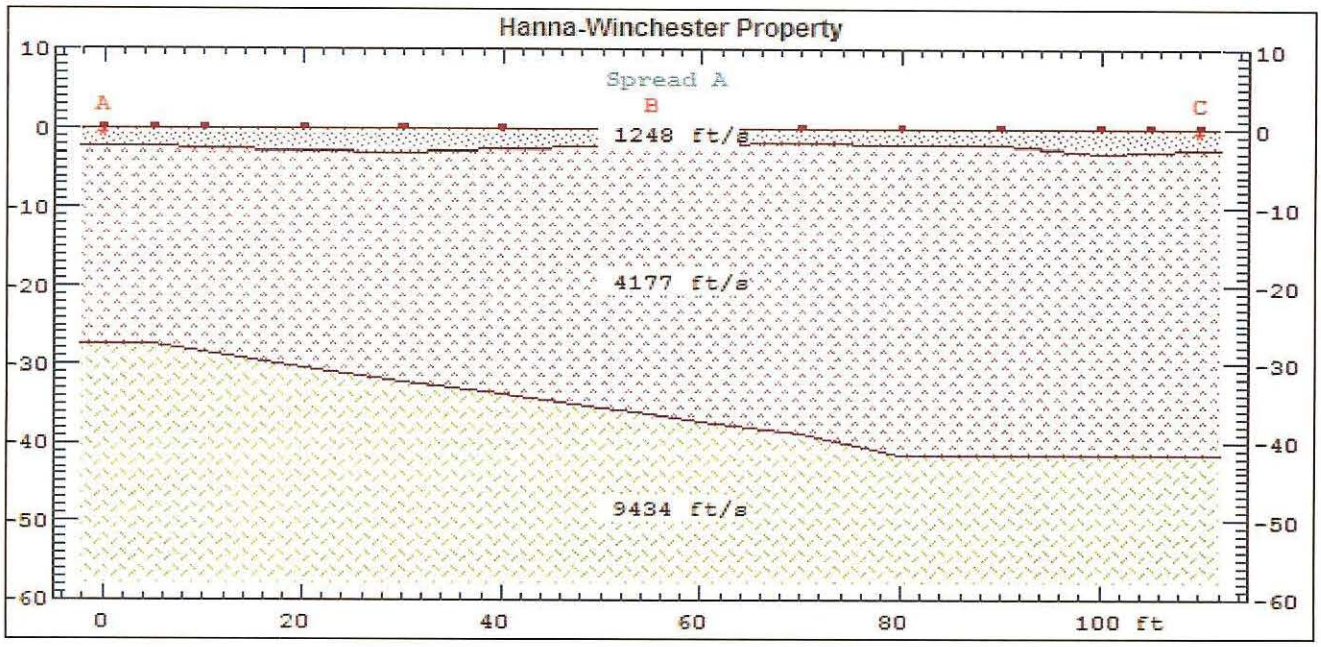
Seismic Line
ST-3



Southeast

Northwest

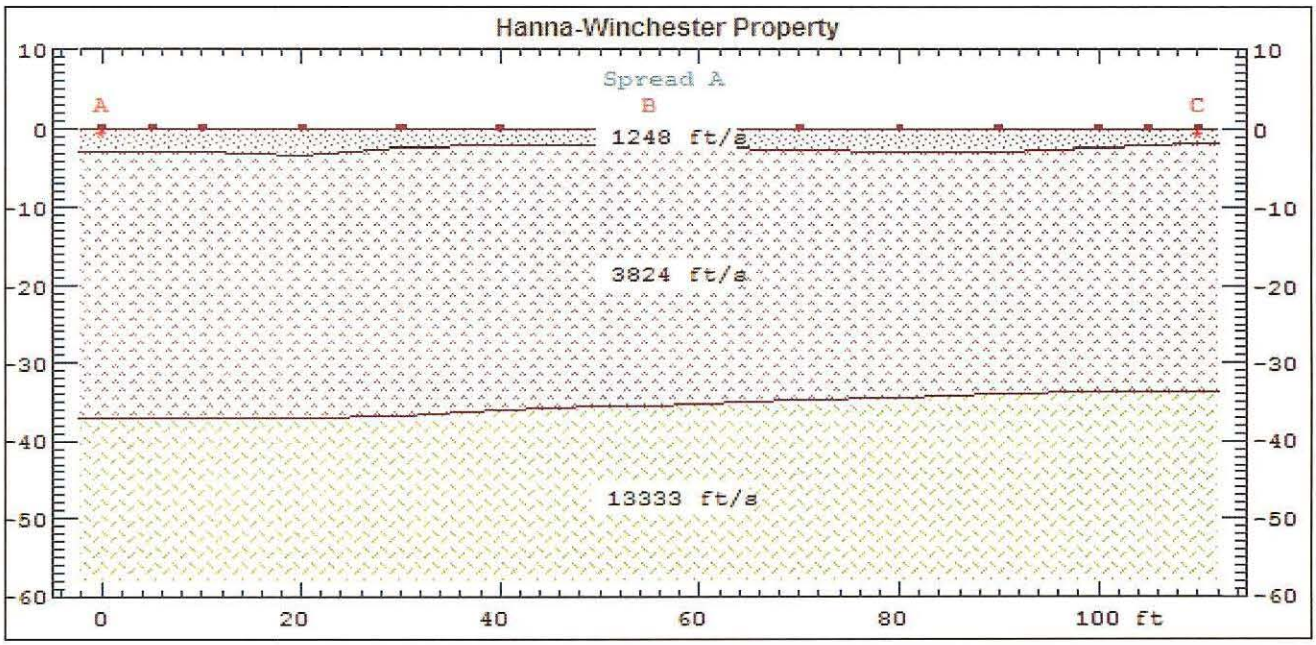
Seismic Line
ST-4



Northwest

Southeast

Seismic Line
ST-5



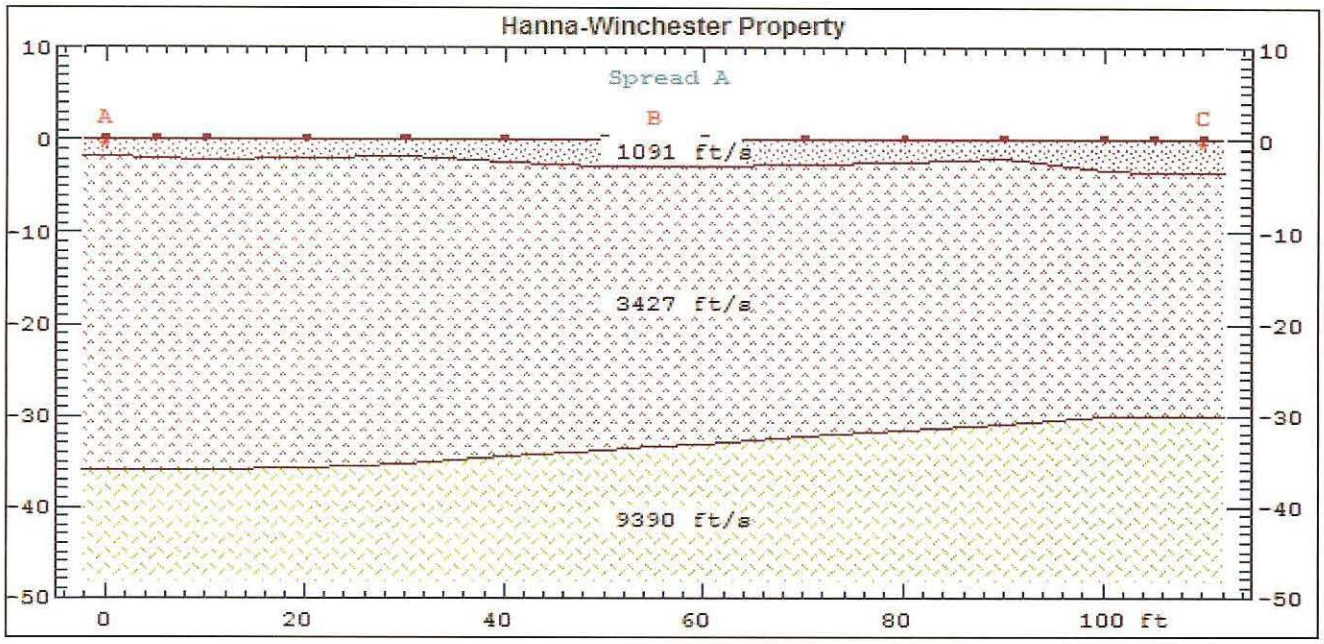
North

South

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 7

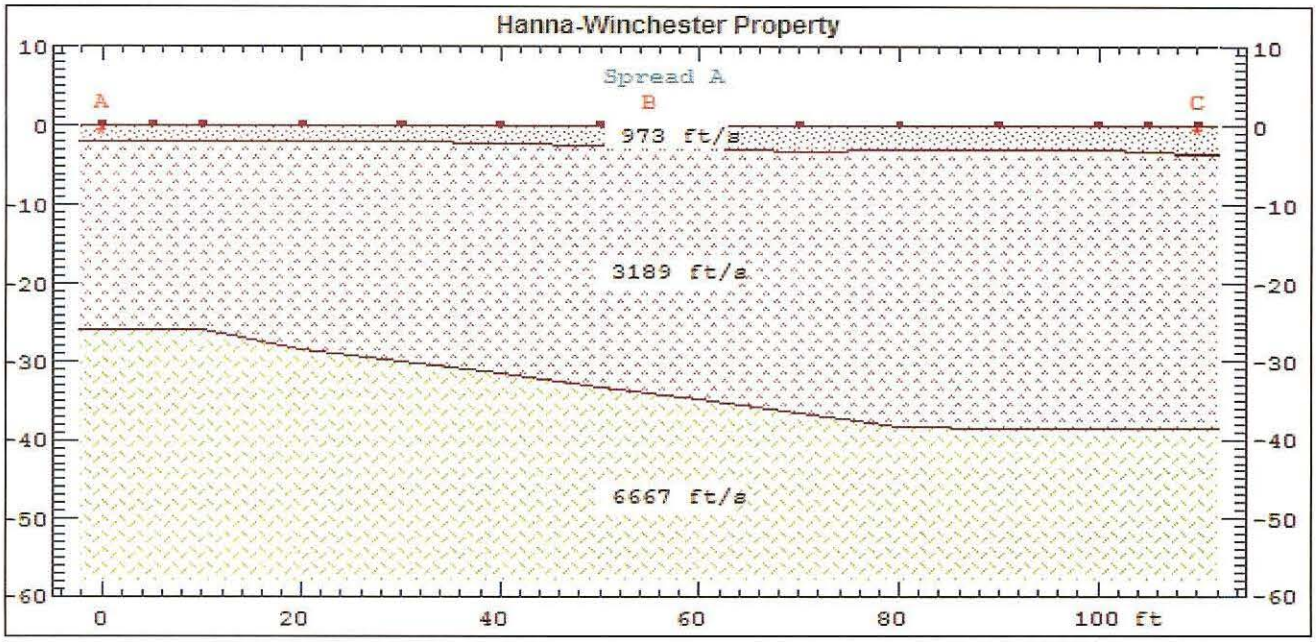
Seismic Line
ST-6



North

South

Seismic Line
ST-7



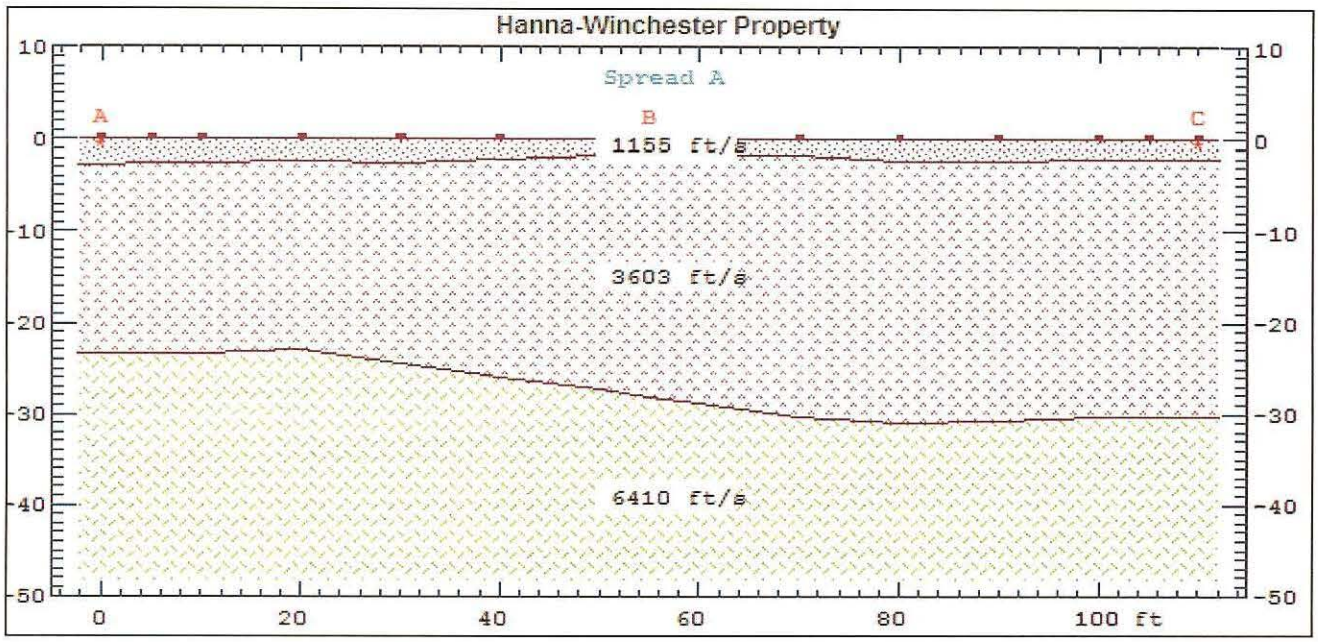
Northwest

Southeast

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 9

Seismic Line
ST-8



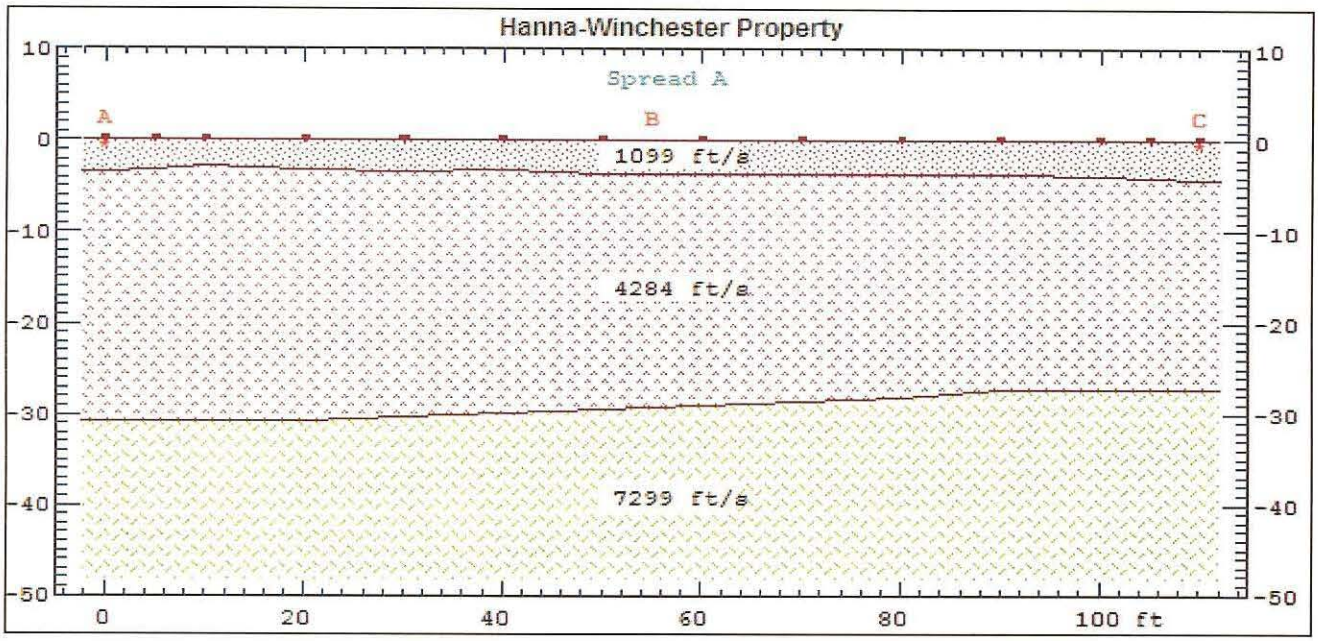
South

North

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 10

Seismic Line
ST-9

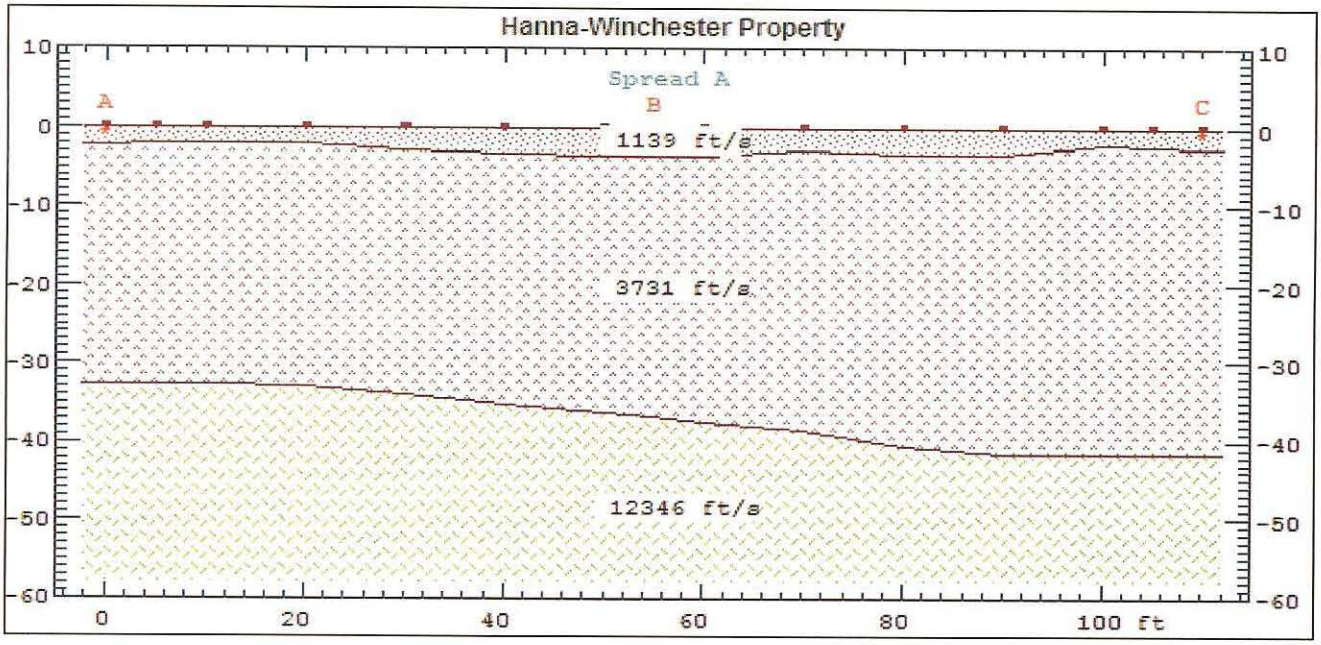


West East

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 11

Seismic Line
ST-10



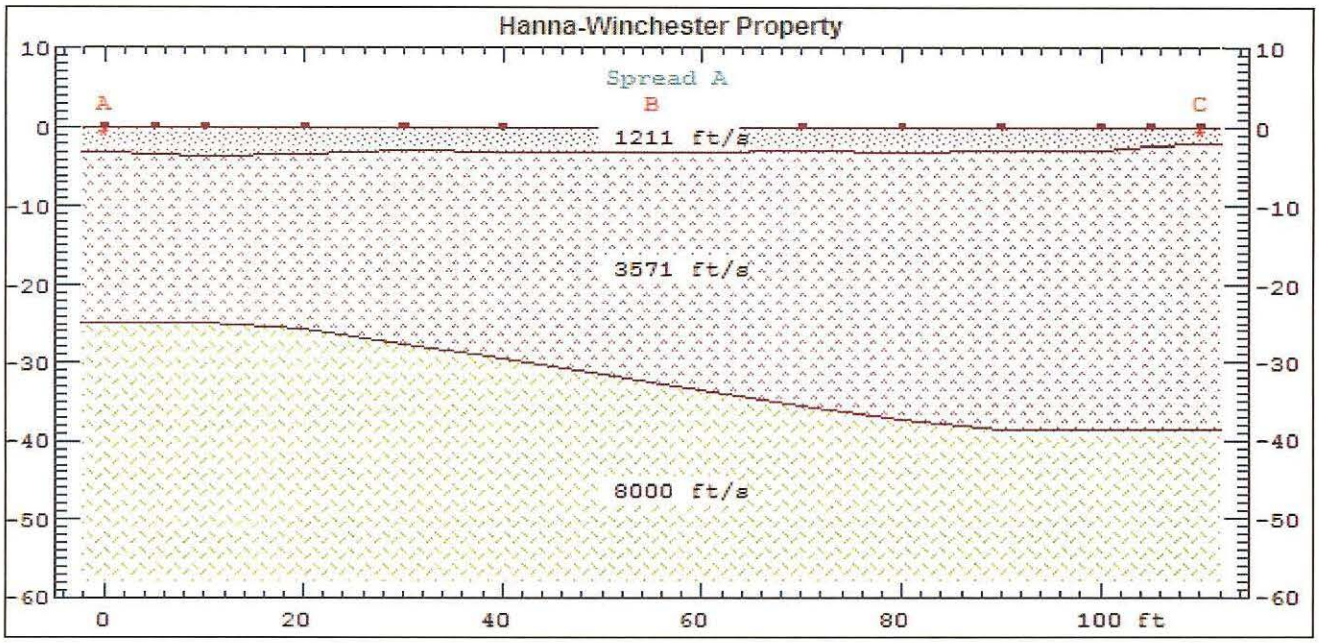
Southeast

Northwest

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 12

Seismic Line
ST-11



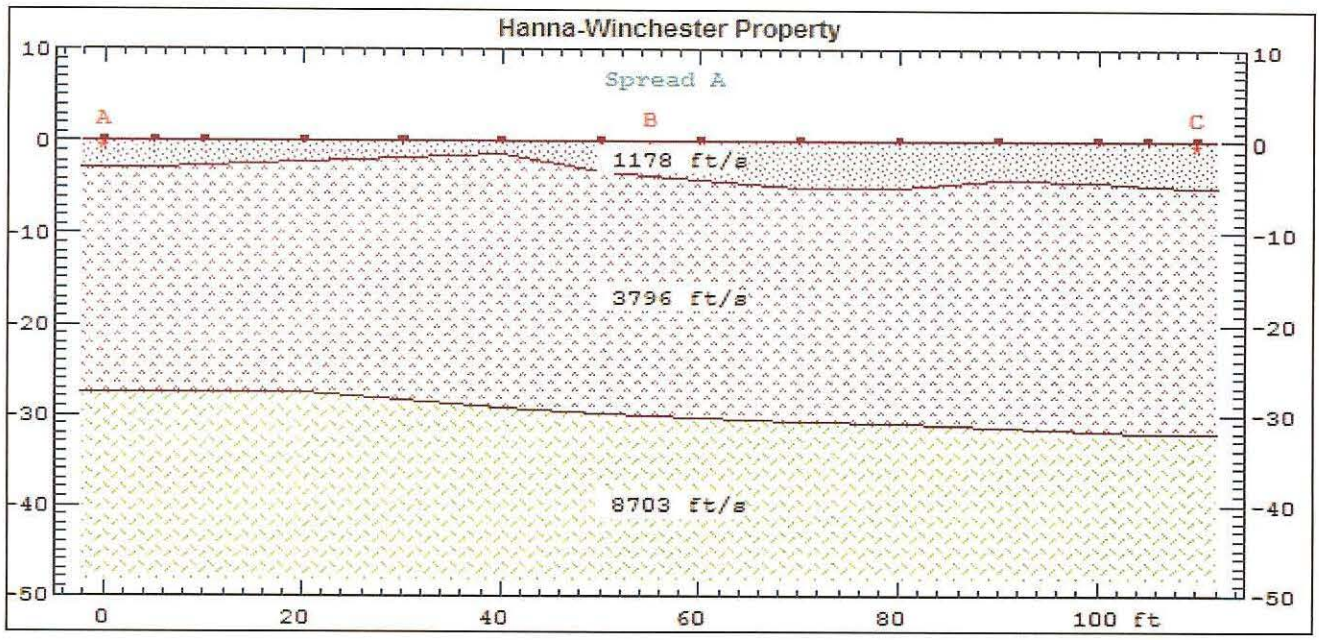
North

South

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 13

Seismic Line
ST-12



East

West

Hanna Marital Trust
Keller Crossing, Riverside County, California

W.O. 5815-A-OC
Figure E - 14

APPENDIX F

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these 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. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), and/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(s), the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that an evaluation 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 geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant 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 geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted codes or agency ordinances, geotechnical report(s), 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, as evaluated by the geotechnical consultant 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 geotechnical consultant.

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 geotechnical consultant. 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 geotechnical consultant 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 (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. 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 geotechnical consultant. 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, mounds, 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 geotechnical consultant. 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 geotechnical consultant, the minimum width of fill keys should be 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 geotechnical consultant 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 evaluated to be suitable by the geotechnical consultant. 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 geotechnical consultant. 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 geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, 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 geotechnical consultant to evaluate it's physical properties and suitability for use onsite. Such testing

should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant 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 geotechnical consultant 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 evaluated by ASTM test designation D-1557, or as otherwise recommended by the geotechnical consultant. 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 geotechnical consultant.

In general, per the 1997 UBC and/or latest adopted version of the California Building Code (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 evaluation 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.

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 geotechnical consultant 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/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, 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 geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant 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 geotechnical consultant 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 geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), 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 geotechnical consultant.

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 geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

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.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per code, and geometry of the proposed

improvements. Recommendations for pools/spas and/or deck flatwork underlain by expansive soils, or for areas with differential settlement greater than ¼-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below.

The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all homeowners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

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

5. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.

6. All pool/spa walls should be designed as "free standing" and be capable of supporting the water in the pool/spa without soil support. The shape of pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with Section 1806.5 of the 1997 UBC. Minimally, the bottoms of the pools/spas, should maintain a distance H/3, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.

7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading and/or re-grading of the pool/spa bottom, and attendant potential for shoring and/or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.

10. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.

11. An elastic expansion joint (flexible waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.

13. In order to reduce unsightly cracking, deck slabs should minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening and/or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of Class 2 base compacted to 90 percent. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable,

improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, or properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be throughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.

16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete utilized should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.

17. Joint and sawcut locations for the pool/spa deck should be determined by the design engineer and/or contractor. However, spacings should not exceed 6 feet on center.

18. 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), should be anticipated. All excavations should be observed by a representative of the geotechnical consultant, including the project geologist and/or geotechnical engineer, prior to workers entering the excavation or trench, and minimally conform to Cal/OSHA ("Type C" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.

19. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.

21. For pools/spas built within (all or part) of the 1997 Uniform Building Code (UBC) setback and/or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the affects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited "cap soils" and built within 1997 UBC setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:

    OPTION A: Shallow foundations with or without overexcavation of the pool/spa "shell," such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater that 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.

    OPTION B: Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations, local code, and the 1997 UBC. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa "leak-free."

22. The temperature of the water lines for spas and pools may affect the corrosion properties of site soils, thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.

23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).

24. Pool and spa utility lines should not cross the primary structure's utility lines (i.e., not stacked, or sharing of trenches, etc.).

25. The pool/spa or associated utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.

27. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, prior to the placement of any reinforcement or pouring of any concrete.

28. Any changes in design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.

29. Disclosure should be made to homeowners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and/or H/3, where H is the height of the slope (in feet), <u>will</u> experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

30. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.

31. Local seismicity and/or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.

32. The information and recommendations discussed above should be provided to any contractors and/or subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

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

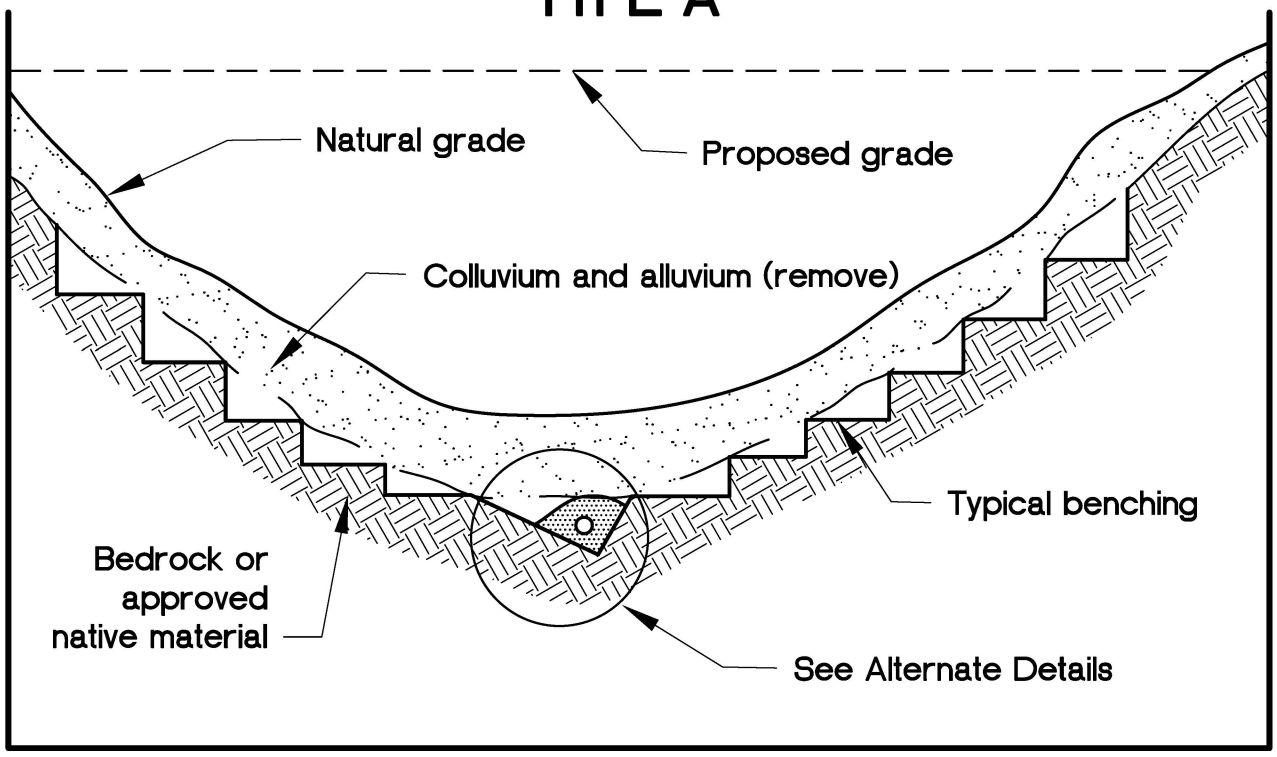
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

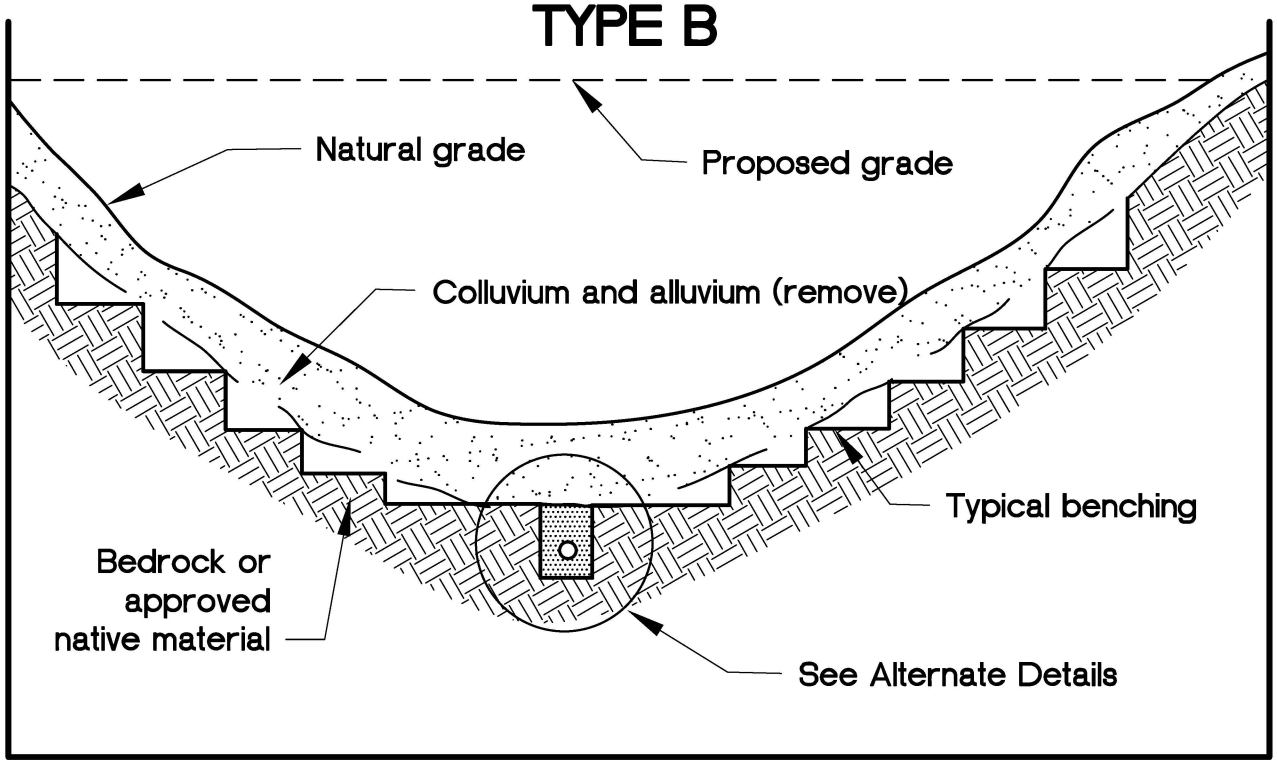
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

TYPE A

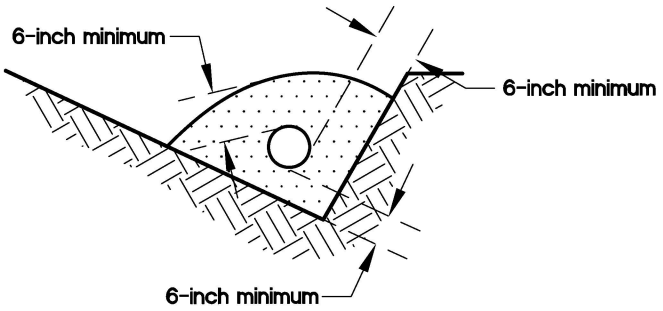


TYPE B

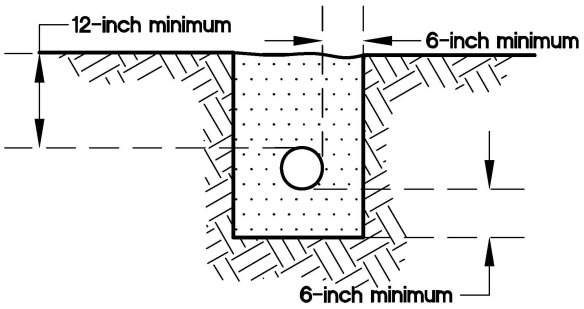


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate F–1



A-1



B-1

Filter material: Minimum volume of 9 cubic feet per lineal foot of pipe.

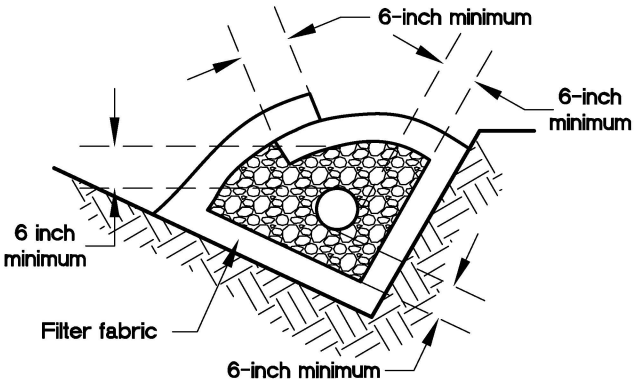
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations (¼-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

For continuous run in excess of 500 feet, use 8-inch-diameter pipe (ASTM D-3034, SDR-35, or ASTM D-1785, Schd. 40).

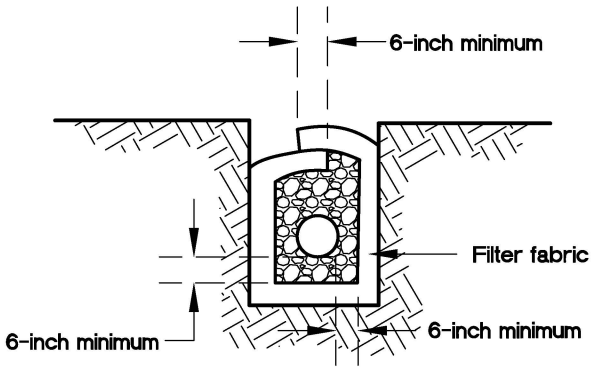
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



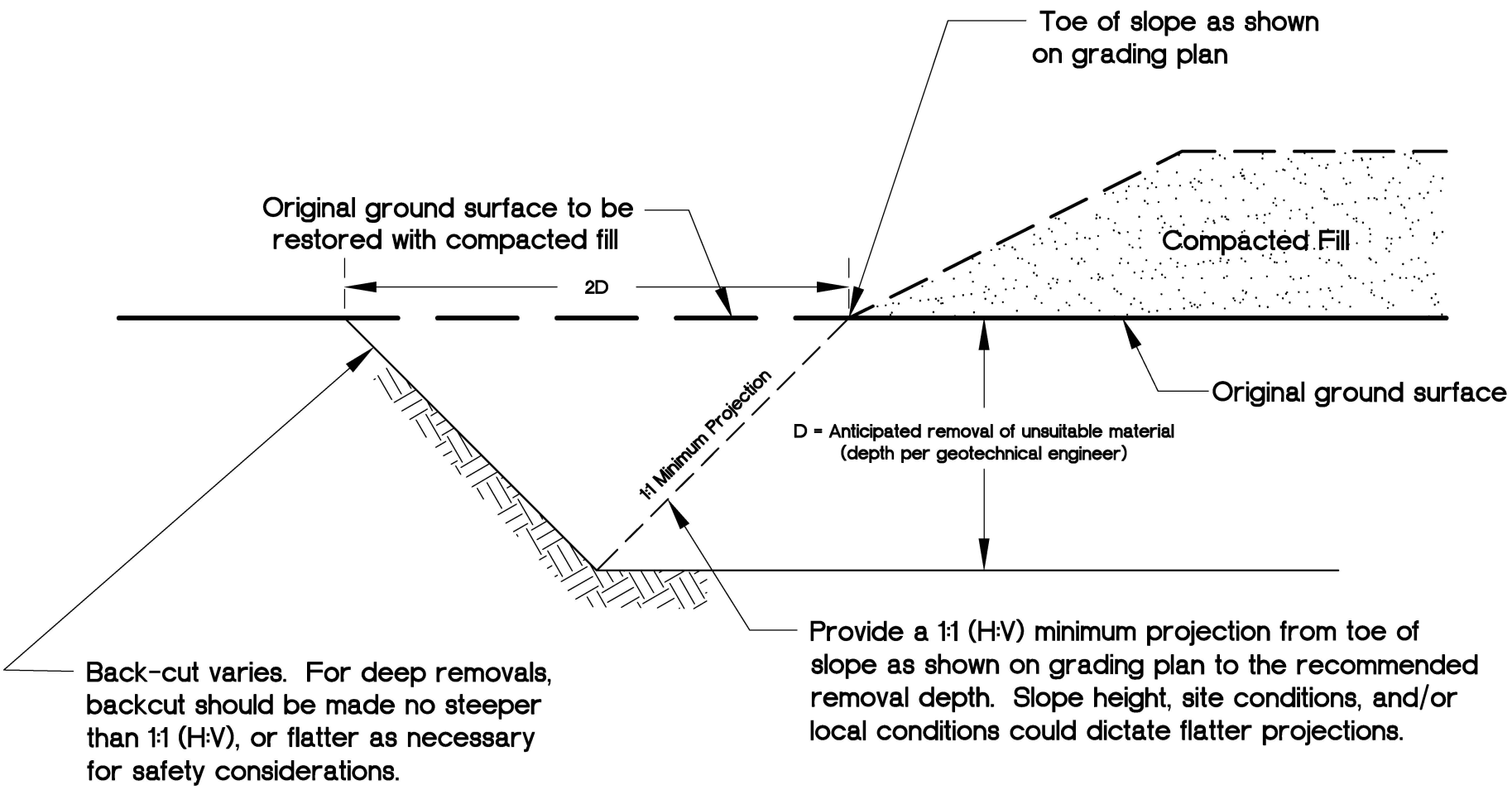
A-2



B-2

Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean ¾-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

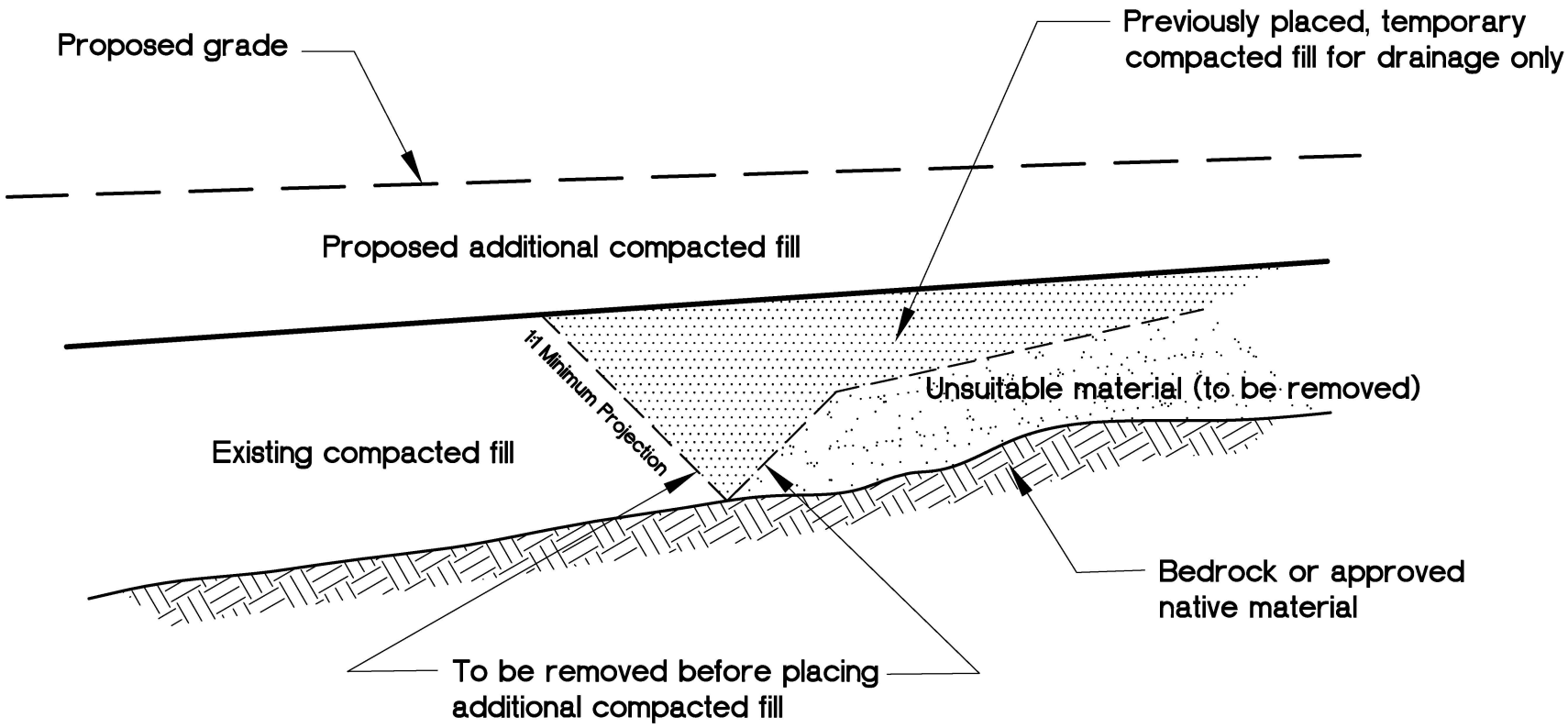
ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

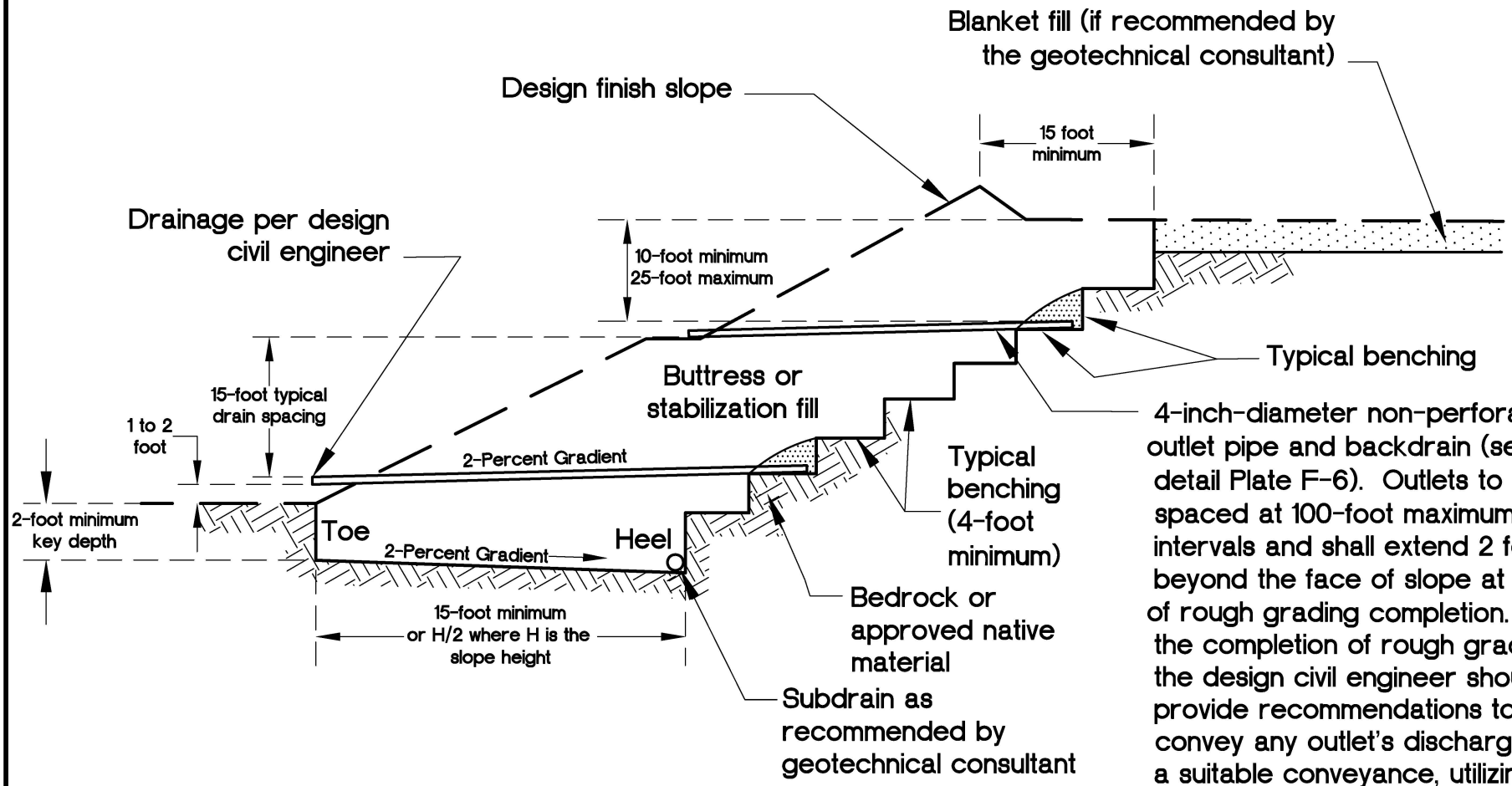
Plate F-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

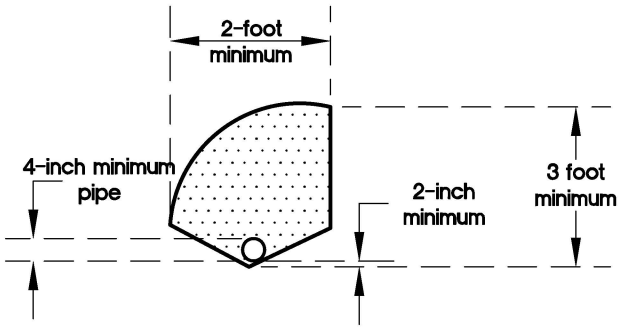
Plate F-4

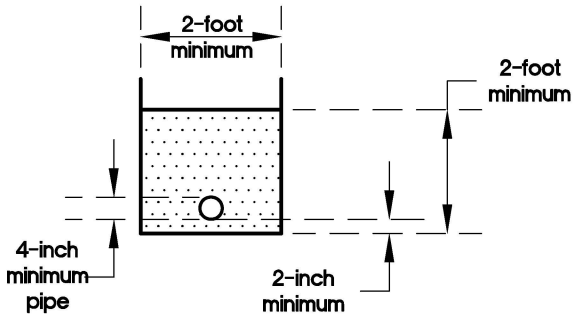


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate F-5





Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal feet of pipe when placed in square cut trench.

Alternative in Lieu of Filter Material: 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 inches in all joints.

Minimum 4-Inch-Diameter Pipe: 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. Must be installed with perforations down at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2 percent to outlet pipe. Outlet pipe to be connected to subdrain pipe with tee or elbow.

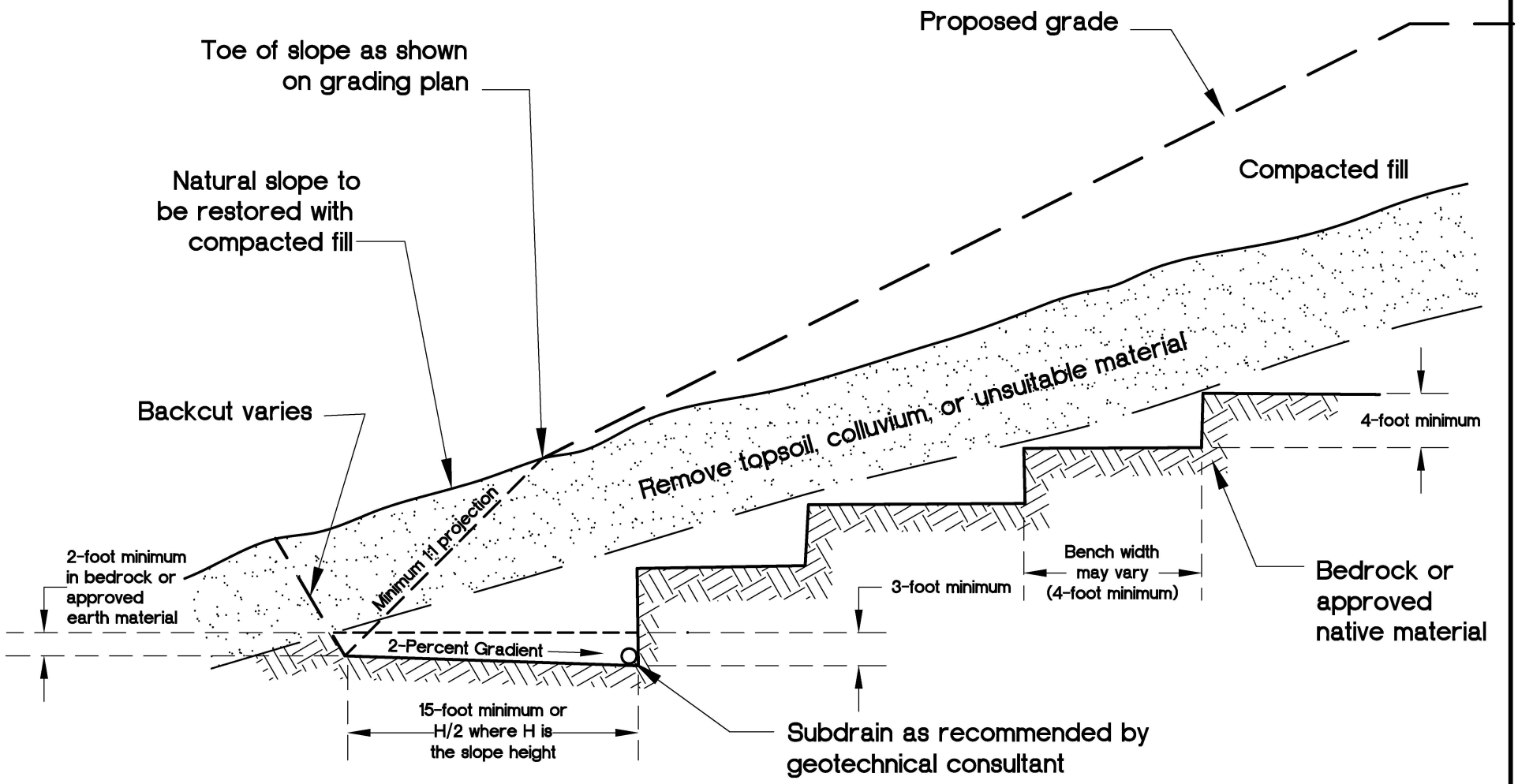
Notes:
1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

Filter Material shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1½ inch	100
No. 4	50
No. 200	8

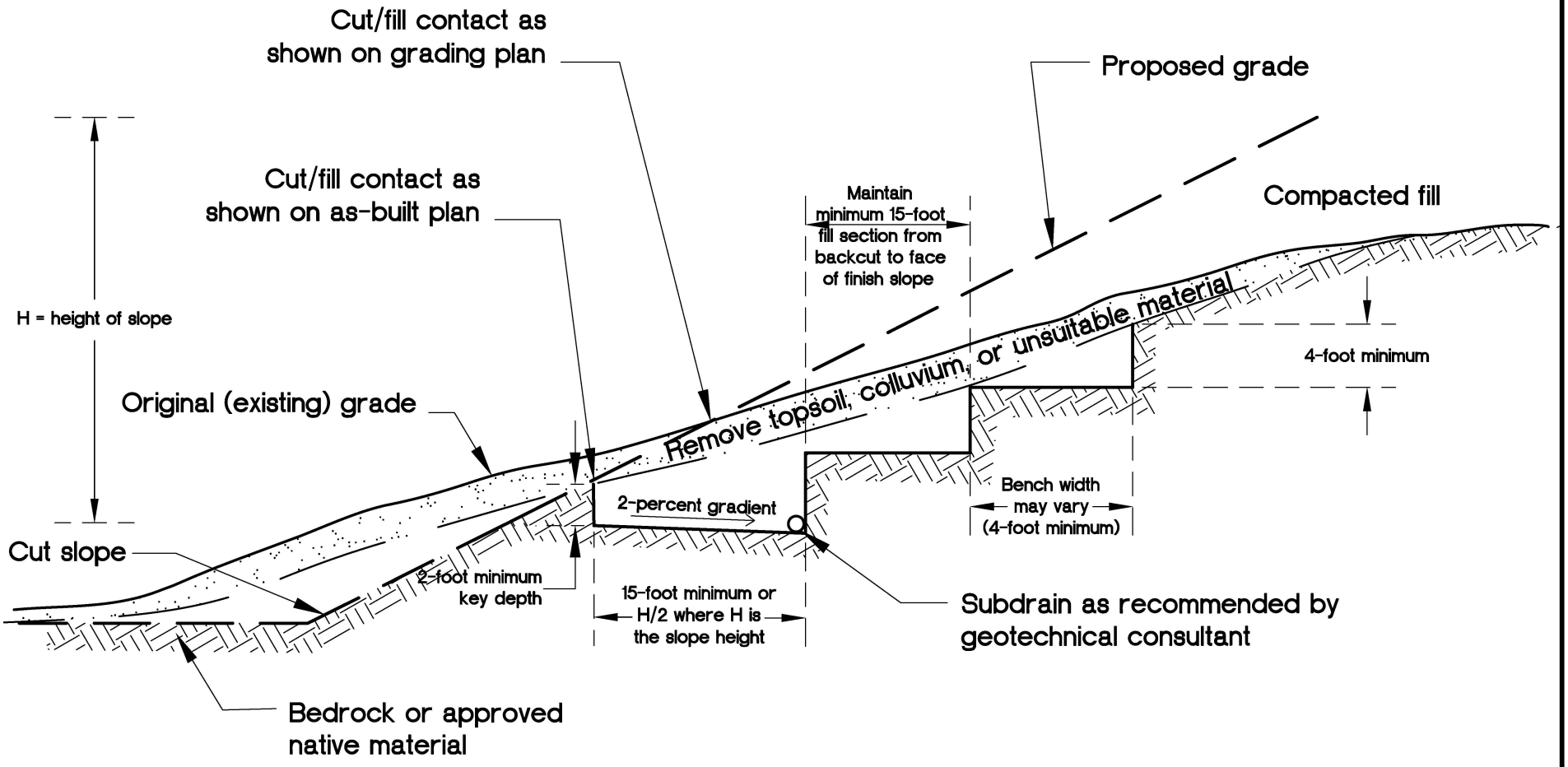


NOTES:

1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.

GeoSoils, Inc.	FILL OVER NATURAL (SIDEHILL FILL) DETAIL	Plate F-7

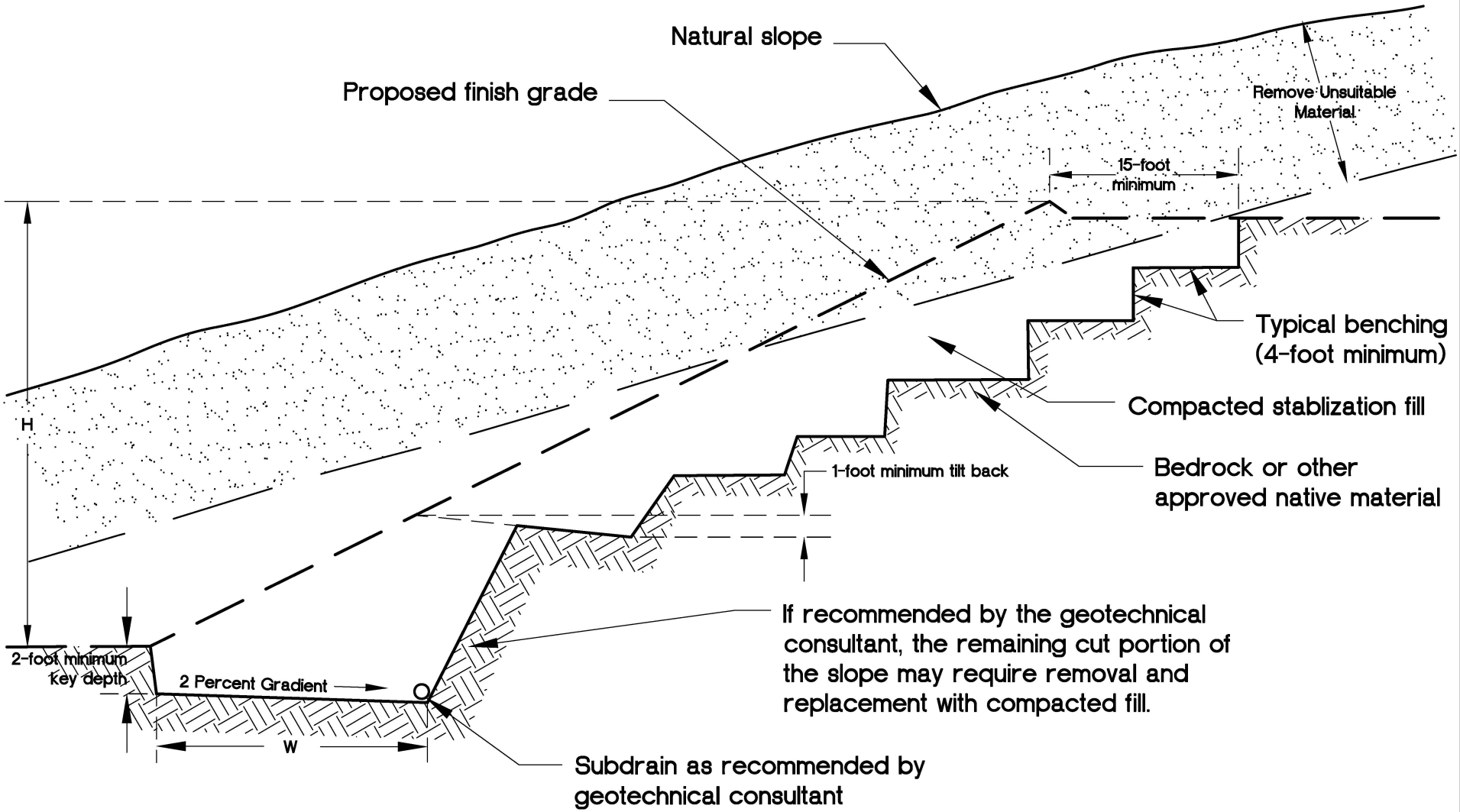


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8

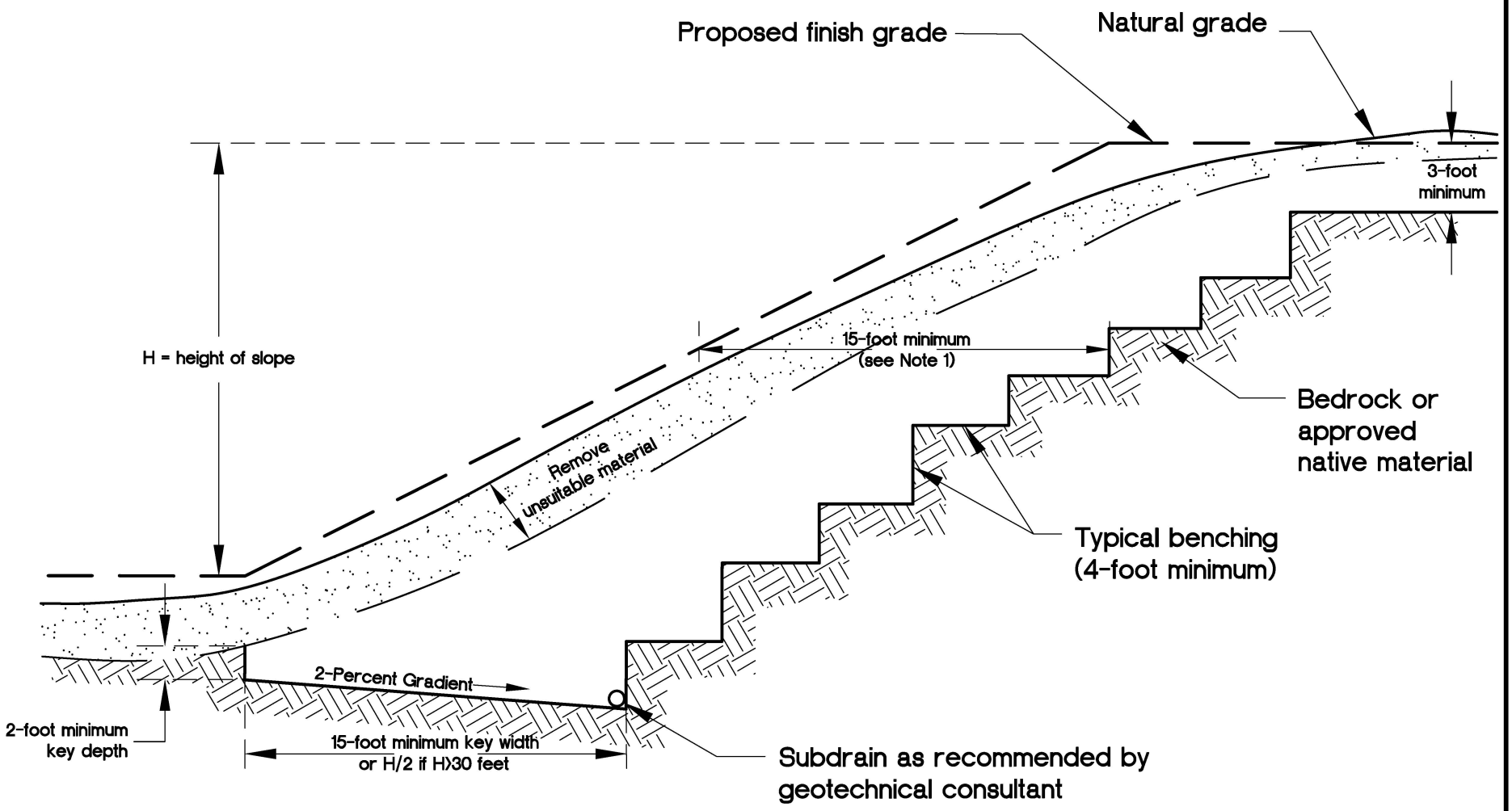


NOTES:

1. Subdrains may be required as specified by the geotechnical consultant.

2. W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

GeoSoils, Inc.	STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL	Plate F-9



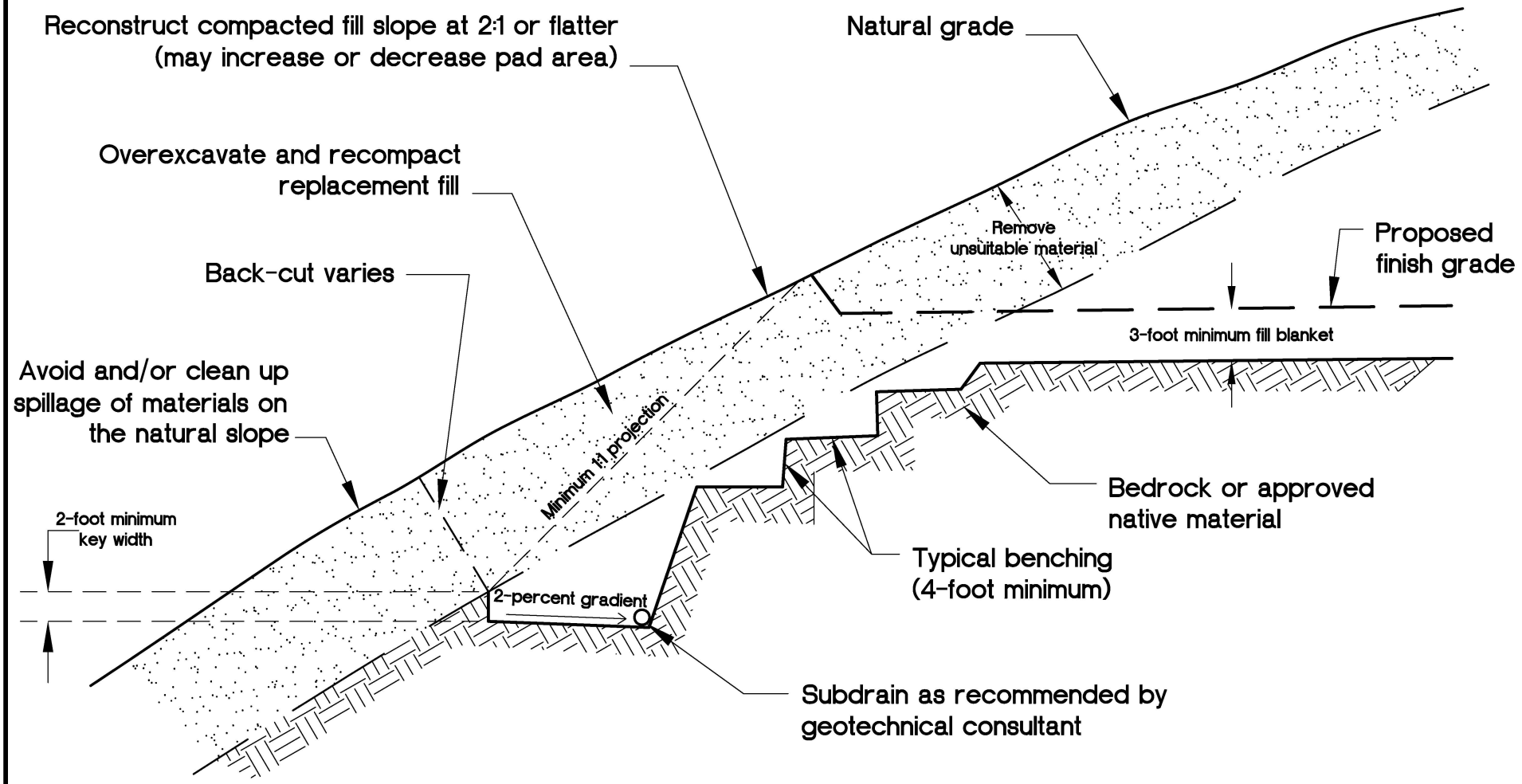
NOTES:

1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical consultant.

GeoSoils, Inc.

SKIN FILL OF NATURAL GROUND DETAIL

Plate F–10



NOTES:

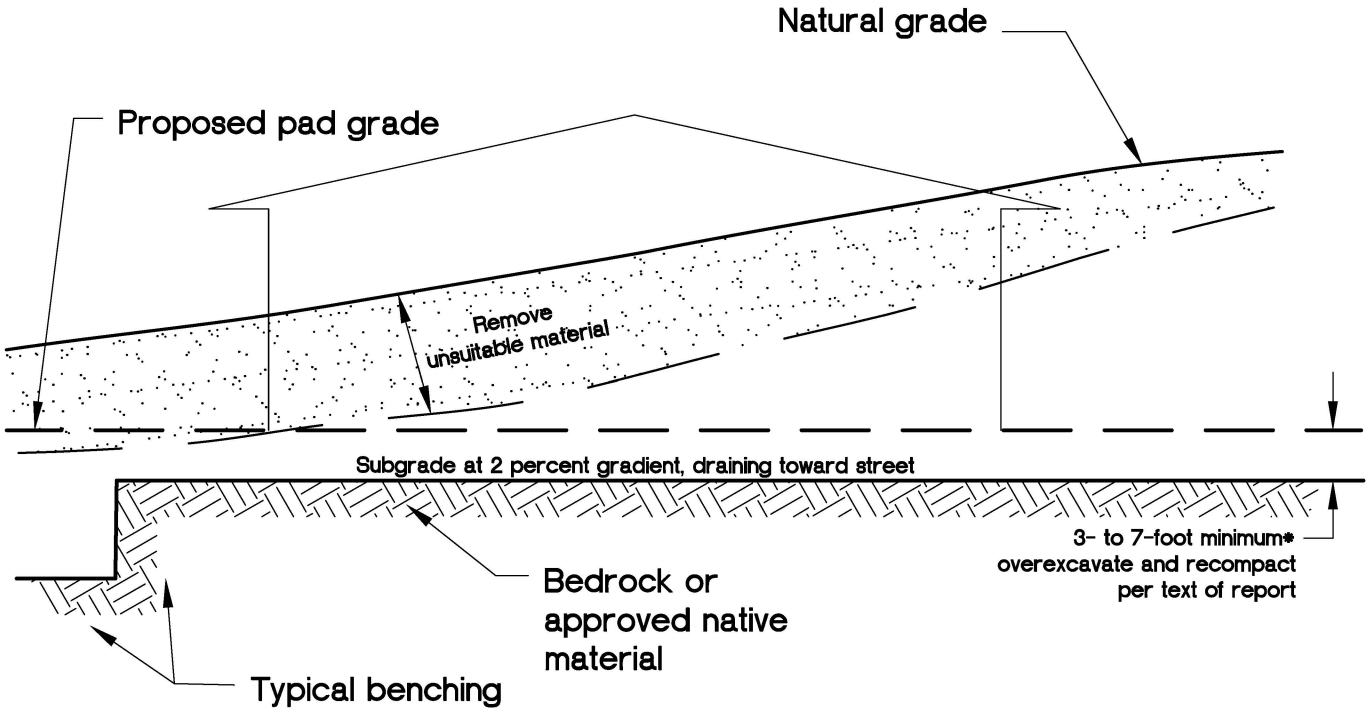
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

2. Pad overexcavation and recompaction should be performed if evaluated necessary by the geotechnical consultant.

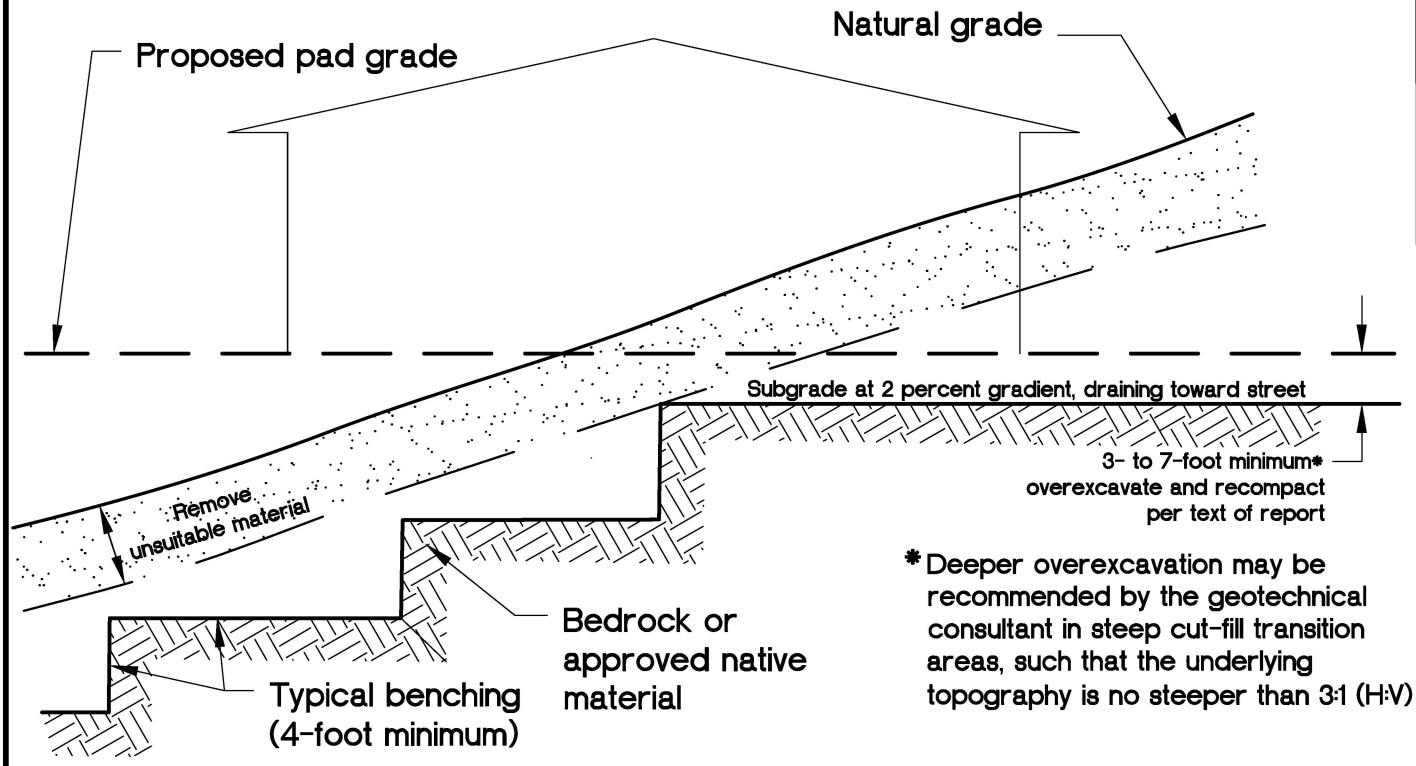
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate F-11



CUT LOT OR MATERIAL-TYPE TRANSITION

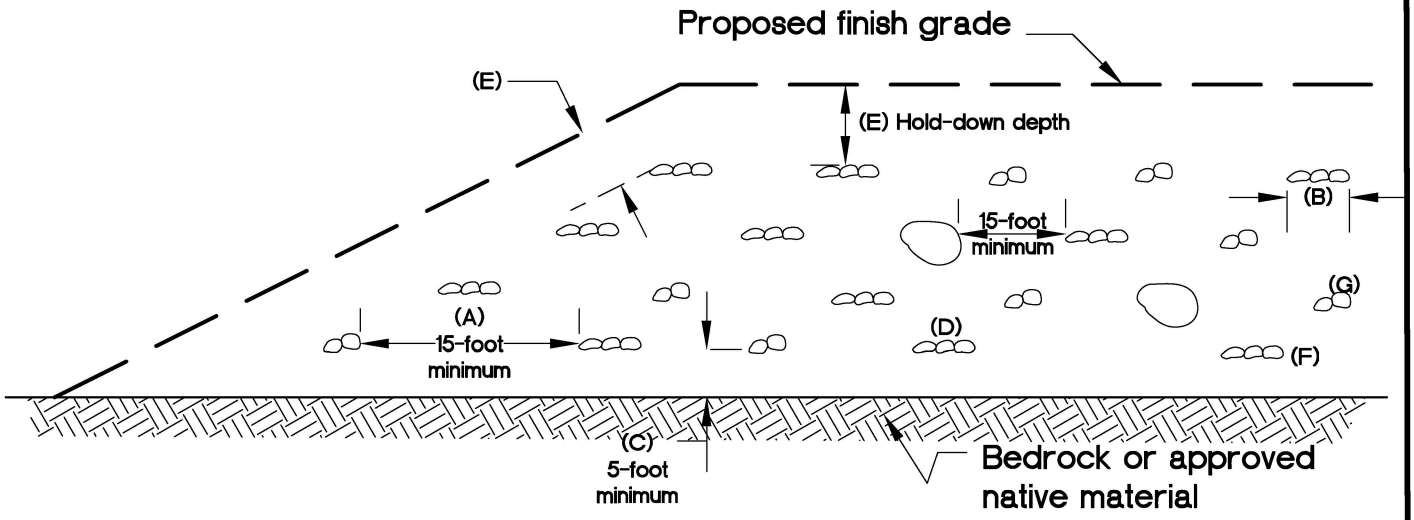


* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

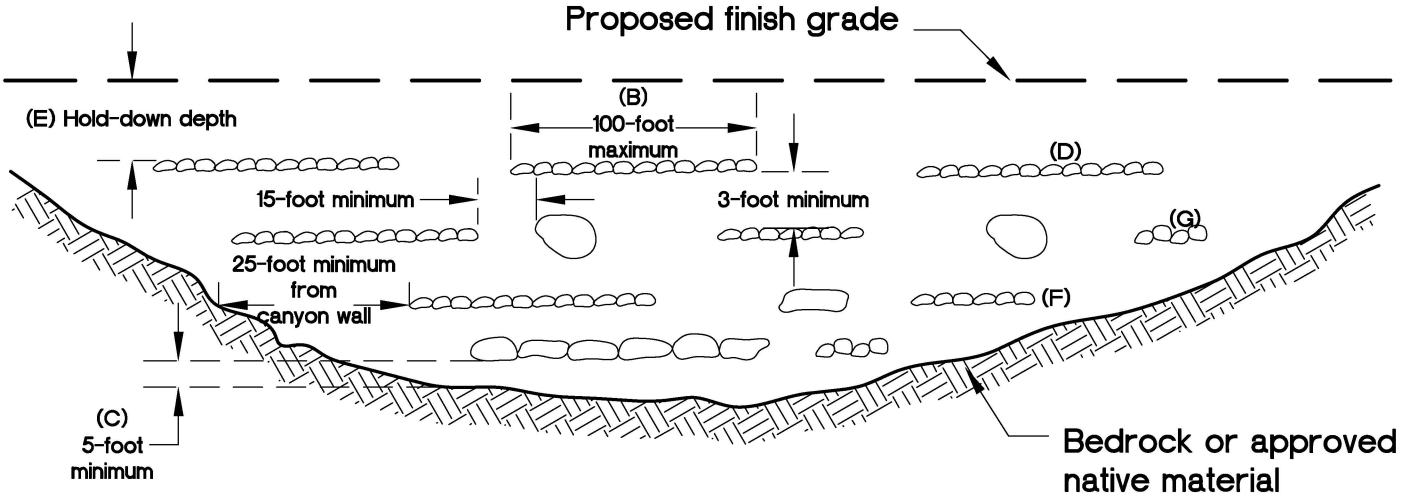
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate F-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).

B. Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.

C. If approved by the geotechnical consultant, 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 geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.

E. Clear area for utility trenches, foundations, and swimming pools: Hold-down depth as specified in text of report, subject to governing agency approval.

F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

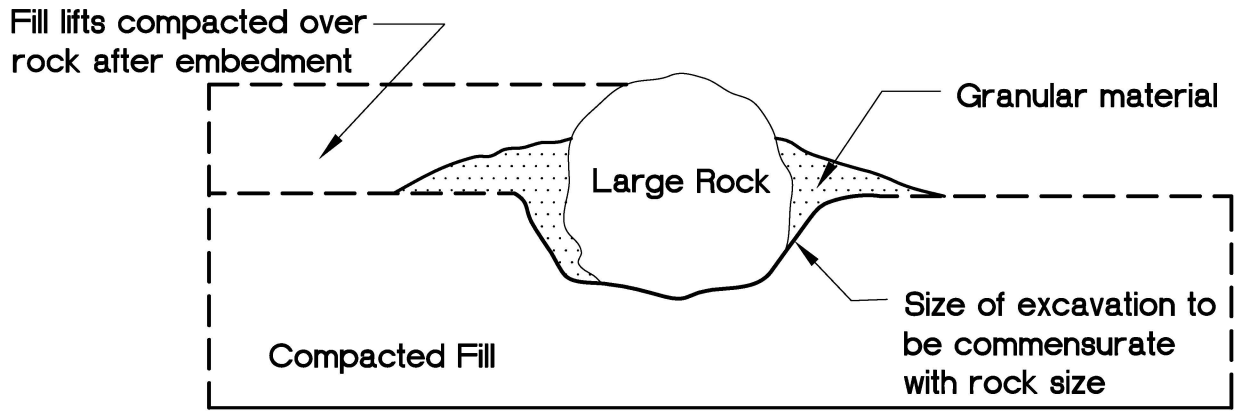
G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE

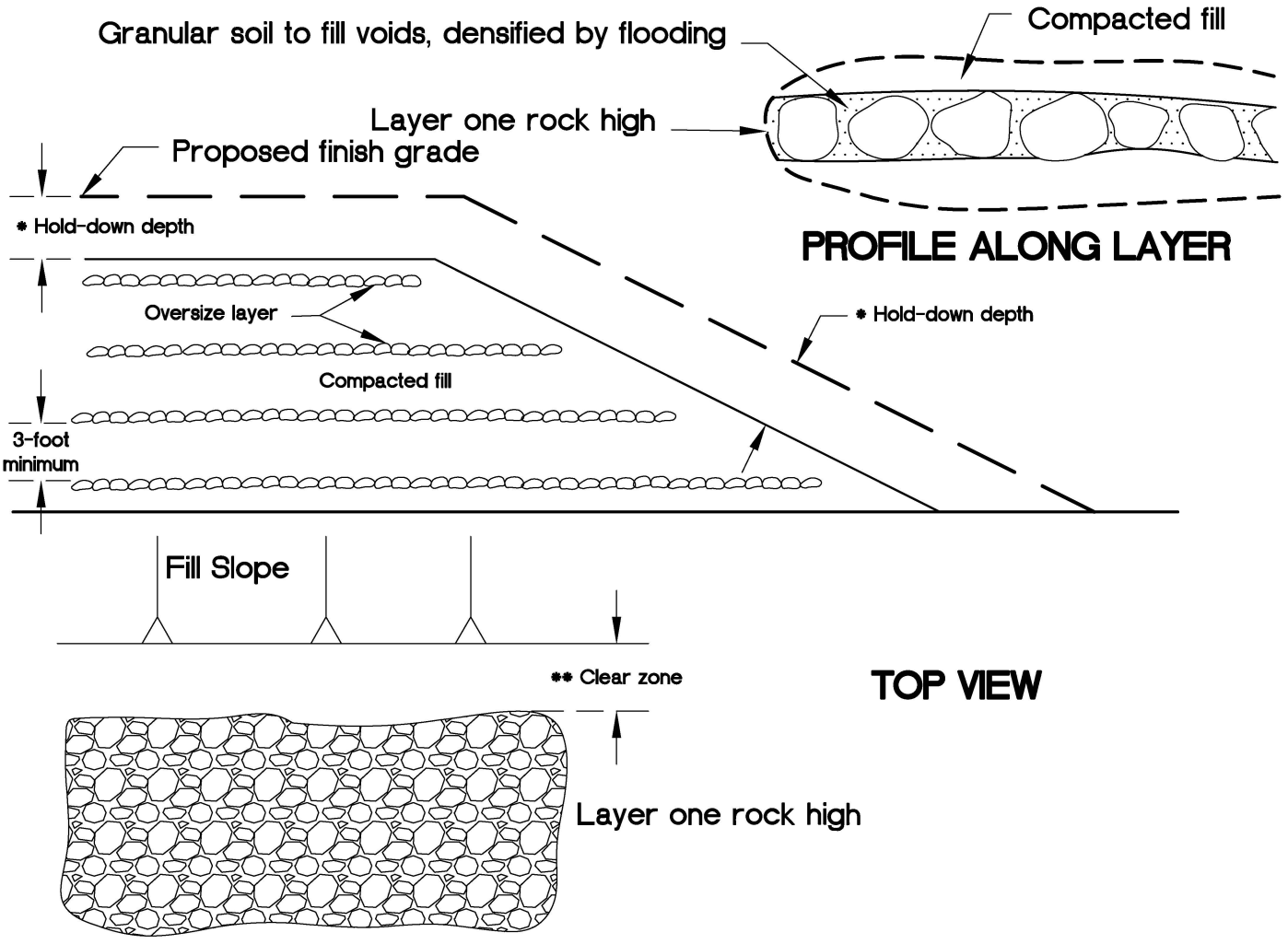
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

GeoSoils, Inc.	OVERSIZE ROCK DISPOSAL DETAIL	Plate F-13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS



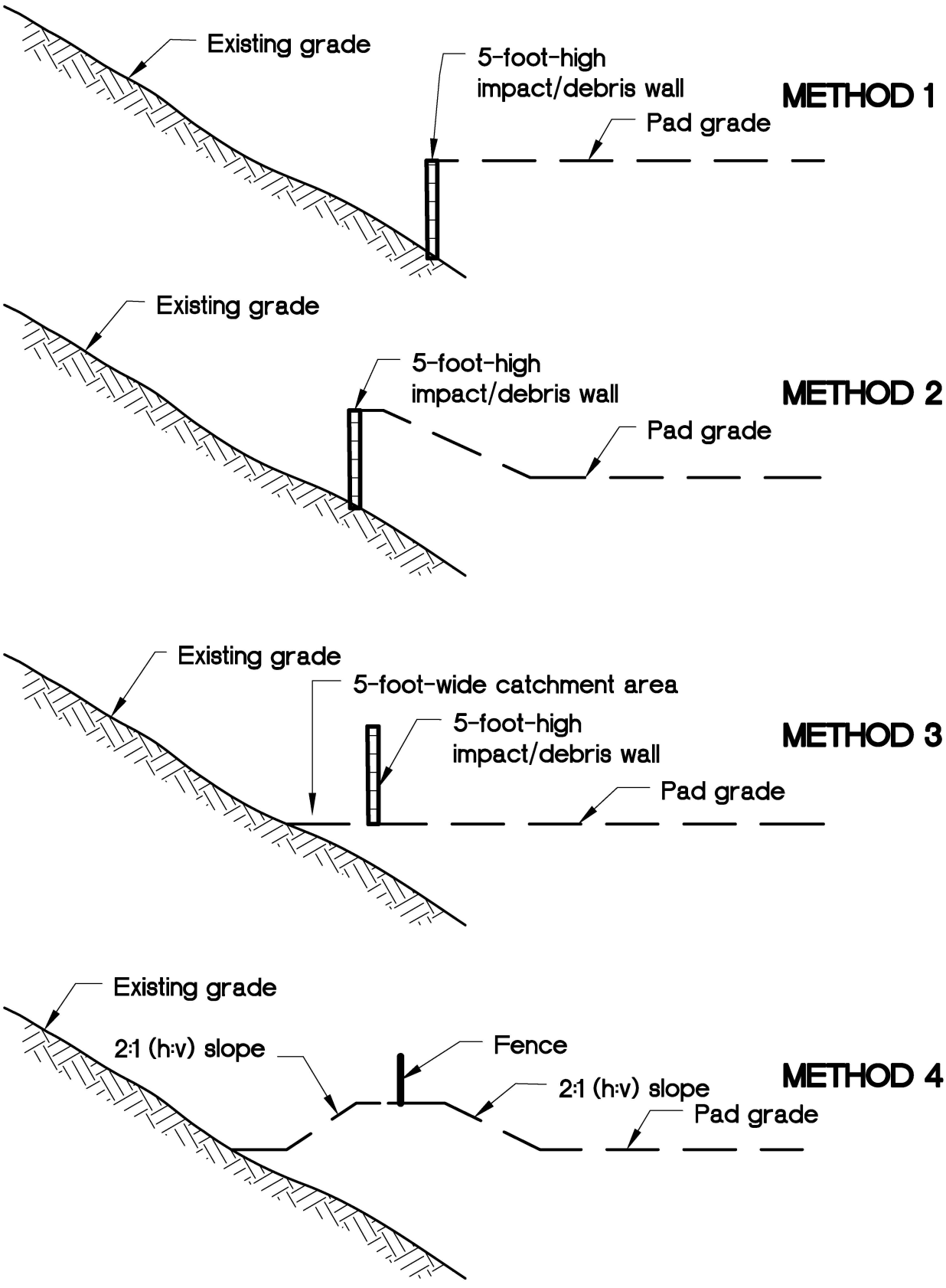
* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE

ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate F-14

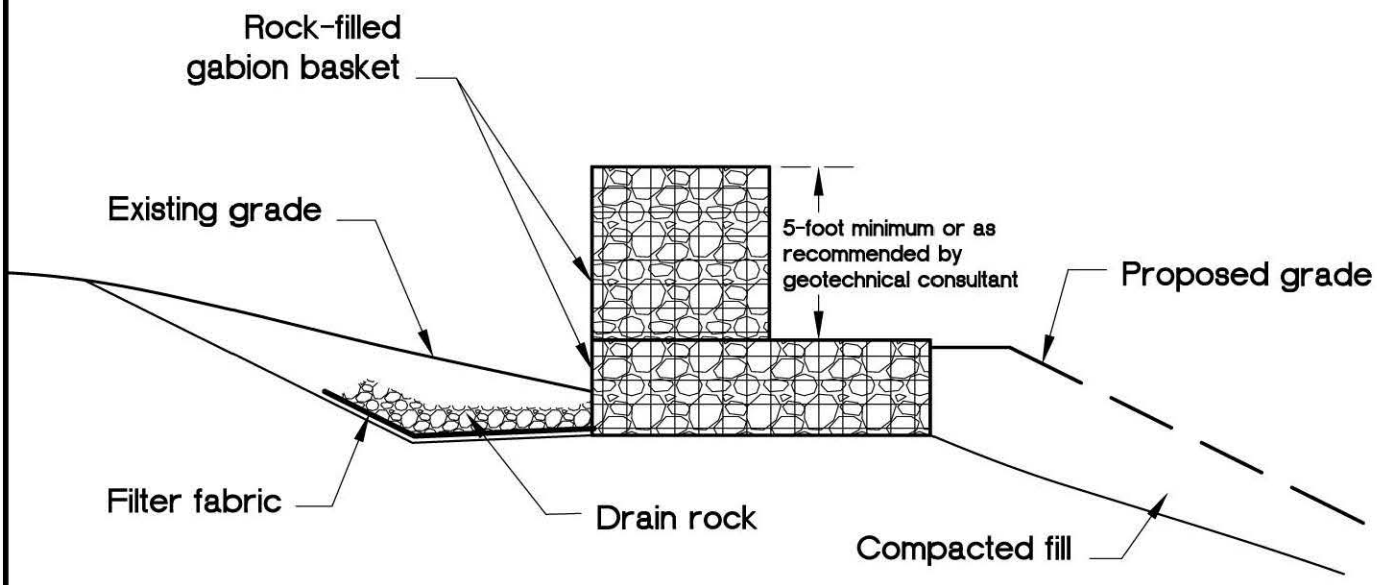


NOT TO SCALE

GeoSoils, Inc.

DEBRIS DEVICE CONTROL METHODS DETAIL

Plate F-15



Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional area for subsequent clean-out. Additional subdrain may be recommended by geotechnical consultant.

From GSA, 1987

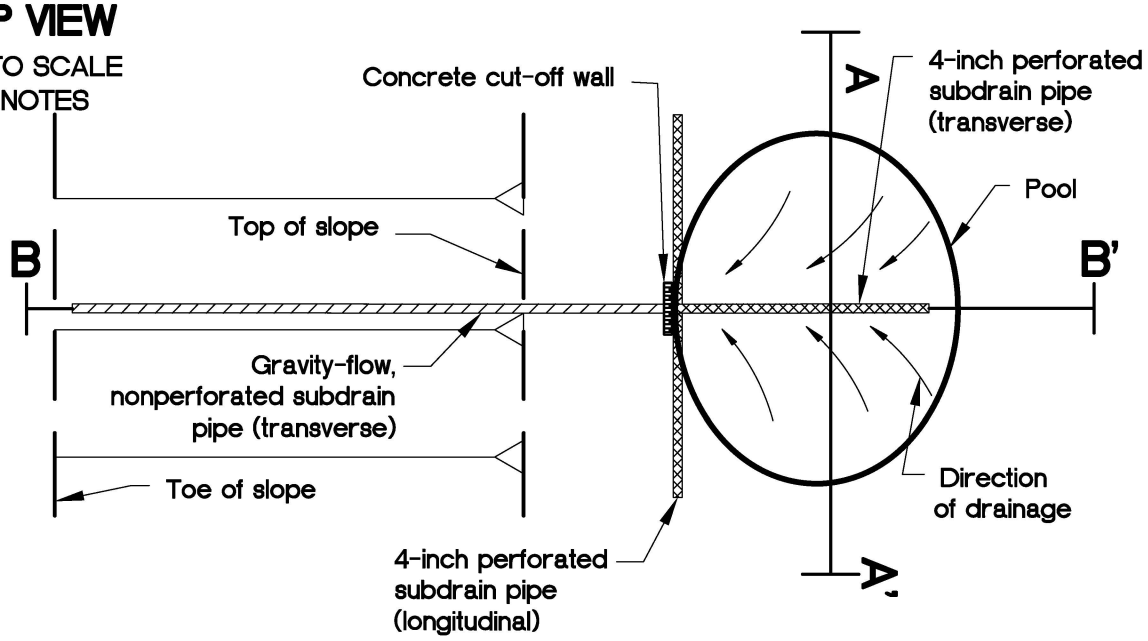
GeoSoils, Inc.

ROCK FALL MITIGATION DETAIL

Plate F-16

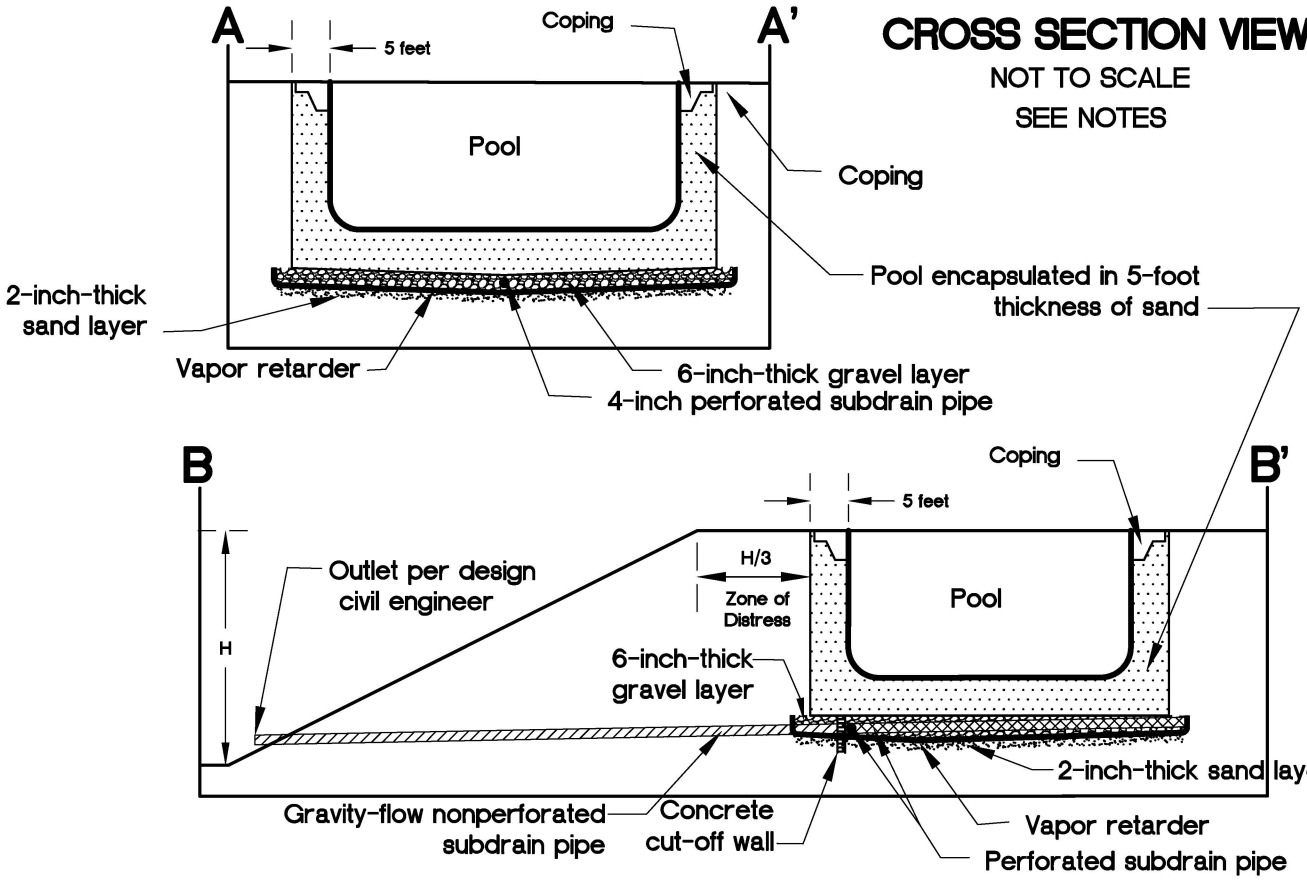
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

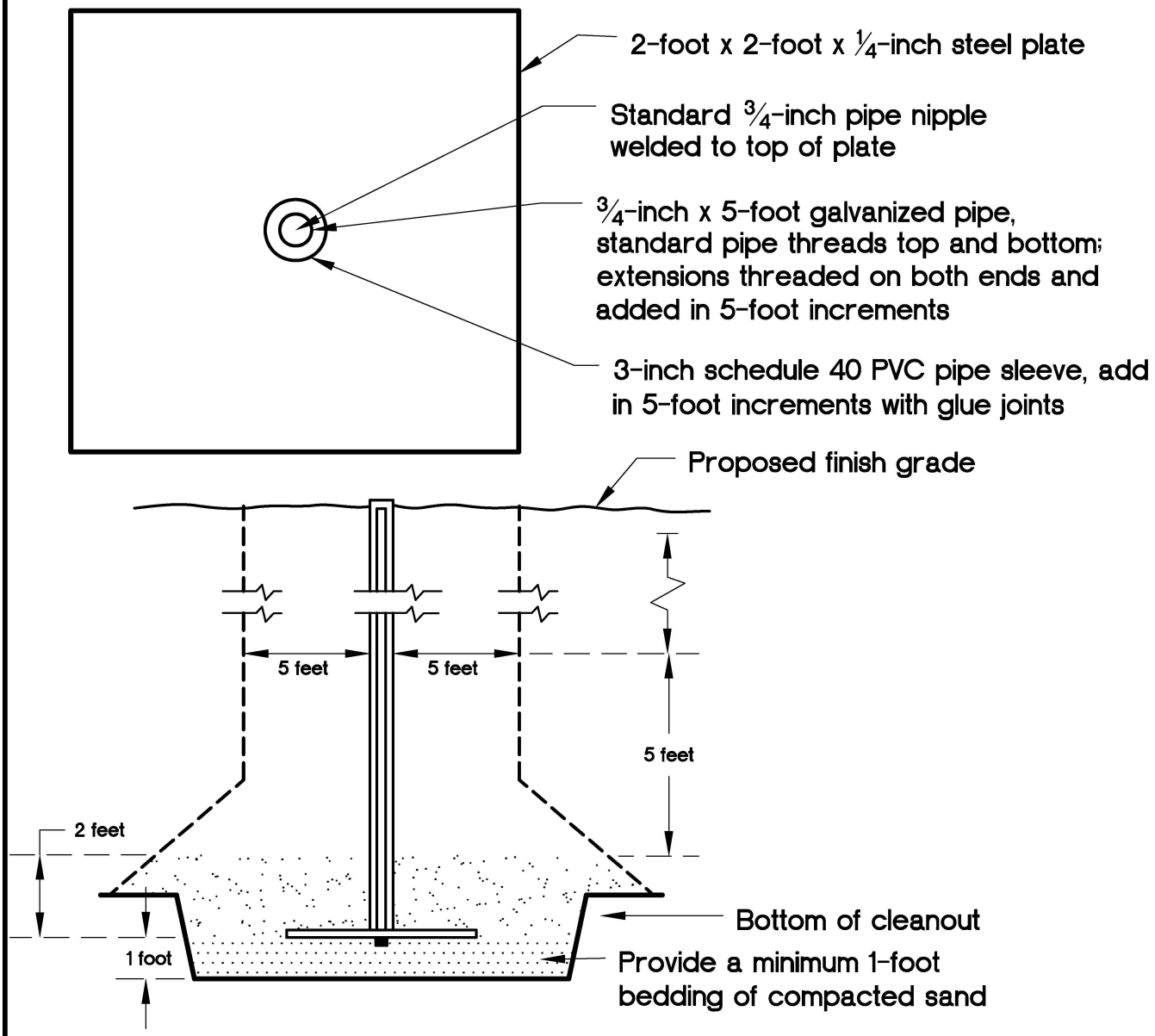
NOT TO SCALE
SEE NOTES



NOTES:

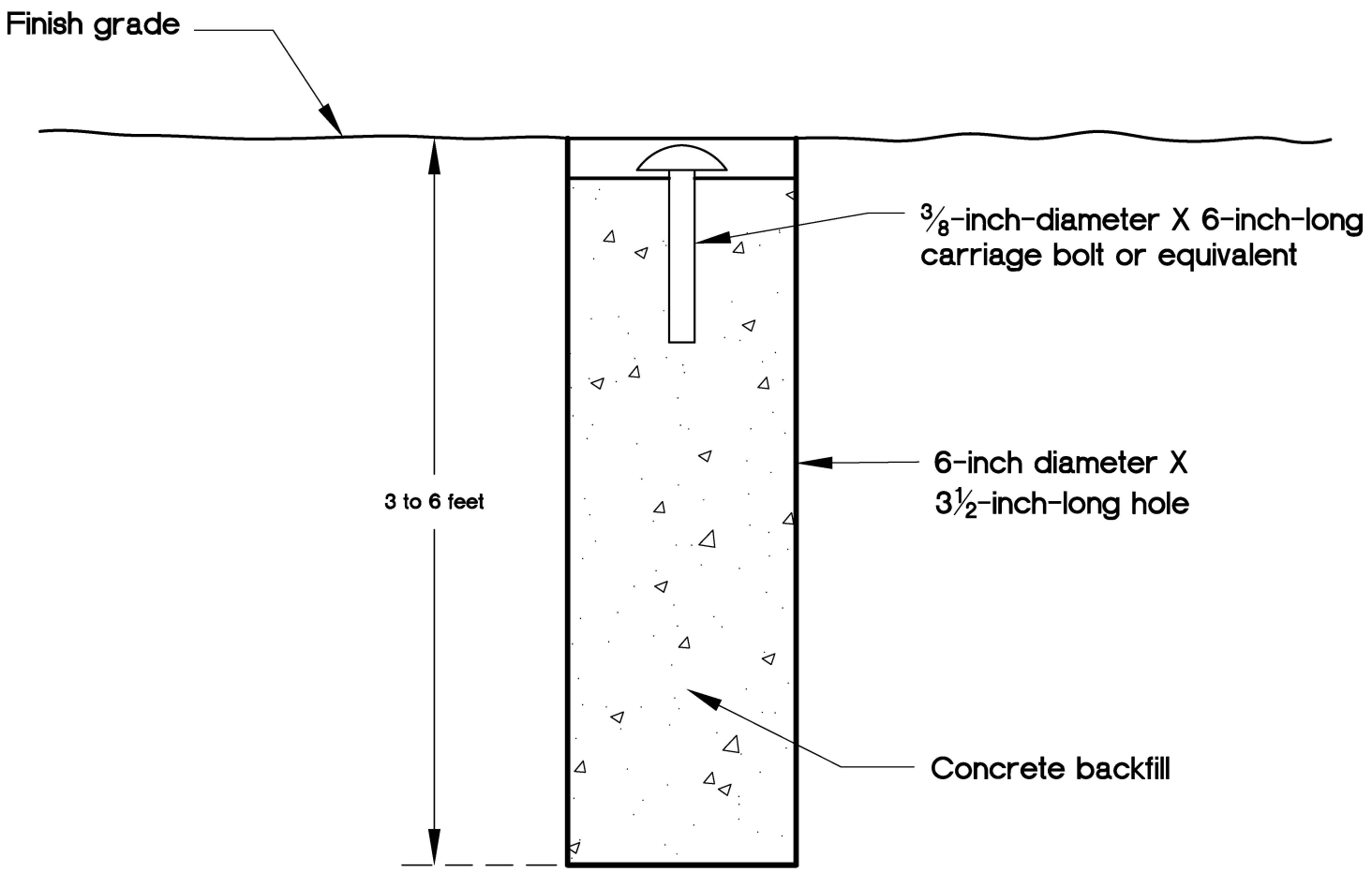
1. 6-inch-thick, clean gravel (¾ to 1½ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe at low point and outlet or to sump pump area.
2. Pools on fills thicker than 20 feet should be constructed on deep foundations; otherwise, distress (tilting, cracking, etc.) should be expected.
3. Design does not apply to infinity-edge pools/spas.

GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate F–17



NOTES:

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-foot radius of plate base and withiin 5 feet (vertical) for heavy equipment. Fill within clearance area should be hand compacted to project specifications or compacted by alternative approved method by the geotechnical consultant (in writing, prior to construction).
3. After 5 feet (vertical) of fill is in place, contractor should maintain a 5-foot radius equipment clearance from riser.
4. Place and mechanically hand compact initial 2 feet 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 geotechnical consultant 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 geotechnical consultant.

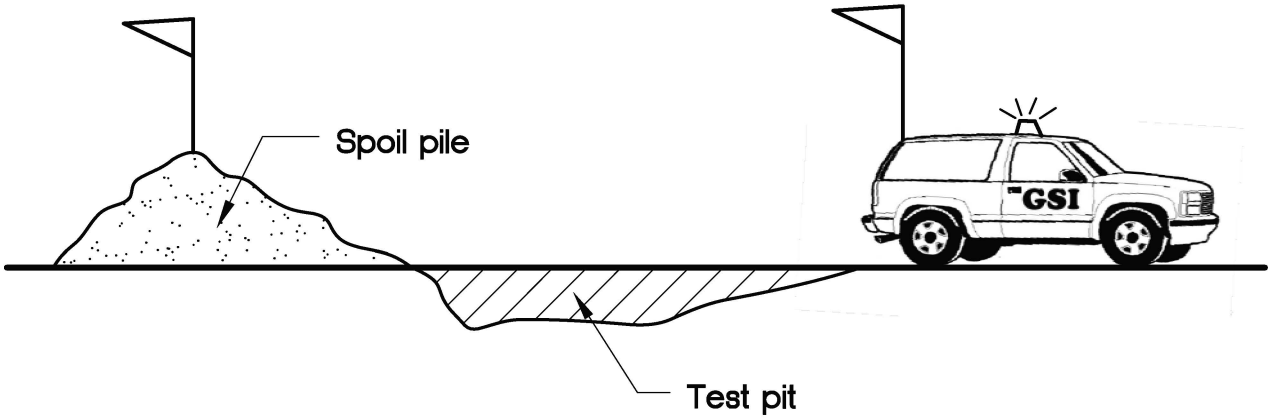


GeoSoils, Inc.

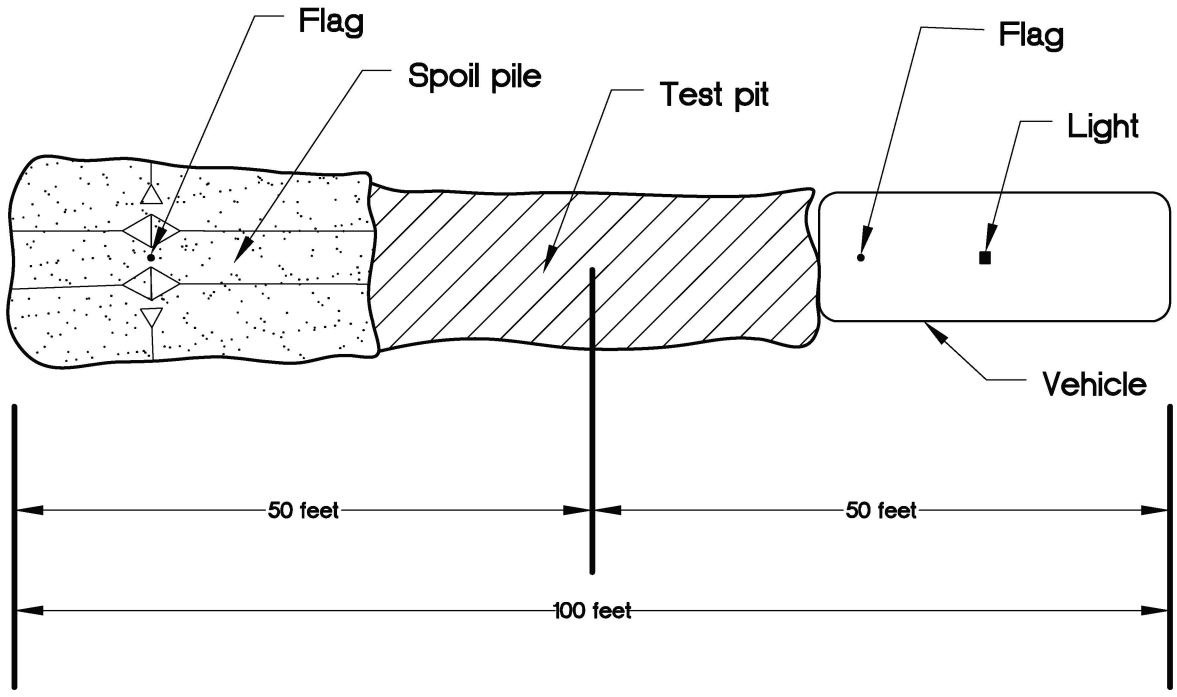
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate F–19

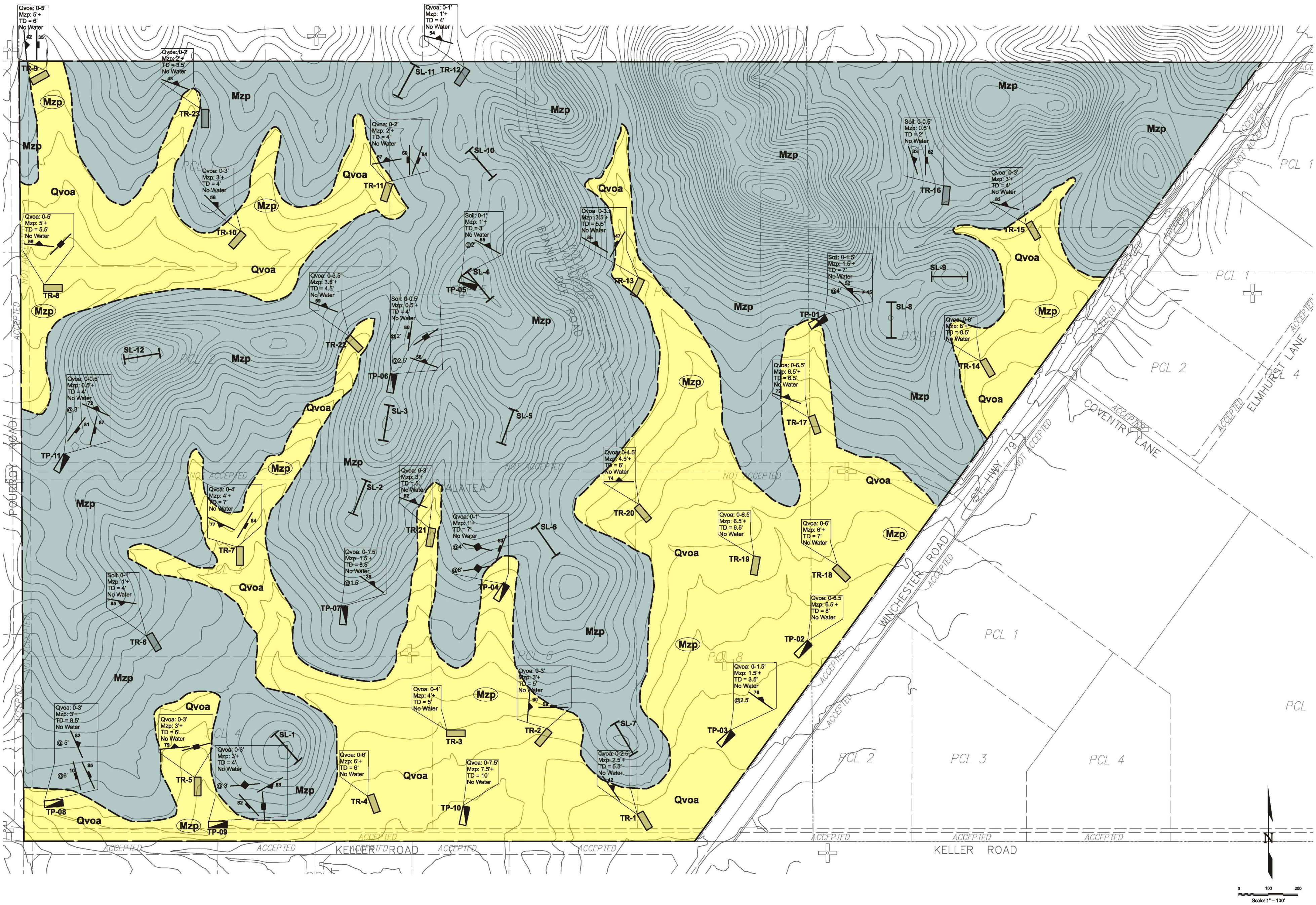
SIDE VIEW



TOP VIEW



GeoSoils, Inc.	TEST PIT SAFETY DIAGRAM	Plate F-20



ALL LOCATIONS APPROXIMATE

This document or efile is not part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

EXPLANATION

Qvoa	Very Old Alluvial Deposits	SL-12	Location of Seismic Refraction Line
Mzp	Phyllite, Circled Where Buried	TP-11	Location of Test Pit (GSI)
— —	Geologic Contact	TR-23	Location of Test Pit (Eberhart/United)
	Metamorphic Foliation Attitude (Vertical)	— —	Property Line
	Joint Attitude (Vertical)		

Base Map: K & A Engineering, Inc. (2007)

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOTECHNIAL MAP

Plate 1 of 1

W.O. 5815-A-OC | DATE 07/09 | SCALE: 1" = 100'