GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE – KNOX VII

SEC Harley Knox Boulevard and Decker Road
Riverside County, California
For
Trammel Crow So. Cal. Development

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

September 14, 2020

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Trammell Crow So. Cal. Development
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

Attention: Mr. Neal Holdridge
Principal/Environmental Manager

Project No.: **20G183-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse – Knox VII
SEC Harley Knox Boulevard and Decker Road
Riverside County, California

Dear Mr. Holdridge:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- Throughout most of the site, the ground surface is immediately underlain by native older alluvium and/or existing undocumented fill. Tonalite bedrock is located beneath the older alluvium as shallow as 3 to 6± feet from the ground surface.
- The existing fill materials and older alluvial soils are not considered suitable for support of the proposed building based on their consolidation/collapse potential and varying densities and strengths.
- The borings, trenches, and previous seismic refraction surveys performed at the subject site generally indicate that the near-surface existing bedrock materials are rippable to the depths of 3 to 10± feet with a D-9 dozer. However, blasting is expected to be required where excavations extend to greater depths, where excavation/grading equipment other than a D-9 dozer is utilized, or within localized areas where very dense bedrock is encountered.

Site Preparation

- Initial site preparation should include stripping of any surficial vegetation. The surficial vegetation, trees, and any organic soils should be properly disposed of off-site.
- Remedial grading is recommended to be performed within the new building pad area. The existing soils within the building pad areas should be overexcavated to a depth of 3 feet below existing grade and to a depth of 5 feet below proposed pad grade, whichever is greater. All existing artificial fill materials and alluvium should also be removed from the new building pad area. The soils within the proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grades. These excavations should extend to a depth of 5 feet below foundation bearing grade in bedrock cut areas.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated, moisture conditioned, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Soils suitable to serve as the structural fill subgrade within the building area should possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slabs

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Reinforcement is not required for geotechnical conditions. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavements

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5	5½	6½
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 20P291, dated July 22, 2020. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southeast corner of Harley Knox Boulevard and the future continuation of Decker Road in an unincorporated portion of Riverside County near Perris, California. The site is bounded to the north by Harley Knox Boulevard, to the west by the Decker Road easement, to the south by a vacant lot, and to the east by an existing warehouse. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of five (5) contiguous parcels totaling 13.6± acres in size. Based on aerial photographs obtained from Google Earth and our site visitation, the site is currently vacant and undeveloped, with the exception of a few dirt access roads. The ground surface cover consists of exposed soil with sparse to moderate native grass and weed growth. Isolated areas of tonalitic bedrock outcrops are exposed throughout the site, with heavier concentrations occurring in the northeast and eastern portions of the site.

Detailed topographic information was not available at the time of this report. However, based on topographic information obtained from Google Earth, the overall site topography slopes gently to the east at a gradient of 3± percent.

3.2 Proposed Development

The most current preliminary site plan, prepared by HPA, Inc., was provided to our office by the client. The plan indicates that the new development will consist of one (1) new warehouse, 256,048± ft^2 in size, located in the north-central region of the subject site. Dock-high doors and a truck court will be constructed on the south side of the proposed building. The new building is expected to be surrounded by asphaltic concrete pavements in the parking and drive areas and Portland cement concrete pavements in the loading dock areas. Several landscaped planters and concrete flatwork are also expected to be included throughout the site.

Detailed structural information has not been provided. However, it is our understanding that the new building will be a single-story structure of tilt-up concrete construction, generally supported on conventional shallow foundations with a concrete slab-on-grade floor. The construction may include second floor mezzanine offices. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 5± feet are expected to be necessary to achieve the proposed site grades.

3.3 Previous Study

Southern California Geotechnical, Inc. (SCG) has reviewed a seismic line study performed at the project site, included in Appendix F of this report. This study is identified as follows:

> Seismic Refraction Survey, Proposed Muranaka Project, SE Corner of Harley Knox Boulevard and Decker Road, Perris Area, Riverside County, California, prepared by Terra Geoscience for Trammel Crow Company, Terra Geoscience Project No. 193303-1, dated November 1, 2019.

As part of this study, a total of eight (8) 125-foot-long seismic refraction survey lines (Seismic Lines S-1 through S-8) were performed. The approximate seismic line locations are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report. The geophysical survey identified three major subsurface layers with respect to the seismic velocities. The first (upper) layer, with velocities ranging from 1,472 to 2,650 feet/second, was considered to represent topsoil, colluvium, older alluvial sediments, and/or completely-weathered and fractured bedrock materials. The first layer extended to depths of ½ to 7½± feet below the existing site grades. The second (middle) layer, with velocities ranging from 3,507 to 6,245 feet/second, was considered to represent either weathered granodiorite or older alluvium which extended to depths of 9 to 37± feet. The third (lower) layer, with velocities ranging from 6,249 to 11,984 feet/second, was considered to represent relatively unweathered, granodiorite bedrock which was encountered at depths ranging from 9 to 37± feet, and extended to depths of at least 31 to 50± feet. The granodiorite bedrock, with velocities greater than 8,000 feet/second, was considered to be non-rippable based on the Caterpillar rippability chart. This study indicated that significant blasting should be anticipated where excavation into the third layer would be required to achieve the desired grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of four (4) borings advanced to depths of 8½ to 25± feet below the existing site grades and four (4) trenches excavated to depths of 3 to 11± feet. Some of the borings and trenches were terminated at depths shallower than proposed after encountering refusal on very dense bedrock. All of the borings and trenches were logged during the drilling and excavation by members of our staff.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. The trenches were excavated using a backhoe with a 36-inch-wide bucket. Representative bulk and undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings (identified as Boring Nos. B-1 through B-4) and trenches (identified as Trench Nos. T-1 through T-4) are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report. The Boring and Trench Logs, which illustrate the conditions encountered at the boring and trench locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at Boring No. B-1 and at Trench No. T-1, extending to depths of 1 to 1½± feet below the existing site grades. The fill soils generally consist of loose to medium dense silty fine to medium sands with varying coarse sand content. The fill soils possess a disturbed mottled appearance, resulting in their classification as artificial fill.

Older Alluvium

Older alluvium was encountered beneath the existing fill soils or at the ground surface at all of the boring and trench locations, extending to depths of 3 to 6± feet below the existing site grades. The older alluvium generally consists of medium dense to very dense clayey fine sands with varying medium to coarse sand and silt content, and silty fine sands with varying medium to coarse sand and clay content.

Bedrock

Val Verde Tonalite bedrock was encountered beneath the older alluvial soils at all of the boring and trench locations, extending to at least the maximum depth explored of 25± feet below the ground surface. The bedrock consists of very dense, light gray brown to gray brown fine to coarse grained tonalite. These materials are generally weathered and friable throughout the depths explored at the site. However, auger refusal conditions were encountered at depths of 8½ and 17± feet on very dense tonalite bedrock materials at Boring Nos. B-2 and B-4, respectively. The backhoe encountered refusal conditions at depths ranging from 3 to 11± feet

Groundwater

Groundwater was not encountered at any of the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 25± feet below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is the Western Municipal Water District and the San Bernardino Valley Water Conservation District Cooperative Well Measuring Program. High water level from the nearest well is included below:

State Well ID	Approximate Distance from Subject Site	High Water Level MSL (feet)
03S/04W-36K/Q	< 3,600 feet	1,430.20

Based on topographic information obtained from Google Earth, the elevation at the subject site ranges from 1,558± feet msl in the northeastern area of the site to 1,597± feet msl in the southwestern region of the site. The elevation of the high-water level in the well is 1,430± feet msl.

Recent water level data was obtained from the California State Water Resources Control Board, GeoTracker, website, http://geotracker.waterboards.ca.gov/. A series of nearby monitoring wells (identified as MW-1 through MW-5) on record are located approximately 2,500-3,000 feet north of the site at elevations ranging from 1,535 feet msl to 1,560 feet msl. Water level readings within these monitoring wells indicate high groundwater levels of 6.29± to 25.8± feet below the ground surface in May 2011.

Based on this well data, the depth of the high water level at the subject site, measured from the lowest elevation at the subject site, is 30± feet below the existing site grades. Therefore, a groundwater depth of 30± feet is considered to be conservative with respect to the more recent site conditions.

4.3 Geologic Conditions

Regional geologic conditions were obtained from the Geologic Map of the Steele Peak 7.5' Quadrangle, Riverside County, California, by Douglas M. Morton published by the California Department of Mines and Geology and United States Air Force, 2001. This map indicates that the majority of the site is underlain by Cretaceous Val Verde Formation tonalite (Map Symbol Kvt). A small portion of the eastern area of the site is underlain by older alluvial deposits (Map Symbol Qvof). The Val Verde Formation is described as gray, weathered, relatively homogeneous, massive, medium- to coarse- grained tonalite. A portion of this map indicating the location of the subject site, is included as Plate 3 in Appendix A of this report.

Bedrock materials were encountered at all of the boring and trench locations extending from beneath the older alluvial soils to depths of at least 25± feet. Based on the bedrock encountered at the boring and trench locations, it is our opinion that the near-surface older alluvium throughout the site is underlain by tonalite bedrock of the Val Verde Formation (Map Symbol Kvt). The bedrock is weathered, friable, and consists of fine- to coarse- grained tonalite.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring and Trench Logs.

Consolidation

Selected soil samples were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-5 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

A representative sample of the near-surface soil was submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.002	Not Applicable (S0)

Corrosivity Testing

One representative bulk sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-1 @ 0 to 5 feet	3,520	7.9	9.4	11

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structures should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, our review of the Riverside County RCIT GIS website that the site is not located within a Riverside County fault zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

Based on standards in place at the time of this report, the proposed development is expected to be designed in accordance with the requirements of the 2019 edition of the California Building Code (CBC), which was adopted on January 1, 2020. The 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2019 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2019 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

As a part of this investigation, SCG subcontracted a licensed geophysicist to perform a shear wave survey within the proposed building area in order to determine the Site Classification (per CBC-2019 1613A.2.2 & ASCE 7-16 Ch. 20) of the project site. A letter reporting the results of this survey, including the average shear wave velocity of the soil and bedrock materials present in the upper 100± feet is included in Appendix E of this report. Based on the results of the shear wave survey, the average sear wave velocity for these materials is 2,763.8 feet per second. In accordance with the referenced sections of the 2019 CBC and ASCE 7-16, sites with average shear wave velocities ranging between 2,500 and 5,000 feet per second may be classified as Site Class B.

It should be noted that Site Class B may only be used at sites with bedrock located no more than 10 feet deeper than the bottom of the footings. Section 20.1 of ASCE 7-16 states that "Site Classes A and B shall not be assigned to a site if there is more than 10 feet of soil between the rock surface and the bottom of the spread footing or mat foundation." Val Verde Tonalite bedrock was encountered at all of the boring and trench locations as shallow as 3 to 6± feet from the existing ground surface. The grading plans for the proposed development were not available at the time of this report. However, based on the existing site grades, and assuming a relatively balanced site, we do not expect that the proposed building pad will be raised more than 5 feet from the existing site grades. Based on the results of the shear wave survey and the presence of relatively shallow bedrock at all of the boring and trench locations, the subject site has been classified as Site Class B in accordance with Section 20 of ASCE 7-16. If the proposed grading at the subject site will result in more than 10 feet of soil between the bottom of the footings and bedrock, it may be necessary to reclassify this site as Site Class C.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.500
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.574
Site Class	---	B
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.500
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.574
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.000
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.383

Liquefaction

Liquefaction is the loss of the strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

Review of the Riverside County RCIT GIS website indicates that the northeastern and northwestern areas of the subject site are located within a zone of moderate liquefaction susceptibility. However, based on the subsurface exploration performed for this project, the entire site is underlain by very dense tonalite bedrock. Based on the shallow bedrock encountered at the boring and trench locations within the subject site, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered at one of the boring and one of the trench locations within the proposed building area, extending to depths of 1 to 1½± feet. No documentation regarding the placement or compaction of these fill soils has been provided nor is expected to be available. The existing fill soils, in their present condition, are not considered suitable to support the foundations loads of the new structure. The subsurface conditions encountered at the remaining boring and trench locations generally consist of older alluvium underlain by tonalite bedrock. The older alluvial soils possess varying strengths and have unfavorable consolidation and collapse characteristics. Therefore, the older alluvial soils will require removal and replacement in order to support the proposed improvements. It is expected that the proposed finish pad elevations will require cuts that will expose dense to very dense tonalite bedrock at pad and/or footing grade. Overexcavation of the bedrock materials is considered warranted in order to mitigate the bedrock/fill transitions and to produce a building pad that will facilitate future foundation and utility construction.

Of primary concern in the development of this site is the presence of tonalite bedrock. Based on the velocities of the soil and bedrock determined by the seismic refraction profiling at the previous seismic lines, the upper 3 to 10± feet of the site appears rippable with heavy construction equipment such as a Caterpillar D-9. Below the older alluvium and weathered bedrock layers, the velocities increase to over 4,000 feet per second. Although the Caterpillar rippability charts indicate that bedrock layers with velocities ranging from 6,800 to 8,000 feet per second are considered rippable, different excavating equipment such as scrapers, track-mounted excavators, and loaders may not correlate well with these velocity ranges. Therefore, bedrock with velocities between 4,000 to 7,000 feet per second are considered to be moderately- to non-rippable. The deeper, less weathered or crystalline bedrock is not considered rippable with conventional construction equipment and will require blasting. The table presented below depicts the expected depths to non-rippable material at each seismic line location:

Seismic Line	Depth to Non-Rippable Material (feet)
S-1	3 to 8
S-2	3 to 8
S-3	5 to 15
S-4	5 to 12
S-5	8 to 12
S-6	4 to 25
S-7	4 to 25
S-8	10 to 35

Based on the assumed grades that will be required within the proposed building pad, significant design consideration should be given in order to help mitigate the cut/fill transitions within the proposed building pad area. Recommendations pertaining to this consideration are included in later sections of this report.

Settlement

The recommended remedial grading will remove all of the existing undocumented fill soils and older alluvial soils, as well as a portion of the bedrock, and replace these materials as compacted fill soils. The underlying bedrock is not considered to be susceptible to settlement from the foundations of the proposed structure. Provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed structure is expected to be within tolerable limits.

Expansion

The near-surface soils consist of silty sands with no appreciable clay content. These materials have been visually classified as very low to non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site. However, it is recommended that expansion index testing be performed at the completion of rough grading in order to confirm the expansion potential of the near-surface soils at this site.

Slope Stability

No evidence of landslides or deep-seated slope instability was noted during our investigation. However, the loose granular soils on sloping ground surfaces could be prone to surficial failures.

Newly constructed fill slopes, comprised of properly compacted engineered fill, at inclinations of 2h:1v will possess adequate gross stability. In addition, cut bedrock slopes within inclinations of 2h:1v are expected to possess adequate stability. Further evaluation of the tonalite bedrock will be necessary at the time of site grading to evaluate the appropriate maximum inclinations.

Cut slopes excavated within the existing granular alluvial soils may be subject to surficial instability due to the lack of cohesion within these materials. Therefore, stability fills may be required within these areas. This condition may affect the proposed cut slopes at the site. The need for stability fills should be determined by SCG as part of the future detailed grading plan review.

Soluble Sulfates

The result of the soluble sulfate testing indicates that the selected sample of the on-site soils corresponds to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the on-site soils possesses a saturated resistivity value of 3,520 ohm-cm, and a pH value of 7.9. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Sulfides, and redox potential are factors that are also used in the evaluation procedure. We have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH, and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are not considered to be corrosive to ductile iron pipe. **However, SCG does not practice in the area of corrosion engineering. Therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

A relatively low concentration (9.4 mg/kg) of chlorides was detected in the sample submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Shrinkage/Subsidence

Removal and recompaction of the near-surface fill and older alluvial soils is estimated to result in an average shrinkage of 5 to 15 percent. Where bedrock materials are excavated and replaced as fill, bulking of 5 to 10 percent should be expected. It should be noted that these shrinkage and bulking estimates are based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

No significant subsidence is expected to occur in excavations that are underlain by bedrock materials.

These estimates are based on previous experience with nearby projects and the subsurface conditions encountered at the boring and trench locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Foundation and Grading Plan Review

Foundation and grading plans were not available at the time of this report. It is therefore recommended that we be provided with copies of all future foundation and grading plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring and trench locations, and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping

Initial site stripping should include removal of any surficial vegetation and topsoil. This should include any weeds, grasses, shrubs, and trees. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove all the existing artificial fill and older alluvial soils. Based on conditions encountered at the boring and trench locations, this will require excavation to depths of 3 to 6± feet. The existing materials within the proposed building pad areas are also recommended to be overexcavated to a depth of at least 5 feet below proposed building pad subgrade elevation and to a depth of at least 3 feet below existing grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade (5 feet in areas where cuts expose bedrock).

The overexcavation should include the entire pad area. The intent of the grading recommendations is to overexcavate the bedrock and replace it as a compacted fill to a depth of at least 5 feet below footing grade in cut areas and to overexcavate all the older alluvial soils prior to fill placement in fill areas. This will limit differential settlements and facilitate future construction activities with respect to excavation of shallow foundations and utilities in cut areas.

Following completion of the overexcavation, the subgrade should be evaluated by the geotechnical engineer to verify its suitability to serve as the structural fill subgrade. Some localized areas of deeper excavation may be required if loose, porous, or low-density materials are encountered at the base of the overexcavation. Materials suitable to serve as the structural fill subgrade within the building area should consist of bedrock or soils which possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. These materials should be moisture conditioned to 0 to 4 percent above optimum moisture content prior to placement of any new fill soils. The previously excavated soils may then be replaced as compacted structural fill.

Deep Fill Areas

In order to reduce the settlement potential of the newly placed fill soils to acceptable levels and avoid excessive differential settlements, fill soils placed at depths greater than 10 feet below proposed building pad grades should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.

Settlement of Deep Fill Soils

Additional consolidation should be expected for fill soils placed at depths greater than 10 feet below proposed building pad grades. The primary settlement associated with these fill soils is expected to occur relatively quickly due to the generally granular nature of the on-site soils. Minor amounts of additional settlement may occur due to secondary consolidation effects. The extent of secondary consolidation is difficult to assess precisely, and will be reduced by the proposed mitigation measures recommended herein, but may be in the range of 0.2 to 0.4 percent of the fill thickness. Based on the expected differential fill thickness that will exist across the building footprints, the structural design will need to consider the distortions that could be caused by the secondary consolidation of the fill soils. Provided that the grading and foundation design recommendations presented in this report are implemented, these settlements are expected to be within the structural tolerances of the proposed buildings.

Cut/Fill Transitions

As discussed above, the proposed grading may result in bedrock/fill transitions within the proposed building area. It is recommended that remedial grading be performed in order to remove and replace a portion of the bedrock as compacted structural fill. This grading is considered warranted, in order to soften the transition from the fill soils to the bedrock, thereby reducing the potential for excessive future differential settlements. Following the review of the grading and foundation plans, additional geotechnical recommendations including, but not limited to, additional overexcavation or increased compaction standards to help mitigate the effects of differential settlement may be required.

Treatment of Existing Soils: Cut and Fill Slopes

New cut and fill slopes will likely be constructed within and around the perimeter of the project. All slopes should be at an inclination of 2h:1v. A keyway should be excavated at the toe of new fill slopes which are not located in fill areas. The keyway should be at least 15 feet wide and 3 feet deep. The recommended width of the keyway is based on 1.5 times the width of typical grading equipment. If smaller equipment is utilized, a smaller keyway may be suitable, at the discretion of the geotechnical engineer. The base of the keyway should slope at least 1 foot downward into the slope. Following completion of the keyway cut, the subgrade soils should be evaluated by the geotechnical engineer to verify that the keyway is founded into competent materials. The resulting subgrade soils should then be scarified to a depth of 10 to 12 inches, moisture conditioned to 0 to 4 percent above optimum moisture content and recompacted. During construction of the new fill slope, the existing slope should be benched in accordance with the detail presented on Plate D-4. Benches less than 4 feet in height may be used at the discretion of the geotechnical engineer.

Cut slopes in bedrock may be cut to grade, or blasted, undercut and replaced as stability fills. Stability fills for cut slopes will provide a more uniform appearance and allow landscaping on the slope. Should a stability fill for cut slope be necessary, the recommendations for the stability fill will be the same as the recommendations for the fill slopes, mentioned above.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 0 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent structures during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Flatwork and Parking Areas

Based on economic considerations, overexcavation of the existing near-surface existing soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new flatwork, parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not mitigate the extent of undocumented fill or compressible/collapsible native alluvium in the flatwork, parking and drive areas. As such, some settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork, parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent of the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. **Some sorting or crushing will likely be required to utilize fill materials derived from excavated bedrock**.

- All grading and fill placement activities should be completed in accordance with the requirements of the current CBC and the grading code of the county of Riverside.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density, unless noted otherwise. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Selective Grading and Oversized Material Placement

If blasting operations are required, significant oversize material may be generated. The presence of particles greater than 6 inches in diameter within the upper 1 foot of the building pad subgrade will impact the utility and foundation excavations. Depending on the depths of fills required within the pad areas, it may be feasible to sort the on-site soils by placing the materials greater than 6 inches in diameter within the lower depths of the fills, and limiting the upper 1 foot of soils to materials less than 6 inches in size.

Large cobbles and boulders (in excess of 12± inches in size) are expected to be encountered throughout the areas where bedrock outcrops are present within the property. In addition, "floaters" will likely be encountered within the weathered tonalite and granitic bedrock materials in other areas of the site. It will likely be necessary to move these larger rocks individually and place them as oversize materials in accordance with the grading guide specifications, enclosed in Appendix D of this report. Alternatively, the oversized materials could be disposed of off-site.

It is recommended that all materials greater than 12 inches in size be excluded from fills that are within 10 feet of proposed finished pad grade or within 3 feet of the proposed finish grade in the parking lot areas. Materials greater than 12 inches in size can be crushed, disposed of off-site or placed in rock blankets. Particles up to 3 feet in size may be placed in rock blankets, located at least 10 feet below finished pad grade and at least 3 feet below the deepest anticipated utility in the disposal area. The rock blankets should be covered with a free-draining granular material (sand), which should then be jetted in-place with water. The placement of sand and the jetting should continue until the oversized materials have been completely covered. The grading contractor must take special care to place fill material completely around all oversized particles. The areas around and above the rock blankets should then be backfilled with compacted structural fill.

The placement of oversized materials and the procedures used to backfill around these materials must be witnessed, approved, and documented by the geotechnical engineer.

Imported Structural Fill

All imported structural fill should consist of very low to non-expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the county of Riverside. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches. Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands. These materials will be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing unless founded in unweathered tonalite bedrock. Temporary excavation slopes should be made no steeper than 1h:1v in unweathered or slightly weathered tonalite bedrock. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

As discussed previously, most areas of the subject site are underlain at a shallow depth by tonalite bedrock. Results of detailed subsurface exploration as well as the previous seismic refraction survey indicate that portions of the near-surface materials will likely be rippable using conventional grading equipment. However, marginal to non-rippable bedrock was also encountered. **In all of these non-rippable and marginally rippable areas, increased grading effort and/or specialized grading techniques will likely be required.**

Groundwater

The static groundwater table is considered to exist at a depth greater than 25± feet or more below the existing grades. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by newly placed structural fill soils, extending 3 to 5 feet below foundation bearing grade, which are underlain by dense to very dense tonalite bedrock. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Conventional Spread Footing Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).
- Minimum foundation embedment: 12 inches into newly placed structural fill soils, and at least 24 inches below adjacent exterior grade.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by one-third when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill or suitable native alluvium (where reduced bearing pressures are utilized), with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, which is underlain by dense to very dense tonalite bedrock. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: Reinforcement is not required for geotechnical conditions. However, slab reinforcement may be required for structural design considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as 15 mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

- The floor slab should be structurally connected to the foundations as detailed by the structural engineer.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades and in loading docks. Retaining walls are also expected within the truck dock areas of the proposed buildings. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the trench and boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. These near-surface soils generally consist of silty fine to medium sands and weathered tonalite bedrock. The sandy older alluvium and/or recompacted bedrock materials are expected to possess friction angles of at least 32 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density. A friction angle of 32 degrees is considered representative of the on-site soil and rock materials.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Silty Sands
Internal Friction Angle (ϕ)		32°
Unit Weight		135 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	At-Rest Condition (level backfill)	63 lbs/ft^3
	Active Condition (2h:1v backfill)	63 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2019 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 3 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back-wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1-foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1-foot thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a pocket of gravel, 2± cubic feet in size, surrounded by a geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.8 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either

PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils and/or bedrock-derived fill materials. The near-surface soils generally consist of silty sands. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 40 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 40. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Parking Stalls (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within Portland cement concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5	5½	6½
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. Any reinforcement within the PCC pavements should be determined by the project structural engineer.

7.0 GENERAL COMMENTS

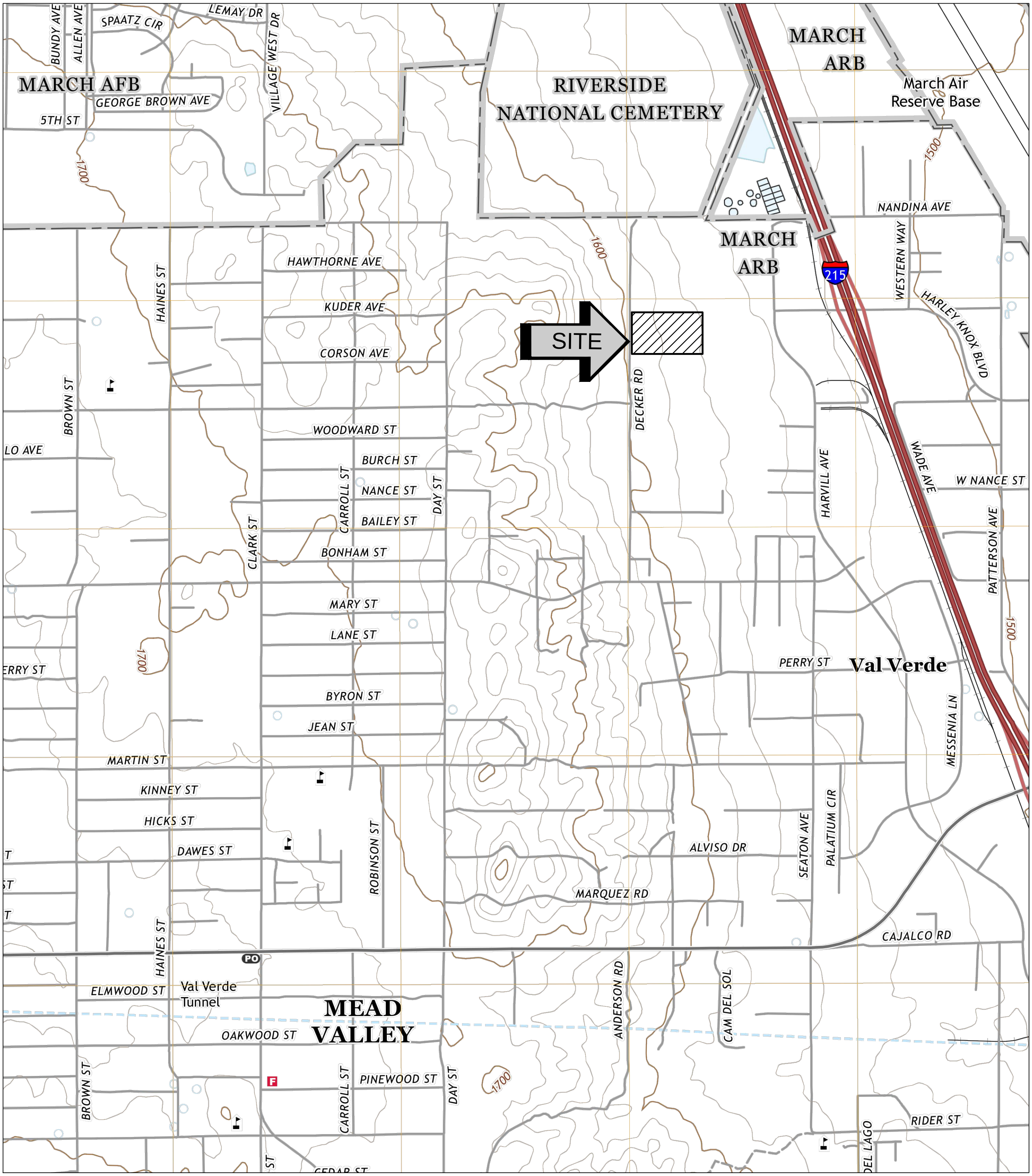
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE STEELE PEAK QUADRANGLE, RIVERSIDE, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED WAREHOUSE - KNOX VII

RIVERSIDE COUNTY, CALIFORNIA

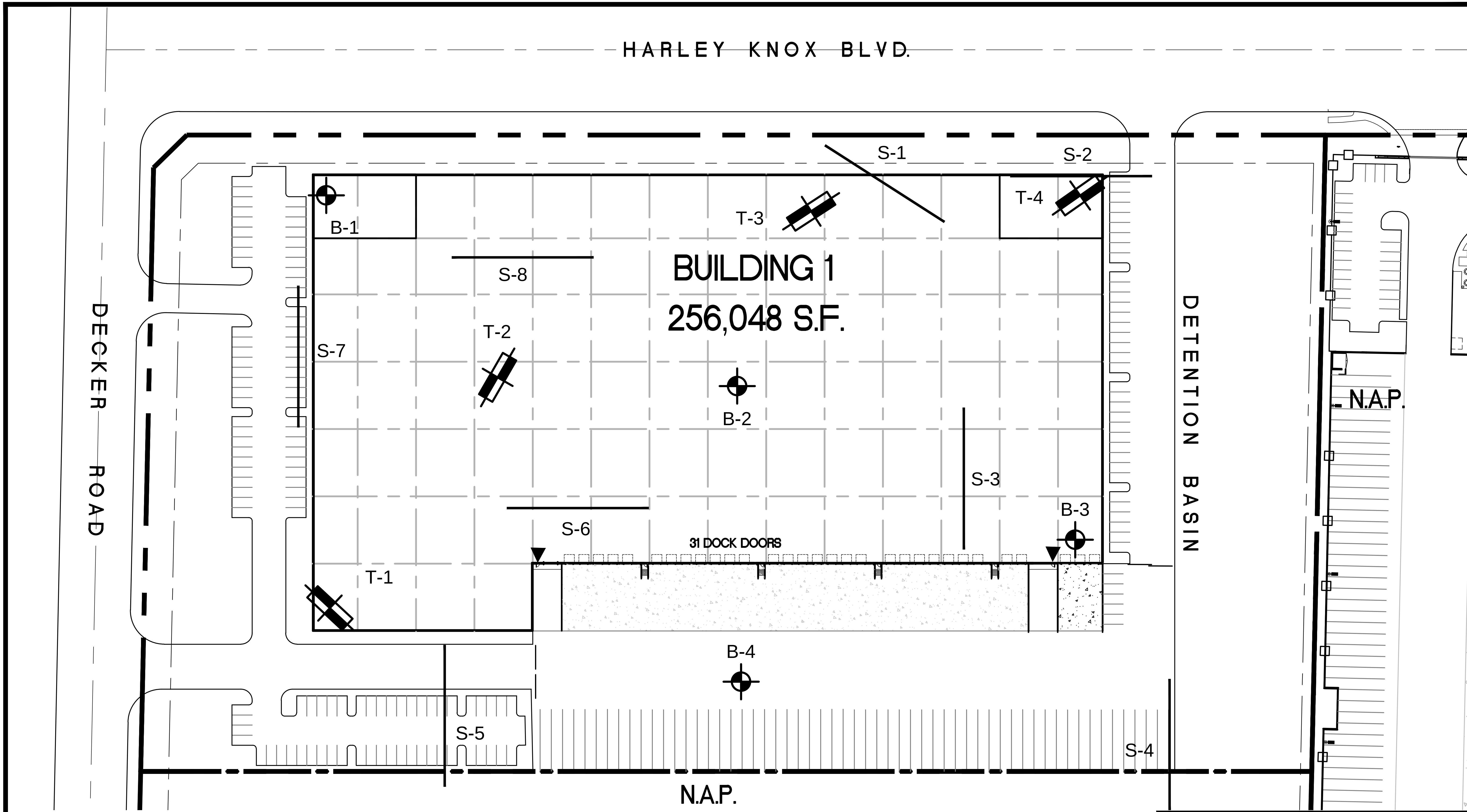
SCALE: 1" = 2000'

DRAWN: JLL
CHKD: GKM

SCG PROJECT
20G183-1

PLATE 1

SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE TRENCH LOCATION

PREVIOUS SEISMIC LINE LOCATION PERFORMED BY TERRA GEOSCIENCES, PROJECT NO. 193303-1

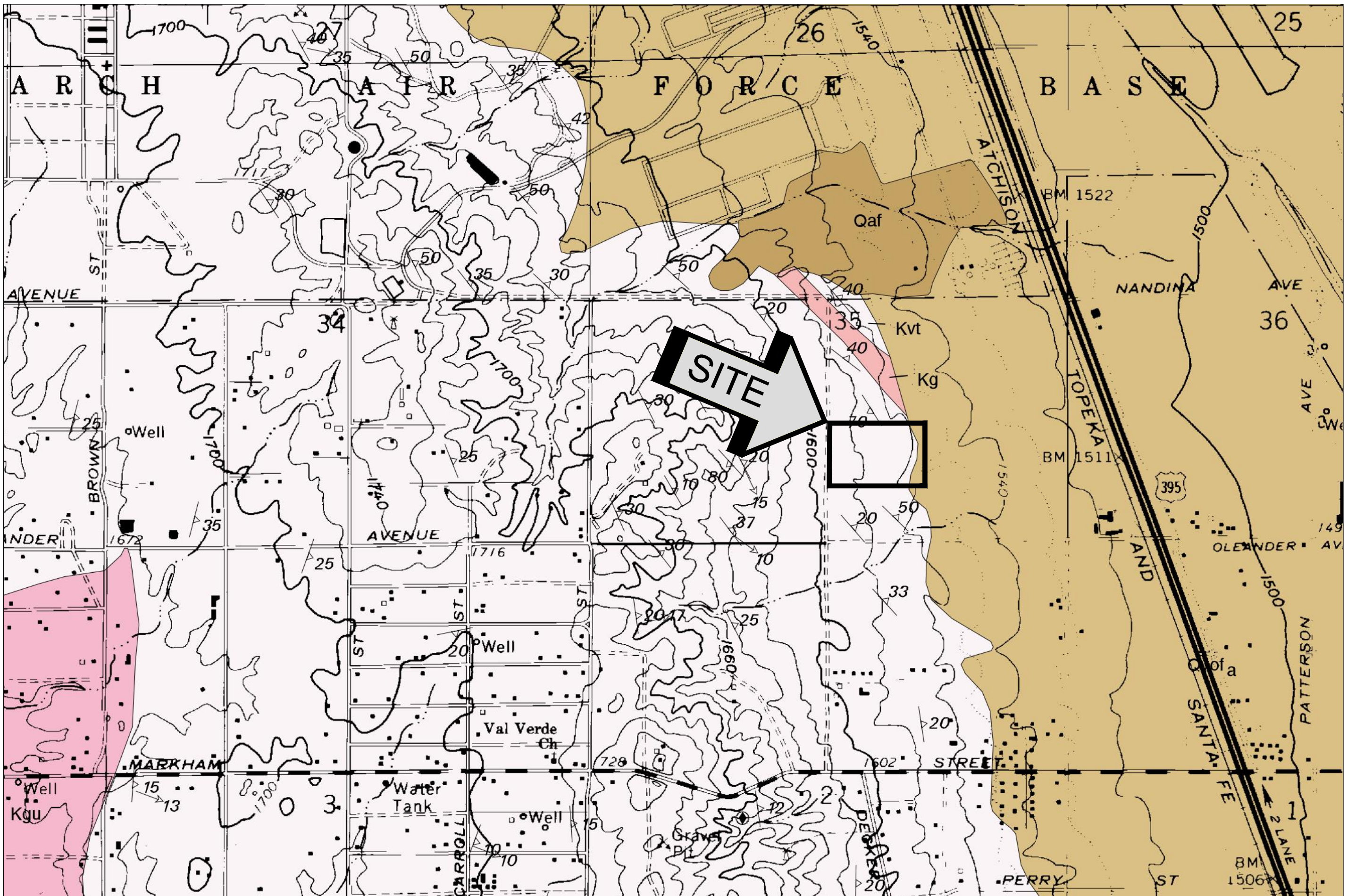
NORTH
SoCalGeo

NOTE: SITE PLAN PREPARED BY HPA, INC.

BORING AND TRENCH LOCATION PLAN
PROPOSED WAREHOUSE - KNOX VII
RIVERSIDE COUNTY, CALIFORNIA

SCALE: 1" = 80'
DRAWN: JLL
CHKD: GKM
SCG PROJECT 20G183-1
PLATE 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



VERY YOUNG SURFICIAL DEPOSITS—Sediment recently transported and deposited in channels and washes, on surfaces of alluvial fans and alluvial plains, and on hillslopes. Soil-profile development is non-existant. Includes:

Qaf — **Artificial fill (late Holocene)**—Deposits of fill resulting from human construction; restricted to construction activities on March Air Force Base in northeastern part of quadrangle and Colorado River aqueduct construction near Cajalco Road.

YOUNG SURFICIAL DEPOSITS—Sedimentary units that are slightly consolidated to cemented and slightly to moderately dissected. Alluvial fan deposits (Qyf series) typically have high coarse:fine clast ratios. Younger surficial units have upper surfaces that are capped by slight to moderately developed pedogenic-soil profiles (A/C to A/AC/$B_{cambric}C_{ox}$ profiles). Includes:

Qyw — **Young alluvial wash deposits (Holocene and late Pleistocene)**—Sand and gravelly sand deposits; unconsolidated. Restricted to drainage roughly followed by Cajalco Road

Qyf — **Young alluvial fan deposits (Holocene and late Pleistocene)**—Gray-hued arkosic, sandy and gravel-sand deposits derived from local Peninsular Ranges batholith granitic bodies. Limited distribution along eastern and western edges of quadrangle

Qya — **Young axial channel deposits (Holocene and late Pleistocene)**—Gray, unconsolidated alluvium consisting of medium- to fine-grained sand and lesser gravel and silt. Found 2 km southwest of Perris, and in western part of quadrangle, east of Gavilan Road

VERY OLD SURFICIAL DEPOSITS—Sediments that are slightly to well consolidated to indurated, and moderately to well dissected. Upper surfaces are capped by moderate to well developed pedogenic soils (A/AB/B/C_{ox} profiles having Bt horizons as much as 2 to 3 m thick and maximum hues in the range 7.5YR 6/4 and 4/4 to 2.5YR 5/6)

Qvof — **Very old alluvial fan deposits (early Pleistocene)**—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Covers large areas adjacent to U.S. Highway 215 in northeastern part of quadrangle and flanking drainage followed by Cajalco Road

Rocks of the Peninsular Ranges batholith

Val Verde pluton (Cretaceous)—Relatively uniform pluton composed of biotite-hornblende tonalite. Termed Perris quartz diorite by Dudley (1935), Val Verde tonalite by Osborn (1939), and included within Bonsall tonalite by Larsen (1948). Name Val Verde adopted by Morton (1999) based on detailed study of Osborn (1939) near Val Verde, a former settlement and railway siding midway between Perris and Riverside. Apparently steep-walled Val Verde pluton is eroded to mid-pluton level. Emplacement age of the pluton is 105.7 Ma_{id}. $^{40}Ar/^{39}Ar$ age of hornblende is 100 Ma, biotite 95 Ma and potassium feldspar 88.5 Ma. Includes:

Kvt — **Val Verde tonalite**—Gray-weathering, relatively homogeneous, massive- to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite; principal rock type of Val Verde pluton. Contains subequal biotite and hornblende, quartz and plagioclase. Potassium feldspar generally less than two percent of rock. Where present, foliation typically strikes northwest and dips moderately to steeply northeast. Northern part of pluton contains younger, intermittently developed, northeast-striking foliation. In central part of pluton, tonalite is mostly massive, and contains few segregational masses of mesocratic to melanocratic tonalite. Elliptical- to pancake-shaped, meso-to melanocratic inclusions are common

Kg — **Granitic dikes. (Cretaceous)**—Includes texturally diverse group of leucocratic granitic dikes composed mainly of quartz and alkali feldspars. Dikes range in thickness from few centimeters to over a meter and are up to several hundred meters in length. Most are tabular; some are texturally and compositionally unzoned, irregular-shaped bodies. Some dike rock has a foliated or gneissoid fabric. Textures are mostly coarse grained and equigranular granitic but range from aplitic to pegmatitic. Accessory minerals include biotite, muscovite, and garnet

SOURCE: "GEOLOGIC MAP OF THE STEELE PEAK 7.5' QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA" MORTON, 2001

GEOLOGIC MAP

PROPOSED WAREHOUSE - KNOX VII

RIVERSIDE COUNTY, CALIFORNIA

SCALE: 1" = 2000'

DRAWN: DRK
CHKD: GKM

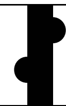
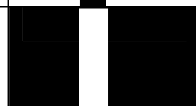
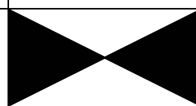
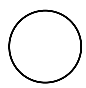
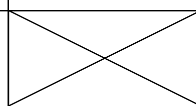
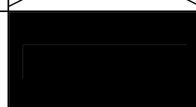
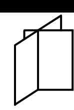
SCG PROJECT
20G183-1

PLATE 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH:	Distance in feet below the ground surface.
SAMPLE:	Sample Type as depicted above.
BLOW COUNT:	Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.
POCKET PEN.:	Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.
GRAPHIC LOG:	Graphic Soil Symbol as depicted on the following page.
DRY DENSITY:	Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.
MOISTURE CONTENT:	Moisture content of a soil sample, expressed as a percentage of the dry weight.
LIQUID LIMIT:	The moisture content above which a soil behaves as a liquid.
PLASTIC LIMIT:	The moisture content above which a soil behaves as a plastic.
PASSING #200 SIEVE:	The percentage of the sample finer than the #200 standard sieve.
UNCONFINED SHEAR:	The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, California

DRILLING DATE: 8/25/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 23 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Light Gray Brown Silty fine to coarse Sand, trace fine root fibers, medium dense-damp							
		43				111	3					
					OLDER ALLUVIUM: Red Brown Clayey fine Sand, trace medium to coarse Sand, little Silt, micaceous, medium dense-damp	123	4					
		83			Light Brown to Brown Silty fine to medium Sand, trace coarse Sand, trace Clay, micaceous, very dense-damp to moist	130	7					
5		50/4"				119	3					
					VAL VERDE TONALITE (Kvt): Light Gray Brown to Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, slightly to highly weathered, friable, very dense-dry to damp	114	5					
		50/5"				115	2					
		50/4"					3					Disturbed Sample
10												
		50/5"					2					
15												
		50/4"										No Sample Recovery
20												
		50/1"					1					
25												

Boring Terminated at 25'

TBL 20G183-1.GPJ SOCALGEO.GDT 9/14/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, California

DRILLING DATE: 8/25/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Brown Clayey fine to medium Sand, trace coarse Sand, little Silt, very dense-damp							
	X	78/11"					7					
					VAL VERDE TONALITE (Kvt): Light Gray Brown to Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, slightly to highly weathered, friable, very dense-damp to moist							
	X	91					7					
5												
	X	50/6"					3					
					Refusal at 8.5' due to very dense bedrock							

TBL 20G183-1.GPJ SOCALGEO.GDT 9/14/20

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, California

DRILLING DATE: 8/25/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

SURFACE ELEVATION: MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				OLDER ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, medium dense-dry to damp							
	X	16			106	2					
				Light Brown Silty fine to coarse Sand, very dense-damp							
	X	50/4"			106	5					
				VAL VERDE TONALITE (Kvt): Light Gray Brown to Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, slightly to highly weathered, friable, very dense-dry to damp							
5	X	50/6"			119	2					
	X	50/4"			112	2					
10	X	50/3"				2					Disturbed Sample
15	X	50/3"				1					
20	X	50/3"				2					
				Boring Terminated at 20'							

TBL 20G183-1.GPJ SOCALGEO.GDT 9/14/20

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, California

DRILLING DATE: 8/25/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					OLDER ALLUVIUM: Light Brown Sitly fine Sand, trace Clay, trace medium to coarse Sand, very dense-damp							
	X	72/8"					6					
					VAL VERDE TONALITE (Kvt): Light Gray Brown to Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, slightly to highly weathered, friable, very dense-dry to damp							
	X	68/11"					5					
5												
	X	50/5"					2					
	X	75/11"					4					
10												
	X	50/4"					1					
15												
					Refusal at 17' due to very dense bedrock							

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-1

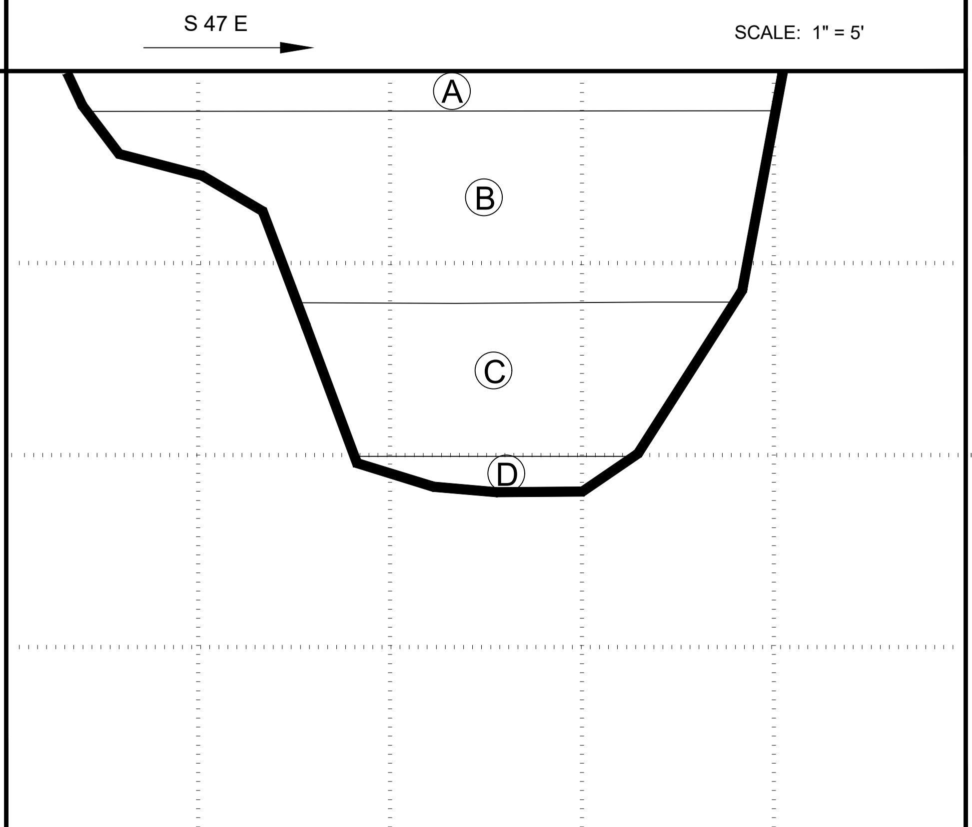
JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, CA
DATE: 8-20-2020

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: S 47 E
TOP OF TRENCH ELEVATION: ---- feet msl

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		4	A: FILL: Brown Silty fine to medium Sand, loose to medium dense - damp
				B: OLDER ALLUVIUM: Dark Brown Silty fine to medium Sand, trace coarse Sand, medium dense - damp to moist
	b		7	
	b		5	
5				
	b		4	C: VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, friable, highly weathered, very dense - damp
	b		3	D: VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, friable, slightly weathered, very dense - damp
10				
	b		6	
				Refusal @ 11 feet due to very dense bedrock
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-5

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-2

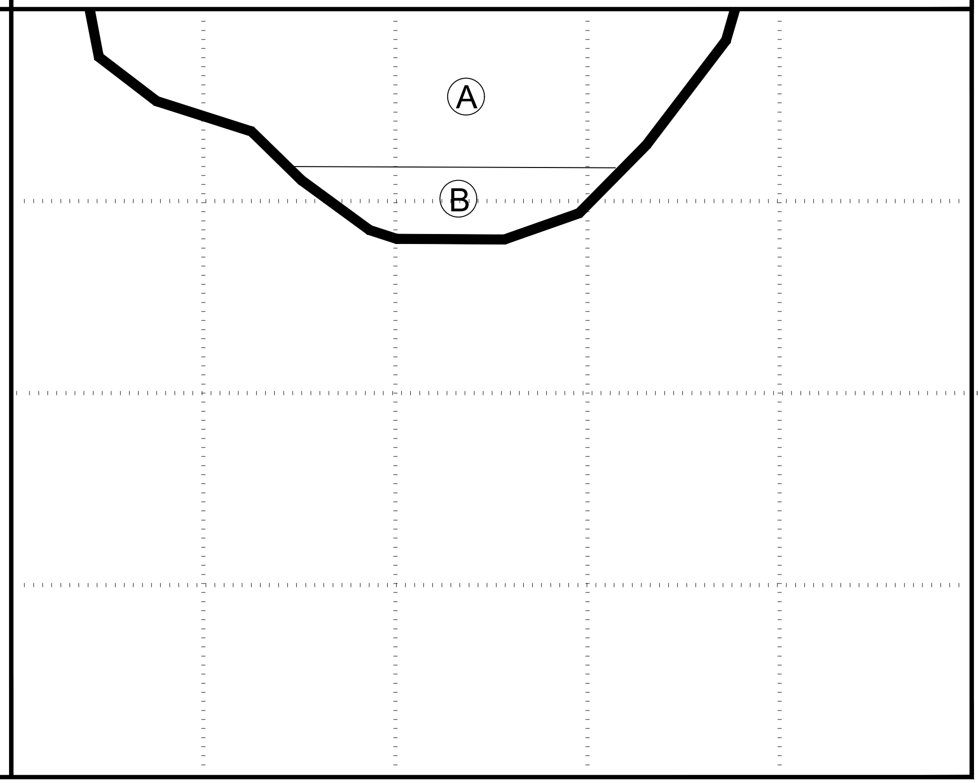
JOB NO.: 20G183-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse - Knox VII	LOGGED BY: Daryl Kas	SEEPAGE DEPTH: Dry
LOCATION: Riverside County, CA	ORIENTATION: S 30 W	READINGS TAKEN: At Completion
DATE: 8-20-2020	TOP OF TRENCH ELEVATION: ---- feet msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		3	A: OLDER ALLUVIUM: Brown Silty fine to medium Sand, medium dense to dense - damp
	b		4	
	b		1	B: VAL VERDE TONALITE (Kvt): Brown fine to coarse grained Tonalite bedrock, phaneritic, friable, slightly weathered, very dense - dry
5	b		1	
				Refusal @ 6 feet due to very dense bedrock
10				
15				

GRAPHIC REPRESENTATION

S 30 W →

SCALE: 1" = 5'



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

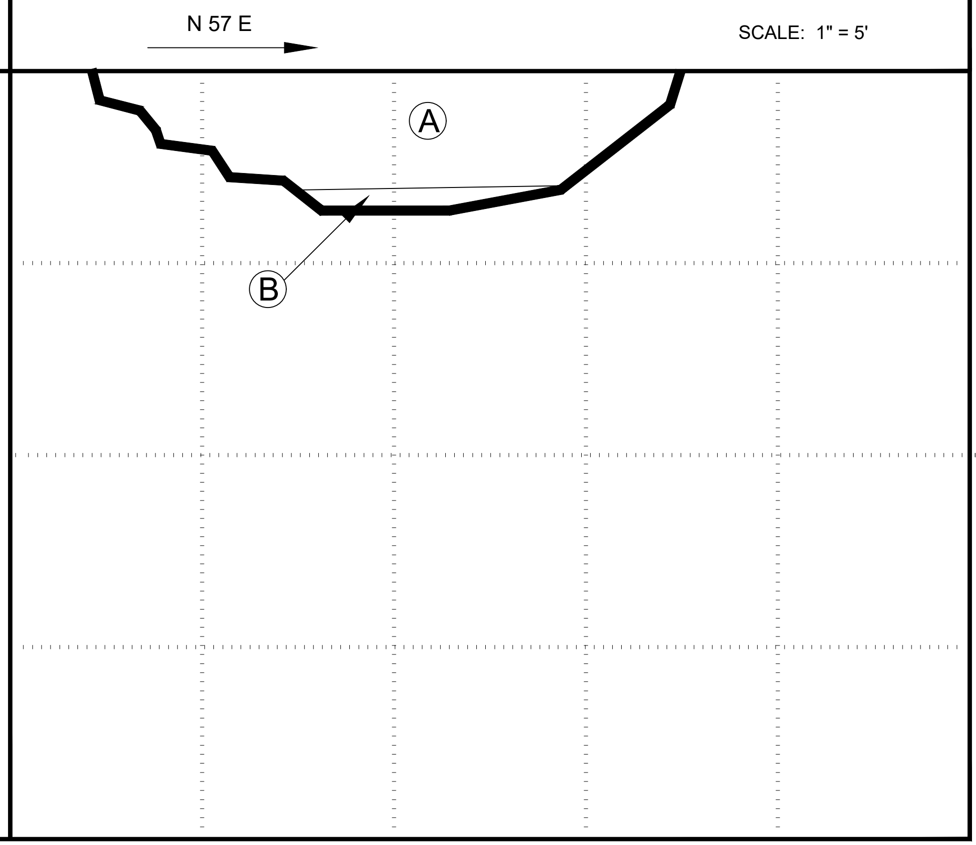
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-3

JOB NO.: 20G183-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Warehouse - Knox VII	LOGGED BY: Daryl Kas	SEEPAGE DEPTH: Dry
LOCATION: Riverside County, CA	ORIENTATION: N 57 E	READINGS TAKEN: At Completion
DATE: 8-20-2020	TOP OF TRENCH ELEVATION: ---- feet msl	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		2	A: OLDER ALLUVIUM: Brown Silty fine Sand, little medium Sand, trace coarse Sand, medium dense - dry to damp
	b		2	B: VAL VERDE TONALITE (Kvt): Gray Brown fine to coarse grained Tonalite bedrock, phaneritic, slightly weathered, very dense - dry to damp
				Refusal @ 3.5 feet due to very dense bedrock
5				
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-7

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-4

JOB NO.: 20G183-1
PROJECT: Proposed Warehouse - Knox VII
LOCATION: Riverside County, CA
DATE: 8-20-2020

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 55 E
TOP OF TRENCH ELEVATION: ---- feet msl

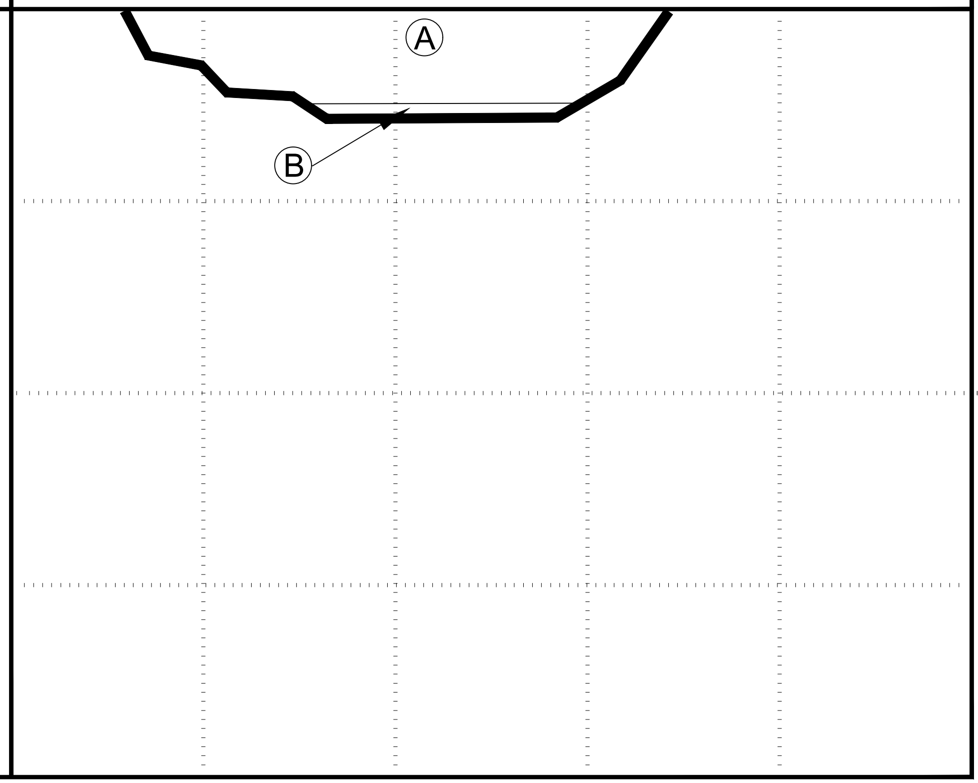
WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: OLDER ALLUVIUM: Red Brown Clayey fine Sand, some Silt, medium dense to dense - dry to damp
	b		4	B: VAL VERDE TONALITE (Kvt): Brown fine to coarse grained Tonalite bedrock, phaneritic, friable, very dense - dry to damp
	b		6	
				Refusal @ 3 feet due to very dense bedrock
5				
10				
15				

GRAPHIC REPRESENTATION

N 55 E

SCALE: 1" = 5'



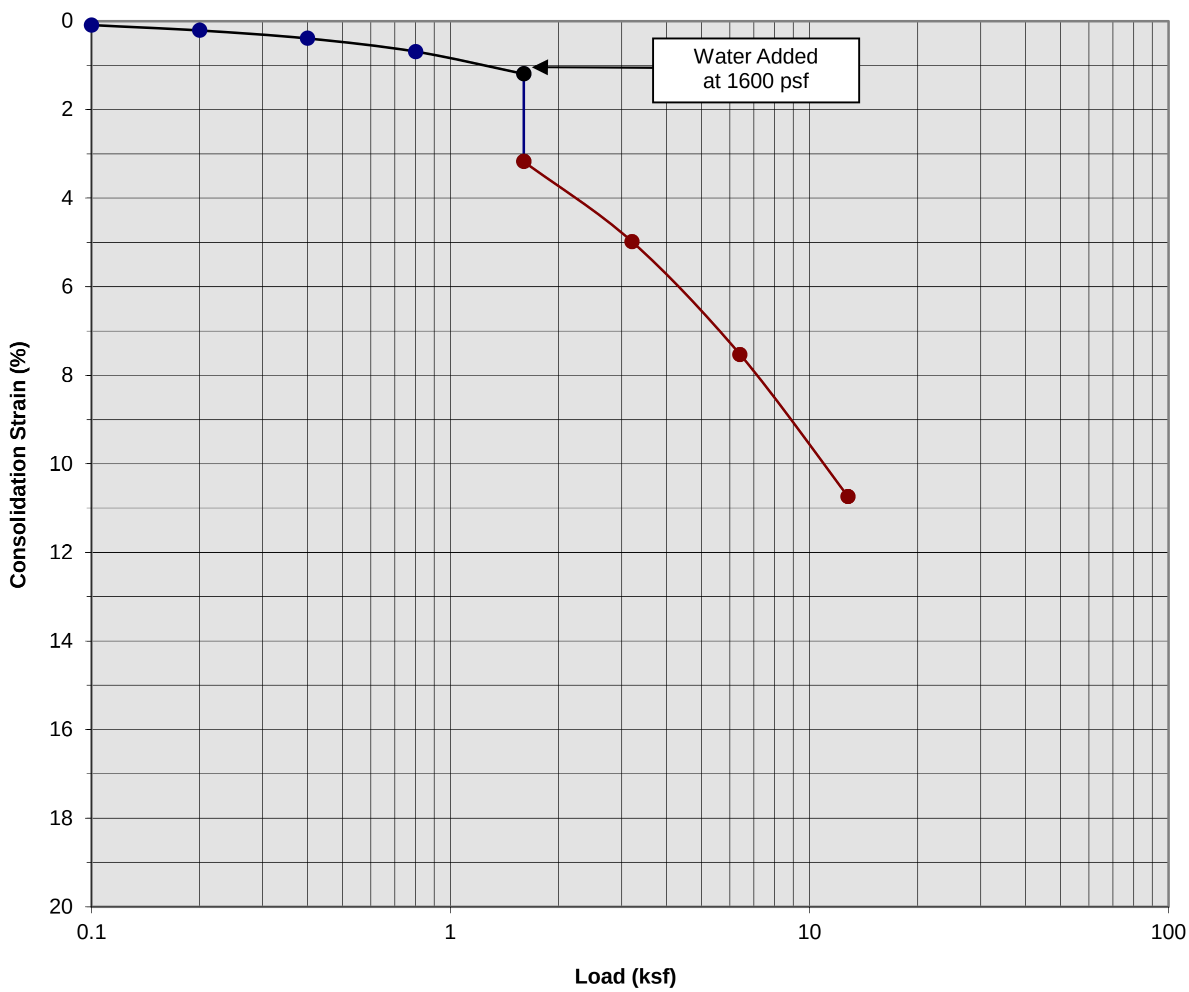
KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-8

APPENDIX C

Consolidation/Collapse Test Results

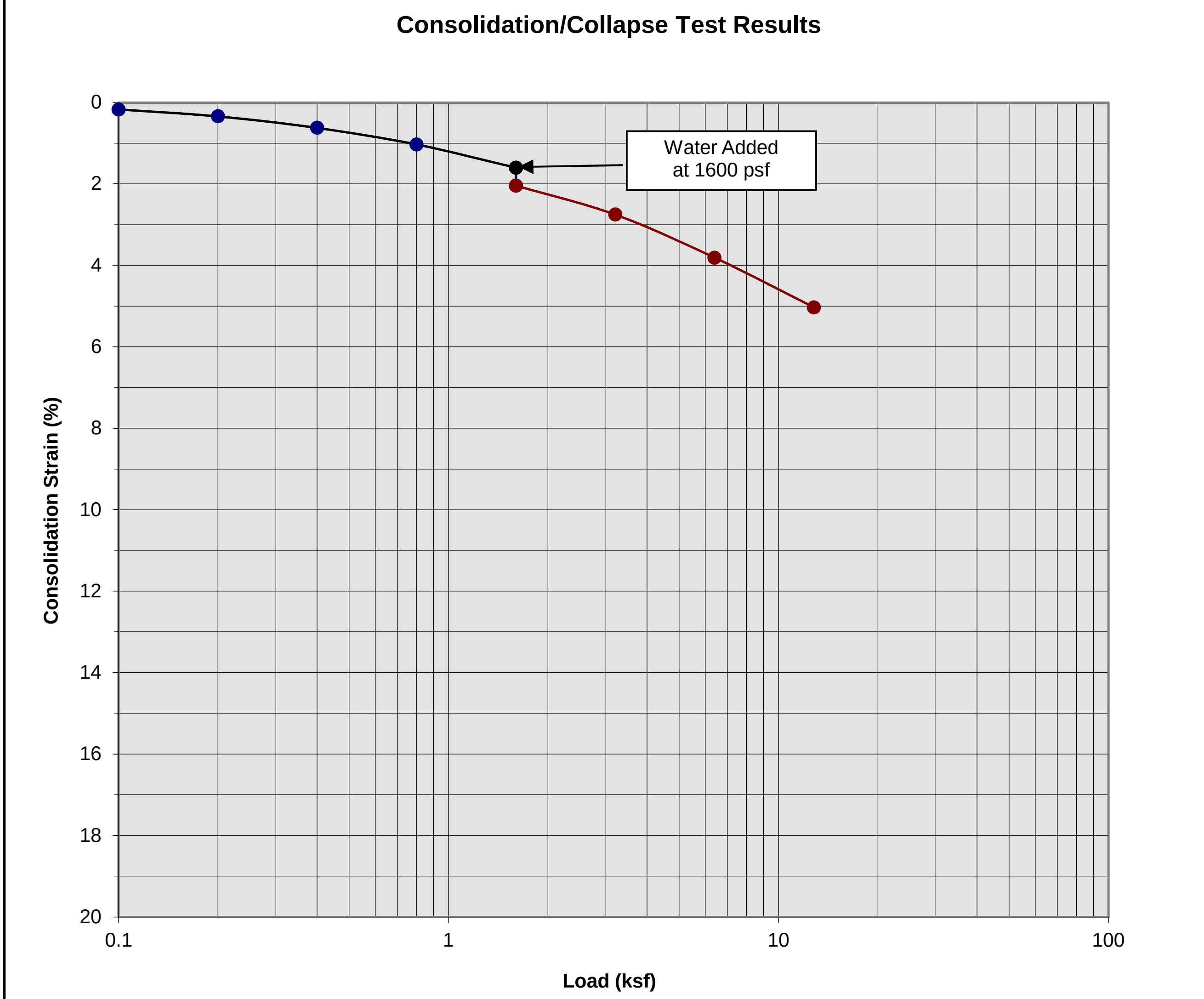


Classification: OLDER ALLUVIUM: Red Brown Clayey fine Sand, trace m-c Sand

Boring Number:	B-1	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	1 to 2	Initial Dry Density (pcf)	123.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	137.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.98

Proposed Warehouse - Knox VII
Riverside County, California
Project No. 20G183-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

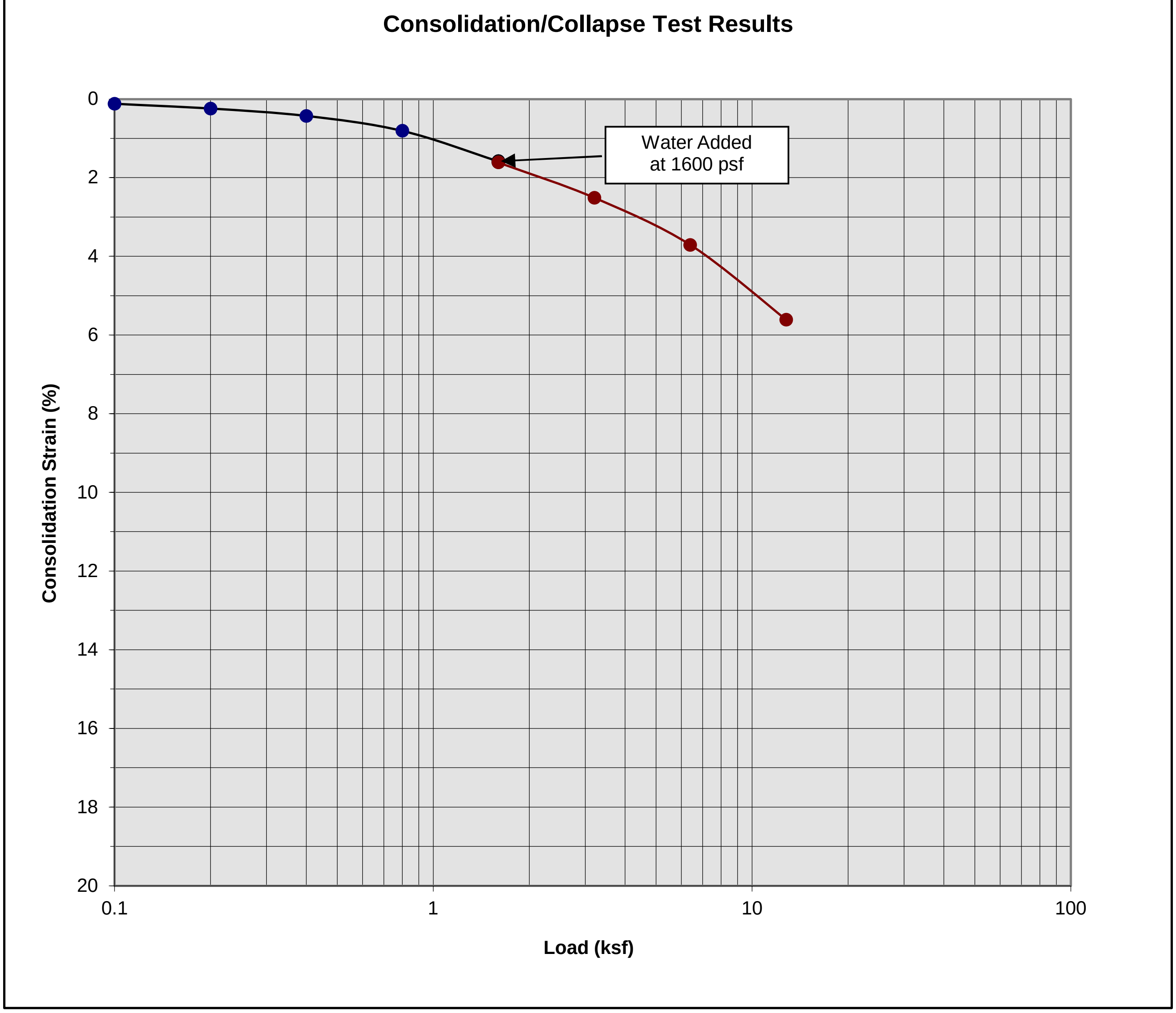


Classification: OLDER ALLUVIUM: Light Brown to Brown Silty fine to medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	3 to 4	Initial Dry Density (pcf)	130.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	135.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.44

Proposed Warehouse - Knox VII
Riverside County, California
Project No. 20G183-1
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

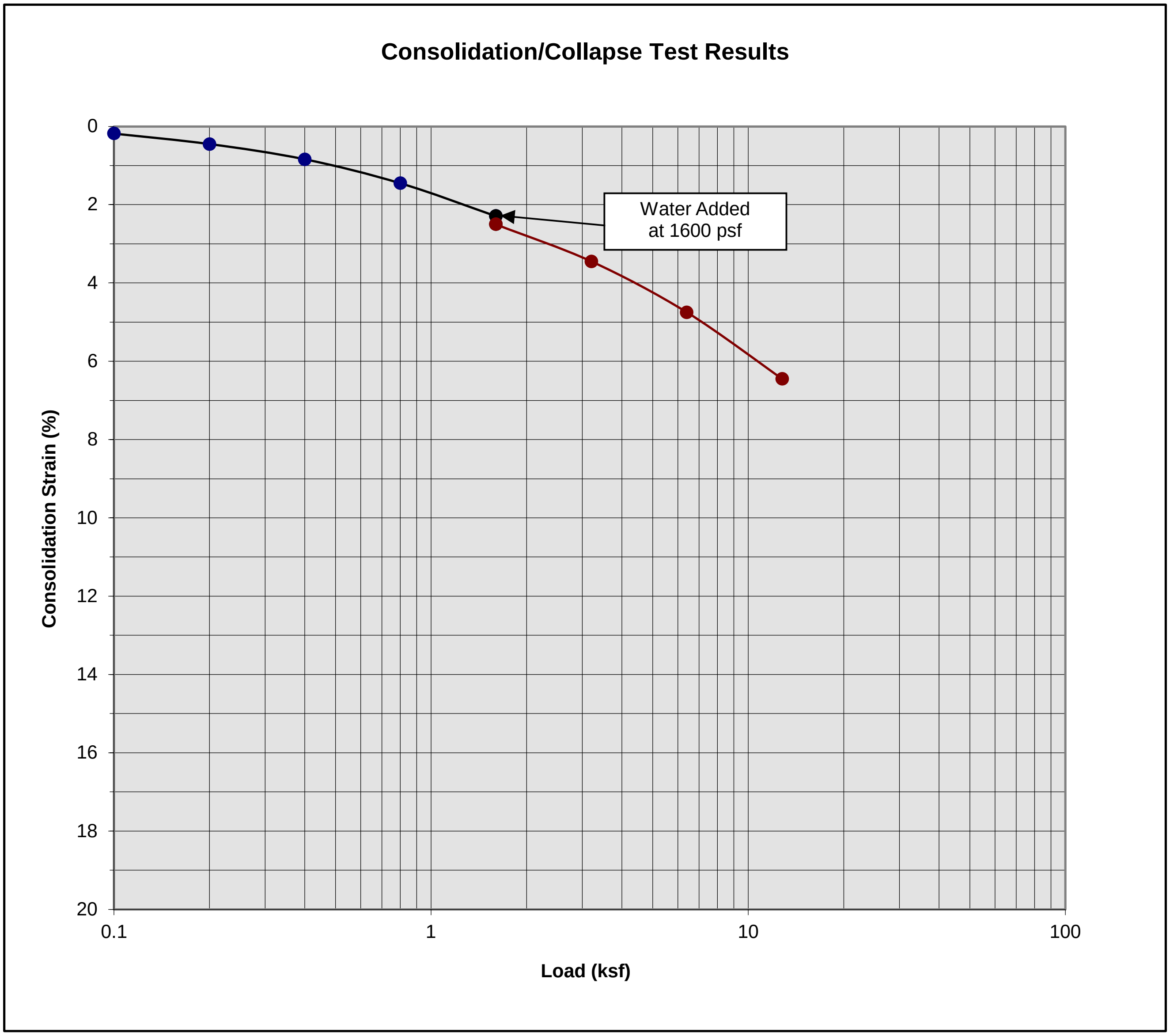


Classification: BEDROCK: Light Gray Brown to Gray Brown fine to coarse grained Tonalite

Boring Number:	B-1	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	5 to 6	Initial Dry Density (pcf)	115.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.03

Proposed Warehouse - Knox VII
Riverside County, California
Project No. 20G183-1
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

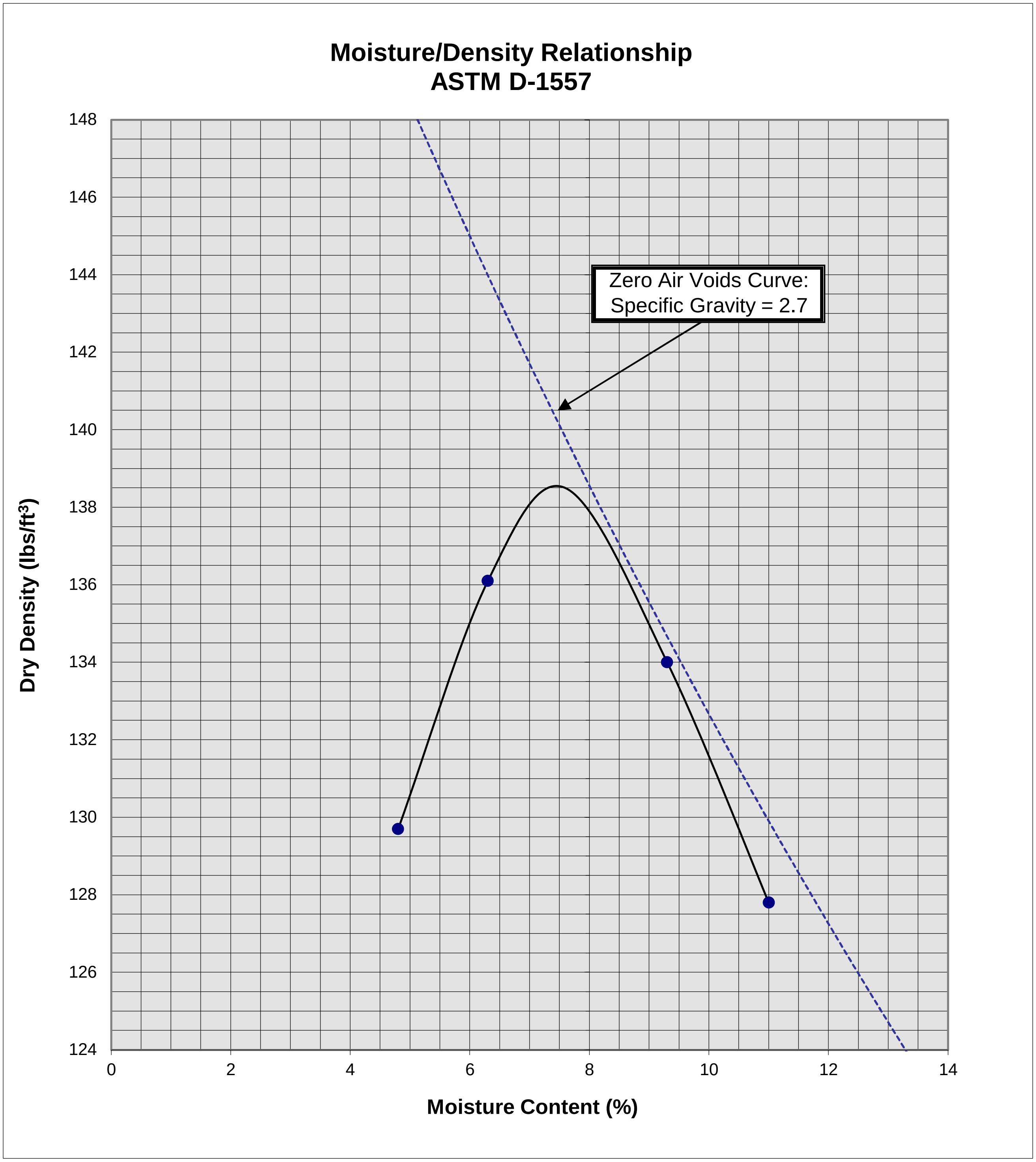


Classification: BEDROCK: Light Gray Brown to Gray Brown fine to coarse grained Tonalite

Boring Number:	B-1	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	7 to 8	Initial Dry Density (pcf)	113.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.21

Proposed Warehouse - Knox VII
Riverside County, California
Project No. 20G183-1
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-1 @ 0-5'
Optimum Moisture (%)		7.5
Maximum Dry Density (pcf)		138.5
Soil Classification	Gray Brown Silty fine to coarse Sand, little Clay	

Proposed Warehouse - Knox VII
Riverside County, California
Project No. 20G183-1
PLATE C-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated,” and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

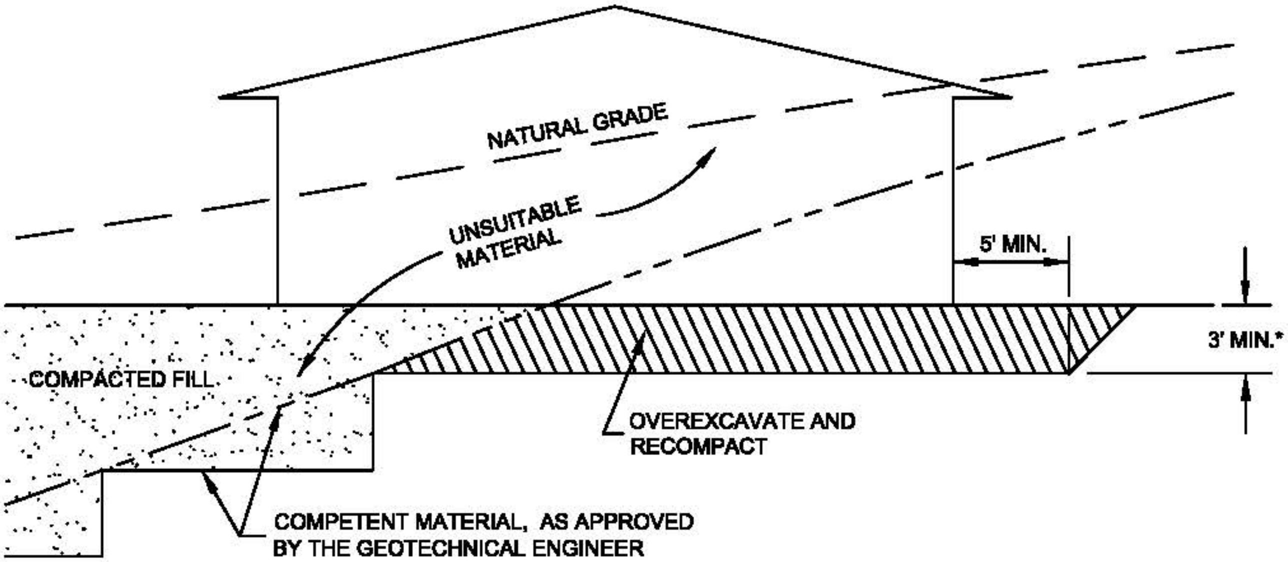
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

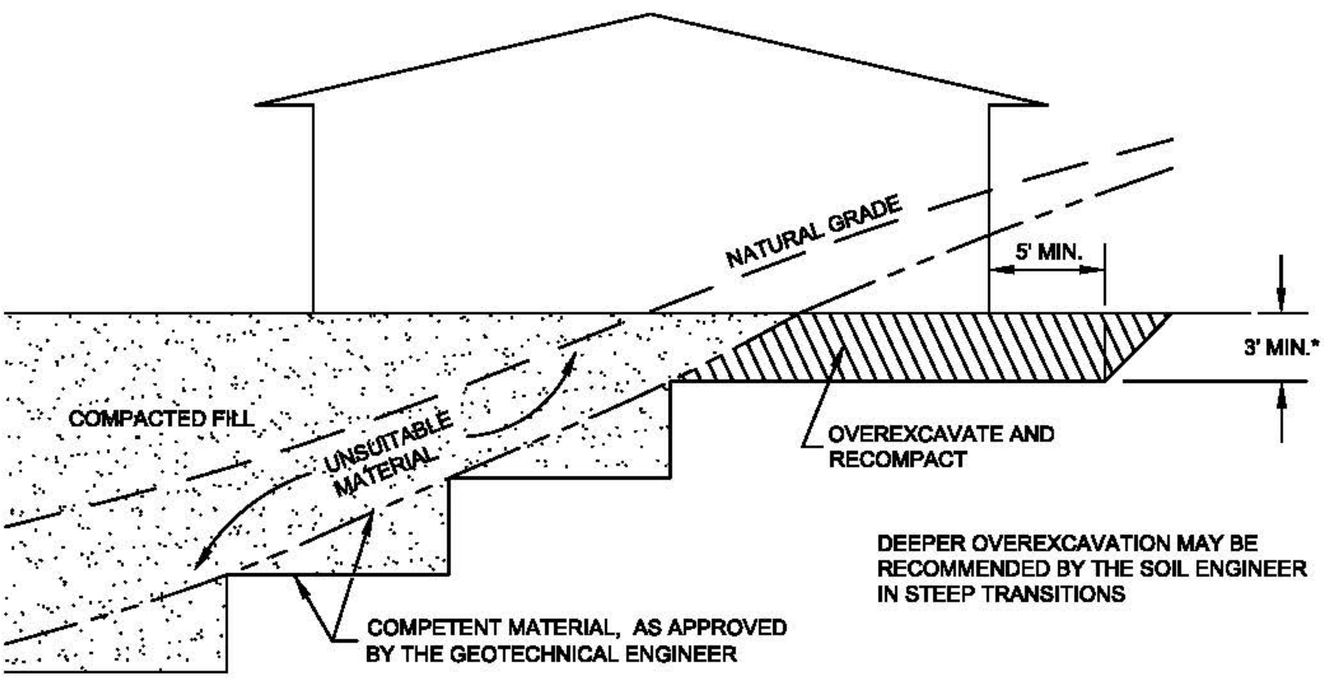
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

CUT LOT



CUT/FILL LOT (TRANSITION)



*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

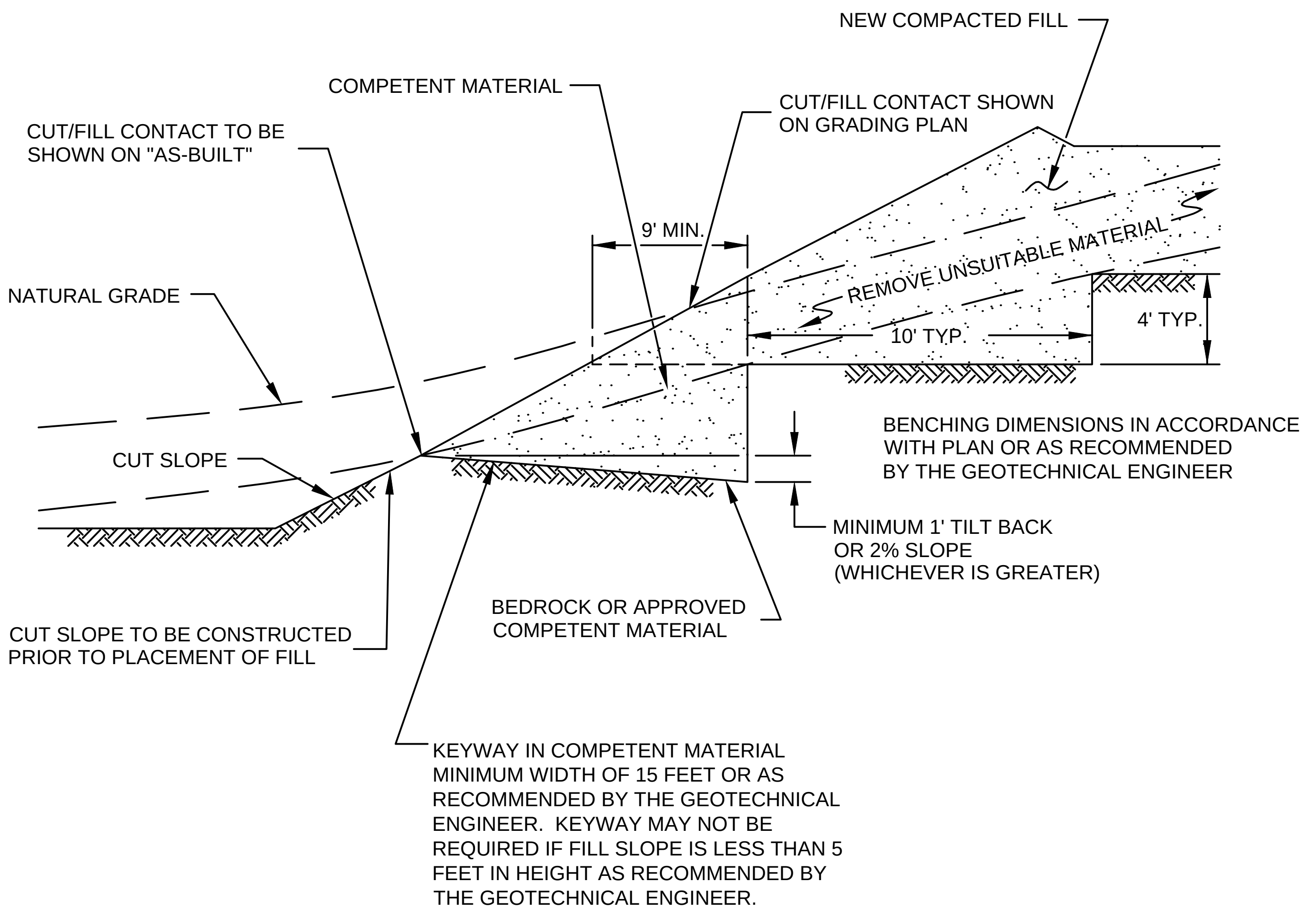
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

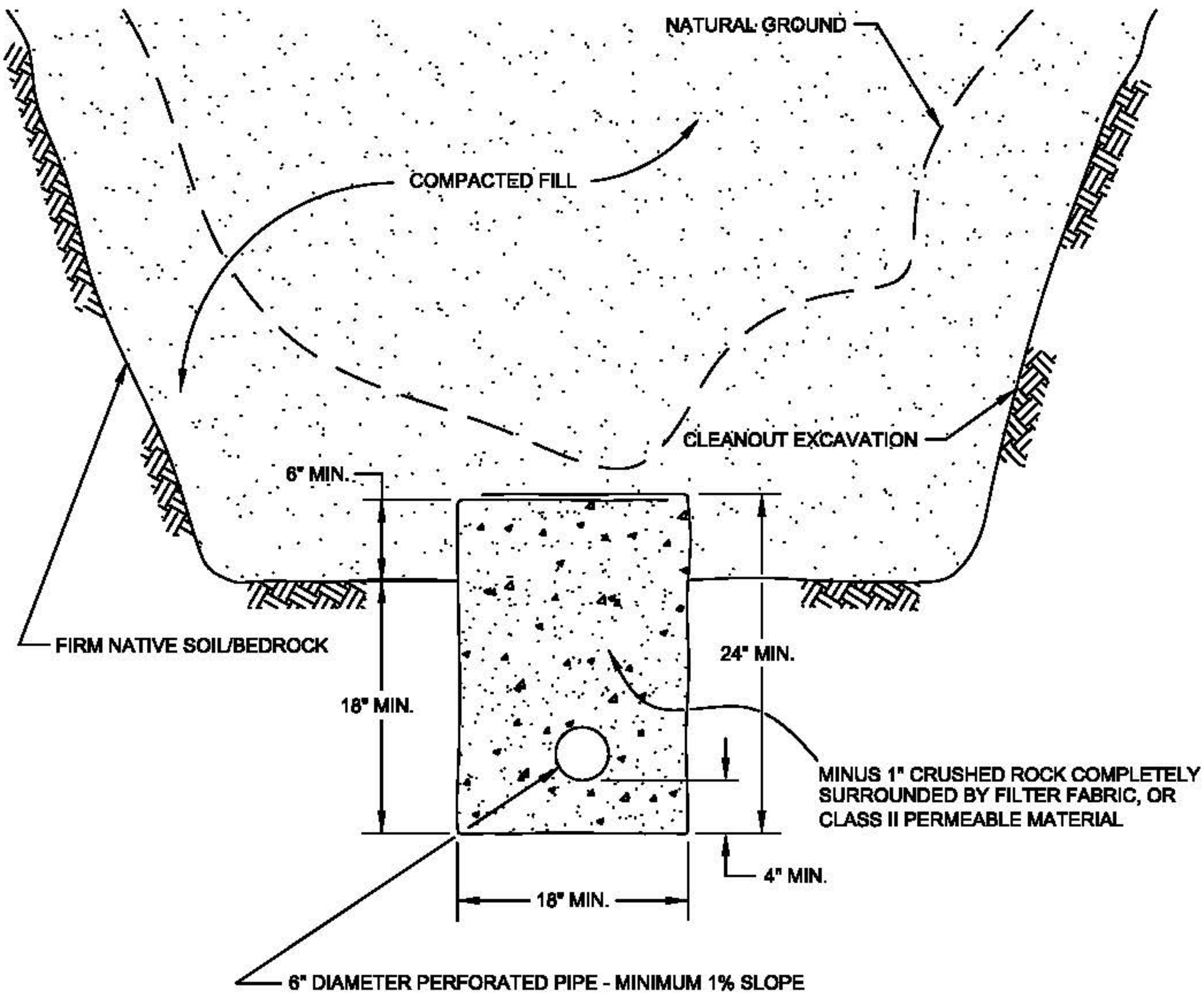
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

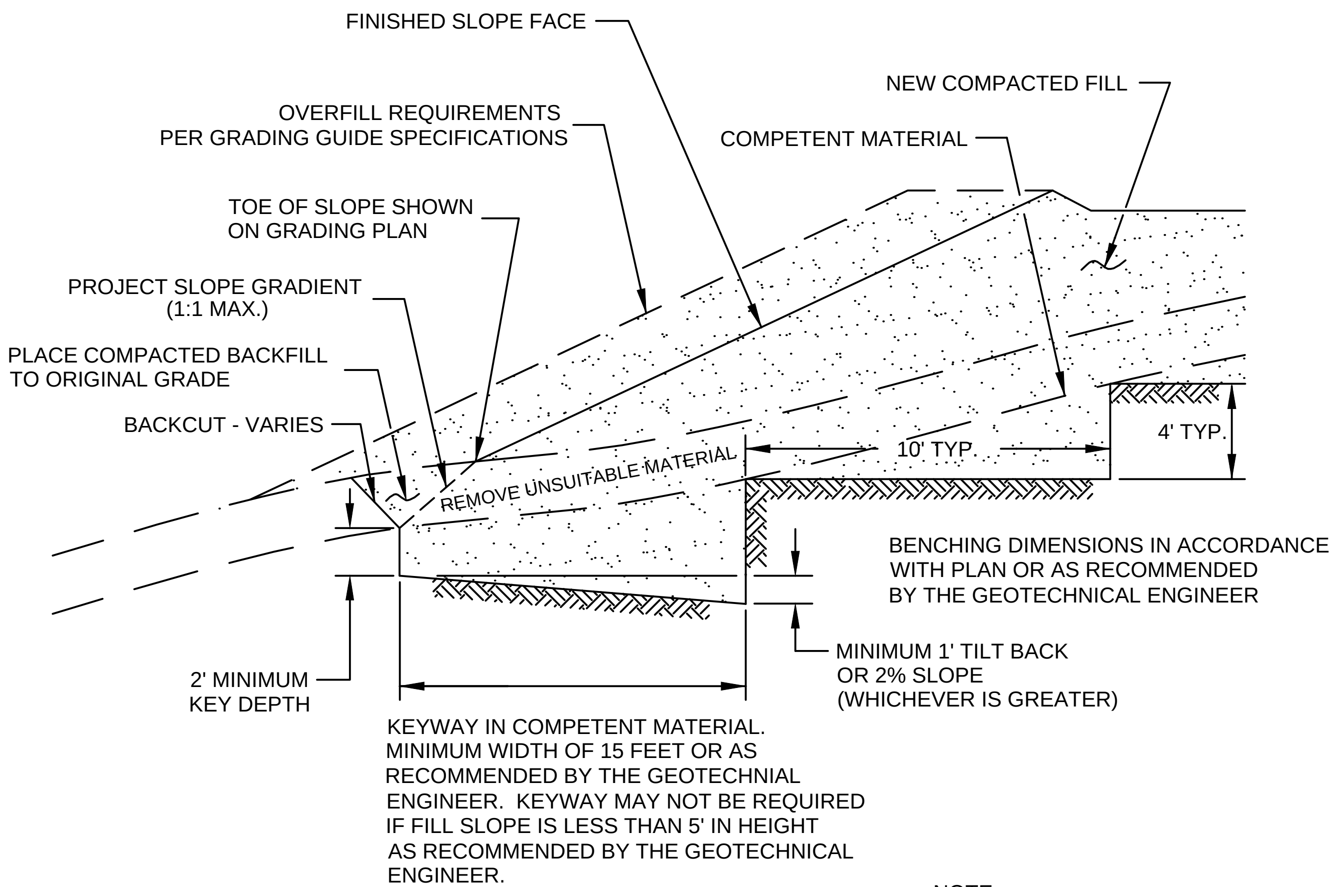
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



NOTE:
BENCHING SHALL BE REQUIRED
WHEN NATURAL SLOPES ARE
EQUAL TO OR STEEPER THAN 5:1
OR WHEN RECOMMENDED BY
THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

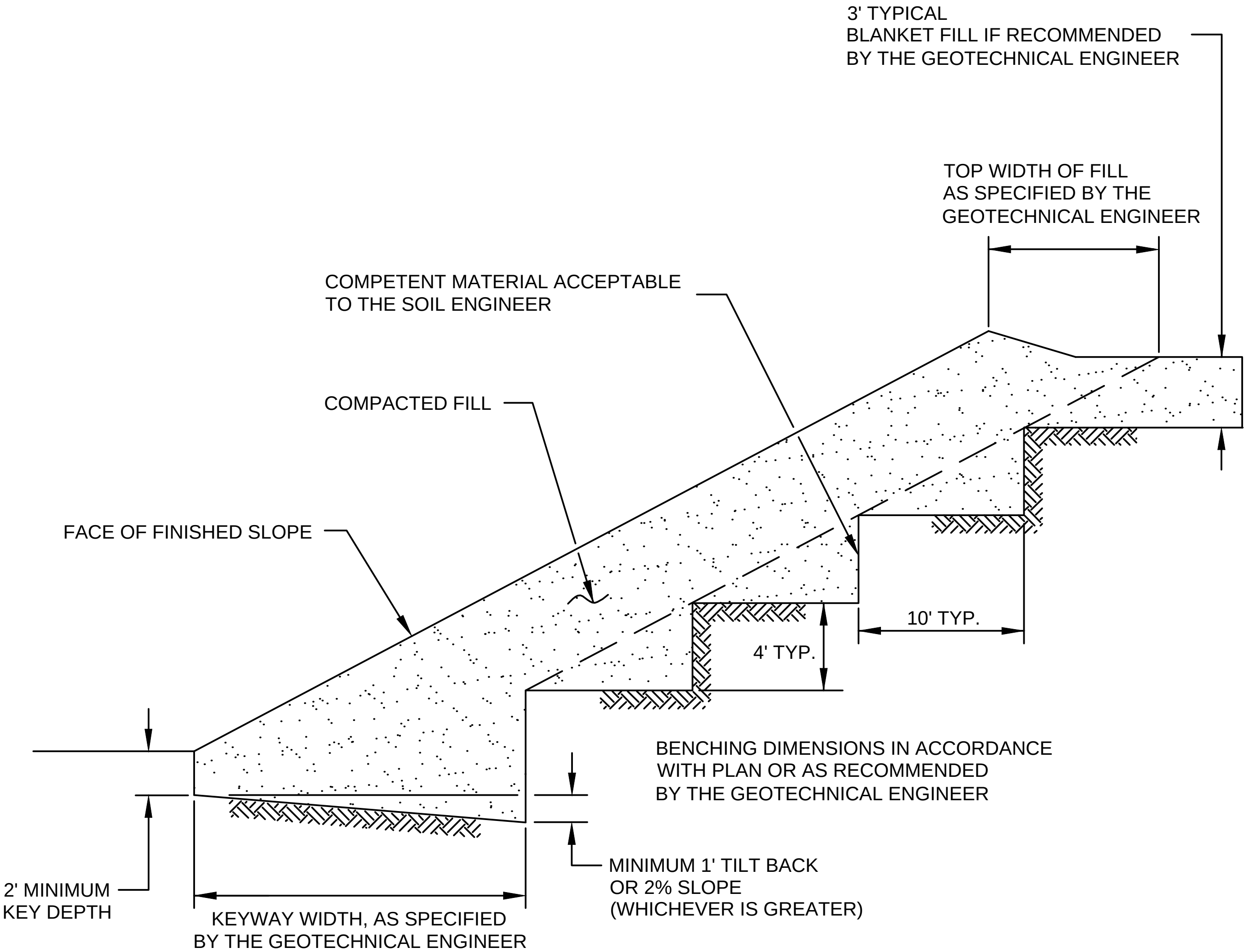
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

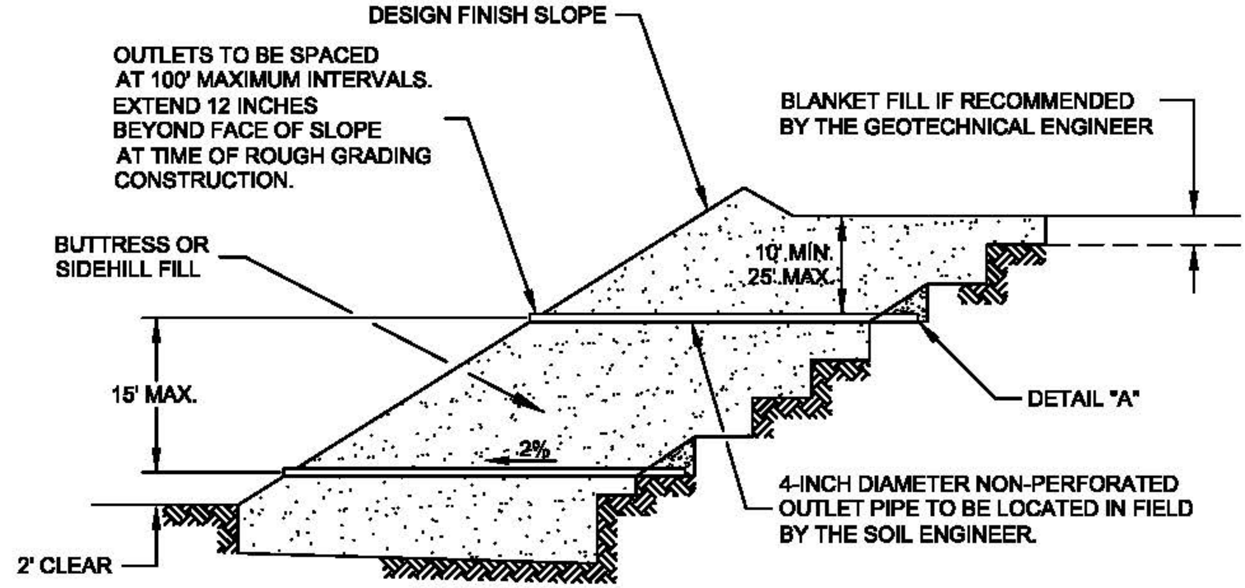
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

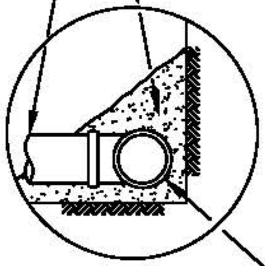
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW



FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

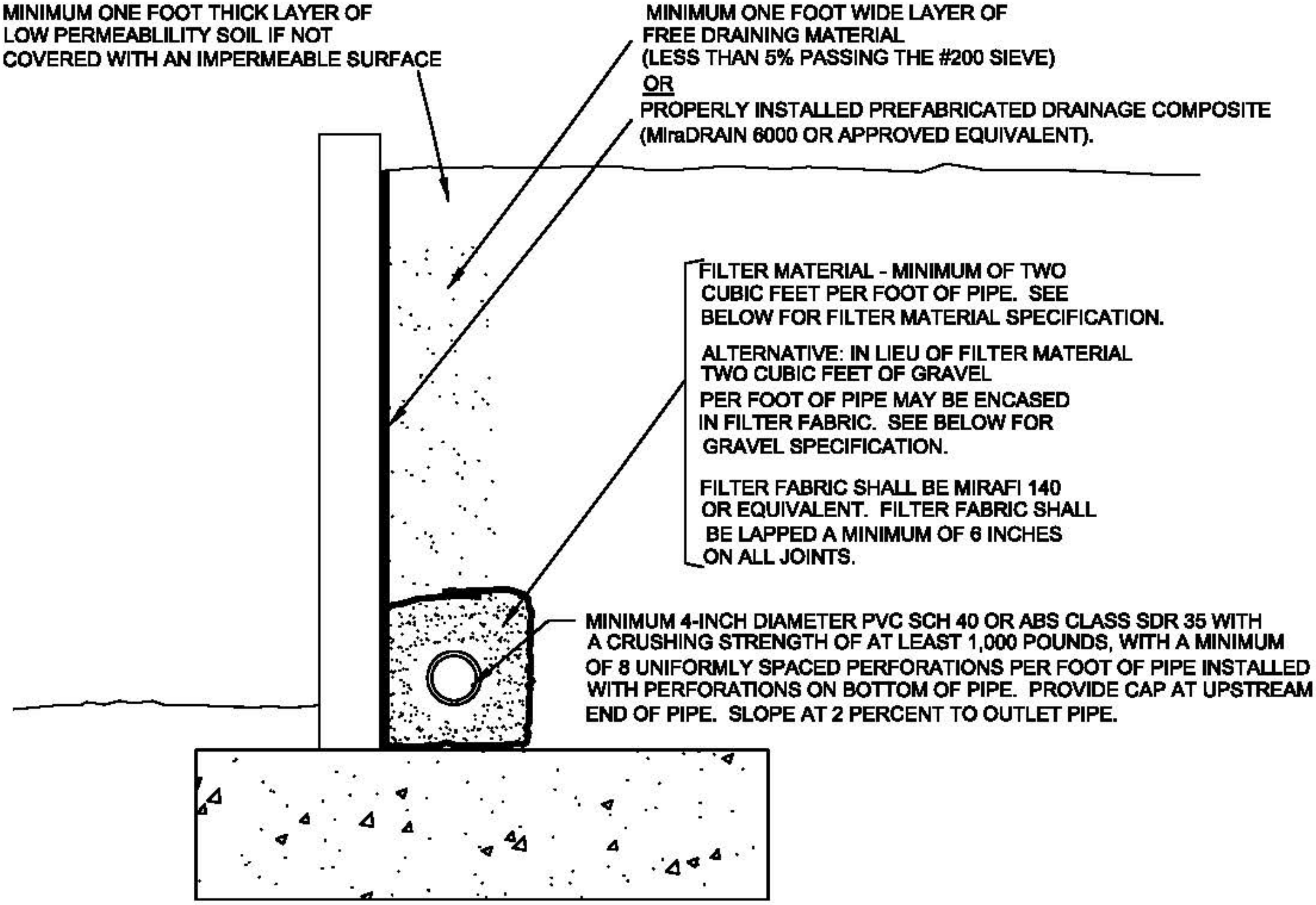
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

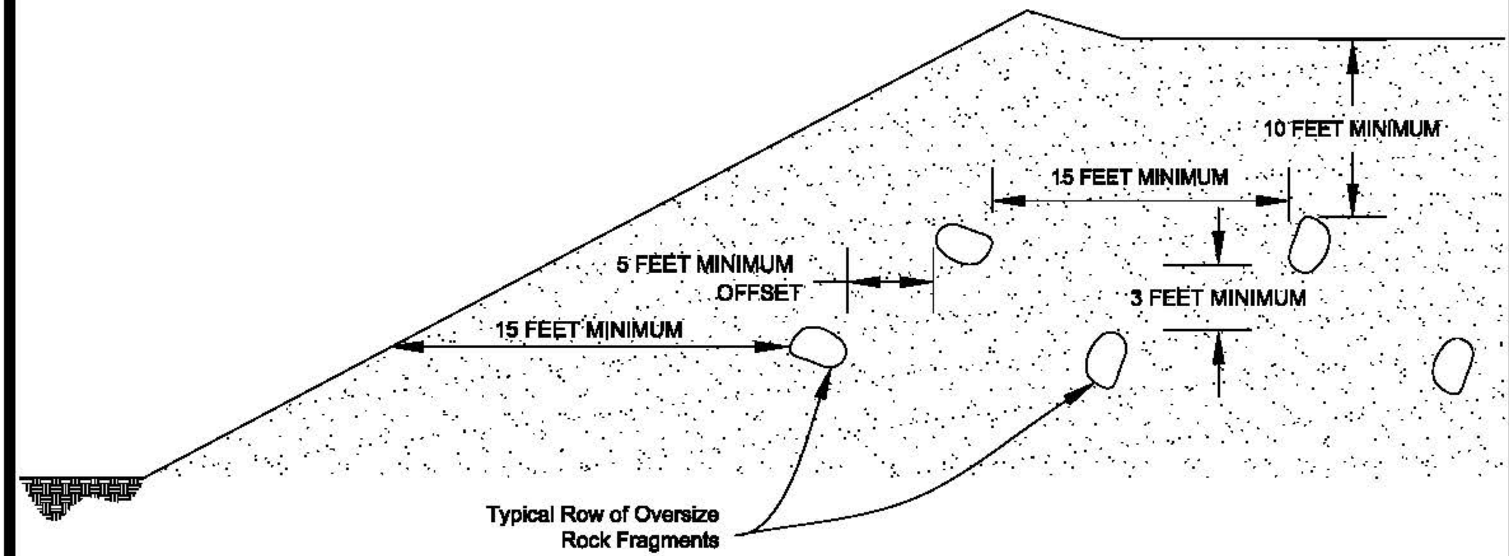
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

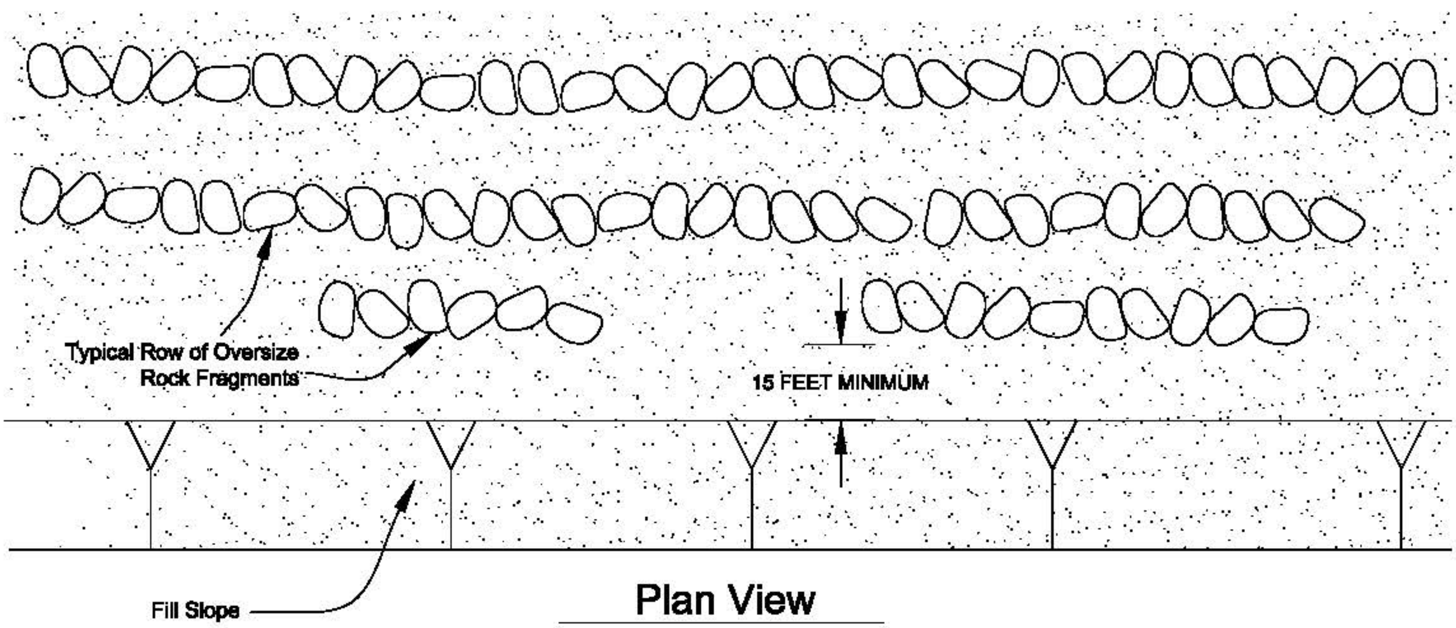
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View

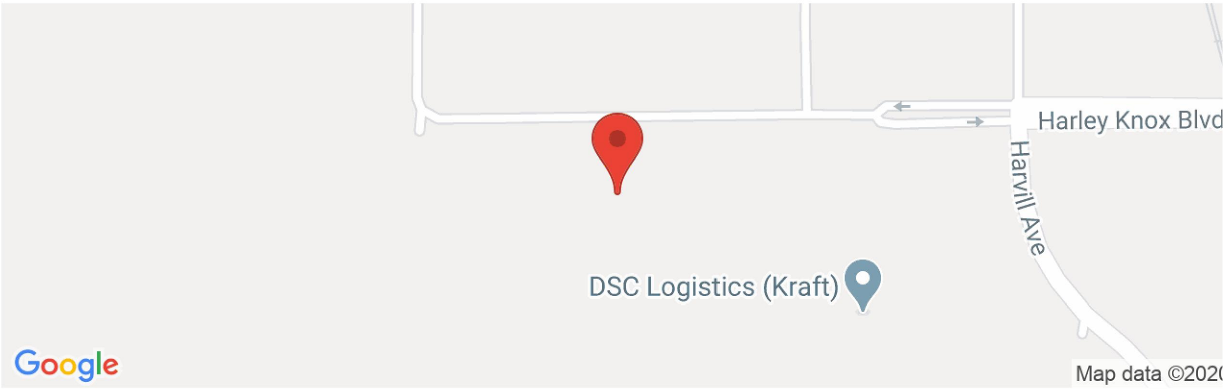


Plan View

PLACEMENT OF OVERSIZED MATERIAL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

Latitude, Longitude: 33.861796, -117.267831



Date	8/27/2020, 10:18:40 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	B - Estimated (see Section 11.4.3)

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.574	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	0.574	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.383	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1	Site amplification factor at 1.0 second
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.5	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.547	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.653	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.574	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.628	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.936	Mapped value of the risk coefficient at short periods
C_{R1}	0.914	Mapped value of the risk coefficient at a period of 1 s

SOURCE: SEAOC/OSHPD Seismic Design Maps Tool
<https://seismicmaps.org/>

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED WAREHOUSE - KNOX VII

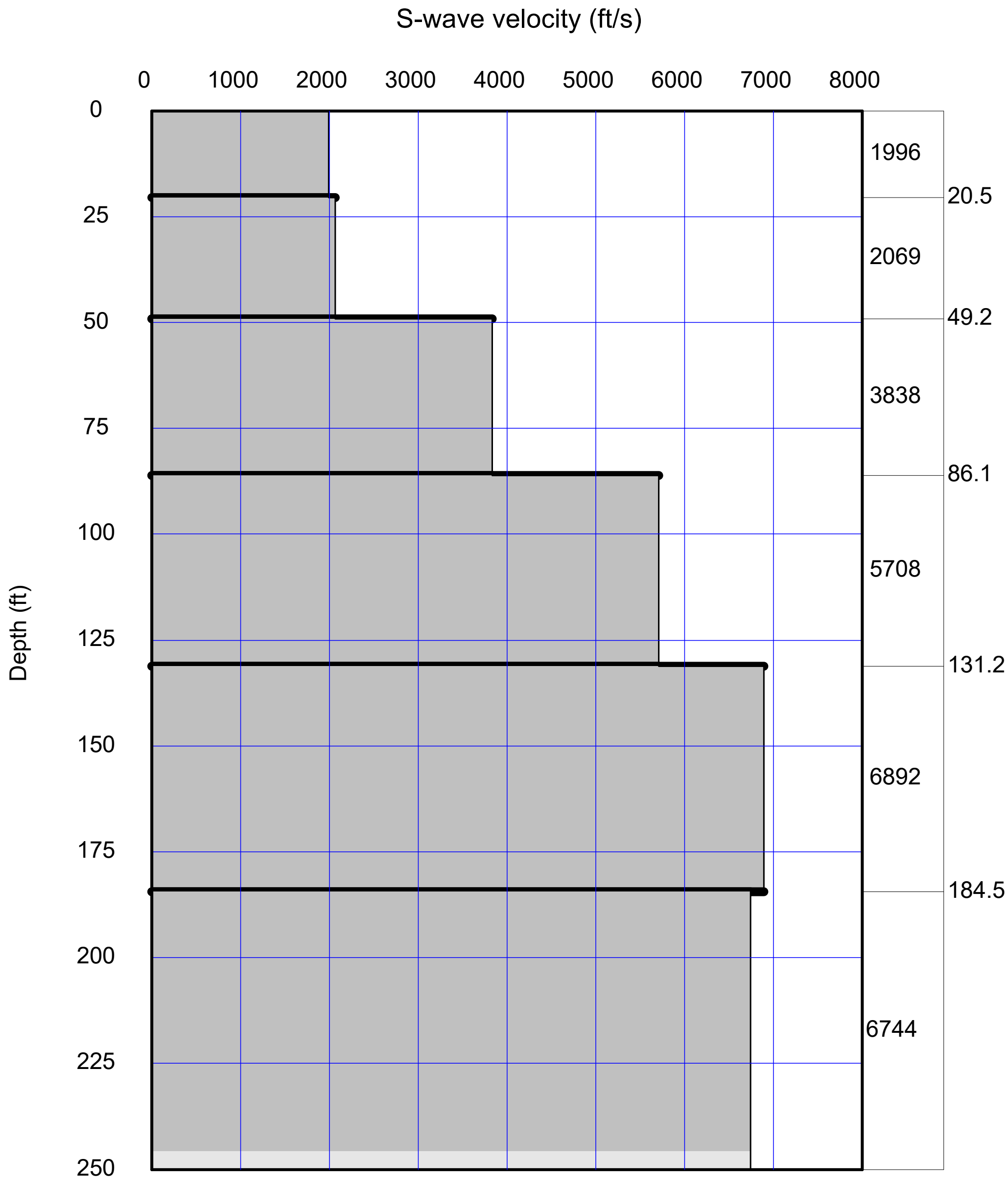
RIVERSIDE COUNTY, CALIFORNIA

DRAWN: JLL
CHKD: GKM

SCG PROJECT
20G183-1

PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 2,763.8 ft/sec

Site Classification (CBC-2019 1613A.2.2 & ASCE 7-16 Ch. 20)- "B" (Rock)

Client: **Southern California Geotechnical, Inc., Project No. 20G183-1**

Project Name: **Proposed Warehouse - Knox VII, Perris Area, California**

Survey Line End Coordinates: **33.86187, -117.26871 / 33.86187, -117.26810**

Date: **9/11/20**

PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

TERRA GEOSCIENCES
P.O. Box 1090
Loma Linda, CA 92354

TG Project No. 193303-3

APPENDIX F

TERRA GEOSCIENCES

SEISMIC REFRACTION SURVEY

PROPOSED MURANAKA PROJECT

SE CORNER OF HARLEY KNOX BOULEVARD AND DECKER ROAD

PERRIS AREA, RIVERSIDE COUNTY, CALIFORNIA

Project No. 193303-1

November 1, 2019

Prepared for:

Trammel Crow Company
3501 Jamboree Road
Suite 239
Newport Beach, CA 92660

Trammel Crow Company
3501 Jamboree Road, Suite 239
Newport Beach, CA 92660

November 1, 2018
Project No. 193303-1

Attention: Mr. Neal Holdridge, Principal

Regarding: Seismic Refraction Survey
Proposed Muranaka Project
SE Corner of Harley Knox Boulevard and Decker Road
Perris Area, Riverside County, California

EXECUTIVE SUMMARY

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high velocity bedrock materials (non-rippable) may be present. Additionally, the structure and seismic velocity distribution of the subsurface earth materials was also assessed. This report will describe in further detail the procedures used and the results of our findings, along with presentation of representative seismic models for the survey traverse.

For this study, eight survey traverses were performed across the subject property, as approved by your office. The traverses were located in the field by use of Google™ Earth imagery (2019) and GPS coordinates. The approximate locations of these traverses are shown on the Seismic Line Location Map, Plate 1, of which the base map is a captured Google™ Earth image (2019).

This opportunity to be of service is sincerely appreciated. If you should have questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

TABLE OF CONTENTS

INTRODUCTION

The subject study area is located at the southeast corner of Harley Knox Boulevard and Decker Road, in the Perris area of Riverside County, California. Geomorphically, the subject study area is located within the northwestern portion of the Perris Block, which is an eroded mass of Cretaceous and older crystalline rock forming generally flat-lying erosion surfaces now present at various elevations. More specifically, the subject property is located within the western transition zone of the southern Peninsular Ranges batholith, along the northwestern portion of the Cretaceous age Lakeview Mountains Valley pluton.

Locally, as shown on Figure 1 below, surficial mapping by Morton (2003) indicates the subject study area to be underlain by Cretaceous age granitic rocks (locally referred to as the Val Verde tonalite) consisting of a gray-weathering, relatively homogeneous, massive to well-foliated, medium- to coarse-grained, biotite-hornblende tonalite (map symbol Kvt). Along the east portion of the site, very old alluvial fan deposits (early Pleistocene age) are shown to be present, comprised of well-indurated sand deposits (map symbol Qvof). These deposits may be surficially present across most of the site.

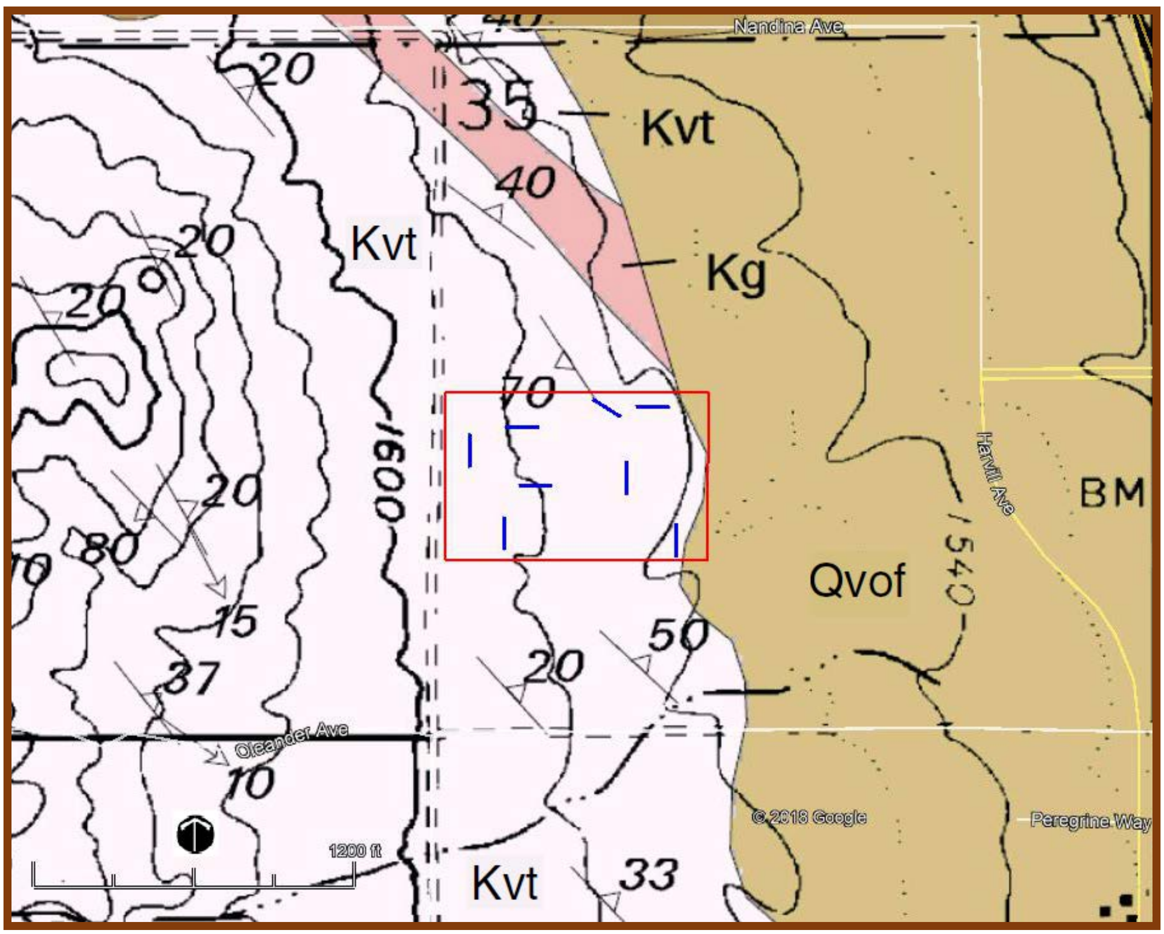


FIGURE 1- Geologic Map (Morton, 2003), Seismic traverses shown as blue lines.

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Eight seismic refraction survey lines (Seismic Lines S-1 through S-8) have been performed along representative areas across the subject study area as selected by you. The traverses were located in the field by use of Google™ Earth imagery (2019) and GPS coordinates and have been delineated on the Seismic Line Location Map, as presented on Plate 1. The survey traverses were each 125 feet in length, which consisted of a total of twenty-four 14-Hertz geophones, spaced at regular five-foot intervals, in order to detect both the direct and refracted waves. A 16-pound sledge-hammer was used as the energy source to produce the seismic waves. Multiple hammer impacts were utilized at each shot point in order to increase the signal to noise ratio, which enhanced the primary seismic “P”-waves.

The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.064 seconds. No acquisition filters were used during data collection.

During acquisition, the seismograph displays the seismic wave arrivals on the computer screen which were used to analyze the arrival time of the primary seismic “P”-waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. If spurious “noise” was observed, the shot location was resampled during relatively quieter periods. Each geophone and seismic shot location were surveyed using a hand level and ruler for topographic correction, with “0” being the lowest point along each survey line.

Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2018); and **Rayfract™** (Intelligent Resources, Inc., 1996-2019). All of the computer programs perform their individual analyses using exactly the same input data, which includes the first-arrival times of the "P"-waves and the survey line geometry.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of six times to improve the accuracy of depths to the top of layer-2. This first-arrival picks were then used to generate the Layer Velocity Models using the **SIPwin** computer program, which presents the subsurface velocities as individual layers and are presented within Appendix A for reference. In addition, the associated Time-Distance Plot for each survey line, which shows the individual data picks of the first "P-wave" arrival times, also appears in Appendix A.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the ABC method, and the Generalized Reciprocal Method (GRM). These methods are used for defining irregular non-planar refractors and are briefly described below. The Delay-Time method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay-time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The ABC (intercept time) method makes use of critically refracted rays converging on a common surface position. This method involves using three surface to surface travel times between three geophones and the velocity of the first layer in an equation to calculate depth under the central geophone and is applied to all other geophones on the survey line. The GRM method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

- **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves which generally indicates the relative structure and velocity distribution of the subsurface using first break energy propagation modeling. An initial 1D gradient model is created using the DeltatV method (Gebrande and Miller, 1985) which gives a good initial fit between modeled and picked first breaks. The DeltatV method is a turning-ray inversion method which delivers continuous depth vs. velocity profiles for all profile stations. These profiles consist of horizontal inline offset, depth, and velocity triples. The method handles real-life geological conditions such as velocity gradients, linear increasing of velocity with depth, velocity inversions, pinched-out layers and outcrops, and faults and local velocity anomalies. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion (Schuster and Quintus-Bosz, 1993).

  WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break. This computer program performs the analysis by using the same first-arrival P-wave times and survey line geometry that were generated during the layer velocity model analyses. The associated Refraction Tomographic Models which display the subsurface earth material velocity structure, is represented by the velocity contours (isolines displayed in feet/second), supplemented with the color-coded velocity shading for visual reference, and are presented within Appendix B.

The combined use of these computer programs provided a more thorough and comprehensive analysis of the subsurface structure and velocity characteristics. Each computer program has a specific purpose based on the objective of the analysis being performed. **SIPwin** and **Refractor** were primarily used for detecting generalized subsurface velocity layers providing "weighted average velocities." The processed seismic data of these two programs were compared and averaged to provide a final composite layer velocity model which provided a more thorough representation of the subsurface. **Rayfract™** provided tomographic velocity and structural imaging that is very conducive to detecting strong lateral velocity characteristics such as imaging corestones, dikes, and other subsurface structural characteristics.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it is important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, relic bedding, etc.), creating anisotropic conditions. Anisotropy (direction-dependent properties of materials) can be caused by "micro-cracks," jointing, foliation, layered or inter-bedded rocks with unequal layer stiffness, small-scale lithologic changes, etc. (Barton, 2007). Velocity anisotropy complicates interpretation and it should be noted that the seismic velocities obtained during this survey may have been influenced by the nature and character of any localized structural discontinuities within the bedrock underlying the subject site.

Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Such variable directions can result in velocity differentials of between 2% to 40% depending upon the degree of the structural fabric (i.e., weakly-moderately-strongly foliated, respectively). Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time.

The first computer method described below used for data analysis is the traditional layer method (**SIPwin** and **Refractor**). Using this method, it should be understood that the data obtained represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders, dikes, or other local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics. In general, the site where locally surveyed, was noted to be characterized by three major subsurface layers (Layers V1, V2, and V3) with respect to seismic velocities. The following velocity layer summaries have been prepared using the **SIPwin** and **Refractor** analysis, with the representative Layer Velocity Model presented within Appendix A along with the respective Time-Distance Plot.

- ***Velocity Layer V1***: This uppermost velocity layer (V1) is most likely comprised of colluvium, topsoil, older alluvium, and/or completely-weathered and fractured bedrock materials. This layer has an average weighted velocity of 1,472 to 2,650 fps, which is typical for these types of unconsolidated surficial earth materials.

- ***Velocity Layer V2***: The second layer (V2) yielded a seismic velocity range of 3,507 to 6,245 fps, which is typical for highly- to moderately-weathered granitic bedrock materials. This velocity range may indicate the presence of homogeneous weathered bedrock with a relatively wide spaced joint/fracture system and/or the possibility of buried relatively-fresher boulders within a very highly-weathered bedrock matrix. Additionally, the presence of older alluvial sediments, such as mapped by Morton (2003) in the local area, may also be locally present in this velocity layer based upon the degree of sediment induration.

- ***Velocity Layer V3***: The third layer (V3) indicates the presence of moderate- to slightly-weathered bedrock, having a seismic velocity range of 6,249 to 11,984 fps. These higher velocities signify the decreasing effect of weathering as a function of depth and could indicate a slightly-weathered bedrock matrix that has a wide-spaced fracture system, or possibly the presence of abundant widely-scattered buried fresh large crystalline boulders in a moderately-weathered matrix, which based on the abundant large surface rock outcrops exposed across the site, appears likely.

The following table summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer, as indicated on the Layer Velocity Models, presented within Appendix A.

TABLE 1- VELOCITY SUMMARY OF SEISMIC SURVEY LINES

Seismic Line	V1 Layer (fps)	V2 Layer (fps)	V3 Layer (fps)
S-1	**2,650**	**6,139**	**10,683**
S-2	**2,239**	**6,245**	**11,728**
S-3	**1,702**	**4,041**	**6,950**
S-4	**1,527**	**4,937**	**11,984**
S-5	**1,472**	**3,994**	**6,480**
S-6	**1,747**	**4,424**	**6,249**
S-7	**1,822**	**4,121**	**6,273**
S-8	**1,835**	**3,507**	**6,688**

Using **Rayfract™**, tomographic models were also prepared for comparative purposes to better illustrate the general structure and velocity distribution of the subsurface, using velocity contour isolines, as presented within Appendix B. Although no discrete velocity layers or boundaries are created, these models generally resemble the corresponding overall average layer velocities as presented within Appendix A.

In general, the seismic velocity of the bedrock gradually increases with depth, with occasional lateral velocity differentials suggesting the local presence of buried corestones and/or dike structures. These corestones are expected as numerous bedrock outcrops are scattered across the subject property. The colors representing the velocity gradients have been standardized on all of the models for comparative purposes.

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

A summary of the generalized rippability characteristics of bedrock based on a compilation of rippability performance charts prepared by Caterpillar, Inc. (2018; see Figure 2, Page 8), Caltrans (Stephens, 1978), and Santi (2006), has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. These seismic velocity ranges and rippability potentials have been tabulated below for reference.

TABLE 2- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

Granitic Rock Velocity	Rippability
< 6,800	Rippable
6,800 – 8,000	Moderately Rippable
> 8,000	Non-Rippable

Additionally, we have provided the Caltrans Rippability Chart as presented below within Table 2 for comparison. These values are from published Caltrans studies (Stephens, 1978) that are based on their experience and which appear to be more conservative than Caterpillar's rippability chart. It should be noted that the type of bedrock was not indicated.

TABLE 3- STANDARD CALTRANS RIPPABILITY CHART

Velocity (feet/sec ±)	Rippability
< 3,500	Easily Ripped
3,500 – 5,000	Moderately Difficult
5,000 – 6,600	Difficult Ripping / Light Blasting
> 6,600	Blasting Required

Table 3 is partially modified from the "Engineering Behavior from Weathering Grade" as presented by Santi (2006), which also provides velocity ranges with respect to rippability potentials, along with other rock engineering properties that may be pertinent.

TABLE 4- SUMMARY OF ROCK ENGINEERING PROPERTIES

ENGINEERING PROPERTY:	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely Weathered
Excavatability	Blasting necessary	Blasting to rippable	Generally rippable	Rippable
Slope Stability	½ :1 to 1:1 (H:V)	1:1 (H:V)	1:1 to 1.5:1 (H:V)	1.5:1 to 2:1 (H:V)
Schmidt Hammer Value	51 – 56	37 – 48	12 – 21	5 – 20
Seismic Velocity (fps)	8,200 – 13,125	5,000 – 10,000	3,300 – 6,600	1,650 – 3,300

The Caterpillar D9R Ripper Performance Chart (Caterpillar, 2018) has been provided on Figure 2 below for reference.

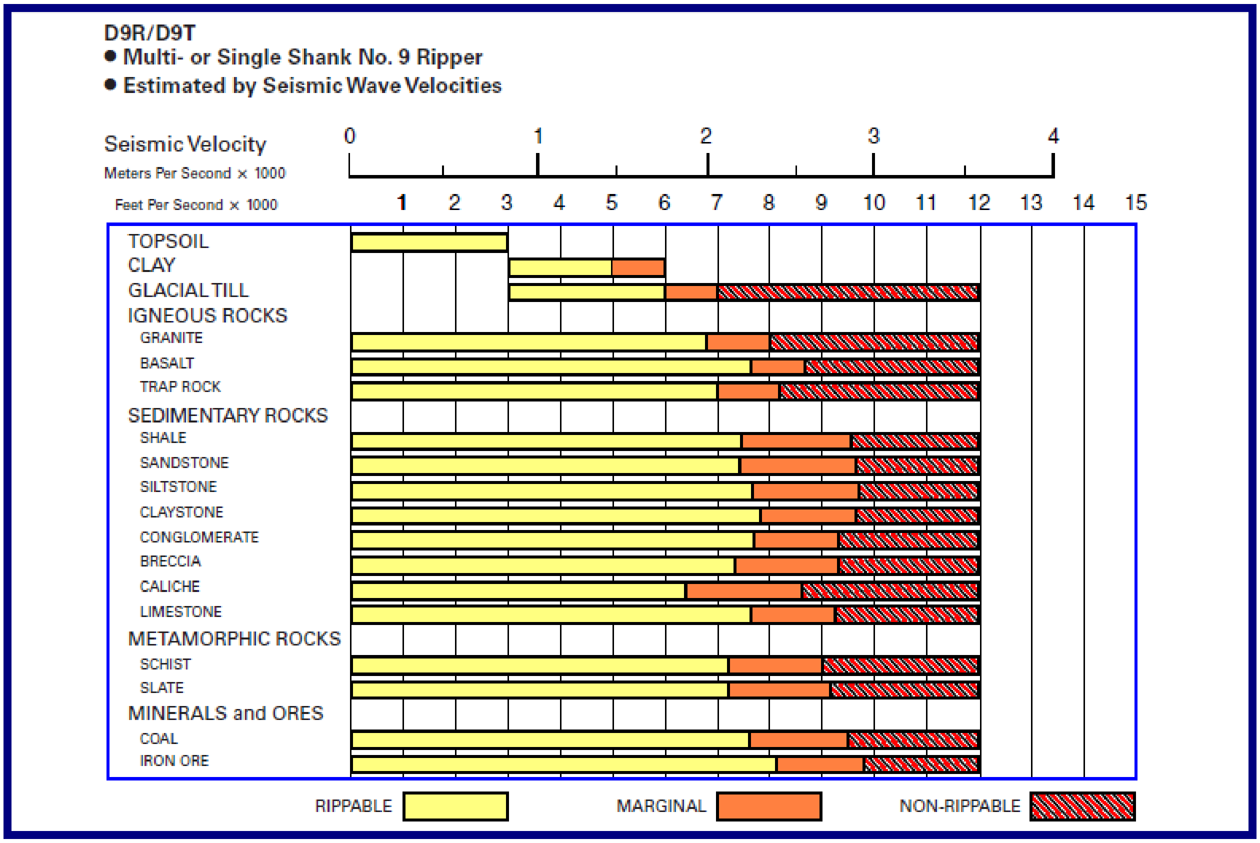


FIGURE 2- Caterpillar D9R Ripper Performance Chart (2018).

For purposes of the discussion in this report with respect to the expected bedrock rippability characteristics, we are assuming that a D9R/D9T dozer will be used as a minimum, such as discussed further below and as shown in Figure 2 above. Smaller excavating equipment will most likely result in slower production rates and possible refusal within relatively lower velocity bedrock materials. It should be noted that the decision for blasting of bedrock materials for facilitating the excavation process is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

A summary of the generalized rippability characteristics of granitic bedrock (such as present within the subject study area) has been provided below to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas that were surveyed. The velocity ranges described below are general averages of Tables 2 and 3 presented in this report (see Page 7) and assume typical, good-working, heavy excavation equipment, such as D9R dozer using a single shank, as described by Caterpillar, Inc. (2000 and 2018).

However, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges as the rippability performance charts are tailored for conventional bulldozer equipment and cannot be directly correlated. Trenching operations which utilize large excavator-type equipment within granitic bedrock materials, typically encounter very difficult to non-productable conditions where seismic velocities are generally greater than 4,000± fps, and less for smaller backhoe-type equipment.

These average seismic velocity ranges are summarized below:

- **Rippable Condition (0 - 4,000 ft/sec)**:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

  Marginally Rippable Condition (4,000 - 7,000 ft/sec):

  This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **Non-Rippable Condition (7,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

GEOLOGIC & EARTHWORK CONSIDERATIONS

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report and/or information gathered for the subject project which may describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults, and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is foliated and/or fractured at depth, this structure could aid in excavation production.

Unfavorable bedrock conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment that will be necessary and the production levels that can be anticipated for this project. A summary of excavation considerations is included within Appendix C in order to provide you and your grading contractor with a better understanding of the complexities of excavation in bedrock materials, so that proper planning and excavation techniques can be employed.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of good quality with minor amounts of ambient “noise” that was introduced during our survey, originating from vehicular traffic along Domenigoni Parkway to the north and wind sources. Analysis of the data and picking of the primary “P”-wave arrivals was therefore performed with little difficulty, with only minor interpolation of some data points being necessary. Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process, integrate, and display the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  No excavating difficulties are expected to be encountered within the uppermost, low-velocity layer V1 (average weighted velocity of 1,472 to 2,650 fps) and should excavate with conventional ripping. This layer is expected to be comprised of topsoil, colluvium, older alluvial sediments, and/or completely-weathered and fractured bedrock materials. Localized boulders should be anticipated based on surficial exposures, which may require more significant excavation techniques.

Velocity Layer V2:

The second layer V2 (average weighted velocity 3,507 to 6,245 fps) is believed to consist of highly- to moderately-weathered granitic bedrock (within higher end of velocity range) and/or possibly older alluvial sediments (within lower end of velocity range). Using the rock classifications as presented within Tables 1 through 3, seismic wave velocities of less than 6,800± fps are generally noted to be within the threshold for conventional ripping. Isolated floaters (i.e., boulders, corestones, etc.) should be expected to be present within this layer and could produce somewhat difficult conditions locally. A wide range of moderate to very difficult ripping conditions should be anticipated. Placement of infrastructure within this velocity layer may require some breaking and/or light blasting to obtain desired grade.

- **Velocity Layer V3**:

  The third V3 layer is believed to consist of moderate- to slightly-weathered bedrock. Extremely hard excavation difficulties within this velocity layer (average weighted velocity range of 6,249 to 11,984 fps) should be anticipated if encountered during grading. This layer may consist of relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Significant blasting should be anticipated throughout this layer to achieve desired grade, including any infrastructure. Caterpillar (2018; see Figure 2) indicates this velocity range to be “moderately-rippable” to “non-rippable” using a D9R dozer or equivalent. Larger equipment may facilitate excavation potentials within this higher velocity layer.

The ray sampling coverage of the subsurface seismic waves that were modeled during the processing of the tomographic models appeared to be of very good quality which was verified by having a Root Mean Square Error (RMS) of 2.7 to 4.9 percent (see lower right-hand corner of each model). The RMS error (misfit between picked and modeled first break times) is normalized, which calculates the average picked time over of all traces modeled. This error is automatically calculated during the processing routine, with a value of less than 5.0% being preferred, of which all of the models obtained.

Based on the tomographic models and typical excavation characteristics observed within granitic bedrock of the southern California region, anticipation of gradual increasing hardness with depth should be anticipated during grading. Significant lateral velocity variations will most likely be encountered across the predominance of the site generally due to the presence of buried corestones and/or dikes such as imaged in some of the tomographic refraction modes and as also expressed as scattered outcrops across the subject site.

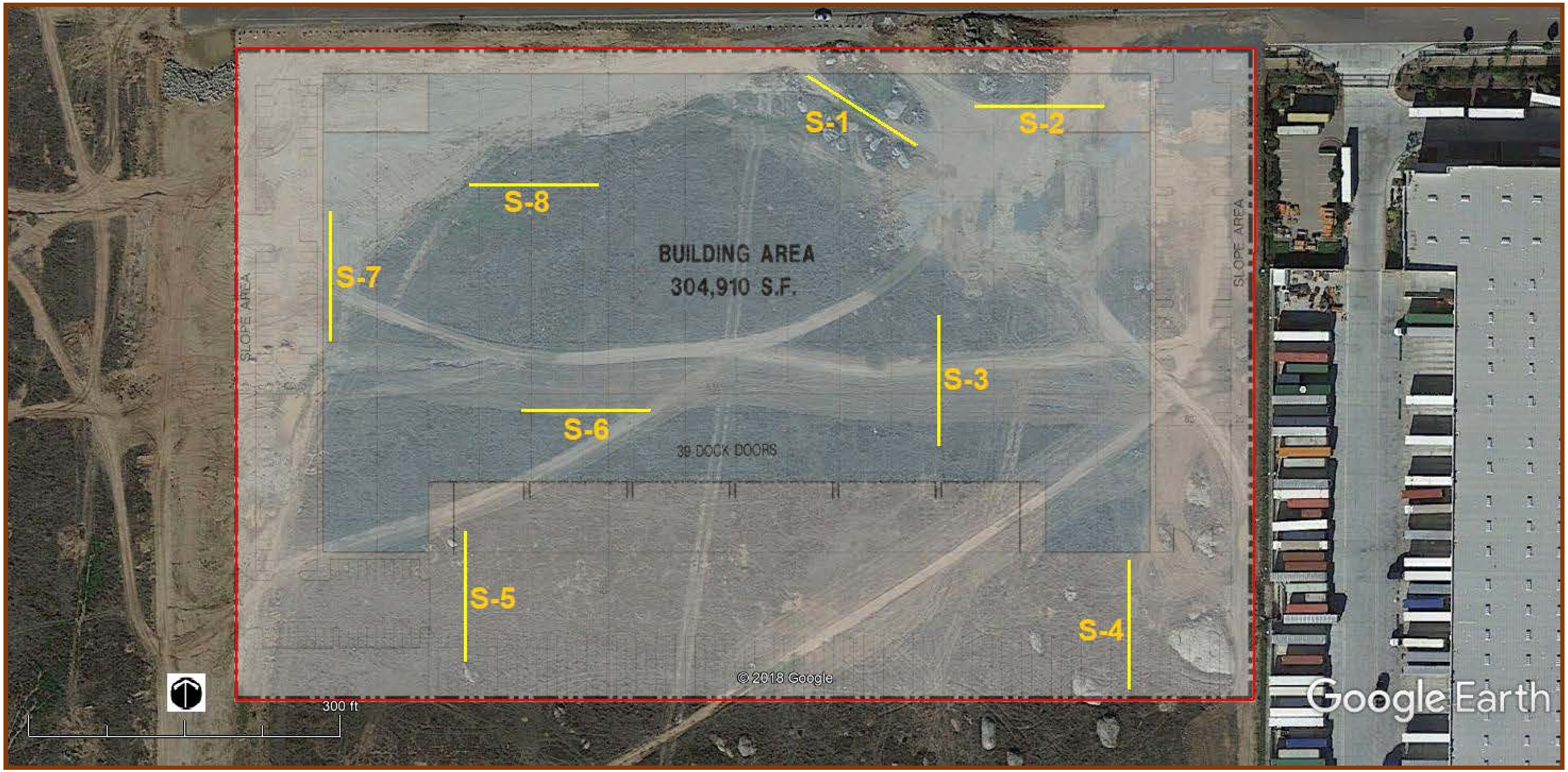
CLOSURE

The field geophysical survey was performed on October 25, 2019 by the undersigned using "state of the art" geophysical equipment and techniques along the selected traverse location. The seismic data was further evaluated using recently developed computerized tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface velocity and structural conditions. It should be noted that our data presented within this report was obtained along eight specific locations therefore other areas in the local may contain different velocity layers and depths not encountered during our field survey. Additional survey traverses may be necessary to further evaluate the excavation characteristics across other portions of the site where cut grading will be proposed, if warranted. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained, and in the interpretation, and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied.

In summary, the results of this seismic refraction survey are to be considered as an aid to assessing the rippability and excavation potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative “test” excavations with the proposed type of excavation equipment for the proposed construction should be considered, so that they may be correlated with the data presented within this report.

SEISMIC LINE LOCATION MAP



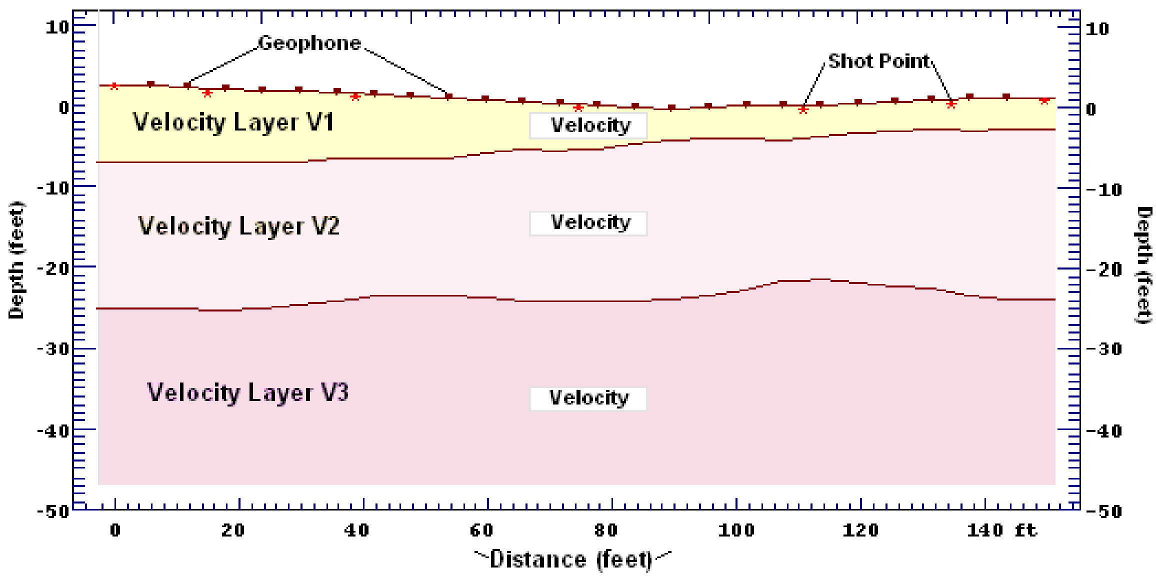
Base Map: Google™ Earth imagery (2019); Seismic traverses shown as yellow lines.

APPENDIX A

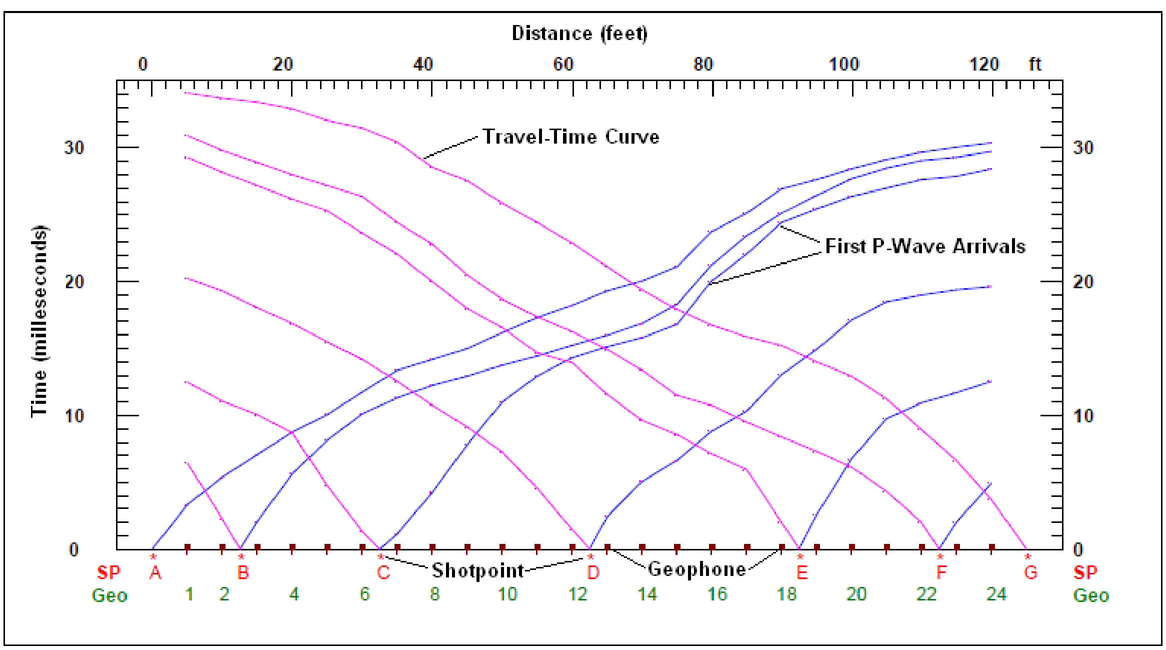
LAYER VELOCITY MODELS

LAYER VELOCITY MODEL LEGEND

LAYER VELOCITY MODEL



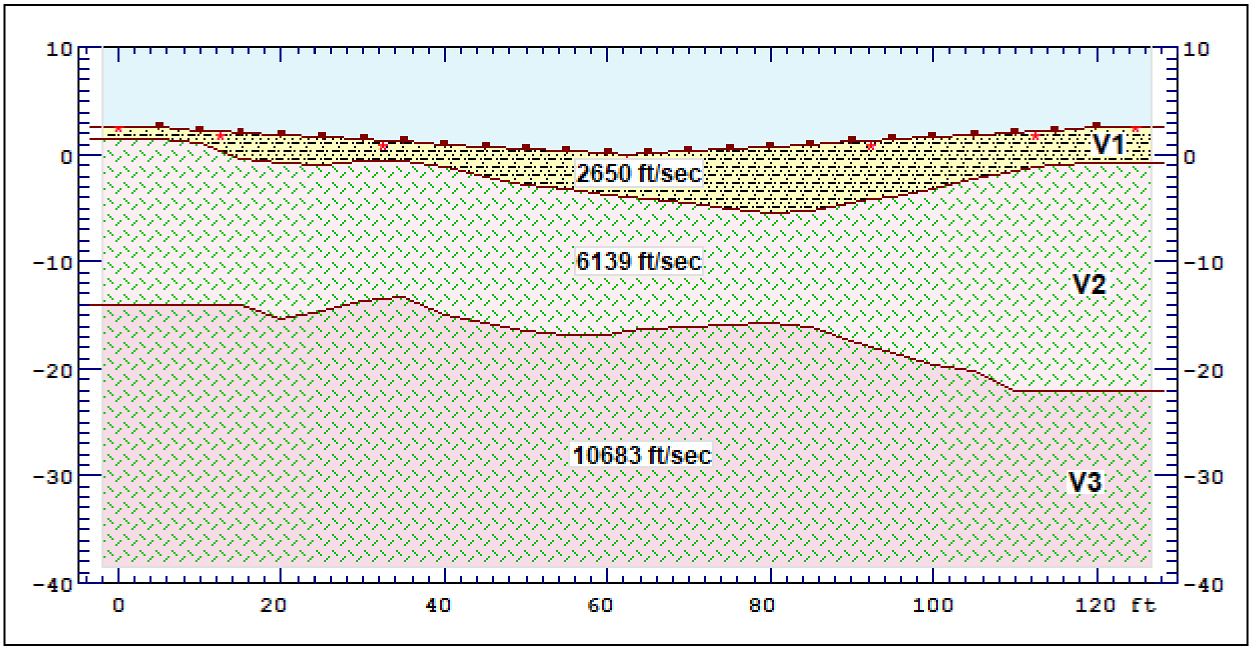
TIME-DISTANCE PLOT



SEISMIC LINE S-1

South 57° East >

LAYER VELOCITY MODEL



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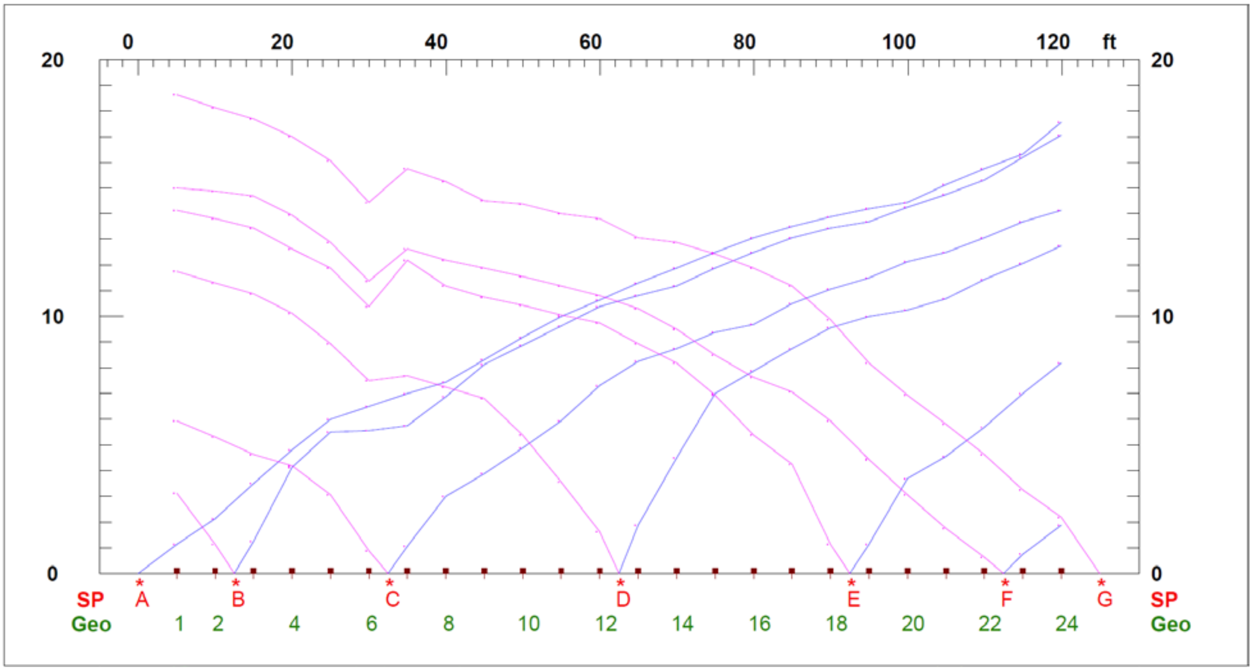
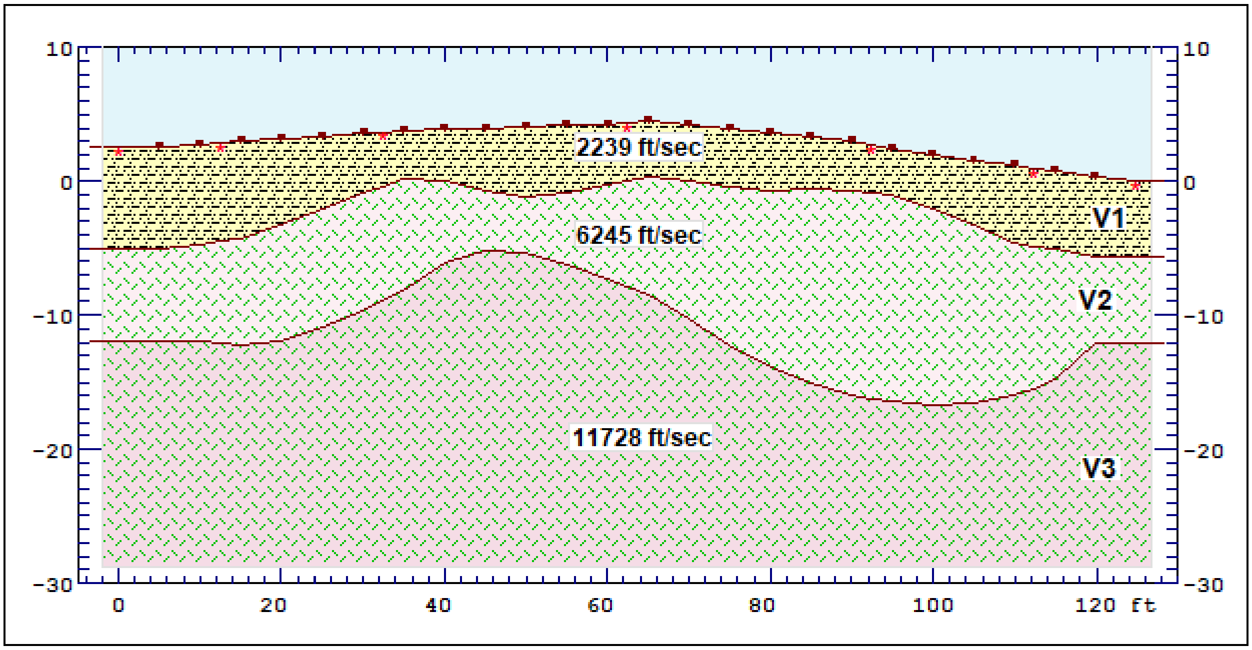


SEISMIC LINE S-2

< West - East >

LAYER VELOCITY MODEL



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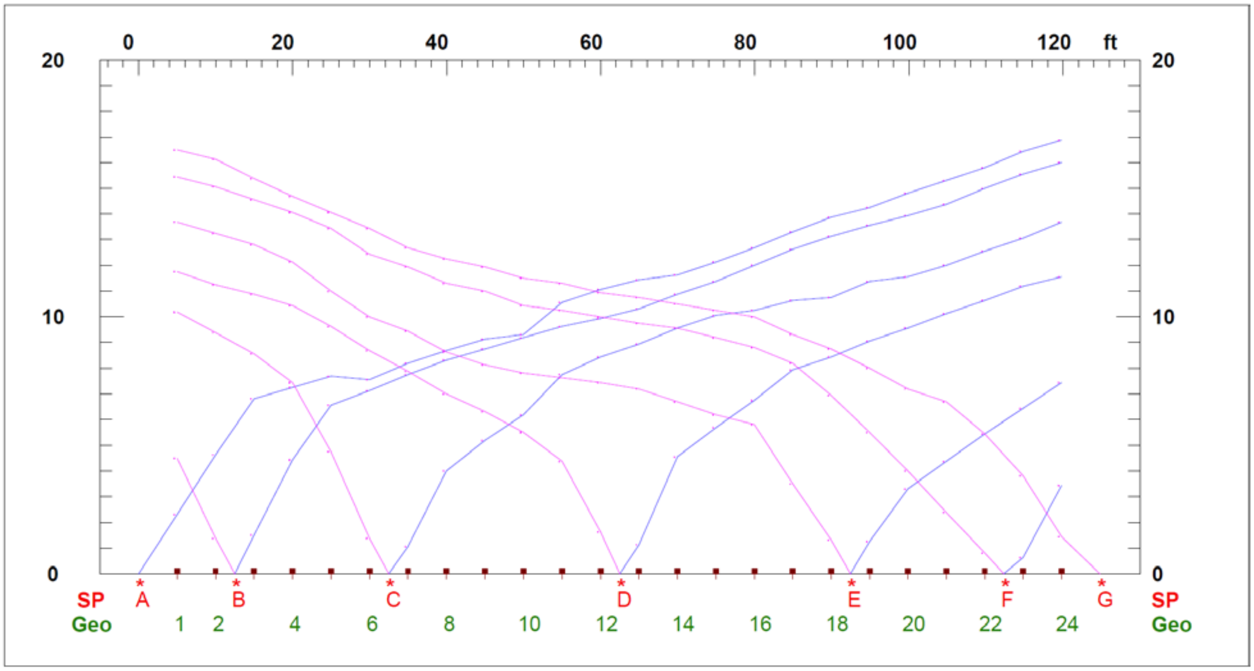
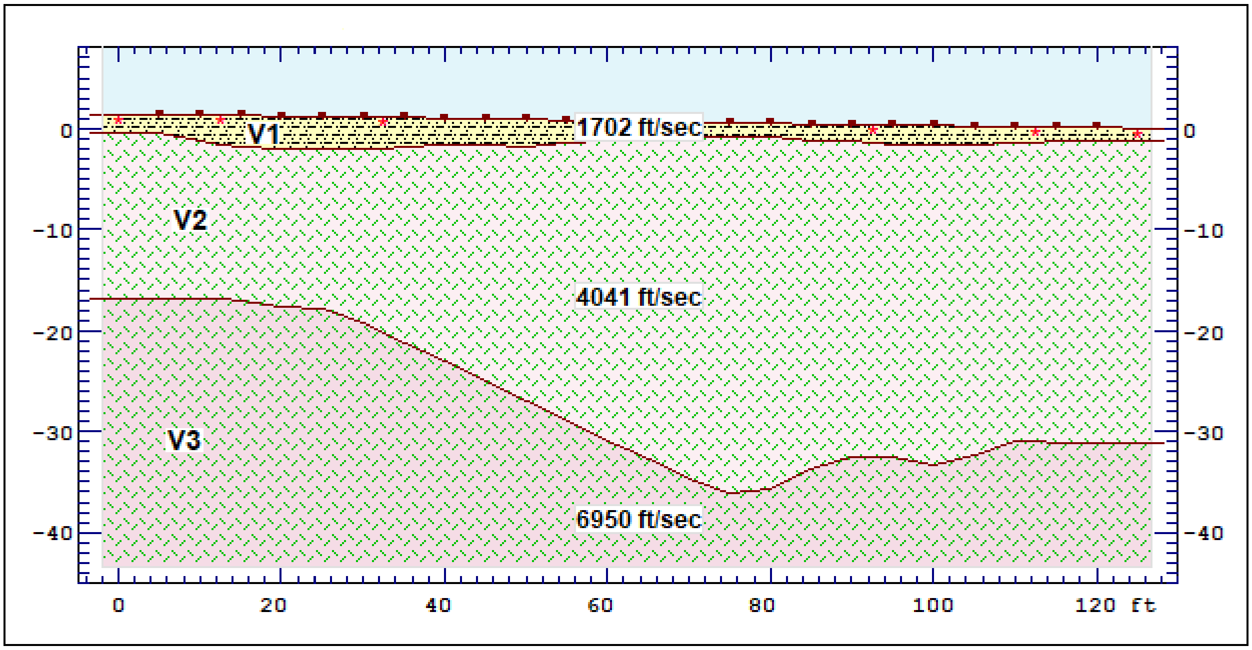


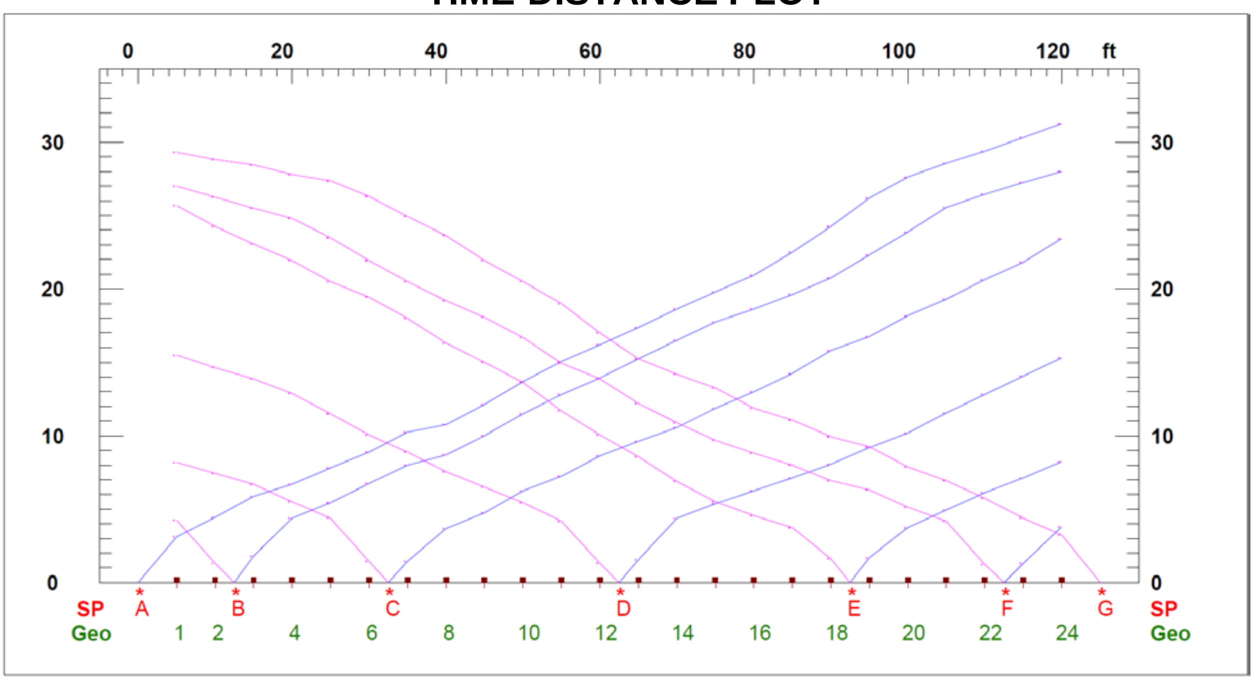
SEISMIC LINE S-3

< North - South >

LAYER VELOCITY MODEL



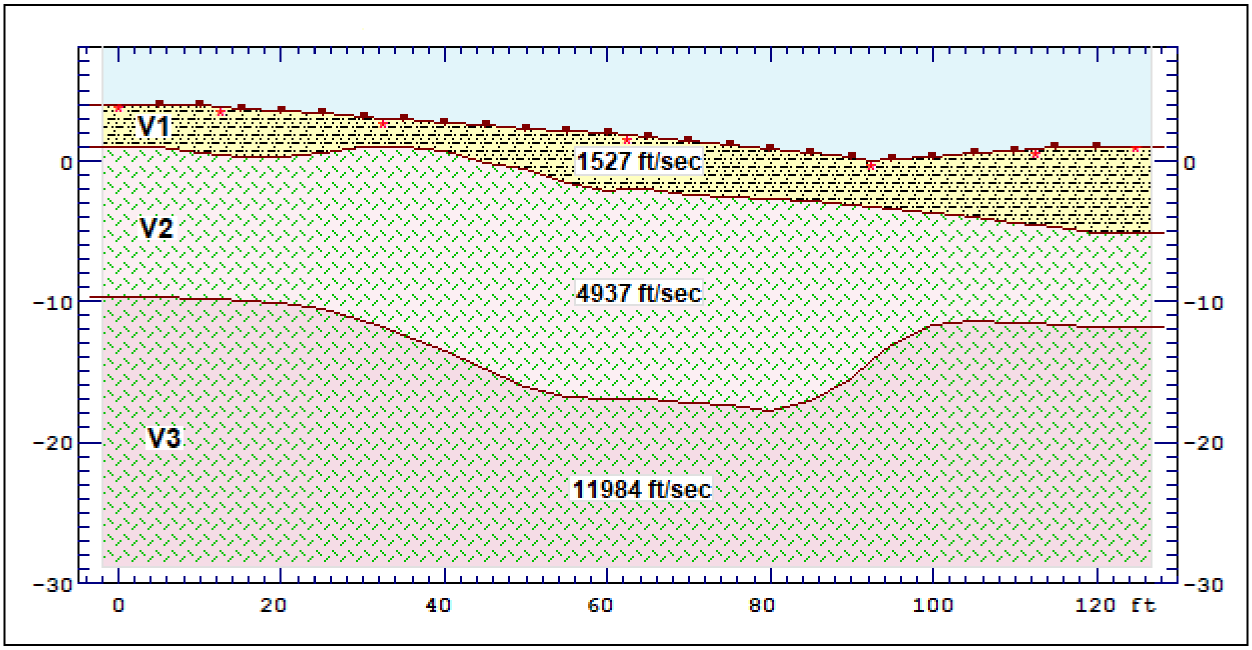
TIME-DISTANCE PLOT



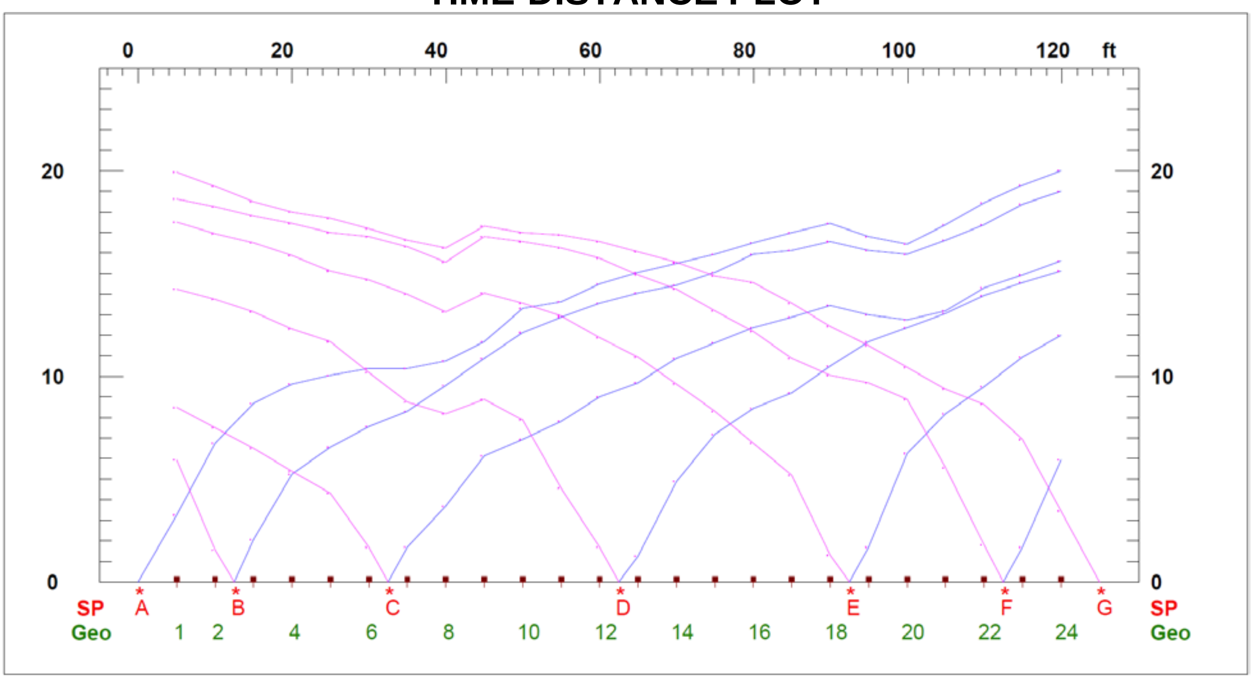
SEISMIC LINE S-4

< North - South >

LAYER VELOCITY MODEL



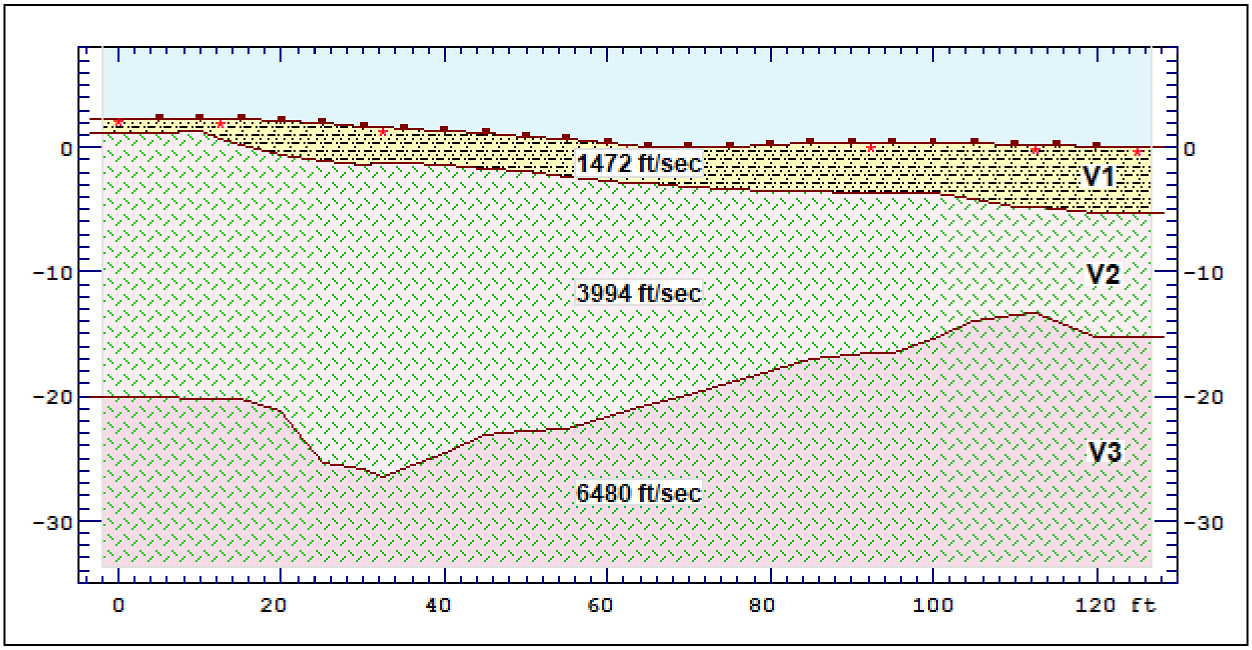
TIME-DISTANCE PLOT



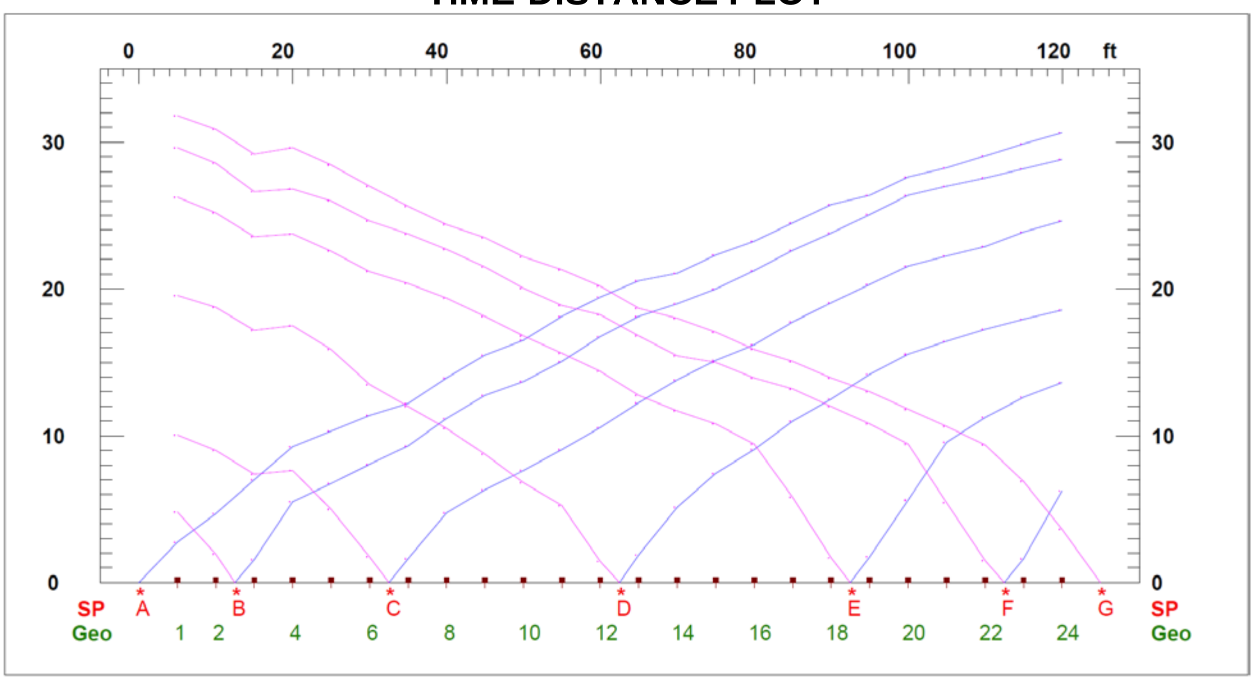
SEISMIC LINE S-5

< North - South >

LAYER VELOCITY MODEL



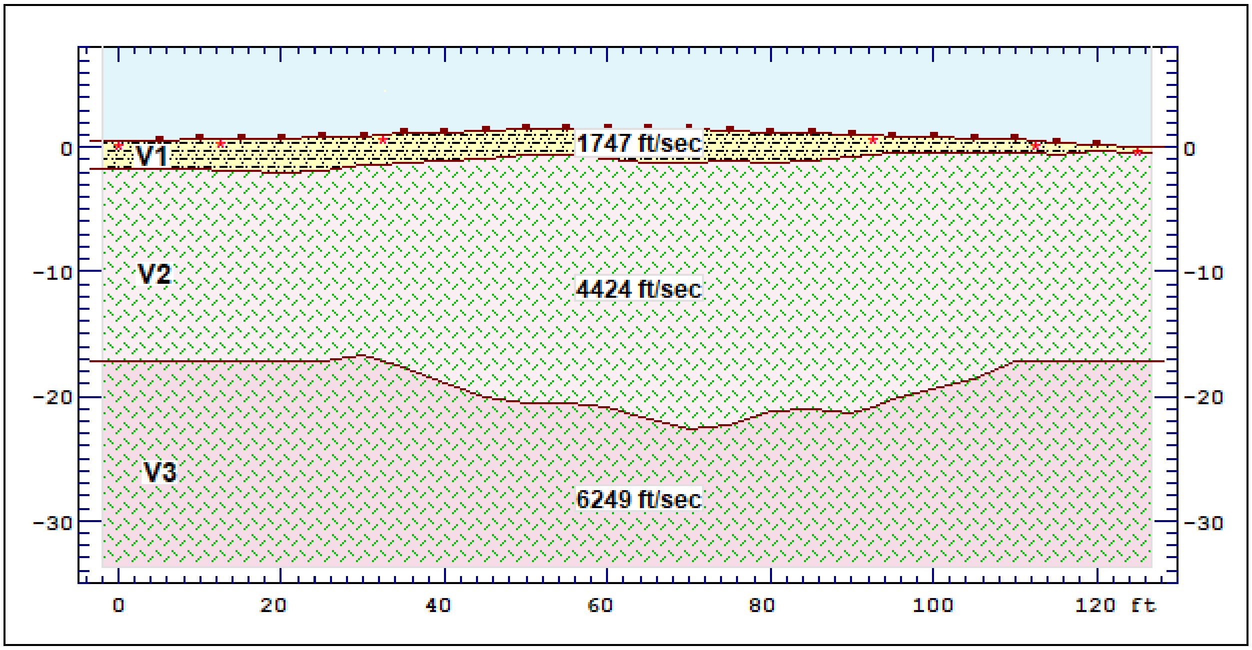
TIME-DISTANCE PLOT



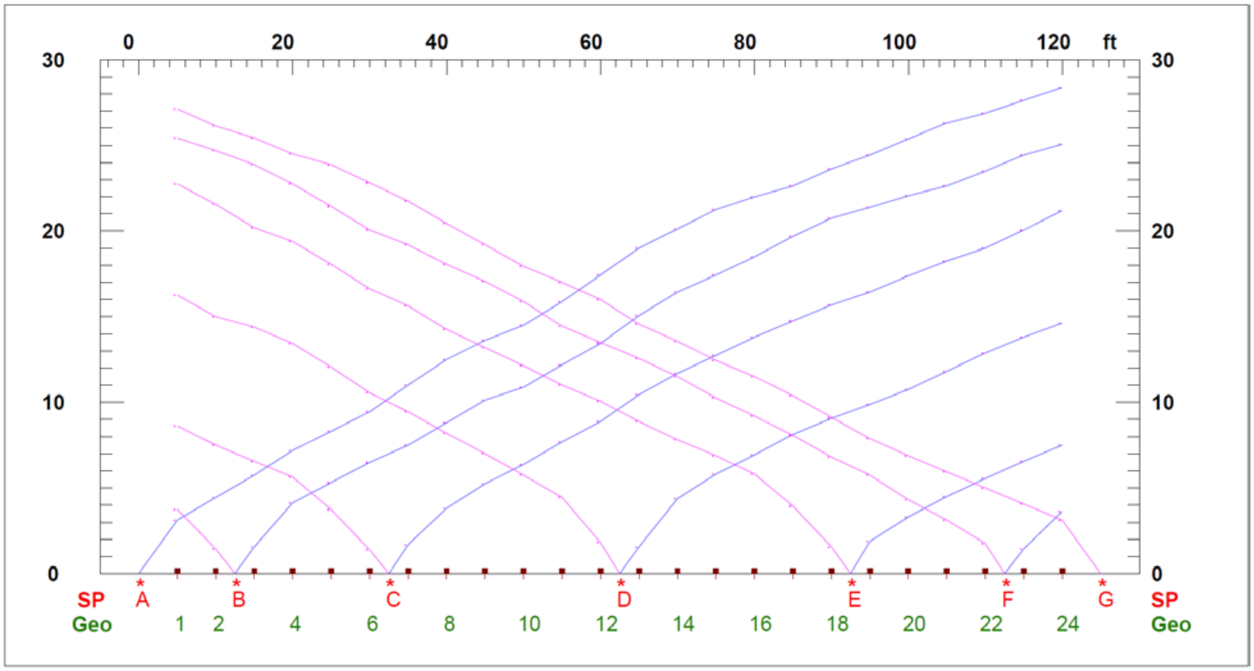
SEISMIC LINE S-6

< West - East >

LAYER VELOCITY MODEL



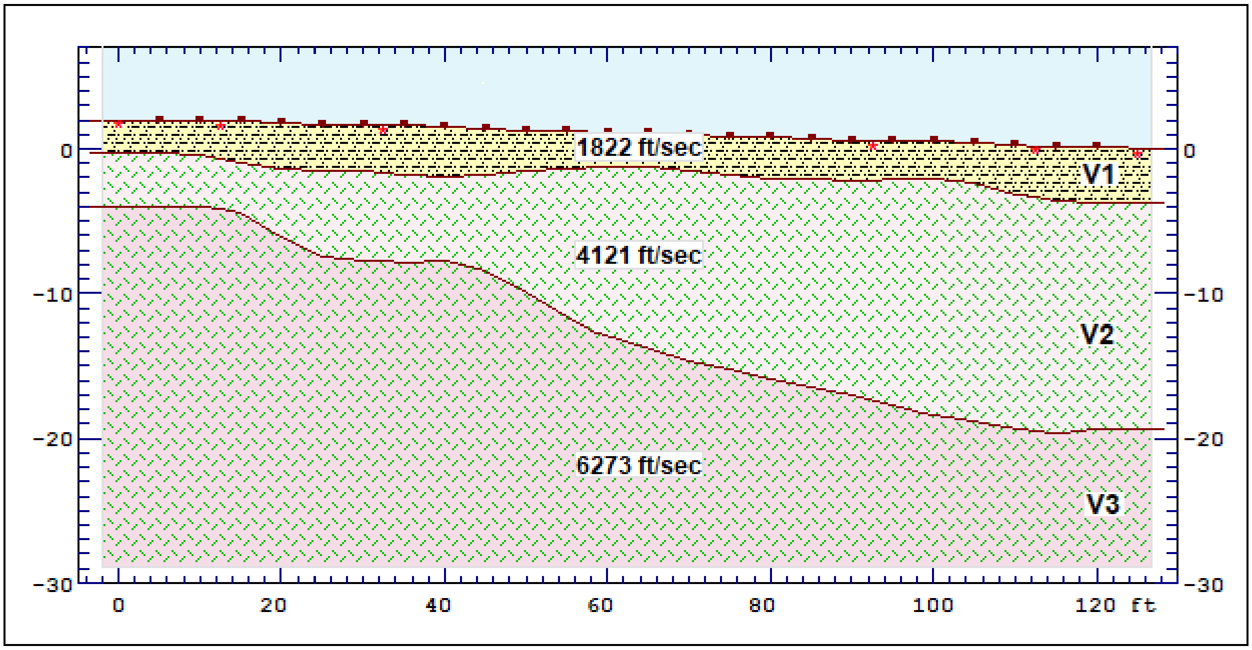
TIME-DISTANCE PLOT



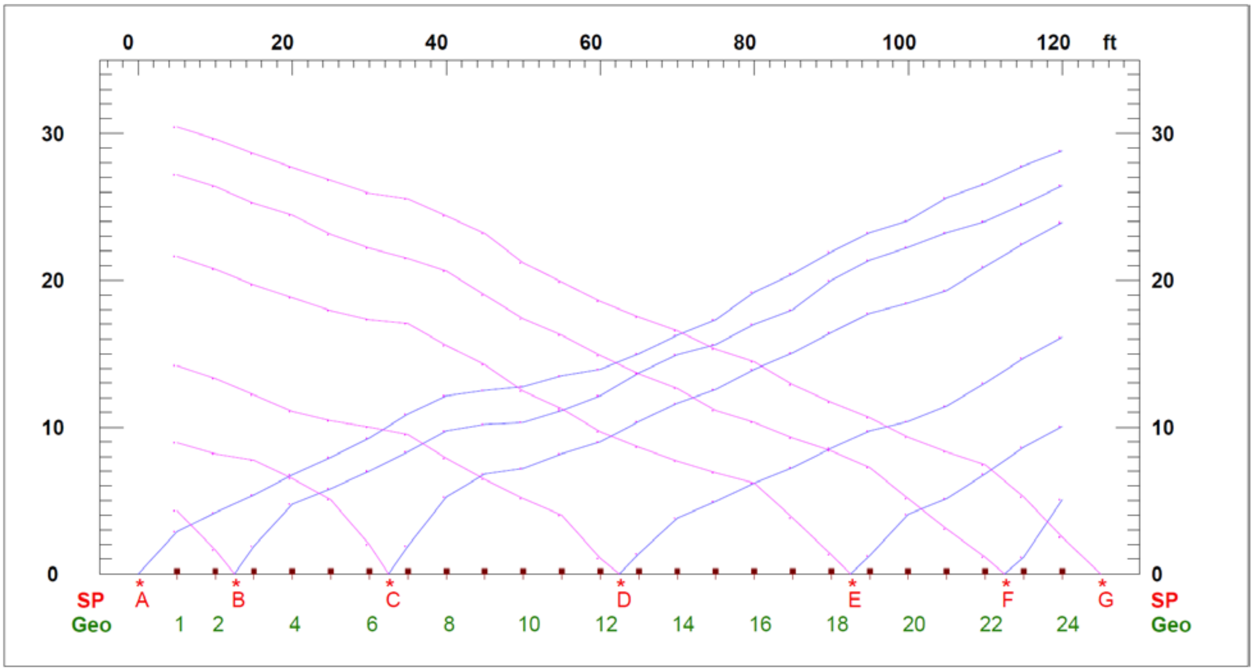
SEISMIC LINE S-7

< North - South >

LAYER VELOCITY MODEL



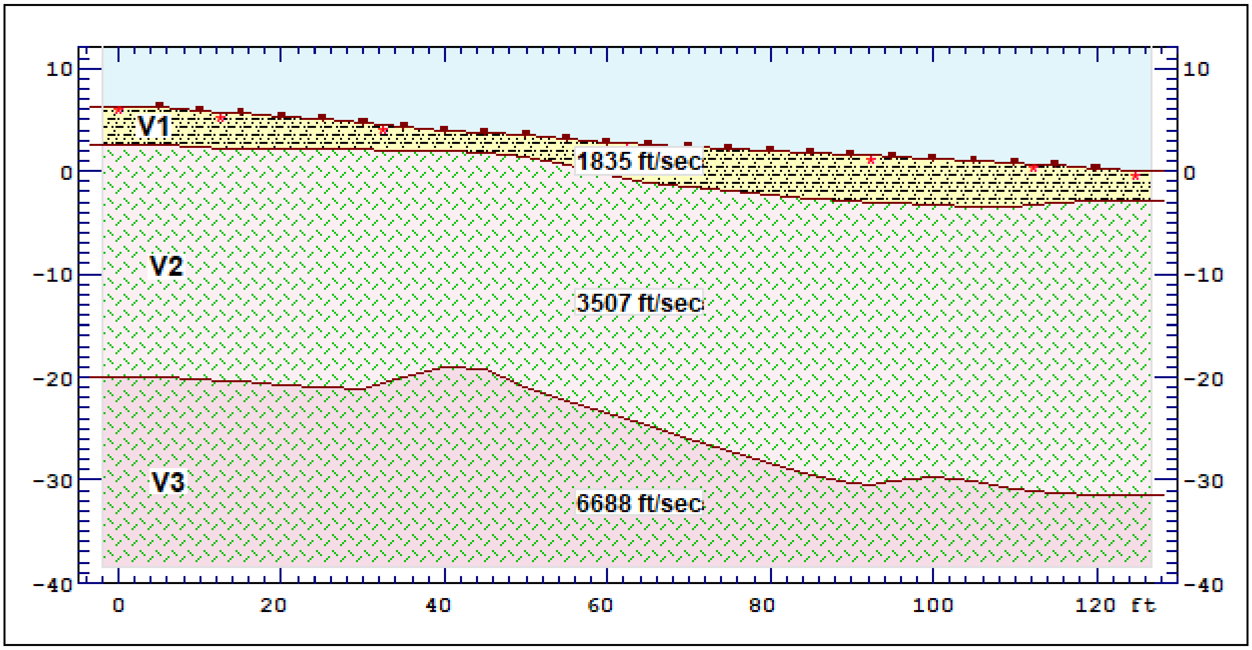
TIME-DISTANCE PLOT



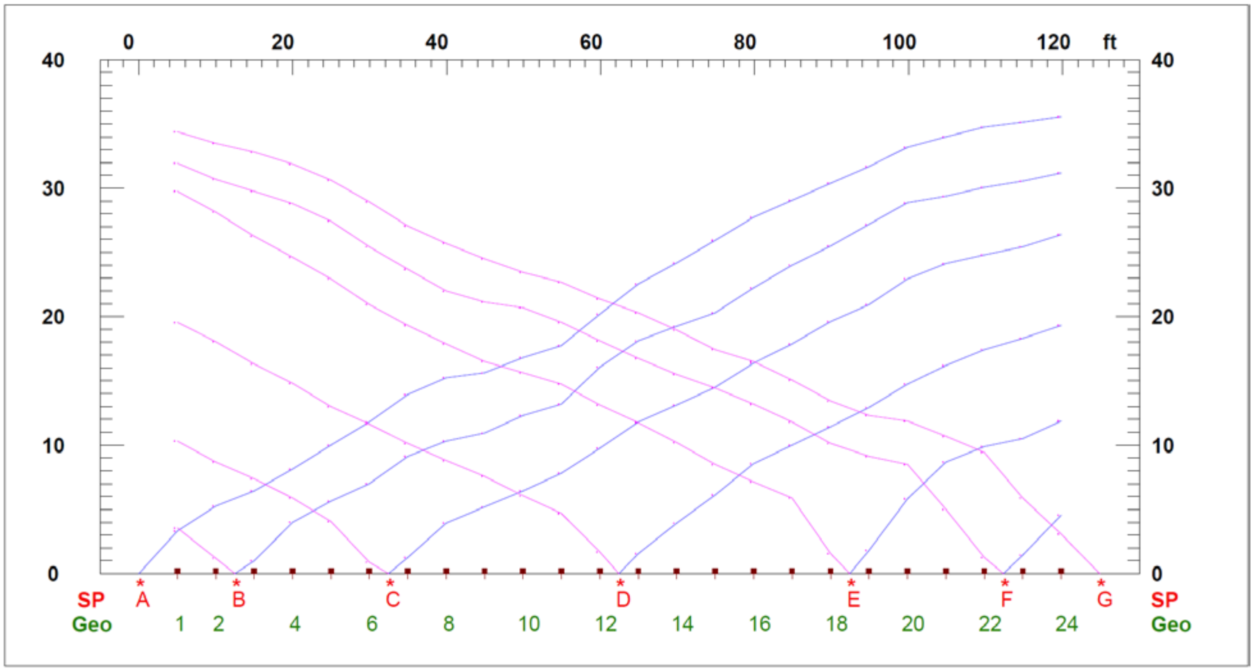
SEISMIC LINE S-8

< West - East >

LAYER VELOCITY MODEL



TIME-DISTANCE PLOT

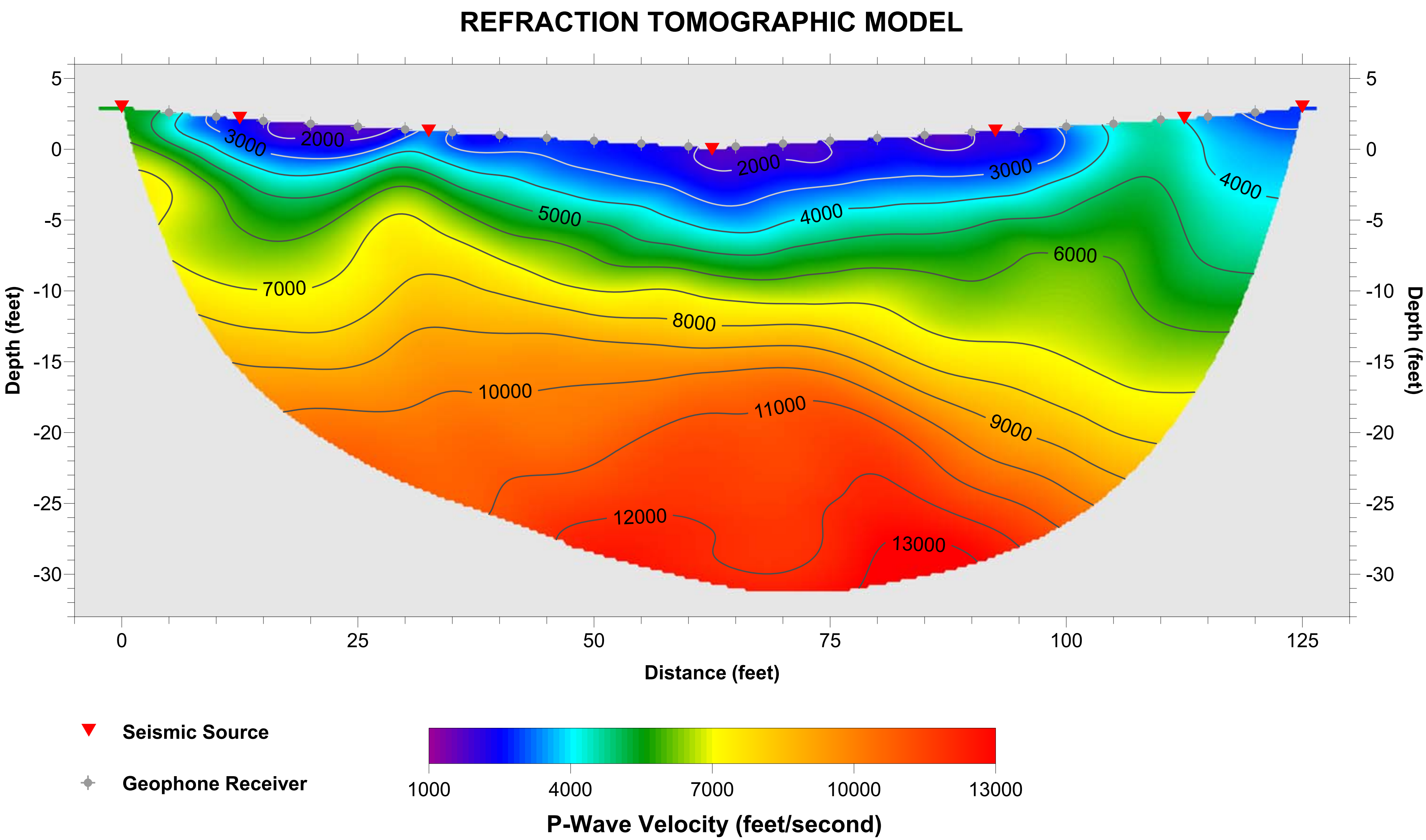


APPENDIX B

REFRACTION TOMOGRAPHIC MODELS

SEISMIC LINE S-1

South 57° East →



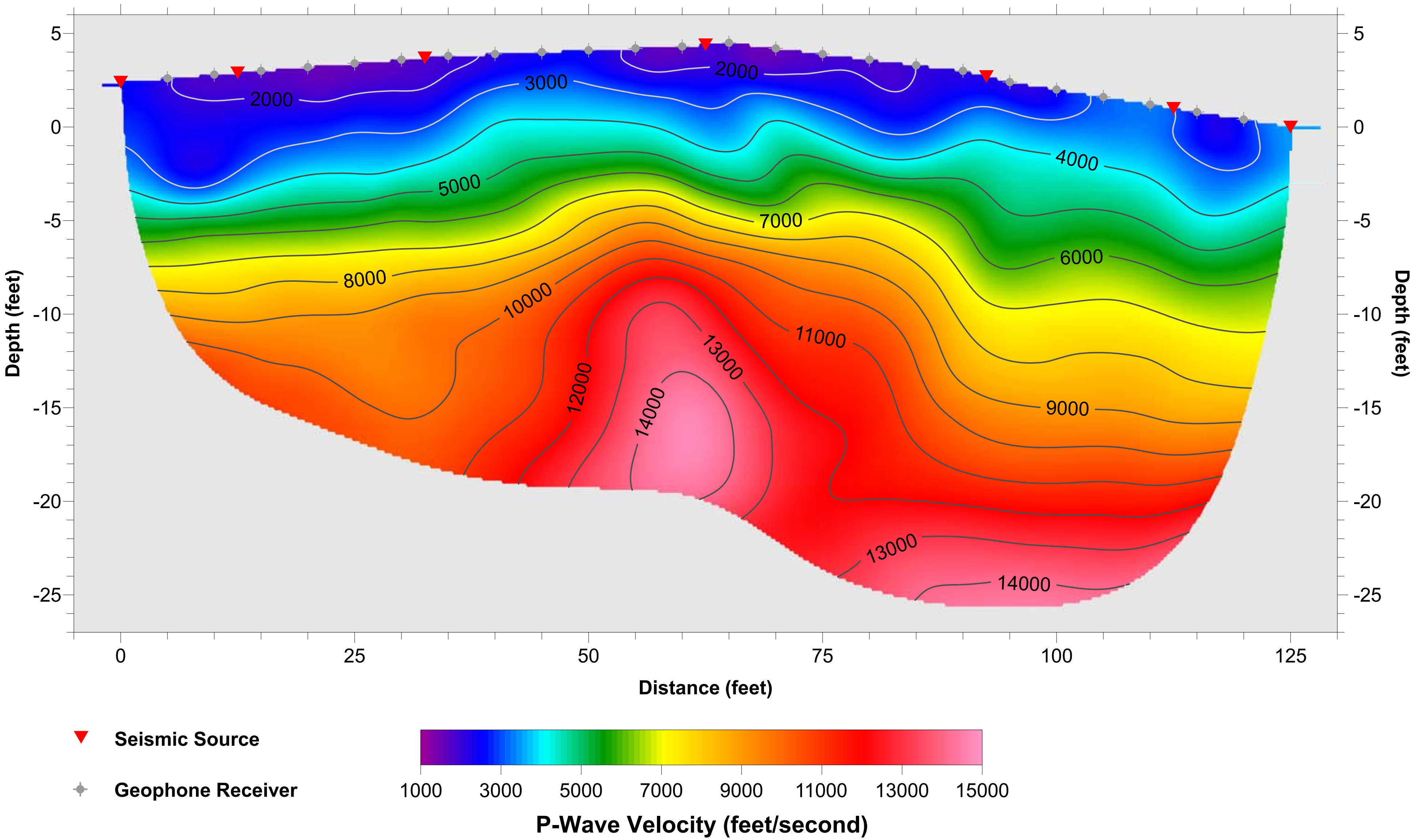
SCALE: Vertical Exaggeration 1.5X

RMS error 4.3%; Rayfract Version 3.36

SEISMIC LINE S-2

< West - East >

REFRACTION TOMOGRAPHIC MODEL



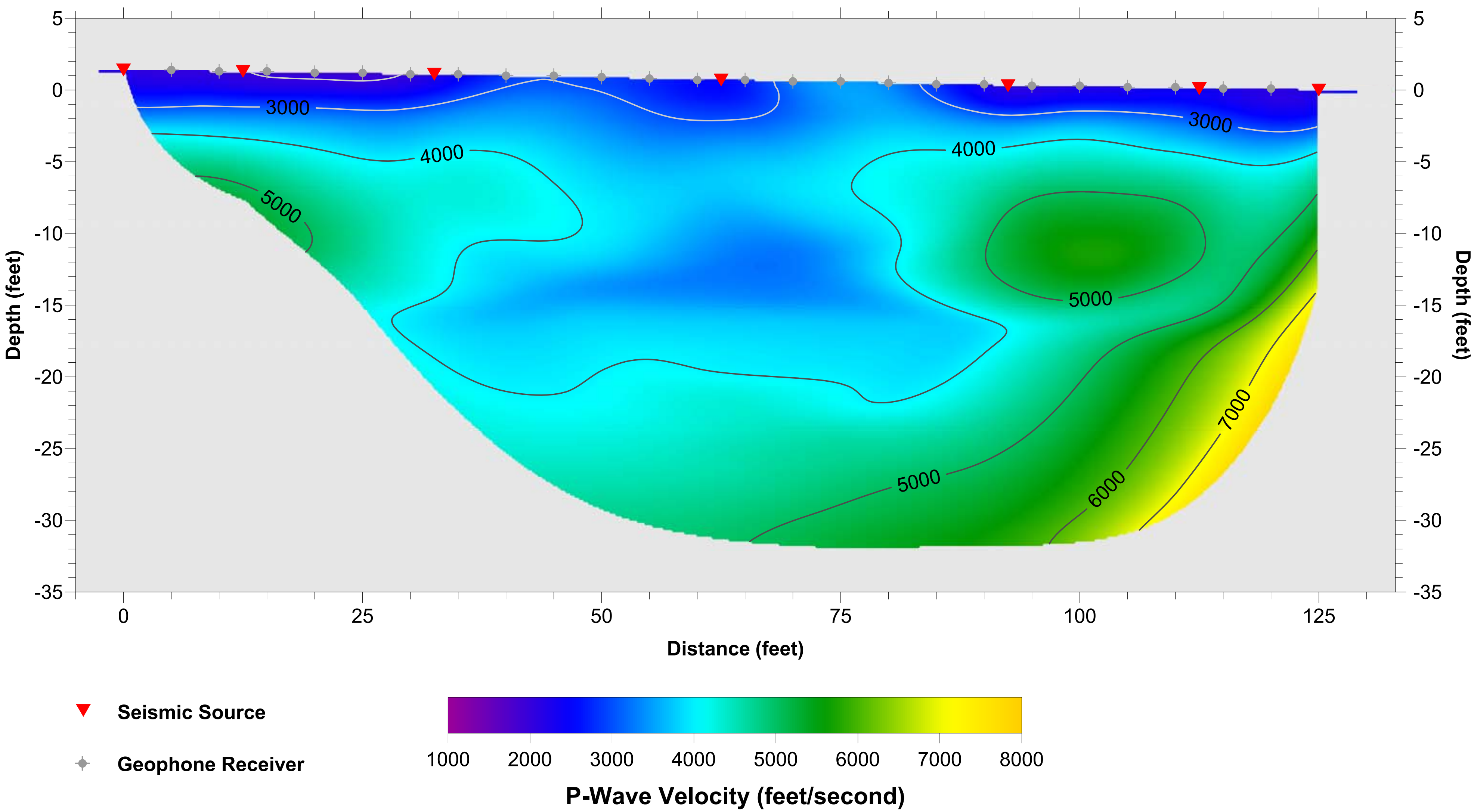
SCALE: Vertical Exaggeration 2X

RMS error 3.9%; Rayfract Version 3.36

SEISMIC LINE S-3

< North - South >

REFRACTION TOMOGRAPHIC MODEL



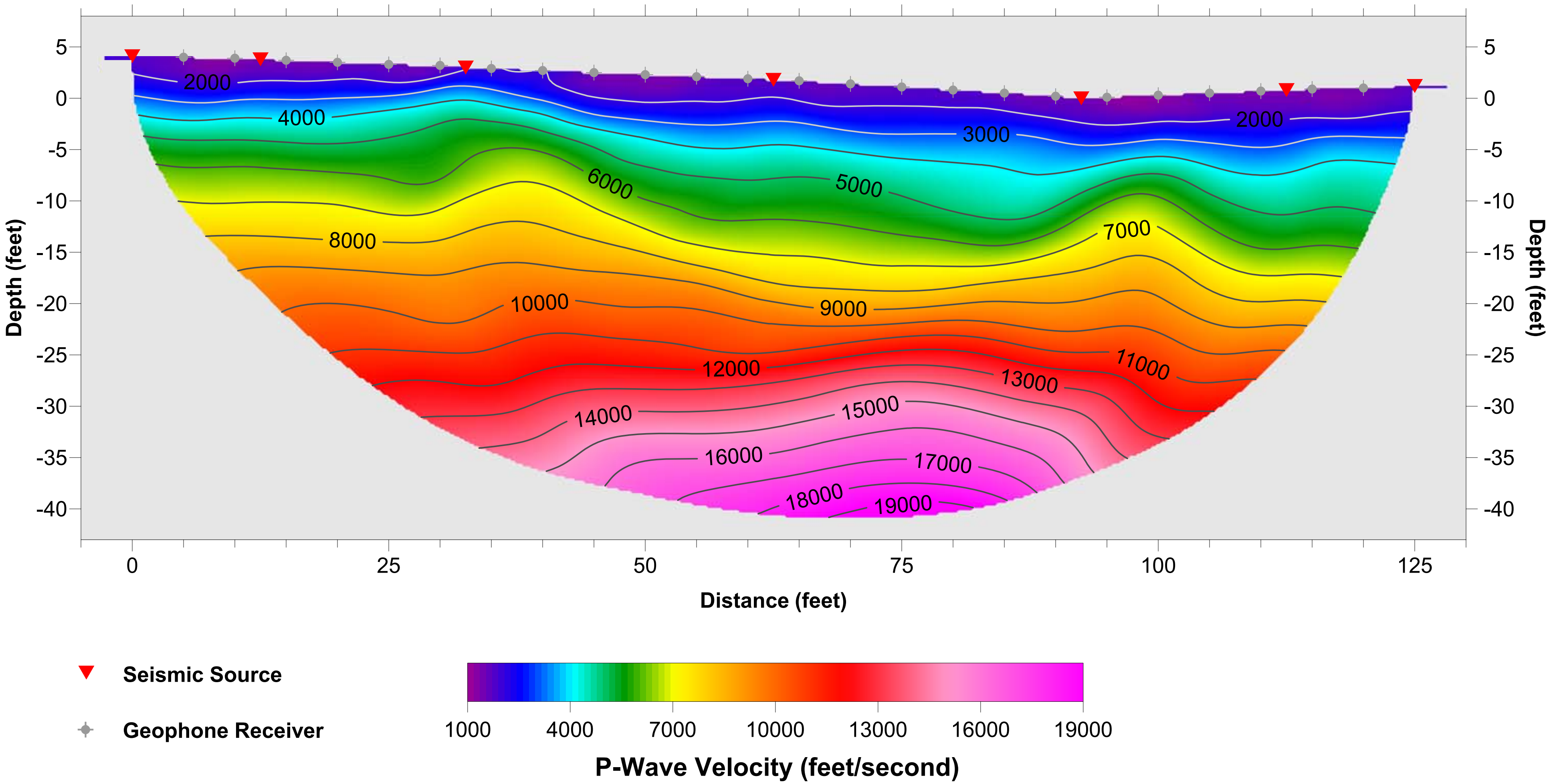
SCALE: Vertical Exaggeration 1.5X

RMS error 4.9%; Rayfract Version 3.36

SEISMIC LINE S-4

< North - South >

REFRACTION TOMOGRAPHIC MODEL



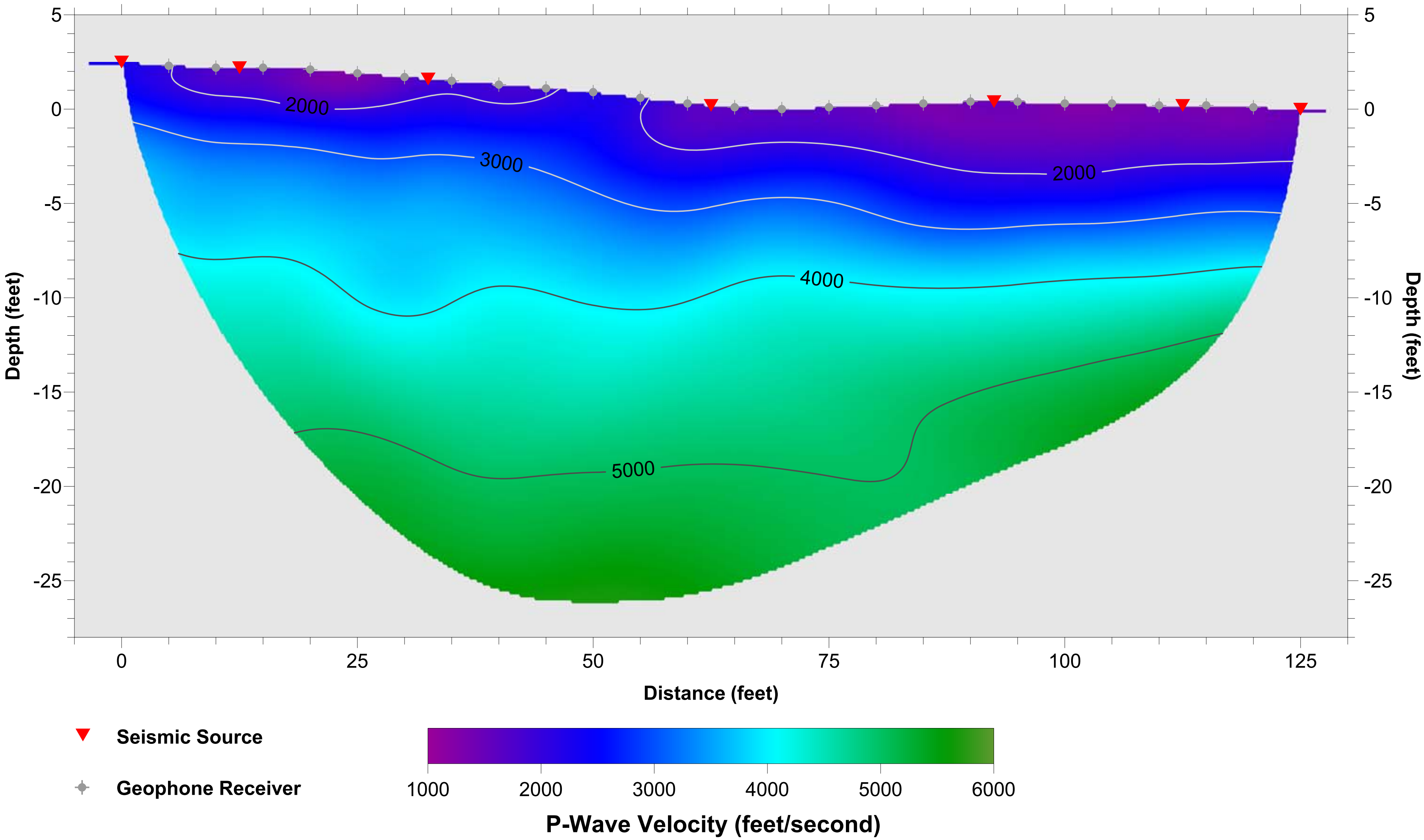
SCALE: 1:1 (Horizontal = Vertical)

RMS error 3.4%; Rayfract Version 3.36

SEISMIC LINE S-5

< North - South >

REFRACTION TOMOGRAPHIC MODEL



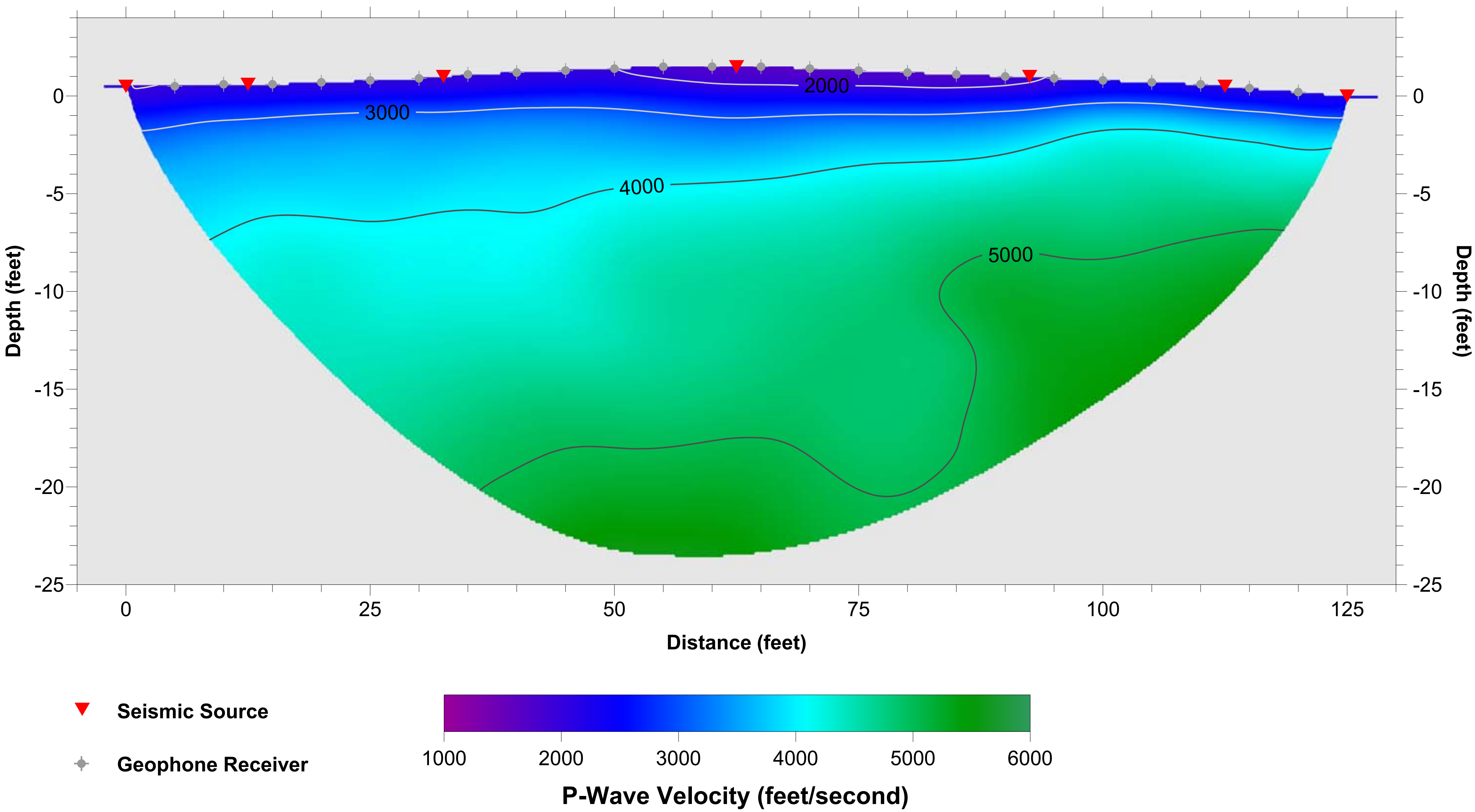
SCALE: Vertical Exaggeration 2X

RMS error 2.7%; Rayfract Version 3.36

SEISMIC LINE S-6

< West - East >

REFRACTION TOMOGRAPHIC MODEL



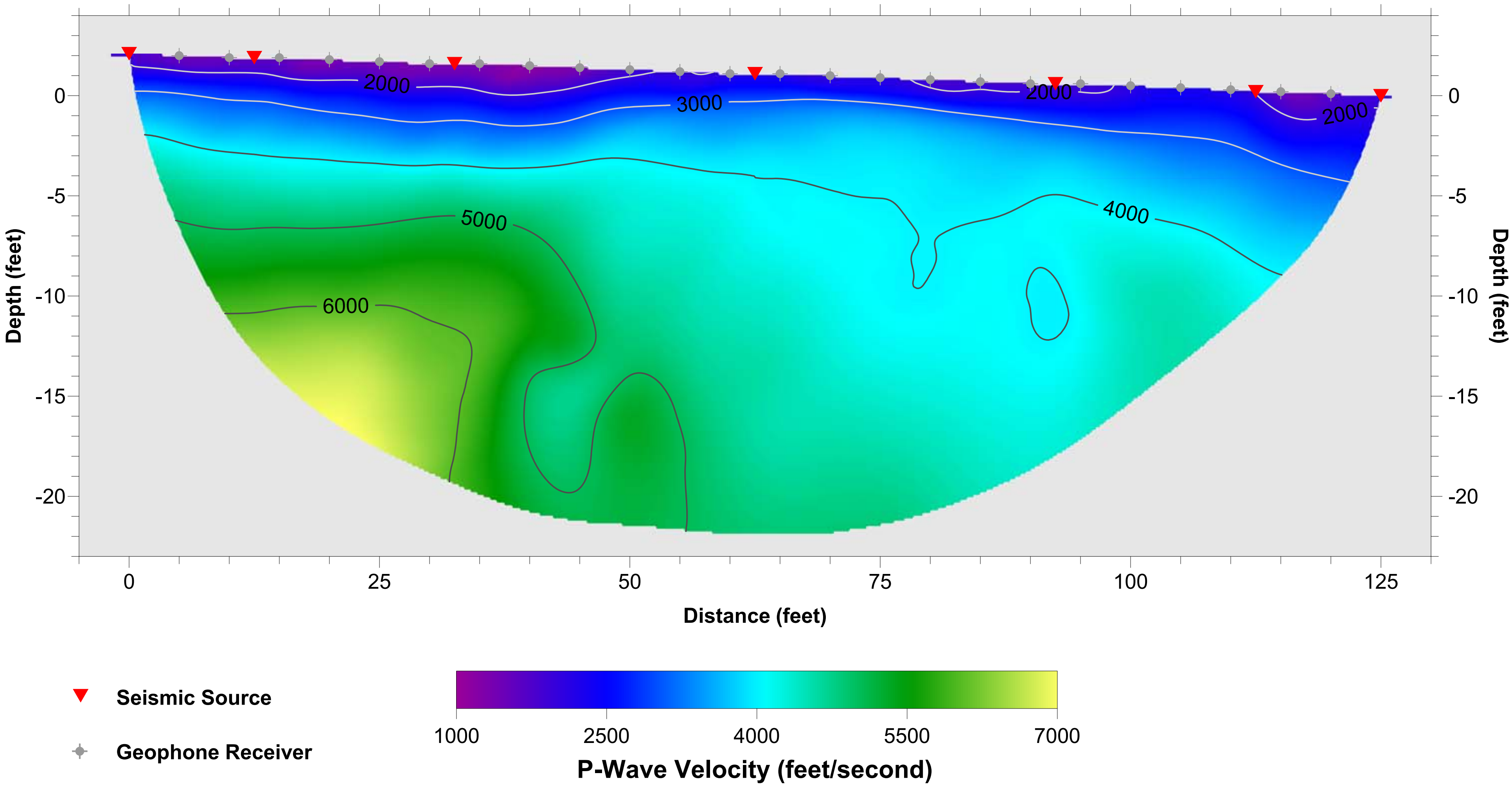
SCALE: Vertical Exaggeration 2X

RMS error 2.7%; Rayfract Version 3.36

SEISMIC LINE S-7

< North - South >

REFRACTION TOMOGRAPHIC MODEL



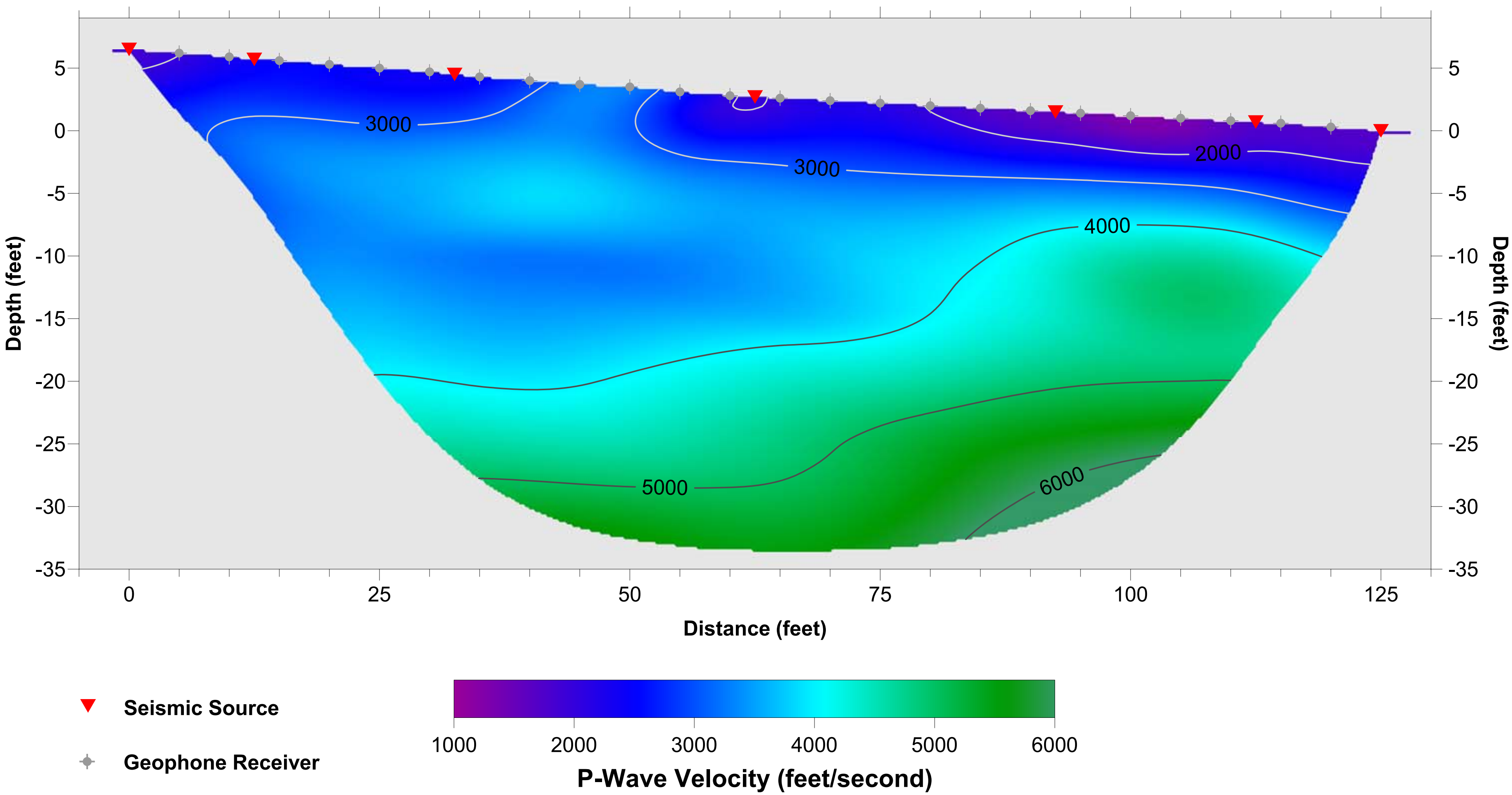
SCALE: Vertical Exaggeration 2X

RMS error 3.1%; Rayfract Version 3.36

SEISMIC LINE S-8

< West - East >

REFRACTION TOMOGRAPHIC MODEL



SCALE: Vertical Exaggeration 1.25X

RMS error 2.9%; Rayfract Version 3.36

APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the client's responsibility to ensure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ***Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,***

- ***Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,***

- ***Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.***

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also, important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to ensure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX D

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REFERENCES

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