UPDATED GEOTECHNICAL AND INFILTRATION EVALUATION

PROPOSED SINGLE-FAMILY RESIDENTIAL DEVELOPMENT

KELLER CROSSING PROJECT

NORTHWEST CORNER OF WINCHESTER ROAD AND KELLER ROAD
WINCHESTER AREA OF RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

D.R. HORTON LOS ANGELES HOLDING COMPANY, INC.
2280 WARDLOW CIRCLE, SUITE 100
CORONA, CALIFORNIA 92878

PREPARED BY

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PROJECT NO. 2453-CR **MAY 25, 2021**

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May 25, 2021
Project No. 2453-CR

D.R. Horton Los Angeles Holding Company, Inc.
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Attention: Mr. Tom Irwin

Subject: Updated Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Development
Keller Crossing Project
Northwest Corner of Winchester Road and Keller Road
Winchester Area of Riverside County, California

Dear Mr. Irwin:

We are pleased to provide the results of our updated geotechnical and infiltration evaluation for the subject project located northwest of the intersection of Winchester Road and Keller Road in the Winchester area of Riverside County, California. This report presents the results of our evaluation and discussion of our findings.

In our opinion, site development appears feasible from a geotechnical viewpoint. Final site development and grading plans should be reviewed by this firm as they become available, as it will be necessary to provide appropriate recommendations for intended specific site development as those plans become refined.

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.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions on the site and provide updated geotechnical and infiltration recommendations as deemed appropriate. Services for this study included the following:

- Research and review of available geologic and geotechnical data, and past reports pertinent to the site,
- Perform a reconnaissance of the site,
- Site evaluation of rock hardness via a seismic refraction survey, performed by a subconsultant,
- Excavation of six percolation test borings and subsequent percolation testing per County of Riverside guidelines,
- Excavation of seven exploratory trenches and eight exploratory borings to assess general subsurface soil conditions of the property and the areas of proposed offsite improvements,
- Collection of relatively undisturbed and bulk samples of the onsite materials,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, and
- Compilation of this updated geotechnical evaluation report which presents our findings, conclusions, and recommendations for the site development.

The intent of this report is to aid in the evaluation of the site for future development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report will likely need to be updated based on our review of final site development plans. These should be provided to GeoTek for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The roughly trapezoidal-shaped site consists of an approximate 200-acre area of land located northwest of the intersection of Winchester Road and Keller Road in the Winchester area of Riverside County, California. The site consists of ten parcels of land identified with Riverside County Assessor's Parcel Numbers (APNs) 472-110-00I through -004, -007 through -009, and -032 through -034. Topographically, the site is characterized by a series of south trending ridges with intervening drainage valleys with the highest ground elevation of about 1,583 feet toward the northeastern portion of the site to the lowest elevation of about 1,423 feet near the southeastern edge of the site. Surface drainage is generally directed toward the south-southeast. The general location of the site is shown in Figure 1.

Dry farming operations are being conducted within the low-lying portions of the site, while the remainder has a light cover of native grass and weeds.

The site is bounded by an east-west trending ridge to the north; Keller Road, followed by single-family homes to the south; Winchester Road, followed by vacant land and a single-family residence to the east; and Pourroy Road, followed by residential properties and vacant land to the west.

2.2 PROPOSED DEVELOPMENT

According to the *Site Plan* prepared by K&A Engineers (undated), the project consists of the grading and construction of 356 single-family residential lots, commercial pads, two retention/water quality basins/areas, underground utilities, a park area, and street improvements. The existing Keller Road alignment is proposed to be realigned in an east-west arcuate orientation to the north of the southernly proposed retention area, and to the east of the intersection of Winchester Road. Approximately 61-acres of land located at the northern edge of the property will remain undeveloped. Based on the Cut-Fill Summary Map prepared by K&A Engineers, dated August 11, 2020 cuts and fills up to 61 and 32 feet, respectively, are anticipated to be required to reach design grades. Also, cut and fill slopes up to about 45 and 30 feet in height and at 2:1 (h:v) maximum gradients, respectively are planned. Retaining walls are also expected, but no plans have been made available at this time. Plans for utility construction were not available at the time of this evaluation, although an offsite improvement plan was used as the base for Figure 2b.

Two water quality basins are proposed within the south-central and southeastern portion of the property. Cuts on the order of 5-18 feet are expected to be required to reach the proposed basin bottom. The locations of the planned basin areas are shown on the enclosed Figure 3 (Infiltration Test Location Map).

Based on discussions with the D.R. Horton, a jack and bore technique is planned for the proposed water line utility construction near the intersection of Keller Road and Winchester Road. The jack and bore will include a receiving pit and a jacking pit of currently unknown plan dimensions and depths; however, it is our undertanding that the utilities at this location will be installed up to 25 feet below existing grade. Temporary shoring system is anticipated to be required to excavate the pits. However, the shoring system to be utilized has not yet been determined.

Based on the proposed *Offsite Utilities Exhibit*, prepared by K&A Engineering, Inc. (2019) offsite improvements will include a recycled water line, as well as water and sewer lines. A 12-inch recycled water line is proposed to extend from within the subject site westerly along Keller Road and terminating at an existing tie-in point near the intersection of Leon Road. A water line is proposed to extend bilaterally from the subject site. The water line is proposed to extend from the subject site in an easterly direction along Keller road then transitioning in a southerly direction along Washington Street terminating at an existing utility tie-in point. The water line is also proposed in a westerly direction from the subject site then transitioning in the southerly direction along Pourroy Road and terminating at an existing utility tie-in point near the intersection of Ruft Road. Additionally, a 12-inch sewer line is proposed to extend from the subject site westerly along Keller Road then transitioning in a southerly direction along Pourroy Road and terminating at an existing tie-in point east of the Winchester Road intersection. The subject utilities are anticipated to be installed via open sloped excavations at depths generally ranging from 5 to 8 feet and possibly deeper. Utilities within Pourroy Road, south of Keller Road, may also be deeper. The proposed locations of the described utilities are shown on the enclosed Figure 2b.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Final site development plans should be reviewed by GeoTek when they become available. Additional geotechnical field exploration, analyses, and recommendations may be necessary upon review of site development plans.

3. REPORT REVIEW

On July 29, 2009, GeoSoils Inc., (GSI) completed a report entitled *Preliminary Geotechnical Investigation* for the subject site. The GSI report also referenced a prior geotechnical report prepared by Eberhart/United Consultants that was performed in 2005. Although a copy of the 2005 report was not provided, copies of the 23 excavation logs and laboratory testing from the 2005 report were included within the GSI report. In 2009, GSI excavated an additional 11 test pits to assess the soil and bedrock conditions at the site. GSI also performed 12 seismic refraction traverses to preliminarily assess the rippability characteristics of the bedrock materials at the site. The 2005 exploratory excavations were extended to depths ranging from about 2 to 9.5 feet below grade and the 2009 test pits by GSI were extended to depths ranging from about 2 to 9 feet below grade. The soils generally encountered in the excavations consisted of surficial soils classified as a firm to hard silty clay, loose to firm clayey silt, loose to dense silt with underlying bedrock. Portions of the shallow soils were also identified as possessing a porous structure. Excavation logs by GSI indicated that the surficial soils included very soft clay and loose clayey sand topsoil to a depth of about 0.5 to 1.5 feet below grade. The depth of soil overlaying the bedrock was indicated to range from about 0.5 to 8.5 feet below existing grade.

GSI indicated that no active faults are known to exist on the site and seismic design parameters were provided based on the 2007 California Building Code (CBC). GSI also noted that the site is not susceptible to soil liquefaction based on a depth to groundwater in excess of 50 feet below grade and the presence of shallow dense soil/bedrock materials across most of the property. GSI indicated that laboratory testing indicated that the near surface soils at the site possess a "very low" to "medium" expansive potential. Plasticity testing indicated a low degree of plasticity. Chemical testing indicated the on-site materials possessed a "negligible" sulfate levels and can be considered to be "extremely corrosive" to "highly corrosive" (Roberge, 2000).

The seismic traverses perform by GSI generally indicated a thin surficial layer of topsoil/colluvium or highly weathered bedrock underlain by apparent weathered bedrock further underlain by relatively unweathered bedrock. The bedrock was estimated by GSI to be rippable using a D-9 Caterpillar dozer to depths ranging from about 23 to 41 feet. GSI indicated that blasting would likely be necessary for excavations extending into the relatively unweathered bedrock. GSI also stated that non-trenchable conditions were present at depths ranging from about 2 to 24 feet below grade. GSI indicated that based on the variable nature of the bedrock, blasting should be anticipated to achieve proposed cut depths and/or street/roadway undercuts for utility installation.

GSI noted that landsliding was not evident at the site and that landslide mitigation was not necessary. GSI further noted an insignificant rock fall hazard for the project.

Preliminary recommendations by GSI indicated that cut and fill slopes constructed at gradients of 2:1 (h:v) or flatter to a height of 30 feet should be grossly and surficially stable. GSI also noted that stabilization fills may be needed due to localized adverse geologic conditions. Preliminarily, GSI recommended over-excavation of surficial soils of about 1 to 2 feet and 3 to 8 feet in the bedrock areas and alluvial areas, respectively. Localized deeper removals were also indicated by GSI to be possible. GSI also noted that over-sized rock fragments (i.e. greater than 12 inches in diameter) should be anticipated during site grading and disposal of over-sized materials should be considered. GSI provided recommendations for disposal over over-size rock (greater than 12 and 36 inches) and disposal of material with rocks less than 12 inches in greatest dimension.

Shrinkage estimates of 5 to10% for colluvium/topsoil; 0 to10% for older alluvium and bulking of between 15-25% for phyllite bedrock were provided by GSI for the materials that will be moved during earthwork construction. Subsidence of 0.10 feet was estimated in non-bedrock areas.

Following site grading, GSI provided foundation recommendations for construction on very low to low expansive soils (PI<15) and for medium expansive soils (PI>15). GSI noted that a mat-type or post-tensioned foundation system would likely be needed in areas where the near surface as-graded soils possess a PI greater than 15. Preliminary recommendations were also provided by GSI for lightly loaded and heavily loaded floor slabs. GSI estimated that localized total static and differential foundation settlements of about 0.75-inch and 0.5-inch should be expected. GSI also indicated that dynamic differential settlement of about 0.25 to 0.5 percent of the fill thickness may occur. GSI stated that on a preliminary basis the footings and floor slabs should be designed to accommodate a vertical static and dynamic settlement of up to 1.5 inches and a differential settlement of 1-inch over a 50-foot span.

Retaining wall design parameters and backfilling recommendations were also presented by GSI along with recommendations for "top of slope" walls. Preliminary pavement design recommendations were presented by GSI for various traffic loading conditions but indicated that the preliminary design recommendations should be verified with R-value testing at completion of site grading.

A copy of the trench logs, seismic refraction lines, and laboratory test results by GSI (2009) are presented in Appendix A of this report.

4. FIELD EXPLORATION, LABORATORY TESTING, AND INFILTRATION EVALUATION

4.1 FIELD EXPLORATION

To supplement previous exploratory excavations by GSI (2009), GeoTek investigated the project site via exploratory trenches and borings which were performed between February 1, 2021 and March 29, 2021. The trenching exploration consisted of seven trenches to depths ranging from 18 to 20 feet and were excavated to log the subsurface materials and examine the rippability and/or hardness of localized areas throughout the proposed residential site. The boring exploration consisted of drilling eight exploratory borings to approximately 10.5 to 31.2 feet below existing grade within the subject site and within areas of offsite utility improvements. Additionally, six percolation test borings were excavated within the proposed basin areas to approximately 5 feet below grade. The trenches were excavated utilizing a Hyundai HX 480 excavator, and the borings were drilled with a CME-75 truck-mounted hollow-stem auger drill rig.

A seismic refraction survey was conducted on February 19, 2021 by a subconsultant (Subsurface Surveys & Associates, Inc.). The seismic refraction survey involved the recording and measuring of man-made energy waves from seven seismic refraction and tomography lines placed in site areas where deep excavations are assumed to be proposed and/or postulated areas of rock hardness. The seismic survey summary report is included in Appendix C. Additionally, GSI performed a seismic refraction survey (2009) which has been included in Appendix A.

The approximate locations of our site explorations, as well as GSI's are shown on the Exploration Location Maps, Figures 2a and 2b. Logs of the exploratory excavations by GeoTek are provided in Appendix B. Logs of the exploratory excavations and the seismic refraction survey by GSI are included in Appendix A.

4.2 LABORATORY TESTING

Laboratory testing was performed by GeoTek and previously by GSI (2009) on selected relatively bulk soil and bedrock samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the subsurface materials encountered and to evaluate the soil/bedrock physical properties for use in the engineering design and analysis. Our test results along with a brief description and relevant information regarding testing procedures are included in Appendix C. Laboratory test results provided by GSI (2009) are provided in Appendix A.

4.3 INFILTRATION STUDY

GeoTek utilized the percolation test procedure (Riverside County, 2011) in order to estimate the infiltration rate of the subsurface materials.

The percolation test borings (Borings I-1 through I-6) were excavated with a hollow-stem auger drill rig within the future basin areas as shown on the referenced Site Plan (K&A Engineering, Inc., 2021). All test borings were drilled to depths of approximately five feet. The borings were approximately eight inches in diameter. A three-inch diameter perforated PVC pipe encapsulated in filter sock was inserted into each of the test holes. The annular space between the test hole sidewalls and PVC pipe was filled with gravel.

Native alluvial materials typically consisting of silty sand were encountered in our test holes. In addition to our test borings, Borings B-1 and B-2 were drilled to 15 feet below grade within the subject areas to confirm the absence of impermeable materials or groundwater. Borings B-1 and B-2 excavated within the southeastern area encountered weathered bedrock at six feet and five feet below existing grade, respectively. No groundwater was encountered in any of the borings. The logs of the borings are presented in Appendix A. The locations of the test borings and deep borings are shown on Figures 2a and 3.

Subsequent to pre-soaking the test holes, percolation testing was performed in the lower 36 inches of each test hole by a representative from our firm. The percolation rates were converted to infiltration rates via the Porchet Method. The infiltration rates, which do not include a factor of safety and were determined after the water levels had stabilized, are presented in the following table.

SUMMARY OF FIELD INFILTRATION RATES			
Area	Test	Depth (Feet)	Infiltration Rate (Inches per hour)
South Basin	I-1	5.0	0.08
	I-2	5.0	0.05
	I-3	5.0	0.05
Southeast Basin	I-4	5.0	0.05
	I-5	5.0	0.08
	I-6	5.0	0.16

Over the lifetime of storm water disposal areas, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rates to design the infiltration systems.

5. GEOLOGIC AND SOILS CONDITIONS

5.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends from the point of contact with the Transverse Ranges geomorphic province, southerly 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. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are mostly found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province, and the San Jacinto fault borders the province adjacent the Colorado Desert province.

The site is located in an area geologically mapped to be underlain mostly by meta-sedimentary (phyllite) bedrock (phyllite and schist, with abundant feldspar and quartz) with areas in the southeast, south, and west of the site underlain by alluvium (Dibblee, T.W., and Minch, J.A., 2003). Based on our field investigations, we have geologically mapped the alluvial areas as being older alluvium. Additionally, the site was geologically mapped by GSI (2009) as being underlain by very old alluvial deposits and Phyllite bedrock. GSI indicated that adverse geologic structure within the Phyllite bedrock is not anticipated to impact the planned development.

No active or potentially active fault is known to exist at the site, nor is the site situated within an “Alquist-Priolo” Earthquake Fault Zone as designated by the State of California. The Riverside County GIS website (https://gis.countyofriverside.us/) also indicates that the property is “not located within a fault zone or fault line,” and the eastern portion of the site has “low” liquefaction potential and is “susceptible” to subsidence.

5.2 EARTH MATERIALS

A brief description of the earth materials reported to be on the site by GSI (2009) and encountered in our explorations for the site and areas of proposed offsite utilities is presented in the following sections.

5.2.1 Artificial Fill

Artificial fill materials were observed in Boring B-7 located west of the subject site within areas of proposed utility improvements. These materials consist of silty sand and sandy silt which has various shades of brown and olive in color, slightly moist, and is in loose condition, based on our field observations. The fill thickness encountered in Boring B-7 was 2 feet. The fill materials apparently were originated from the construction of Keller Road.

5.2.2 Older Alluvium

Older Alluvium was encountered in all the exploratory trenches excavated within the future tract site and within the majority of the exploratory borings drilled along the proposed utility alignments. These materials consist of silty sand, sandy silt, and clayey sand and extended from the ground surface to depths of about 2 to 8.5 feet, with average depths of 4 to 6 feet. The older alluvium was brown, red and gray in color, slightly moist to moist, and medium dense to very dense, based on our field observations.

5.2.3 Metasedimentary Bedrock

Metasedimentary bedrock was observed at the subject site and under the proposed off-site utility alignments at typical depths of 4 to 6 feet and in some areas as deep as approximately 8.5 feet. The regional geologic map shows the bedrock is being of schist (phyllite) composition, generally with foliations in a northwest/southeast orientation with inclinations ranging from 52 degrees to 75 degrees to the northeast. GSI describes the bedrock as fissile black phyllite with a predominantly east-west strike with steep northerly inclinations. GSI states the rocks are generally moderately weathered in the near surface and become hard to very hard at depth.

The on-site bedrock consists of phyllite which is moderately to highly weathered within its upper portions and is generally recovered as gray, brown, and red fine to medium sand and clayey sand when excavated. The bedrock becomes less weathered with depth. All of our exploratory trenches were able to be excavated to a maximum depth of 20 feet, with increasing difficulty below 15 feet below ground surface.

The seismic refraction survey generally identified three zones of subsurface materials. The uppermost zone comprises mostly soil and highly weathered bedrock and is estimated to extend up to 5 feet below grade. The middle zone was noted to correspond to weathered bedrock to depths ranging generally from about 18 to 60 feet with velocities ranging from 3,700 to 4,600 fps. The bottom zone was noted to comprise less weathered bedrock with velocities ranging from 7,100 to 10,800 fps. A tomographic model conducted for seismic line 3 indicated that relatively fresh bedrock exists near the center portion of the line at about 30 feet below existing grade.

To estimate the approximate depth to rippable bedrock and rippable trenching (utility construction) using the seismic refraction data collected at the site, we have utilized cut-off velocities of 5,000 fps and 4,000 fps, respectively. We have also used our field observations during the excavation of the recent site trenches. Based on the above and per the proposed grades shown on the referenced *Cut-Fill Summary Map* (K&A Engineers, 2020) and assuming a maximum wet utility depth of 8 feet below street grade and over-excavation of about 5 feet deep for cut lots into bedrock, we estimate that grading operations within the north-central, elevated portions of the development area will encounter marginally rippable bedrock. Hard, marginally rippable bedrock is anticipated to exist in that area at general depths ranging from 20 to 40 feet, while proposed excavations could be up to 60 feet deep. The seismic refraction traverses performed by GSI (2009) also indicated hard bedrock in the northeastern end of the planned site development area with depth to marginally rippable rock ranging from 25 to 30 feet and potential excavations up to 33 feet deep. While these materials may still be rippable with a Caterpillar D-9 Ripper, excavations may be slow and other excavation techniques could be necessary.

The seismic refraction lines by GeoTek and GSI also suggest that very difficult to non-rippable trenching could be encountered in areas of Lines 1 and 3 through 5 by GeoTek and Lines 4 and 9 by GSI at about 5 to 10 feet below existing grades, based on a cut-off velocity of 4,000 psf. Most of these lines are also situated within the north-central and northeastern ends of the planned development area. The rest of the lines appear to indicate that rippable trenching could achieve to 20 to 40 feet below grade. However, the majority of our exploratory trenches noted slow excavation below 15 feet.

5.2.4 Granitic Bedrock

Granitic bedrock was encountered in our exploratory boring B-7 located on Keller Road within an area of proposed utilities. These materials generally excavated as fine to coarse sand and were found at a depth of about 3 feet. The granitic bedrock was found to be gabbroic in composition that was orange and brown in color, slightly moist to moist, hard, and highly weathered, based on our field observations.

Detailed logs of the subsurface conditions of the site are presented in Appendices A and B. Report of the seismic refraction survey by GeoTek is presented in Appendix C.

5.3 SURFACE WATER AND GROUNDWATER

5.3.1 Surface Water

Surface water was not noted during our field work. If encountered during earthwork construction, surface water on this site is the result of precipitation or possibly some minor surface run-off from immediately surrounding properties. Overall site area drainage is generally to the south-southeast, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

5.3.2 Groundwater

Groundwater was not encountered in any of our exploratory borings or trenches excavated to a maximum depth of 32.1 feet. Groundwater was not encountered by GSI to an explored depth of 9 feet below existing ground surface. GSI stated that the regional groundwater level is estimated to be greater than 50 feet below the existing ground surface.

The California Department of Water Resources, Water Data Library, was searched for groundwater data. However, none of the wells in the general vicinity had groundwater depths more recent than 1968. Therefore, it is our opinion that these wells are not representative of current groundwater condition on the subject site. The nearest well was found on GeoTracker approximately 3 miles southeast of the site. Groundwater was encountered in this well (NM-MW4S) at 45 feet below ground surface. Based on the above, groundwater is not anticipated to be a factor during the site grading. However, seasonal perched groundwater may be encountered during grading within the lower elevations of the site.

GeoTek should review grading plans once available to determine if groundwater is anticipated to adversely affect the proposed developments.

5.4 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 a State of California designated *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986).

The County of Riverside has designated the site as "not in a fault zone" and "not in a fault line."

5.4.1 Seismic Design Parameters

The site is located at approximately 33.6310 Latitude and -117.0963 Longitude. Site spectral accelerations (S_a and S_1), for 0.2 and 1.0 second periods for a Class "D" site, were determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. While the site has shallow bedrock, some areas will require up to 32 feet of engineered compacted fill; thus, a Site Class "D" is considered appropriate. The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.372g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.509g
Site Coefficient for Site Class "C", F_a	1.0
Site Coefficient for Site Class "C", F_v	1.791
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.372g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.912g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.914g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.608g
Site Modified Peak Ground Acceleration, PGA_M	0.585g

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

5.4.2 Surface Fault Rupture

The site is in a seismically active region; however, 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 (Bryant and Hart, 2007). No faults are identified on geologic maps readily available and reviewed by this firm for the immediate study area. The nearest known active fault zone is the Elsinore Fault – Temecula Section located approximately 7.9 miles southwest of the site. Therefore, the potential for surface rupture at the site is considered negligible.

5.4.3 Liquefaction and Seismically Induced Settlement

The County of Riverside has designated the site having "low" liquefaction potential and "susceptible" to subsidence.

Liquefaction is not considered to be a hazard at the subject site due the presence of shallow bedrock materials. Also, the potential for seismically induced settlement at the property is considered to be nil because of the minimal thickness of soil atop bedrock. Additionally, GSI (2009) concluded the absence of a liquefaction hazard at the site and no further investigation or analysis of this nature are warranted.

5.4.4 Other Seismic Hazards

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazard such as a tsunami is considered negligible due to site elevation and great distance to the ocean.

Diamond Valley Lake is located approximately 2.8 miles northeast of the site. This man-made lake was designed for a maximum water elevation 1756 feet msl, which is about 320 feet above the lowest elevation of the subject site. However, the design and construction of the lake's three dams was recently completed (2002) and included dam safety for a strong seismic event. It is our opinion that the risk of a seiche associated with Diamond Valley Lake is minor. Additionally, Skinner Reservoir is located approximately 2.4 miles southeast of the site. This man-made reservoir was designed for a maximum water elevation of 1,497 feet about msl which is about 75 feet above the lowest elevation of the subject site. However, site area surficial drainage west of the skinner reservoir dam is generally to the west-southwest with topographic high areas between the dam and the subject site. Therefore, it is our opinion that a risk of seiche or flooding from dam failure is considered to be minor.

Other bodies of water such as Heritage Lake and Canyon Lake are situated at greater distances and/or lower elevations than the site. Thus, the hazard of seiche related to these lakes is anticipated to be low.

Rock fall hazards are considered nil based on the lack of perched loose rock materials and dense nature of the underlying bedrock. Once grading plans are available, areas of proposed cut slopes on the subject site should be evaluated for potential rock fall hazards by an engineering geologist from this firm.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Development of the site appears feasible from a geotechnical viewpoint. The following recommendations should be incorporated into the design and construction phases of development. The following recommendations are preliminary and may be subject to change upon review of the site rough grading plans.

6.2 EARTHWORK CONSIDERATIONS

6.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the County of Riverside, the 2019 California Building Code (CBC), and recommendations contained in this report. The General Grading Guidelines included in Appendix E 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 E.

Final site grading plans should be reviewed by this office when they become available. Additional recommendations will likely be offered subsequent to review of these plans.

6.2.2 Site Clearing

Site preparation should start with removal of any existing improvements, deleterious materials, and vegetation within the planned development areas of the site. These materials should be properly disposed of off-site.

6.2.3 Remedial Grading

All undocumented fill, loose older alluvium, and highly weathered bedrock should be removed to expose competent native materials. Competent native materials are defined as either older alluvium which is not visibly porous having and in-place compaction of at least 85 percent of the soil's maximum dry density (per ASTM D 1557) or firm, unyielding bedrock. A representative of this firm should observe and approve the bottom of all excavations.

Based on the data available, removals generally on the order of three to four feet from existing grade or to a minimum of three feet below proposed grades, whichever is greater, should be performed below structural areas in fill. Deeper removals should be anticipated in some drainage areas. Actual depths of removal/over-excavation should be determined in the field based on observation and in-place density testing. As a minimum, removals should extend down and away from foundation elements at a 1:1 (h:v) projection to the recommended removal depth, or a minimum of five feet laterally, whichever is greater. The bottom of the removals should be graded to drain toward the front of the lot at a gradient of at least two percent.

In order to facilitate footing excavation and installation of house services, consideration should be given to over-excavate cut lots to a minimum depth of five feet below proposed grades. We recommend that the entire lot be over-excavated. The bottom of the over-excavation should be graded to drain toward the front of the lot at a gradient of at least two percent.

To prevent potential differential settlement, the cut portions of transition (i.e. cut/fill) lots should be overexcavated a minimum of five feet below proposed grades or to a depth of one-half of the maximum fill thickness on the lot, whichever is greater. The horizontal extent of over-excavation could comprise the entire lot or extend at least five feet outside the structural area, or a distance equal to the depth of over-excavation below the bottom of the structural elements, whichever is greater. Over-excavation bottoms should be graded to drain toward the front of the lot (two percent minimum).

We also recommend that utility alignments be overexcavated to at least one foot below the depth of the lowest underground utility.

The approved removal/over-excavation bottom exposed should then be scarified to a depth of about six inches, be moisture conditioned to slightly above the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density, per ASTM D 1557.

6.2.4 Canyon Subdrains

Canyon subdrains should be installed in major drainage swales that will be filled with at least 10 feet of fill. A typical canyon subdrain detail is provided as Figure F-1 in Appendix F. The actual locations of subdrains should be determined by a GeoTek representative during grading based on the conditions encountered. The canyon subdrains should be tied into the project storm drain system (where possible) or daylighted as appropriate. The final 20-foot segment of the subdrain should consist of a non-perforated pipe. Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

6.2.5 Engineered Fill

The onsite materials are considered suitable for reuse as engineered fill provided the materials are free from vegetation, roots, and rock/hard lumps greater than six inches in maximum dimension.

The undercut areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in six- to eight-inch loose lifts, moisture conditioned to the optimum moisture content, and compacted to a minimum relative compaction of 90 percent as determined by ASTM D 1557. Placement of engineered fill should be observed and tested on a full-time basis by a GeoTek representative during grading activities.

Our site excavations observed that bedrock generally breaks into silty fine sands with some cobble- and boulder-sized materials. However, given the relatively soft condition of the rock, farther crushing of the cobbles and boulders should be able to reduce their size to six inches or less. Thus, special procedures to dispose over-sized particles (greater than six inches in maximum dimension) are not anticipated to be required.

6.2.6 Excavation Characteristics

The preliminary *Site Cut/Fill Plan* (K&A, 2020) indicates that the deepest cuts (up to 60 feet) are proposed to be conducted within the north-central region of the site. The results of the seismic refraction surveys by GSI and us (Appendices A and C) and our trenching exploration suggest that bedrock materials marginally rippable with a Caterpillar D9R Ripper may be encountered within this zone starting at general depths of about 20 to 30 feet. Also, marginally rippable bedrock is anticipated at about 20 to 30 feet in the northeastern portion of the site with projected cuts up to 40 feet.

Similarly, the data suggests that very difficult to non-rippable trenching conditions may be experienced within the future utility areas located within the cited zones starting at depths of 5 to 10 feet, due to hard unweathered bedrock. However, much of our trenches conducted with a large excavator noted slow progress started at about 15 feet. Thus, localized chipping or other techniques may be necessary to dislocate and remove corestones or more resistant bedding (quartzite).

Once final design elevations are available, GeoTek should review the plans to discern if the excavation of the metamorphic bedrock to the proposed design grades is expected to be generally feasible with heavy-duty grading equipment in good operating condition. 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 on-site materials should be stable at 1:1 (h:v) inclinations for cuts less than 10 feet in height.

6.2.7 Slope Construction

GSI identified numerous geologic attitudes within the metasedimentary bedrock with the shallowest inclination being 42 degrees to the northeast. These conditions are generally favorable within cut slopes designed at a 2:1 (h:v) or flatter gradient. Additionally, GSI states that stabilization of such slopes is not anticipated, although locally adverse geologic conditions (i.e., daylighted joints/fractures or severely weathered bedrock) may be encountered which may require grading or laying back of the slope to an angle flatter than the adverse geologic condition. Similar conditions were encountered in our exploratory trenches. An engineering geologist should observe all cut slopes during grading. Cut slopes should expose competent bedrock. If adverse structure or incompetent materials are exposed and identified in the cut slopes, stabilization fills may be recommended during grading.

Fill slopes constructed at maximum gradients of 2:1 (h:v), in accordance to industry standards, are anticipated to be both grossly and surficially stable. Where fill is to be placed against sloping terrain with gradients of 5:1 (h:v) or steeper, the sloping ground surface should be benched to remove loose and disturbed surface soil to assure that the new fill is placed in direct contact with competent bedrock and to provide horizontal surfaces for fill placement. A minimum 10- to 15-foot wide keyway should be constructed at the toe of the fill slope areas extending at least 2 to 3 feet vertically into competent natural material.

The base of the keyways and benches should be sloped back into the hillside at a gradient of at least two percent. The base of the benches should be evaluated by a representative of GeoTek prior to processing. Upon approval, the exposed materials should be moistened to at least the optimum moisture content and densified to a relative compaction of at least 90 percent (ASTM D 1557).

Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slope to provide a dense, erosion resistant surface.

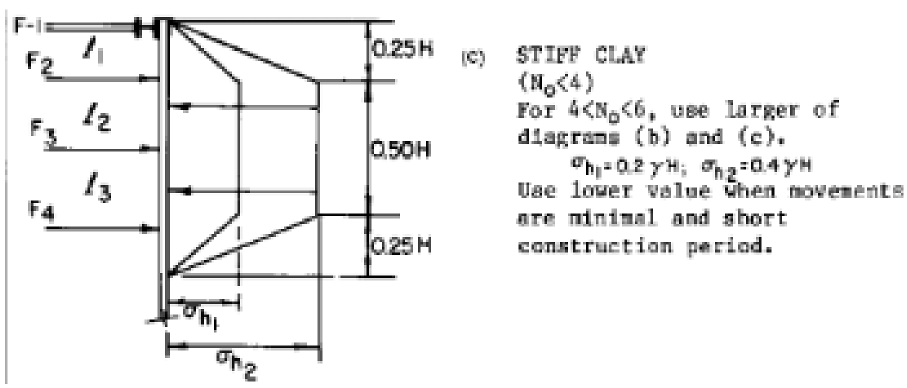
6.2.8 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at 1:1 (h:v) inclinations for short durations during construction and where cuts do not exceed ten feet in height. We anticipate that temporary cuts to a maximum height of four feet can be excavated vertically.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

6.2.9 Temporary Shoring

We anticipate that a temporary shoring system will be used to excavate the proposed receiving and jacking pits near the intersection of Keller Road and Winchester Road. The specific shoring system to be utilized is unknown. If a braced wall system is used, we recommend that a uniform lateral earth pressure of 25H psf (where H is the height of the retained material) be utilized for design of the system. This earth pressure was estimated using the Terzaghi and Peck distribution for temporary braced walls in stiff clays (NAVFAC, 2009). The figure below shows the approximate earth pressure distribution.



*from NAVFAC – Figure 26-Pressure Distribution for Brace Loads in Internally Braced Flexible Walls

An additional surcharge of about 100 pounds per square foot (psf) over the upper 10 feet of the excavation is recommended to be applied due to the live load associated with the nearby roadways. The project structural engineer should design the shoring system using a suitable factor of safety.

The above recommendations are preliminary. When details of the shoring are defined, they should be provided to GeoTek to verify or supplement our recommendations.

6.2.10 Excavation Backfill

Utility trench backfill should be compacted to at least 90 percent relative compaction (as determined per ASTM D 1557). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. On-site materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than six inches are removed.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

6.2.11 Shrinkage and Bulking

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage is primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 10 percent may be considered for the surficial soils. Weathered bedrock materials may bulk up to 10 percent, and relatively unweathered bedrock could bulk from 10 to 20 percent. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction.

Subsidence is anticipated to be up to 0.1 foot in alluvial areas and nil in bedrock areas.

6.3 DESIGN RECOMMENDATIONS

6.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Based on our results of laboratory testing, the on-site materials have mostly "very low" ($0 \leq EI \leq 20$) expansion potential with occasional materials having "low" expansion potential, per ASTM D 4829. Additional laboratory testing should be performed at the completion of site grading to verify the expansion potential of the near-surface soils.

A summary of our preliminary foundation design recommendations is presented in the table below:

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALY REINFORCED SHALLOW FOUNDATIONS		
Design Parameter	"Very Low" Expansion Potential (0≤EI≤20)	"Low" Expansion Potential (21≤EI≤50)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	One- and Two-Story – 12	One- and Two-Story – 12
Minimum Foundation Width (Inches)*	One- and Two-Story – 12	One- and Two-Story – 12
Minimum Slab Thickness (actual)	4 inches	4 inches
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab	6" x 6" – W2.9/W2.9 welded wire fabric or No. 3 rebars at 24 inches on center each way placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 Reinforcing bars, one top and one bottom	Two No. 4 Reinforcing bars, one top and one bottom
Effective Plasticity Index**	<15	15
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% to a depth of 12 inches	Minimum 110% to a depth of 12 inches

*Code minimums per Table 1809.7 of the 2019 CBC.
** Effective plasticity index should be verified at the completion of the rough grading.

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This value may be increased by 400 psf for each additional 12 inches in depth and by 400 psf for each additional 12 inches in width to a maximum value of 3,000 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

Based on the recommended site grading, we estimate a total static settlement of less than 1 inch. A differential static settlement of about ½ inch over a 30-foot span is also estimated. Seismically induced total and differential settlement are considered to be negligible.

The passive earth pressure may be computed as an equivalent fluid having a density of 270 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. 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.

A grade beam, a minimum of 12 inches wide and 12 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

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 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2019 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as the 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. It is GeoTek's opinion that a minimum ten 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 atmospheric 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.

We recommend that control joints be placed in two directions spaced approximately 24 to 36 times the thickness of the slab in inches. These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

6.3.2 Miscellaneous Foundation Recommendations

To minimize moisture penetration beneath the slab-on-grade areas, utility trenches should be backfilled with engineered fill, lean concrete, or concrete slurry where they intercept the perimeter footing or thickened slab edge.

Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

6.3.3 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 outside bottom edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least 5 feet and need not to exceed 15 feet. Where a retaining wall is constructed at the toe of the slope, the height of the slope should be measured from top of the wall to the top of the slope.
- 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 footing.
- The bottom of any proposed 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.

6.4 RETAINING WALL DESIGN AND CONSTRUCTION

6.4.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical walls retaining up to six feet of soil. Additional review and recommendations should be requested for higher walls.

Retaining wall foundations embedded a minimum of 12 inches below the lowest adjacent grade and should rest on either 24 inches of compacted fill placed on competent bedrock or on competent bedrock. Wall footings should be designed using an allowable bearing capacity of 2,000 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 270 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 the table below for specific slope gradients of retained materials.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Native Materials*
Level	40
2:1	65

*The design pressures assume the native backfill material has an expansion index less than or equal to 20. Backfill zone includes area between the back of the wall and footing to a plane (1:1 h:v) up from the bottom of the wall foundation to the ground surface.

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

6.4.2 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, or as otherwise determined by the structural engineer.

6.4.3 Wall Backfill and Drainage

Retaining wall backfill should be free of deleterious and/or oversized materials and should have and expansion index of less than 20. Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one-cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining wall backfill should be placed in lifts no greater than eight inches in thickness and compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D 1557. The wall backfill should also include a minimum one-foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The rock should be separated from the earth with filter fabric. The upper 24 inches should consist of compacted on-site soil.

As an alternative to the drain rock and fabric, Miradrain 2000, or approved equivalent, may be used behind the retaining wall. The Miradrain 2000 should extend from the base of the wall to within two feet of the ground surface. The subdrain should be placed at the base of the wall in direct contact with the Miradrain 2000.

The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. Proper surface drainage needs to be provided and maintained. Walls from two to four feet in height may be drained using localized gravel packs behind weep holes at eight 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 the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

6.4.3.1 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

6.4.4 Pavement Design Considerations

Pavement design for proposed on-site and off-site street improvements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Based on traffic indices (TIs) of 5.5 to 7.0 generally associated with these types of projects and using an assumed design R-value of 22 (GSI, 2009), the following preliminary sections were calculated:

PRELIMINARY PAVEMENT SECTIONS				
Street Category	TI	R-Value	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
Local	5.5	22	3*	9
Enhanced Local	6.5		4*	10
Collector	7.0		4*	12

*Minimum pavement structural section per County of Riverside Street Standards

The TIs used in our pavement design are considered reasonable values for the proposed street areas and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper 12 inches) after final grading has been completed.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density (modified proctor).

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 City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557). If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

6.4.5 Soil Corrosivity

The soil resistivity was tested in the laboratory on three samples collected during our field exploration. The results of the testing (1,640 to 2,600 ohm-cm) indicate that the tested samples are "highly corrosive" to buried metals, based on the guidelines provided in *Corrosion Basics: An Introduction* (Roberge, 2005). Soil resistivity testing performed by GSI (2009) revealed a "extremely" corrosive to "highly" corrosive category for the on-site materials (850 to 1,500 ohm-cm). Chloride content of the samples tested by GeoTek (8 and 33 ppm) was found to be negligible. GSI reported similar findings (50 to 73 ppm). Consideration should be given to consulting with a corrosion engineer. A preliminary corrosion report for the project was prepared by HDR, a corrosion engineering consultant, and is included in Appendix D.

6.4.6 Soil Sulfate Content

The sulfate content was determined in the laboratory for three soil samples obtained during our field investigation. The results (0.00076 and 0.0037 percent) indicate that the tested water-soluble sulfate is negligible, per Table 4.2.1 of ACI 318. GSI (2009) reported similar results regarding the soil sulfate content. Based upon the test results, no special concrete mix design is required by Code for sulfate attack resistance. Additional sampling and testing should be performed once the site grading is complete.

6.4.7 Import Soils

Import soils should have expansion characteristics similar to the on-site soils. GeoTek also recommends that the proposed import soils be tested for expansion and sulfate potential. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

6.4.8 Concrete Flatwork

6.4.8.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks, and driveways should be designed using a four-inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in industrial construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior flatwork with "very low" expansion potential should be pre-saturated to a minimum of 100 percent of optimum moisture content to a depth of at least 12 inches. Soils with a "low" expansion potential should be pre-saturated to about 110 percent of optimum moisture content to a minimum depth of 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the County of Riverside specifications, and under the observation and testing of GeoTek and a county inspector, if necessary.

6.4.8.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 0.125-inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent upon a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control 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 they are. GeoTek suggests that control joints be placed in two orthogonal directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

6.5 POST CONSTRUCTION CONSIDERATIONS

6.5.1 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. 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 foundations. This type of landscaping should be avoided. Due to the presence of high expansive soils, irrigation should be minimized adjacent to the buildings. Planters within 30 feet of the buildings should be above ground and underlain by a concrete slab. Waterproofing of the foundation and/or subdrains may be warranted and advisable. We could discuss these issues, if desired, when plans are made available.

6.5.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times, as directed by the project civil engineer. 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 adjacent to the footings and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

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.

6.6 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications, retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. Additional recommendations and subsurface exploration may be necessary based on these reviews. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of 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 when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Test the fill for field density and relative compaction.
- Test the near-surface soils to verify proper moisture content.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can 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.

7. LIMITATIONS

This evaluation does not and should in no way be construed to encompass any areas beyond the specific area 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 understanding of the project and the client's needs, our proposal (Proposal No. P-0704720-CR dated August 14, 2020) and geotechnical engineering standards normally used on similar projects in this region.

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 conclusions 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.

8. SELECTED REFERENCES

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Keller Crossing Project
Winchester, Riverside County, California

Project No. 2453-CR

Scale: As Shown

0 0.8 mi

Figure 1

Site Location Map

GEOTEK

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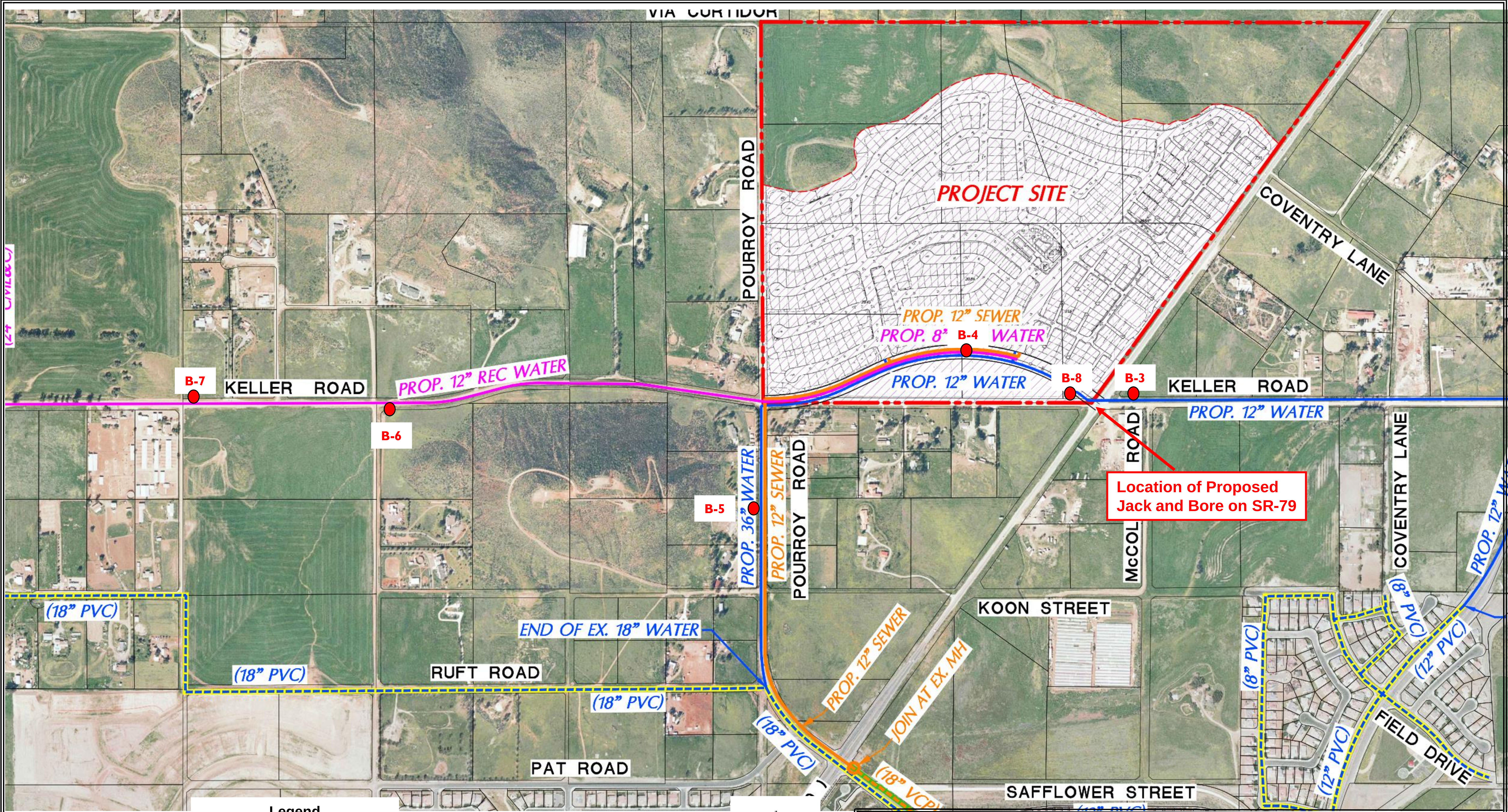
Keller Crossing Project
Winchester, Riverside County, California

GeoTek Project No. 2453-CR

GEOTEK

Figure 2a

Geotechnical and Exploration Location Map



Legend

(Locations are approximate)

B-8
- Exploratory Geotechnical Boring

SCALE: 1"=300'
0 300 600 900 1,200

NORTH

DR Horton Los Angeles Holding Company, Inc.

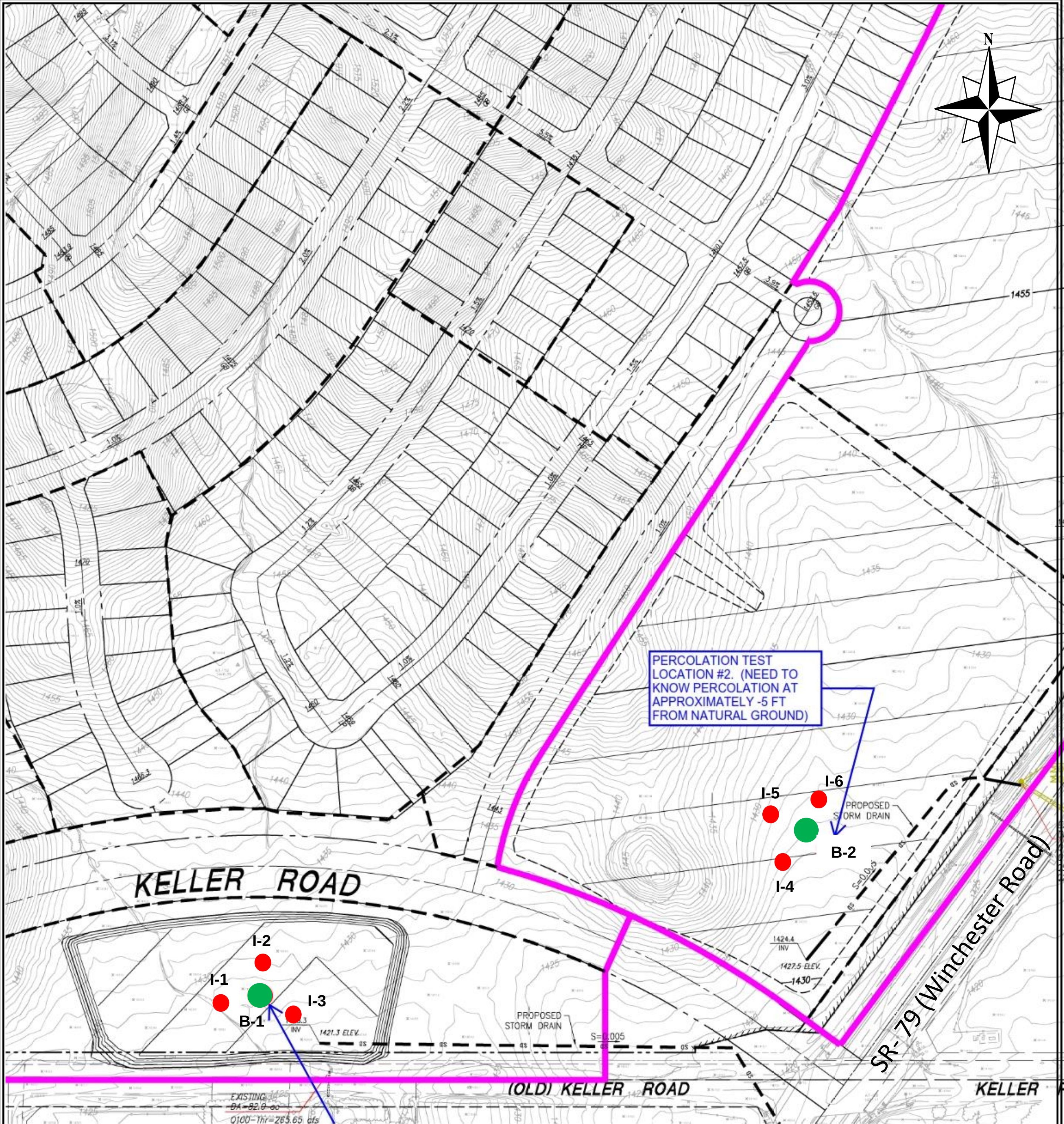
Keller Crossing Project
Winchester, Riverside County, California

Project No. 2453-CR

GEOTEK

Figure 2b

Exploration Location Map



Legend
(Locations are Approximate)

I-6 Infiltration Testing Boring

B-2 Geotechnical Exploratory Boring

Scale: As Shown

0 120 ft

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Keller Crossing Project
Riverside County, California

GeoTek Project No. 2453-CR

Figure 3

Infiltration Location Map

GEOTEK

APPENDIX A

LOGS OF EXPLORATORY EXCAVATIONS, LABORATORY TEST RESULTS, AND SEISMIC REFRACTION SURVEY RESULTS BY GSI (2009)

Updated Geotechnical and Infiltration Evaluation

Keller Crossing Project, Winchester Area, Riverside County, California

Project No. 2453-CR

LOG OF TEST PIT TP-01

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

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

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

LOG OF TEST PIT TP-02

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

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

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

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W.O. 5815-A-OC

LOG OF TEST PIT TP-03

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

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

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

LOG OF TEST PIT TP-04

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

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

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

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LOG OF TEST PIT TP-05

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

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

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

LOG OF TEST PIT TP-06

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

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

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

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LOG OF TEST PIT TP-07

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

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

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

LOG OF TEST PIT TP-08

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

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

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

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LOG OF TEST PIT TP-09

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

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

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS

TOPSOIL @ Surface:
Sandy CLAY (CL), fine to coarse grained, brown, slightly moist, soft, with pore holes, rootlets, and trace gravel

BEDROCK (Mzp) @ 2.75'
Biotite phyllite, black, thinly foliated, with limonite and manganese staining, weathered to Clay in places
AND
Quartz syenite (alkali feldspar) schist, light gray and white, moderately thick foliation, very hard
Contact between trending similar to foliation
F @ 3': N85E/90, N60E/88SE
J @ 3': N42W/82SW, N5W/90

Total Depth = 4.5 Feet, Refusal on Hard Bedrock
No Groundwater Encountered, No Caving
Hole Backfilled with Cuttings

LOG OF TEST PIT TP-10

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

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

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE	HAND PEN. (tsf)	MOISTURE (%)	DRY UNIT WT. (pcf)	LAB TESTS
					6.2	98.0	
					17.8	106.9	EI, AL CN
					13.1	103.0	

TOPSOIL @ Surface:
Clayey SAND (SC), fine to medium grained, brownish gray, dry to slightly moist, loose

VERY OLD AXIAL CHANNEL DEPOSITS (Qvoa) @ 1.5':
Sandy CLAY (CL), fine to medium grained, brown, slightly moist to moist, medium stiff to stiff, with pore holes and trace, angular gravel
@ 3.5': with increasing Clay
@ 4': CLAY (CL), reddish brown, moist, stiff, with trace, coarse graines Sand and gravel
@ 5': CLAY with Sand (CL), fine grained, dark gray, slightly moist, stiff, with limonite staining and limonite-filled root holes and trace gravel, Munsell 2.5YR 4/1

BEDROCK (Mzp) @ 7.5'
Biotite phyllite, black, thinly foliated, with limonite, manganese, and trace sulfur

Total Depth = 9 Feet, Refusal on Hard Bedrock
No Groundwater Encountered, No Caving
Hole Backfilled with Cuttings

LOG OF TEST PIT TP-11

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

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

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

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APPENDIX B2

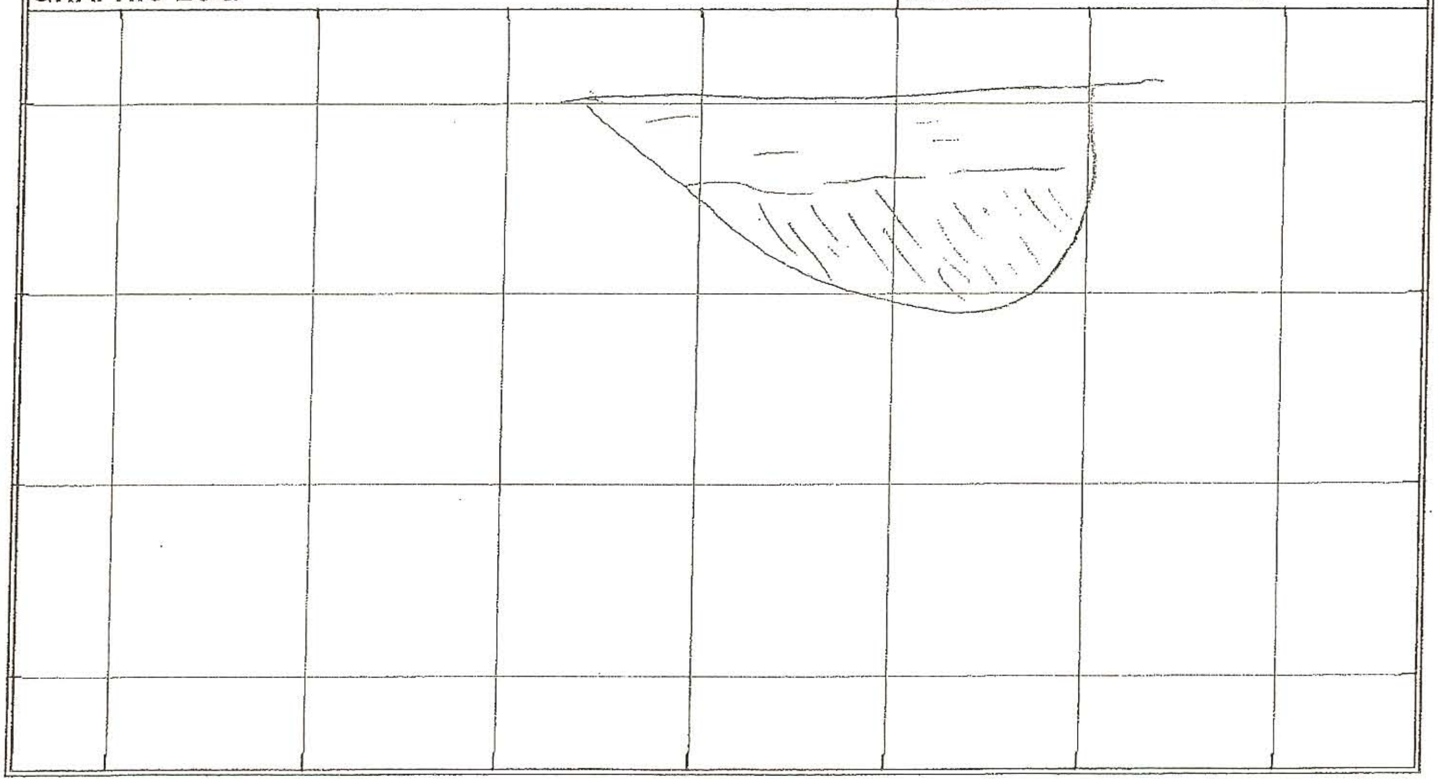
TRENCH LOGS BY EUC

TRENCH LOG No. 1

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

SUBSURFACE CONDITIONS	**REMARKS** Logged By: jd
Surficial Soil 0-2.5 Silty Clay, dark brown, dry, hard. At 12" becomes moist, stiff Bedrock (Mzp) 2.5-5.5 Phyllite, dark gray, damp, fissile. Excavates with effort.	 Foliation: N40W,42NE

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



TRENCH LOG No. 2

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

SUBSURFACE CONDITIONS	**REMARKS** **Logged By:** jd
Surficial Soil 0-3 Silty Clay, dark gray, moist, firm, porous. Upper 12" dry and loose. Grades to bedrock. Bedrock (Mzp) 3-5 Phyllite, dark gray, damp to moist, fissile. Excavates with ease 3-4 Excavates with effort 4-5.	Bag Sample 1-2 4-5 Drive Sample Blows 3 12 Foliation N15E,60E Joint E-W, 58N

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

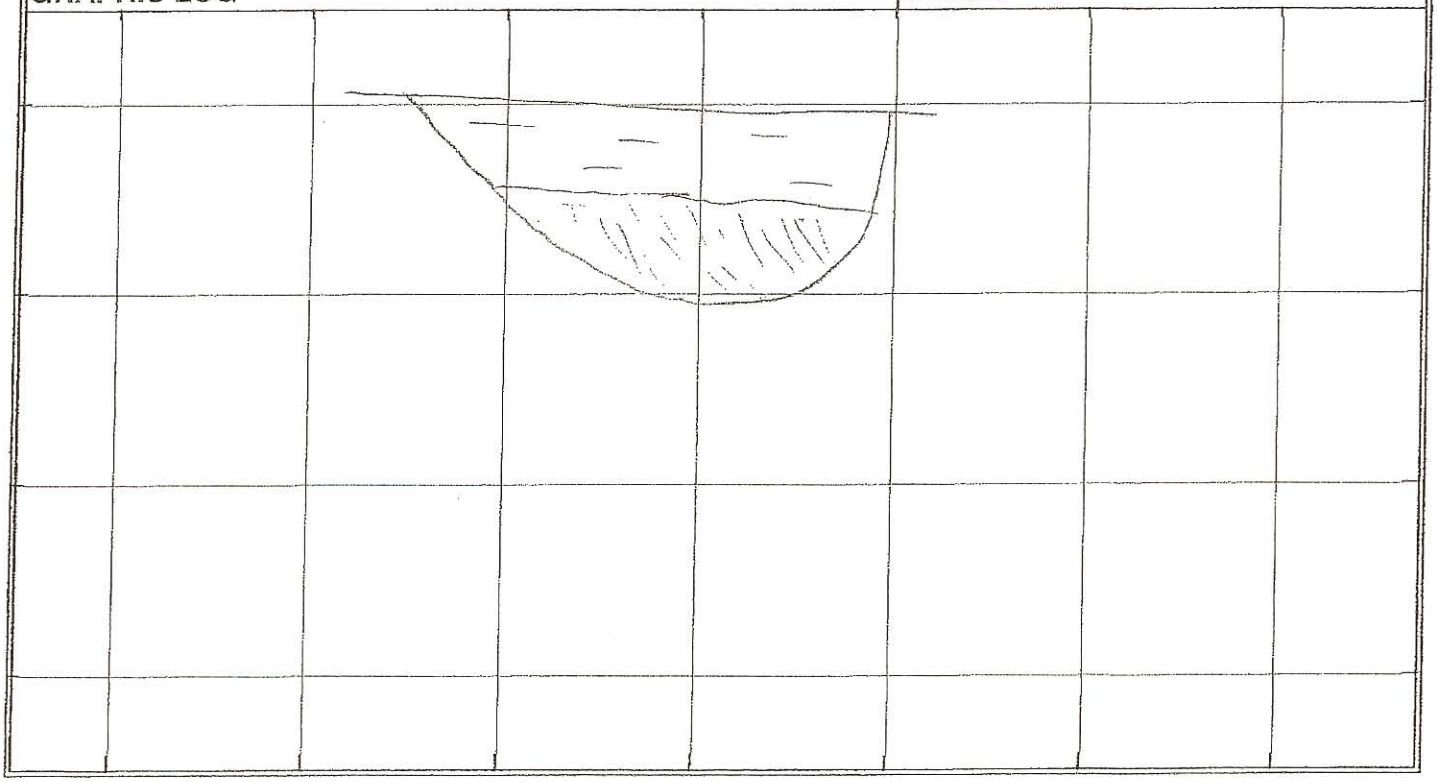


Plate: A - 2

TRENCH LOG No. 3

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

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

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

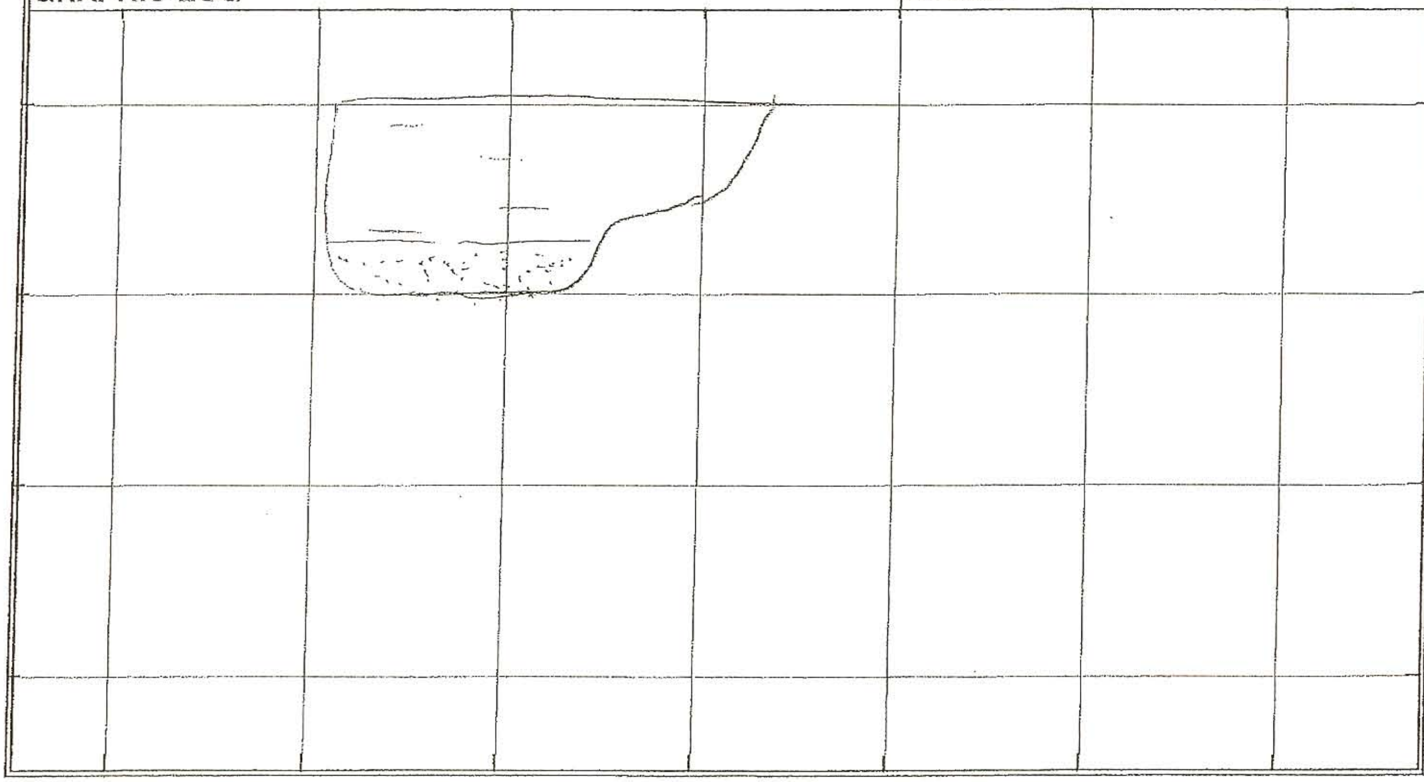


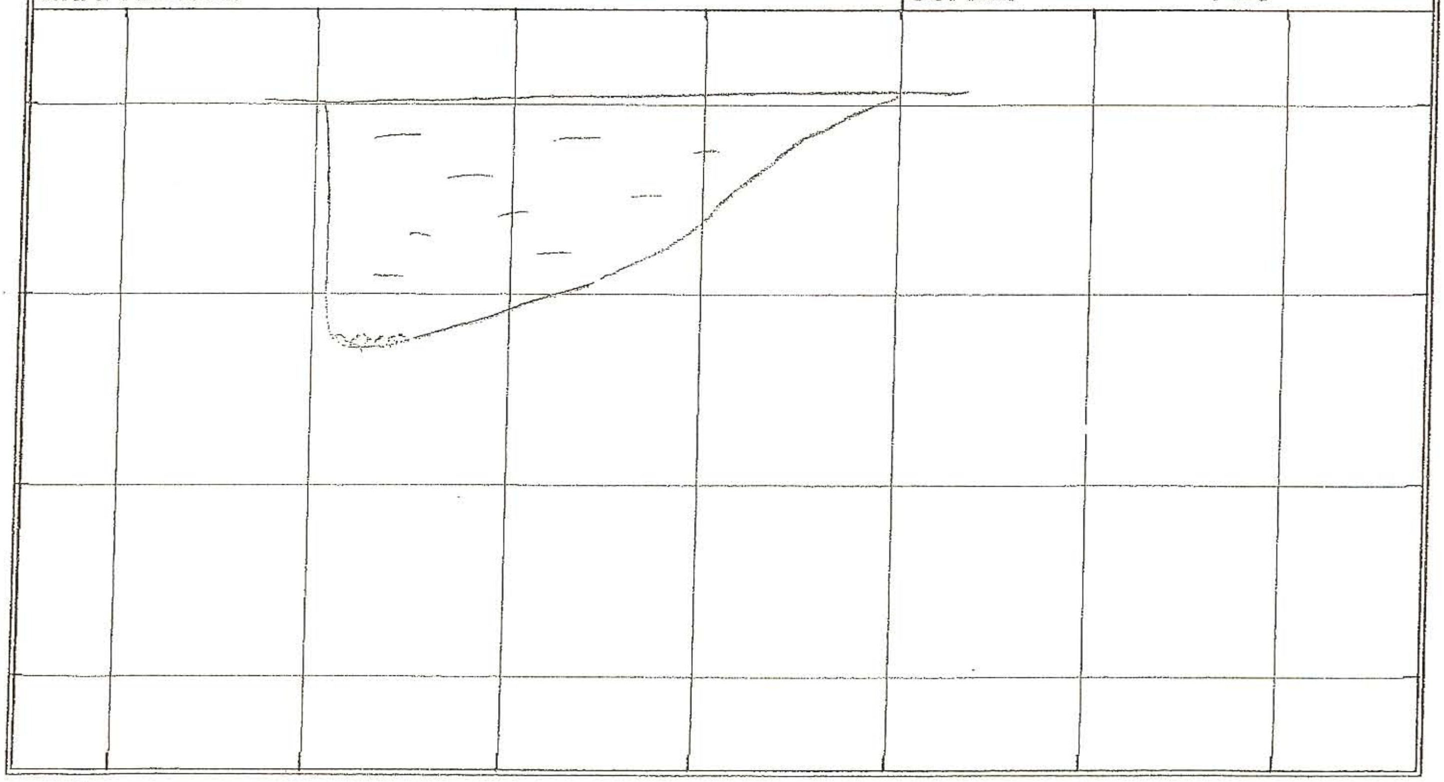
Plate: A - 3

TRENCH LOG No. 4

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

SUBSURFACE CONDITIONS	**REMARKS** — **Logged By:** jd
Very old Channel Deposits (Qvoa) 0-6 Clayey silt, dark gray, moist, firm, porous. Upper 12" dry and loose.	Bag Sample 2-3
Bedrock (Mzp) @6' Gray aphanitic mafic rock, very hard. Refusal at 6 feet.	Drive Sample: 2' — Blows: 10

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

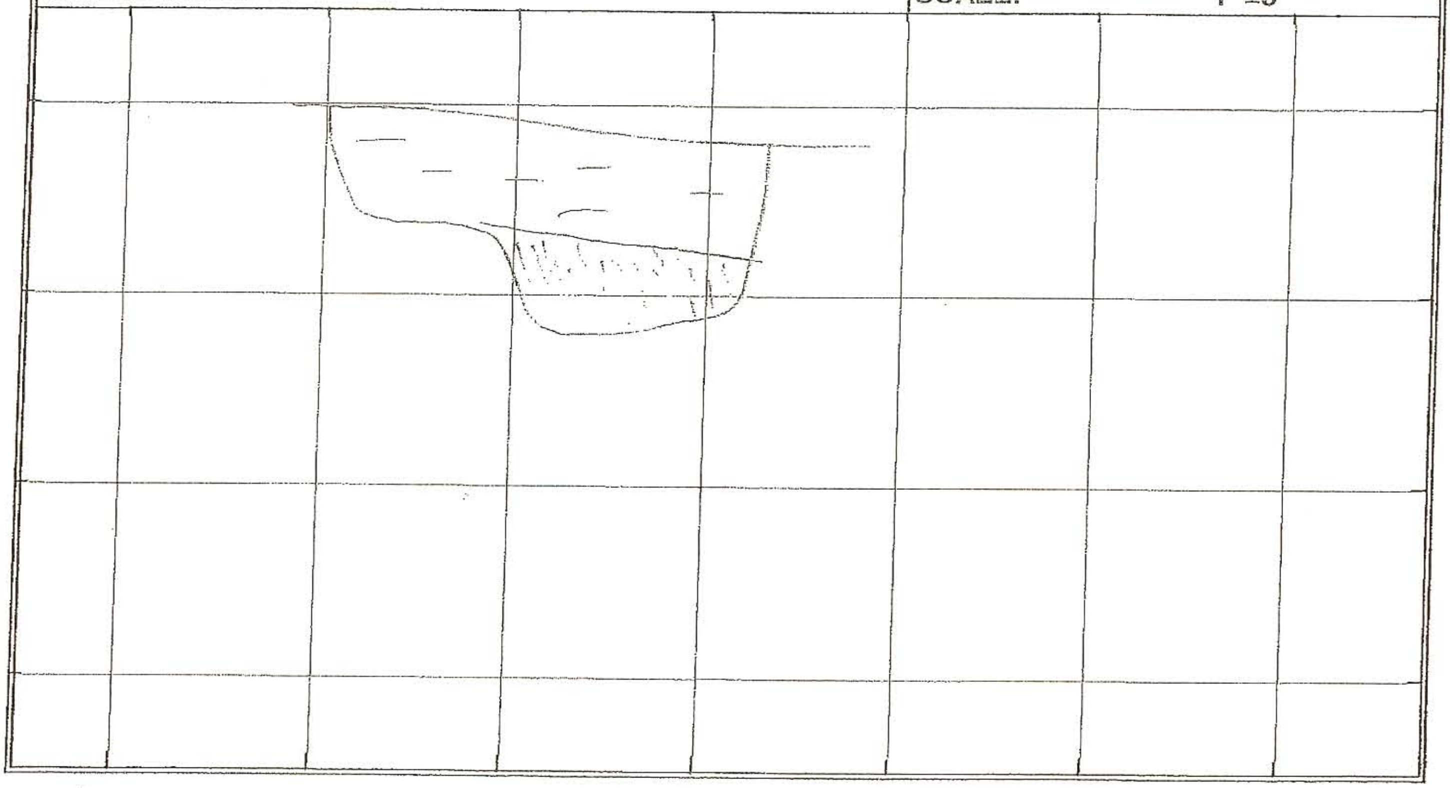


TRENCH LOG No. 5

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

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

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

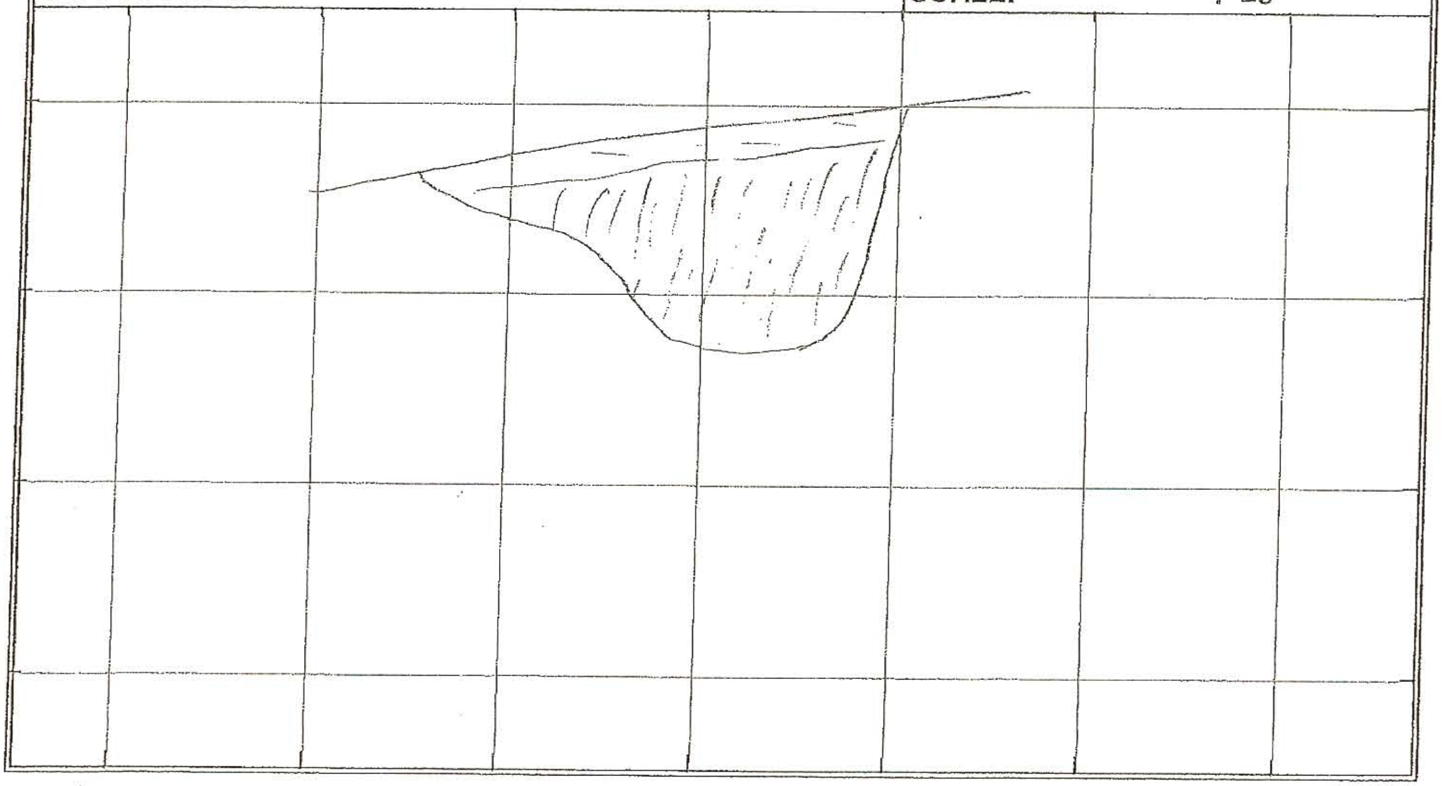


TRENCH LOG No. 6

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

SUBSURFACE CONDITIONS	**REMARKS** **Logged By:** jd
Surficial Soil 0-1 Clayey Silt, gray, dry to damp, loose.	
Bedrock (Mzp) 1-4 Phyllite, dark gray, damp, fissile, soft rock. Excavates with difficulty 3-4 Refusal at 4	Foliation N80W,85S

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



TRENCH LOG No. 7

Client:	**Brookfield Land**	**W.O.**	81-02081-0011
Project:	Hanna Property	**Date:**	15-Aug-05
Location:	Winchester	**Sheet:**	1 of 1

Est. Surface Elev: 1461	**Total Depth:** 7	**Subcontractor:** Al-Roy Drilling
Orientation: N--->	**Length:** 13	**Equip. type:** JD330

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

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

Bedrock (Mzp)

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

REMARKS

Logged By: jd

Bag Sample
1-3

Drive Sample	Blows	
3	20	(nr)
7	50	

Foliation N70W,77SW
Joint N17E,84E

GRAPHIC LOG

SCALE: 1"=5'

TRENCH LOG No. 8

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

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

0-2 Silt, with trace of clay, brown, damp, firm, porous

2-5 Silt with sub angular chunks of phyllite and quartzite difficult excavation

Bedrock (Mzp)

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

REMARKS

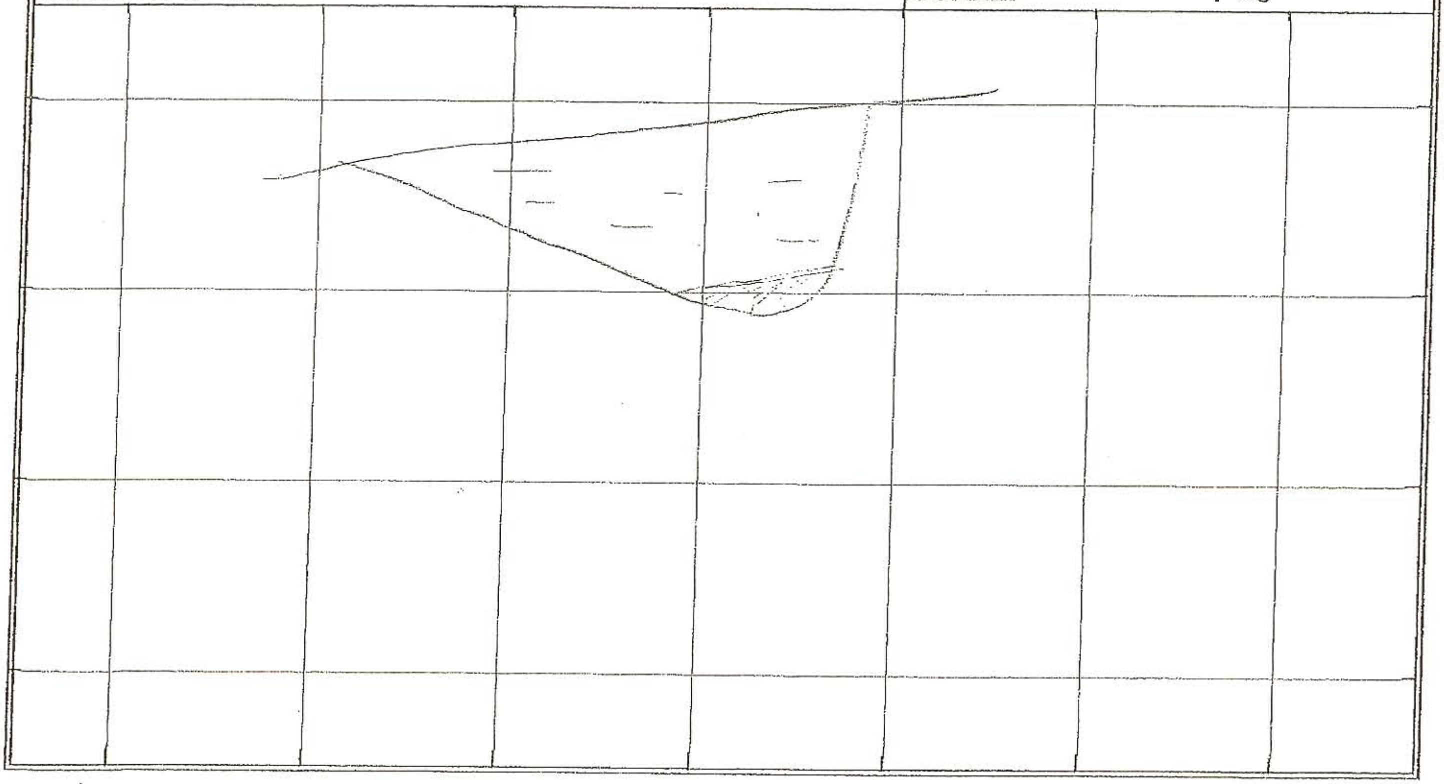
Logged By: jd

Bag Sample
1-2

Foliation NN88W,56N
Joint N40E,90

GRAPHIC LOG

SCALE: 1"=5'



TRENCH LOG No. 9

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

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

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

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

Bedrock (Mzp)

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

REMARKS

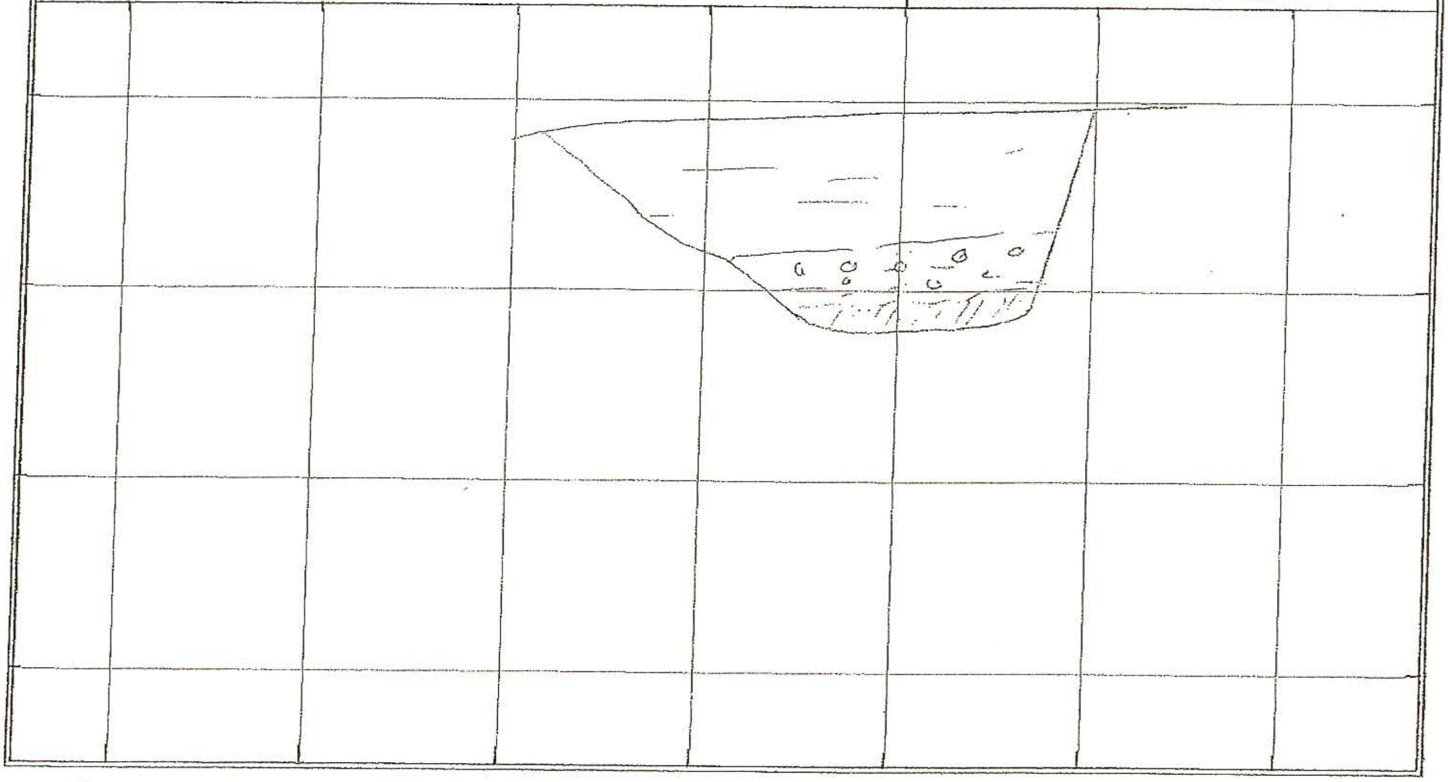
Logged By: jd

Drive Sample	Blows
2	6

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

GRAPHIC LOG

SCALE: 1"=5'



TRENCH LOG No. 10

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

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

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

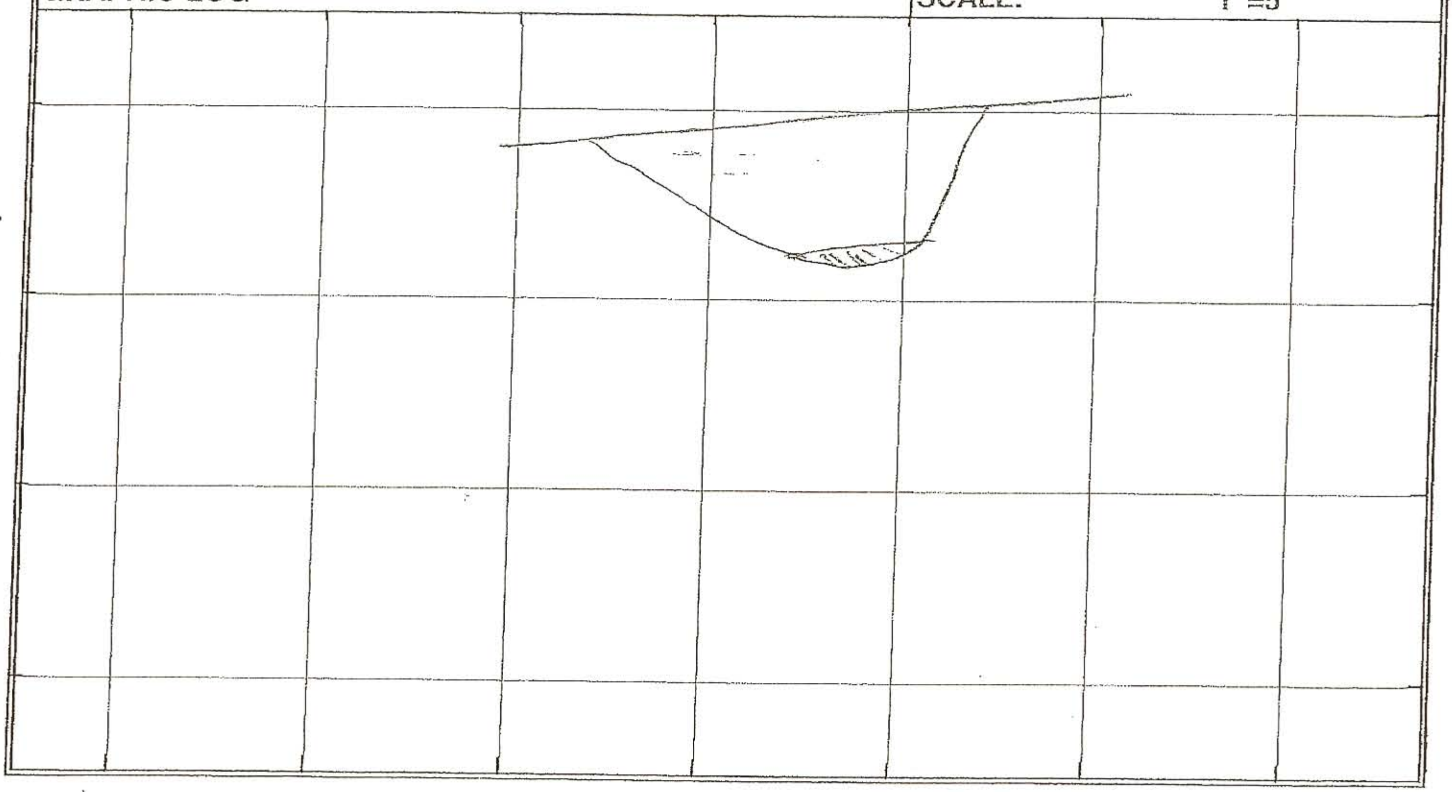


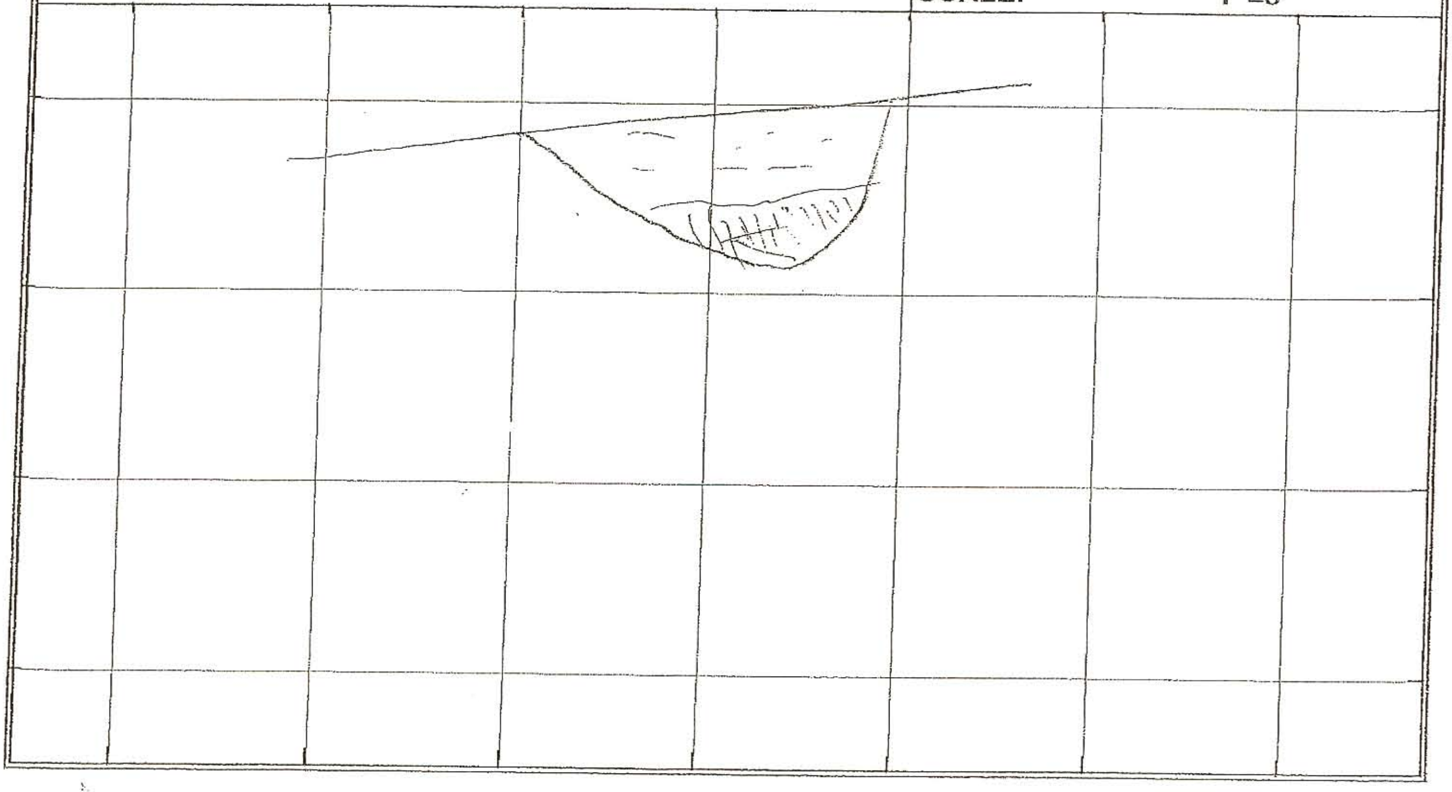
Plate: A - 10

TRENCH LOG No. 11

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

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

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

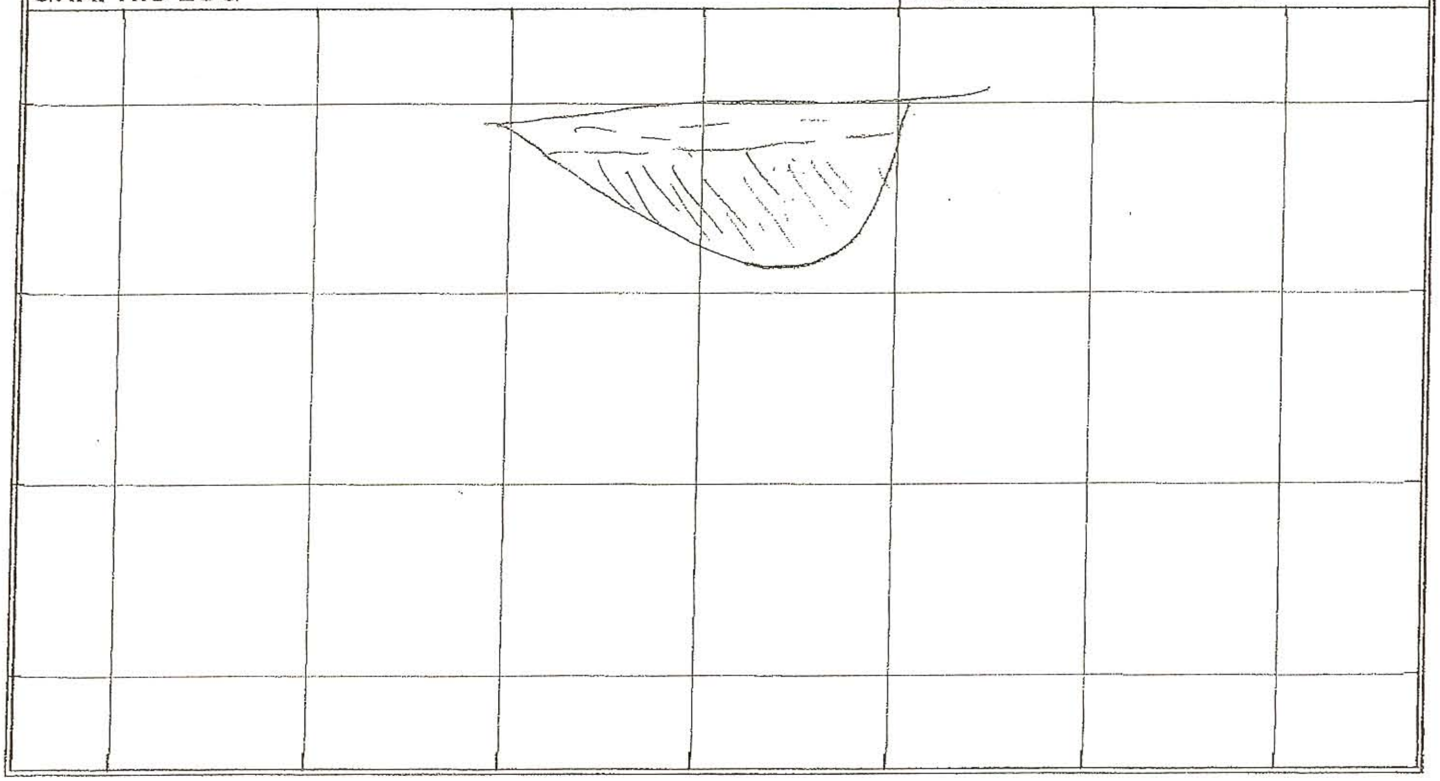


TRENCH LOG No. 12

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

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

GRAPHIC LOG — **SCALE:** 1"=5'



TRENCH LOG No. 13

Client:	**Brookfield Land**	**W.O.**	81-02081-0011
Project:	Hanna Property	**Date:**	15-Aug-05
Location:	Winchester	**Sheet:**	1 of 1

Est. Surface Elev: 1465	**Total Depth:** 5	**Subcontractor:** Al-Roy Drilling
Orientation: N20E--->	**Length:** 10	**Equip. type:** JD330

SUBSURFACE CONDITIONS

Surficial Soil

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

@3 Cobble layer.

Bedrock (Mzp)

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

REMARKS

Logged By: jd

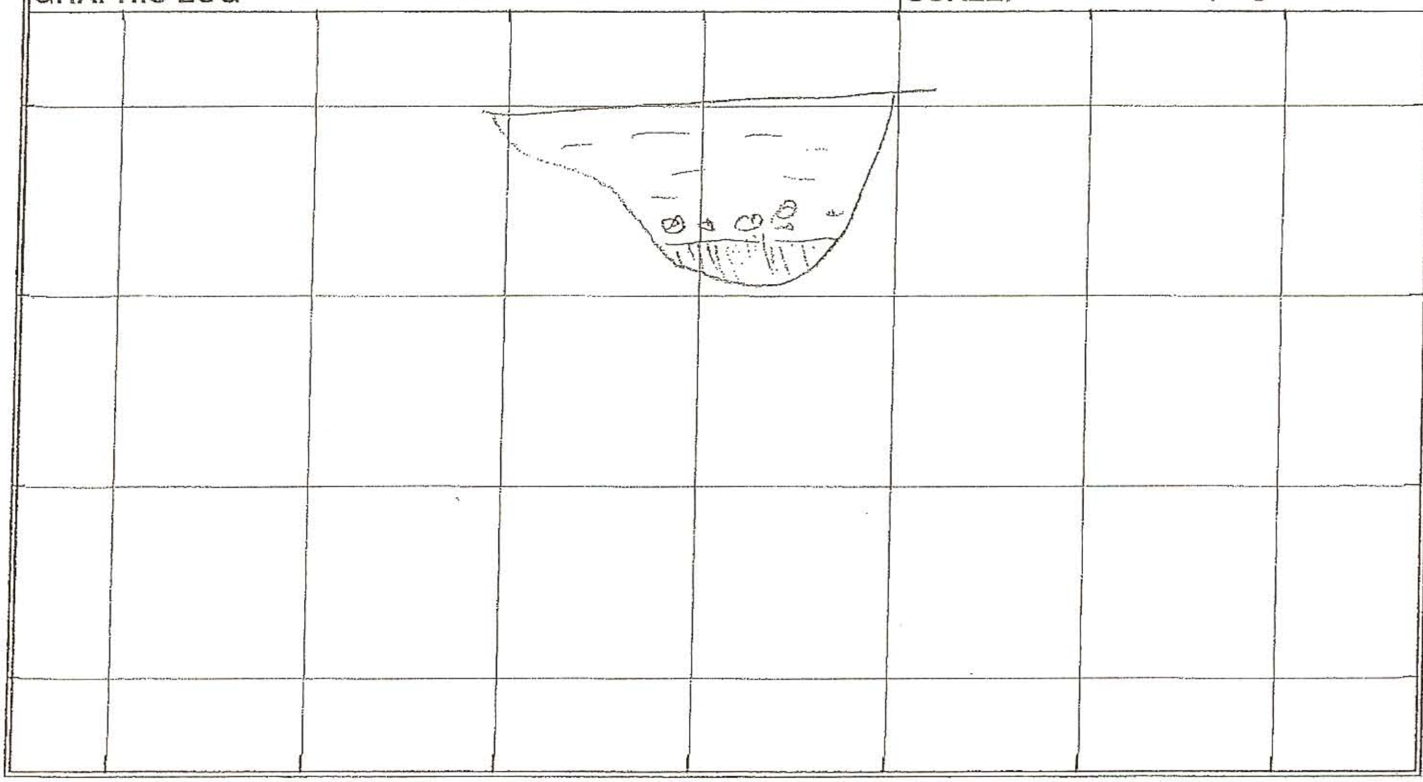
Bag Sample
1-3

Drive Sample	Blows
3	25

Foliation N67W,85N
Joint N27E,47NW

GRAPHIC LOG

SCALE: 1"=5'



TRENCH LOG No. 14

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

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

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

@3 Cobble layer

@7 Cobble layer

Bedrock (Mzp)

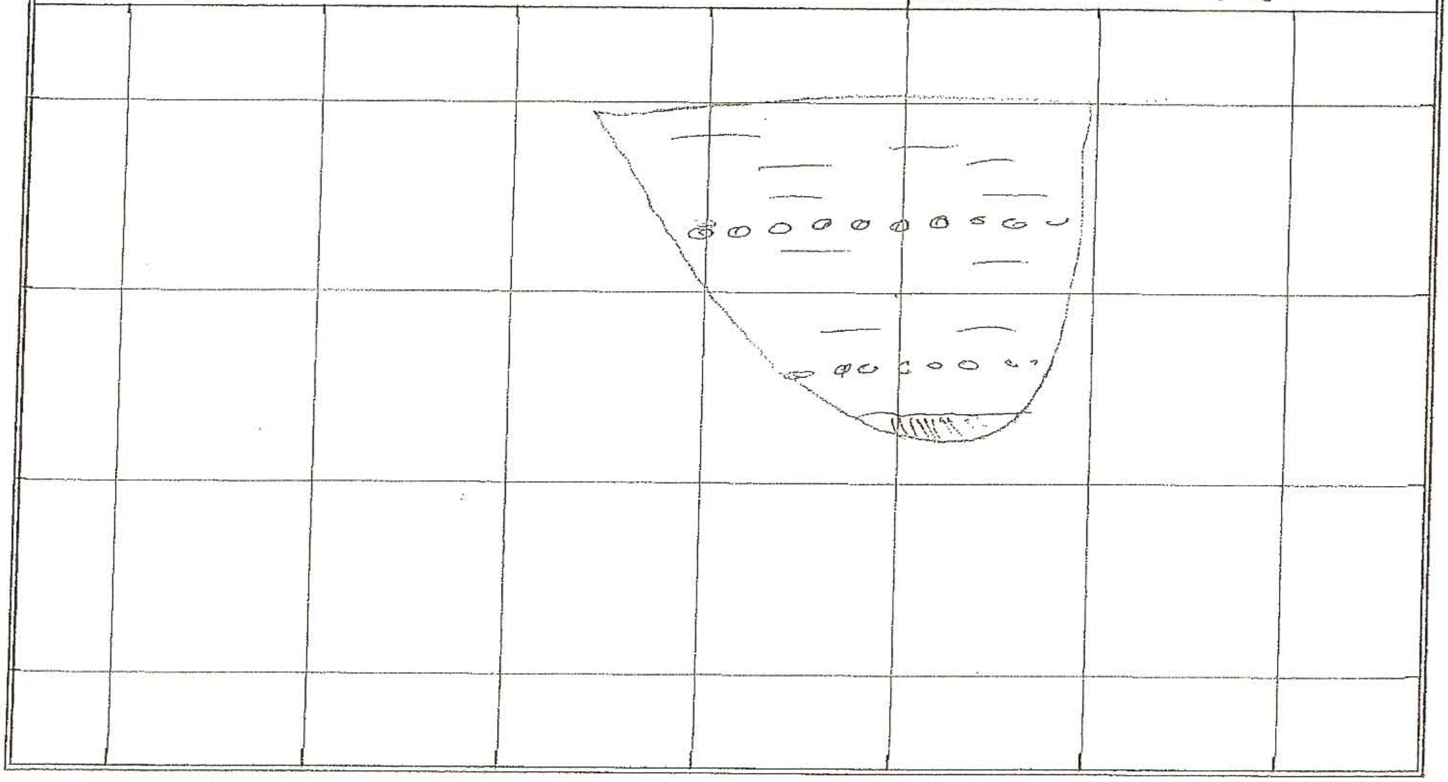
8-8.5 Phyllite, dark gray, damp, fissile, soft rock. Thin quartzite interbeds . Excavates with ease

REMARKS **Logged By:** jd

Bag Sample
1-3

Drive Sample	Blows
3	18
5	33

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

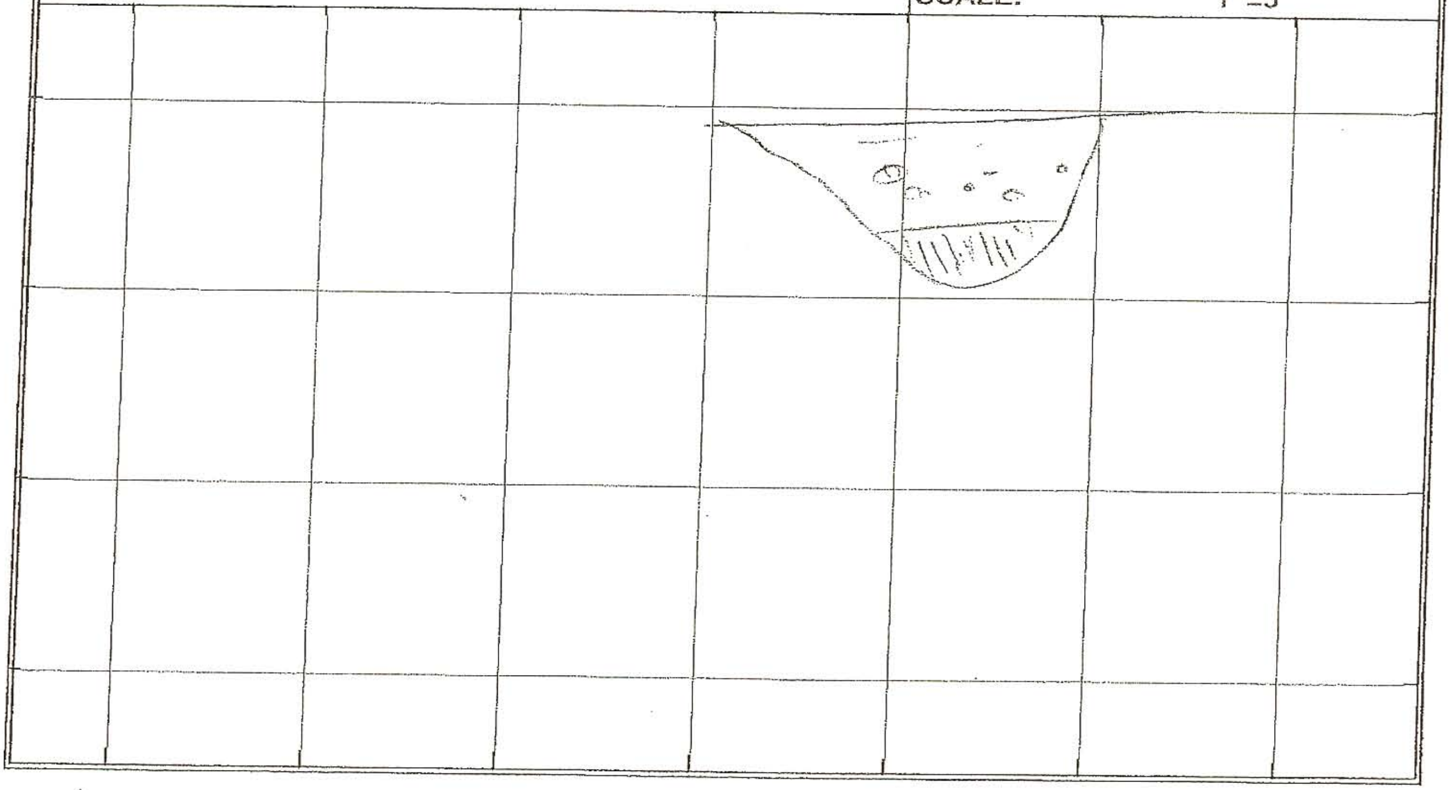


TRENCH LOG No.15

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

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

GRAPHIC LOG — **SCALE:** 1"=5'



TRENCH LOG No. 16

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

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

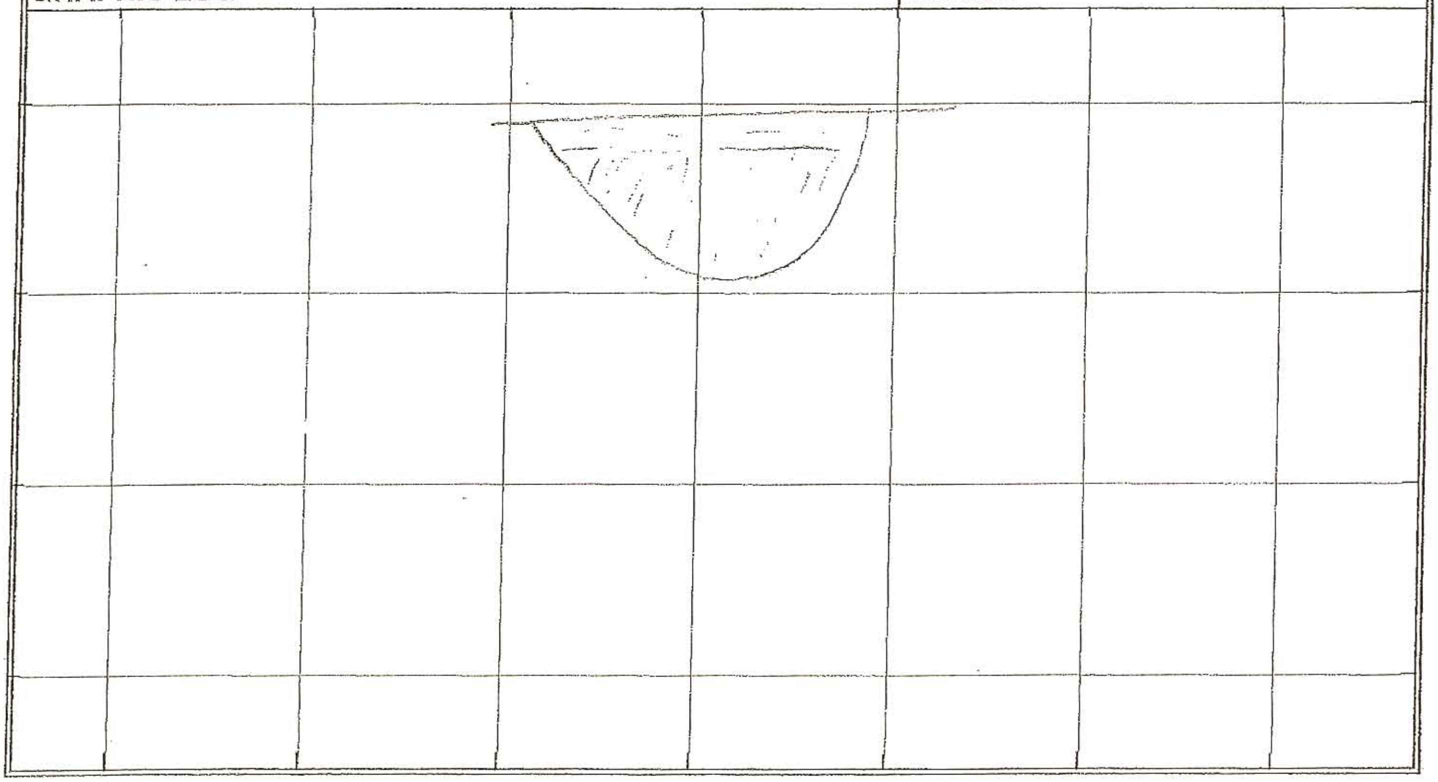
GRAPHIC LOG **SCALE:** 1"=5'

TRENCH LOG No. 17

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

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

GRAPHIC LOG — **SCALE:** 1"=5'



TRENCH LOG No. 18

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

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

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

Bedrock (Mzp)

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

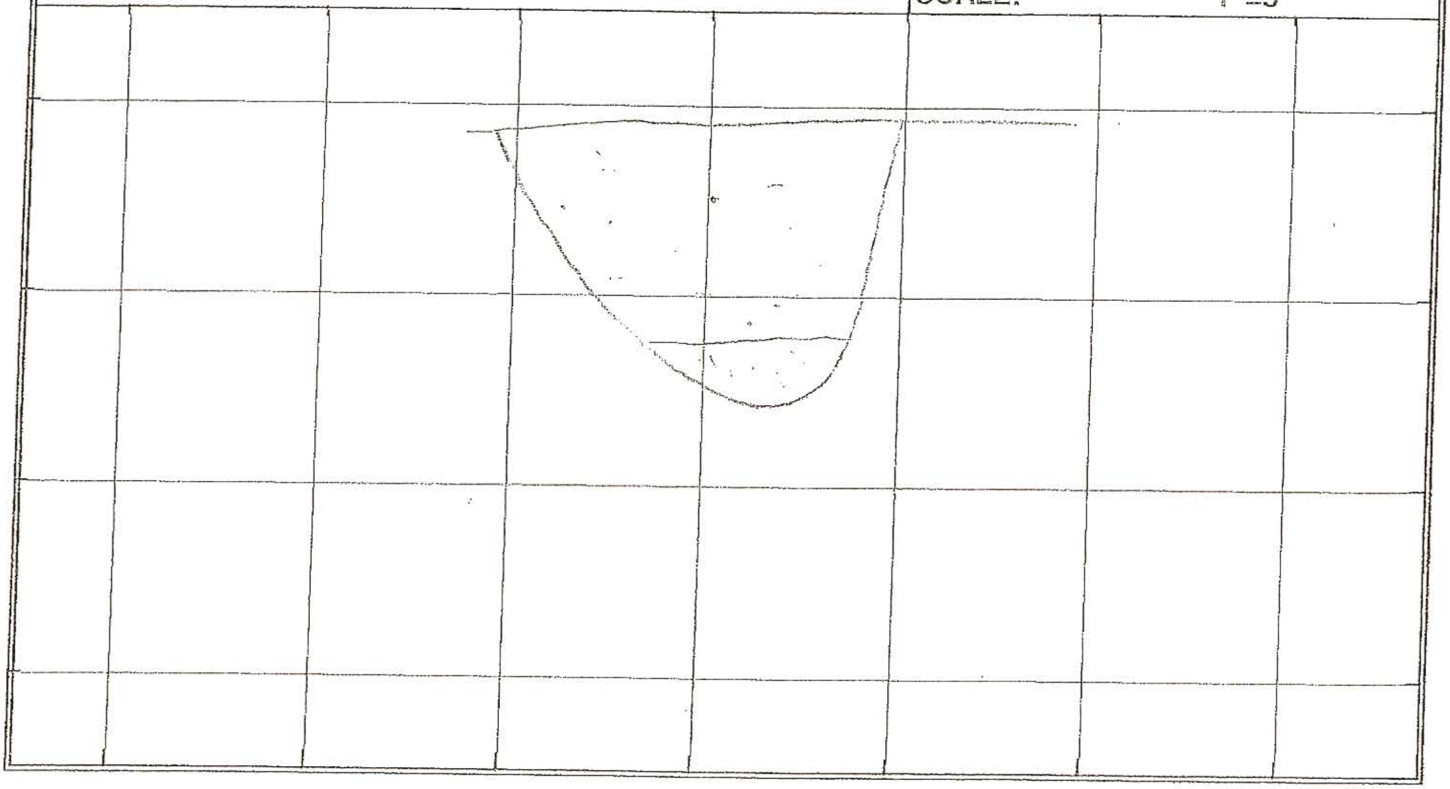
REMARKS

Logged By: jd

Drive Sample	Blows
3	6

GRAPHIC LOG

SCALE: 1"=5'



TRENCH LOG No. 19

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

SUBSURFACE CONDITIONS

Surficial Soil
- 0-6.5 Sandy silt, brown, dry, soft/loose
- @1 Becomes moist, soft to firm and porous

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

REMARKS **Logged By:** jd

Bag Sample
3-5

Drive Sample	Blows
3	9

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

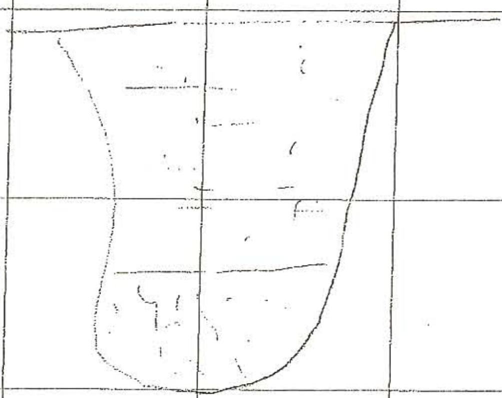


Plate: A - 19

EBERHART / UNITED CONSULTANTS, INC.

REV. 6/22/04tl1

TRENCH LOG No. 20

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

SUBSURFACE CONDITIONS

Very old Channel Deposits (Qvoa)

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

Bedrock (Mzp)

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

REMARKS

Logged By: jd

Foliation E-W,74N

GRAPHIC LOG

SCALE: 1"=5'

TRENCH LOG No. 21

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

SUBSURFACE CONDITIONS

Surficial Soil

0-3 Clayey silt, dark gray, moist, soft to firm, porous

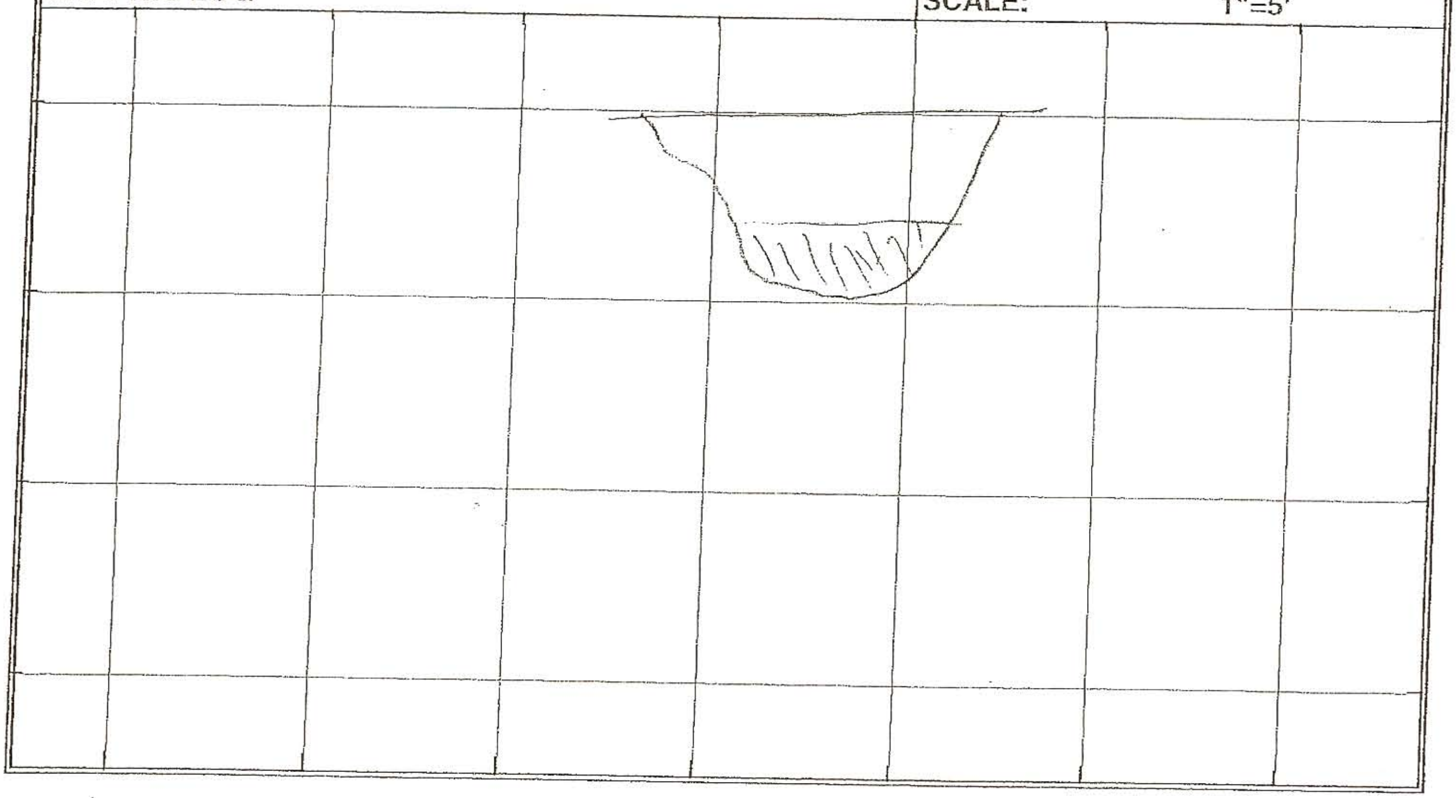
Bedrock (Mzp)

3-5 Phyllite, dark gray, damp, fissile, soft rock. Excavates with effort.

REMARKS **Logged By:** jd

Foliation N80W, 62NE

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

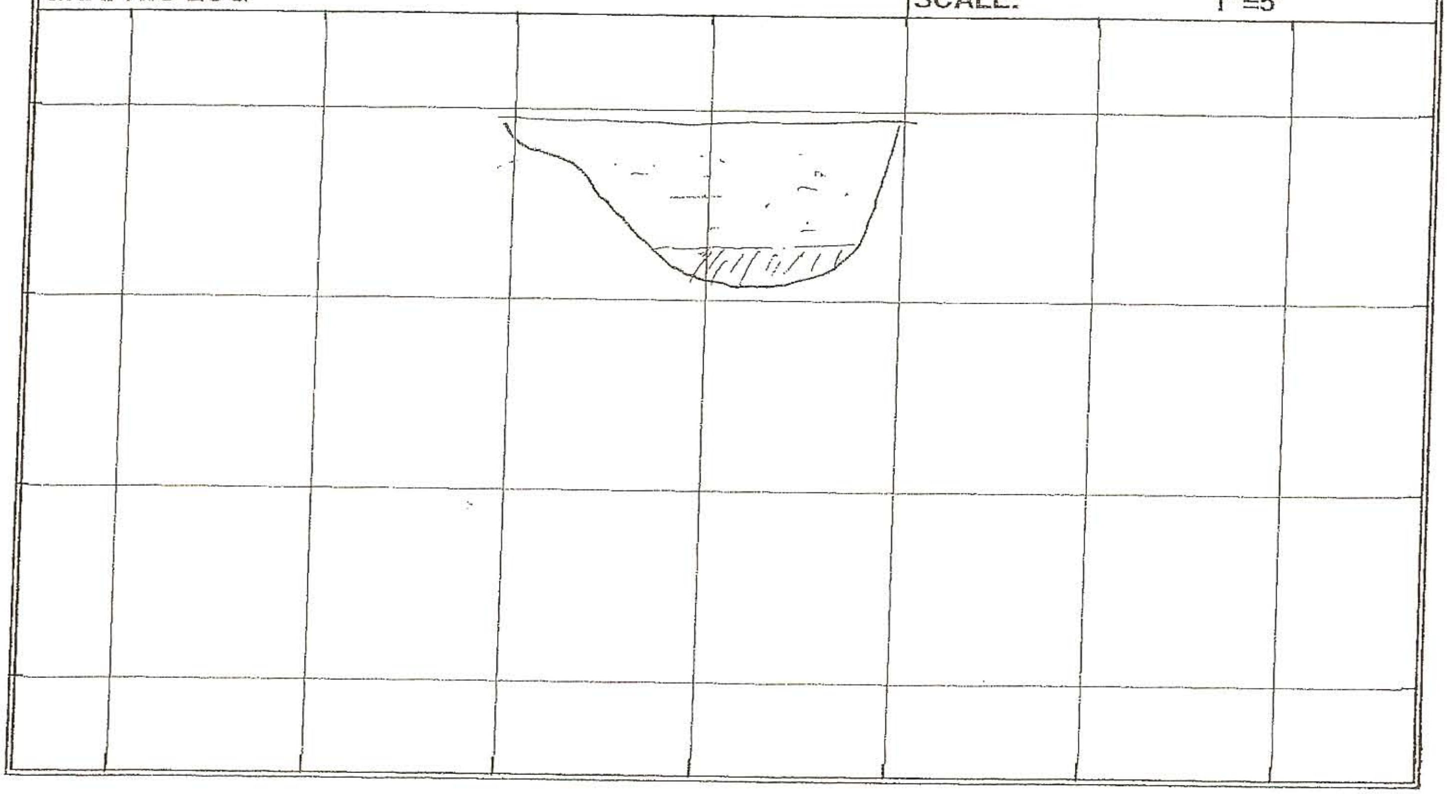


TRENCH LOG No. 22

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

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

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



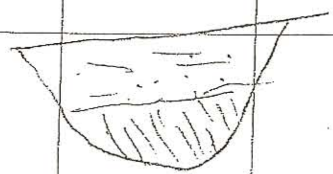
TRENCH LOG No. 23

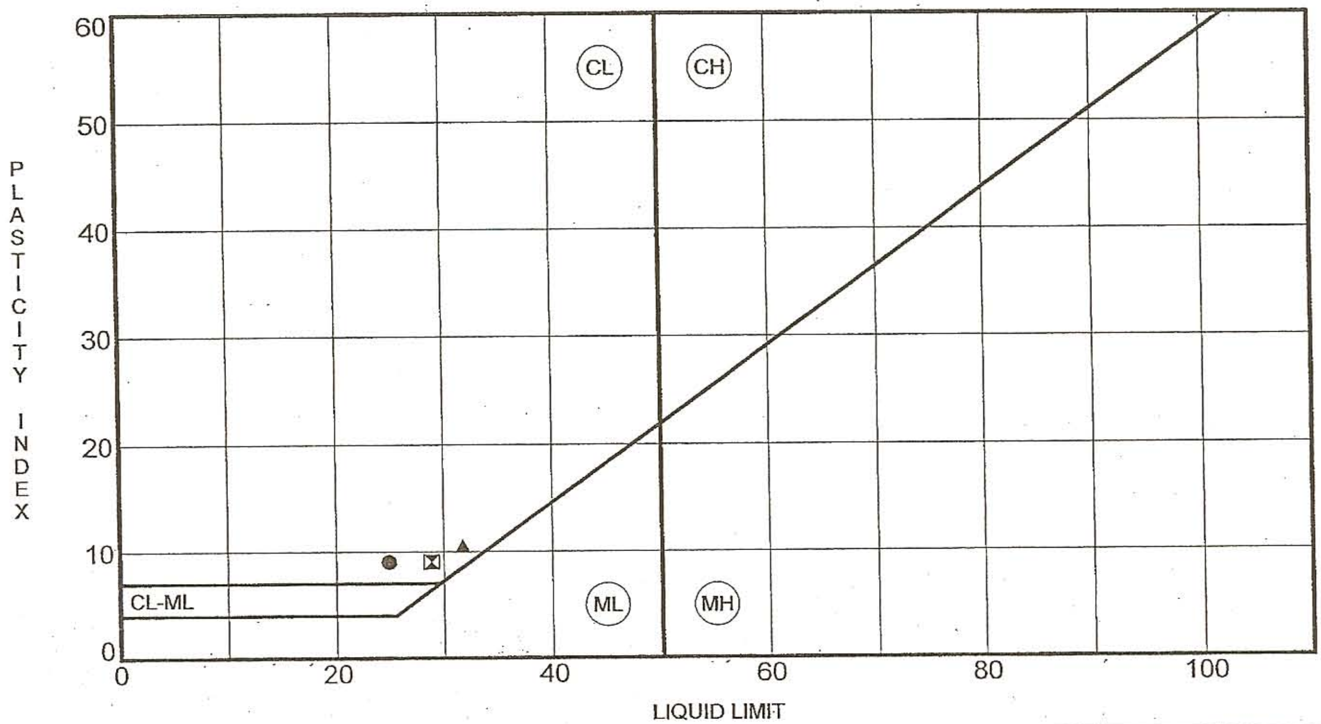
Client:	**Brookfield Land**	**W.O.**	81-02081-0011
Project:	Hanna Property	**Date:**	16-Aug-05
Location:	Winchester	**Sheet:**	1 of 1

Est. Surface Elev: 1516	**Total Depth:** 3.5	**Subcontractor:** Al-Roy Drilling
Orientation: N--->	**Length:** 6	**Equip. type:** JD330

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

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





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

GSI

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

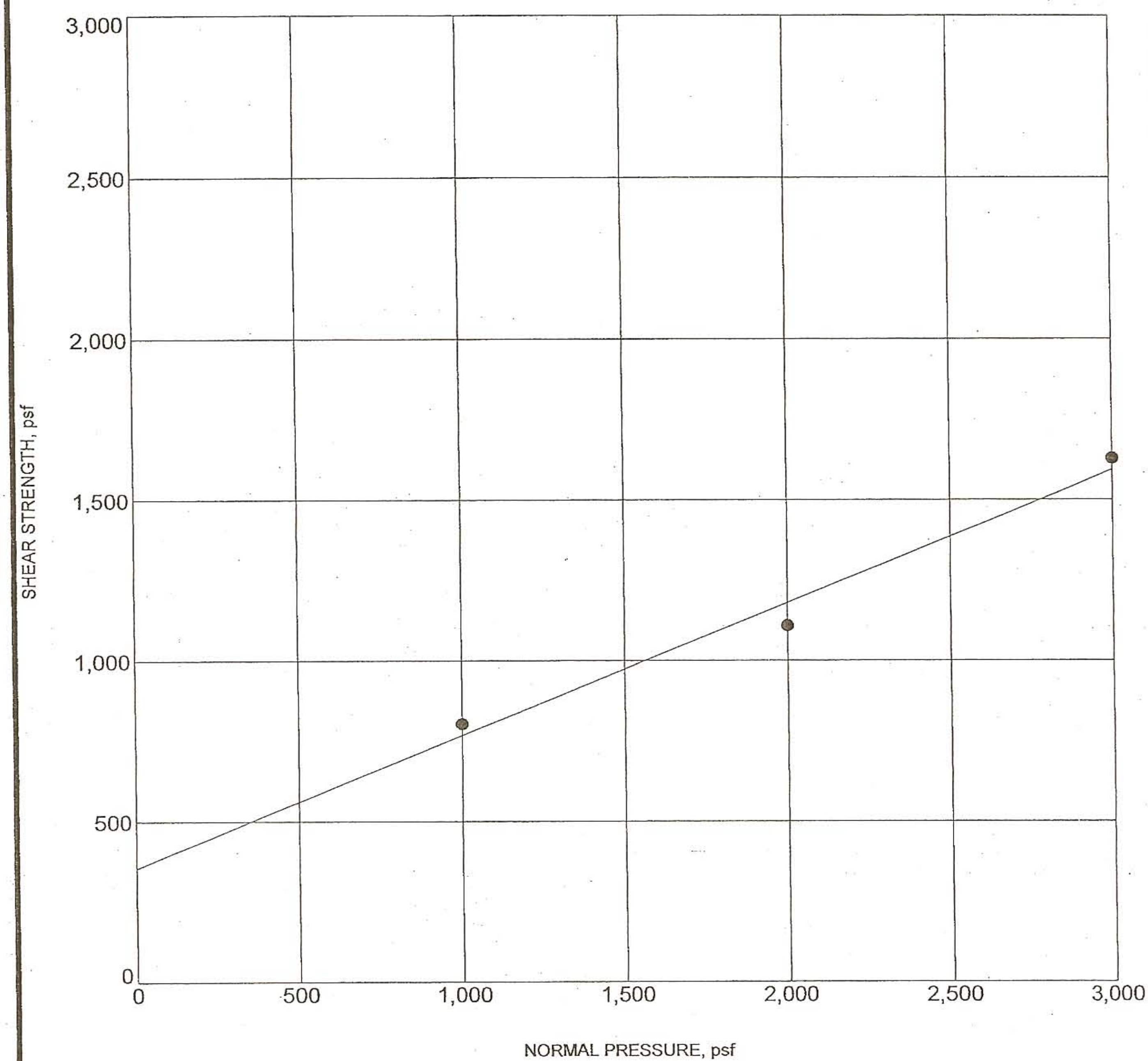
ATTERBERG LIMITS' RESULTS

Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE D - 1



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

GSI

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

DIRECT SHEAR TEST

Project: Hanna Property

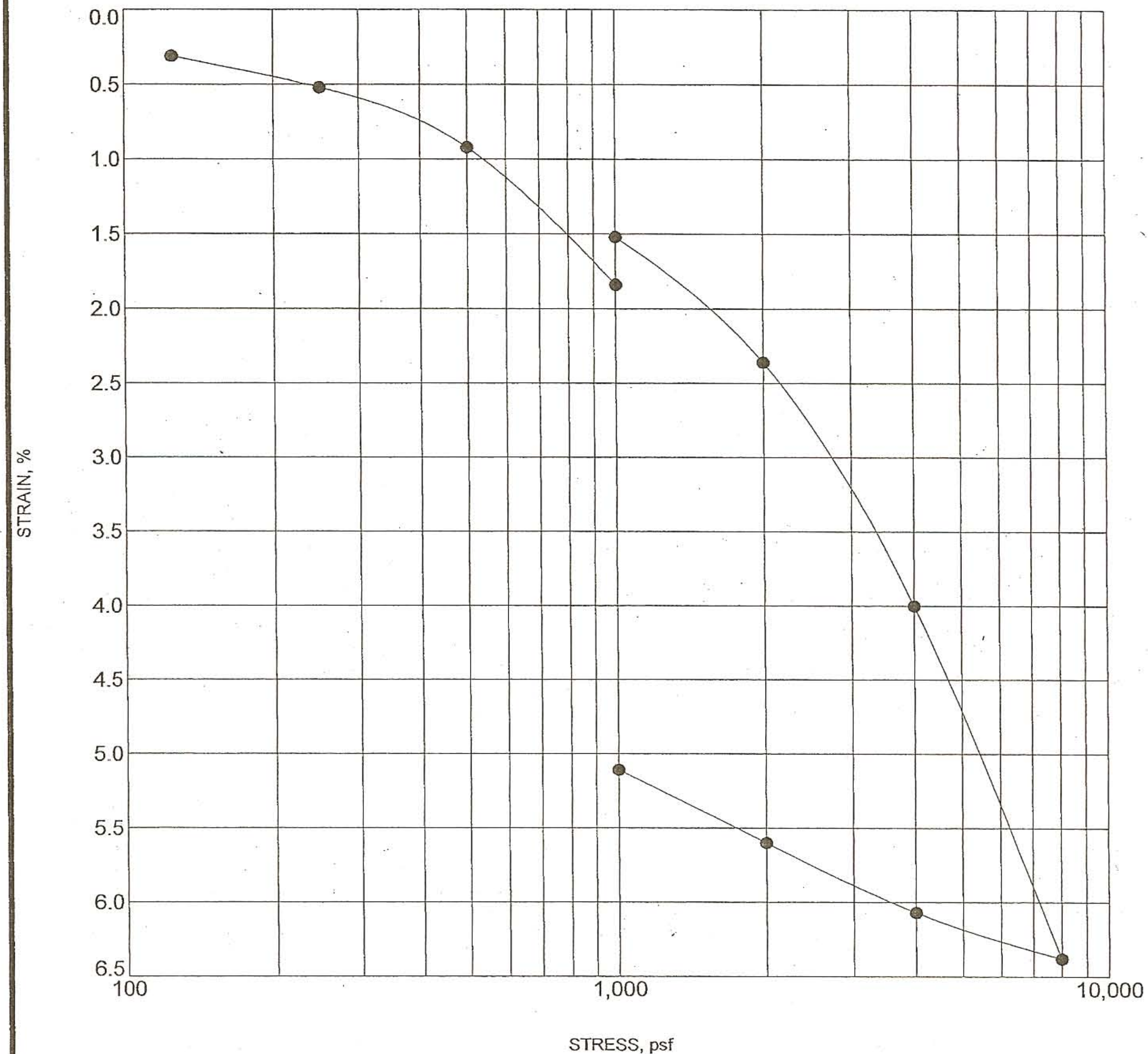
Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE

D - 2

US_DIRECT_SHEAR_5815A.GPJ US_LAB.GDT 2/25/09



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

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

GSI

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

CONSOLIDATION TEST

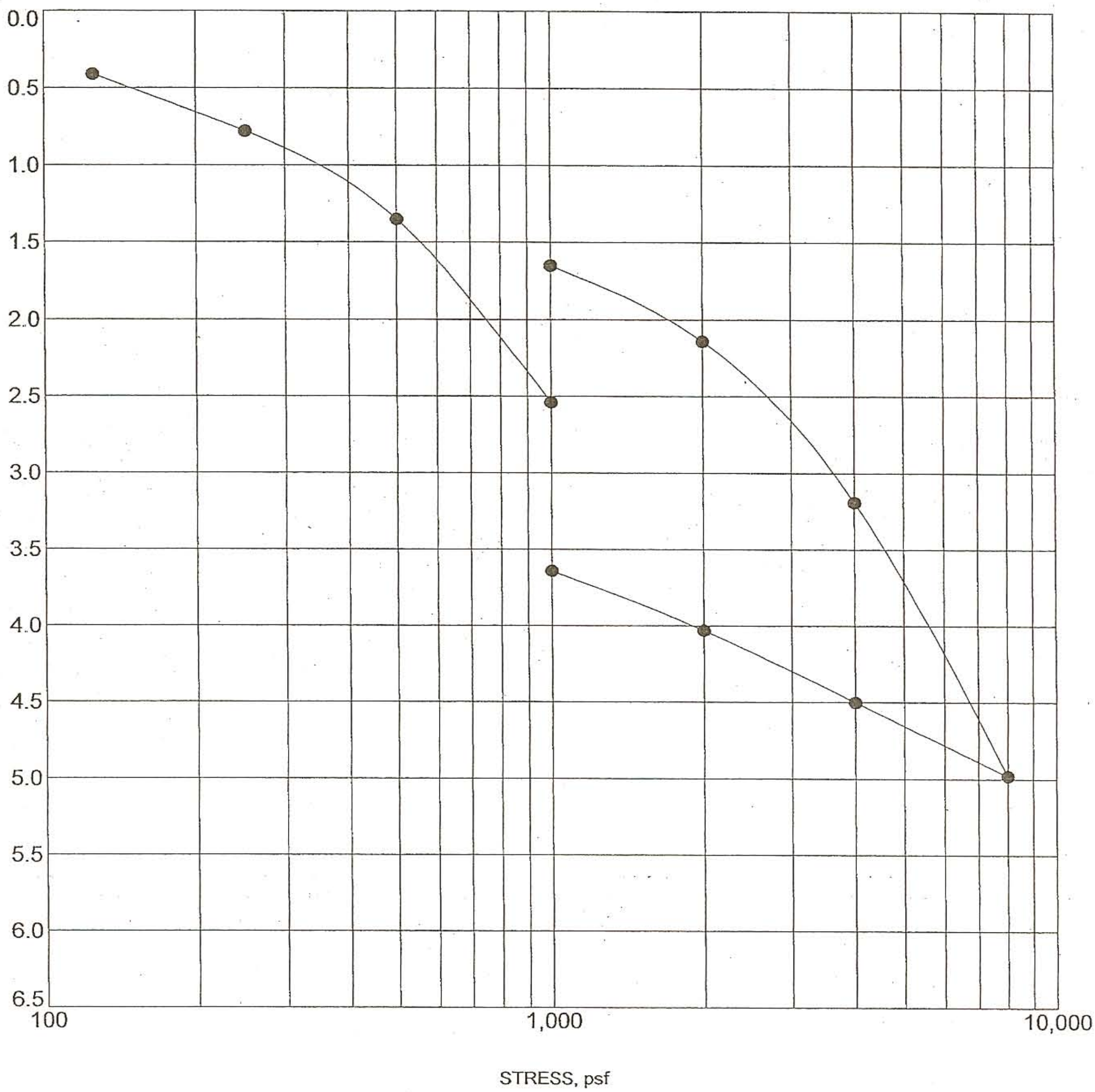
Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE

D - 3



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

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

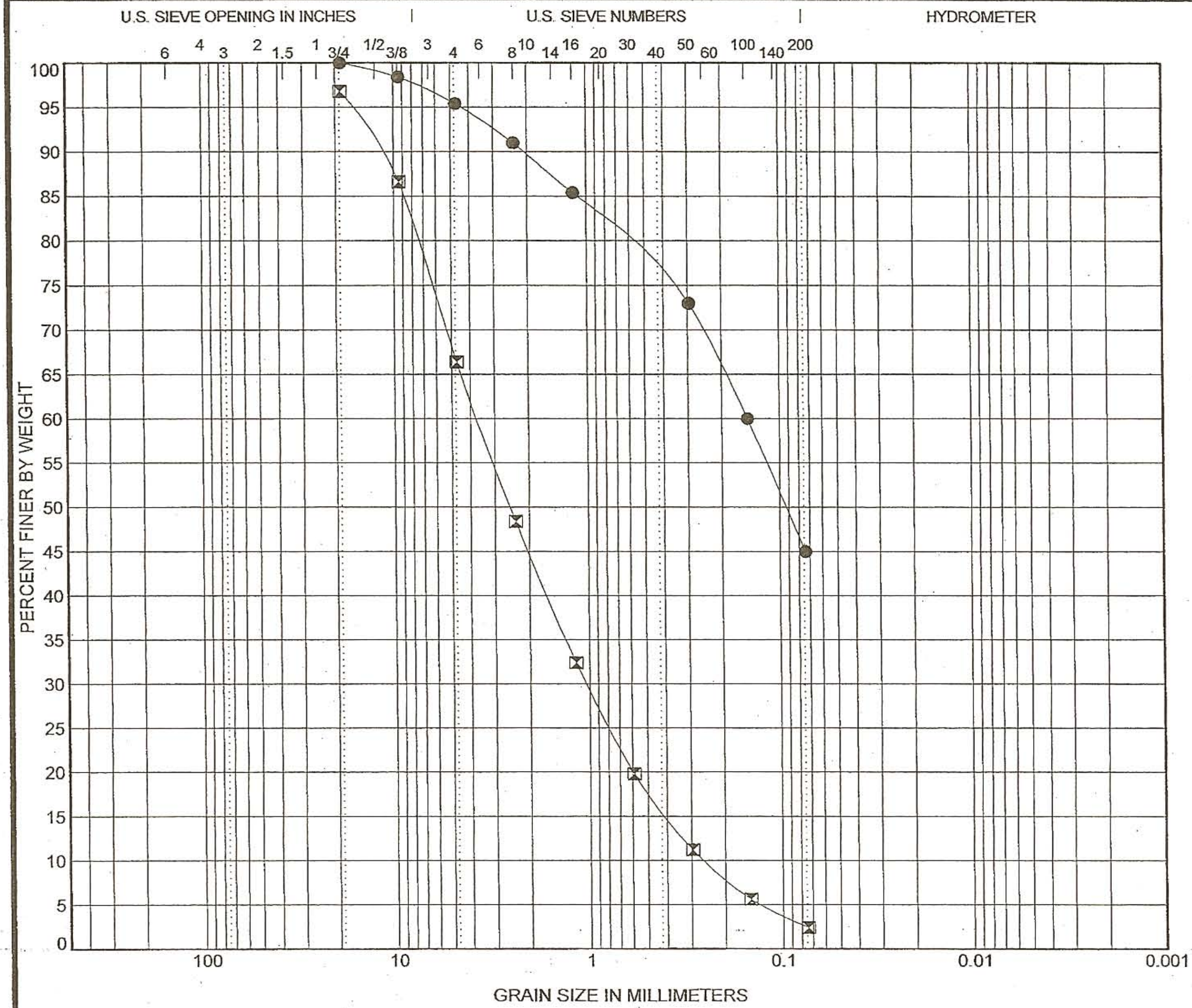
GSI

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

CONSOLIDATION TEST

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

PLATE
D - 4



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

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

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

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

GSI

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

GRAIN SIZE DISTRIBUTION

Project: Hanna Property

Location: Keller and Winchester Rds, Riv. Co.

Number: W.O. 5815-A-OC

PLATE

D - 5

SUMMARY OF LABORATORY TEST DATA

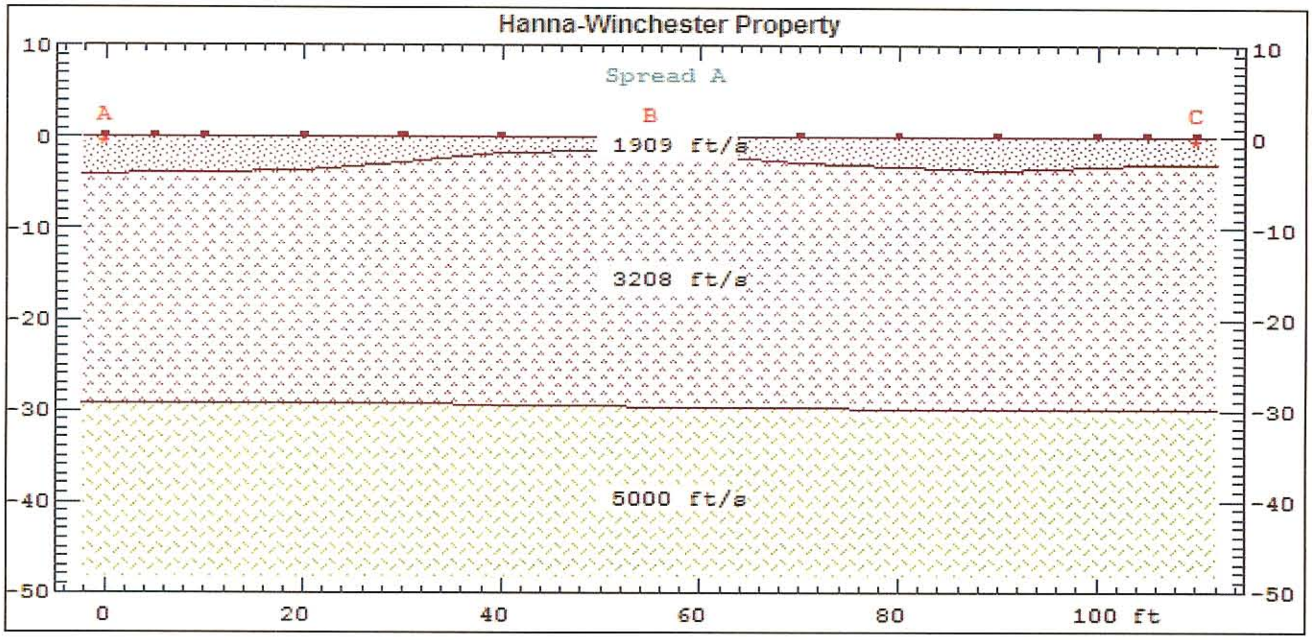
GeoSoils, Inc.
3812 E. La Palma Avenue
Anaheim, CA 92807

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

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

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

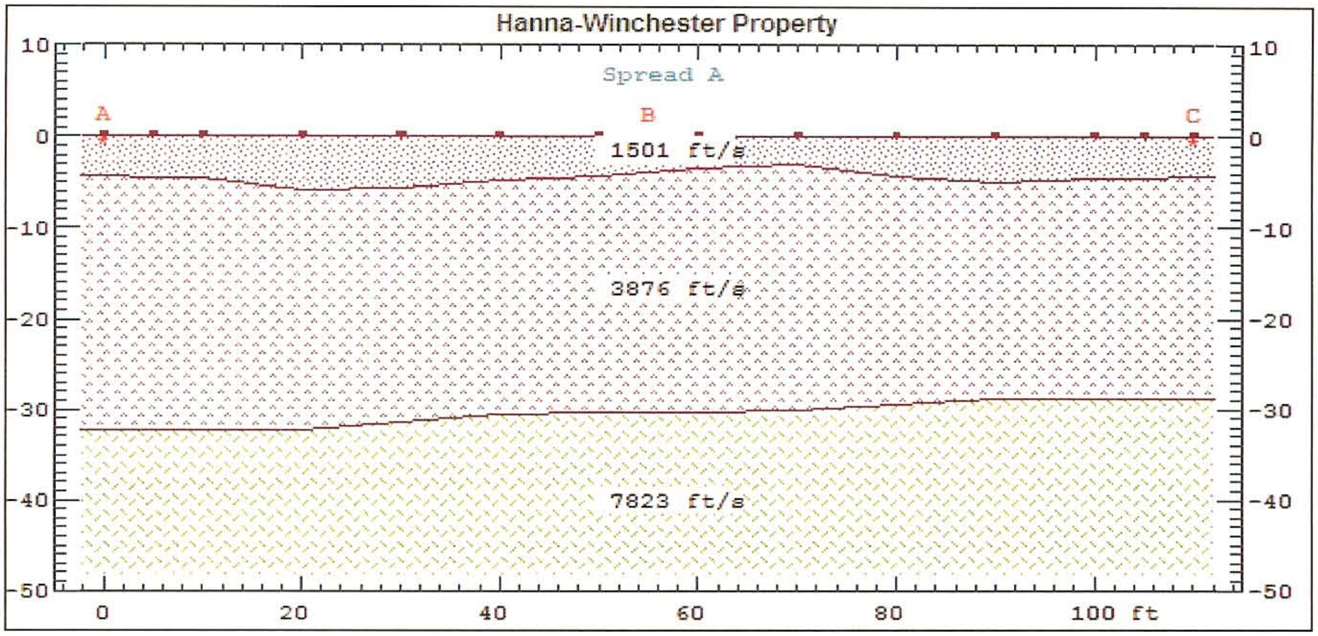
Seismic Line
ST-1



Northwest

Southeast

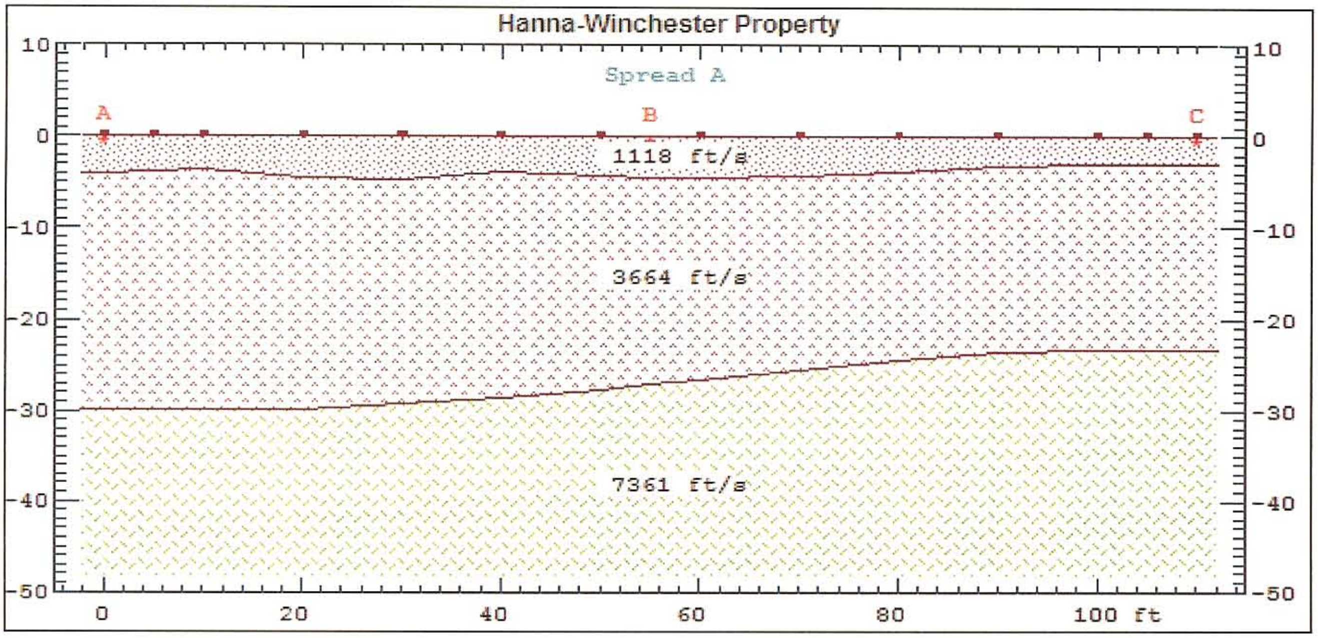
Seismic Line
ST-2



Southeast

Northwest

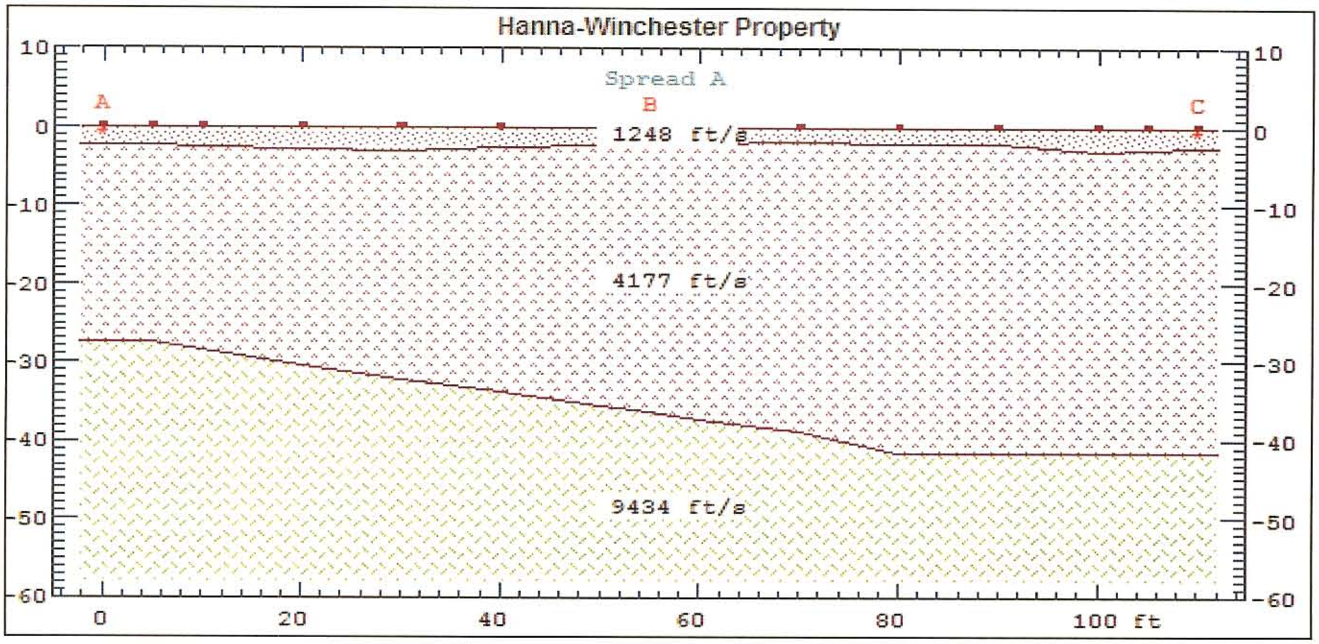
Seismic Line
ST-3



Southeast

Northwest

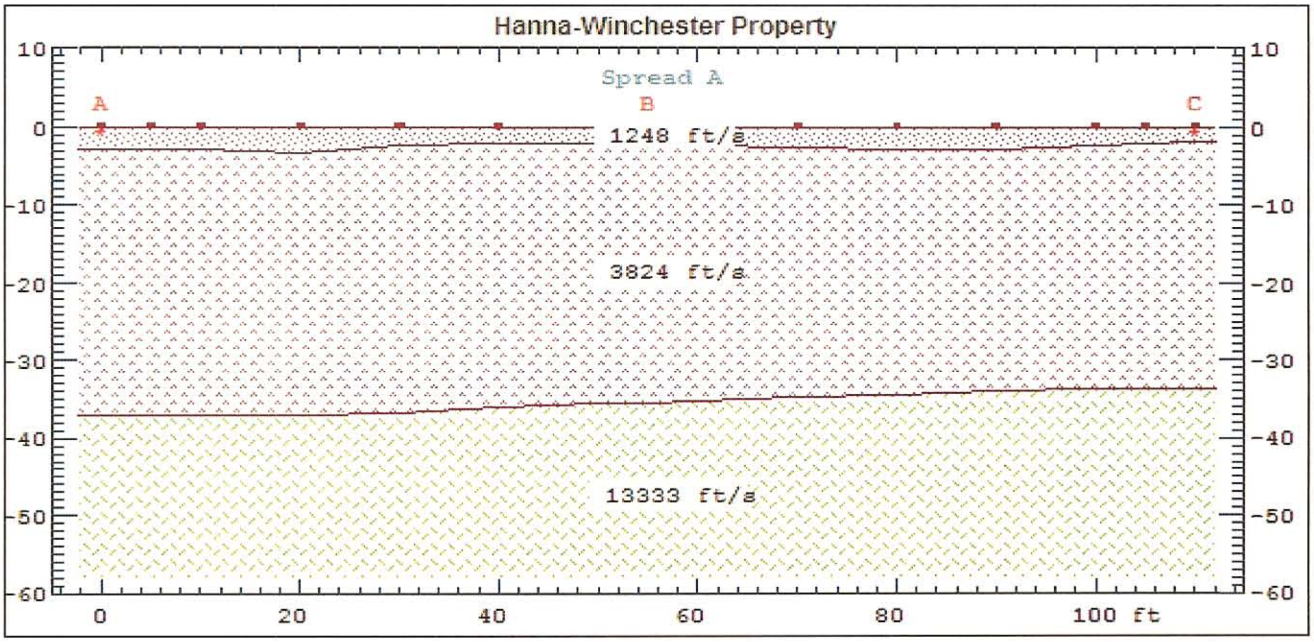
Seismic Line
ST-4



Northwest

Southeast

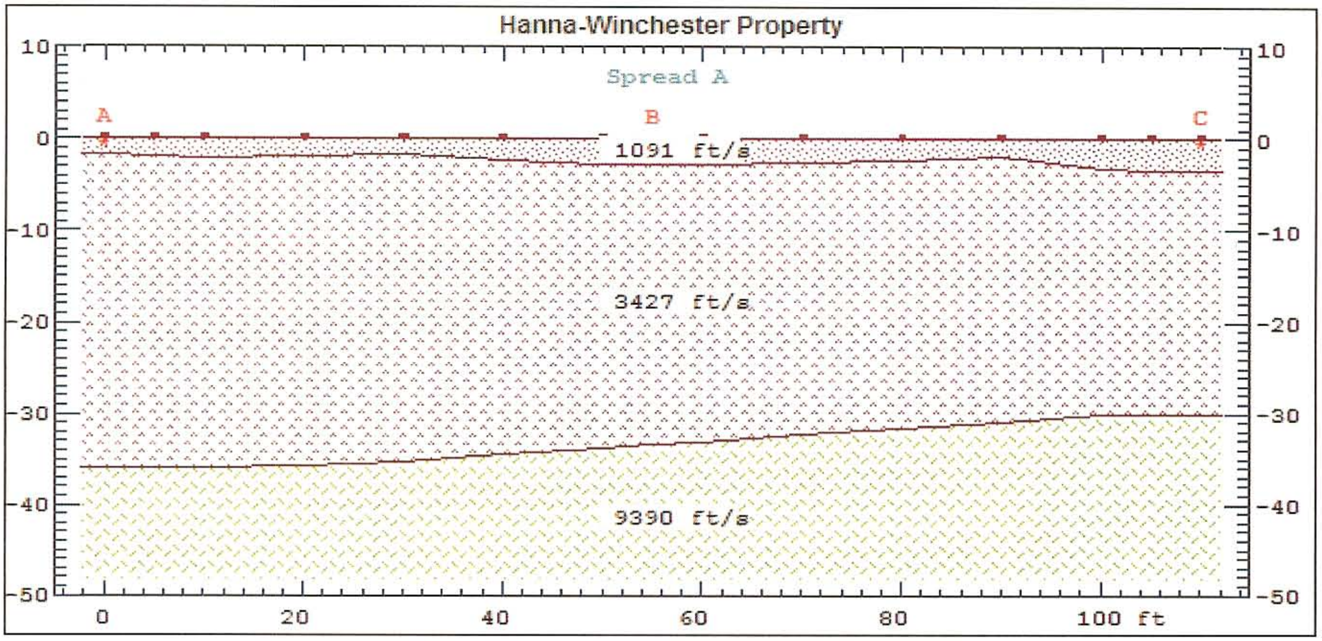
Seismic Line
ST-5



North

South

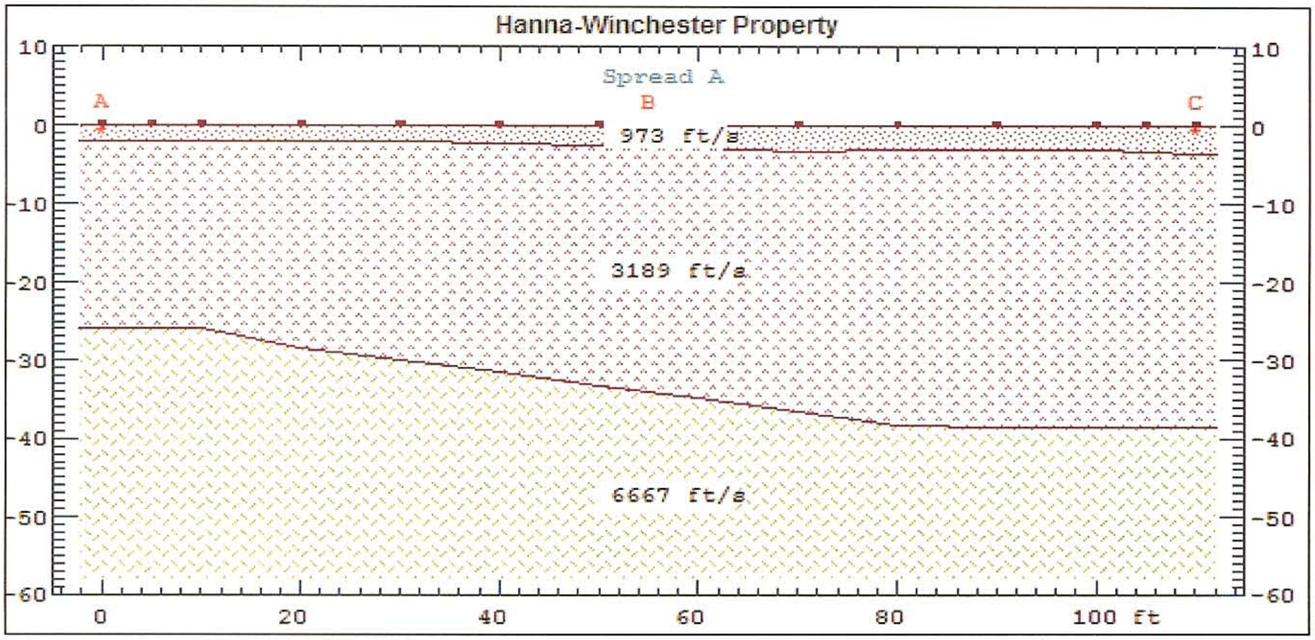
Seismic Line
ST-6



North

South

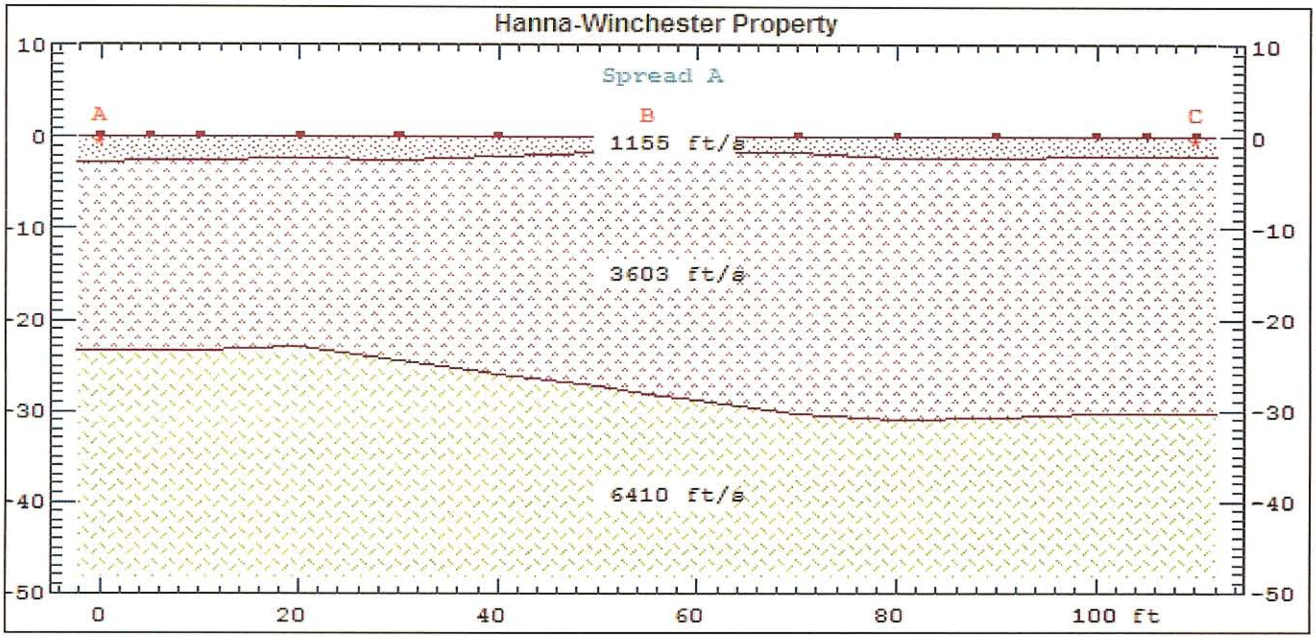
Seismic Line
ST-7



Northwest

Southeast

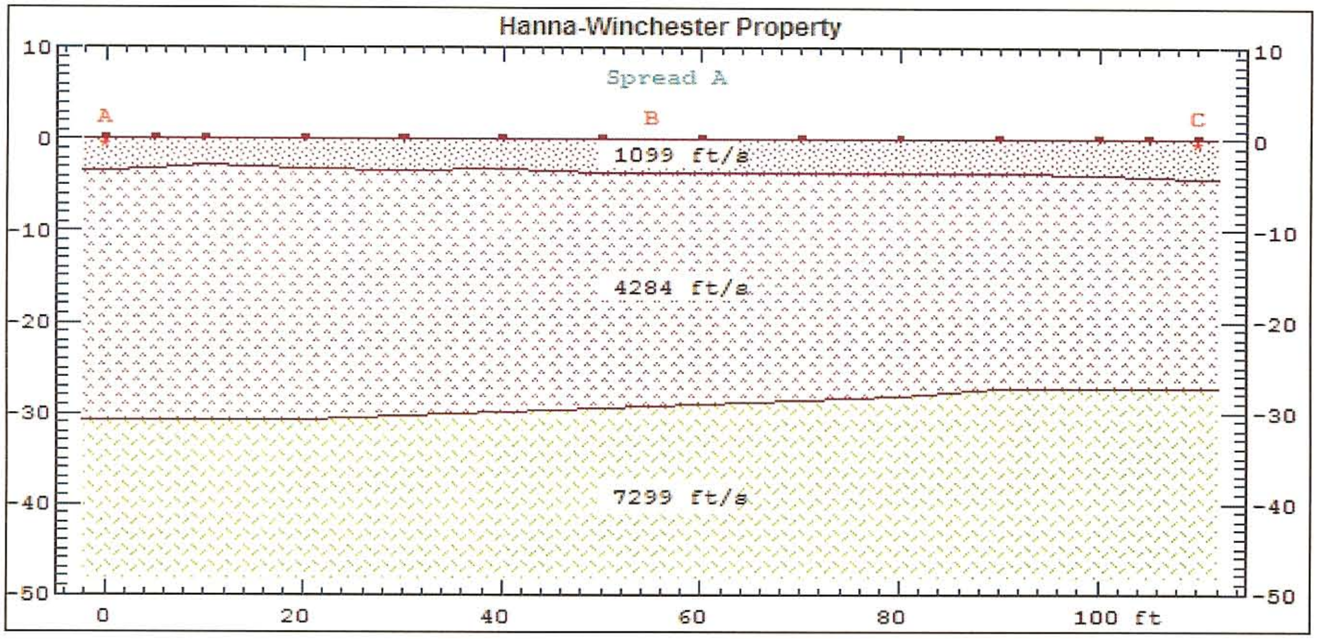
Seismic Line
ST-8



South

North

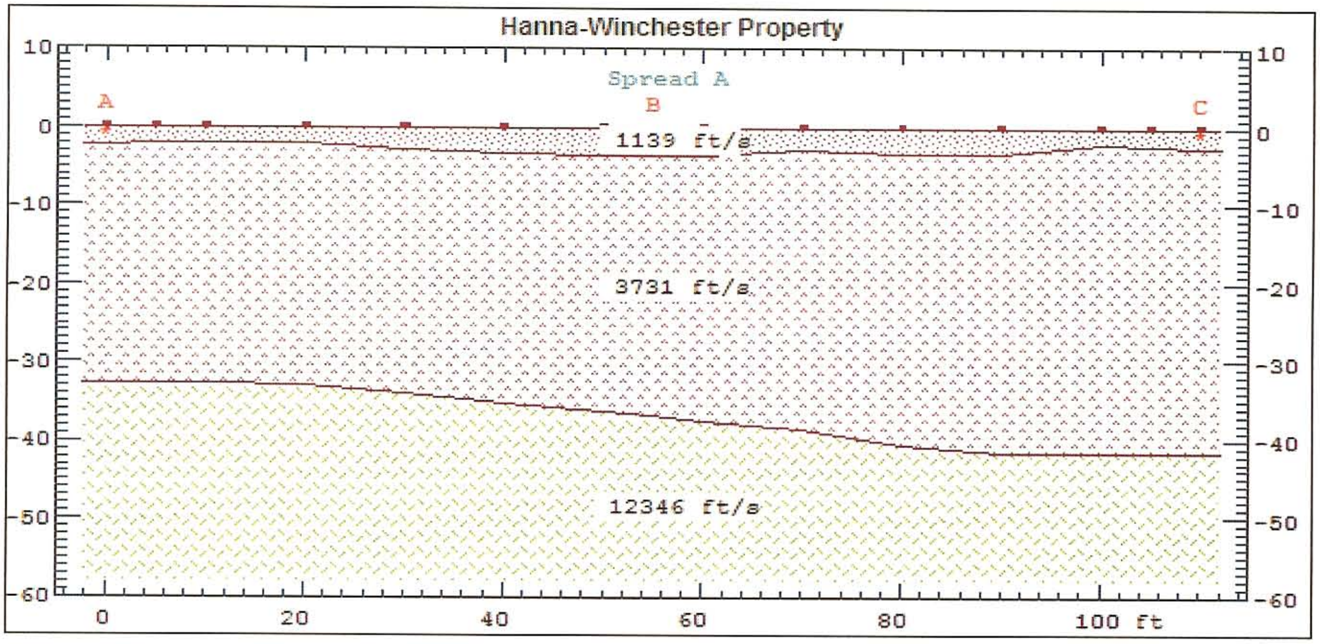
Seismic Line
ST-9



West

East

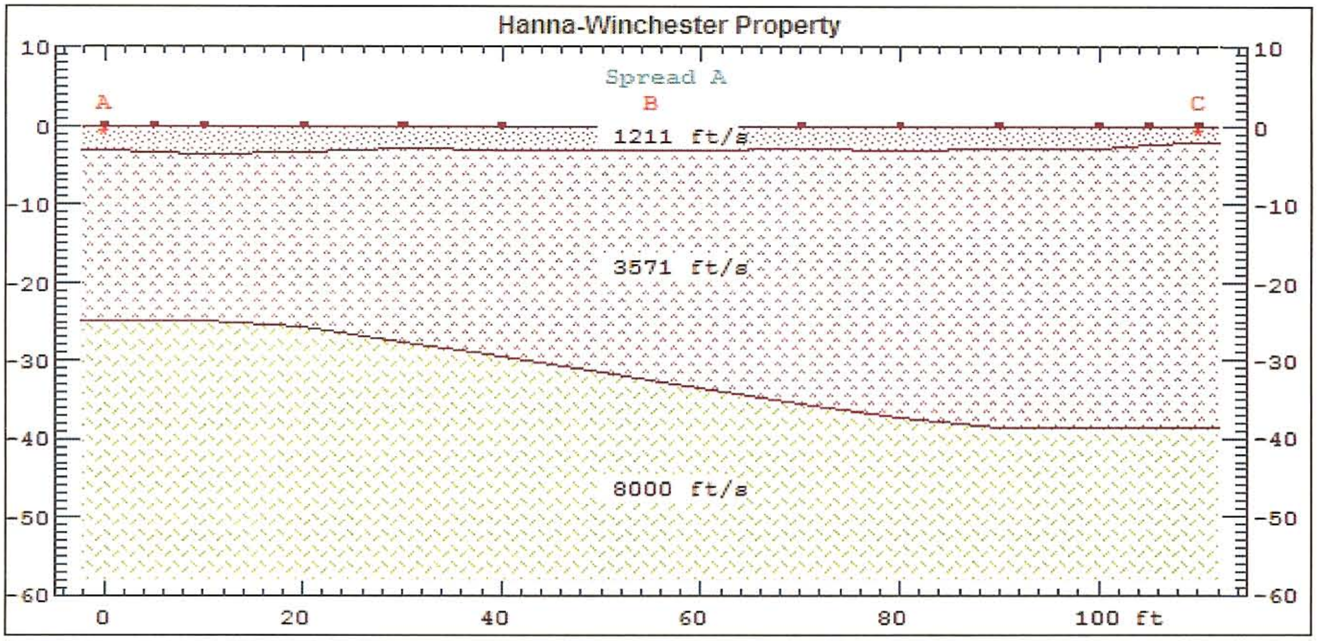
Seismic Line
ST-10



Southeast

Northwest

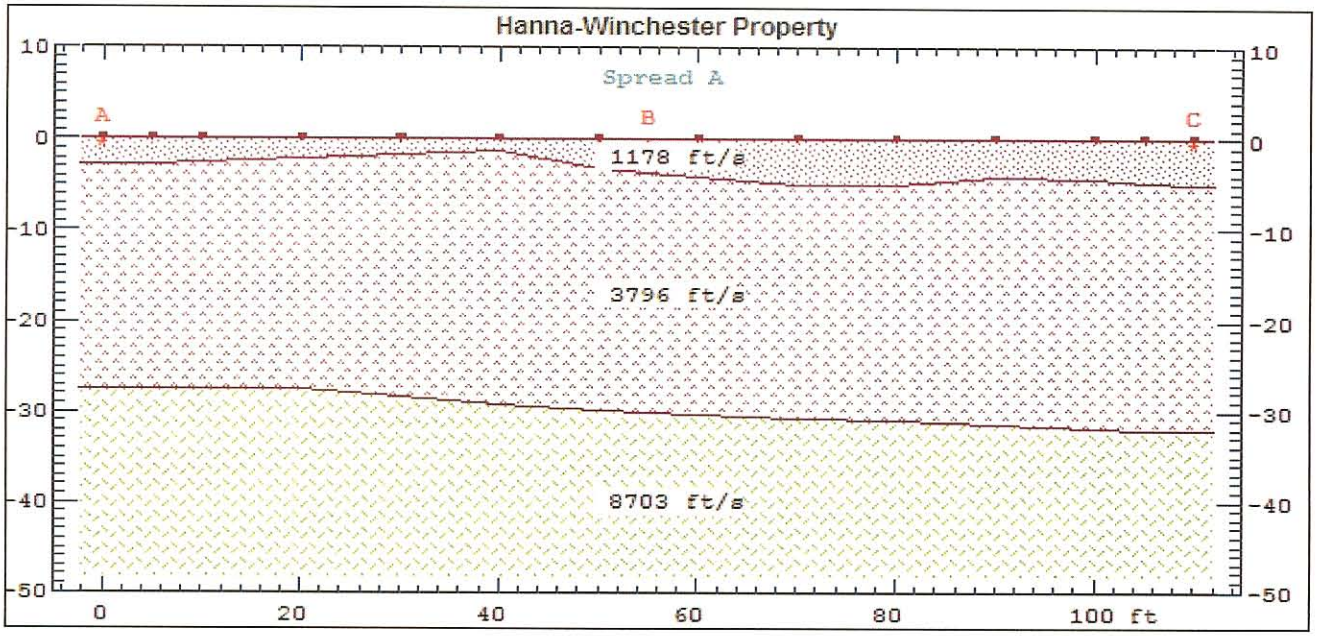
Seismic Line
ST-11



North

South

Seismic Line
ST-12



East

West

APPENDIX B

LOGS OF EXPLORATORY EXCAVATIONS BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
Keller Crossing Project, Winchester Area, Riverside County, California
Project No. 2453-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

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/BORING LOG LEGEND

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

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 the trench/boring logs

(Additional denotations and symbols are provided on the log of trench/boring)

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	Trench No.: T-1 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0			SM	**Older Alluvium:** Silty f-m SAND, light brown-gray, slight moisture, loose, some gravel, many cobble			
		0:46	SM				
		1:31		**Weathered Bedrock:**			
5				Excavates as Silty f-m SAND, light brown-gray, slight moisture, moderately hard, some gravel and cobble sized materials			
		4:40		Same as above			
		7:28		Same as above			
10		9:37		Mostly gravel and cobble sized materials, easy to excavate, full buckets			
		12:10		Same as above			
		15:26		Same as above			
15							
		17:42		Same as above			
		19:36		Same as above			
20		21:01		Same as above, excavator was fully extended			
				TRENCH TERMINATED AT 20 FEET			
				No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Trench No.: T-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0	Large Bulk		SM	**Older Alluvium:** Silty f SAND, light brown, slight moisture, loose to medium dense, some gravel			MD, EI
		0:38					
		2:30		**Weathered Bedrock:** Excavates as silty sand with cobbles, dark gray/brown, slight moisture, moderately hard			
5							
		4:40		Becomes silty f-m sand			
		5:58		Same as above			
10		7:40		increased cobble sized materials			
		11:26		Increased hardness, same as above			
		13:05		Same as above			
15							
		15:14		Same as above			
		16:57					
20		23:07		Excavates as cobbles, somewhat easy to excavate, becomes dark brown/gray Excavator fully extended			
				TRENCH TERMINATED AT 20 FEET			
				No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Trench No.: T-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				Older Alluvium:			
0			SM	Silty f-m SAND, light brown, slightly moist, medium dense			
		0:35					
		3:50		**Weathered Bedrock:**			
				Excavates as Silty f-c SAND, light brown, slightly moist, moderately hard to hard			
5				B-N72E, 58NE			
		7:20		Few boulder sized materials (1-2 ft diamters) observed			
		10:53		Same as above			
10		12:46		Same as above			
		14:21		Increasing hard to excavate, 2-3 scratches for full bucket			
		16:17		Same as above			
15							
		19:56		Becomes very hard to excavate, 3-5 scratches for 1/2 bucket			
		21:12		Same as above			
20		23:05		Same as above, excavator fully extended			
				TRENCH TERMINATED AT 20 FEET			
				No groundwater encountered			
				Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Trench No.: T-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
0			SM	**Older Alluvium:** Silty f-m SAND, light gray-brown, slight moisture, loose, some gravel			
		1:41		**Weathered Bedrock:** excavated as Silty f-m SAND, light gray-brown, slight moisture, loose			
		3:31		Excavates as cobble sized blocky material			
5							
		7:01		Becomes very hard to excavate, 3-5 scrapes for full bucket			
		11:20		Same as above			
10		14:31		Same as above			
		17:52		Same as above			
		23:36		Same as above			
15							
		38 m 12 s		Still very hard to excavate, 3-5 scrapes for 1/2 bucket			
		41 m 14 s		Same as above			
				TRENCH TERMINATED AT 18 FEET			
20				No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Trench No.: T-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0	Large Bulk		SM	**Older Alluvium:** Silty f-m SAND, light brown, slight moisture, loose			MD, EI
		0:57		**Weathered Bedrock:** Excavates as mostly silty SAND with cobbles and few boulder sized materials, light brown-brown, slightly moist, hard			
		2:49		Excavates as mostly cobble and few small boulder sized materials (1-2 ft)			
5							
		8:26		Becomes hard to excavate, 1-3 scratches for full bucket			
		9:57		Same as above			
10		11:52		Same as above			
		15:31		Same as above			
		19:13		Becomes very hard to excavate, 3-5 buckets for 1/2 buckets			
15							
		22:56		Same as above			
		25 m 21 s		Same as above			
20		28 m 14 s		Same as above, excavator fully extended			
				TRENCH TERMINATED AT 20 FEET			
				No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

Trench No.: T-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0			SM	**Older Alluvium:** Silty f-m SAND, brown-gray, slight moisture, medium dense, some gravel			
		0:52		**Weathered Bedrock:** Excavates as gravelly cobble, brown-gray, slight moisture, loose 1-3 scratches for full buckets			
		3:13		Excavate as many cobble and few boulder sized materials (1-2 feet diameter)			
5							
		5:49		Same as above			
		8:26		Same as above			
10		10:16		Same as above			
		12:08		Same as above			
		16:58		Becomes more cobble with some boulders			
15							
		21:37		Becomes very hard, 3-5 scrapes for 1/2 buckets			
		25 m 01 s		5+ scrapes for 1/2 bucket			
20		31 m 45 s		Same as above, excavator arm fully extended			
				TRENCH TERMINATED AT 20 FEET No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY TRENCH

CLIENT:	D.R. Horton	**LOGGED BY:**	GP
PROJECT NAME:	Keller Crossing	**EQUIPMENT:**	Hyundai HX480 108k lbs
PROJECT NO.:	2453-CR	**DATE:**	2/25/2021
LOCATION:	See Trench Location Map		

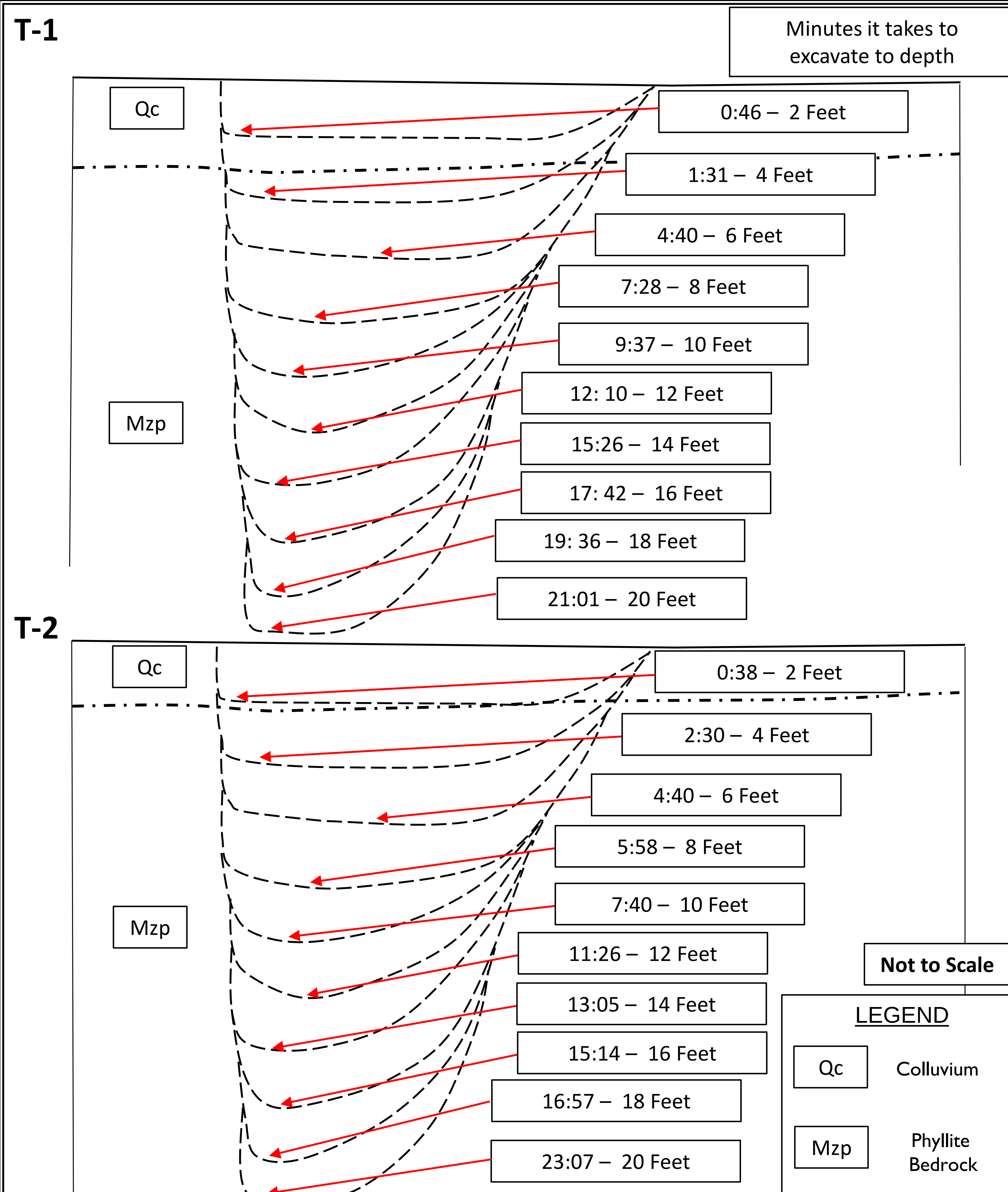
Trench No.: T-7

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0			SM	**Older Alluvium:** Silty f-m SAND dark brown-gray, slight moisture, medium dense some gravel, many cobble			
		0:40		**Weathered Bedrock:** Excavates as Silty f-c SAND, dark brown-gray, slight moisture, moderately hard, easy to excavate, 1-3 scrapes for full bucket			
		4:12					
5							
		7:41		Same as above			
		9:36		hard, few small boulder sized materials observed			
10		13:40		Same as above			
		16:11		Same as above			
		20:45		Same as above			
15							
		22:31		Increasingly hard to excavate, 1-2 scratches for 1/2 to full bucket			
		25: 53 s		Same as above			
20		27 m 19 s		Same as above, excavator fully extended			
				TRENCH TERMINATED AT 20 FEET			
				No groundwater encountered Trench backfilled with excavated soils			
25							
30							

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

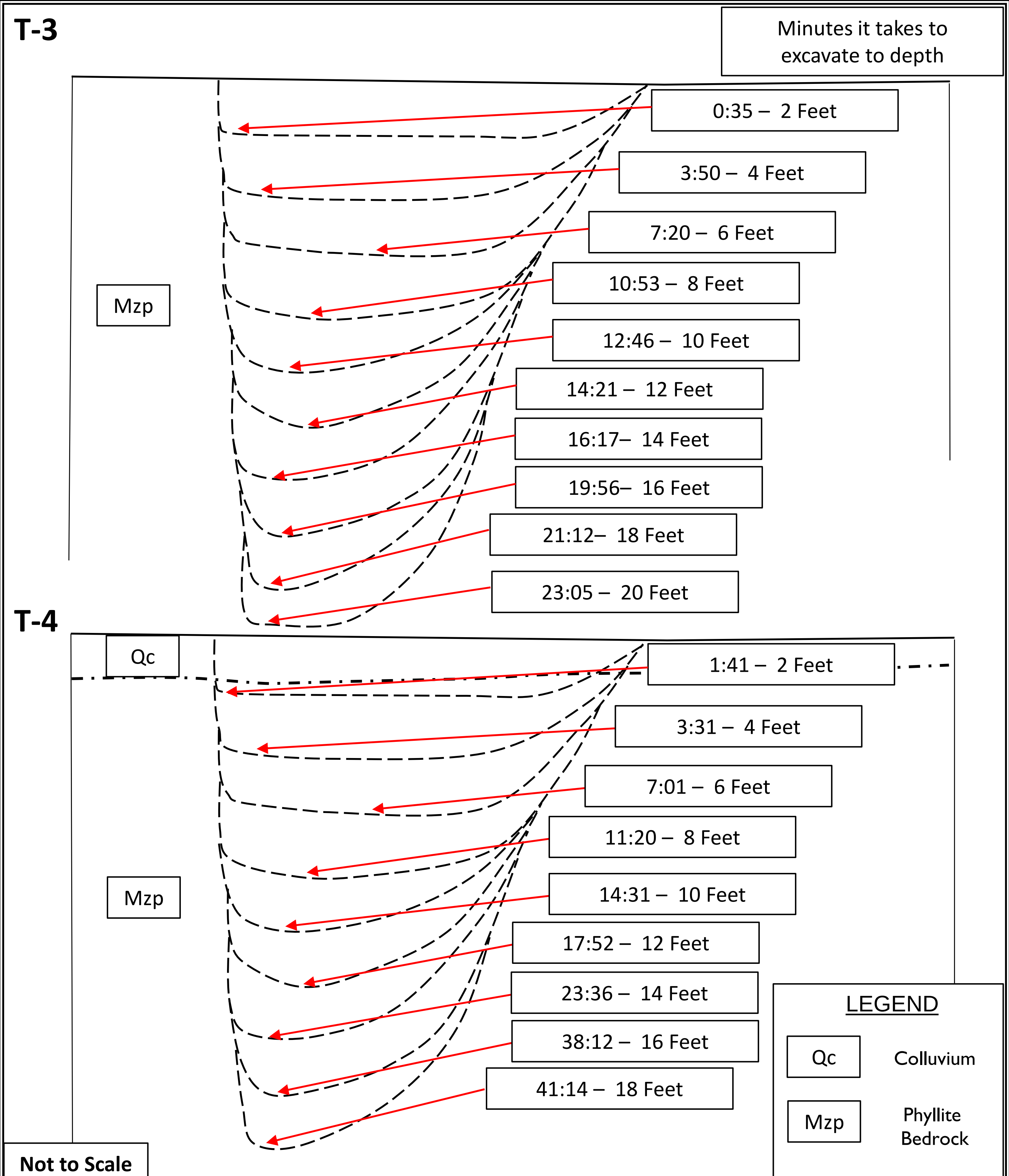


DR Horton Los Angeles Holding Company
Keller Crossing
Winchester, Riverside County, California

GeoTek Project No. 2453-CR

Graphic Trench Logs

GEOTEK

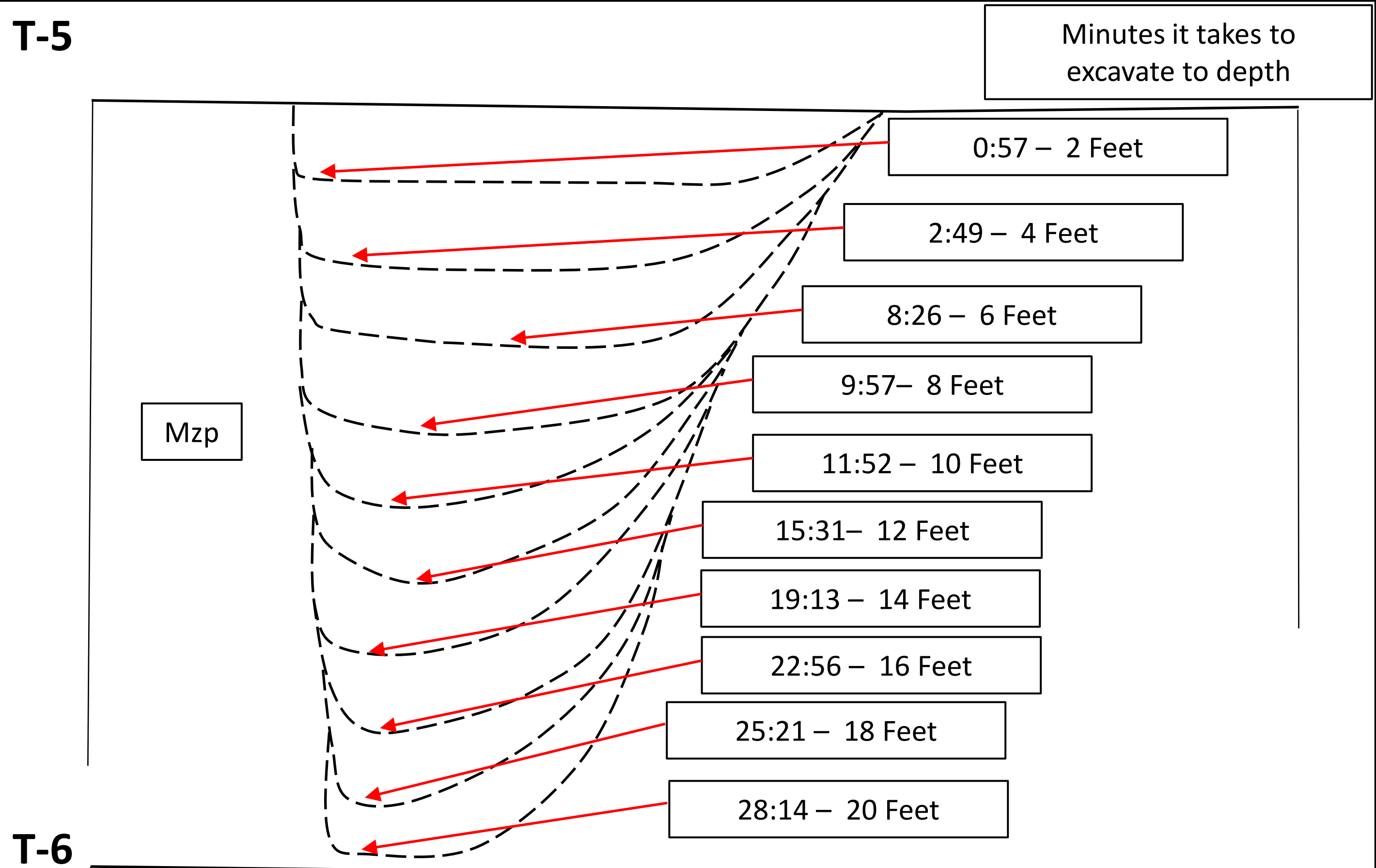


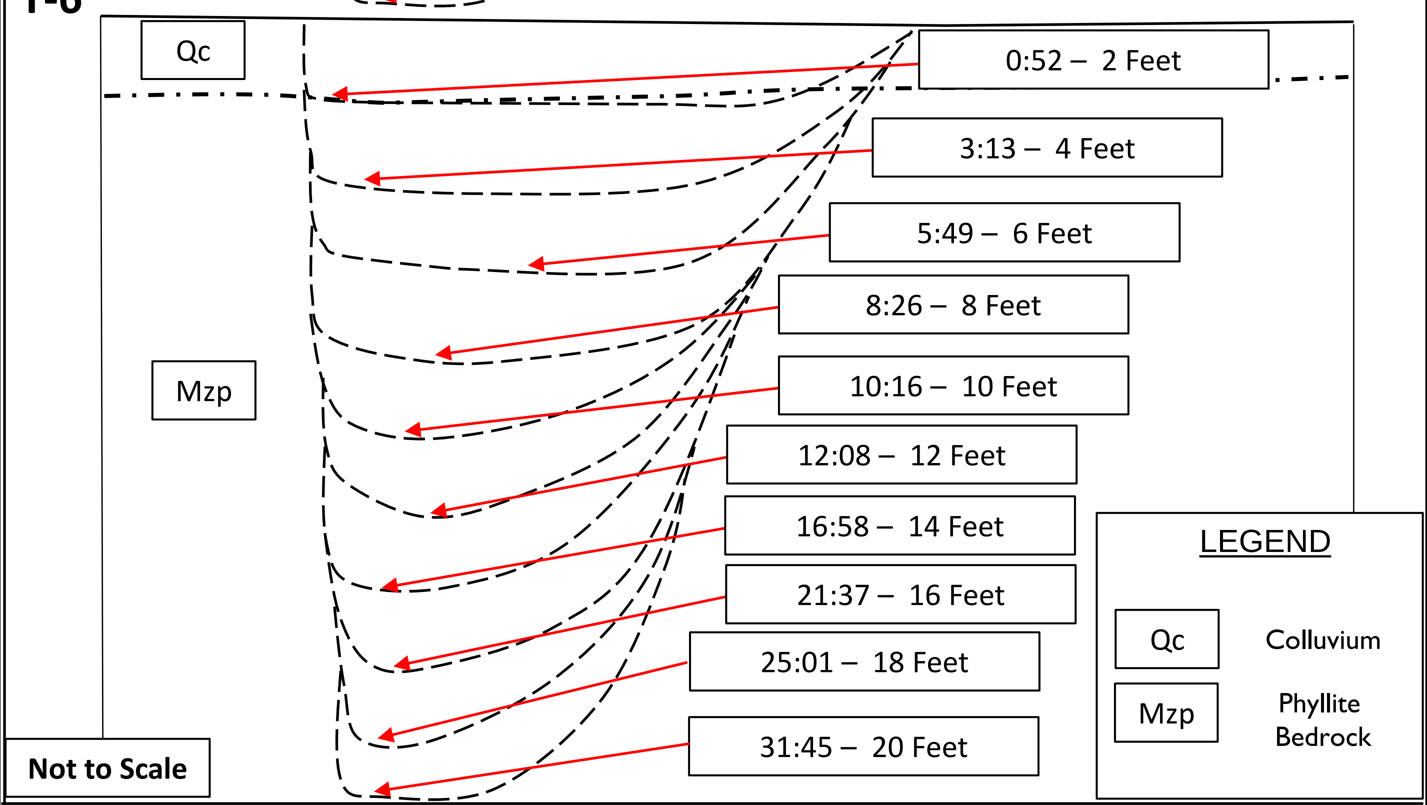
DR Horton Los Angeles Holding Company
Keller Crossing
Winchester, Riverside County, California

GeoTek Project No. 2453-CR

Graphic Trench Logs

GEOTEK





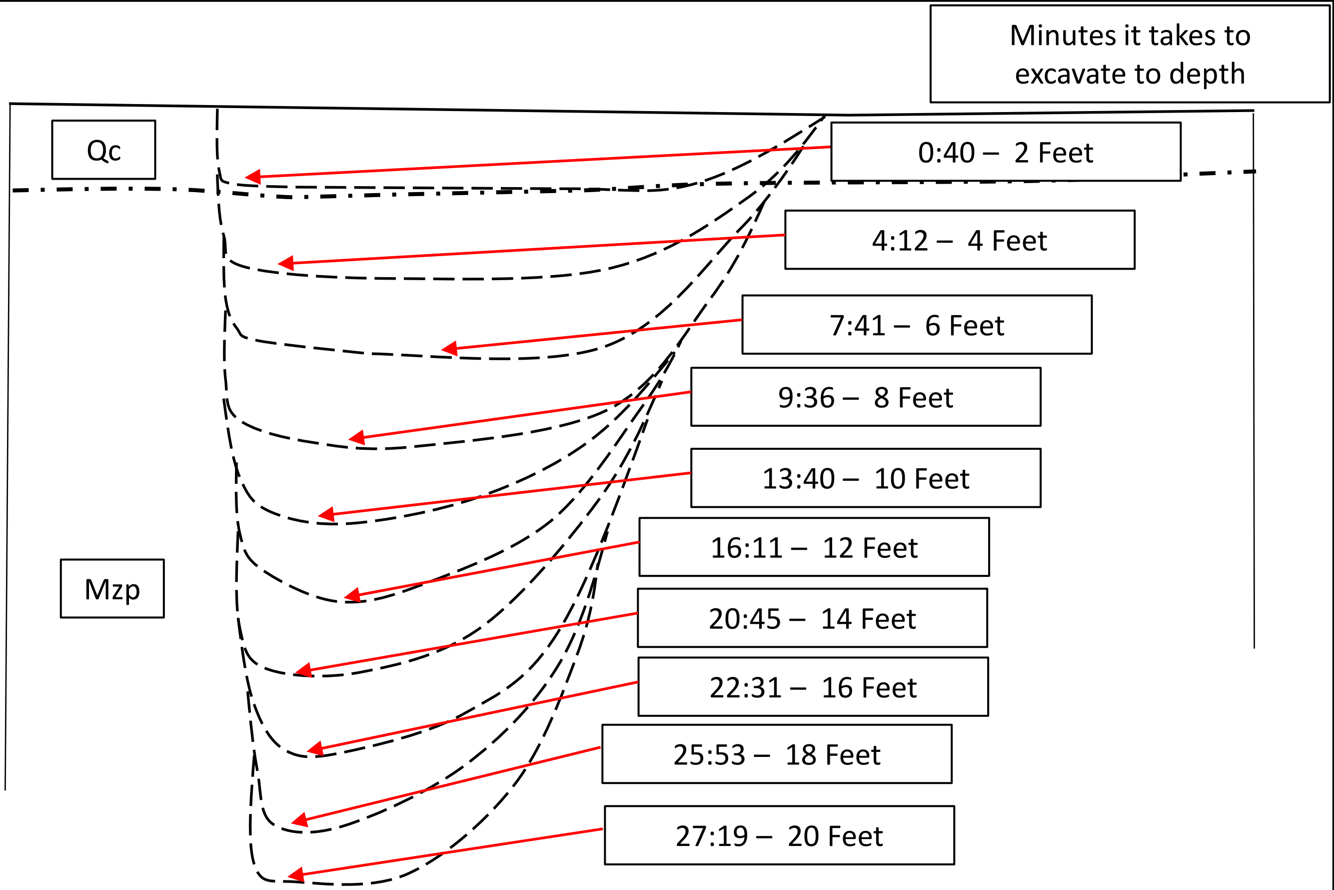
DR Horton Los Angeles Holding Company
Keller Crossing
Winchester, Riverside County, California

GeoTek Project No. 2453-CR

Graphic Trench Logs

GEOTEK

T-7



LEGEND

Qc	Colluvium
Mzp	Phyllite Bedrock

Not to Scale

DR Horton Los Angeles Holding Company
Keller Crossing
Winchester, Riverside County, California

GeoTek Project No. 2453-CR

Graphic Trench Logs

GEOTEK

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Boring No.: B-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f-m SAND, light gray to brown, moist, dense, trace f gravel			
5	SPT	19 50/6			becomes very dense			
					Phyllite Bedrock: Excavates as silty m/c sand with gravel, light gray- brown, slight moisture, very hard,			
10	SPT	50/2			Same, no recovery			MD, EI, SR
15	SPT	50/3			Same, no recovery			
					Boring terminated @ 16.5 ft. No groundwater Spoils backfilled			
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test
SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: D.R. Horton	**DRILLER:** 2R Drilling, Inc.	**LOGGED BY:** G. Pocius	
PROJECT NAME: Keller Crossing, Winchester	**DRILL METHOD:** Hollow-Stem Auger	**OPERATOR:** Juan/Reece	
PROJECT NO.: 2453-CR	**HAMMER:** 140 lbs/30 inches	**RIG TYPE:** CME-75	
LOCATION: See Figure 2a		**DATE:** 2/1/2021	

Boring No.: B-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0–5	Large Bulk			SM	**Older Alluvium:** Silty f SAND, light gray to brown, moist, slightly dense, many gravel sized particles.			MD, EI
5	Ring	50/4			**Phyllite Bedrock:** Excavates as silty f sand with gravel, light gray- brown, moist, hard			
10	Ring	44 50/2			Same as above			
15	Ring	17 27 41			Excavates as clayey silt, light olive green, moist, dense			
16.5					**Boring terminated @ 16.5 ft.** No groundwater Spoils backfilled			
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test
SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
		50/6	R1	SM	Silty f SAND, brown, slightly moist, very dense, trace pinhole pores, rootlets	9.7	106.6	MD SH
5		30 50/5	R2		**Metasedimentary Bedrock:** METASILTSTONE, dark gray, moist, hard, highly weathered to decomposed, oxidized			
10		41 50/2	R3		Same, moderately weathered	8.7	117.6	
15		50/6	R4		Same, slightly moist, laminated bedding			SH
20		50/4	R5		Same	10.7	139.9	SR
25		50/6	R6		F Sandy METASILTSTONE, medium to light gray, slightly moist to moist, hard, little weathering			SH
30		50/4	R7		Same, slightly moist, partially disturbed sample	8.7	89.5	
					BORING TERMINATED AT 30.4 FEET No groundwater encountered Boring backfilled with soil cuttings			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | HC= Consolidation | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
	Ring	26 18 28	R1	SC	Clayey f SAND, dark brown, slightly moist, dense, metasedimentary fragments (up to 3", 2-5% typical)	11.7	109.5	
5					More difficult excavation noted by driller			
					Metasedimentary Bedrock:			
	Ring	42 50/4	R2		METASILTSTONE, dark gray, slightly moist, hard, highly weathered			
10								
	Ring	18 38 50/4	R3		Clayey METASILTSTONE, light gray, yellowing brown, moist, hard, sulfurous, highly weathered	18.5	114.8	
15								
	Ring	29 29 50/4	R4		Same, laminated bedding	23.5	97.1	
20								
	Ring	27 50/3	R5		METASILTSTONE, yellowish gray, moist, hard, highly weathered	26.3	102.0	
25					Drilling slowed			
	No Recovery	50/6			No recovery Becoming very hard Drilling slowed			
30	No Recovery	50/3			No recovery			
	No Recovery	50/2			No recovery			
					BORING TERMINATED AT 31.2 FEET No groundwater encountered, boring backfilled with soil cuttings			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium:			
	Large Bulk / Ring	7 19 25	R1	SM	Silty f SAND, brown, moist, dense, roothairs	12.5	108.9	
5	Ring	50/6	R2	SM-ML	Silty f SAND/Sandy SILT, reddish brown, slightly moist, very dense, abundant caliche, oxidation	20.3	94.2	SH
					Metasedimentary Bedrock?			
					METASILTSTONE?, reddish brown, moist, hard, logged from soil cuttings			
10	No Recovery	50/2			No recovery			
					Drilling slowed			
	No Recovery	50/1			No recovery			
15	No Recovery	50/1			No recovery			
					Drilling slowed, strong steaming			
	No Recovery	50/1			No recovery			
					BORING TERMINATED DUE TO REFUSAL AT 17.1 FEET			
					No groundwater encountered			
					Boring backfilled with soil cuttings			
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium:			
		7 14 19	R1	SM	Silty f SAND, dark reddish brown, slightly moist, medium dense, caliche stringers, roothairs			
5		24 50/6	R2		Same, dry to slightly moist, very dense	12.5	110.3	
					Metasedimentary Bedrock			
		23 50/5	R3		METASILTSTONE, reddish brown, olive gray, moist, hard, decomposed, highly oxidized, sulfurous			
10		50/6	R4		Same, highly weathered to decomposed	9.9	99.8	
					BORING TERMINATED AT 10.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-7

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Artificial Fill			
	Ring	4 5 4	R1	SM-ML	Silty f SAND/Sandy SILT, brown, olive brown, moist, loose			
					Granitic bedrock:			
	Ring	50/6	R2		GRANITICS(GABBRO), orangish brown, slightly moist to moist, hard, oxidized, highly weathered to decomposed	7.3	95.6	SR
5								
	Ring	32 50/6	R3		Same, breaks down to f-c SAND			
10	Ring	25 50/6	R4		Same, highly weathered	5.7	118.9	
					BORING TERMINATED AT 11.0 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling Inc.	**LOGGED BY:**	KM
PROJECT NAME:	Keller Crossings	**DRILL METHOD:**	Hollow stem Auger	**OPERATOR:**	Cody
PROJECT NO.:	2453-CR	**HAMMER:**	140lbs/30in.	**RIG TYPE:**	CME-75
LOCATION:	See Boring Location Map			**DATE:**	3/29/2021

BORING NO.: B-8

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium:			
	Ring	50/4	R1	SM	Silty f SAND, orangish brown, dry to slightly moist, hard, rootlets, trace pinohle pores	10.8	100.4	
5	Large Bulk				**Metasedimentary Bedrock:**			
	Ring	50/6	R2		METASILTSTONE, dark gray, slightly moist, hard, laminated bedding, highly weathered	6.6	105.3	
10								
	Ring	50/5	R3		Same	6.1	-	
					Drilling slowed			
15								
	Ring	50/4	R4		Sandy METASILTSTONE, yellowish gray, moist, hard			
20								
	No Recovery	50/2			No recovery Drilling slowed, some steaming			
	No Recovery	50/0			No recovery			
					BORING TERMINATED DUE TO REFUSAL AT 24.0 FEET			
25					No groundwater encountered Boring backfilled with soil cuttings			
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	Boring No.: I-1 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f SAND, light brown to gray, slightly moist to moist, medium dense			
5					**Boring terminated @ 5 ft**			
					No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | HC= Consolidation | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	Boring No.: I-2 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f-m SAND, light brown to gray, slightly moist to moist, medium dense			
5					**Boring terminated @ 5 ft** No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Depth (ft)	SAMPLES			USCS Symbol	**Boring No.: I-3**	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number		**MATERIAL DESCRIPTION AND COMMENTS**	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium:			
				SM	Silty f-m SAND, light brown to gray, slightly moist to moist, medium dense some f gravel			
5					**Boring terminated @ 5 ft**			
					No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test
SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Boring No.: I-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f-m SAND, light brown to gray, slightly moist to moist, medium dense some f gravel			
5					**Boring terminated @ 5 ft** No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Boring No.: I-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f-m SAND, light brown to gray, slightly moist to moist, medium dense some f gravel			
5					**Boring terminated @ 5 ft**			
					No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	D.R. Horton	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	G. Pocius
PROJECT NAME:	Keller Crossing, Winchester	**DRILL METHOD:**	Hollow-Stem Auger	**OPERATOR:**	Juan/Reece
PROJECT NO.:	2453-CR	**HAMMER:**	140 lbs/30 inches	**RIG TYPE:**	CME-75
LOCATION:	See Figure 2a			**DATE:**	2/1/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	**Boring No.: I-6** MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
				SM	Silty f-m SAND, light brown to gray, slightly moist to moist, medium dense some f gravel			
5					**Boring terminated @ 5 ft**			
					No groundwater			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test
SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

APPENDIX C

SEISMIC REFRACTION SURVEY RESULTS BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
Keller Crossing Project, Winchester Area, Riverside County, California
Project No. 2453-CR

GEOTEK

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

GeoTek. Inc.
1548 North Maple Street
Corona, CA 92880

February 27, 2021

Attn: Gabriela Pocius

Re: Seismic Survey Summary Report
Project 2453-CR, Winchester, CA

This report covers the results of a seismic refraction survey performed at 33972 Winchester Road in Winchester, 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, grading, and other earthwork.

The field work was conducted on February 19, 2021. Seven 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 the San Bernardino and Santa Ana 30' x 60' quadrangles, California ", (USGS Open File Report 2006-1217, 2006) indicates the survey area is underlain by fissile dark brown to black phyllite of Triassic age (Trmp). Surface deposits are colluvium on the hillsides and alluvium in the low lying areas.

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 which provided a line length of 240 feet.. Each spread used five shotpoints, one off each end (5-foot offset) and three within the interior of the spread. Depth of investigation was approximately 45 to 60 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 field records 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 assigned a elevation value of 0.0 feet. This datum point served as the reference elevation for all other measurements.

Labeled wooden stakes were placed at the beginning and end of each spread and a Garmin handheld GPS receiver was used to record the latitude and longitude coordinates of the stakes. The coordinates were used to make the location map shown 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.

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 7 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

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:

$$Vt = 2(Vu \times Vd)/(Vu + Vd)$$

where Vt = true velocity
Vu = apparent up dip velocity
Vd = 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.

Significant lateral changes in velocity were observed below Line 3, especially along the top of the unweathered bedrock interface. This made the layer modeling approach unusable. A colored tomographic inversion model was prepared to show the velocity distribution beneath this line.

The modeling program used 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

Results from refraction analysis show a three layer solution beneath all lines, except Line 3 (see Figures 5-11). 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 with rock fragments but may also include highly weathered bedrock. Thickness is generally less than 5 feet.

Layer 2 - is interpreted to be weathered bedrock. The velocity range is 3740-4613 ft/sec which is considered rippable with a D-9 Cat.

Layer 3 - represents hard unweathered bedrock.

Table 1. Cross Section Summary Velocity in (ft/sec), Depth in (feet)

Line	Velocity Layer 1	Velocity Layer 2	Velocity Layer 3	Depth Range Unweathered Interface
1	1327	4167	7516	46 - 54
2	1699	3779	7706	44 - 60
3	see tomographic model			
4	1520	4613	7140	18 - 31
5	1854	4404	9756	30 - 48
6	1433	3862	10885	29 - 48
7	1385	3740	9987	47 - 63

Line 3 showed evidence of a localized high velocity zone (7000+ ft/sec) beneath the center of the spread but was not laterally continuous beneath the entire traverse. This precluded the preparation of a layered velocity model. As an alternative, a colored tomographic cross section made (see Figure 9).

Weathering tends to be gradational for most rock types and usually produces a gradual increase in velocity with depth. In this metamorphic complex, the seismic records showed unusually high velocity gradients within layers 2 and 3. Consequently, variation of $\pm$ 25% from the posted averages may occur between the top and bottom of these two layers. For example, the average velocity for layer 2 on Line 5 is 4404 ft/sec. However, the overall range, top to bottom, could vary from 3300 to 5500 ft/sec.

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. 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 changes in metamorphism of the bedrock and the presence of fractures and joints.

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.

PAWalen

Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Seismic Survey Location Map

Keller Crossing Project Site -- Winchester, California

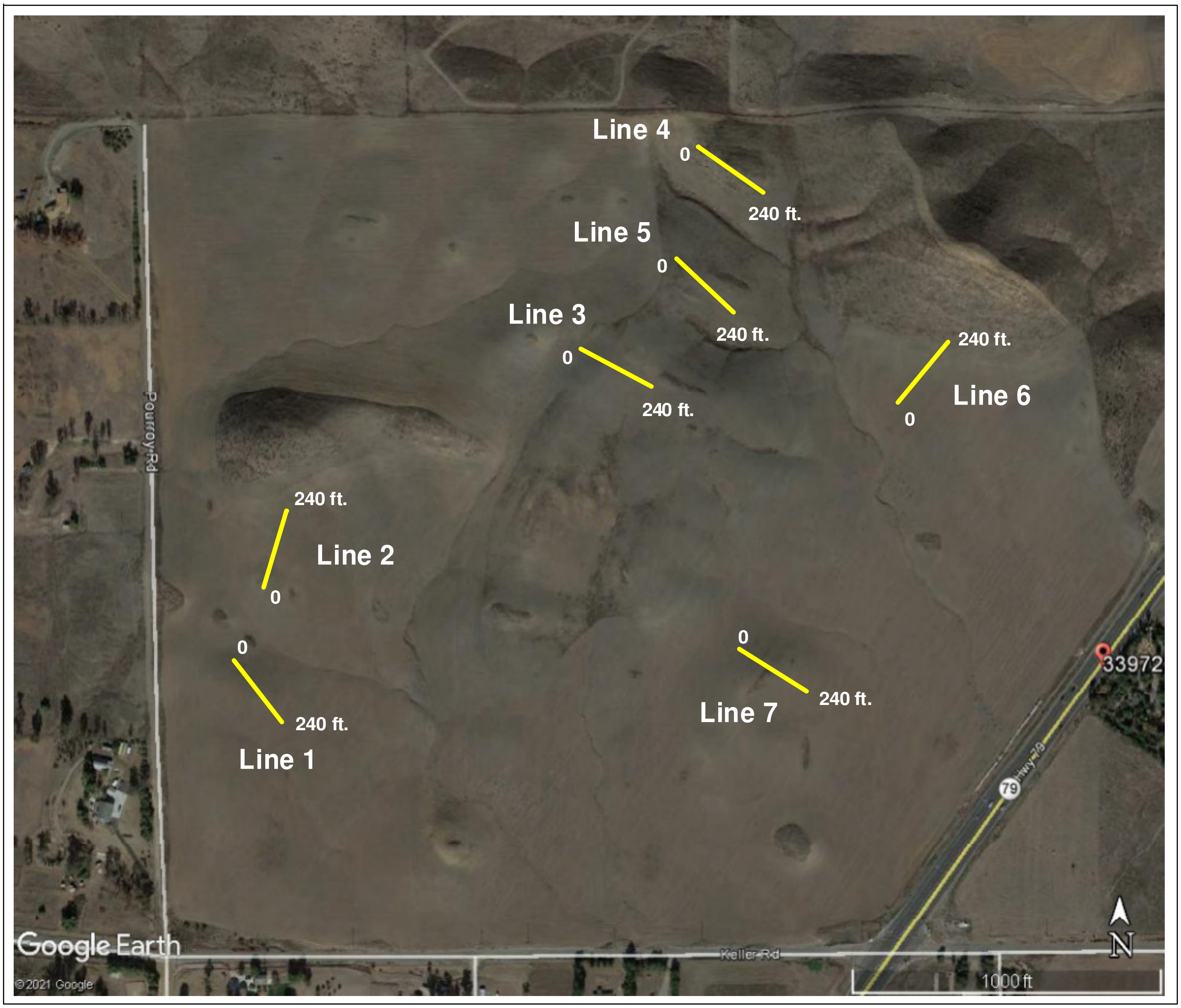


Figure 1

Example Seismic Field Records

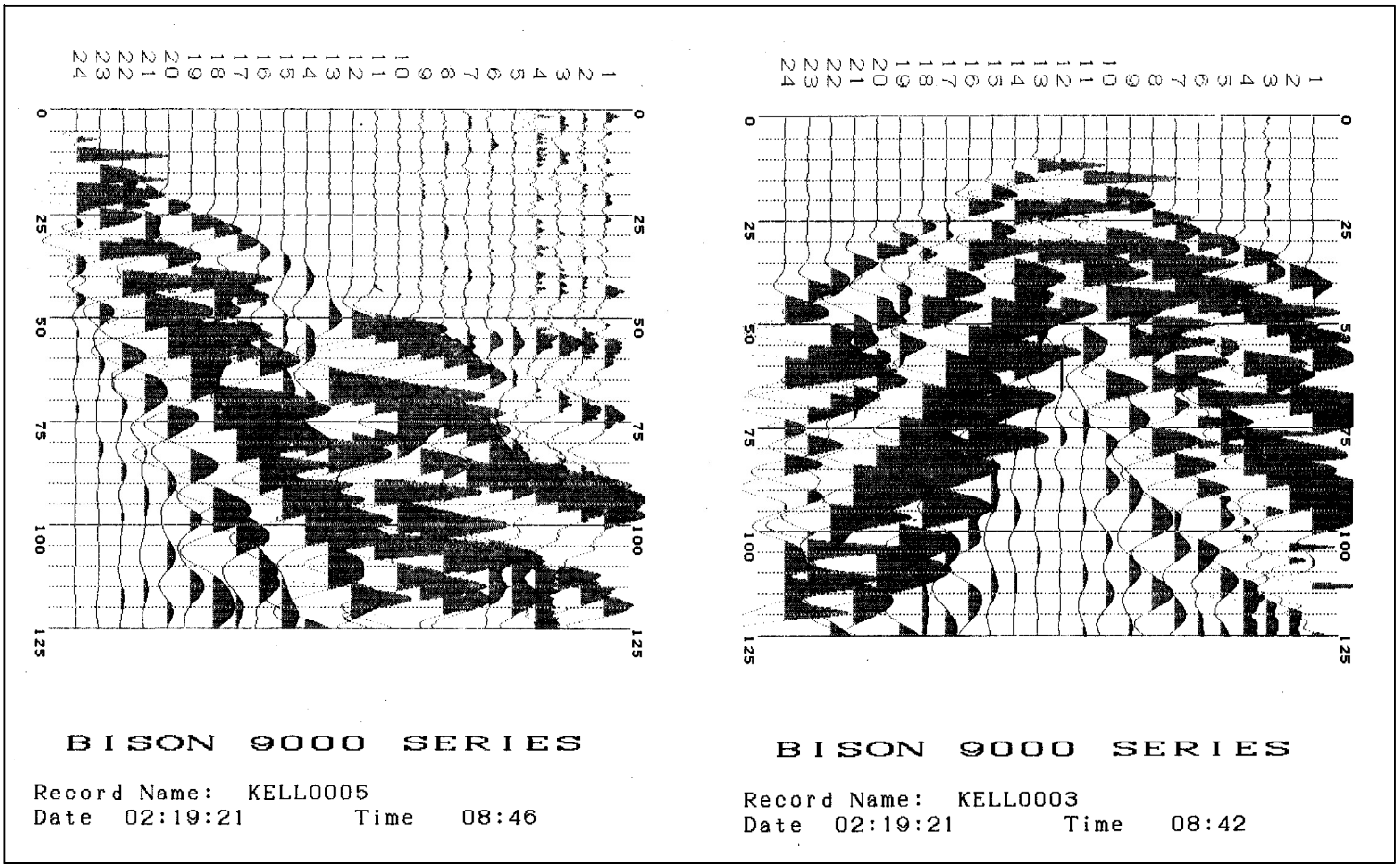
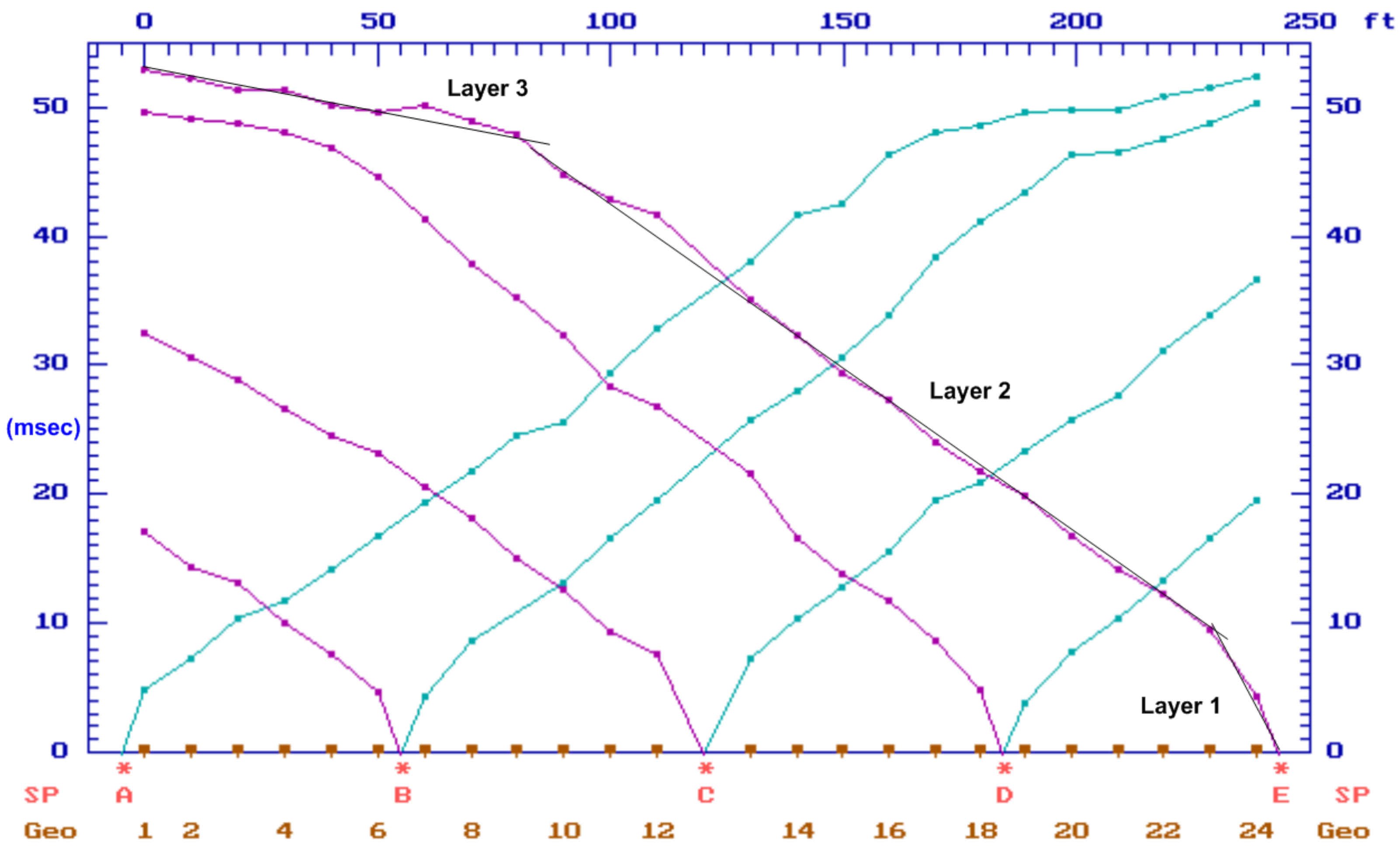


Figure 2

Example Time-distance Graph -- Line 7



Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California

Figure 3

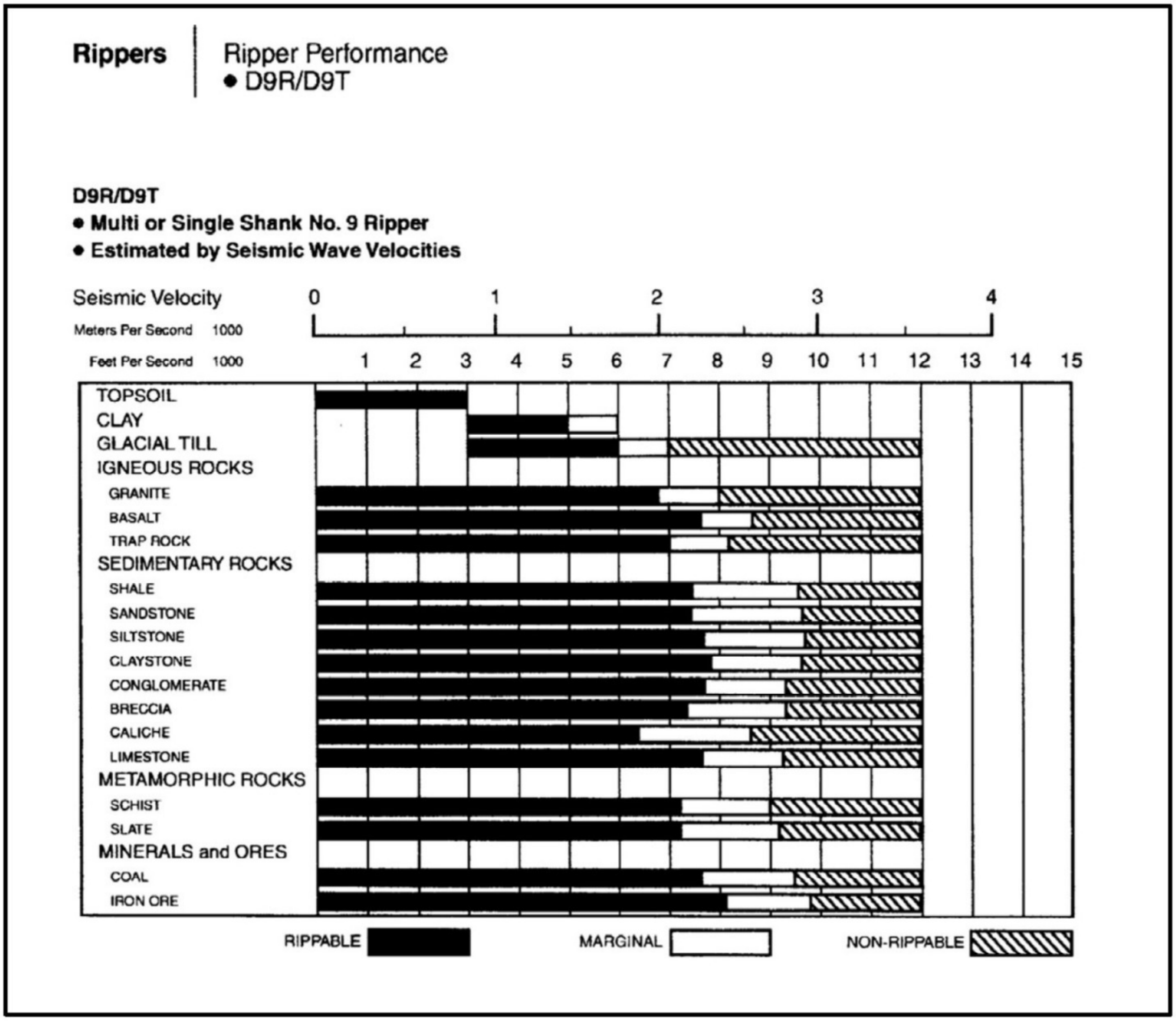
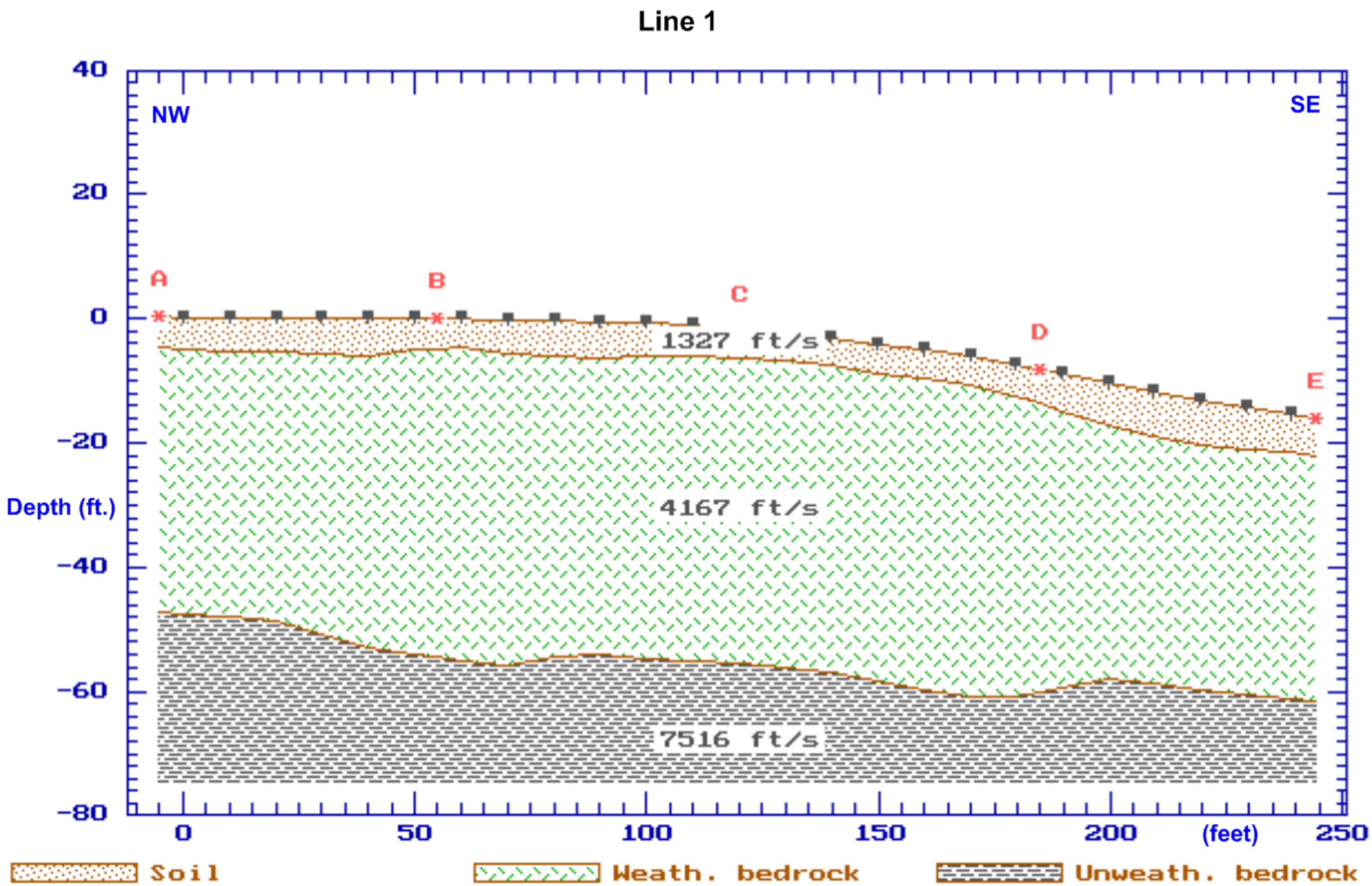
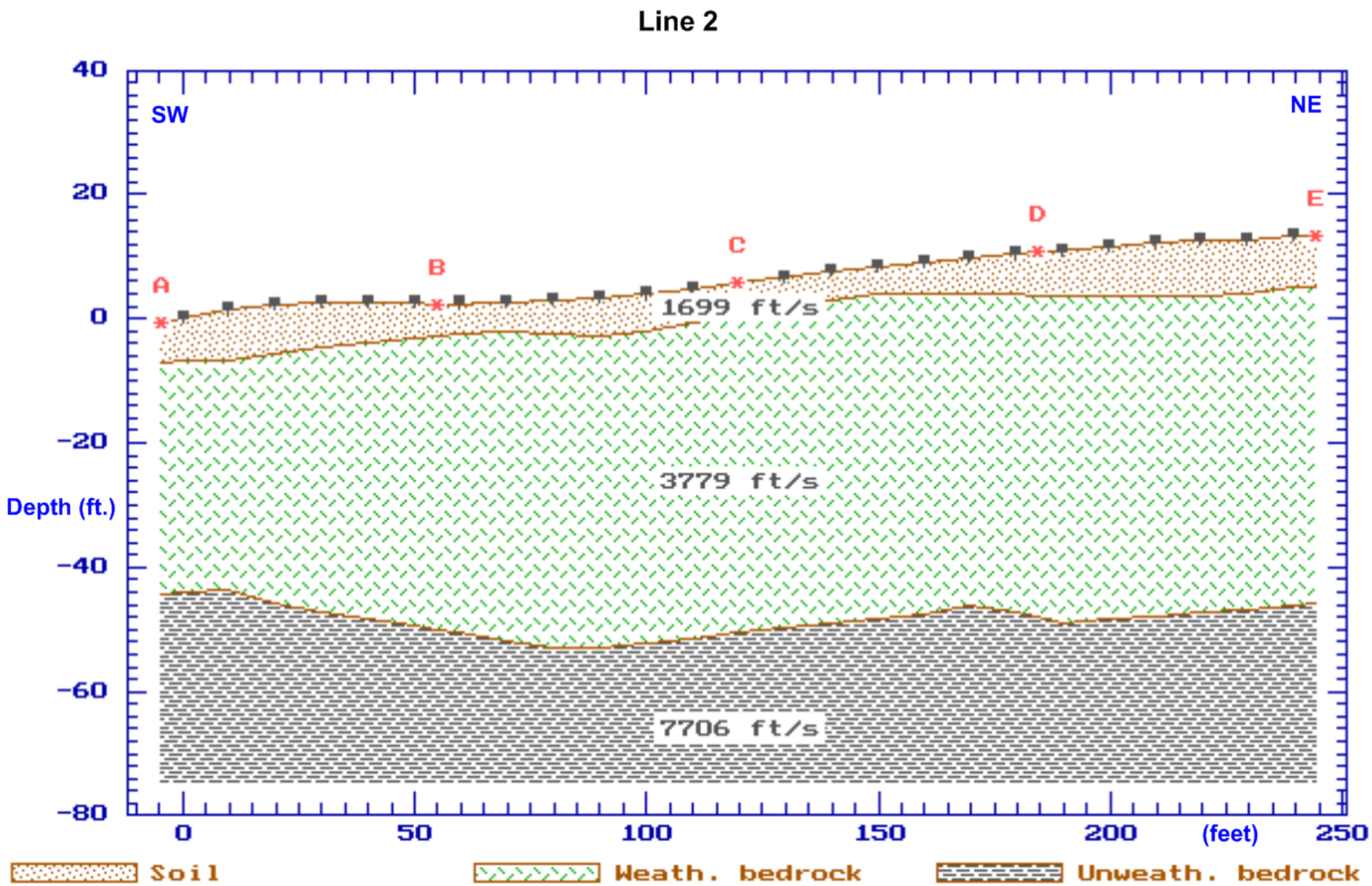

Rippers
Ripper Performance
• D9R/D9T
D9R/D9T
• Multi or Single Shank No. 9 Ripper
• Estimated by Seismic Wave Velocities
Seismic Velocity
Meters Per Second 1000
Feet Per Second 1000
0
1
2
3
4
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
TOPSOIL
CLAY
GLACIAL TILL
IGNEOUS ROCKS
GRANITE
BASALT
TRAP ROCK
SEDIMENTARY ROCKS
SHALE
SANDSTONE
SILTSTONE
CLAYSTONE
CONGLOMERATE
BRECCIA
CALICHE
LIMESTONE
METAMORPHIC ROCKS
SCHIST
SLATE
MINERALS and ORES
COAL
IRON ORE
RIPPABLE
MARGINAL
NON-RIPPABLE

Figure 4

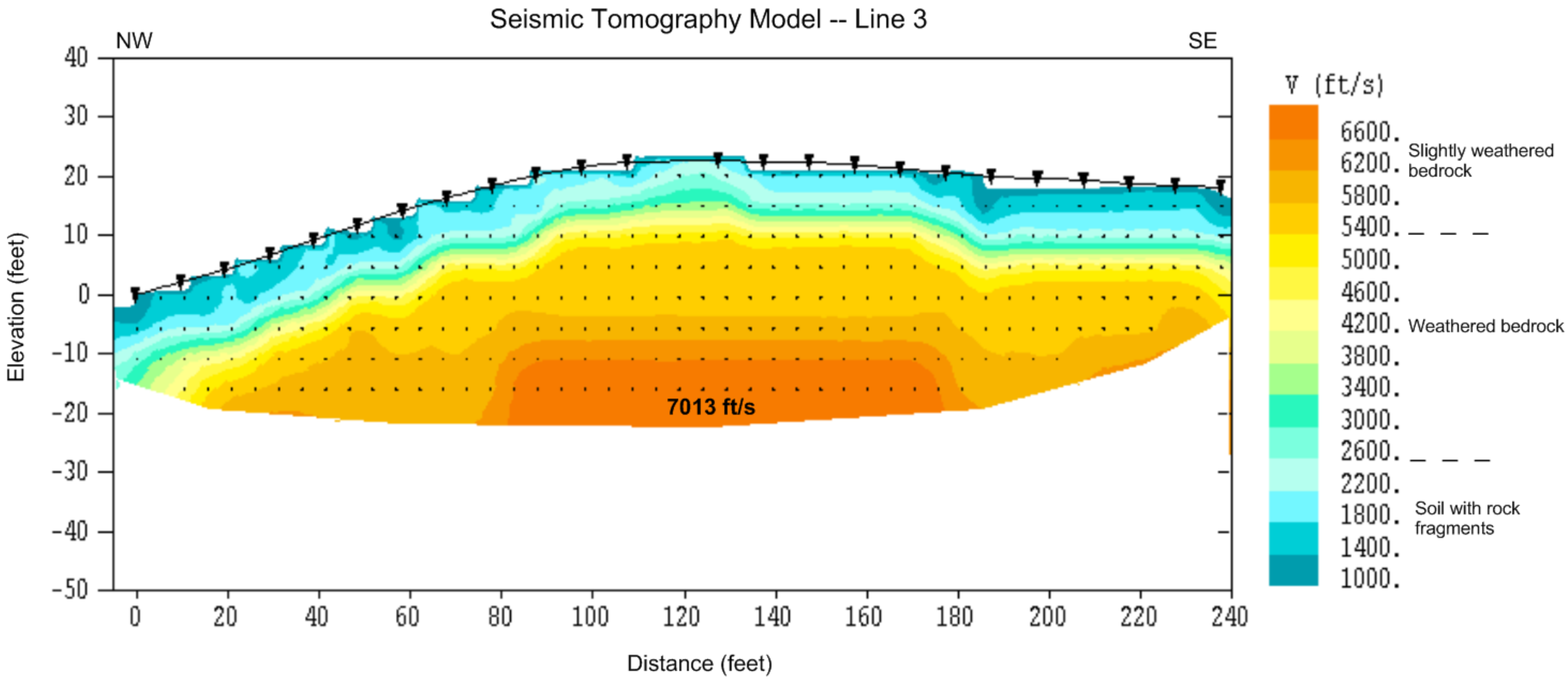


Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California

Figure 5

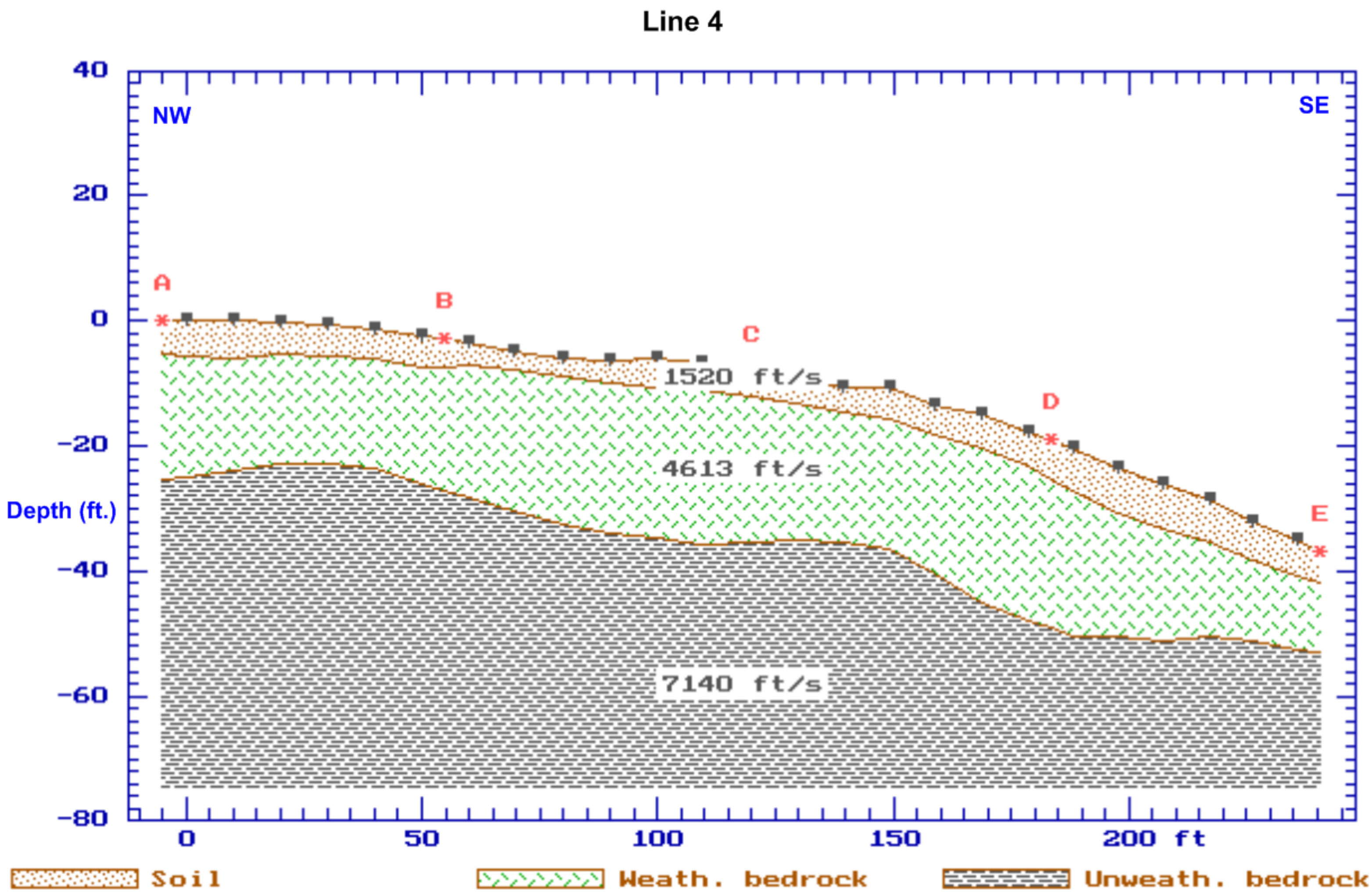


Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California **Figure 6**



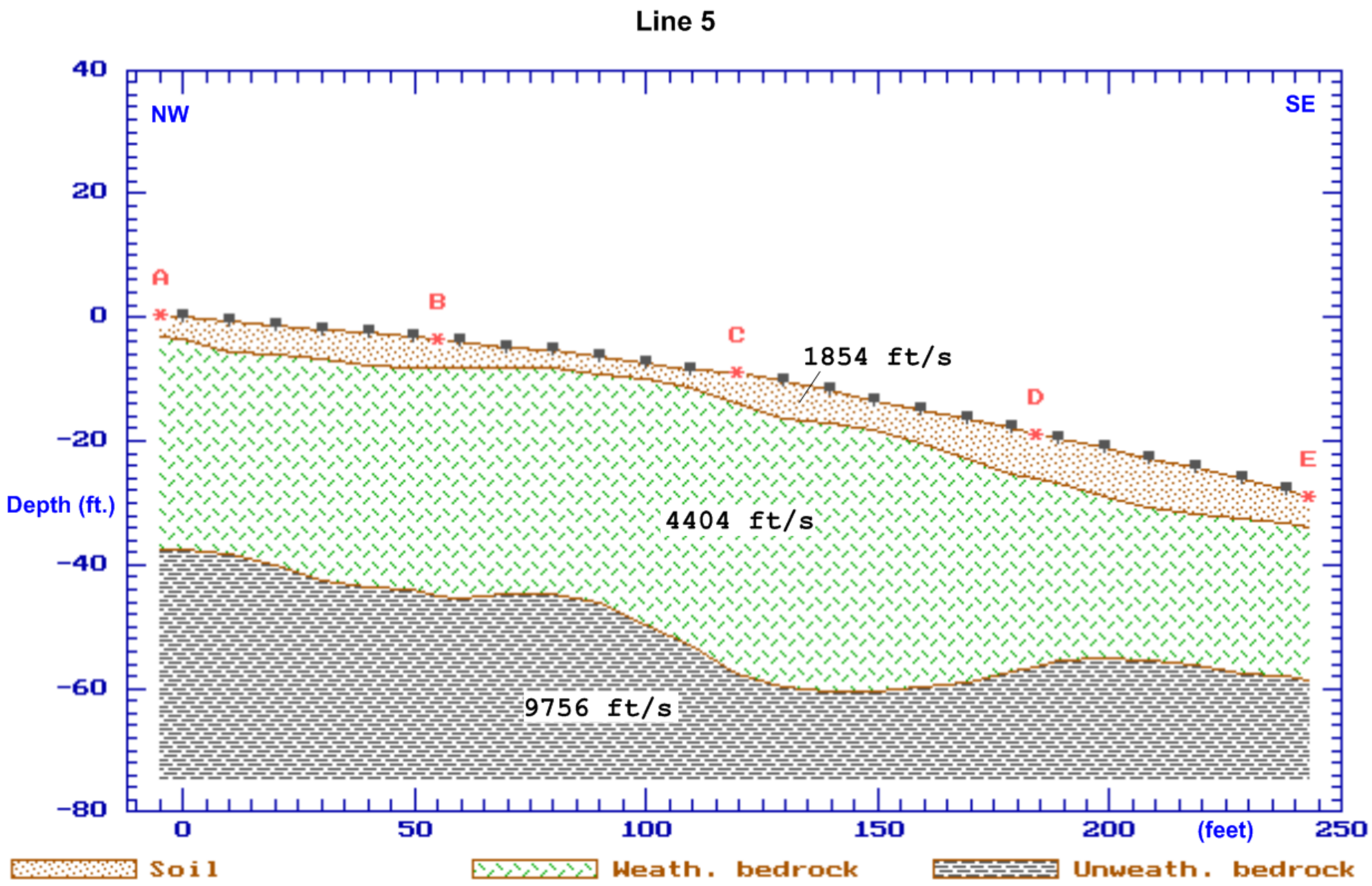
Seismic Tomography Model -- Line 3
NW
SE
Elevation (feet)
40
30
20
10
0
-10
-20
-30
-40
-50
0
20
40
60
80
100
120
140
160
180
200
220
240
Distance (feet)
7013 ft/s
V (ft/s)
6600.
6200.
5800.
5400.
5000.
4600.
4200.
3800.
3400.
3000.
2600.
2200.
1800.
1400.
1000.
Slightly weathered bedrock
Weathered bedrock
Soil with rock fragments

Figure 7

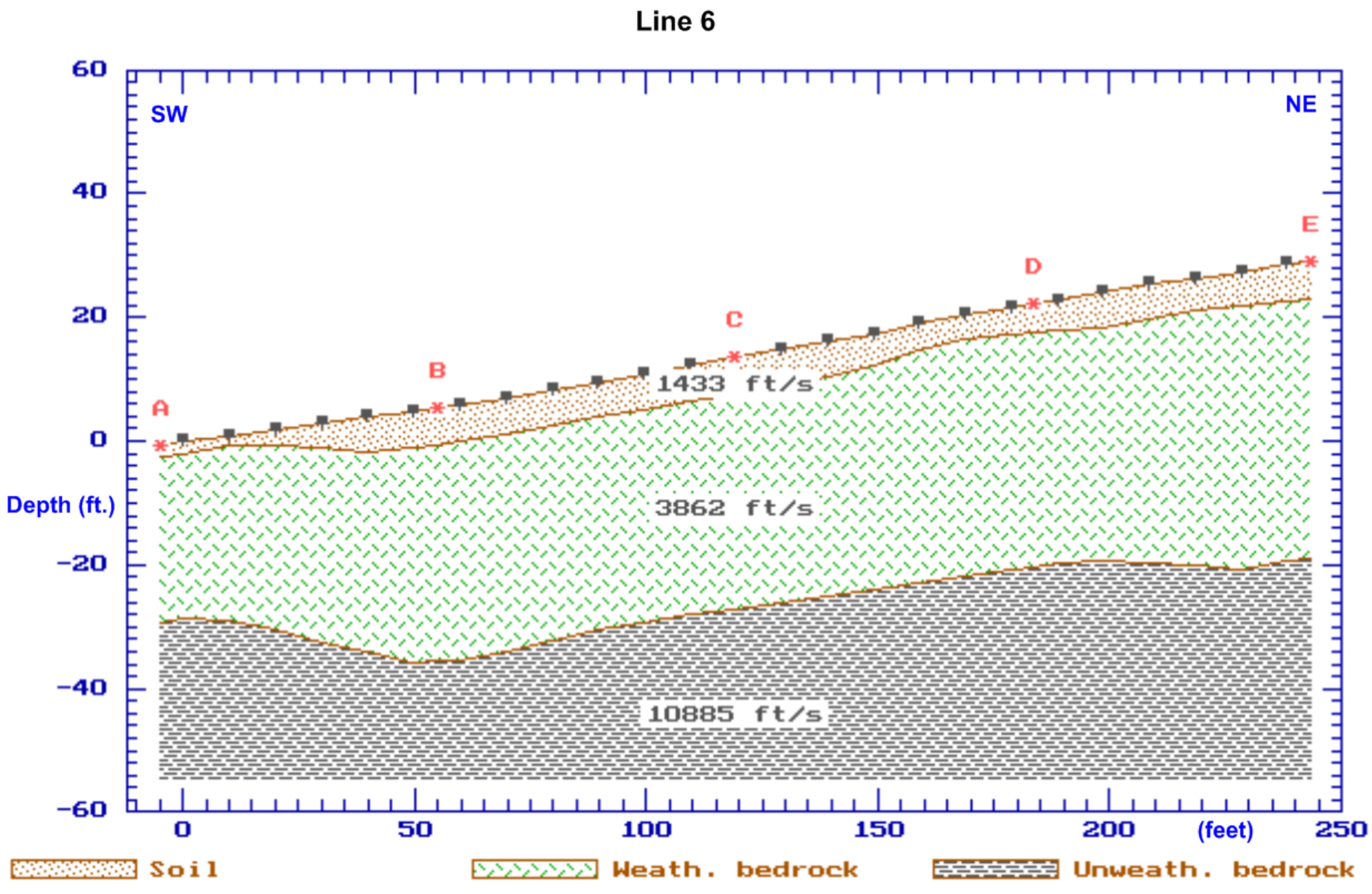


Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California

Figure 8

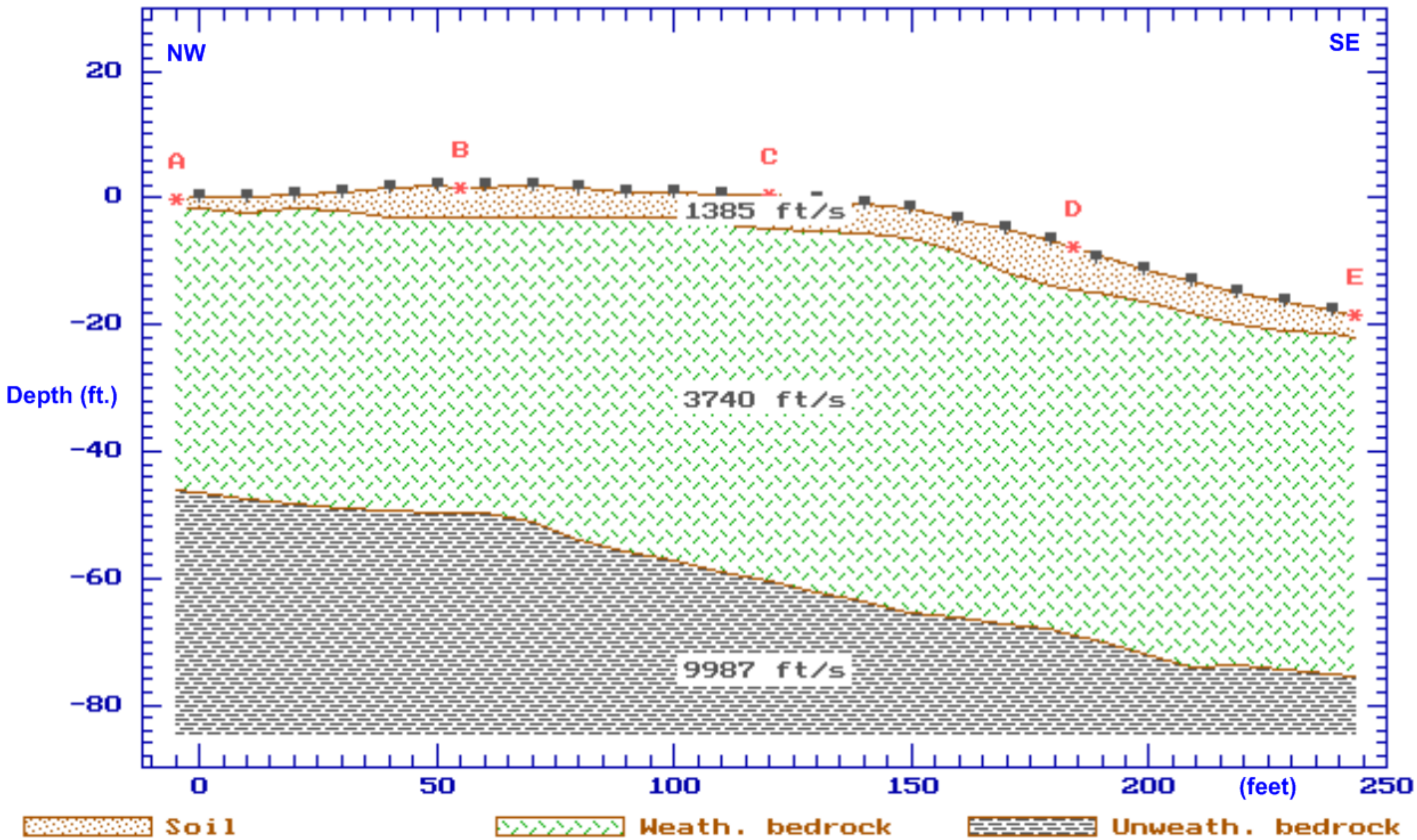


Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California **Figure 9**



Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California **Figure 10**

Line 7



Seismic Refraction Survey 02/19/2021	Keller Crossing Project
Geotek Project No. 2453-CR	Winchester, California Figure 11

APPENDIX D

RESULTS OF LABORATORY TESTING BY GEOTEK

Updated Geotechnical and Infiltration Evaluation
Keller Crossing Project, Winchester Area, Riverside County, California
Project No. 2453-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of trenches and borings in Appendix B.

Moisture-Density Relationship
Laboratory testing was performed on various samples obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content was determined in general accordance with ASTM D 1557. The results of the testing are provided herein.

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rate of deformation was approximately 0.035 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. Testing was performed on remolded soil samples (90% of the maximum dry density per ASTM D 1557) or undisturbed samples. The shear test results are presented herein.

Expansion Index
Expansion Index testing was performed on several site samples. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided herein.

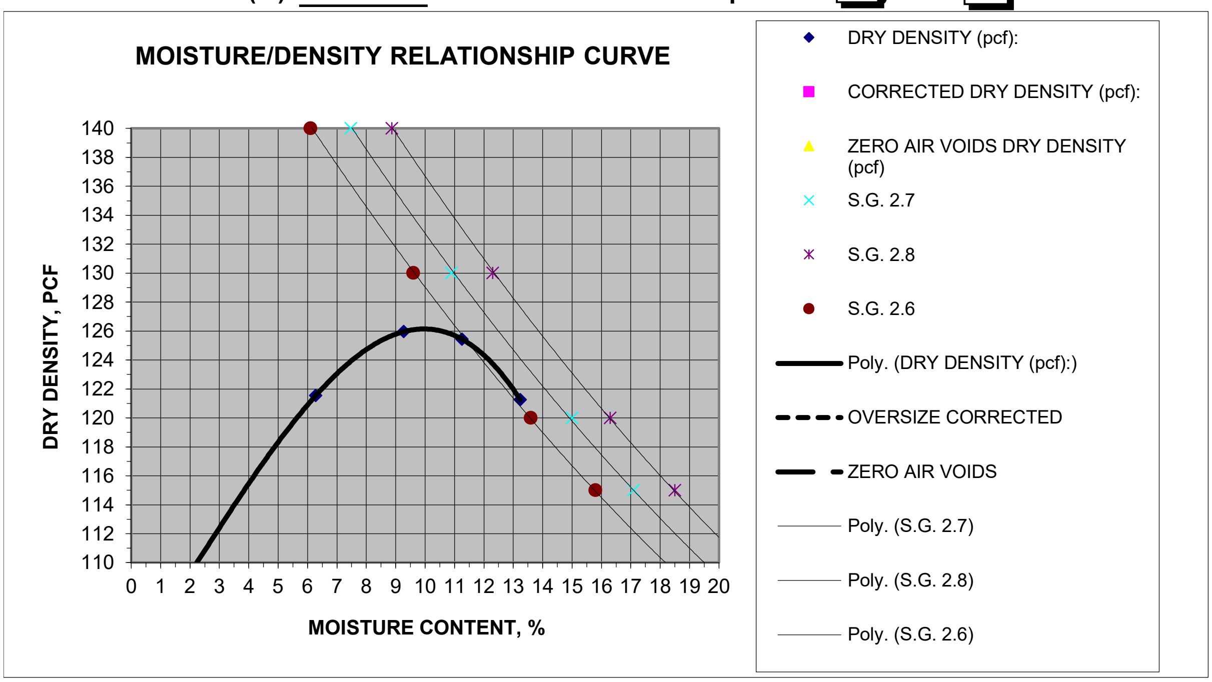
Sulfate, Resistivity, and Chloride Content
Testing to determine the water-soluble sulfate content, resistivity, and chloride content was performed by others. The results are presented herein.

GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client: Dr. Horton
Job No.: 2453-CR
Project: Keller Crossing
Lab No.: Corona
Location: Winchester
Material Type: Greyish Silty Sand/ Sandy Silt
Material Supplier: -
Material Source: -
Sample Location: B1 @ 10-15'
-
Sampled By: GP
Date Sampled: 2/1/2021
Received By: RJ
Date Received: 2/1/2021
Tested By: FS
Date Tested: 2/3/2021
Reviewed By: RJ
Date Reviewed: 2/4/2021

Test Procedure: ASTM D1557
Method: A
Oversized Material (%): 0.0
Correction Required: ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	126.0	**@ Optimum Moisture, %**	10.0
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)

Atterberg Limits:

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

Classification:

Unified Soils Classification: ____________

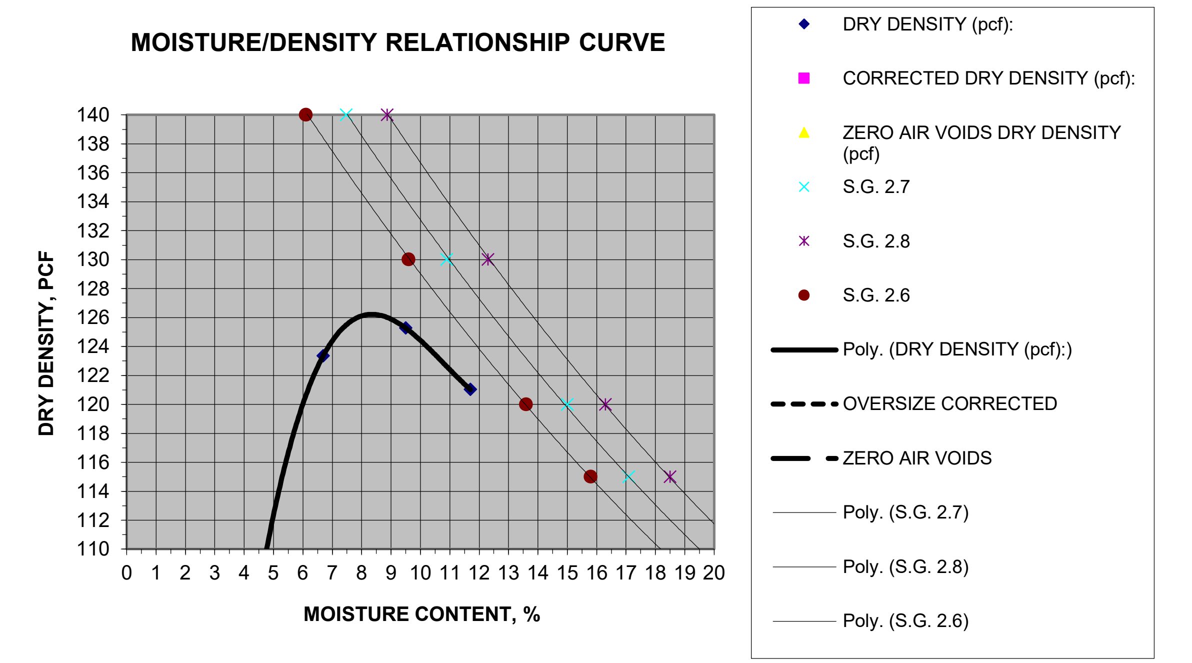
AASHTO Soils Classification: ____________

MOISTURE/DENSITY RELATIONSHIP

Client:	Dr. Horton	**Job No.:**	2453-CR
Project:	Keller Crossing	**Lab No.:**	Corona
Location:	Winchester		
Material Type:	Greyish Silty Sand/ Sandy Silt		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B2 @ 0-5'		
	-		
Sampled By:	GP	**Date Sampled:**	2/1/2021
Received By:	RJ	**Date Received:**	2/1/2021
Tested By:	RL	**Date Tested:**	2/4/2021
Reviewed By:	RJ	**Date Reviewed:**	2/5/2021

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 0.0 **Correction Required:** [ ] yes [x] no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	126.0	**@ Optimum Moisture, %**	8.5
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)

Atterberg Limits:

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

Classification:

Unified Soils Classification:

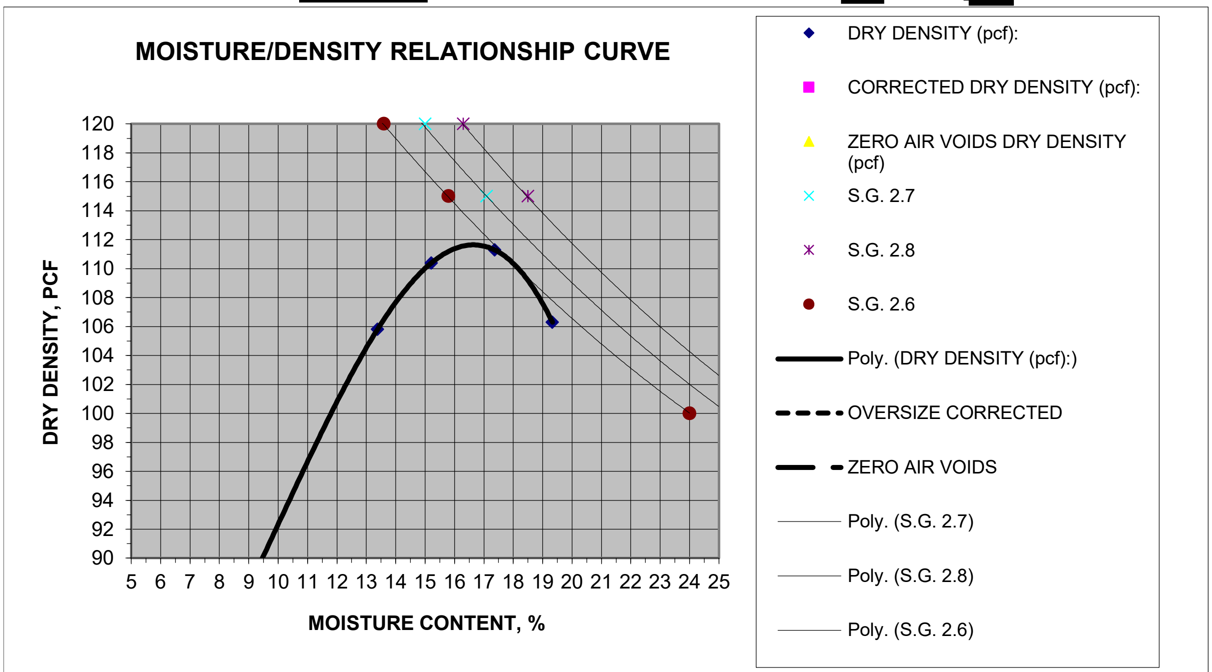
AASHTO Soils Classification:

MOISTURE/DENSITY RELATIONSHIP

Client:	DR Horton	**Job No.:**	2453-CR
Project:	Keller Crossing	**Lab No.:**	Corona
Location:	Winchester		
Material Type:	Greyish Brown Silty Sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B3 @ 0-5'		
	-		
Sampled By:	N/A	**Date Sampled:**	3/31/2021
Received By:	RJ	**Date Received:**	3/31/2021
Tested By:	RL	**Date Tested:**	4/6/2021
Reviewed By:	RJ	**Date Reviewed:**	4/7/2021

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 11.0 **Correction Required:** ☒ **yes** ☐ **no**



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	111.5	**@ Optimum Moisture, %**	16.5
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)

Atterberg Limits:

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

Classification:

Unified Soils Classification: ______

AASHTO Soils Classification: ______

GEOTEK

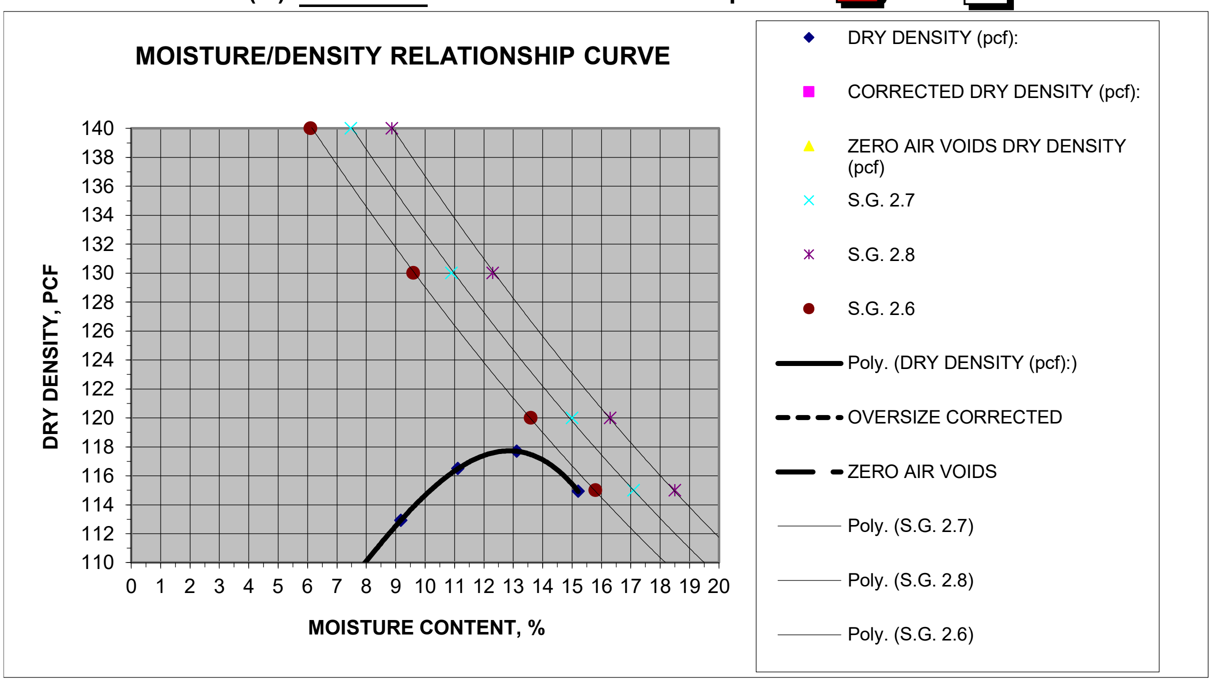
MOISTURE/DENSITY RELATIONSHIP

Client: DR Horton
Project: Keller Crossing
Location: Winchester
Material Type: Brown Decomposed Bedrock
Material Supplier: -
Material Source: -
Sample Location: T2 @ 0-5'
-
Sampled By: GP
Received By: RJ
Tested By: RL
Reviewed By: RJ

Job No.: 2453CR
Lab No.: Corona

Date Sampled: 2/25/2021
Date Received: 2/25/2021
Date Tested: 3/8/2021
Date Reviewed: 3/9/2021

Test Procedure: ASTM D1557 **Method:** B
Oversized Material (%): 23.0 **Correction Required:** yes no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	118.0	**@ Optimum Moisture, %**	13.0
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)

Atterberg Limits:

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

Classification:

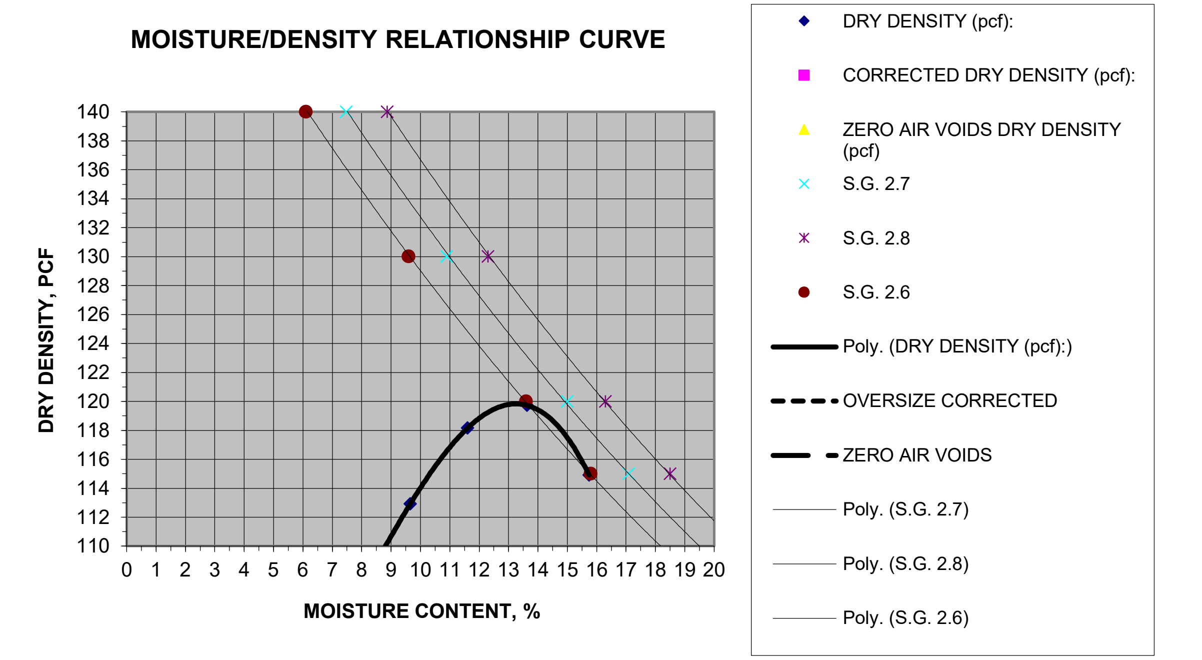
Unified Soils Classification:
AASHTO Soils Classification:

MOISTURE/DENSITY RELATIONSHIP

Client:	DR Horton	**Job No.:**	2453CR
Project:	Keller Crossing	**Lab No.:**	Corona
Location:	Winchester		
Material Type:	Greyish Brown Decomposed Bedrock		
Material Supplier:	-		
Material Source:	-		
Sample Location:	T5 @ 0-5'		
	-		
Sampled By:	GP	**Date Sampled:**	2/25/2021
Received By:	RJ	**Date Received:**	2/25/2021
Tested By:	RL	**Date Tested:**	3/8/2021
Reviewed By:	RJ	**Date Reviewed:**	3/9/2021

Test Procedure: ASTM D1557 **Method:** B

Oversized Material (%): 23.5 **Correction Required:** yes no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	120.0	**@ Optimum Moisture, %**	13.5
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)

Atterberg Limits:

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

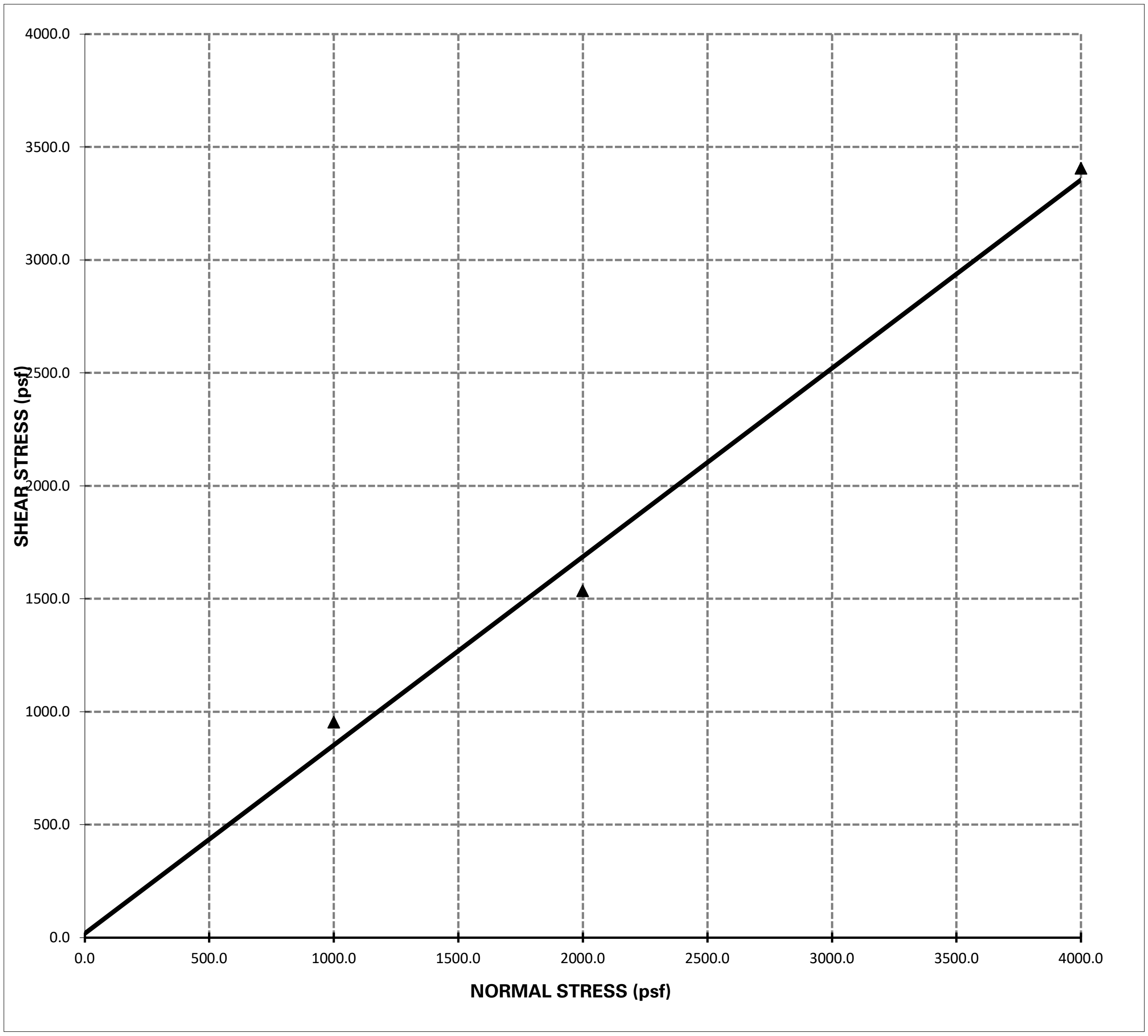
Classification:

Unified Soils Classification:

AASHTO Soils Classification:

DIRECT SHEAR TEST

Project Name:	DR Horton	***Sample Location:***	B-3 @ 0-5'
Project Number:	2453-CR	***Date Tested:***	4/8/2021



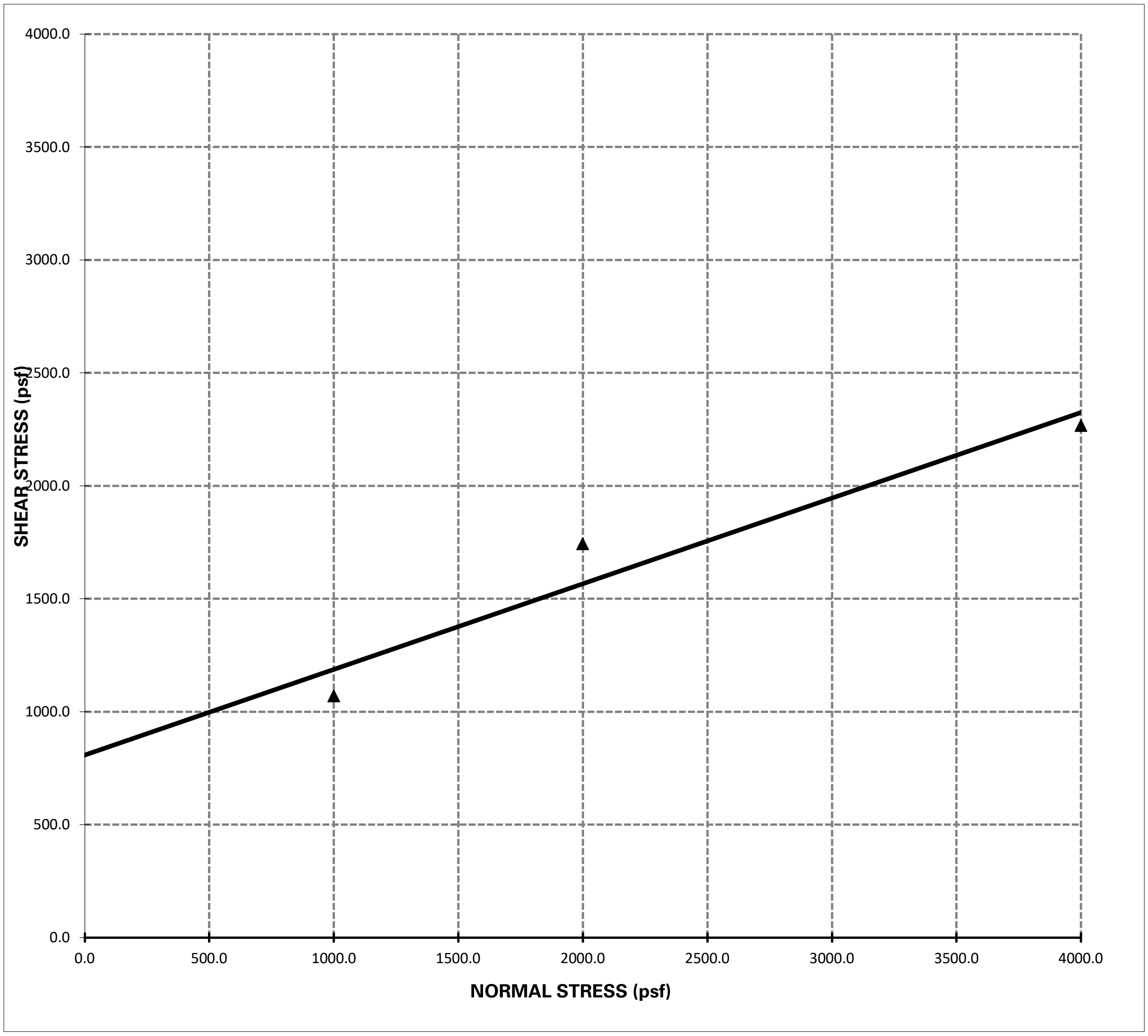
Shear Strength: Φ = 40 °, C = 18 psf

Notes:
1 - The soil specimens sheared were "remolded" samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

GEOTEK

Project Name:	Dr Horton	***Sample Location:***	B-3 @ 15'
Project Number:	2453-CR	***Date Tested:***	4/16/2021



Shear Strength: $\Phi =$ **21 °** , **C = 808 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.

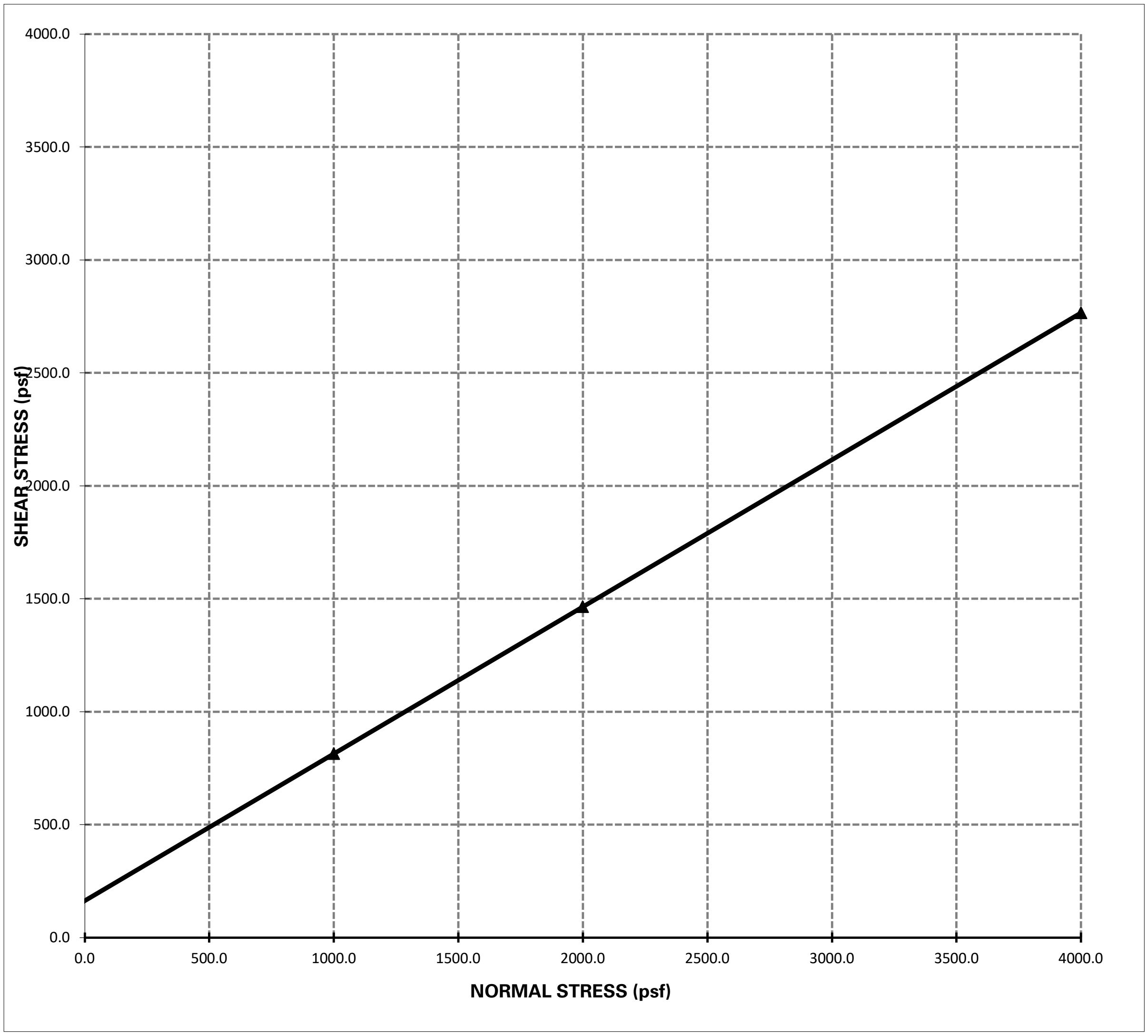
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

DIRECT SHEAR TEST

GEOTEK

Project Name: DR Horton
Project Number: 2543-CR
Sample Location: B-3 @ 25'
Date Tested: 4/7/2021

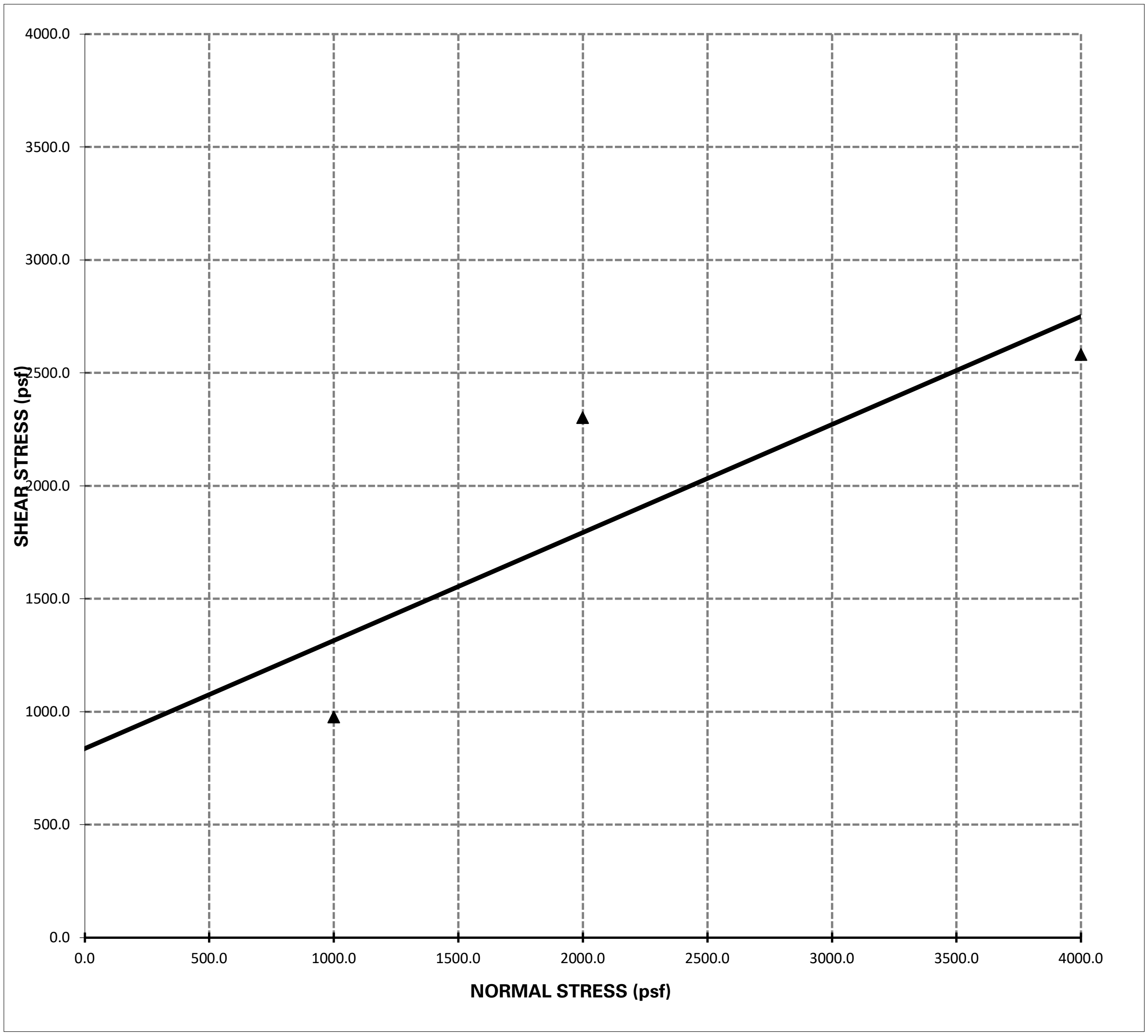


Shear Strength: Φ = 33 ° , C = 163 psf

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name: DR Horton
Project Number: 2453-CR
Sample Location: B-5 @ 5'
Date Tested: 4/8/2021



Shear Strength: Φ = 26 ° , C = 837 psf

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: DR Horton

Project Number: 2453-CR

Project Location: Keller Crossing, Winchester

Tested/ Checked By: FS Lab No Corona

Date Tested: 2/8/2021

Sample Source: B2 @ 0-5'

Sample Description:

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	779.1
B	Weight of ring (gm)	368.1
C	Net weight of sample (gm)	**411.0**
D	Wet Density, lb / ft3 (C*0.3016)	**124.0**
E	Dry Density, lb / ft3 (D/1.F)	**113.7**

SATURATION DETERMINATION

F	Moisture Content, %	9.0
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.5**

READINGS

DATE	TIME	READING	
2/8/2021		0.4170	Initial
2/8/2021		0.4170	10 min/Dry
2/9/2021		0.4420	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
807.3	**15.9**

EXPANSION INDEX = 25

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	DR Horton	***Tested/ Checked By:***	FS Lab No Corona
Project Number:	2453-CR	***Date Tested:***	2/8/2021
Project Location:	Keller Crossing, Winchester	***Sample Source:***	B1 @ 10-15'
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	771.0
B	Weight of ring (gm)	366.3
C	Net weight of sample (gm)	**404.7**
D	Wet Density, lb / ft3 (C*0.3016)	**122.1**
E	Dry Density, lb / ft3 (D/1.F)	**111.2**

SATURATION DETERMINATION

F	Moisture Content, %	9.8
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**51.3**

READINGS

DATE	TIME	READING	
2/8/2021		0.4740	Initial
2/8/2021		0.4740	10 min/Dry
2/9/2021		0.4770	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
798.1	**16.5**

EXPANSION INDEX = 3

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: DR Horton

Project Number: 2453-CR

Project Location: Keller Crossing, Winchester

Tested/ Checked By: RL Lab No Corona

Date Tested: 3/9/2021

Sample Source: T5 @ 0-5

Sample Description:

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	736.4
B	Weight of ring (gm)	364.1
C	Net weight of sample (gm)	**372.3**
D	Wet Density, lb / ft3 (C*0.3016)	**112.3**
E	Dry Density, lb / ft3 (D/1.F)	**98.9**

SATURATION DETERMINATION

F	Moisture Content, %	13.5
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**51.8**

READINGS

DATE	TIME	READING	
3/9/2021		0.1200	Initial
3/9/2021		0.1200	10 min/Dry
3/10/2021		0.1320	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
760.2	**19.9**

EXPANSION INDEX = 12

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: DR Horton

Project Number: 2453-CR

Project Location: Keller Crossing, Winchester

Tested/ Checked By: RL Lab No Corona

Date Tested: 3/9/2021

Sample Source: T2 @ 0-5'

Sample Description:

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	730.5
B	Weight of ring (gm)	363.1
C	Net weight of sample (gm)	**367.4**
D	Wet Density, lb / ft3 (C*0.3016)	**110.8**
E	Dry Density, lb / ft3 (D/1.F)	**98.1**

SATURATION DETERMINATION

F	Moisture Content, %	13.0
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.9**

READINGS

DATE	TIME	READING	
3/9/2021		0.1220	Initial
3/9/2021		0.1220	10 min/Dry
3/10/2021		0.1220	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
747.4	**17.6**

EXPANSION INDEX = 0

Table 1 - Laboratory Tests on Soil Samples

Geotek, Inc.
Keller Crossing
Your #2453-CR, HDR Lab #21-0120LAB
11-Feb-21

Sample ID			B-1 @ 10-15'
Resistivity		**Units**	
as-received		ohm-cm	7,200
saturated		ohm-cm	1,640
pH			7.6
Electrical Conductivity		mS/cm	0.11
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	32
magnesium	Mg^{2+}	mg/kg	9.4
sodium	Na^{1+}	mg/kg	80
potassium	K^{1+}	mg/kg	2.1
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	14
bicarbonate	HCO_3^{1-}	mg/kg	159
fluoride	F^{1-}	mg/kg	6.4
chloride	Cl^{1-}	mg/kg	33
sulfate	SO_4^{2-}	mg/kg	37
nitrate	NO_3^{1-}	mg/kg	144
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

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Phone: 909.962.5485 · Fax: 909.626.3316

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotek, Inc.
Keller Crossing
Your #2453-CR, HDR Lab #21-0284SCS
8-Apr-21

Sample ID			B-3 @ 20'	B-7 @ 7'
Resistivity		**Units**		
as-received		ohm-cm	7,200	5,600
saturated		ohm-cm	1,480	2,600
pH			7.7	7.8
Electrical Conductivity		mS/cm	0.04	0.03
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	32	28
magnesium	Mg^{2+}	mg/kg	2.1	3.9
sodium	Na^{1+}	mg/kg	14	18
potassium	K^{1+}	mg/kg	7.1	6.0
ammonium	NH_4^{1+}	mg/kg	ND	ND
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	134	149
fluoride	F^{1-}	mg/kg	4.4	3.1
chloride	Cl^{1-}	mg/kg	9.7	8.4
sulfate	SO_4^{2-}	mg/kg	22	7.6
nitrate	NO_3^{1-}	mg/kg	4.8	12
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

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Phone: 909.962.5485 · Fax: 909.626.3316

May 24, 2021

via email: elamont@geotekusa.com

Geotek, Inc.
1548 N. Maple St.
Corona, CA, 92880

Attention: Ed LaMont

Re: Soil Corrosivity Study
Keller Crossing
Winchester, CA
HDR #21-0284SCS, Geotek #2453-CR

Introduction

Laboratory tests have been completed on three soil samples provided for the referenced project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping and concrete structures. HDR assumes that the provided samples are representative of the most corrosive soils at the site. Additional soil samples will be provided for laboratory testing and will be added to this report at a future date.

The proposed structures include single and two story residential buildings with no subterranean levels, underground utilities, and two retention basins. The site is located in Winchester, California. The water table depth is reportedly greater than 50 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per ASTM International (ASTM) G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and American Water Works Association (AWWA) Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory numbers 21-0120LAB and 21-0284SCS. The full set of test results are shown in the attached Table A1 and Table A2, respectively.

Discussion

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil. A correlation between electrical resistivity and corrosivity toward ferrous metals is shown in Table 1.[1]

Table 1: Soil Corrosivity Categories.

Soil Resistivity (ohm-cm)	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the moderately corrosive category with as-received moisture. When saturated, the resistivities were in the moderately corrosive to corrosive categories.

Soil pH values varied from 7.6 to 7.8. This range is mildly alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low. Chloride and sulfate were found at low concentrations. Per American Concrete institute (ACI) 318, the soil is classified as S0 with respect to sulfate concentration.[3]

The nitrate concentration was high enough to be aggressive to copper. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

In conclusion, this soil is classified as corrosive to ferrous metals, aggressive to copper, and negligible (S0) for sulfate attack on concrete.

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[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*
[3] *American Concrete Institute (ACI) 318-19 Table 19.3.1.1.*

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion. The following recommendations are based on the evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE International (NACE) SP0286 from:

    a. Dissimilar metals.

    b. Dissimilarly coated piping (cement-mortar vs. dielectric).

    c. Above ground steel pipe.

    d. All existing piping.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable dielectric coating intended for underground use such as:

   i. Polyurethane per AWWA C222 *or*

   ii. Extruded polyethylene per AWWA C215 *or*

   iii. A tape coating system per AWWA C214 *or*

   iv. Hot applied coal tar enamel per AWWA C203 *or*

   v. Fusion bonded epoxy per AWWA C213.

b. Apply cathodic protection to steel piping as per NACE SP0169.

OPTION 2

As an alternative to the coating systems described in Option 1 and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some steel piping systems, such as oil, gas, insulated, or high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable coating intended for underground use such as:

   i. Polyethylene encasement per AWWA C105; *or*

   ii. Epoxy coating; *or*

   iii. Polyurethane; *or*

   iv. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to ductile iron piping as per NACE SP0169.

OPTION 2

As an alternative to the coating systems described in Option 1 and cathodic protection, encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe. Use the following parameters for clean sand backfill:

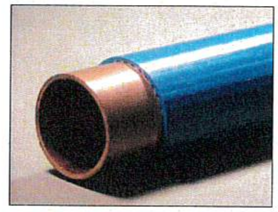
   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

   b. pH between 6.0 and 8.0.

   c. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]
2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.
3. Electrically insulate cold water piping from hot water piping systems.
4. Protect buried copper tubing by one of the following measures:
   a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; or
   b. Install copper pipe with a factory-applied coating that is at least 25 mils in thickness. Use Kamco's Aqua Shield™, Mueller Streamline's Plumbshield™, or equal. The coating must be continuous with no cuts or defects.



   c. Insulate the pipe by installing 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.
2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately designed cathodic protection system per NACE SP0169.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

NOTE: Interior surfaces of concrete structures and pipe used to transport wastewater may require additional corrosion protection measures, such as linings, based on the flow conditions and wastewater characteristics. These considerations are beyond the scope of this report.

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[4] 2016 California Plumbing Code (CPC), July 1, 2018 Supplement, Section 604.3.
[5] 2018 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[6] 2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[7] 2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[8] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65

Expanded Analysis

Recommendations are based on preliminary soil corrosivity data. Additional soil samples will be submitted to the laboratory for corrosivity testing and will be amended to this report.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

DRAFT

Steven Pierce, EIT
Corrosion EIT

DRAFT

Marc E N Wegner, PE
Sr Corrosion Project Manager

Enc: Table A1 – Laboratory Tests on Soil Samples
Table A2 – Laboratory Tests on Soil Samples

Table A1 - Laboratory Tests on Soil Samples

Geotek, Inc.
Keller Crossing
Your #2453-CR, HDR Lab #21-0120LAB
11-Feb-21

Sample ID			B-1 @ 10-15'
Resistivity		**Units**	
as-received		ohm-cm	7,200
saturated		ohm-cm	1,640
pH			7.6
Electrical Conductivity		mS/cm	0.11
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	32
magnesium	Mg^{2+}	mg/kg	9.4
sodium	Na^{1+}	mg/kg	80
potassium	K^{1+}	mg/kg	2.1
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	14
bicarbonate	HCO_3^{1-}	mg/kg	159
fluoride	F^{1-}	mg/kg	6.4
chloride	Cl^{1-}	mg/kg	33
sulfate	SO_4^{2-}	mg/kg	37
nitrate	NO_3^{1-}	mg/kg	144
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

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HDR

Table A2 - Laboratory Tests on Soil Samples

Geotek, Inc.
Keller Crossing
Your #2453-CR, HDR Lab #21-0284SCS
8-Apr-21

Sample ID

		Units	B1 @ 20'	B5 @ 7'
Resistivity				
as-received		ohm-cm	7,200	5,600
saturated		ohm-cm	1,480	2,600
pH			7.7	7.8
Electrical Conductivity		mS/cm	0.04	0.03
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	32	28
magnesium	Mg^{2+}	mg/kg	2.1	3.9
sodium	Na^{1+}	mg/kg	14	18
potassium	K^{1+}	mg/kg	7.1	6.0
ammonium	NH_4^{1+}	mg/kg	ND	ND
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	134	149
fluoride	F^{1-}	mg/kg	4.4	3.1
chloride	Cl^{1-}	mg/kg	9.7	8.4
sulfate	SO_4^{2-}	mg/kg	22	7.6
nitrate	NO_3^{1-}	mg/kg	4.8	12
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

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APPENDIX E

RESULTS OF INFILTRATION TESTS

Updated Geotechnical and Infiltration Evaluation
Keller Crossing Project, Winchester Area, Riverside County, California
Project No. 2453-CR

GEOTEK

Client:	**DR Horton**
Project:	**Keller Crossing, Winchester**
Project No:	**2453-CR**
Date:	**2/2/2021**

Boring No. **I-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.75
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	35.25
$\Delta H = \Delta D = H_O - H_F$ =	0.75
$Havg = (H_O+H_F)/2$ =	35.625

I_t = 0.08 **Inches per Hour**

GEOTEK

Client:	**DR Horton**
Project:	**Keller Crossing, Winchester**
Project No:	**2453-CR**
Date:	**2/2/2021**

Boring No. **I-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	35.5
$\Delta H = \Delta D = H_O - H_F$ =	0.5
Havg = $(H_O+H_F)/2$ =	35.75

I_t = 0.05 **Inches per Hour**

GEOTEK

Client: DR Horton
Project: Keller Crossing, Winchester
Project No: 2453-CR
Date: 2/2/2021

Boring No. I-3

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	35.5
$\Delta H = \Delta D = H_O - H_F$ =	0.5
$Havg = (H_O+H_F)/2$ =	35.75

I_t = 0.05 **Inches per Hour**

GEOTEK

Client: DR Horton

Project: Keller Crossing, Winchester

Project No: 2453-CR

Date: 2/2/2021

Boring No. I-4

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	35.5
$\Delta H = \Delta D = H_O - H_F$ =	0.5
Havg = $(H_O+H_F)/2$ =	35.75

I_t = 0.05 **Inches per Hour**

GEOTEK

Client:	**DR Horton**
Project:	**Keller Crossing, Winchester**
Project No:	**2453-CR**
Date:	**2/2/2021**

Boring No. **I-5**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	24.75
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	35.25
$\Delta H = \Delta D = H_O - H_F$ =	0.75
$Havg = (H_O+H_F)/2$ =	35.625

I_t = 0.08 **Inches per Hour**

GEOTEK

Client:	**DR Horton**
Project:	**Keller Crossing, Winchester**
Project No:	**2453-CR**
Date:	**2/2/2021**

Boring No. **I-6**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	25.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	24
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	36
$H_F = D_T - D_F$ =	34.5
$\Delta H = \Delta D = H_O - H_F$ =	1.5
Havg = $(H_O+H_F)/2$ =	35.25

I_t = 0.16 **Inches per Hour**

GEOTEK

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester	Project No:	2453-CR	Date:	2/2/21
Test Hole No:	I-1	Tested By:	G. Pocius		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM		

Test Hole Dimensions (inches)		Length	Width
Diameter (if round)= 8	Sides (if rectangular)=		

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:17	7:42	25	24	25.25	1.25	N
2	7:42	8:07	25	24	25.25	1.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	8:07	8:37	30	24	25.25	1.25	
2	8:37	9:07	30	24	25.25	1.25	
3	9:07	9:37	30	24	25.25	1.25	
4	9:37	10:07	30	24	25	1	
5	10:07	10:37	30	24	25	1	
6	10:37	11:07	30	24	25	1	
7	11:07	11:37	30	24	25	1	
8	11:37	12:07	30	24	25	1	
9	12:07	12:37	30	24	24.75	0.75	
10	12:37	1:07	30	24	24.75	0.75	
11	1:07	1:37	30	24	24.75	0.75	
12	1:37	2:07	30	24	24.75	0.75	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester	**Project No:**	2453-CR	**Date:**	2/2/21
Test Hole No:	I-2	**Tested By:**	G. Pocius		
Depth of Test Hole, D_T:	5	**USCS Soil Classification:**	SM		

Test Hole Dimensions (inches)			Length	Width	
Diameter (if round)=	8	Sides (if rectangular)=			

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:15	7:40	25	24	25.25	1.25	N
2	7:40	8:05	25	24	25.25	1.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_0 Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	8:05	8:35	30	24	25	1	
2	8:35	9:05	30	24	25	1	
3	9:05	9:35	30	24	25	1	
4	9:35	10:05	30	24	25	1	
5	10:05	10:35	30	24	25	1	
6	10:35	11:05	30	24	24.75	0.75	
7	11:05	11:35	30	24	24.75	0.75	
8	11:35	12:05	30	24	24.75	0.75	
9	12:05	12:35	30	24	24.75	0.75	
10	12:35	1:05	30	24	24.5	0.5	
11	1:05	1:35	30	24	24.5	0.5	
12	1:35	2:05	30	24	24.5	0.5	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester		Project No:	2453-CR		Date:	2/2/21
Test Hole No:		I-3	Tested By:	G. Pocius			
Depth of Test Hole, D_T:		5	USCS Soil Classification:		SM		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		8	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:19	7:49	25	24	25.25	1.25	N
2	7:49	8:09	25	24	25.25	1.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	8:09	8:39	30	24	25	1	
2	8:39	9:09	30	24	25	1	
3	9:09	9:39	30	24	25	1	
4	9:39	10:09	30	24	25	1	
5	10:09	10:39	30	24	25	1	
6	10:39	11:09	30	24	24.75	0.75	
7	11:09	11:39	30	24	24.75	0.75	
8	11:39	12:09	30	24	24.75	0.75	
9	12:09	12:39	30	24	24.75	0.75	
10	12:39	1:09	30	24	24.5	0.5	
11	1:09	1:39	30	24	24.5	0.5	
12	1:39	2:09	30	24	24.5	0.5	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester	Project No:	2453-CR	Date:	2/2/21
Test Hole No:	I-4	Tested By:	G. Pocius		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM		

Test Hole Dimensions (inches)			Length	Width	
Diameter (if round)=	8	Sides (if rectangular)=			

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:07	7:32	25	24	25.25	1.25	N
2	7:32	7:57	25	24	25.25	1.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	7:57	8:27	30	24	25	1	
2	8:27	8:57	30	24	25	1	
3	8:57	9:27	30	24	25	1	
4	9:27	9:57	30	24	25	1	
5	9:57	10:27	30	24	24.75	0.75	
6	10:27	10:57	30	24	24.75	0.75	
7	10:57	11:27	30	24	24.75	0.75	
8	11:27	11:57	30	24	24.75	0.75	
9	11:57	12:27	30	24	24.5	0.5	
10	12:27	12:57	30	24	24.5	0.5	
11	12:57	1:27	30	24	24.5	0.5	
12	1:27	1:57	30	24	24.5	0.5	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester		Project No:	2453-CR		Date:	2/2/21
Test Hole No:		I-5	Tested By:	G. Pocius			
Depth of Test Hole, D_T:		5	USCS Soil Classification:		SM		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		8	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:09	7:34	25	24	25.5	1.5	N
2	7:34	7:59	25	24	25.5	1.5	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	7:59	8:29	30	24	25.25	1.25	
2	8:29	8:59	30	24	25.25	1.25	
3	8:59	9:29	30	24	25.25	1.25	
4	9:29	9:59	30	24	25.25	1.25	
5	9:59	10:29	30	24	25	1	
6	10:29	10:59	30	24	25	1	
7	10:59	11:29	30	24	25	1	
8	11:29	11:59	30	24	25	1	
9	11:59	12:29	30	24	25	1	
10	12:29	12:59	30	24	24.75	0.75	
11	12:59	1:29	30	24	24.75	0.75	
12	1:29	1:59	30	24	24.75	0.75	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Keller Crossing, Winchester	Project No:	2453-CR	Date:	2/2/21
Test Hole No:	I-6	Tested By:	G. Pocius		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM		

Test Hole Dimensions (inches)		Length	Width
Diameter (if round)=	8		
Sides (if rectangular)=			

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	7:11	7:36	25	24	26	2	
2	7:36	8:01	25	24	26	2	

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	8:01	8:31	30	24	26	2	
2	8:31	9:01	30	24	26	2	
3	9:01	9:31	30	24	26	2	
4	9:31	10:01	30	24	25.75	1.75	
5	10:01	10:31	30	24	25.75	1.75	
6	10:31	11:01	30	24	25.75	1.75	
7	11:01	11:31	30	24	25.75	1.75	
8	11:31	12:01	30	24	25.5	1.5	
9	12:01	12:31	30	24	25.5	1.5	
10	12:31	1:01	30	24	25.5	1.5	
11	1:01	1:31	30	24	25.5	1.5	
12	1:31	2:01	30	24	25.5	1.5	
13							
14							
15							

COMMENTS:

APPENDIX F

GENERAL GRADING GUIDELINES

Updated Geotechnical and Infiltration Evaluation
Keller Crossing Project, Winchester Area, Riverside County, California
Project No. 2453-CR

GEOTEK

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 California Building Code, CBC (2019) 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 F-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 schematic diagrams Plate F-1, F-2 and F-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 Plates 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 F-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 F-2, F-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 F-1, F-2, and F-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 F-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 F-2.
4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate F-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 F-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 F-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 advice 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 affect 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 contractor's 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 recommend 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 contractor's 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 contractor's 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 contractor's 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.

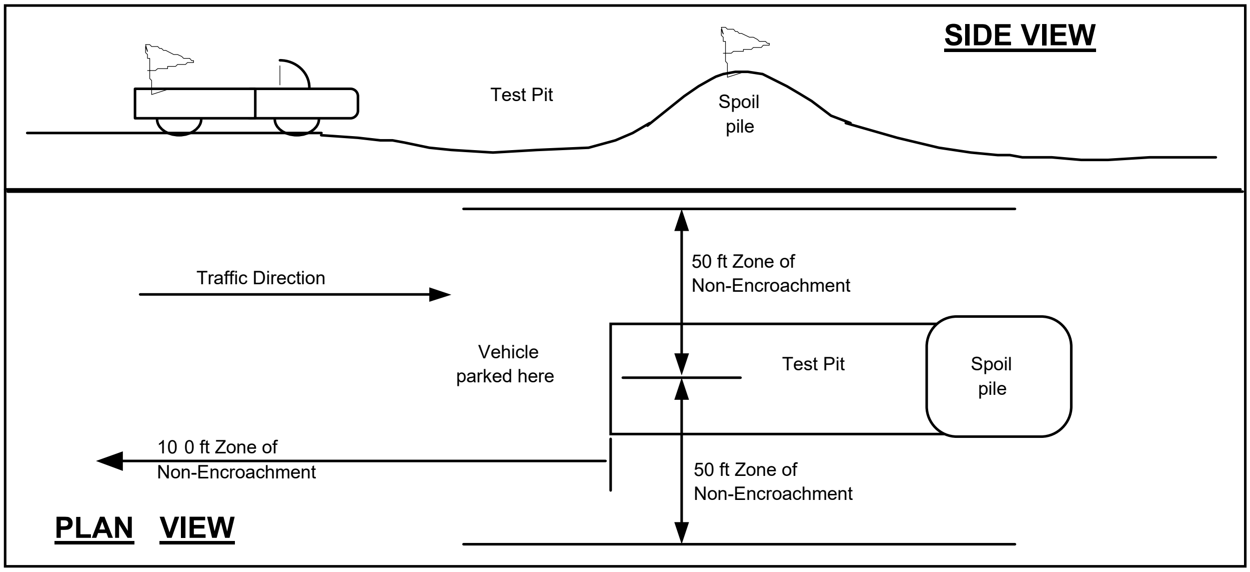
Test Pits Location, Orientation and Clearance

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 affect 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 affect 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 technician's attention and notify our project manager or office. Effective communication and coordination between the contractors' 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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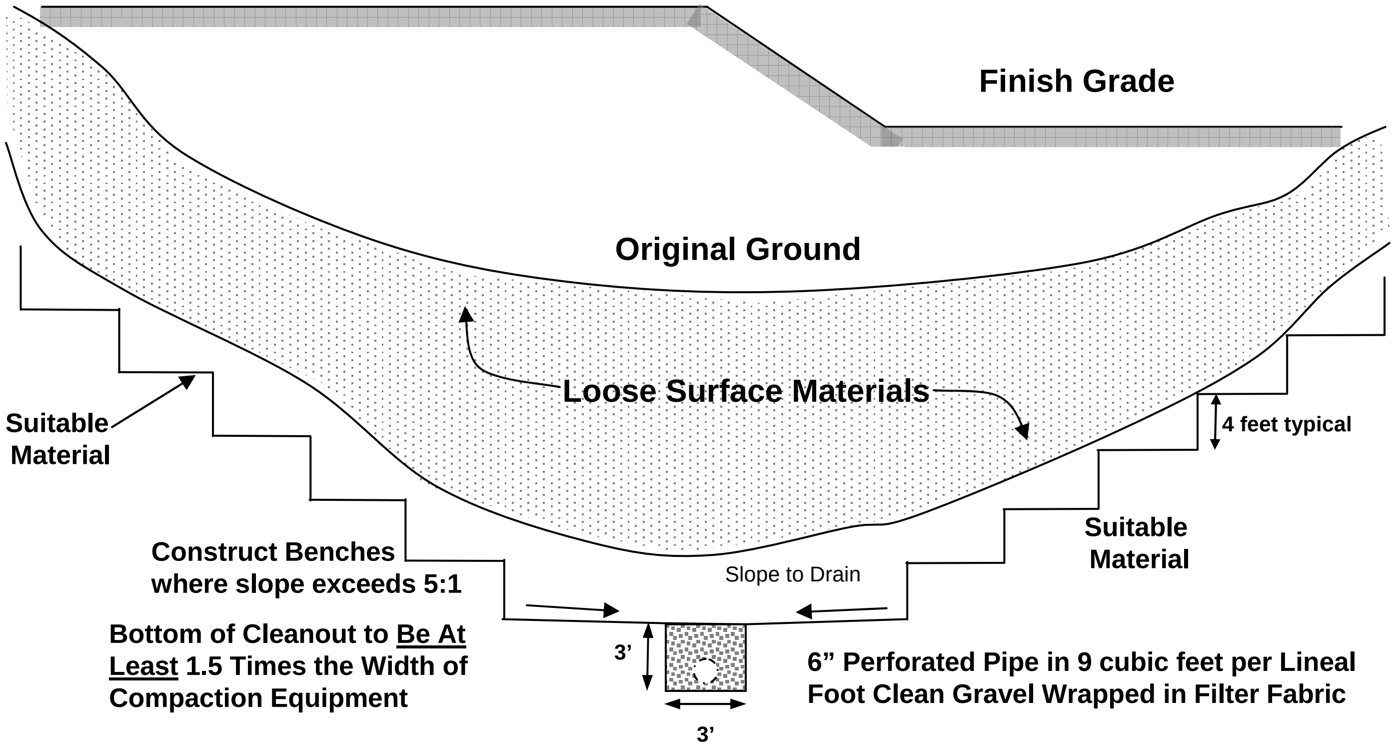
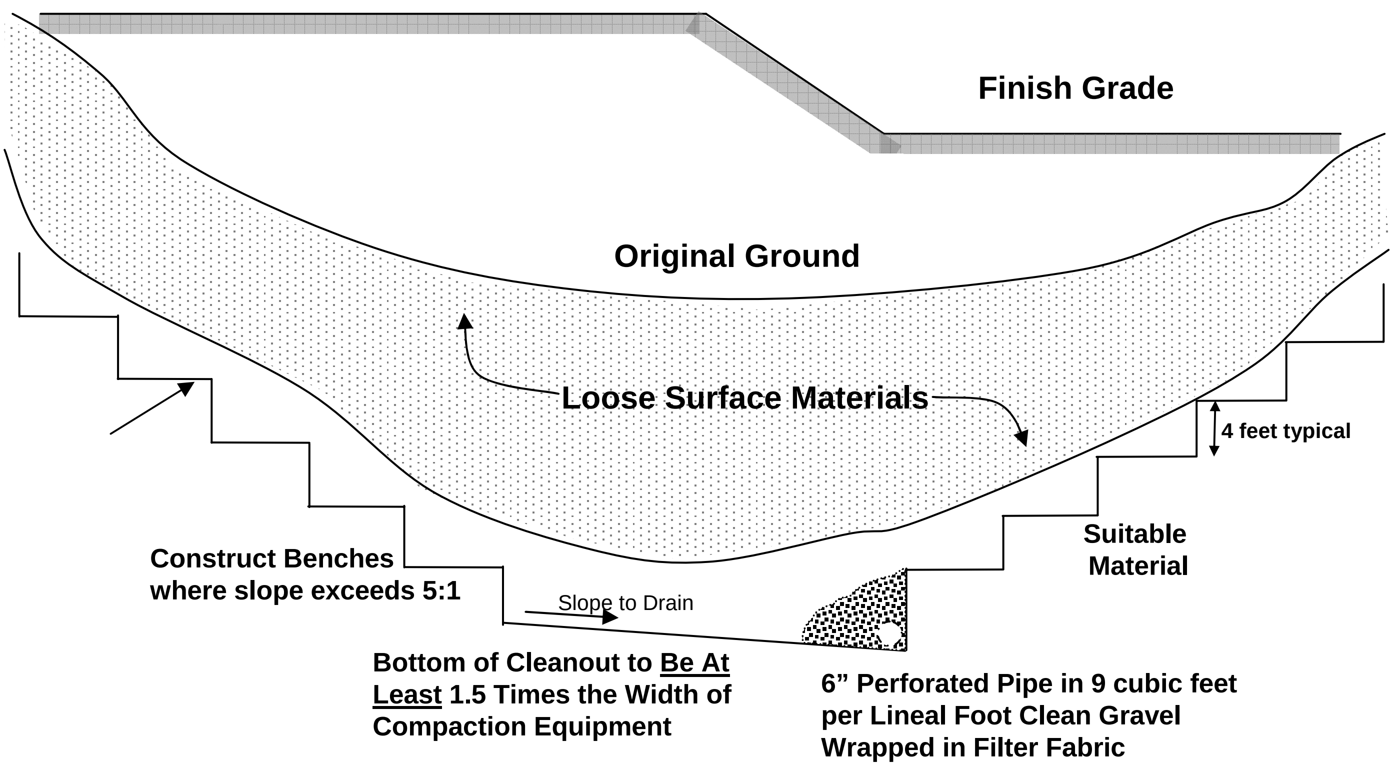
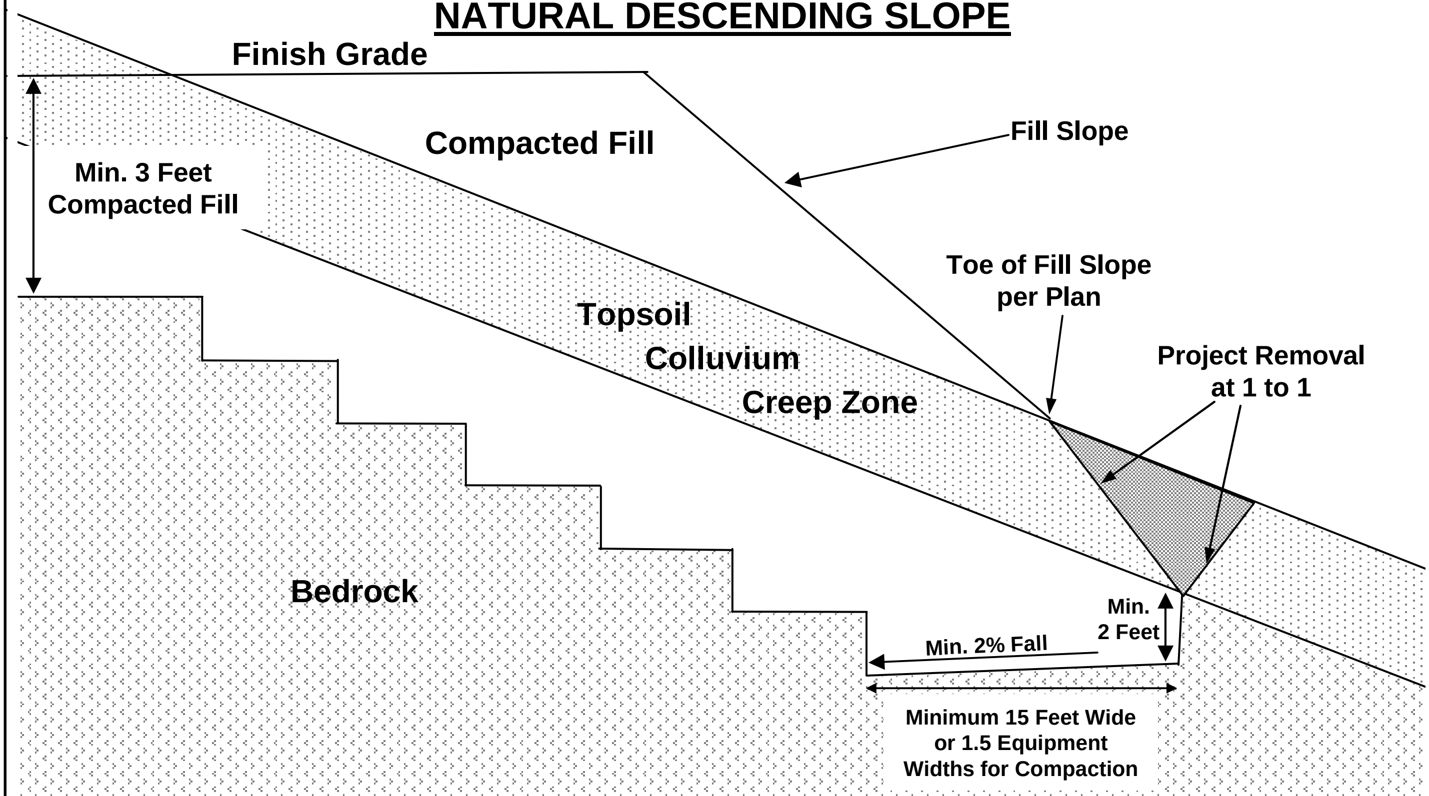
ALTERNATES



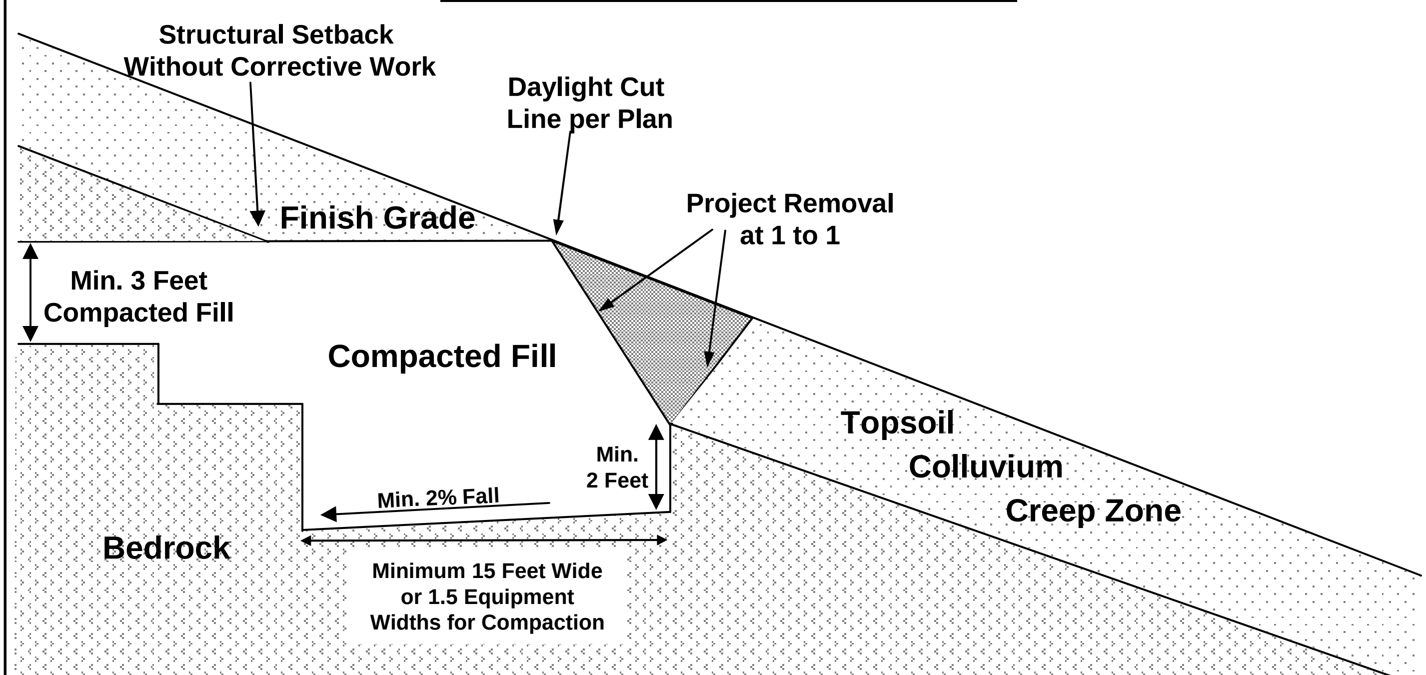


GEOTEK	1548 North Maple Street Corona, California 92880	TYPICAL CANYON CLEANOUT	STANDARD GRADING GUIDELINES PLATE F-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



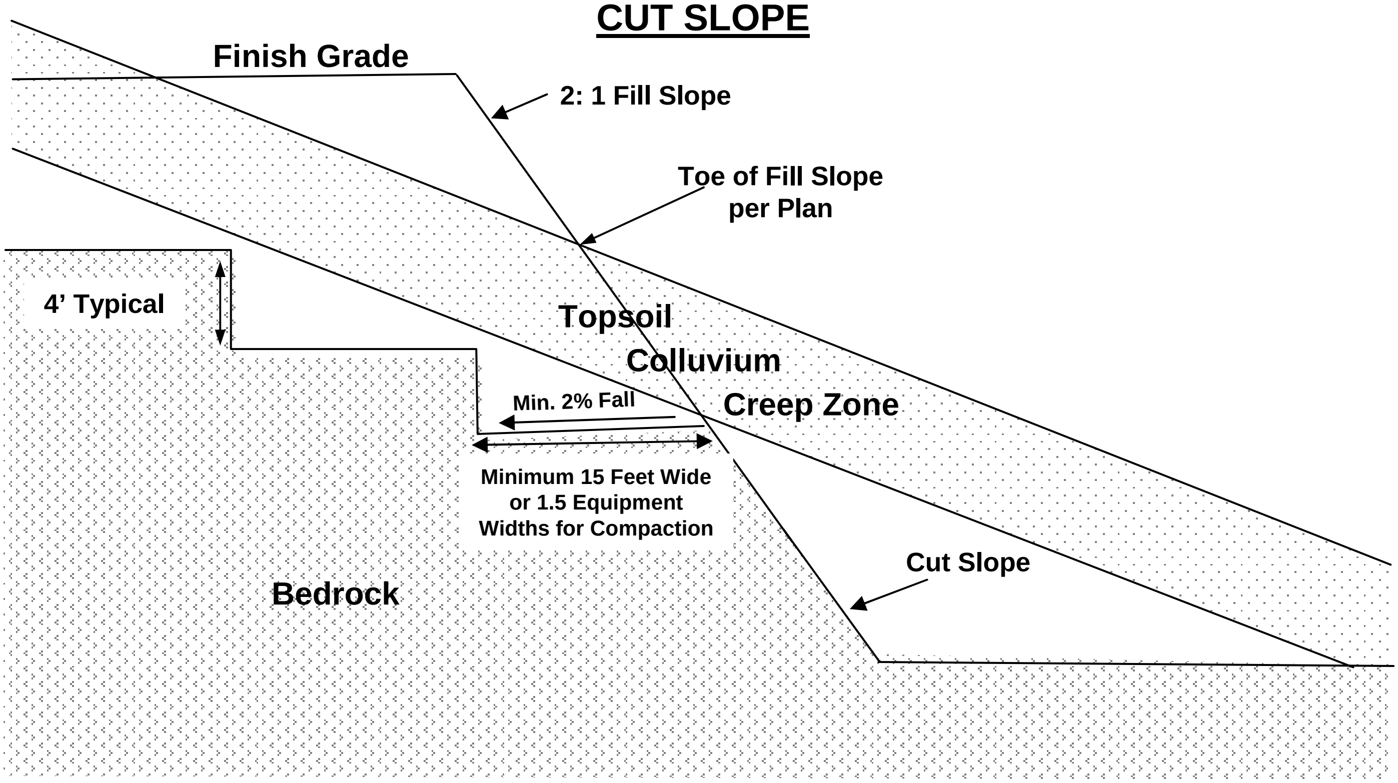
GEOTEK
1548 North Maple Street
Corona, California 92880

TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE F-2

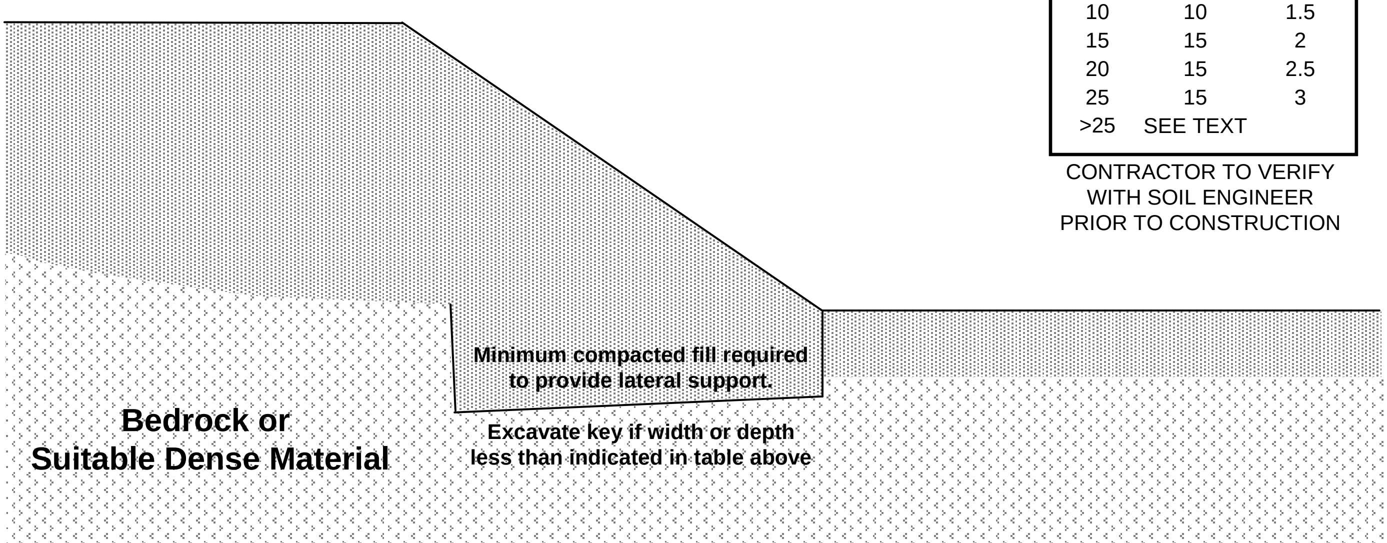
TYPICAL FILL SLOPE OVER CUT SLOPE



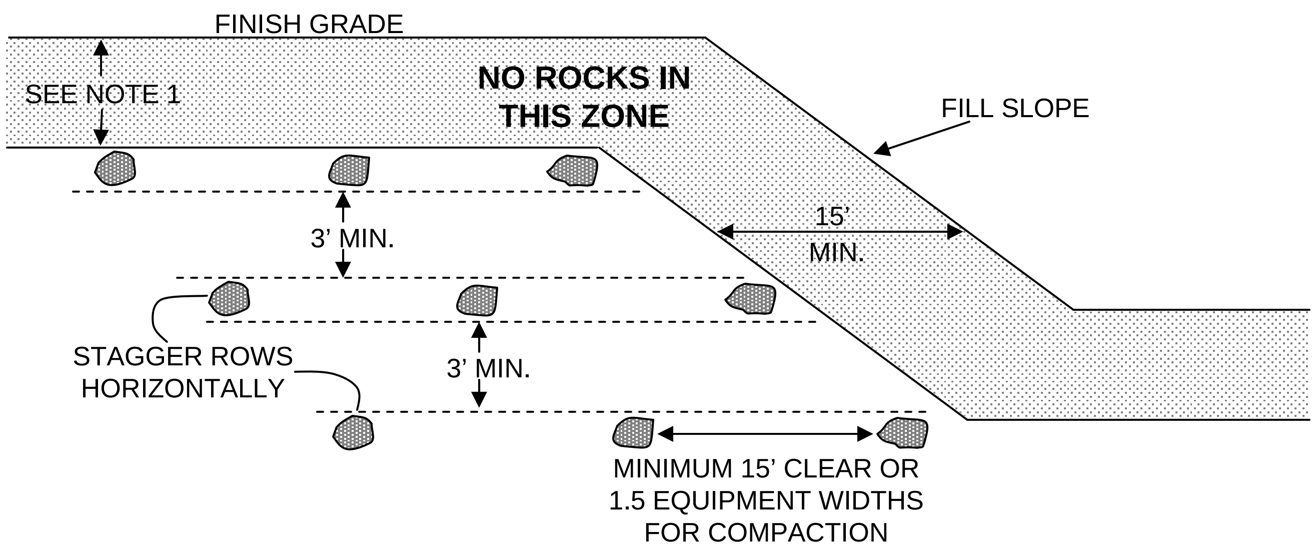
TYPICAL FILL SLOPE

SLOPE HEIGHT	MIN. KEY WIDTH	MIN. KEY DEPTH
5	7	1
10	10	1.5
15	15	2
20	15	2.5
25	15	3
>25	SEE TEXT	

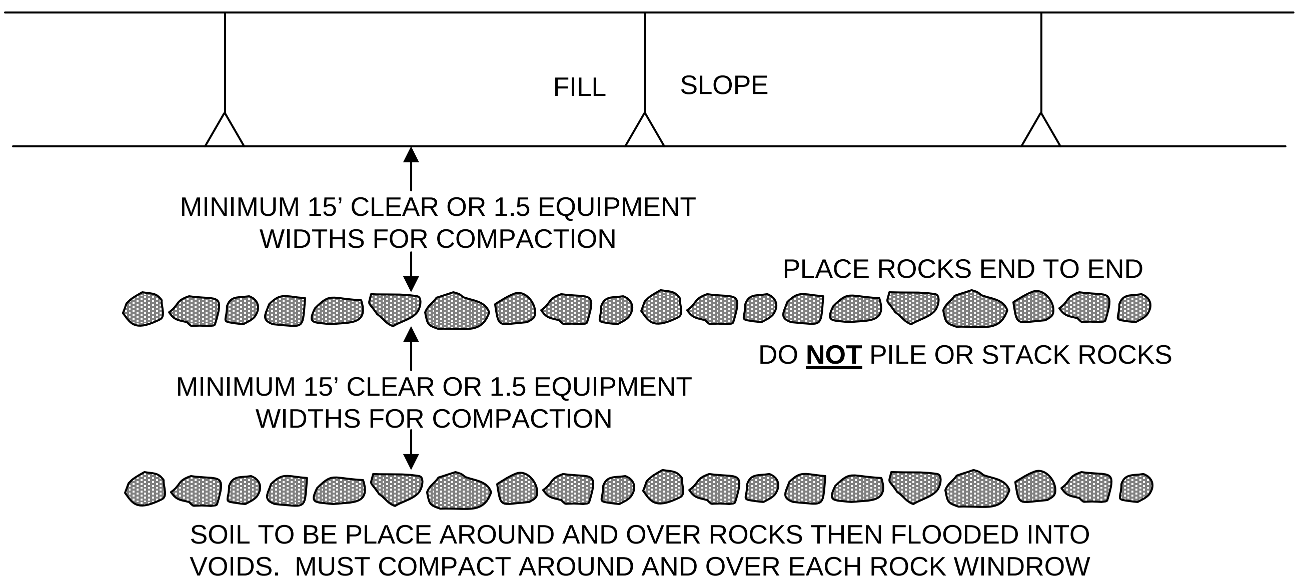
CONTRACTOR TO VERIFY WITH SOIL ENGINEER PRIOR TO CONSTRUCTION



CROSS SECTIONAL VIEW



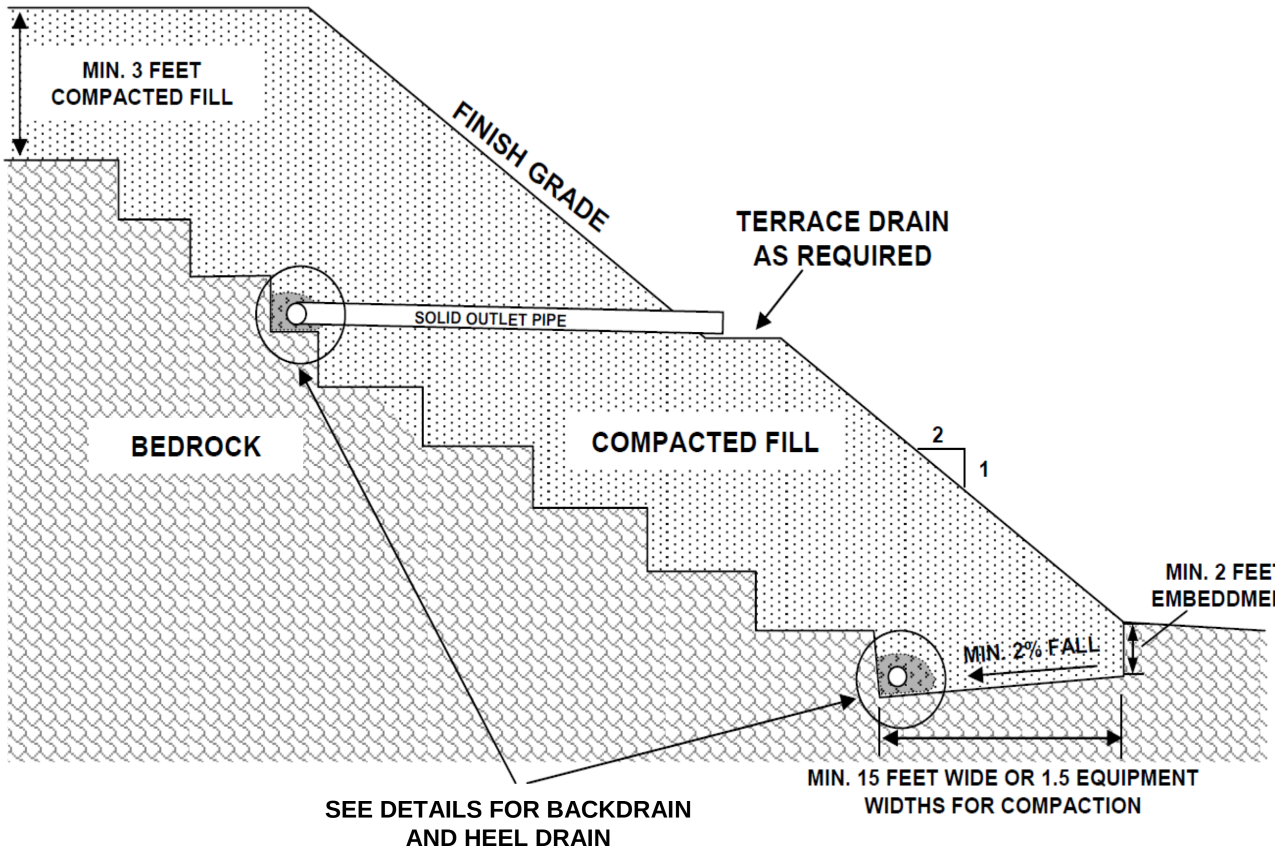
PLAN VIEW

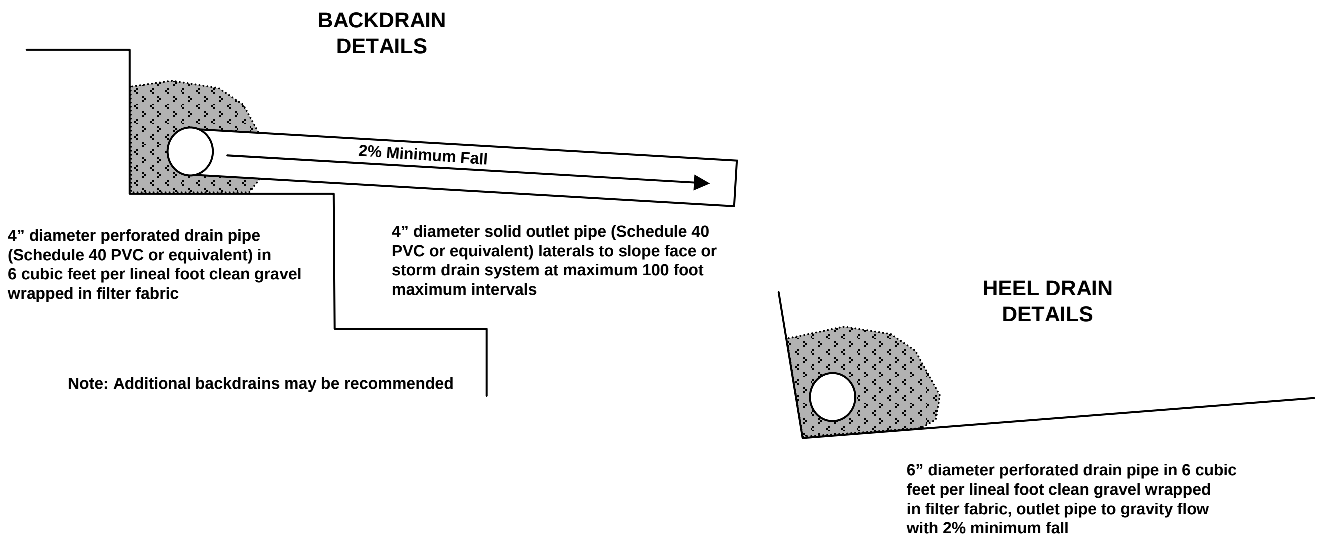


NOTES:

1) SOIL FILL OVER WINDROW SHOULD BE 7 FEET OR PER JURISDUICTIONAL STANDARDS AND SUFFICIENT FOR FUTURE EXCAVATIONS TO AVOID ROCKS
2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET IN DIAMETER
3) SOIL AROUND WINDROWS TO BE SANDY MATERIAL SUBJECT TO SOIL ENGINEER ACCEPTANCE
4) SPACING AND CLEARANCES MUST BE SUFFICIENT TO ALLOW FOR PROPER COMPACTION
5) INDIVDUAL LARGE ROCKS MAY BE BURIED IN PITS.

GEOTEK	1548 North Maple Street Corona, California 92880	ROCK BURIAL DETAILS	STANDARD GRADING GUIDELINES PLATE F-4





GEOTEK	1548 North Maple Street Corona, California 92880	TYPICAL BUTTRESS AND STABILIZATION FILL	STANDARD GRADING GUIDELINES PLATE F-5