

**GEOTECHNICAL EVALUATION
FOR
SOUTH SHORE II PROJECT, TRACT 36567
LAKE ELSINORE, RIVERSIDE COUNTY, CALIFORNIA**

PREPARED FOR

**SOUTH SHORE II, LLC
515 AVOCADO AVENUE
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PREPARED BY

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April 8, 2013
Project No. 0967-CR3

South Shore II, LLC

515 Avocado Avenue

Corona Del Mar, California 92625

Attention: Mr. Erik Lunde

Subject: Geotechnical Evaluation
South Shore II Project, Tract 36567
Proposed Residential Development
Lake Elsinore, Riverside County, California

Dear Mr. Lunde:

GeoTek, Inc. (GeoTek) is pleased to provide herein the results of our geotechnical evaluation for the subject project, located in the City of Lake Elsinore, Riverside County, California. This report presents the results of our evaluation, discussion of our findings, and provides geotechnical information for the site and preliminary recommendations for foundation design and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development. The recommendations in this report are subject to further review/evaluation and potential modification once more complete grading plans become available.

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

Respectfully submitted,
GeoTek, Inc.



Edward H. LaMont
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Principal Geologist

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Ronald A. Reed
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I. EXECUTIVE SUMMARY

Based on the results of this evaluation, the major geotechnical findings or design and construction considerations are summarized as follows:

- The site is primarily underlain by metasedimentary and granitic bedrock units, which are mantled with a variable thickness layer of colluvial and/or alluvial materials. Alluvial deposits are identified and expected in the main drainage areas dissecting the site (see Plate I). Localized undocumented fill materials are also noted on the site, mostly associated with existing access trails and roadways, and additional areas of undocumented fill will also likely be encountered during grading. These undocumented fills appear limited and surficial in nature, and are not identified on the Geotechnical Map (Plate I).
- For planning purposes, complete removal and recompaction of all undocumented fill materials, colluvium, alluvium and highly weathered bedrock materials should be anticipated.
- The majority of the onsite materials appear to be suitable for foundation support or re-use as fill provided they are prepared in accordance with the grading requirements included in this report. Site soils generally appear to have a “very low” expansion potential ($El < 21$), based on preliminary test results and experience in the area.
- For planning purposes, a shrinkage factor of 10% to 15% may be applied for the surficial materials (colluvium and alluvium) requiring removal and recompaction. However, bulking of up to 10 to 15% should be anticipated for granitic bedrock, and 15 to 25% for metasedimentary bedrock. Actual bulking and shrinkage percentages will vary depending on location, depth of material, material type (i.e. geologic units), level of compaction attained during rough grading, and other factors.
- Strong ground shaking is expected at this site due to the known earthquake faults in the vicinity. However, the risks of seismic densification and liquefaction are considered low due to the dense nature of the underlying soils, absence of a shallow groundwater table and the recommended remedial earthwork provided herein.
- Deep fills anticipated for the site (up to $100 \pm$ feet) will likely necessitate settlement monitoring subsequent to rough grading. Fills in excess of $40 \pm$ feet in thickness will likely be recommended to be monitored for settlement prior to construction of structural improvements. Increased compaction standards, including attaining a minimum of 93% relative compaction for all fills greater than 40 feet from finish grade, are recommended.

- Canyon subdrains and stabilization slope backdrains are recommended for the site. General recommendations for subdrain and backdrain construction are included in Appendix D.
- The onsite alluvial, colluvial, weathered granitic bedrock and metasedimentary bedrock materials are anticipated to be largely excavatable with conventional heavy-duty grading equipment. Excavation(s) in the less-weathered bedrock materials (i.e., granitic and metasedimentary materials) may locally require special excavation techniques. Oversize rocks (>8 inches) should be anticipated and hard corestones (i.e., floaters) may be encountered at a shallow depth and may also require special handling and placement. Localized more indurated metasedimentary beds (quartzite units) may also require special handling and placement.
- Deep cuts anticipated for the site (up to 100± feet) may locally expose materials necessitating special excavation techniques and rock placement methods.
- In general, cut and fill slopes constructed to the heights currently proposed at 2:1 (h:v) gradients and shallower are considered grossly stable on this site.
- Although considered grossly stable, cut slopes exposing metasedimentary bedrock materials are anticipated to locally be very blocky and difficult to establish landscaping upon. Consideration should be given to replacing some cut slope areas with engineered fill for this reason. This will also likely be a factor for some cut slope areas exposing granitic bedrock. Finish cut slopes should be mapped an engineering geologist to evaluate the exposed surface for obvious surficial slope stability or rock fall concerns.
- Conventionally reinforced spread and/or continuous wall footings are considered a suitable foundation system for the proposed buildings, provided further verification and review of settlement characteristics and soil expansion potential are performed at the completion of final site grading. Consideration should be given to the use of post-tensioned foundation systems in deeper fill areas (i.e., fills in excess of 40 feet in depth).

2. INTENT AND SCOPE OF SERVICES

It is the intent of this report to aid in the design and planning of the proposed development. Implementation of the advice or findings included in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained herein are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

This evaluation does not and should in no way be construed to encompass any areas beyond the specific areas of proposed construction as indicated to us by the client. Further, no



evaluation of any existing site improvements is included. The scope is based on our current understanding of the project and the client's needs, our proposal dated February 15, 2013 (P3-0901512r3), and geotechnical engineering standards normally used on similar projects in this region.

The scope of our evaluation is limited to the area explored that is shown on the Geotechnical Map (Plate I). Plate I utilizes a 1"=100' scale map provided by K&A Engineering, Inc. as a base. Additional subsurface exploration and supplementary laboratory testing may be necessary at a later date once site grading plans become more complete. Services provided for this study have included the following:

- Research and review of available geologic data and general information pertinent to the site (see referenced reports and literature),
- Site exploration consisting of the geologic field mapping and excavation, logging, and sampling of nine (9) exploratory test trenches, and five (5) seismic refraction survey lines to evaluate rock hardness/excavatability,
- Laboratory testing on soil samples collected during the field investigation,
- Review and evaluation of site seismicity, and
- Compilation of this geotechnical report which presents our findings, conclusions, and recommendations for site development.

3. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

3.1 SITE DESCRIPTION

The subject site, which consists of approximately 72 acres, is located northeast of Interstate Highway 15 and roughly one mile east of Lake Elsinore, in the City of Lake Elsinore, Riverside County, California (see Figure 1). More specifically, the irregularly shaped site is located roughly one-quarter mile to the northeast of Camino del Norte Street, just north of the closed Elsinore Sanitary Landfill site. The subject property is comprised of primarily ungraded and vacant land. The site can be considered as having hillside terrain, with variable surface drainage but generally directed toward the south and southwest (see Figure 2) via natural drainages. Total relief across the site is on the order of 300± feet. A portion of the southeastern-most site area appears to have been used as a borrow site, as an open excavation area is present.

Site vegetation consists of native brush, weeds and grasses. Some small trees are also located in drainages and low-lying site areas. The property ranges in elevation from a low of approximately 1520 feet above mean seal level (msl) toward the southwestern edge of the property, to roughly 1820 feet above msl near the northern site boundary.

Surrounding properties to the east, west and north are in an apparent natural condition and undeveloped. A former landfill (Elsinore Sanitary Landfill) is located adjacent the southeast of the subject property. This landfill is understood be owned by the City of Lake Elsinore and maintained by the Riverside County Waste Management Department, and was closed in 1986. An aerial photo of the site area is included on Figure 3.

3.2 PROPOSED DEVELOPMENT

We understand that most of the subject site is planned to be subdivided into typical single-family residential lots, with a portion of the southeastern site area possibly being utilized as a park site. A detention basin is also conceptually planned for the southwestern corner of the site. The single-family residential development is expected to predominantly consist of typical one or two-story wood-framed structures with conventionally reinforced slab-on-grade and/or spread and continuous wall footings. Structural loads are expected to be typical for such construction.

A 100-scale grading plan was available for our review at the time of the herein evaluation. This grading plan was used as the base for our Geotechnical Map (Plate I). The 100-scale plan was provided by K&A Engineering, Inc. Based on review of the plan, it is anticipated that cuts of up to $100\pm$ feet and fills of up to $100\pm$ feet will be required to achieve proposed finish grades. If site development plans change, GeoTek should review from a geotechnical perspective. Additional evaluation and/or subsurface exploration(s) may be recommended at a future date if deemed necessary subsequent to review of future development plans.

4. FIELD EXPLORATION AND LABORATORY TESTING

4.1 PREVIOUS FIELD EXPLORATIONS

No previous geotechnical field explorations or reports are known to have been completed for the subject site. GeoTek has completed a *Geotechnical Evaluation* for the property immediately adjacent and west of the subject site (GeoTek, 2011). A geotechnical report for a proposed development of the property immediately to the northwest of the subject site (Lawson & Associates Geotechnical Consulting, Inc., 2004) was also reviewed in conjunction with



completing the herein evaluation, as was a *Semi-Annual Groundwater and General Site Monitoring Report for the Elsinore Sanitary Landfill* (County of Riverside Waste Management Department, 2012) for the landfill to the southeast of the subject site.

4.2 CURRENT FIELD EXPLORATION

The herein reported field exploration was conducted by GeoTek in March of 2013. An engineering geologist from our firm logged the excavations and collected samples for use in the laboratory testing. Logs of exploratory excavations are included in Appendix A. Locations of the excavated trenches are shown on the Geotechnical Map (Plate I). Field geologic mapping of the site was also performed.

4.3 LABORATORY TESTING

Laboratory testing was performed on selected soil and bedrock samples collected during the field investigation. The purpose of the laboratory testing was to confirm the field classification of the soil materials encountered and to evaluate their physical properties for use in the engineering design and analysis. The results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included in Appendix B.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends from the Transverse Ranges geomorphic province and the Los Angeles Basin, 975 miles south to the tip of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Three major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zones trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

5.2 GENERAL SOIL CONDITIONS

A brief description of the earth materials encountered on the subject property is presented in the following sections. More detailed descriptions of these materials are provided in the logs of the exploratory trenches included in Appendix A. Based on our site reconnaissances, field mapping, subsurface excavations, and review of the referenced reports and published geologic maps, the site is underlain to the depths explored by Quaternary-age alluvial deposits, Cretaceous-age granitic and Jurassic-age metasedimentary bedrock units. Colluvial materials locally mantle the bedrock. Localized undocumented fill soils are also scattered across the site, and other fill materials may also be encountered or identified during earthwork for the project.

5.2.1 Undocumented Fill (Unmapped)

Localized undocumented fill soils were observed scattered across the site. The existing fills noted are largely associated with existing dirt trails and roadways that traverse the site, and some fill materials also appear to be associated with a former onsite borrow pit/excavation area (near the southeastern portion of the site, immediately north of the adjacent landfill property). These undocumented fill materials are expected to be suitable for reuse as engineered fill provided they are free from deleterious materials and oversize rocks.

5.2.2 Colluvium (Mapped as Qc)

Colluvial materials locally mantle site bedrock units. Thicker accumulations of colluvial soils noted on the site are identified on the Geotechnical Map (Plate I). In general, these materials typically consist of cobbly and/or gravelly silty sand, and may be reused as engineered fill provided they are processed and meet project specifications. Based on our experience with similar soils, the onsite colluvial materials likely possess a “very low” to “low” expansion potential (Expansion Index, $EI < 51$) when tested and classified in general accordance with ASTM D 4829.

5.2.3 Alluvium (Mapped as Qal)

Quaternary-age alluvium was encountered in most of the exploratory excavations toward the southern portion of the site, and has also been mapped in the primary site drainages. In general, these materials consist of silty sand to gravelly silty sand. Based on our lab testing and experience with similar soils, the onsite alluvial materials possess a “very low” expansion potential ($0 \leq EI < 21$).

GeoTek recently excavated test trenches TP-1 through TP-9 in the primary site drainages. The intent of excavating these test pits was to help evaluate the approximate depth of alluvium in these drainage areas, and to assess the suitability of these materials from a geotechnical

perspective with respect to proposed site development. The thickest alluvium encountered onsite in our exploratory excavation was approximately 10 feet, in our test pit TP-7 (see Plate I). More detailed descriptions of the alluvium encountered in the site drainages explored are included on the logs in Appendix A. In general, site alluvial materials may be reused as engineered fill provided they are processed and meet project specifications.

5.2.4 Granitic Bedrock (Mapped as Kgr)

Mesozoic-aged (Cretaceous) granitic bedrock has been mapped toward the southern and southwestern site areas (see Plate I). Surficial granitic boulders were also observed along portions of the western edge of the property, and may be underlying portions of that site area as well at relatively shallow depth. The granitics are plutonic in origin, and intruded into the older/pre-existing now metasedimentary units described below. Where weathered, the granitic material generally decomposes into brown, gray, black and orange mottled, fine- to coarse-grained sand with gravel, cobble and boulder size clasts. Where relatively unweathered, the granitic bedrock can be very hard. Discussion of excavatability and slope stability are presented in later sections of this report. Onsite granitic materials are expected to possess a “very low” expansion potential ($0 \leq EI < 21$).

5.2.5 Metasedimentary Bedrock (Mapped as Ju)

Upper Jurassic age metasedimentary bedrock has been mapped across the majority of the site (see Geotechnical Map, Plate I). This mottled dark unit is primarily comprised of inter-bedded quartzite, phyllite and slate units. The thinly bedded slate and phyllite units are generally heavily fractured, jointed and folded. Relict bedding is commonly measurable in this material. The quartzite units are relatively dense and are more resistant to weathering, and are commonly exposed along most site ridgelines. Further discussion of excavatability and slope stability are presented in later sections of this report. Metasedimentary bedrock materials are expected to possess a “very low” expansion potential ($0 \leq EI < 21$).

5.3 GEOLOGIC STRUCTURE

Granitic bedrock is structurally massive, with overall irregular to locally regular joint patterns likely. Relict bedding within the metasedimentary bedrock units has been measured at various locations across the site (see Geotechnical Map, Plate I). Where measured, the bedrock strike generally trends to the northwest, with dips of 40+ degrees down to the northeast. Prominent joint and/or fracture patterns have also been noted at various locations on the site, and also appear to locally control site topography (orientation of joints commonly results in differential weathering patterns, which helps to form topography).

No landslides have been identified on the site. Near surface slope instabilities may also locally occur across the site, on steeper slope areas in particular, and should be further assessed during site earthwork construction by an engineering geologist.

5.4 SURFACE AND GROUNDWATER

5.4.1 Surface Water

Existing natural site drainage channels are found at various locations across the property, with the most prominent channels identified on the Geotechnical Map (Plate I). No surface water was encountered or noted during our recent field exploration for the project, or in any of the referenced geotechnical reports. Any surface water on this site is the likely result of direct precipitation or surface run-off from surrounding sites that may be directed to the subject property. Overall site drainage is generally directed in a westerly direction, with local variations common. Provisions for surface drainage will need to be accounted for by the project civil engineer. All site drainage should be reviewed and designed by the project civil engineer.

5.4.2 Groundwater

Groundwater was not encountered in any of our exploratory excavations, and is not anticipated to be a factor in currently proposed site development. The presence of natural groundwater conditions or perched groundwater can not be precluded, however. Groundwater depth in the site vicinity is expected to be well over 100 feet below natural ground surface elevations (California DWR). However, perched groundwater or localized seepage can occur due to variations in rainfall, irrigation practices, and other factors not evident at the time of this investigation. Some perched groundwater should be anticipated in the vicinity of the existing drainages throughout the property, or where relatively impermeable bedrock units are present and/or exposed during grading operations. Some perched groundwater is understood to exist in the vicinity of the landfill located adjacent to the southeastern portion of the site (County of Riverside Waste Management District, 2012). Natural groundwater gradient in the site area is understood to be to the southwest.

Subdrain systems are recommended and should be placed subsequent to performing removals during the earthwork construction in the area of the existing natural drainages, or where seepages are encountered during grading, based on exposed conditions and planned development (see Plate G-I, Appendix D for generalized subdrain construction guidelines).

5.5 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *Alquist-Priolo* Earthquake Fault Zone or a Special Studies Zone. No County of Riverside designated fault or fault zones have been designated or are known to exist on the site.

Fault studies previously completed by other consultants, including Geowest (1987) and Lawson & Associates (2005), have indicated no active faulting in the immediate site area. GeoTek concurs with these findings (the Elsinore fault is now generally indicated as being located several miles to the west of the subject property on most geologic maps of the region). Based on review of information available in our library and review of air photos for the site in our library, no other faulting has been reported on the subject property. No active site faulting has been identified or mapped by this firm.

6. SEISMIC CRITERIA

The site is located at approximately 33.6795 Latitude and -117.3079 Longitude. Site spectral accelerations (S_s and S_I), for 0.2 and 1.0 second periods for a Class "D" site (portions of the project site can be classified as Class "B" at a later time, depending on final graded configurations), were determined from the USGS Website, Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude/Longitude, 2009 Data. The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.501g
Mapped 1.0 sec Period Spectral Acceleration, S_I	0.600g
Site Coefficient for Site Class "D", F_a	1.0
Site Coefficient for Site Class "D", F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 0.2 Second, SMS	1.501g
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1 second, SMI	0.900g
Design Spectral Response Acceleration for Parameter for 0.2 Second, SDS	1.001g
Design Spectral Response Acceleration for Parameter 1.0 Second, SDI	0.600g

6.1 DYNAMIC SETTLEMENT

The potential for liquefaction and associated settlement on this site is considered low due to recommended removal of the existing alluvium and colluvium, and the relatively dense nature of the underlying materials and lack of shallow groundwater table.

6.2 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigations, aerial photograph and/or literature review. Thus, the potential for existing landslides is considered low based on the reviewed data. Geologic observation and mapping should be performed by an engineering geologist during earthwork operation.

The potential for secondary seismic hazards such as seiche and tsunami are considered to be negligible due site elevation and distance from an open body of water.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 GENERAL

The proposed development of the site appears feasible from a geotechnical viewpoint provided that the recommendations provided herein are incorporated into the design and construction phases of development. However, these recommendations are considered preliminary and should be subject to further review and evaluation once site grading plans become more complete.

7.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in general accordance with the applicable grading ordinances of the City of Lake Elsinore, County of Riverside, the 2010 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix D outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix D.

7.2.1 Site Clearing

In areas of planned grading or improvements, the site should be cleared of any existing improvements or structures, vegetation, roots and debris. These deleterious materials should

be properly disposed of offsite. Any holes resulting from site clearing, tree removal, and/or the trenches excavated during this or possibly other prior geotechnical studies of the site should be replaced with engineered fill materials compacted to the project standards.

7.2.2 Removals

If not removed by the proposed grading, all undocumented fill materials, colluvial and alluvial materials are subject to complete removal and recompaction within the limits of grading. If applicable, structural removals should extend a minimum 1:1 (horizontal to vertical) projection down and away from proposed finish grade elevations.

7.2.3 Fills

The onsite soils are considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Remedial removal and/or undercut areas should be brought to final subgrade elevations with fill that is placed and compacted in accordance with the general grading guidelines presented in Appendix D.

Some portion of the onsite bedrock may generate moderate to significant quantities of oversized rock (i.e., material greater than 8 inches). These materials should be placed using the procedures outlined in Appendix D. Alternatively, oversized materials may be disposed of off-site.

Due to the anticipated fill depths in some proposed areas of the project, settlement monitoring is recommended in order to help evaluate whether the estimated remaining settlement is within the project specification prior to construction of structural improvements. This monitoring is recommended for representative site areas where fill depth exceeds $40 \pm$ feet. A more detailed settlement monitoring program can be developed when grading plans become more finalized, and during earthwork construction, when as-graded site conditions become apparent.

Compaction levels for deeper site fills are also recommended to be relatively higher than otherwise prescribed. For fills greater than 40 feet in depth, a minimum relative compaction of 93% is recommended. Maximum dry density should be determined in general accordance with ASTM D 1557.

7.2.4 Cut and Transition Lots

All cut lots and cut portions of transition lots should be overexcavated a minimum of three (3) feet below finish pad grade or two (2) feet below bottom of deepest footing, whichever is



deeper, and replaced with engineered fill in an attempt to provide a more uniform condition and decrease the potential for differential settlement. Lot overexcavation bottoms should be cut to drain to an adjacent street at a gradient of at least 1%, to reduce the potential for possible ponding of perched groundwater subsequent to development. In addition, cut/fill transition gradients are not recommended to exceed 3:1 (horizontal to vertical) gradient. Consideration should also be given to overexcavating site streets to the depth of the deepest utility, in order to facilitate subsequent trenching for underground utilities (sewer, storm drain, water, joint utilities, etc.). Overexcavation recommendations should be provided based on review of finalized site grading plans, and conditions exposed during grading.

7.2.5 Excavation Characteristics

Rippability studies and opinions regarding site area excavatability have previously been provided and/or performed nearby project sites by others, including GeoTek (2011), GeoSoils, Inc. (GSI, 1990 and 1991), Lawson & Associates Geotechnical Consulting, Inc. (2004) and others. In addition, results of seismic refraction traverses recently completed for the subject site are included in Appendix A.1, at the back of this report. Approximate locations of the refraction traverses reported on herein are indicated on our Geotechnical Map (Plate I).

The seismic refraction traverses recently completed for the subject site appear very similar to results for adjacent properties. Based on review of the results, site bedrock appears to generally be excavatable with conventional earthmoving equipment to depths ranging from roughly 30 to 90+ feet, depending on location (see results presented in Appendix A.1). However, some resistant bedrock units may require the use of a single-shank ripper on a Caterpillar D-10 or larger bulldozer (or equivalent) at shallower depths, or locally where conditions dictate (possible quartzite units or granitic corestones).

The majority of the subject site consists of mapped metasedimentary and granitic bedrock units. The data produced by seismic lines and other data are subject to interpretation. However, based on knowledge of earthwork completed on nearby projects in similar bedrock units, excavations to the depths currently anticipated for the subject project are feasible without the need for blasting. The ultimate need for special excavation techniques and/or blasting will only be known during rough earthwork construction.

Further exploration can be conducted at a later date when more site specific grading plans become available if additional assessment of this topic is desired.

Oversize rocks (>8 inches) should be anticipated in both the granitic and metasedimentary bedrock units on this site and hard “floaters” may also be encountered at relatively shallow depths and require specialized excavation techniques.

Excavation in the colluvial and alluvial materials is expected to be easy using heavy-duty grading equipment. Saturated alluvium may be encountered in some of the drainage channels, depending on the time of year. However, at the present time it is not generally anticipated that significant quantities of saturated alluvium will be encountered.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the onsite materials should be stable at 1:1 (h:v) inclinations for cuts less than 10 feet in height.

7.2.6 Slope Stability

Based on our field mapping and experience with similar soil and bedrock types, 2:1 (horizontal to vertical) fill slopes constructed with the onsite soils in accordance with the grading requirements presented in this report will have calculated factors of safety in excess of 1.5 under static conditions and 1.1 for seismic conditions for slopes with a maximum height of 90± feet (see slope stability analyses presented in Appendix C). Shear strengths used in the analyses ($C = 100\text{psf}$; $\phi = 35^\circ$) are a “composite” strength based on the results of the remolded samples tested. Additional review and analyses for higher slopes should be completed when final grading plans become available, if different than the herein site development plan.

In general, proposed 2:1 (h:v) gradient cut slopes excavated into site bedrock materials are anticipated to be grossly stable. Mapped metasedimentary bedrock structure is generally favorable to proposed cut slopes in any orientation due to the relatively steep dip angles measured within this unit across the site (see Cross-Section A-A' on Plate 2). However, local folding, joint patterns and fracturing of this bedrock may locally reveal adverse structure upon excavation of some slopes proposed to be cut into this material and should be mapped by an engineering geologist upon excavation. Appendix C presents global stability analysis for Cross-Section B-B' which is the highest 2:1 fill slope proposed. Global stability analyses of this Cross-Section B-B' indicate factors of safety in excess of code minimums for an engineered fill condition.

Granitic bedrock is anticipated to be structurally massive, but also could expose adverse structural features (joint and/or fracture patterns). If adverse structure is exposed and

identified in proposed cut slopes upon excavation, stabilization fills may be subsequently recommended. In addition to the above, surficial stability and rock fall potential will need to be reviewed upon excavation of all cut slopes on the site. Subsequent to excavating, project cut slopes should be observed and mapped by an engineering geologist in order to determine if additional corrective grading and/or recommendations are needed (specifically with respect to rock fall potential, localized out-of-slope bedding, joint or fracture patterns).

Some of the onsite materials used as engineered fill have relatively low cohesion values. Placement of these materials at/ear finish slop grade will result in surficial slope stability factor of safety of less than 1.5 and should be avoided. If such materials are placed at/ear finish slope grade, then additional mitigative measures such as jute mesh, slope netting, etc. may be required. Additional evaluation may be prudent at the grading plan review stage.

7.2.7 Shrinkage, Bulking, and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, bulking, subsidence (not expected on this site), trench spoil from utilities and footing excavations, and final pavement section thicknesses as well as the accuracy of topography.

Shrinkage and bulking are primarily dependent upon the degree of compactive effort achieved during construction, in-place density prior to removal and recompaction, and methods and techniques used to remove, transport and compact the respective material(s). For planning purposes, a shrinkage factor of 10 to 15 percent may be considered for the surficial materials (colluvium and alluvium) requiring removal and recompaction. A bulking factor of 10 to 15 percent may be considered for the granitic bedrock, and 15 to 25 percent for the metasedimentary bedrock. Variations in bulking percentages are primarily due to variations in material types (i.e. slate, phyllite, quartzite, etc.) for the metasedimentary materials, hardness, density, depth of cut, degree of weathering, rippability, method of fill placement, etc.

The County of Riverside has designated a portion of the site as being susceptible to subsidence on their Transportation and Land Management website. The susceptibility to subsidence is an issue that affects large regions within Riverside County and no site specific designation constraints are generally imposed by this designation. The subject site is almost entirely underlain by hard bedrock, and all “soft” sediments are recommended to be completely removed as part of remedial site earthwork. Subsidence is not considered to be a geotechnical constraint for the subject project, and the post-earthwork potential for subsidence is considered to be negligible.

The shrinkage and bulking estimates provided above are primarily based on laboratory testing on samples collected for the herein study (see Appendices A and B), information from consultants and developers working on recently graded sites just to the north of the subject property (by Lennar Homes and Centex Homes), and previous experience with similar site bedrock and surficial materials on nearby projects.

The above estimates are intended as an aid for project engineers in determining earthwork quantities. It is recommended that site development be planned to include an area(s) that could be raised or lowered to accommodate final site balancing.

7.3 DESIGN RECOMMENDATIONS

7.3.1 Foundation Design Criteria

Preliminary foundation design criteria for conventional foundation system in conformance with the 2010 CBC are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Based on the results of this evaluation and our past experience, the majority of the onsite soils may be classified as “very low” to “low” expansive soils ($0 \leq E.I. < 5$) with a Plasticity Index (PI) of less than 15. Laboratory testing should be performed at the completion of site grading to verify the expansion potential of the exposed soils. Thus, for planning purposes, preliminary foundation design criteria are presented in Table 7.3.1 below. Actual as graded conditions will determine the applicable foundation design criteria.

TABLE 7.3.1 – MINIMUM DESIGN REQUIREMENTS

DESIGN PARAMETER	E.I. $\leq$ 20 P.I. $\leq$ 15	20 $\leq$ E.I. $\leq$ 50 P.I. $\leq$ 15
Foundation Depth or Minimum Perimeter Beam depth (inches below lowest adjacent grade)	1 floor – 12 2 floor – 18	1 floor – 12 2 floors – 18
Foundation Width (Inches)	1 floor – 12 2 floors – 15	1 floor – 12 2 floors – 15
Footing Reinforcement	Two No. 4 bars, one top one bottom	Two No. 4 bars, one top one bottom
Minimum Slab Thickness (inches)	4	4
Minimum Slab Reinforcing	No. 3 bars, placed 18” on- center	No. 3 bars, placed 18” on-center

Presaturation of Subgrade Soil (Percent of Optimum/Depth in inches)	Subgrade well wetted before pouring concrete	100/12
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An allowable bearing capacity of 1500 pounds per square foot (psf) may be used for design of perimeter or continuous footings 12 inches deep, and pad footings 24 inches square and 12 inches deep. This value may be increased by 200 psf for each additional 12 inches in depth and by 100 psf for each additional 12 inches in width to a maximum of 2500 psf. The allowable bearing value may be increased by one-third when considering short-term live loads (e.g. seismic and wind loads).

The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2500 pounds per square foot. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

7.3.2 Settlement

Based on the anticipated loading conditions described in Section 3.2 and our evaluation of settlement characteristics, the differential settlement for the site is expected to be less than 1 inch in a 40-foot span at the completion settlement monitoring. Due to the depths of fill anticipated at the site, site settlement parameters should be reviewed when site development plans become more finalized. Monitoring deeper site fills (greater than 40 feet) for settlement is recommended to evaluate the remaining settlement is within the anticipated design value. The preliminary location of areas subject to settlement monitoring and locations of the settlement monuments can be presented at a later date subsequent to review of more finalized site development plans.

7.3.3 Foundation Set Backs

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of $H/3$ (where H is the slope height) from the face of any descending slope. The setback should be at least 7 feet and need not exceed 40 feet.

- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom inside edge of the wall stem.
- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 (h:v) projection upward from the bottom of the nearest excavation.

7.3.4 Underslab Vapor Retarder

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2010 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2010 CBC Section 1910.1. It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction.

Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6 mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeance) to achieve the desired performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation. More restrictive local jurisdictional requirements may exist and should be complied with.

7.3.5 Concrete Cracking

Control joints should be provided to help minimize random cracking. One of the simplest means to control cracking is to provide weakened joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced. We recommend that

control joints be placed in two directions spaced the numeric equivalent of two times the thickness of the slab in inches changed to feet (e.g. a 4 inch slab would have control joints at 8 feet centers). As a practical matter, this is not always possible nor is it a widely applied standard.

7.3.6 Soil Corrosivity

The soil resistivity at this site should be tested in the laboratory on representative samples collected at the site subsequent to rough grading. Preliminary test results indicate “moderate” corrosion potential for the site (see results in Appendix B). A determination of corrosivity potential and sulfate content of site soils, and their affect on proposed site development, should then be provided prior to site development and subsequent to earthwork construction.

7.4 CONCRETE CONSTRUCTION

7.4.1 General

Concrete construction should follow the CBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, we could provide quality control testing of the concrete during construction.

7.4.2 Concrete Mix Design

The sulfate content of site soils collected near finish grades will help to determine concrete mix design (cement type, water/cement ratio, etc.) to be recommended for the project. Preliminary test results indicate sulfate contents of less than 0.1% by weight, which is considered “not applicable” (i.e., negligible) as per Table 4.2.1 of ACI 318. Additional testing should be performed as the project progresses. Final design should be based on samples obtained at/near finish grades.

7.4.3 Concrete Flatwork

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered “non-structural” components. Cracking of these features is fairly common due to various factors. While cracking is not usually detrimental, it is unsightly. We suggest that the same standards of care be applied to these features as to the structure itself.

7.5 RETAINING WALL DESIGN AND CONSTRUCTION

7.5.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of 10 feet. Additional review and recommendations should be requested for higher walls.

Retaining walls embedded a minimum of 12 inches into engineered fill or dense formational materials should be designed using a net allowable bearing capacity of 1,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2,500 psf. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in Table 6.5.1 below for specific slope gradients of retained materials.

TABLE 7.5.1 – ACTIVE EARTH PRESSURES

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (Select Backfill*) (PCF)	Equivalent Fluid Pressure (Native Backfill) (PCF)
Level	35	40
2:1	55	60

*Select backfill should extend a minimum distance of H/2 behind the wall and have an EI of less than 21 and Sand Equivalent of 30 or greater.

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

7.5.2 Wall Backfill and Drainage

The onsite very low expansive soils are suitable for backfill provided they are screened of greater than 3-inch size gravels. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs. The backfill materials should be placed in

lifts no greater than 8-inches in thickness and compacted at 90% relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained.

Retaining walls should be provided with an adequate pipe and gravel back drain system to prevent build up of hydrostatic pressures. Backdrains should consist of a 4-inch diameter (Schedule 40 or SDR 35) perforated collector pipe embedded in a minimum of one cubic foot per lineal foot of 3/4 to one inch clean crushed rock or equivalent, wrapped in filter fabric. The drain system should be connected to a suitable outlet. A minimum of two outlets should be provided for each drain section. Spacing between drain outlets should not exceed 100 feet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Walls from 2 to 4 feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of wall.

7.5.3 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 60 pcf, plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner.

7.5.4 Preliminary Pavement Recommendations

For preliminary planning purposes only, the following pavement design sections are provided for the subject development. Actual recommendations will need to be based on R-Value testing, to be completed after rough grading of the project is complete for the subject street areas.

The following table summarizes the preliminary pavement sections based on an estimated R-Value result for the site. Traffic indices used for design should be provided by the governing agency, or the project civil engineer. The pavement sections are subject to the review and approval of the City of Lake Elsinore and/or County of Riverside. Performance of the pavement sections will ultimately be based largely on construction methods and traffic loading, and subgrade performance.

MINIMUM RECOMMENDED PAVEMENT SECTIONS

Location	Traffic Index	Assumed R-Value	Depth of Asphaltic Concrete (inches)	Depth of Aggregate Base (inches)
Interior Streets & Cul-de Sacs	4.5	40	3.0*	4.0*
Interior Local Streets	5.0	40	3.0*	4.0*
Enhanced Local Streets	6.0	40	4.0	5.0
Collector Streets	7.0	40	4.0	7.0

*Indicates minimum GeoTek recommended section.

It is also recommend that six (6) inches of Portland Cement Concrete (PCC) pavement be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches. The PCC pavement should be placed on a minimum of four (4) inches of aggregate base. Requirements of Section 302-6 of "Standard Specifications for Public Works Construction" (Greenbook) regarding mixing and placing concrete should be followed. Keyed joints should be provided in the longitudinal direction spaced at a maximum of 10 feet on center. Crack control joints should be provided in the transverse direction spaced at a maximum of 10 feet on center and at corners. Proper reinforcement should be considered for the concrete construction.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the County of Riverside specifications, and under the observation and testing of GeoTek and a County Inspector where required.

The aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Class II Aggregate Base (Section 26 of the Standard Caltrans Specification) or Crushed Aggregate Base (Section 200-2 of the Greenbook). Minimum compaction requirements should be 90 percent for subgrade and 95 percent for aggregate base, as per ASTM D 1557 (modified proctor). Where no base section is provided or constructed, the upper 6 inches of subgrade should be compacted to a minimum of 95 percent relative compaction. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

7.6 POST CONSTRUCTION CONSIDERATIONS

7.6.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded

slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be prudent and advisable. We could discuss these issues, if desired, when plans are made available.

As previously indicated herein, it may be difficult to establish landscaping on some cut slopes currently planned for the project, due to the anticipated rocky and hard zones anticipated to be locally exposed at finish grades. Consideration should be given to overexcavating some of these cut slope areas.

7.6.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

7.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications, and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. These representatives should perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement including utility trenches. Also, test the fill for field density and relative compaction.
- Observe and probe foundation materials to confirm suitability of bearing materials.

If requested, GeoTek will provide a construction observation and compaction report to comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

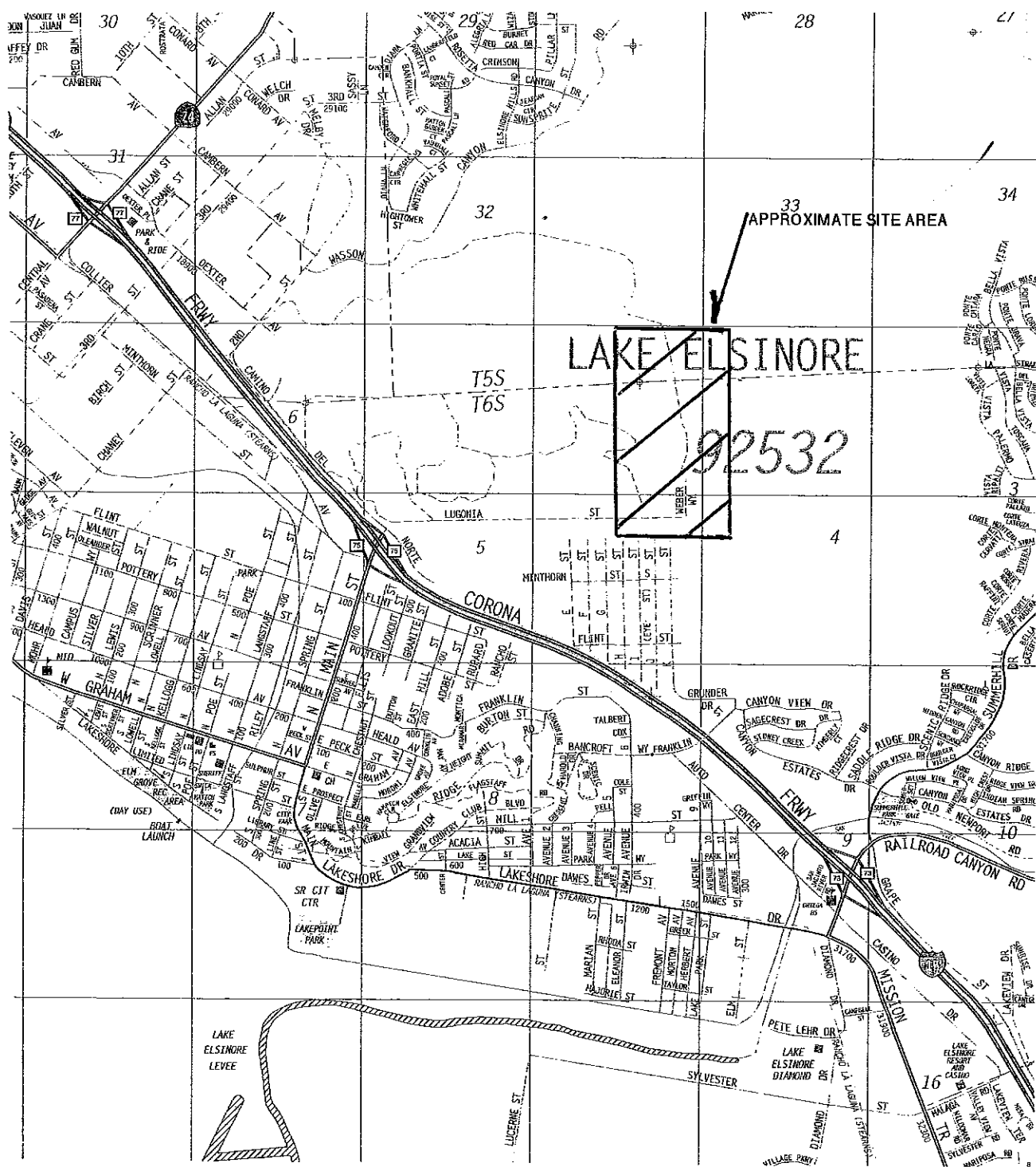
8. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusion and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

9. SELECTED REFERENCES

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South Shore II Project
Lake Elsinore Area
Riverside County, California

GeoTek Project No.: 0967-CR3

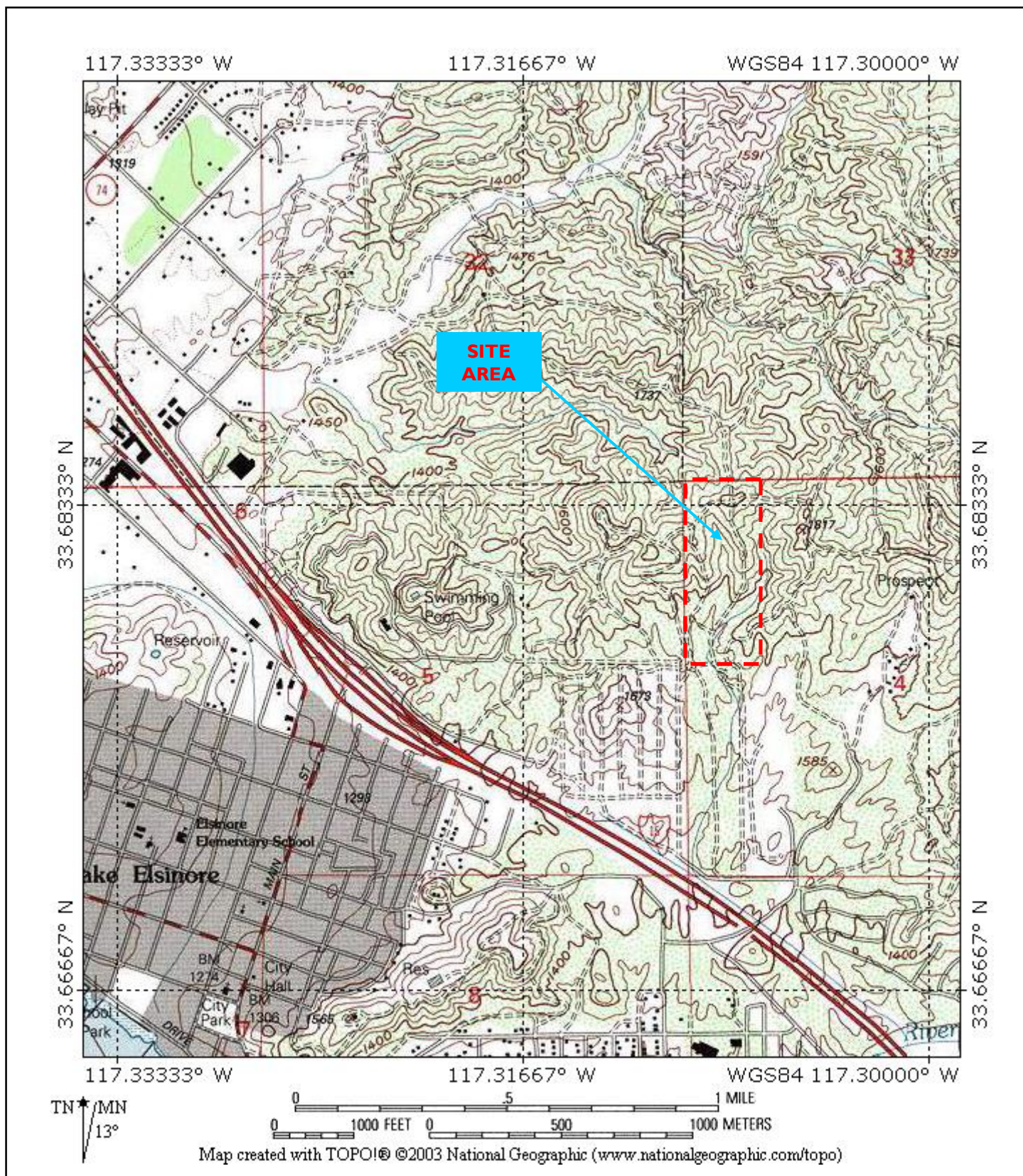


Modified from The
Thomas Guide,
Riverside County,
Scale: 1" = 2,400'

Figure 1

Site Location
Map

G
GEOTEK



South Shore II Project
Lake Elsinore Area
County of Riverside, California

GeoTek Project No. 0967-CR3



Modified from USGS
7.5 Topographic Map

Figure 2

**General Site
Topography
Map**





South Shore II Project
 City of Lake Elsinore
 County of Riverside, California

GeoTek Project No. 0967-CR3



Figure 3

**Aerial Photo
 Map of Site
 Area**



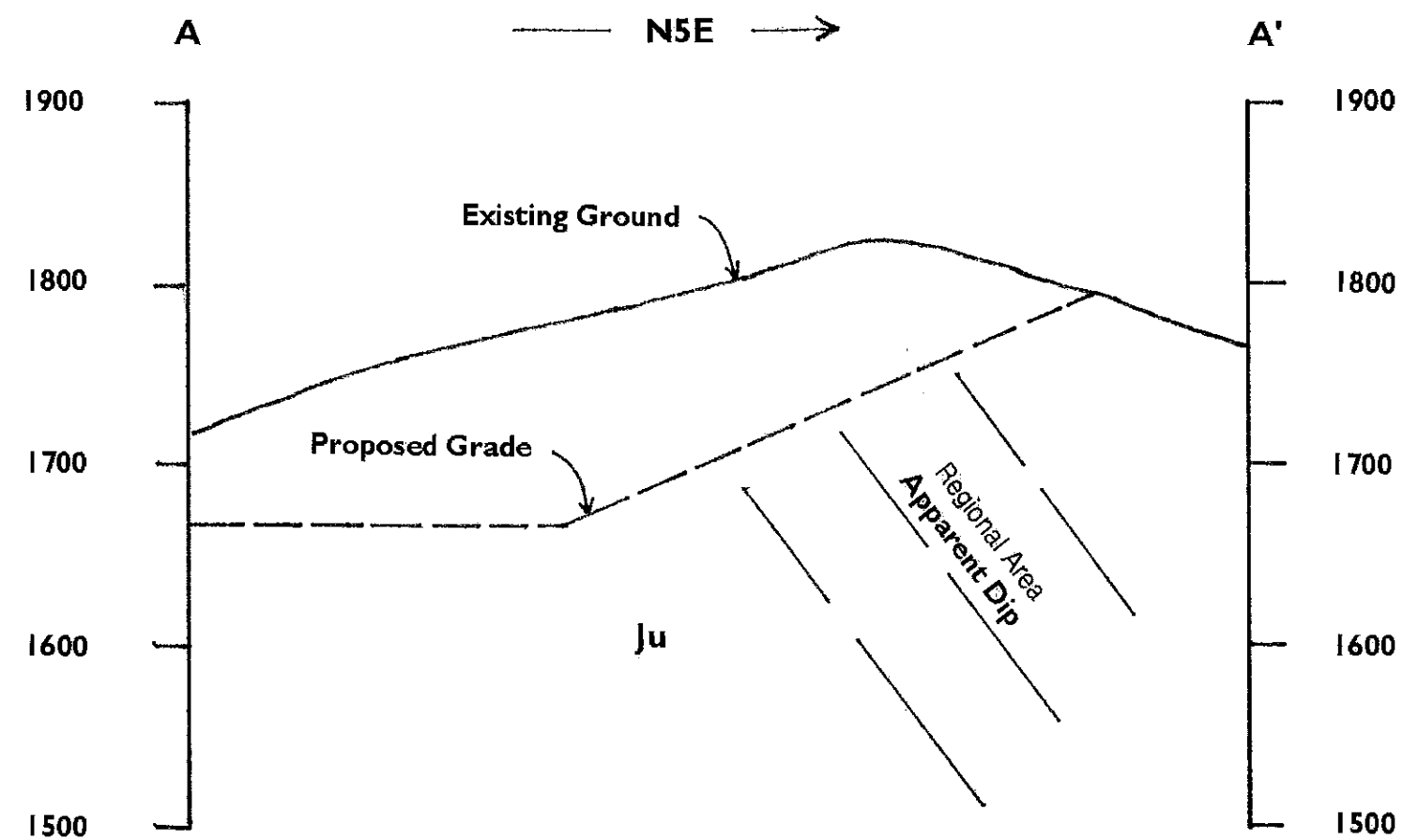


LEGEND

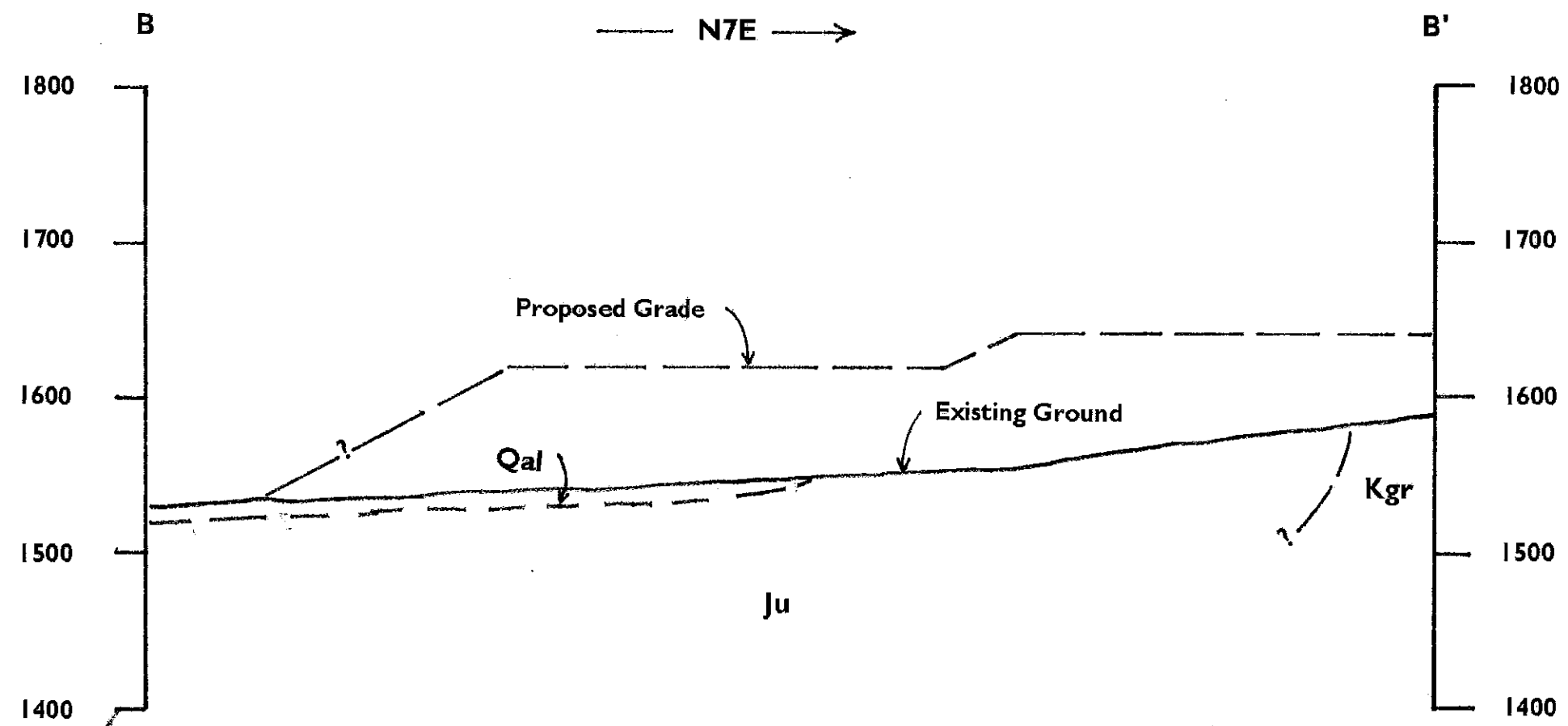
- Qal Quaternary Alluvium
- Qc Quaternary Culluvium
- Kgr Cretaceous Granitic Bedrock
- Ju Upper Jurassic Metasedimentary Bedrock
- TP-9 Approximate location of Exploratory Trench
- SL-5 Approximate Location of Seismic Line Refraction Line
- B-B' Approximate Location of Geologic Cross-Section
- - - Approximate Location of Geologic Contact
- 35° Strike and Dip of Bedding
- 80° Strike and Dip of Joint/Fraction

SCALE: 1" = 100'

Geotechnical Map		Plate 1
		710 E. Parkridge Ave. Suite 105 Corona, California 92879
GeoTek® Project No. 0967-CR3		Date 3/28/13



Cross Section A - A'	Plate 2
Project No. 0967-CR3	Scale: 1" = 100'



Cross Section B - B'	Plate 3
 GEOTEK	
Project No. 0967-CR3	Scale: 1" = 100'

APPENDIX A

LOGS OF EXPLORATORY EXCAVATIONS

**South Shore II Project, Tract 32013
Lake Elsinore, Riverside County, California
Project No. 0967-CR3**



A - FIELD TESTING AND SAMPLING PROCEDURES

The Standard Penetration Test (SPT)

The SPT is performed in accordance with ASTM Test Method D 1586. The SPT sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The split-barrel sampler has an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The samples of earth materials collected in the sampler are typically classified in the field, bagged, sealed and transported to the laboratory for further testing.

The Modified Split-Barrel Sampler (Ring)

The Ring sampler is driven into the ground in accordance with ASTM Test Method D 3550. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Bulk Samples (Large)

These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)

These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B - TRENCH LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of trenches:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C:	Contact line
.....	Dashed line denotes USCS material change
————	Solid Line denotes unit / formational change
————	Thick solid line denotes end of trench

(Additional denotations and symbols are provided on the logs of trenches)

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-I	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	EI = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-2	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
				MATERIAL DESCRIPTION AND COMMENTS			
5			SM	Alluvium: Gravelly silty f-m SAND (SM), light red brown, slightly moist, loose, porous, roots @ 1.5', becomes cobbly gravelly silty f-m SAND (SM), red brown, slightly moist, medium dense, metamorphic and granular clasts, layered			
10				Highly Weathered Granitic Bedrock: Excavates as f-m silty SAND, light red brown, damp			
15				TRENCH TERMINATED AT 10 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-3	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
5			SM	Alluvium: Clayey silty f-m SAND (SM), red brown, slightly moist, loose to medium dense, porous			
				Highly Weathered Granitic Bedrock: Excavates as f-m silty SAND, light red brown, damp			
10				TRENCH TERMINATED AT 8 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			
15							

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density
		SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test
				SA = Sieve Analysis
				CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-4	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
5			SM	Alluvium: Gravelly silty f-m SAND (SM) with cobbles, gray brown, damp, loose; meta-sedimentary clasts of gravel			EI
				Highly Weathered Meta-sedimentary Bedrock: Excavates as blocky, cobbly gravelly SAND, mottled yellow brown, very dense			
				TRENCH TERMINATED AT 6 FEET			
10				No Groundwater Encountered			
				No Caving			
15				Backfilled with Trench Spoils			

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	EI = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-5	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
			SM	Alluvium: Clayey silty f-m SAND (SM), red brown, slightly moist, loose, porous			
				Highly Weathered Meta-sedimentary Bedrock: Excavates as blocky, cobbly gravelly SAND, mottled yellow brown, very dense; Practical refusal @ 2'			
				TRENCH TERMINATED AT 2 FEET			
5				No Groundwater Encountered No Caving Backfilled with Trench Spoils			
10							
15							

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density
		SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test
				SA = Sieve Analysis
				CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-6	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
			SM	Alluvium: Clayey silty f-m SAND (SM), red brown, slightly moist, loose, porous			
5				Highly Weathered Granitic Bedrock: Excavates as f-m silty SAND, light red brown, damp			
10				TRENCH TERMINATED AT 9 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			
15							

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-7	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
				MATERIAL DESCRIPTION AND COMMENTS			
5			SM	Alluvium: Gravelly, cobbly, silty f-m SAND (SM), medium red brown, slightly moist, loose to medium dense, layered, meta-sedimentary clasts			
10				Highly Weathered Granitic Bedrock: Excavates as f-m silty SAND, light red brown, damp			
15				TRENCH TERMINATED AT 11.5 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-8	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
5			SM	Alluvium: Gravelly, cobbly, silty f-m SAND (SM), medium red brown, slightly moist, loose to medium dense, layered, meta-sedimentary clasts			
				Highly Weathered Granitic Bedrock: Excavates as f-m silty SAND, light red brown, damp			
10				TRENCH TERMINATED AT 8 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			
15							

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density
		SR = Sulfate/Resistivity Test	SH = Shear Testing	SA = Sieve Analysis
				RV = R-Value Test
				CO = Consolidation

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

PROJECT NO.: 0967-CR3
PROJECT NAME: Lake Elsinore
CLIENT: South Shore II, LLC
LOCATION: See Geotechnical Map

LOGGED BY: EHL
EQUIPMENT: Rubber Tire Backhoe
DATE: 3/18/2013

Depth (ft)	SAMPLES		USCS Symbol	TRENCH NO.: TP-9	Field Testing		Laboratory Testing
	Sample Type	Sample Number			Water Content (%)	Dry Density (pcf)	Others
	MATERIAL DESCRIPTION AND COMMENTS						
				Highly Weathered Meta-sedimentary Bedrock: Excavates as blocky, cobbly gravelly SAND, mottled yellow brown, very dense; Practical refusal @ 2'			SH, SR
				TRENCH TERMINATED AT 2 FEET No Groundwater Encountered No Caving Backfilled with Trench Spoils			
5							
10							
15							

LEGEND	Sample Type:	 --- Ring Sample	 --- Large Bulk Sample	 ---Water Table
	Laboratory Testing:			
	AL = Atterberg Limits	El = Expansion Index	MD = Maximum Density	SA = Sieve Analysis
	SR = Sulfate/Resistivity Test	SH = Shear Testing	RV = R-Value Test	CO = Consolidation

APPENDIX A.I

SEISMIC REFRACTION DATA

**South Shore II Project, Tract 32013
Lake Elsinore, Riverside County, California
Project No. 0967-CR3**



Subsurface Surveys & Associates, Inc.
2075 Corte Del Nogal, Suite W Carlsbad, CA 92011
 Phone: (760) 476-0492 Fax: (760) 476-0493

GeoTek. Inc.
710 East Parkridge Avenue, Suite 105
Corona, CA 92879

April 2, 2013

Attn: Ed LaMont Re: Seismic Survey Summary Report
 South Shore II Project Site – Lake Elsinore, CA

This report covers the results of a seismic refraction survey performed at the South Shore II Development Site in Lake Elsinore, California. The purpose of the survey was to measure the compressional wave velocity of bedrock for rippability assessment and to provide cross sections showing thickness of the weathered zone and depth to the unweathered interface. This should be useful for planning cuts and other earthwork.

The field work was conducted on March 19, 2013. Five seismic lines were recorded at locations selected by GeoTek. A survey location map is provided on Figure 1 that shows the position and orientation of the traverses.

GEOLOGIC SETTING

A review of the “Geologic Map of California, Santa Ana Sheet”, (California Division of Mines and Geology, 1966) indicates the local area is underlain by Mesozoic rock of three different types. They are described as follows: 1) marine sedimentary rock (Upper Jurassic age) that is locally metamorphosed 2) granitic rocks that are primarily tonalite and diorite and 3) basic intrusive rock. Surface deposits are mainly colluvium on the hillsides.

An unnamed fault is mapped between the I-15 freeway and the survey area. This feature trends NW-SE and is roughly parallel to the Glen Ivy Fault, which is 1-1.5 miles to the southwest.

DATA ACQUISITION AND FIELD METHODS

Seismic refraction data were recorded with a Bison 9024 signal enhancement seismograph and 30 Hz geophones. The standard spread layout used 24 geophones with a 10-foot spacing. Each spread used five shotpoints, one off each end (10-foot offset) and three within the interior of the spread. Depth of investigation was approximately 60-70 feet.

Compressional wave energy was created by sledge hammer impacts on a metal plate. The signal enhancement feature of the seismograph allowed returns from repeated hits to be stacked, thus

improving the signal. Each record was stored digitally on an internal hard disk and printed copies of each seismogram were made in the field on thermal paper. Example seismic records from this survey are shown on Figure 2.

Relative elevations of all shotpoints and geophones were determined by differential leveling with a hand level. Geophone 1 (distance = 0 ft.) at the beginning of each line was the reference station. All other elevation measurements along the line are relative to this point.

The endpoints of each seismic spread are marked by a labeled wooden stake. The latitude and longitude coordinates of the stakes were recorded with a hand held Garmin GPS receiver and used to prepare the line location map on Figure 1.

SEISMIC REFRACTION METHOD

The refraction method involves measuring the total time for compressional waves to travel from a shotpoint through the subsurface to a set of geophones placed linearly along the ground. Based on Snell's Law, when two or more layers are present with increasingly higher acoustic velocity, waves become critically refracted across the layer boundaries and begin traveling at the speed of the underlying layer. The advancing waves then generate new wavefronts back to the ground surface. The first surge of energy hitting the geophone is termed the "first arrival" and is depicted on the seismogram as a high angle deflection along each trace. Example field records from this survey that show the first arriving energy are provided on Figure 2.

Recognition of direct wave arrivals (non-refracted) verses refracted waves is a key element of refraction interpretation. To assist this process, the first arrival times measured from the seismic records are plotted on graphs of time verses distance called Time-Distance graphs. An example T-D graph from Line 2 is shown on Figure 3. Based on changes in slope on the graphs, a preliminary layer number (i.e. 1, 2, 3) is assigned to each segment of the graph. The layer assignments together with time, distance and elevation data are input to a computer for additional processing.

DATA REDUCTION AND VELOCITY DETERMINATION

Seismic data from this survey was processed using two methods for generating seismic velocity cross sections. One method produces layered earth models and uses the average velocity across the spread to calculate the thickness of the layers. This is the most widely used approach for rippability surveys because depths to refracting (velocity) horizons can be measured directly from the cross sections. Layered models are best applied when there is not a significant lateral variation in velocity along the line and the layer interfaces are relatively flat.

The second modeling approach uses what is referred to as tomographic inversion and produces velocity gradient cross sections in color. Tomography does not perform refraction layer calculations or attempt to measure discrete depths. Instead, the main objective is to create a

velocity distribution grid in the subsurface. Each node of the grid has a specific velocity associated with it. The goal is to adjust or “iterate” the velocity matrix so that the computer derived travel-time curves match what was recorded in the field. The final velocity grid is then loaded into a contouring program that produces color-filled cross sections. This method is typically used for imaging the shape and configuration of complex structures such as faults, landslides and intrusions, and areas where strong lateral velocity gradients are suspected within the weathered profile.

Layered Models

Processing and interpretation of this data set was accomplished with “SIPT2”, an interactive inversion modeling program developed by James Scott for the U.S. Bureau of Mines. The inversion algorithm uses the delay time method to construct a first pass depth model. The model is then adjusted by an iterative ray tracing process that attempts to minimize the discrepancies between the total travel times calculated along ray paths and the observed travel times measured in the field.

This program calculates refractor velocity in two ways. First, apparent velocities from each shot are determined by the inverse slope of a best fit (least squares) line through datum-corrected travel times. True velocity is estimated from the apparent velocities by using the following equation:

$$V_t = 2(V_u \times V_d)/(V_u + V_d)$$

where V_t = true velocity
 V_u = apparent up dip velocity
 V_d = apparent down dip velocity

The second method uses a more sophisticated set of equations (the Hobson-Overton formula) developed by the Canadian Geological Survey. The final velocity assigned to the refractor is a weighted average of the results of the two methods. The weighting is based on the number of arrival times used in the computations.

Velocity Gradient Models

The tomographic modeling program used for this survey is SeisOpt Version 3.5 from Optim LLC. It uses a proprietary inversion algorithm that applies a non-linear optimization technique called generalized simulated annealing to adjust the velocity grid points for the best statistical match. It is referred to as an optimization because it attempts to find the model that has the least minimum travel-time error between the calculated and observed (field) measurements.

SUMMARY OF RESULTS

Layered Velocity Cross Sections

Data from Lines 1, 2, and 3 were processed using our standard layer modeling software for rippability surveys. Results from refraction analysis show a three layer solution beneath these lines. Velocities posted on the cross sections represent averages as described in the previous section. Therefore, minor localized changes in velocity may occur along any profile. A description of the layers is provided below and a cross section summary is shown in Table 1.

Layer 1 - is mostly colluvium and highly decomposed and fractured bedrock.

Layer 2 - is interpreted to be weathered bedrock. The velocity range is 3114-4142 ft/sec and is considered rippable with a Caterpillar D-9.

Layer 3 - represents hard slightly weathered to unweathered bedrock.

Table 1. Cross Section Summary

<u>Line</u>	<u>Velocity Layer 1</u>	<u>Velocity Layer 2</u>	<u>Velocity Layer 3</u>	<u>Depth Range Layer 2/3 Interface</u>
1	1559	4722	8479	38 - 44
2	1635	3629	7407	37 - 50
3	1474	3507	ND	> 60

Velocity in (ft/sec), Depth in (feet)

ND - not detected

Weathering tends to be gradational for most rock types and usually produces a gradual increase in velocity with depth. Consequently, variation of $\pm 10\%$ from the posted averages may occur between the top and bottom of Layer 2.

Figure 4 presents a rippability chart (courtesy of Caterpillar Tractor Co.) for a D9R Ripper. Bar graphs show the relationship between seismic compressional wave velocity and ripper performance for various rock types in three categories: rippable, marginal, and non-rippable. Metamorphic rocks are listed as marginally rippable at approximately 7200 ft/sec and are considered non-rippable above 9000 ft/sec. Granitic rocks are marginally rippable at 6800 ft/sec and non-rippable above 8000 ft/sec. This chart is provided only as a guide and should not be considered absolute. Other geologic factors that may influence bedrock rippability at this site include the degree and grade of metamorphism, the presence of fractures, joints, possible intrusive bodies and changes in rock composition.

Velocity Gradient Models

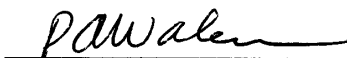
Data from Lines 4 and 5 showed highly irregular refraction horizons and significant lateral changes in velocity across the spreads. The layer modeling program, that does ray tracing and uses the average velocity of each layer to calculate depths, was smoothing and flattening the main features and produced unreliable results. To maintain the true geometry and provide better resolution, a more robust modeling approach was applied that uses tomographic inversion processing. Results are displayed on Figures 8 and 9.

The geologic structures shown on the color models could be related to granitic intrusion or zones of highly metamorphosed and hardened metasediments. These complex features were not observed on Lines 1, 2, and 3.

The tomographic modeling for Line 4 produced a velocity range of 1192-6151 ft/sec and the range for Line 5 was 1181-7527 ft/sec. The high end values are close to or within the marginally rippable range for granitic and metamorphic rock. If ridge line cuts below Lines 4 and 5 are greater than 35-40 feet, ripping may be difficult across these high velocity zones.

All data acquired during this survey is considered confidential and is available for review by your staff at any time. We appreciate the opportunity to participate in this project.

Please call if there are any questions.



Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Seismic Survey Location Map
South Shore II Project Site -- Lake Elsinore, CA

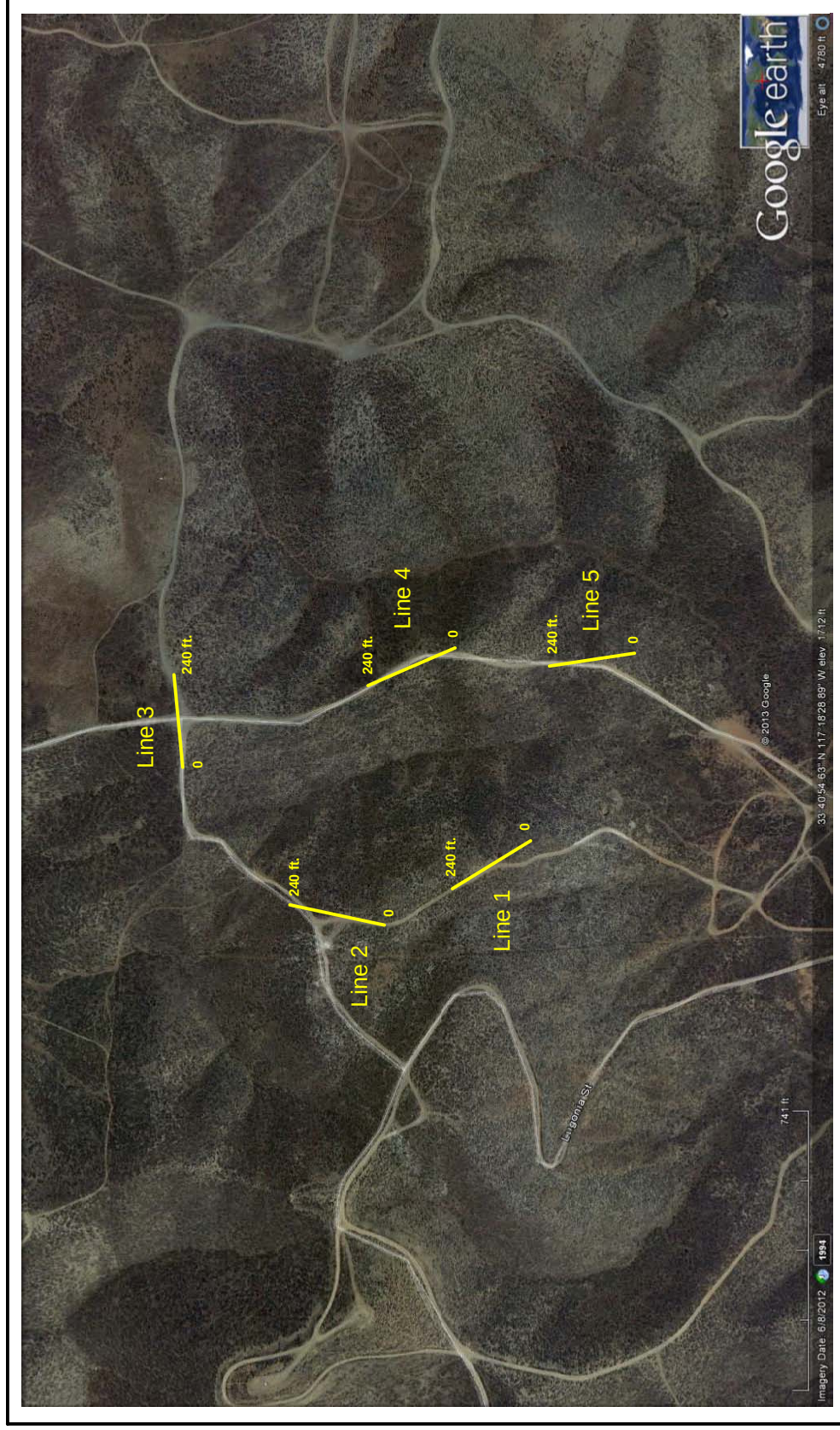


Figure 1

Example Seismic Field Records

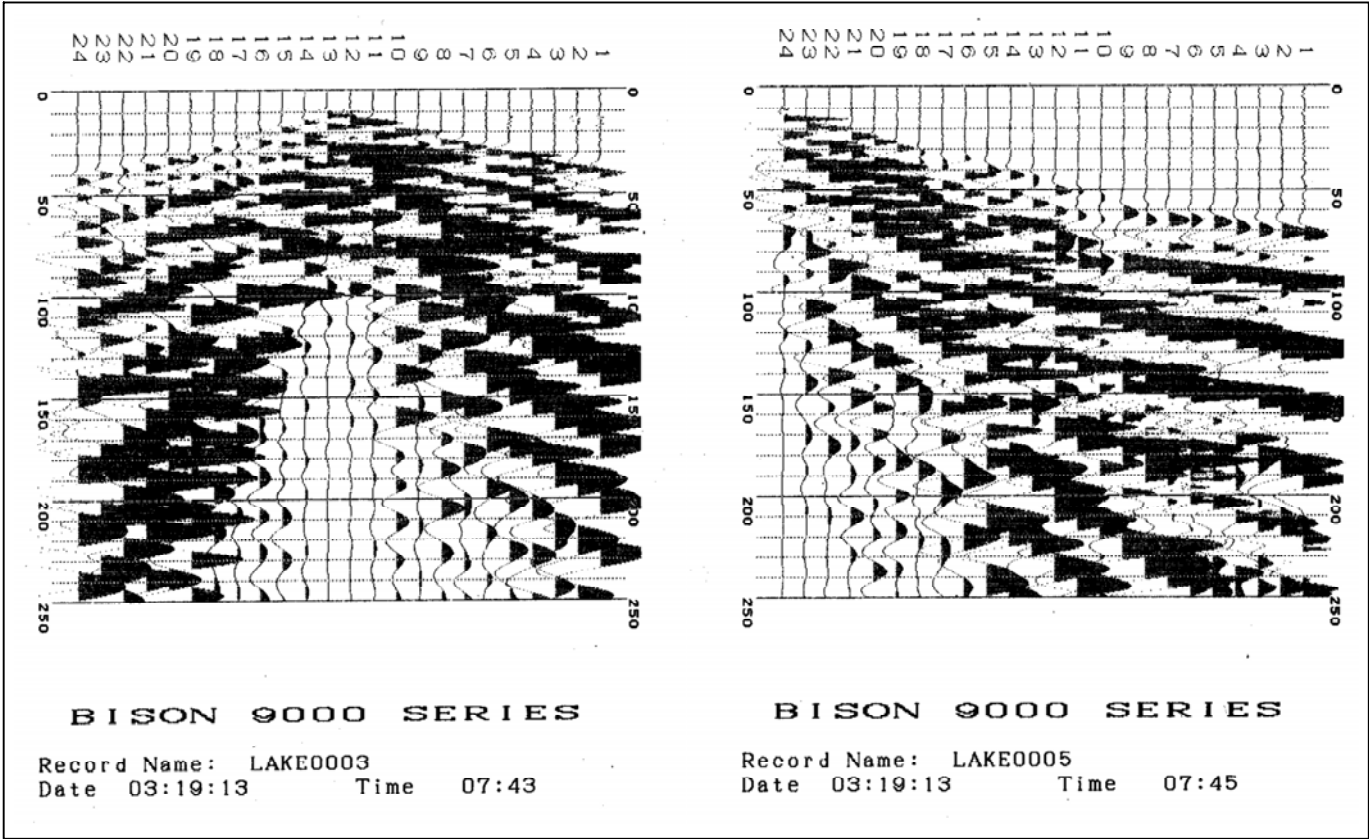
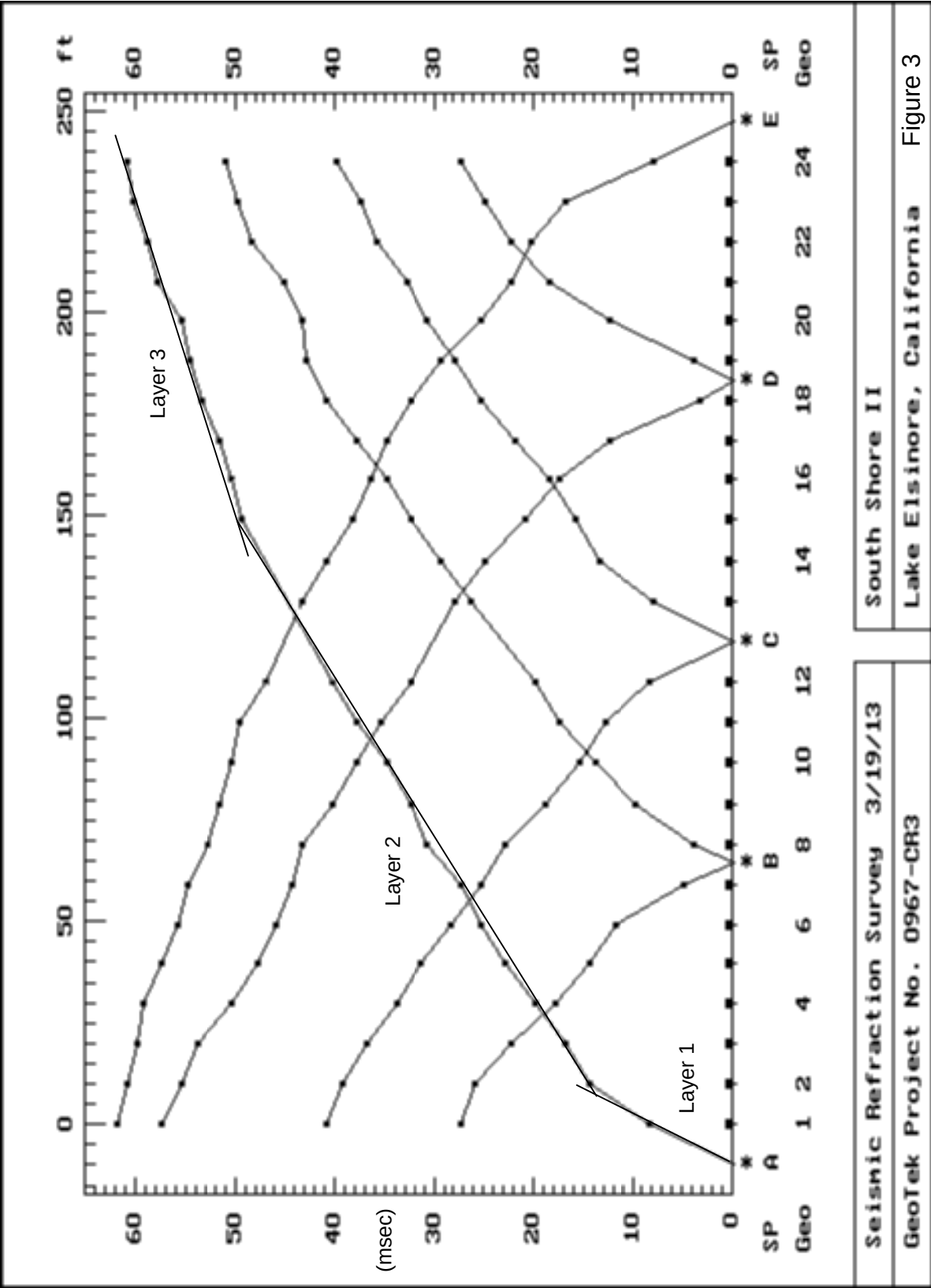


Figure 2

Example Time-Distance Graph -- Line 2



Rippers

Ripper Performance

• D9R/D9T

D9R/D9T

• Multi or Single Shank No. 9 Ripper

• Estimated by Seismic Wave Velocities

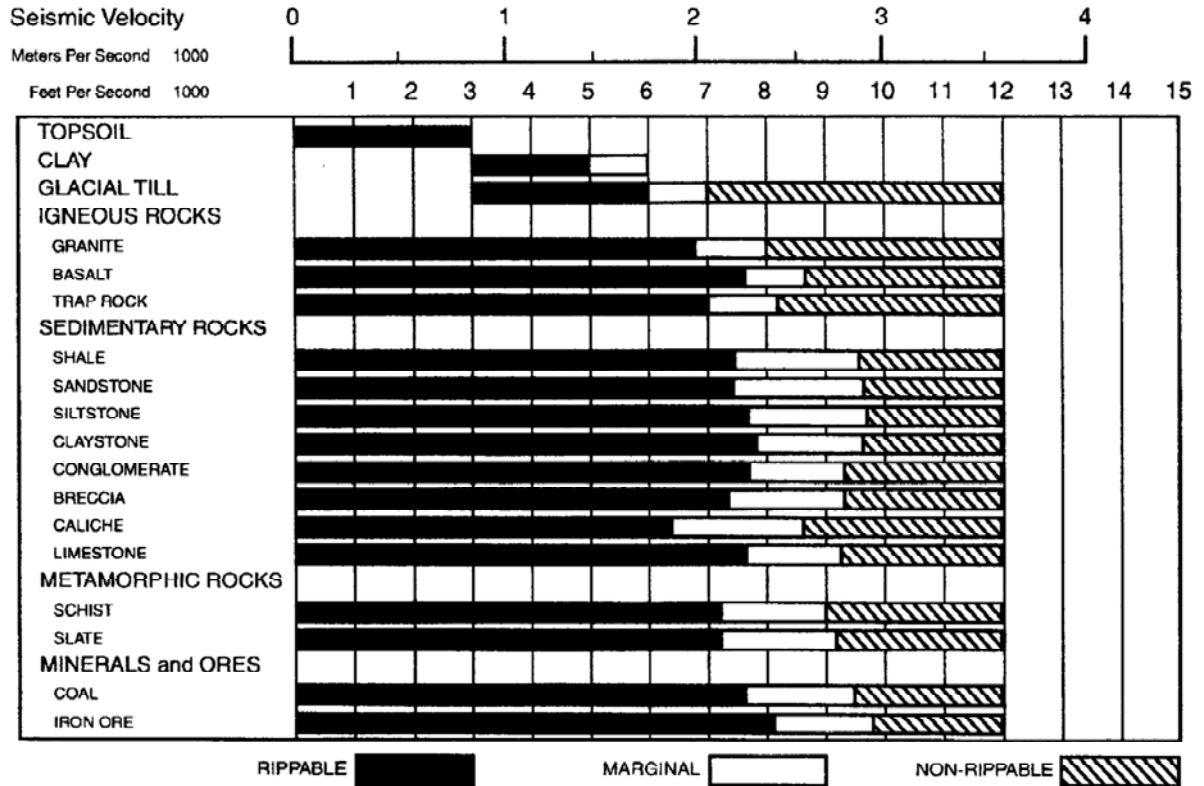
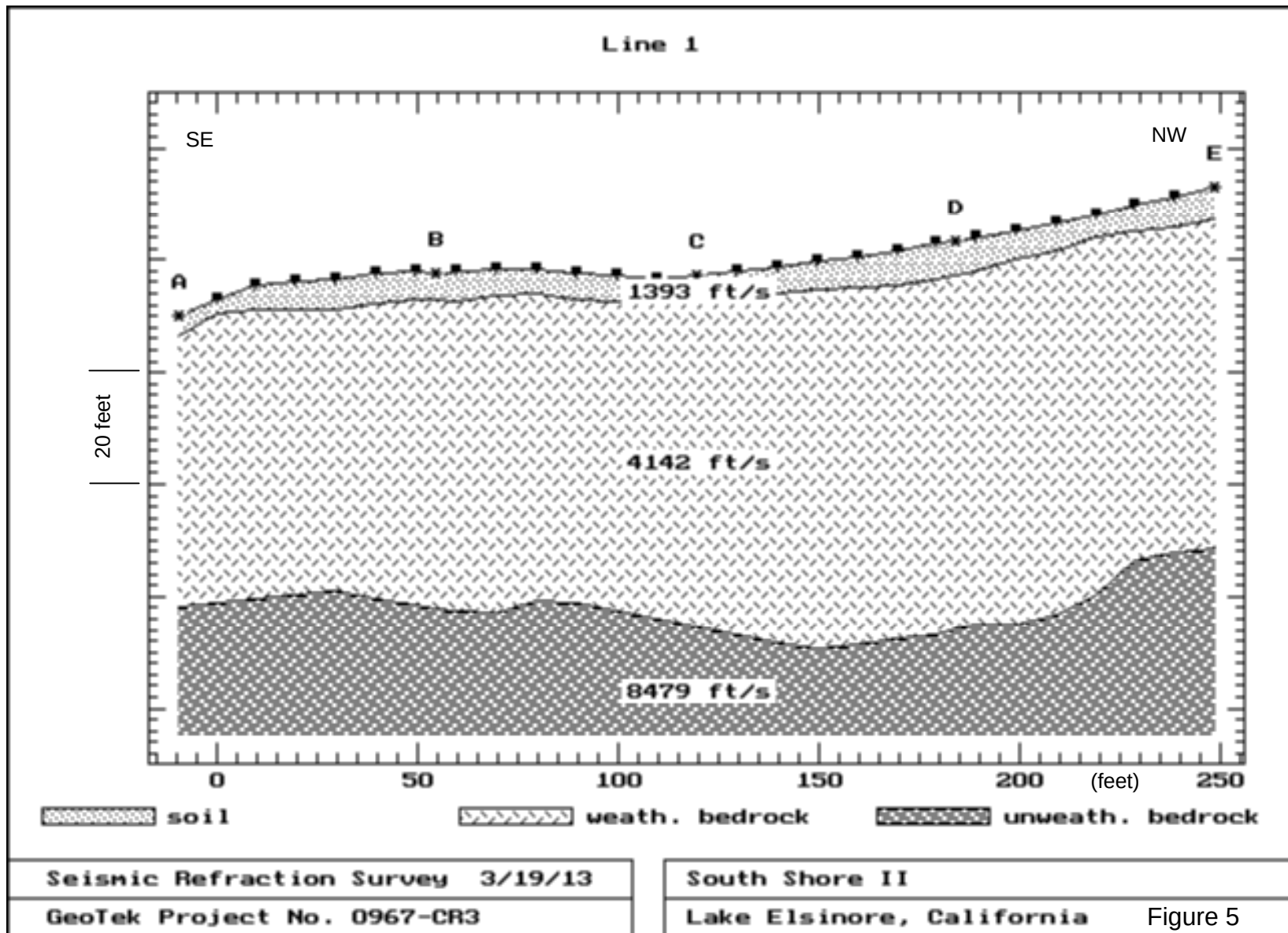
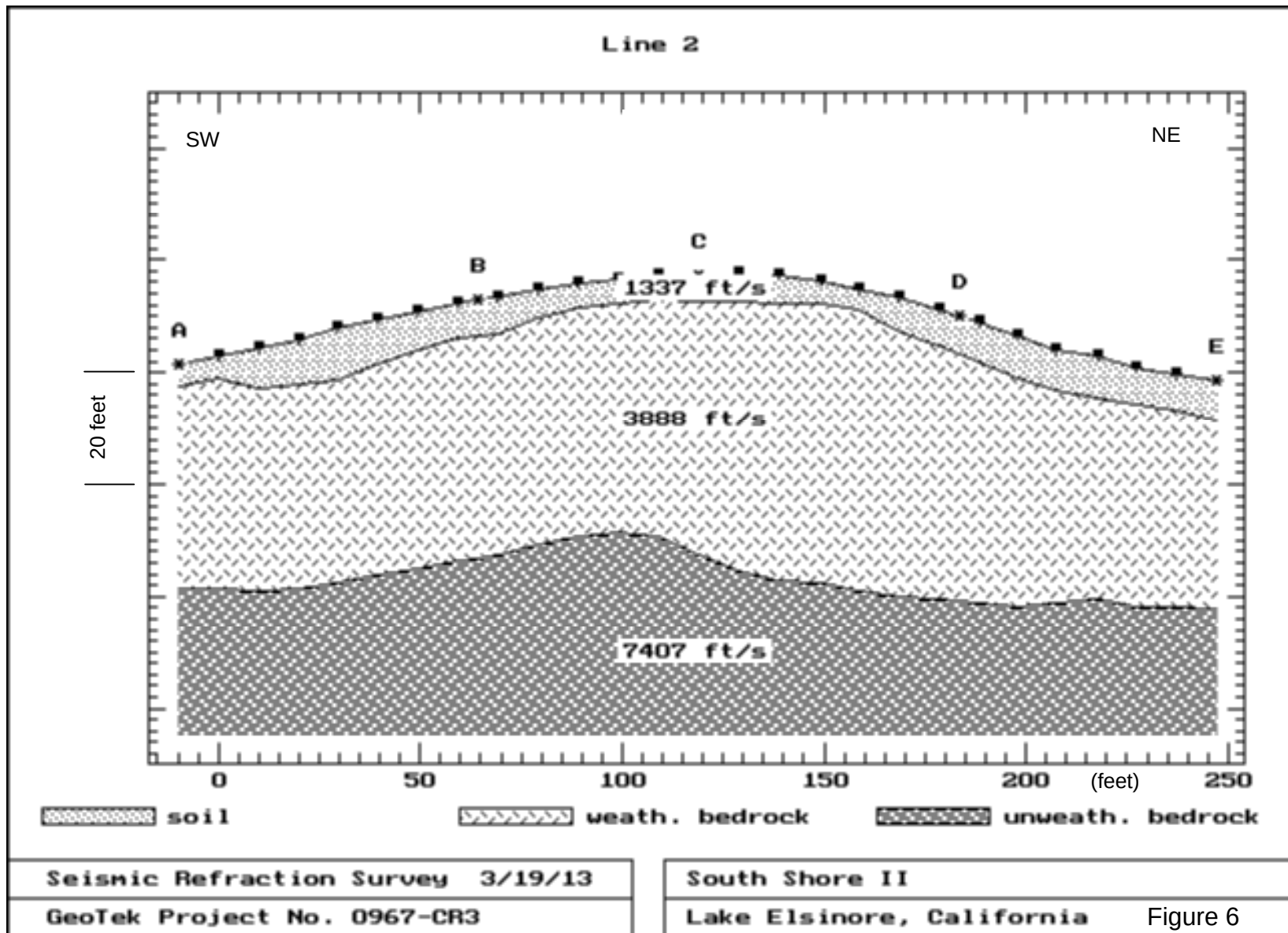
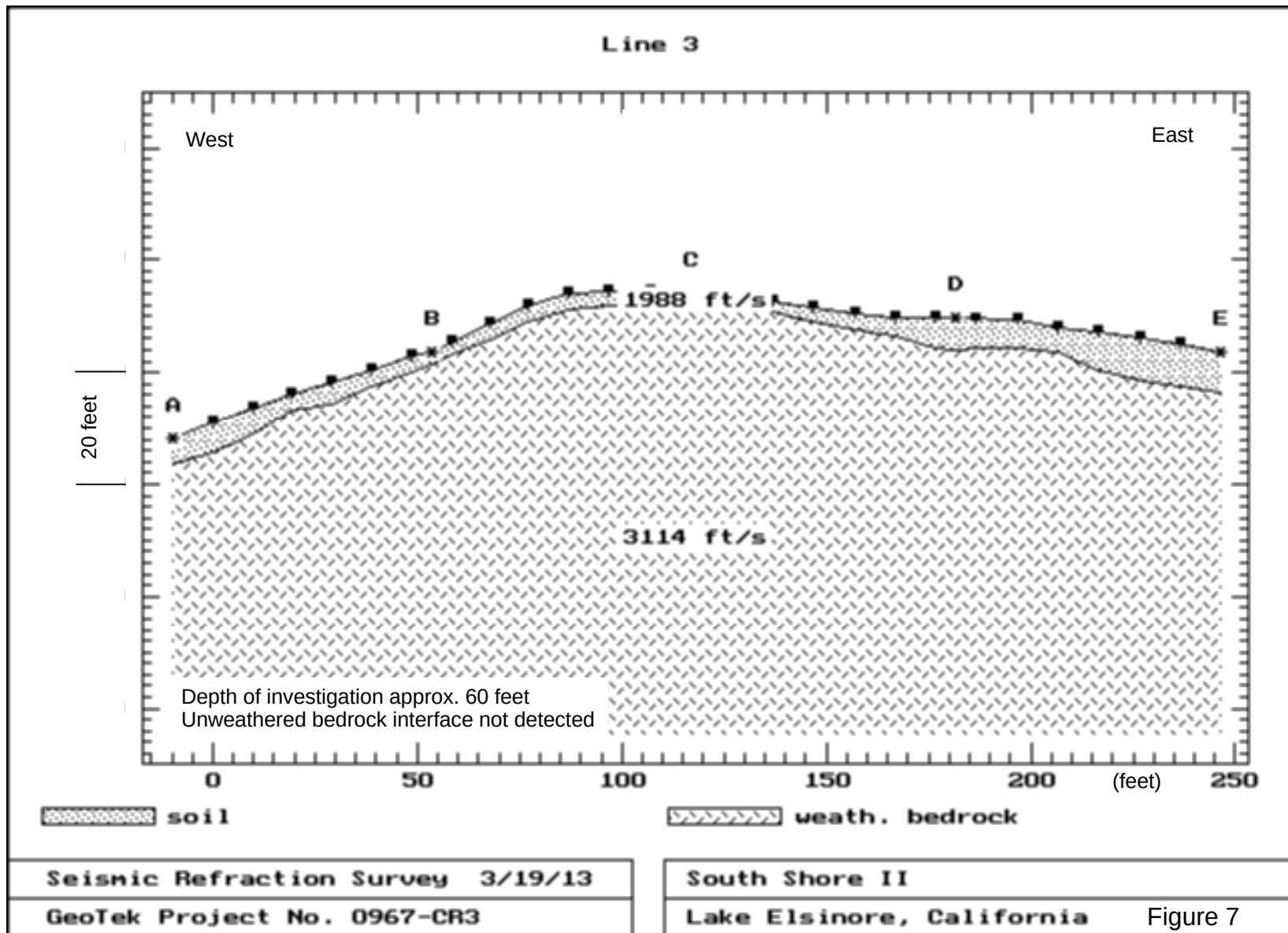


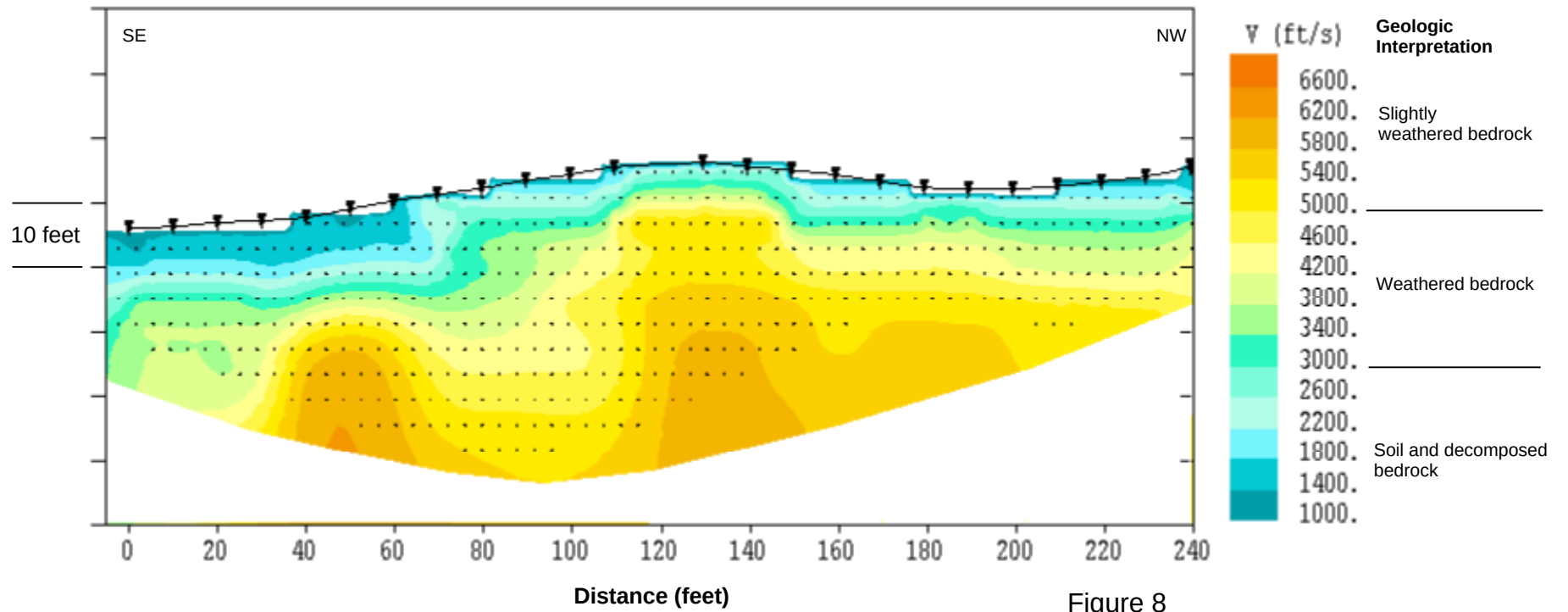
Figure 4



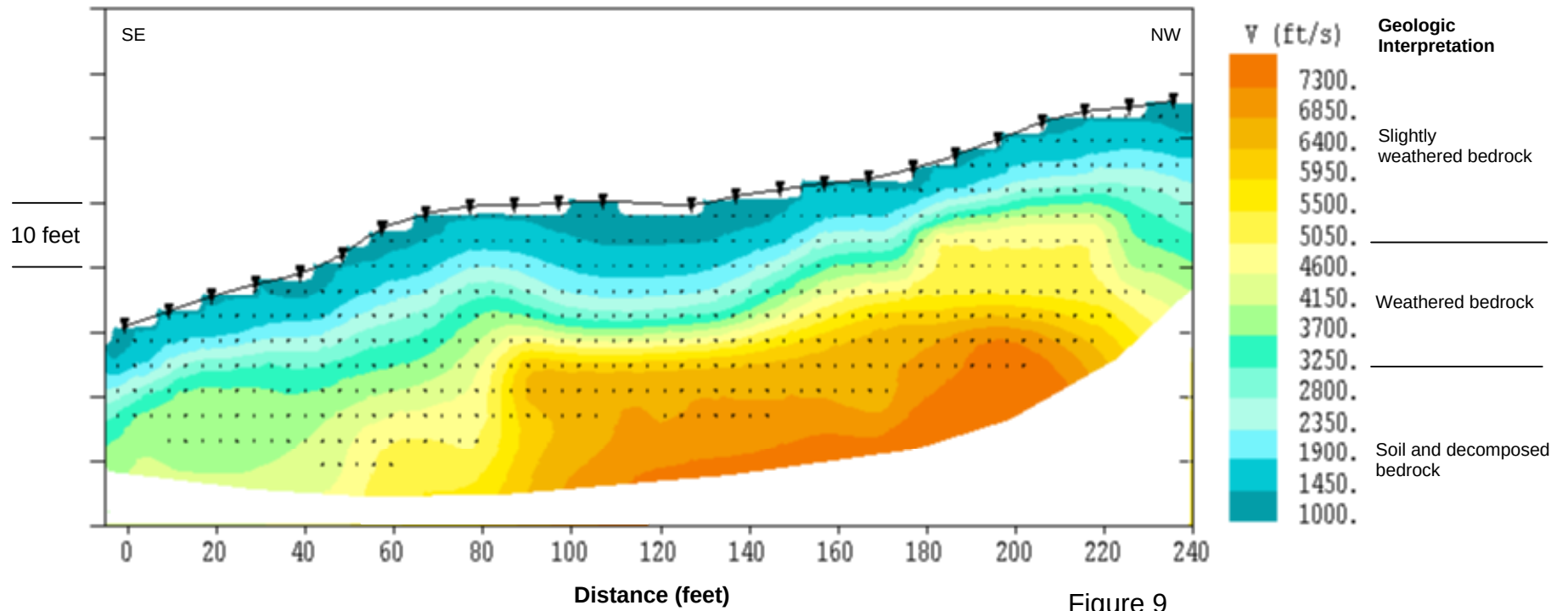




Velocity Gradient Model -- Line 4



Velocity Gradient Model -- Line 5



APPENDIX B

RESULTS OF LABORATORY TESTING

**South Shore II Project, Tract 32013
Lake Elsinore, Riverside County, California
Project No. 0967-CR3**



SUMMARY OF LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soil Classification System (ASTM Test Method D2487). The soil classifications are shown on the logs of exploratory trenches included in Appendix A.

Moisture-Density Relations

Laboratory testing was performed on a representative site samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for representative soil types was determined in general accordance with test method ASTM Test Procedure D 1557. The most recent results are included herein on Plates MD-1 through MD-3.

Expansion Index

Expansion Index testing was performed on soil samples collected during our field explorations. Testing was performed in general accordance with ASTM Test Method D 4829. The Expansion Index (EI) test results indicate a very low expansion potential for the materials tested. The results are included herein on Plates EI-1 through EI-2.

Direct Shear

Shear testing was performed on remolded samples of the site soil materials in general accordance with ASTM Test Method D 3080. The test results are included herein on Plates SH-1 and SH-2.

Sulfate Content

Analyses to determine the water-soluble sulfate content was performed by others in general accordance with California Test No. 417. The results are included herein.

Resistivity

Soil samples collected at the site were tested by others for resistivity in accordance with California Test 643. The results of the testing are included herein.



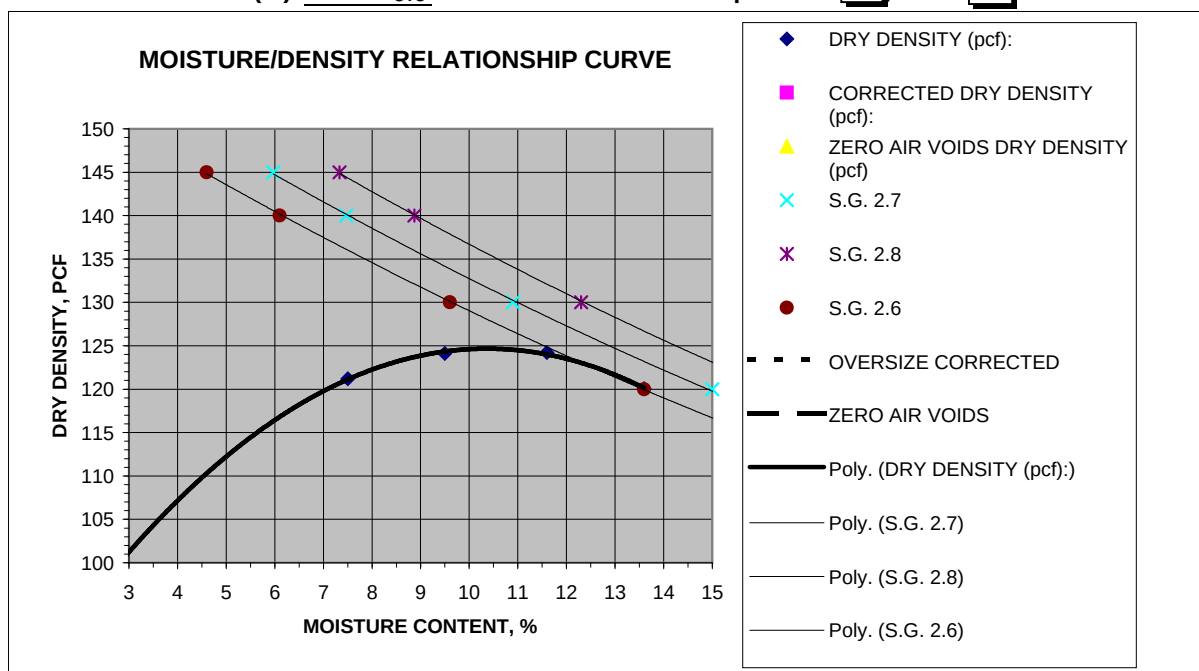
MOISTURE/DENSITY RELATIONSHIP

Client: South Shore II, LLC
 Project: South Shore II
 Location: Lake Elsinore
 Material Type: Light Brown Sand
 Material Supplier: N/A
 Material Source: N/A
 Sample Location: T - 1
@ 8 - 9'
 Sampled By: EHL
 Received By: N/A
 Tested By: DI
 Reviewed By: N/A

Job No.: 0967-CR3
 Lab No.: Corona

Date Sampled: 18-Mar-13
 Date Received: 19-Mar-13
 Date Tested: 22-Mar-13
 Date Reviewed: 3-Apr-13

Test Procedure: ASTM 1557 Method: A
 Oversized Material (%): 0.0 Correction Required: ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	125.0	@ Optimum Moisture, %	10.5
Corrected Maximum Dry Density, pcf		@ Optimum Moisture, %	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Classification:

Unified Soils Classification: _____
 AASHTO Soils Classification: _____

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

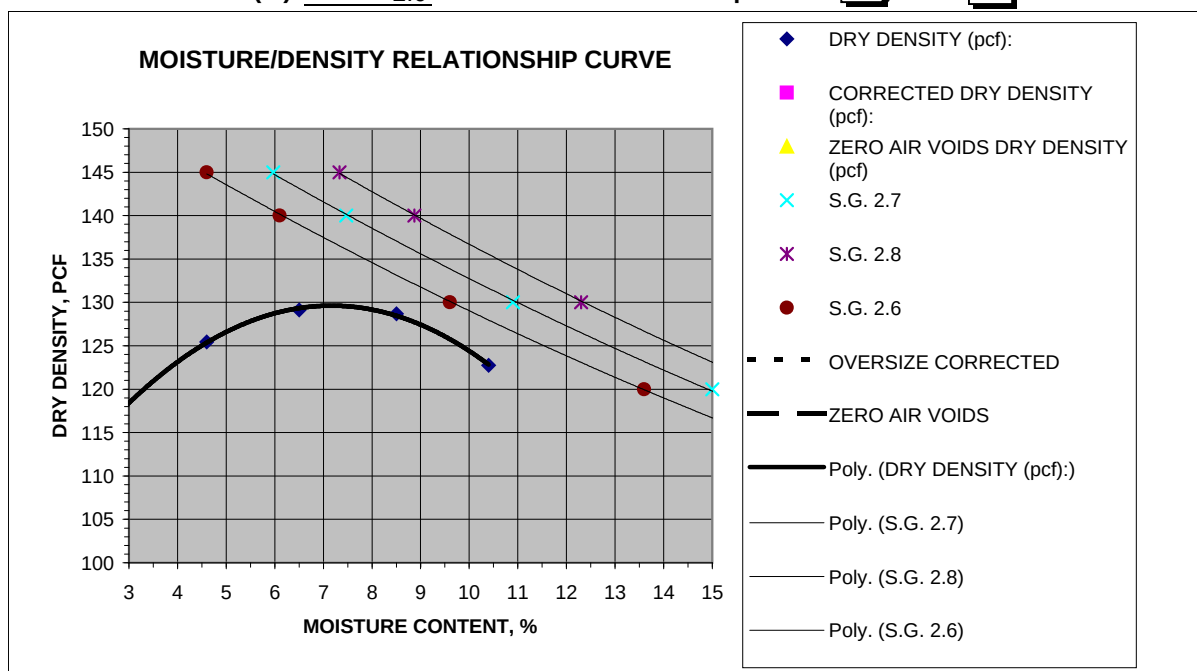


Job No.: 0967-CR3

Lab No.: Corona

Date Sampled:	18-Mar-13
Date Received:	19-Mar-13
Date Tested:	25-Mar-13
Date Reviewed:	3-Apr-13

Test Procedure: ASTM 1557 Method: A
Oversized Material (%): 2.0 Correction Required: ☐ yes ☒ no



Maximum Dry Density, pcf	129.5	@ Optimum Moisture, %	7.5
Corrected Maximum Dry Density, pcf		@ Optimum Moisture, %	

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %



EXPANSION INDEX TEST

(ASTM D4829)

Client: South Shore II, LLC
Project Number: 0967-CR3
Project Location: South Shore II

Tested/ Checked By: DI Lab No Corona
Date Tested: 3/22/2013
Sample Source: T - 1 @ 1 - 2'
Sample Description: Brown Silty Sand

Ring #: _____ Ring Dia. : 4.01" Ring Ht. 1.1"
Loading weight: 5516. grams

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	786.3
B	Weight of ring (gm)	365.8
C	Net weight of sample (gm)	420.5
D	Wet Density, lb / ft ³ (C*0.3016)	127.0
E	Dry Density, lb / ft ³ (D/1.1.F)	117.8

SATURATION DETERMINATION

F	Moisture Content, %	7.8
G	Specific Gravity, assumed	2.7
H	Unit Wt. of Water @ 20 °C, (pcf)	62.3
I	% Saturation	49

READINGS		
DATE	TIME	READING
3/22/2013	9:17	0.0810
3/22/2013	9:27	0.0810
3/22/2013	9:28	0.0810
3/22/2013	9:33	0.0810
3/22/2013	13:05	0.0810
3/25/2013	5:30	0.0810

Initial
10 min/Dry
1 min/Wet
5 min/Wet
Random
Final

FINAL MOISTURE		
Weight of wet sample & tare	Weight of dry sample & tare	% Moisture
604.6	536.6	18.5%

EXPANSION INDEX = 0

PLATE EI-1



EXPANSION INDEX TEST

(ASTM D4829)

Client: South Shore II, LLC
Project Number: 0967-CR3
Project Location: South Shore II

Tested/ Checked By: DI Lab No Corona
Date Tested: 3/22/2013
Sample Source: T - 4 @ 2 - 3'
Sample Description: Gray Brown Gravelly Silty Sand

Ring #: _____ Ring Dia. : 4.01" Ring Ht. 1.1"
Loading weight: 5516. grams

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	770.5
B	Weight of ring (gm)	363.7
C	Net weight of sample (gm)	406.8
D	Wet Density, lb / ft ³ (C*0.3016)	122.9
E	Dry Density, lb / ft ³ (D/1.1.F)	112.7

SATURATION DETERMINATION

F	Moisture Content, %	9.0
G	Specific Gravity, assumed	2.7
H	Unit Wt. of Water @ 20 °C, (pcf)	62.3
I	% Saturation	49

READINGS		
DATE	TIME	READING
3/22/2013	9:43	0.2330
3/22/2013	9:53	0.2330
3/22/2013	9:54	0.2330
3/22/2013	9:59	0.2330
3/22/2013	13:05	0.2370
3/25/2013	5:30	0.2370

Initial
10 min/Dry
1 min/Wet
5 min/Wet
Random
Final

FINAL MOISTURE		
Weight of wet sample & tare	Weight of dry sample & tare	% Moisture
580.4	507.4	20.7%

EXPANSION INDEX = 4

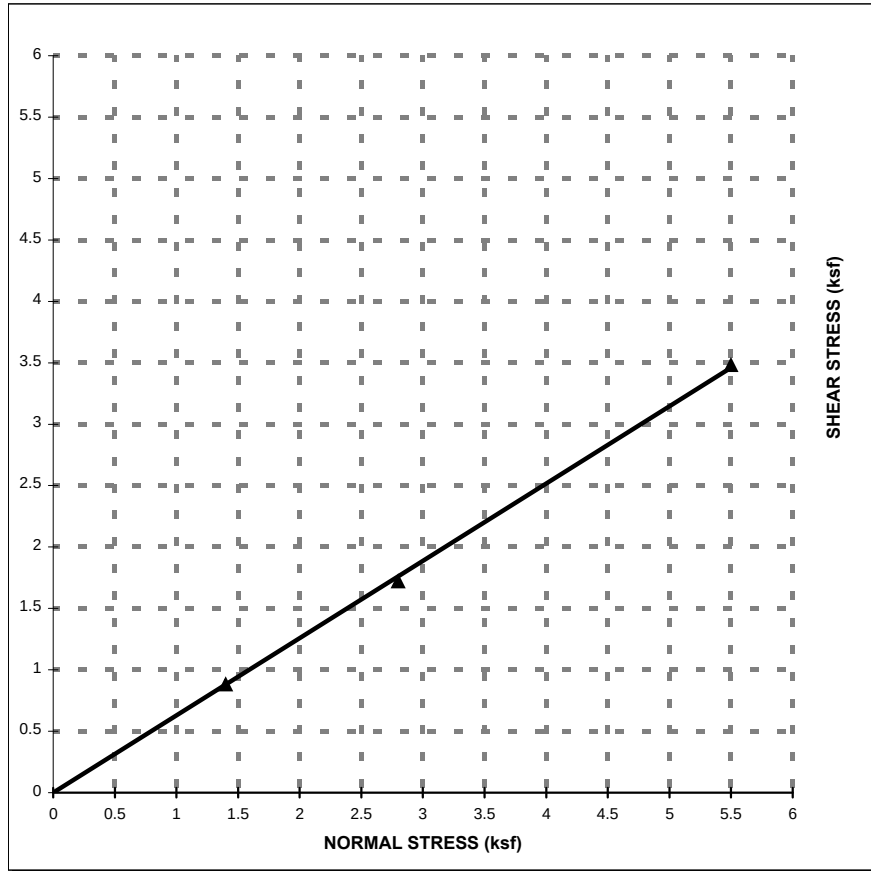
PLATE EI-2



DIRECT SHEAR TEST

Client: South Shore II, LLC
Project Number: 0967-CR3
Soil Description: Light Brown Sand

Sample Source: T-1 @ 8-9'
Date Tested: 3/29/2013



Shear Strength: $\phi = 32.2^{\circ}$; **C = 0.00 ksf**

Test No.	Load (ksf)	Final Water Content (%)	Final Dry Density (pcf)
1	1.4	18.0	108.9
2	2.8	17.3	109.2
3	5.5	16.3	108.8

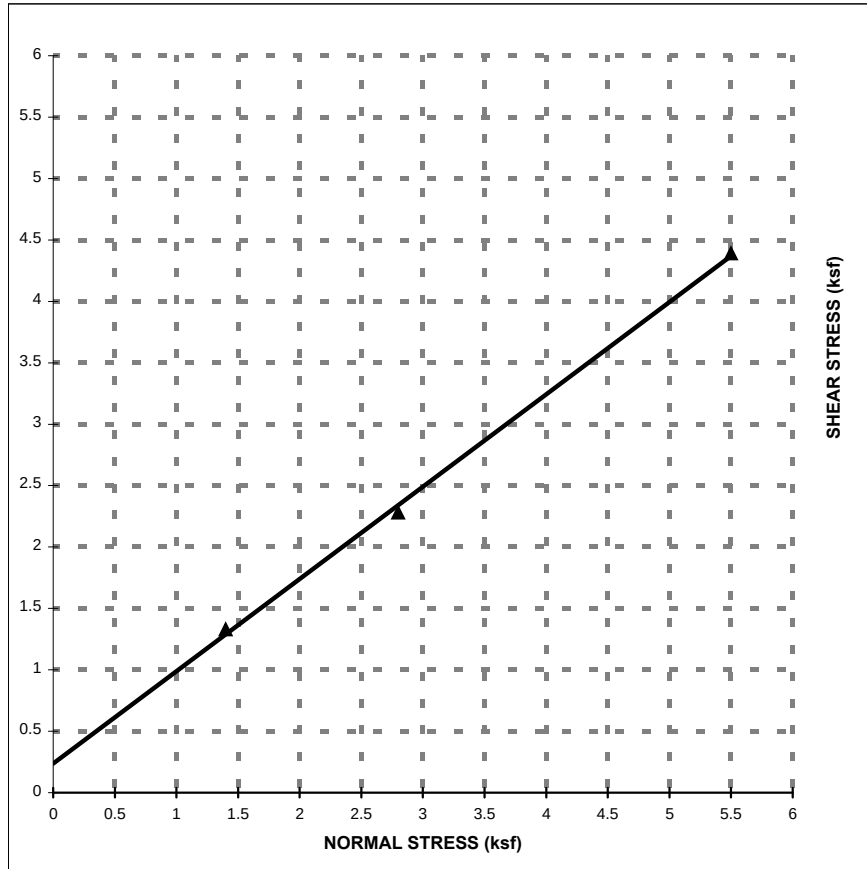
- Notes:**
- 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% from a bulk sample collected during the field investigation.
 - 2 - The above reflect residual shear strength at saturated conditions.
 - 3 - The tests were ran at a shear rate of 0.025 in/min.



DIRECT SHEAR TEST

Client: South Shore II, LLC
Project Number: 0967-CR3
Soil Description: Gray Brown Gravelly Silty Sand

Sample Source: T-9 @ 0-2'
Date Tested: 3/29/2013



Shear Strength: $\phi = 36.9^\circ$; $C = 0.23 \text{ ksf}$

Test No.	Load (ksf)	Final Water Content (%)	Final Dry Density (pcf)
1	1.4	17.8	112.1
2	2.8	17.4	113.9
3	5.5	17.1	113.8

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% from a bulk sample collected during the field investigation.
 2 - The above reflect residual shear strength at saturated conditions.
 3 - The tests were ran at a shear rate of 0.025 in/min.

Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental & Civil Engineering

March 25, 2013

Geo Tek Inc.
710 East Parkridge Avenue, Suite 105
Corona, California 92879

Attn: Mr. Edward Lamont

RE: LABORATORY TEST RESULTS/REPORT

Client: So. Shore LLC
W.O. 0967- CR3
Project: So. Shore II
QCI Job No.: 13-167-03d

Gentlemen:

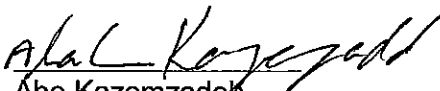
We have completed the testing program conducted on samples from the above project. The tests were performed in accordance with testing procedures as follows:

TEST	METHOD
Corrosion Potential	CT- 417, CT- 422, CT-532 (643)

Enclosed is Summary of Laboratory Test Results.

We appreciate the opportunity to provide testing services to Geo Tek, Inc. Should you have any questions, please call the undersigned.

Respectfully submitted,
Cal Land Engineering, Inc. (CLE)
dba Quartech Consultants (QCI)


Abe Kazemzadeh
Laboratory Manager

Enclosure

Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental, and Civil Engineering

For: GeoTek, Inc.
W.O.: 0967 - CR3
Client: So. Shore LLC
Project: So. Shore II

Date: March 25, 2013
QCI Project No.: 13-167-03d
Summarized by: ABK

Corrosivity Test Results

Sample ID	Depth (Feet)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 (% By Weight)	Resistivity CT-532 (643) (ohm-cm)
TP-1	1 - 2	8.86	95	0.0050	5,100
TP-1	8 - 9	8.24	127	0.0020	6,900
TP-9	0 - 2	7.84	141	0.0015	3,800

APPENDIX C

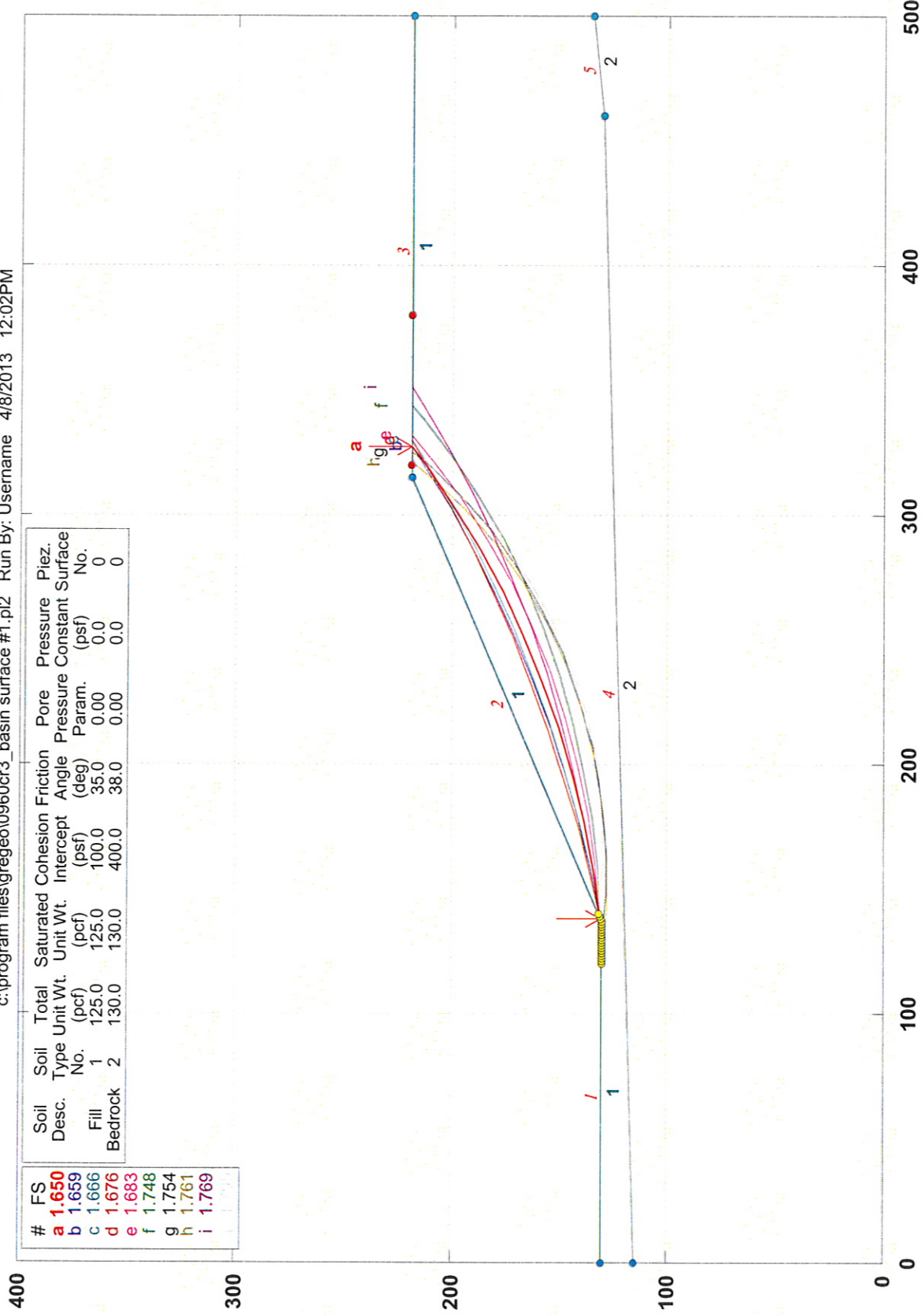
COMPUTER PRINTOUTS OF SLOPE STABILITY ANALYSES

**South Shore II Project, Tract 32013
Lake Elsinore, Riverside County, California
Project No. 0967-CR3**



Proj. No. 0967-CR3 Highest 2:1 (h:v) Fill Slope Static

c:\program files\gregeol\0960cr3_basin surface #1.pl2 Run By: Username 4/8/2013 12:02PM



GSTABL7 v.2 FSmin=1.650

Safety Factors Are Calculated By The Modified Bishop Method



*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/Pile, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 4/8/2013

Time of Run: 12:02PM

Run By: Username

Input Data Filename: C:\Program Files\GREGE0\0960cr3_basin Surface #1.in

Output Filename: C:\Program Files\GREGE0\0960cr3_basin Surface #1.OUT

Unit System: English

Plotted Output Filename: C:\Program Files\GREGE0\0960cr3_basin Surface #1.PLT

PROBLEM DESCRIPTION: Proj. No. 0967-CR3 Highest

2:1 (h:v) Fill Slope Static

BOUNDARY COORDINATES

3 Top Boundaries

5 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	130.00	137.00	130.00	1
2	137.00	130.00	315.00	219.00	1
3	315.00	219.00	500.00	219.00	1
4	0.00	115.00	460.00	130.00	2
5	460.00	130.00	500.00	135.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	100.0	35.0	0.00	0.0	0
2	130.0	130.0	400.0	38.0	0.00	0.0	0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

500 Trial Surfaces Have Been Generated.

25 Surface(s) Initiate(s) From Each Of 20 Points Equally Spaced
 Along The Ground Surface Between X = 120.00(ft)
 and X = 140.00(ft)

Each Surface Terminates Between X = 320.00(ft)
 and X = 380.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is Y = 0.00(ft)

20.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial

Failure Surfaces Evaluated. They Are
 Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 500

Number of Trial Surfaces With Valid FS = 500

Statistical Data On All Valid FS Values:

FS Max = 4.096 FS Min = 1.650 FS Ave = 2.915

Standard Deviation = 0.660 Coefficient of Variation = 22.66 %

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	137.895	130.447
2	157.617	133.766

3	177.122	138.189
4	196.348	143.701
5	215.233	150.286
6	233.718	157.922
7	251.744	166.585
8	269.254	176.248
9	286.194	186.881
10	302.509	198.449
11	318.148	210.916
12	327.193	219.000

Circle Center At X = 89.175 ; Y = 481.202 ; and Radius = 354.122

Factor of Safety

*** 1.650 ***

Slice No.	Width (ft)	Weight (lbs)	Individual data on the 12 slices		Tie		Earthquake		Surcharge Load (lbs)
			Water Force Top (lbs)	Water Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Force Hor (lbs)	Force Ver (lbs)	
1	19.7	8064.7	0.0	0.0	0.	0.	0.0	0.0	0.0
2	19.5	22448.5	0.0	0.0	0.	0.	0.0	0.0	0.0
3	19.2	33457.9	0.0	0.0	0.	0.	0.0	0.0	0.0
4	18.9	41078.4	0.0	0.0	0.	0.	0.0	0.0	0.0
5	18.5	45365.7	0.0	0.0	0.	0.	0.0	0.0	0.0
6	18.0	46444.1	0.0	0.0	0.	0.	0.0	0.0	0.0
7	17.5	44504.8	0.0	0.0	0.	0.	0.0	0.0	0.0
8	16.9	39802.6	0.0	0.0	0.	0.	0.0	0.0	0.0
9	16.3	32651.7	0.0	0.0	0.	0.	0.0	0.0	0.0
10	12.5	19437.8	0.0	0.0	0.	0.	0.0	0.0	0.0
11	3.1	3674.3	0.0	0.0	0.	0.	0.0	0.0	0.0
12	9.0	4570.1	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	137.895	130.447
2	157.340	135.127
3	176.571	140.617
4	195.556	146.907
5	214.261	153.988
6	232.652	161.847
7	250.698	170.468
8	268.368	179.839
9	285.628	189.941
10	302.451	200.758
11	318.805	212.270
12	327.562	219.000

Circle Center At X = 36.620 ; Y = 594.758 ; and Radius = 475.227

Factor of Safety

*** 1.659 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	140.000	131.500
2	159.471	136.070
3	178.732	141.455
4	197.750	147.646
5	216.491	154.631
6	234.920	162.399
7	253.007	170.935
8	270.719	180.225
9	288.024	190.251
10	304.892	200.997
11	321.293	212.443
12	329.893	219.000

Circle Center At X = 41.799 ; Y = 594.419 ; and Radius = 473.220

Factor of Safety

*** 1.666 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	137.895	130.447
2	157.219	135.603
3	176.349	141.438
4	195.260	147.947
5	213.929	155.121
6	232.333	162.950
7	250.448	171.426
8	268.252	180.537
9	285.723	190.273
10	302.838	200.620
11	319.577	211.566
12	330.110	219.000

Circle Center At X = 2.572 ; Y = 677.076 ; and Radius = 563.130
Factor of Safety
*** 1.676 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	140.000	131.500
2	159.918	133.313
3	179.663	136.493
4	199.143	141.024
5	218.265	146.886
6	236.938	154.050
7	255.073	162.482
8	272.585	172.143
9	289.391	182.986
10	305.410	194.960
11	320.567	208.009
12	331.685	219.000

Circle Center At X = 123.834 ; Y = 420.626 ; and Radius = 289.577
Factor of Safety
*** 1.683 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	138.947	130.974
2	158.913	132.147
3	178.756	134.645
4	198.390	138.456
5	217.726	143.565
6	236.680	149.948
7	255.168	157.576
8	273.108	166.417
9	290.421	176.431
10	307.029	187.574
11	322.859	199.797
12	337.842	213.045
13	343.737	219.000

Circle Center At X = 131.304 ; Y = 431.537 ; and Radius = 300.661
Factor of Safety
*** 1.748 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	127.368	130.000
2	147.290	128.228
3	167.289	128.411
4	187.175	130.548
5	206.756	134.616
6	225.847	140.579
7	244.264	148.378
8	261.830	157.939
9	278.378	169.171
10	293.750	181.966

11 307.799 196.201
 12 320.389 211.741
 13 325.176 219.000
 Circle Center At X = 155.420 ; Y = 332.527 ; and Radius = 204.460
 Factor of Safety
 *** 1.754 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	122.105	130.000
2	141.971	127.686
3	161.969	127.399
4	181.893	129.141
5	201.538	132.895
6	220.700	138.621
7	239.184	146.260
8	256.797	155.735
9	273.359	166.947
10	288.699	179.780
11	302.658	194.102
12	315.093	209.767
13	321.004	219.000

Circle Center At X = 154.795 ; Y = 324.252 ; and Radius = 196.983
 Factor of Safety
 *** 1.761 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	140.000	131.500
2	159.748	134.664
3	179.337	138.699
4	198.728	143.596
5	217.884	149.346
6	236.767	155.937
7	255.339	163.356
8	273.566	171.590
9	291.410	180.622
10	308.838	190.434
11	325.815	201.007
12	342.307	212.320
13	351.177	219.000

Circle Center At X = 78.352 ; Y = 579.431 ; and Radius = 452.154
 Factor of Safety
 *** 1.769 ***

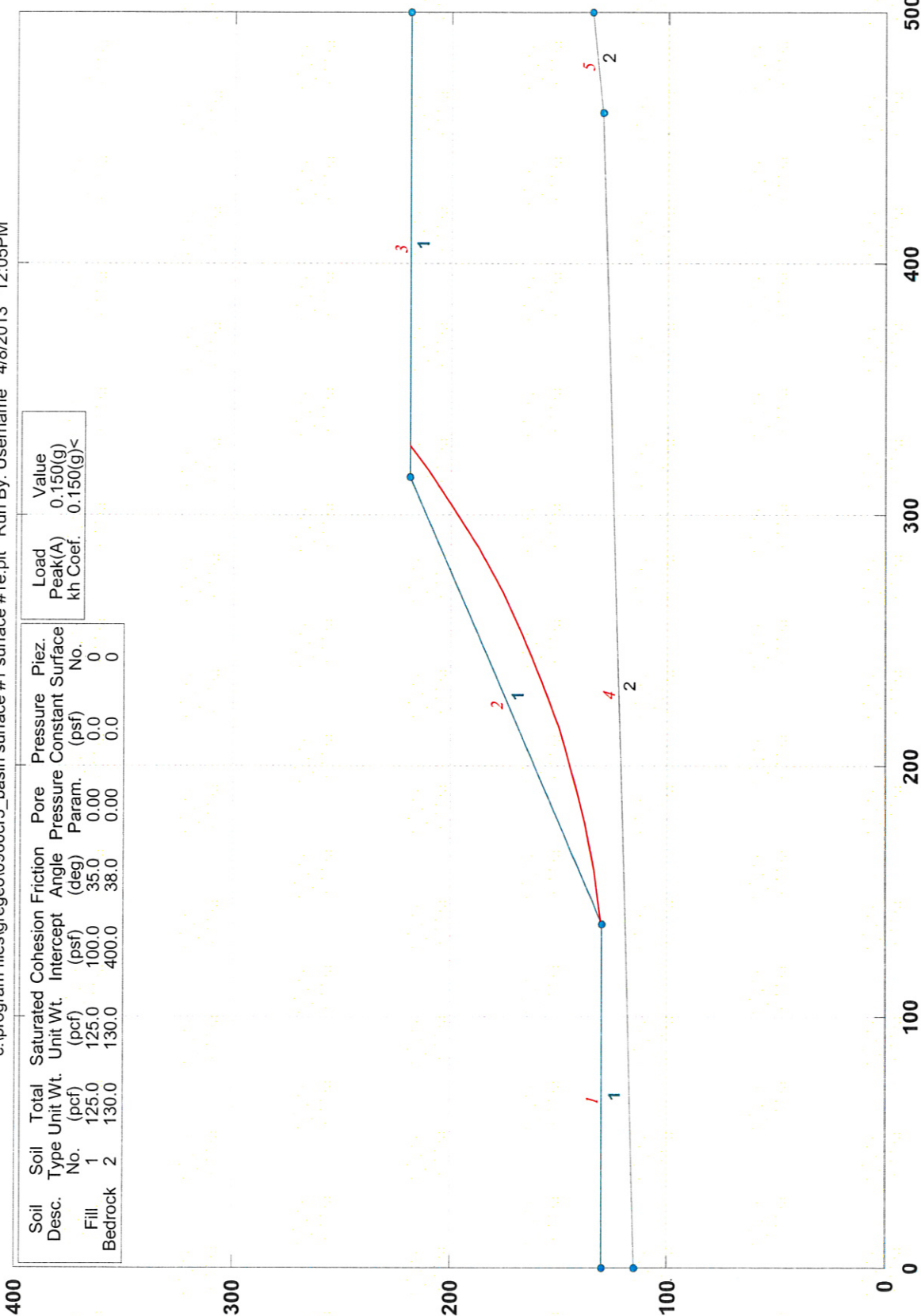
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	140.000	131.500
2	159.899	129.493
3	179.898	129.719
4	199.746	132.176
5	219.197	136.831
6	238.006	143.629
7	255.940	152.482
8	272.774	163.282
9	288.297	175.892
10	302.316	190.156
11	314.656	205.896
12	322.747	219.000

Circle Center At X = 167.946 ; Y = 307.721 ; and Radius = 178.423
 Factor of Safety
 *** 1.795 ***
 ***** END OF GSTABL7 OUTPUT *****

Proj. No. 0967-CR3 Highest 2:1 (h:v) Fill Slope Pseudostatic

c:\program files\geogeo\0960cr3_basin surface #1 surface #1e.plt Run By: Username 4/8/2013 12:05PM



GSTABL7 v.2 FSmin=1.186

Factor Of Safety Is Calculated By The Modified Bishop Method



*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D., P.E., D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.

(Includes Spencer & Morgenstern-Price Type Analysis)

Including Pier/File, Reinforcement, Soil Nail, Tieback,

Nonlinear Undrained Shear Strength, Curved Phi Envelope,

Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water

Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 4/8/2013

Time of Run: 12:05PM

Run By: Username

Input Data Filename: C:\Program Files\GREGEO\0960cr3_basin Surface #1 Surface #1e

.in

Output Filename: C:\Program Files\GREGEO\0960cr3_basin Surface #1 Surface #1e

.OUT

Unit System: English

Plotted Output Filename: C:\Program Files\GREGEO\0960cr3_basin Surface #1 Surface #1e

.PLT

PROBLEM DESCRIPTION: Proj. No. 0967-CR3 Highest

2:1 (h:v) Fill Slope Pseudostatic

BOUNDARY COORDINATES

3 Top Boundaries

5 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	130.00	137.00	130.00	1
2	137.00	130.00	315.00	219.00	1
3	315.00	219.00	500.00	219.00	1
4	0.00	115.00	460.00	130.00	2
5	460.00	130.00	500.00	135.00	2

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param. (psf)	Pressure Constant (psf)	Piez. Surface No.
1	125.0	125.0	100.0	35.0	0.00	0.0	0
2	130.0	130.0	400.0	38.0	0.00	0.0	0

Specified Peak Ground Acceleration Coefficient (A) = 0.150(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

Trial Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	137.895	130.448
2	157.617	133.766
3	177.122	138.189
4	196.348	143.701
5	215.233	150.286
6	233.718	157.922
7	251.744	166.585
8	269.254	176.248
9	286.194	186.881
10	302.509	198.449
11	318.148	210.916

12 327.193 219.000
 DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE (Excluding Last Segment)
 Angle/Segment No. Deflection (Deg) Segment Length (ft)

1	3.23	20.00
2	3.22	20.00
3	3.23	20.00
4	3.22	20.00
5	3.22	20.00
6	3.22	20.00
7	3.22	20.00
8	3.22	20.00
9	3.22	20.00

Circle Center At X = 89.178(ft) ; Y = 481.195(ft); and Radius = 354.115(ft)

* * Factor Of Safety Is Calculated By The Modified Bishop Method * *

Factor Of Safety For The Preceding Specified Surface = 1.186

Table 1 - Individual Data on the 12 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force		Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force		Surcharge Load (lbs)
			Top (lbs)	Bot (lbs)			Hor (lbs)	Ver (lbs)	
1	19.7	8064.5	0.0	0.0	0.0	0.0	1209.7	0.0	0.0
2	19.5	22448.4	0.0	0.0	0.0	0.0	3367.3	0.0	0.0
3	19.2	33459.2	0.0	0.0	0.0	0.0	5018.9	0.0	0.0
4	18.9	41079.0	0.0	0.0	0.0	0.0	6161.9	0.0	0.0
5	18.5	45366.2	0.0	0.0	0.0	0.0	6804.9	0.0	0.0
6	18.0	46444.0	0.0	0.0	0.0	0.0	6966.6	0.0	0.0
7	17.5	44503.9	0.0	0.0	0.0	0.0	6675.6	0.0	0.0
8	16.9	39803.7	0.0	0.0	0.0	0.0	5970.6	0.0	0.0
9	16.3	32651.9	0.0	0.0	0.0	0.0	4897.8	0.0	0.0
10	12.5	19438.3	0.0	0.0	0.0	0.0	2915.8	0.0	0.0
11	3.1	3674.8	0.0	0.0	0.0	0.0	551.2	0.0	0.0
12	9.0	4570.0	0.0	0.0	0.0	0.0	685.5	0.0	0.0

Table 2 - Base Stress Data on the 12 Slices

Slice No.	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	9.55	147.76	20.00	351.42	126.28
2	12.78	167.37	20.00	798.94	410.22
3	16.00	186.74	20.00	1127.75	697.33
4	19.22	205.79	20.00	1346.08	959.60
5	22.45	224.48	20.00	1462.01	1171.07
6	25.67	242.73	20.00	1483.34	1309.72
7	28.89	260.50	20.00	1417.85	1357.83
8	32.12	277.72	20.00	1273.48	1302.93
9	35.34	294.35	20.00	1058.42	1138.53
10	38.56	308.75	15.97	809.01	898.04
11	38.56	316.57	4.03	623.86	674.25
12	41.79	322.67	12.13	297.07	292.85

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing Soil Nail, and Applied Forces if applicable) = 225507.47 (lbs)

Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing, Soil Nail, and Applied Forces if applicable) = 1063.06(psf)

Sum of the Driving Forces = 190082.80 (lbs)

Average Mobilized Shear Stress = 896.06(psf)

Total length of the failure surface = 212.13(ft)

CAUTION - Factor Of Safety Is Calculated By The Modified Bishop Method. This Method Is Valid Only If The Failure Surface Approximates A Circular Arc.

**** END OF GSTABL7 OUTPUT ****

APPENDIX D

GENERAL GRADING GUIDELINES FOR EARTHWORK CONSTRUCTION

**South Shore II Project, Tract 32013
Lake Elsinore, Riverside County, California
Project No. 0967-CR3**



GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2010) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.

5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
 - a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
 - b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed (see Plates G-1, G-2 and G-3) unless otherwise specifically indicated in the text of this report.



2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Subdrainage

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to schematic diagrams G-1 and G-5, and be acceptable to our representative.
2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.
3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.
4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.
5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.
6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.
7. Consideration should be given to having subdrains located by the project surveyors.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:

- a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
 - b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
 - a) They are not placed in concentrated pockets;
 - b) There is a sufficient percentage of fine-grained material to surround the rocks;
 - c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal (see Plate G-4). On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.

5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

Keyways, Buttress and Stabilization Fills

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates G-2, G-3). As the fill is elevated, it should be benched through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates G-1, G-2, and G-3).
2. Fill over cut slopes should be constructed in the following manner:
 - a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
 - b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction), and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
 - c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate G-3 for schematic details.)
3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate G-2.
4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate G-2.
5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate G-3 for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate G-5 shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions, and need to maintain a minimum fill width and provide working room for the equipment.
2. On longer slopes, backcuts and keyways should be excavated in maximum 250 feet long segments. The specific configurations will be determined during construction.
3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent, whichever is greater.
4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate G-5.

5. Benching of backcuts during fill placement is required.

Lot Capping

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advise based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.
2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g. lots above stabilization fills, along daylight lines, above natural slopes, etc.) should be capped with a minimum three foot thick compacted fill blanket.
3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

ROCK PLACEMENT AND ROCK FILL GUIDELINES

It is anticipated that large quantities of oversize material would be generated during grading. It's likely that such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

Limited Larger Rock

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inches) should be placed in windrows.
 - a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.
 - b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).
 - c) The maximum rock size allowed in windrows is four feet
2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate G-4).
3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.

4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
 - a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
 - b) The over size rock trenches should be no closer together than 15 feet from any slope face.
 - c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
 - d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

Structural Rock Fills

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may effect foundation design.
2. Rock fills are required to be placed in horizontal layers that should **not exceed two feet in thickness, or the maximum rock size present, which ever is less**. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:
 - a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill ,
 - b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,
 - c) the portion of the rock above grade is covered with a second lift.
3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

Compaction Procedures

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.



- a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.
 - b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.
 - c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). (Water should be applied before and during spreading.)
2. Rock fill should be generously watered (sluiced)
 - a) Water should be applied by water trucks to the:
 - i) dump piles,
 - ii) front face of the lift being placed and,
 - iii) surface of the fill prior to compaction.
 - b) No material should be placed without adequate water.
 - c) The number of water trucks and water supply should be sufficient to provide constant water.
 - d) Rock fill placement should be suspended when water trucks are unavailable:
 - i) for more than 5 minutes straight, or,
 - ii) for more than 10 minutes/hour.
3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber tired compactors may be required.
 - a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
 - b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.
4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:
 - a) the general segregation of rock size,
 - b) for any unfilled spaces between the large blocks, and
 - c) the matrix compaction and moisture content.
5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
 - a) A lift should be constructed by the methods proposed, as proposed
6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractors procedures.
7. A minimum horizontal distance of 15 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 15 feet should be built of conventional fill materials.

Piping Potential and Filter Blankets

Where conventional fill is placed over rock fill, the potential for piping (migration) of the fine grained material from the conventional fill into rock fills will need to be addressed.



The potential for particle migration is related to the grain size comparisons of the materials present and in contact with each other. Provided that 15 percent of the finer soil is larger than the effective pore size of the coarse soil, then particle migration is substantially mitigated. This can be accomplished with a well-graded matrix material for the rock fill and a zone of fill similar to the matrix above it. The specific gradation of the fill materials placed during grading must be known to evaluate the need for any type of filter that may be necessary to cap the rock fills. This, unfortunately, can only be accurately determined during construction.

In the event that poorly graded matrix is used in the rock fills, properly graded filter blankets 2 to 3 feet thick separating rock fills and conventional fill may be needed. As an alternative, use of two layers of filter fabric (Mirafi 700 x or equivalent) could be employed on top of the rock fill. In order to mitigate excess puncturing, the surface of the rock fill should be well broken down and smoothed prior to placing the filter fabric. The first layer of the fabric may then be placed and covered with relatively permeable fill material (with respect to overlying material) 1 to 2 feet thick. The relative permeable material should be compacted to fill standards. The second layer of fabric should be placed and conventional fill placement continued.

Subdrainage

Rock fill areas should be tied to a subdrainage system. If conventional fill is placed that separates the rock from the main canyon subdrain, then a secondary system should be installed. A system consisting of an adequately graded base (3 to 4 percent to the lower side) with a collector system and outlets may suffice.

Additionally, at approximately every 25 foot vertical interval, a collector system with outlets should be placed at the interface of the rock fill and the conventional fill blanketing a fill slope

Monitoring

Depending upon the depth of the rock fill and other factors, monitoring for settlement of the fill areas may be needed following completion of grading. Typically, if rock fill depths exceed 40 feet, monitoring would be recommended prior to construction of any settlement sensitive improvements. Delays of 3 to 6 months or longer can be expected prior to the start of construction.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that “worked” on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
 - a) shallow (12 + inches) under slab interior trenches and,
 - b) as bedding in pipe zone.

The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.

3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.



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

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our 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.

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.

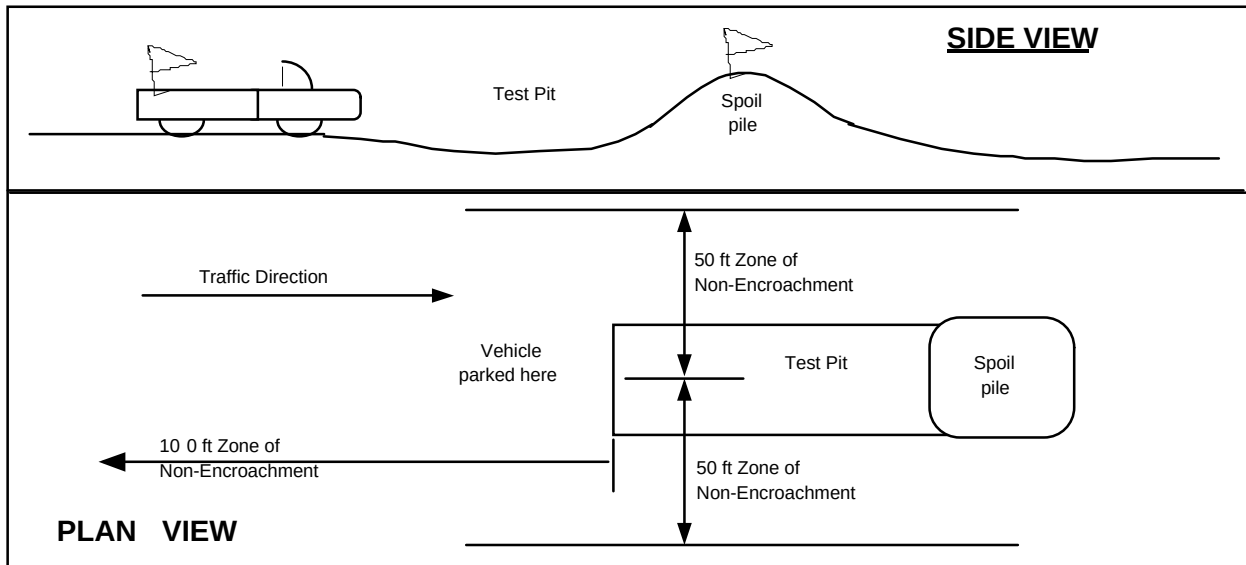
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

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

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during 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.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

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

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

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

Procedures

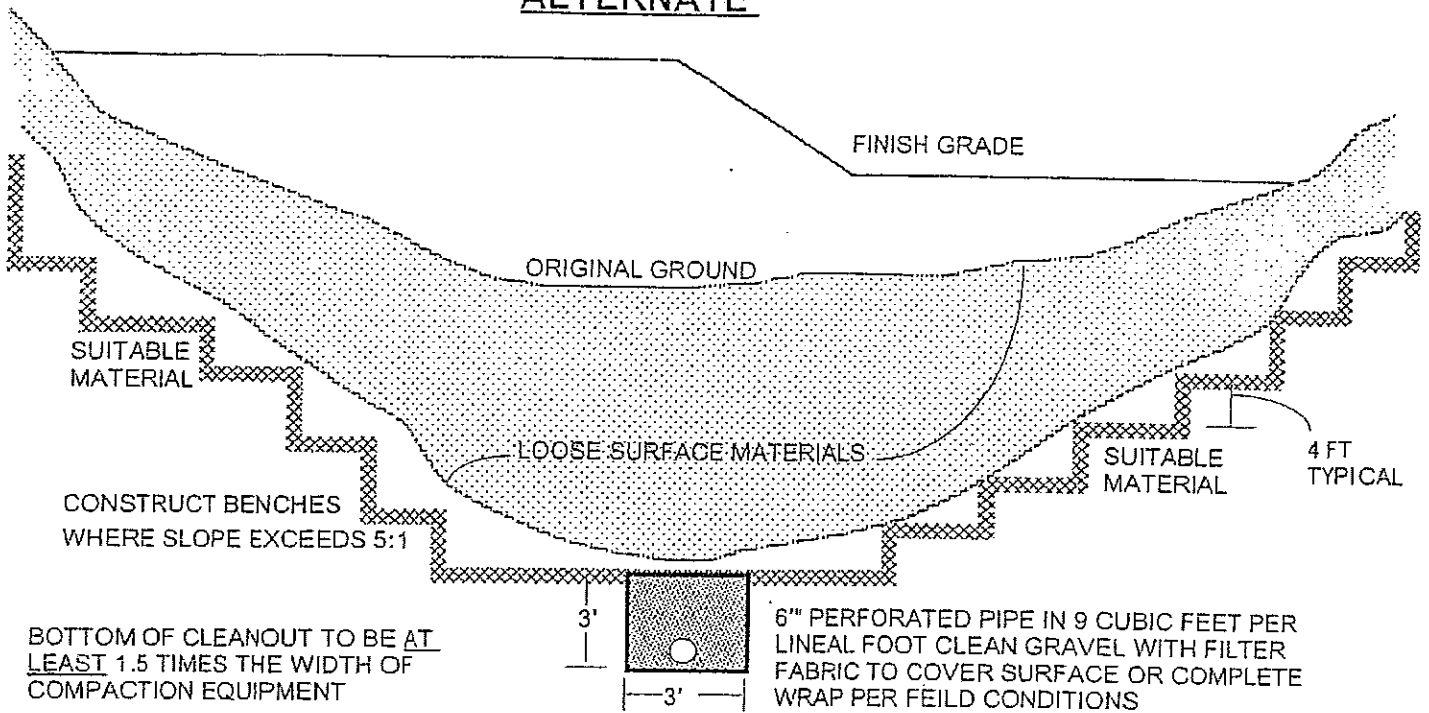
In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim 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 technicians attention and notify our project manager or office. Effective communication and coordination between the contractor's representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

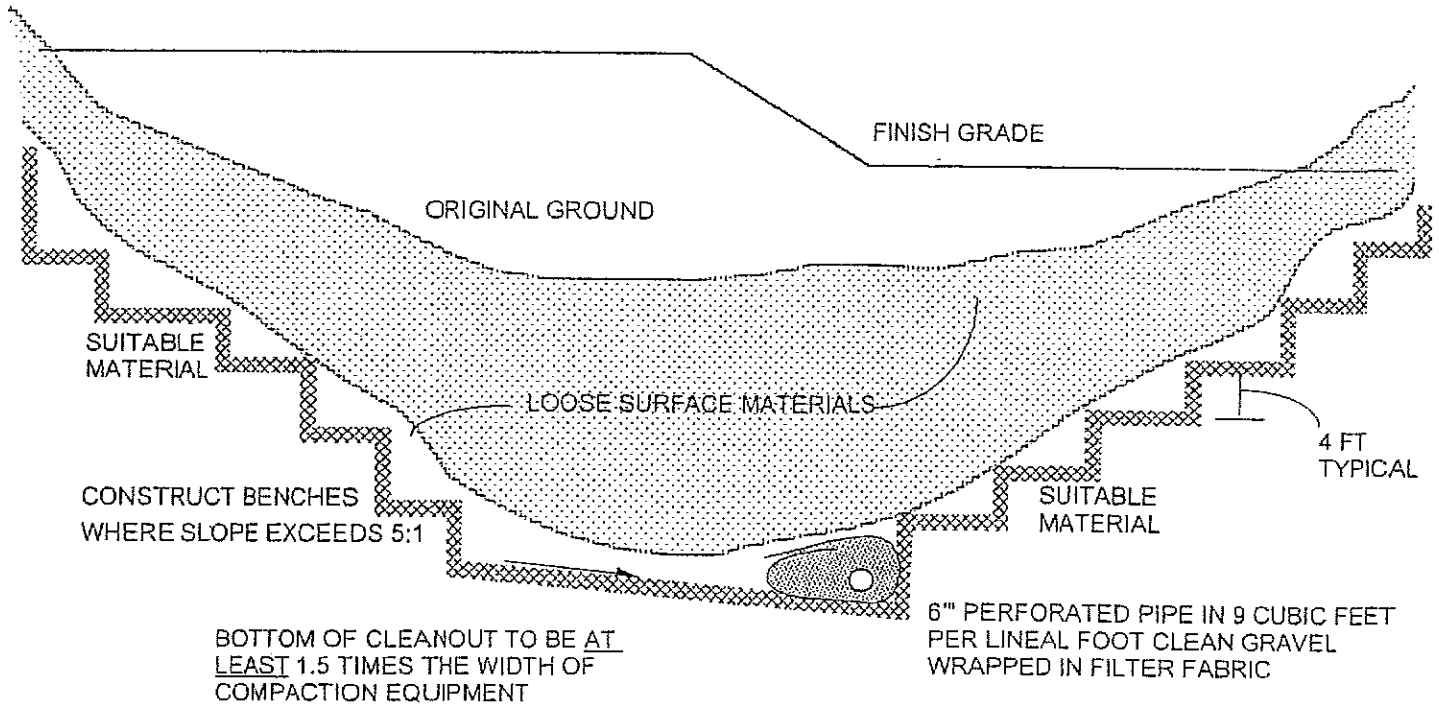
The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

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ALTERNATE



ALTERNATE



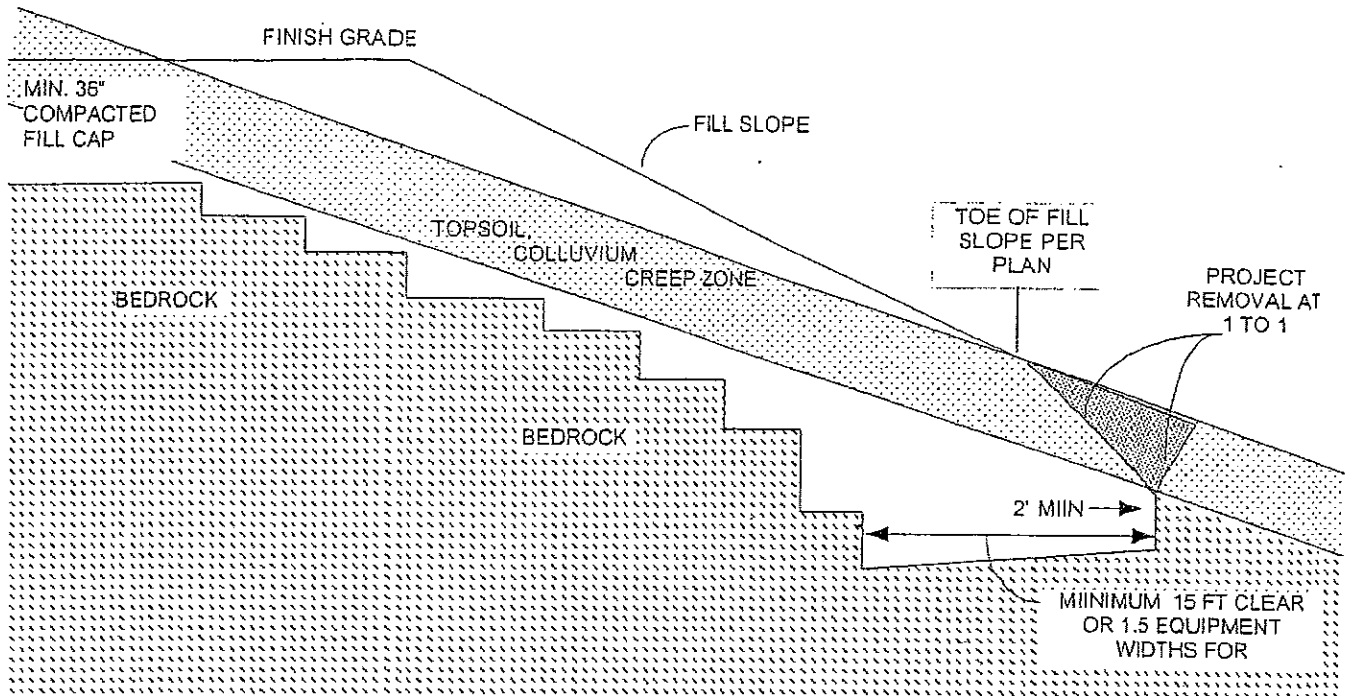
STANDARD GRADING GUIDELINES

TYPICAL CANYON
CLEANOUT

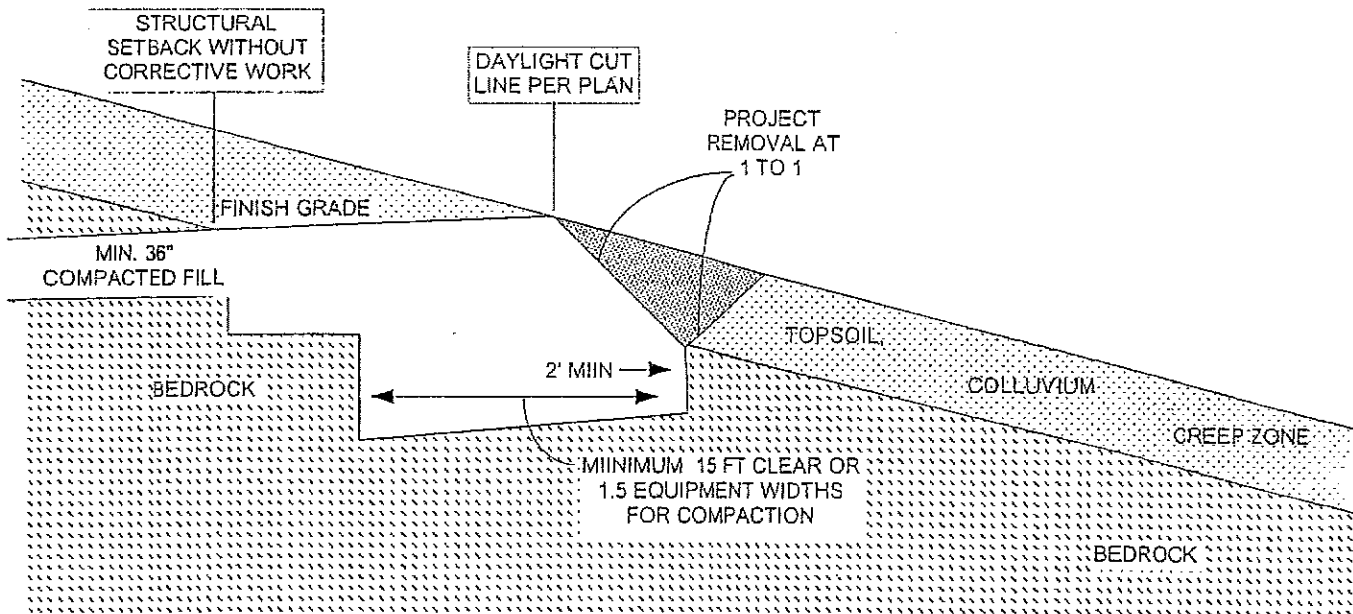
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PLATE G - 1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



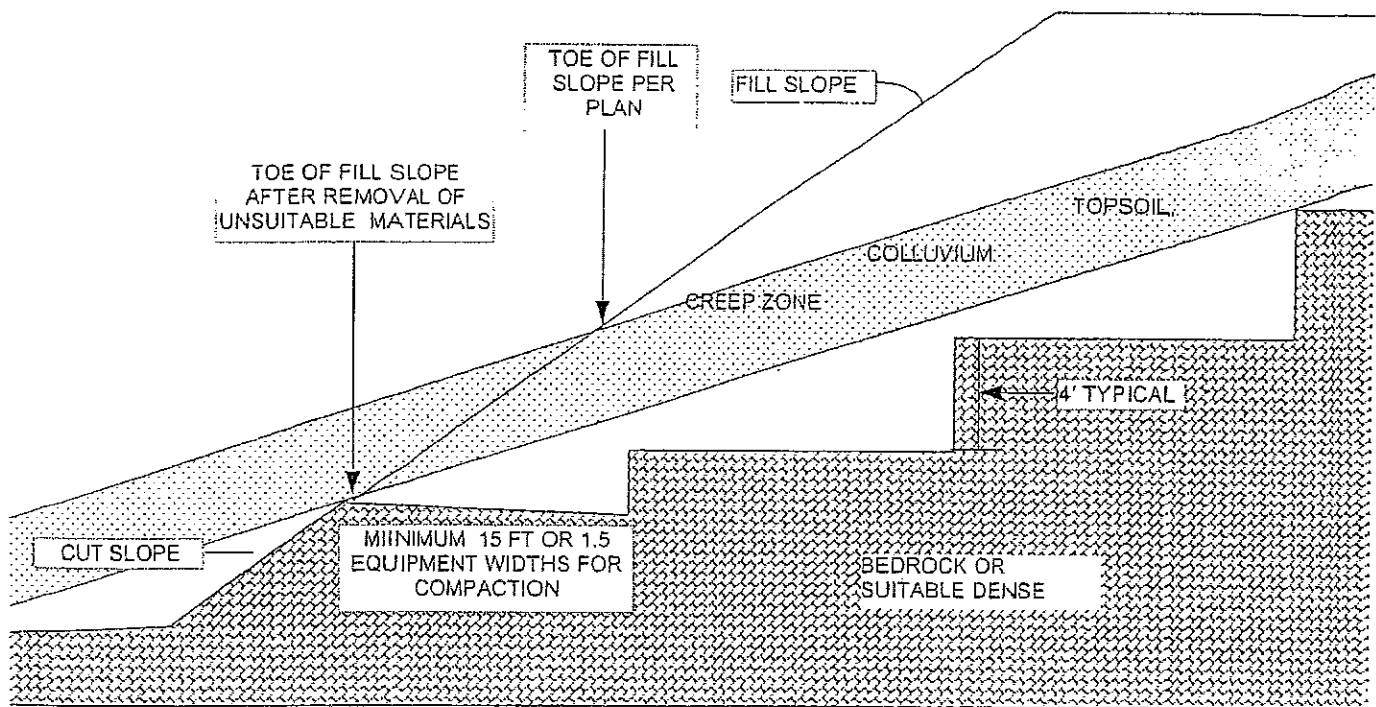
STANDARD GRADING GUIDELINES

TREATMENT ABOVE
NATURAL SLOPES

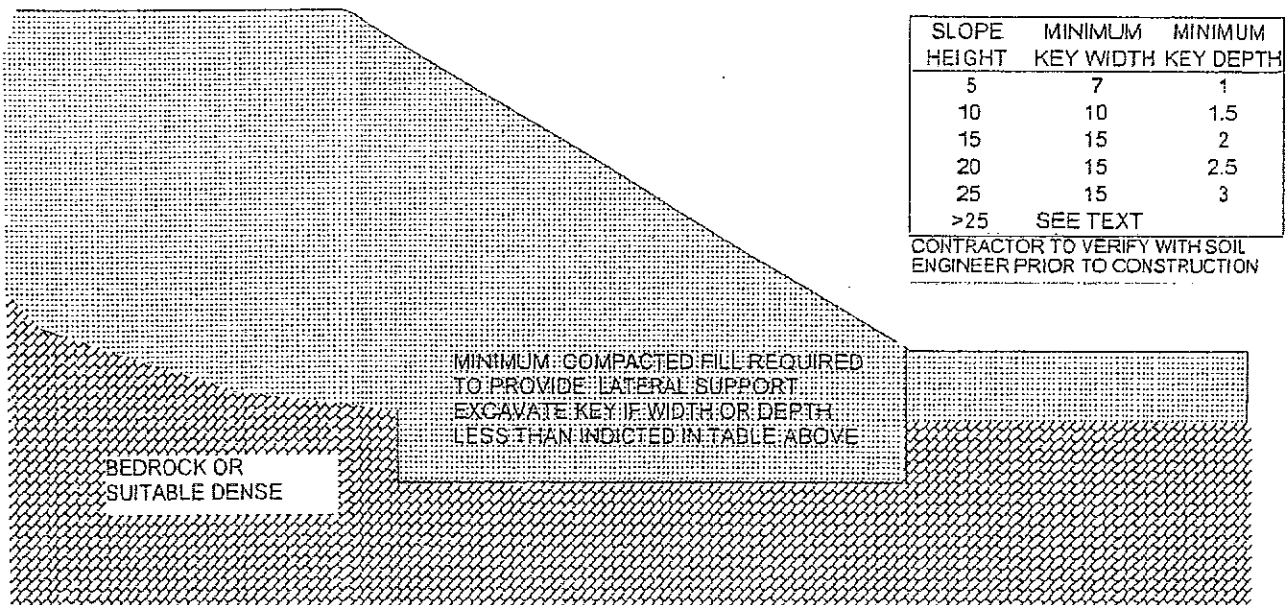
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PLATE G - 2

TYPICAL FILL SLOPE OVER PROPOSED CUT SLOPE



TYPICAL FILL SLOPE



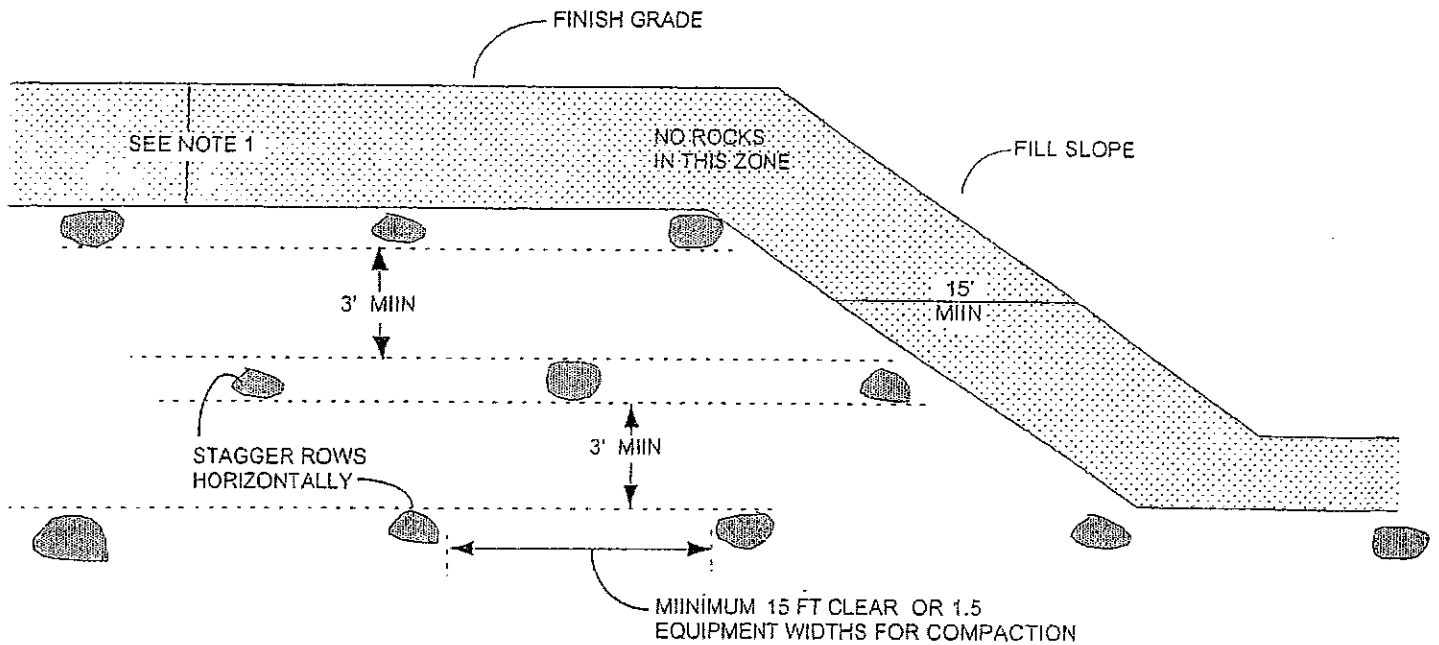
STANDARD GRADING GUIDELINES

COMMON FILL
SLOPE KEYS

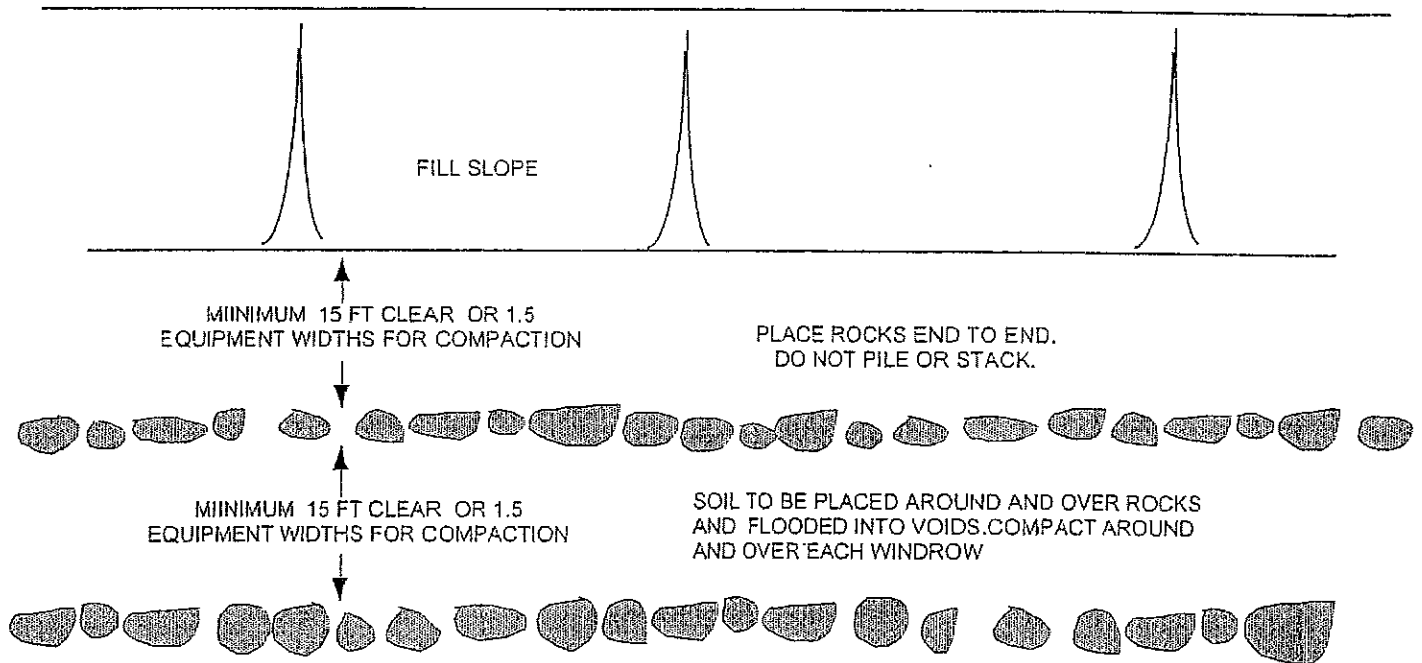
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PLATE G - 3

CROSS SECTIONAL VIEW



PLAN VIEW



NOTES:

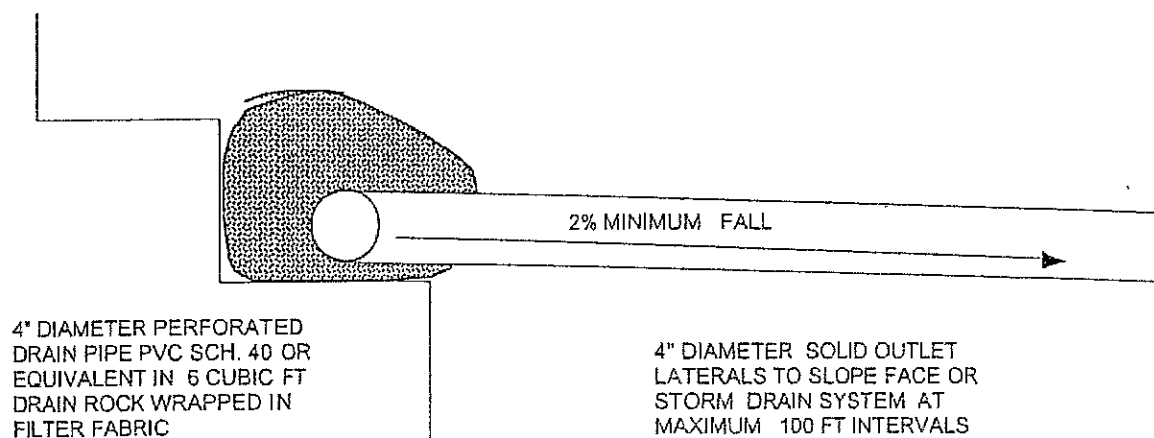
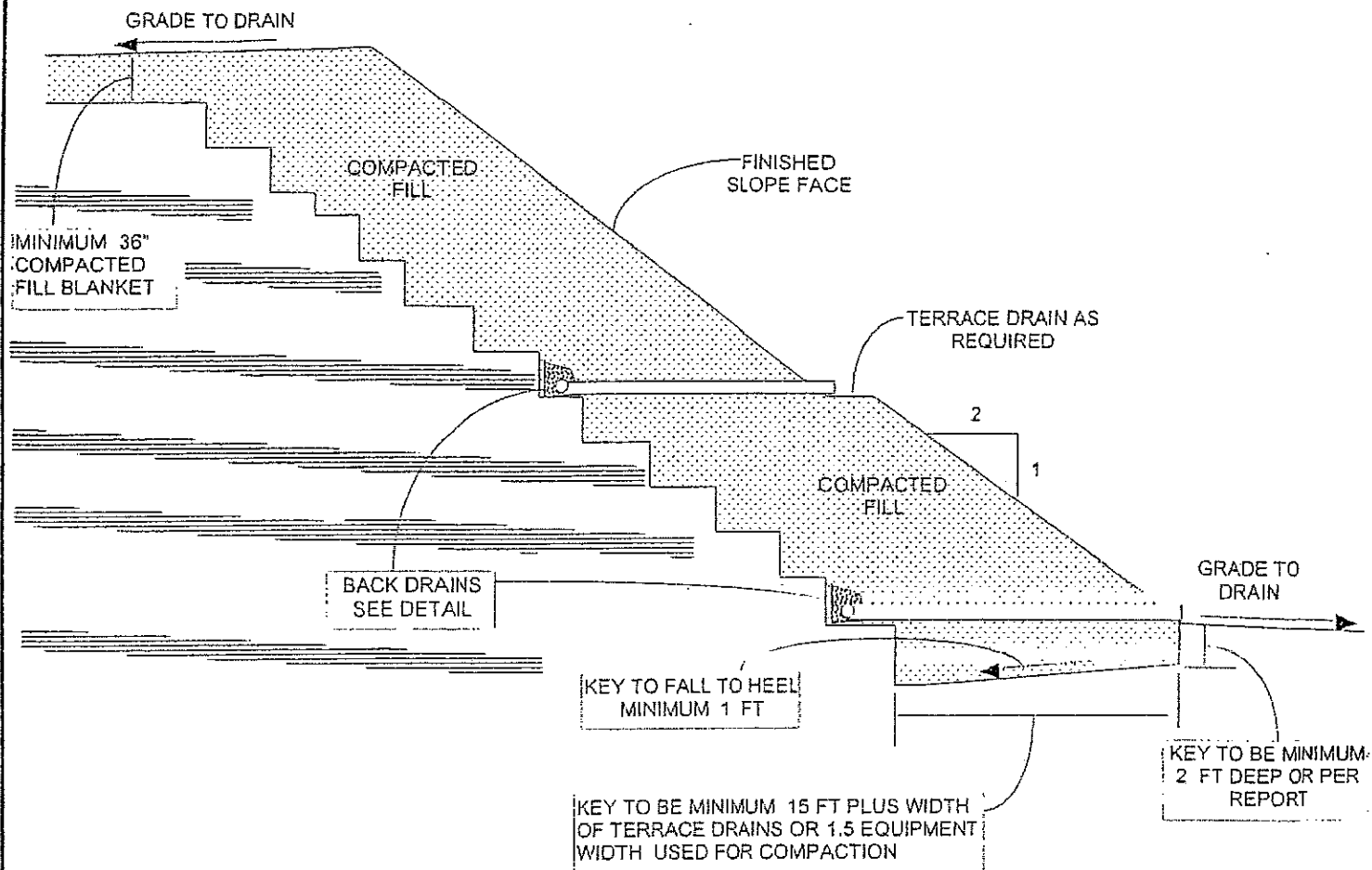
- 1) MINIMUM SOIL FILL OVER WINDROWS SHOULD BE 7 FEET AND SUFFICIENT FOR FUTURE EXCAVATIONS (e.g. SWIMMING POOLS) TO AVOID ROCKS.
- 2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET MINIMUM DIAMETER.
- 3) SOIL AROUND WINDROWS TO BE SANDY MATERIAL SUBJECT TO ACCEPTANCE BY SOIL ENGINEER
- 4) ALL SPACING AND CLEARANCES MUST BE SUFFICIENT TO ALLOW FOR PROPER COMPACTION.

STANDARD GRADING GUIDELINES

ROCK BURIAL
DETAILS

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PLATE G - 4



NOTE: ADDITIONAL BACKDRAINS MAY BE RECOMMENDED

STANDARD GRADING GUIDELINES

BUTTRESS AND
STABILIZATION SLOPES

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PLATE G - 5