ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

July 7, 2015
J.N.: 2265.00

Mr. Bryon Ely
National Community Renaissance
National CORE
9065 Haven Avenue, Suite 100
Rancho Cucamonga, CA 91730

Subject: Preliminary Geotechnical Investigation, Proposed Multi- Family Residential Development "Villa Serena Apartments", 339 and 340 Marcos Street, San Marcos, California.

Dear Mr. Ely;

Pursuant to your request, ***Albus-Keefe & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report for the proposed residential development at the subject site. This report presents the results of our review of the referenced literature and surrounding areas, subsurface exploration, seismic refraction survey, laboratory testing and engineering/geologic analyses. Geotechnical conclusions and recommendations pertaining to the proposed site development are also provided herein.

We appreciate this opportunity to be of service to you. If you have any questions regarding the contents of this report, please do not hesitate to call this office.

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

David E. Albus
Principal Engineer

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purposes of this investigation were to evaluate the geotechnical conditions within the project area and to provide preliminary conclusions and recommendations relevant to design and construction of the proposed residential development at the site. The scope of this investigation included the following:

- Review of the referenced plans
- Review of readily available geologic and seismic data for the site and surrounding area
- Exploratory drilling and soil sampling
- Seismic Refraction Surveying
- Laboratory testing of selected soil samples
- Engineering and geologic analyses of data obtained from our review, exploration and laboratory testing
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site consists of two rectangular-shaped properties located at 339 and 340 Marcos Street, in the city of San Marcos, California. The property located at 339 Marcos Street is bordered by Richmar Avenue on the south, Marcos Street to the west, residential properties to the north and by a Fitzpatrick Road to the east. The property located at 340 Marcos Street is bordered by Richmar Avenue on the south, Liberty Drive to the east, residential properties to the north and by Marcos Street to the east. The location of the site and its relationship to the surrounding areas is shown on Figure 1, Site Location Map.

The property located at 339 Marcos Street is currently occupied by an apartment complex with multi-story buildings, an asphalt paved parking lot, concrete walkways, a playground and various residential improvements. Perimeter block walls and fencing, as well as overhead power lines, are present along the northern margin of the property. Topography within the property generally slopes down from the north to the south. Elevations within the property vary from approximately 617 feet above mean sea level (MSL) to 592 feet (MSL). Vegetation is present in localized areas of the property and consists primarily of landscaped ground cover, shrubs, and trees.

The property located at 340 Marcos Street is also currently occupied by an apartment complex with multi-story buildings, an asphalt paved parking lot, concrete walkways, a playground, a swimming pool and various other residential improvements. Overhead power lines and an ascending graded

2:1 (h:v) slope up to approximately 8 feet in height are also present along the northerly margin of the site. Topography within the property generally slopes down from the north to the south. Elevations within the property vary from approximately 608 feet above mean sea level (MSL) to 592 feet (MSL). Vegetation is present in localized areas of the property and consists primarily of landscaped shrubs, trees and ground cover.

1.3 PROPOSED DEVELOPMENT

Based on our review of the referenced architectural site plans site plans, the two properties are to be completely demolished and then redeveloped for multifamily residential use. The proposed development for the property at 339 Marcos Street, (Referred to as Phase 2 on the plans) will generally involve the construction of two buildings over a podium parking structure. The parking structure will be one subterranean level. The buildings will be an additional 3 stories in height above the podium deck and consist of a total of 63 dwelling units. Surface parking is also proposed. Based on the conceptual grading plans, cut and fill grading, minor slope construction and retaining wall construction will be required to achieve the desired grades. Proposed cuts and fills are generally less than 8 feet. The most significant cuts (approximately 8 feet) are for the subterranean garage.

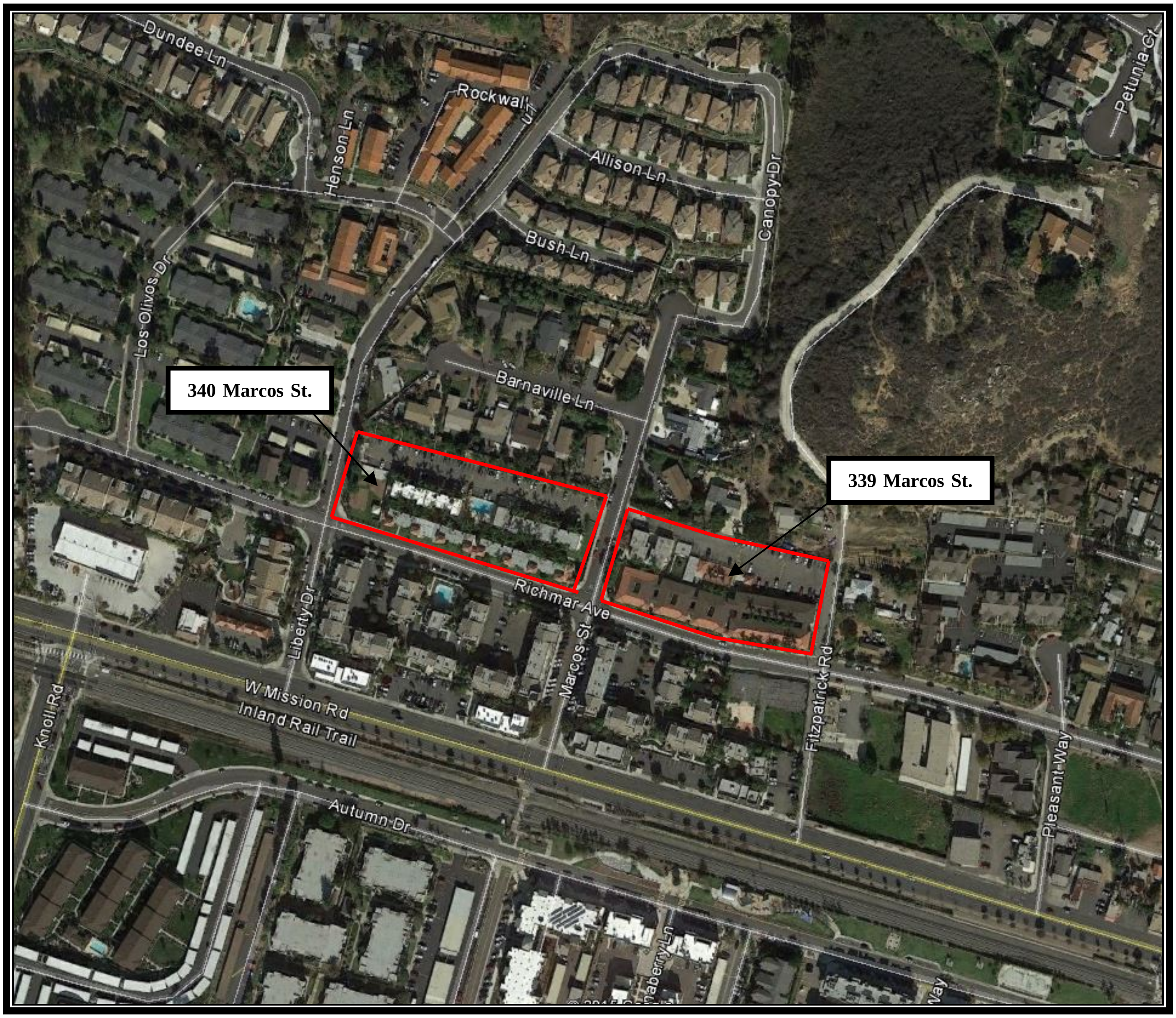
The proposed development for the property at 340 Marcos Street (Referred to as Phase 1 on the plans) will generally involve the construction of two buildings and a separate parking structure. The buildings will be 3 stories in height and will consist of a total of 85 dwelling units. The parking structure will be 2 stories. The rear of the parking structure is anticipated to have one level of subterranean while the front will be at grade. Based on the conceptual grading plans, cut and fill grading, minor slope construction and retaining wall construction will be required to achieve the desired grades. Proposed cuts and fills are generally less than 10 feet. The most significant cuts (up to approximately 10 feet) are for the parking structure at the rear of the complex.

Details of the residential and parking structures are not available at this time. However, we anticipate residential structures will utilize Type V wood-frame construction that will yield relatively light loads. The parking structures are anticipated to utilize poured-in-place concrete and CMU wall construction. Column and wall loads for the parking structures are anticipated to be moderate (up to 100 kips and 6 kips/ft).

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications, and maps for the site and nearby vicinity (see references). Relevant data from our review was utilized to develop some of the conclusions and recommendations presented herein.



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SITE LOCATION MAP

Proposed Multi- Family Residential Development
"Villa Serena Apartments"
339 and 340 Marcos Street, San Marcos, California

NOT TO SCALE

FIGURE 1

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on March 17, 2014. Subsurface exploration consisted of drilling a total of five (5) exploratory borings within the parking lots located in the northerly portions of the two properties. The borings were drilled to about 8 to15.5 feet below the existing ground surface utilizing a truck-mounted, hollow-stem-auger drill rig. Representatives of Albus-Keefe & Associates, Inc. logged the exploratory excavations. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented on the Exploration Logs in Appendix A. The approximate locations of the exploratory borings completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

Bulk, relatively undisturbed and Standard Penetration Test (SPT) samples were obtained at selected depths within the exploratory borings. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained using a standard, unlined SPT soil sampler. During each sampling interval, the sampler was driven 18 inches with successive drops of a 140-pound automatic hammer free falling approximately 30 inches. The number of blows required to advance the split-spoon and SPT samplers was recorded for each six inches of advancement. A representative “blow count” for each sample is recorded on the exploration logs. Samples were placed in sealed containers or plastic bags and transported to our laboratory for analyses and testing. Upon completion of sampling, the borings were backfilled with soil cuttings.

2.3 SEISMIC REFRACTION SURVEY

In addition to our exploratory drilling, we conducted a seismic refraction survey within the site on April 21, 2014. The seismic refraction survey was performed at our direction by Terra Geosciences. The purpose of the seismic refraction survey was to gain a better understanding of the subsurface conditions particularly within the southerly portion of the site where conventional drilling was not practical due to existing site improvements and to aid in our evaluation of the rippability characteristics of the underlying bedrock materials. Five seismic traverses (S-1 through S-5) were conducted in the subject property. The approximate locations of the survey lines are shown on the enclosed Geotechnical Map, Plate 1. The seismic refraction profiles (Layer Velocity Models and Refraction Tomographic Models) prepared for this investigation are presented in Appendix C.

2.4 LABORATORY TESTING

Selected samples of representative earth materials encountered during our subsurface exploration were tested in the laboratory. Tests consisted of in-situ moisture content and dry density, maximum dry density and optimum moisture content, expansion index, soluble sulfate content, direct shear strength, grain-size/hydrometer analyses, and Atterberg limits, minimum resistivity, pH value and resistance value (R-Value). Descriptions of laboratory test criteria and a summary of the test results are presented in Appendix B and on the exploration logs in Appendix A.

3.0 GEOLOGIC CONDITIONS

3.1 GEOLOGIC SETTING

The subject site is located within the mountainous interior of northern San Diego County in the Peninsular Ranges Geomorphic province of Southern California. This portion of the province is characterized by uplifted and dissected mid-Cretaceous and Jurassic igneous and metamorphic basement rocks consisting of granitic plutons and mildly metamorphosed sedimentary and volcanic rocks.

Results of our investigation indicate the subject site is underlain by Cretaceous-age granitic bedrock. Unconformably overlying the granitic bedrock is Eocene-age sedimentary rocks of the Santiago Formation. Surficial units consisting of residual soil and artificial fill are present in the near surface. Detailed descriptions of each of the units are provided in the following section.

3.2 GEOLOGIC UNITS

3.2.1 Artificial Fill

Although not encountered in our exploratory boring excavations, artificial fill associated with the current site development (i.e. underground utilities and retaining walls) are likely present within the site. The characteristics and extent of these fills is uncertain.

3.2.2 Residual Soil (No Map Symbol)

Residual soil materials were encountered in the near surface of the site mantling the Santiago Formation. The residual soil materials consist of red-brown sandy clay and clayey sand that is fine to medium grained, moist, medium stiff and/or medium dense, with some local pores. The thickness of the residual soil encountered varies from approximately 2 feet to 4 feet.

3.2.3 Santiago Formation (Tsa)

Sedimentary rock assigned to the Eocene-age Santiago Formation was encountered in our exploratory excavations within the northerly portion of the site. This unit consists of weakly-cemented, reddish brown to light gray (on less weathered exposures) fine- to coarse-grained clayey sandstone to sandstone. These materials are generally moist, moderately hard to hard, massive, and moderately weathered with some pinhole pores in the near surface. Some gravel and badly weathered granitic bedrock clasts were also noted in this unit. Results of our seismic refraction survey indicate that this unit (interpreted as velocity layer V2 in the Layer Velocity Models) underlies the southerly portion of the site as well and maybe locally as much as 42 feet in thickness.

Based on review of previous work by this firm on a nearby site to the northwest (Albus-Keefe 2002 & 2009), the formation is massive with poorly-developed joints that strike at various random orientations and dip from 28 to 80 degrees. Our previous work did not indicate any preferential joint patterns and the joints tend to be widely spaced.

3.2.4 Granitic Bedrock (Kgr)

Cretaceous-age “Granitic Bedrock” was encountered at depth within all of our exploratory borings and generally resulted in refusal. The granitic rock materials are generally light brown, fine- to coarse-grained, damp, and very hard. Based on our subsurface exploration and interpretation of the seismic refraction survey data, the depth to granitic bedrock beneath the upper northerly portion of the site generally varies from 8 feet to 15 feet (bgs). Beneath the lower southerly margin of the site, along Richmar Drive, the depth to granitic bedrock generally varies from 14 feet to possibly as much as 42 feet (bgs).

Based on review of previous work by this firm on a nearby site to the northwest (Albus-Keefe 2002 & 2009), the formation exhibits joints that trend northeast and northwest. The joints are medium- to widely-spaced (8 inches to 6 feet) and are steeply dipping.

3.3 FAULTING

Evidence of active faulting within and adjacent the site was not encountered during this investigation. The site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active faults are the Newport Inglewood Connected alt 1 & 2 faults located approximately 12.4 miles (19.7 km) from the site. A summary of active faults located within 20 miles of the site are summarized in Table 3.1.

TABLE 3.1
Summary of Active Faults

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Newport Inglewood Connected alt 1	12.35	1.3	89	strike slip	0	208
Newport Inglewood Connected alt 2	12.35	1.3	90	strike slip	0	208
Rose Canyon	12.35	1.5	90	strike slip	0	70
Newport-Inglewood (Offshore)	15.09	1.5	90	strike slip	0	66
Elsinore;GI+T	16.36	5	90	strike slip	0	78
Elsinore;GI+T+J	16.36		86	strike slip	0	153
Elsinore;GI+T+J+CM	16.36		86	strike slip	0	195
Elsinore;J	16.36	3	84	strike slip	0	76

3.4 LANDSLIDES AND ROCKFALLS

No evidence of landslides or major rock falls were identified within or adjacent the subject site.

3.5 GROUNDWATER

Groundwater or seepage was not encountered in our exploratory excavations to the depths explored during this investigation (15.5 feet). We were unable to locate any published data concerning historical ground water levels near the site.

4.0 ANALYSES

4.1 SEISMICITY

We have performed probabilistic seismic analyses utilizing the web-based U.S. Seismic Design Maps web application by the U.S. Geological Survey (USGS). From this we obtain a PGA of 0.381 in accordance with Figure 22-7 of ASCE 7-10. The F_{PGA} factor for site class C is 1.019. Therefore, the PGA_M = 1.019 x 0.381 = 0.388g. The mean event associated with a probability of exceedance equal to 2% over 50 years to have a moment magnitude of 6.50 and the mean distance to the seismic source of 11.0 miles.

4.2 SETTLEMENT

Due to the limited thickness and extent of surficial soil deposits, relatively few samples were obtained for testing and analysis of consolidation characteristics. From the limited data, the upper 1 to 2 feet of the residual soils exhibit relatively low dry density and visual porosity. Based on previous experience with similar materials, these soils would likely exhibit collapse upon wetting (hydrocollapse). We estimate that proposed foundations could induce settlement of about 1 to 3 inches if the soils became wetted after construction of the structures. The bedrock materials are expected to exhibit very low compressibility and a high degree of overconsolidation. We estimate settlement of foundation supported by these materials would be negligible.

4.3 SLOPE STABILITY

While no significant permanent slopes are anticipated from site development, construction is anticipated to result in temporary cuts up to about 10 feet in height. Temporary cuts are anticipated to expose up to about 4 feet of residual soil over Santiago Formation bedrock. Granitic bedrock is generally not anticipated in excavations. The Santiago Formation is anticipated to have moderate-dipping joints that could daylight locally in vertical cuts. Joints are anticipated to exhibit an effective friction angle of about 33 to 35 degrees. Using simple friction block mechanics, cuts in the Santiago Formation that are sloped steeper than about 1.5 to 1 (H:V) could exhibit a factor of safety less than 1. Vertical cuts in the residual soils up to 5 feet in height are estimated to have a factor of safety greater than 1.25.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible provided the recommendations presented in this report are incorporated into the design and construction of the project. Furthermore, it is also our opinion that proposed development will not adversely impact the stability of adjoining properties. Key issues that could have significant fiscal impacts on the geotechnical aspects of the proposed site development are discussed in the following sections of this report.

5.2 SEISMIC HAZARDS

5.2.1 Ground Rupture

No known active faults are known to project through the site nor does the site lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The potential for ground rupture due to an earthquake beneath the site is considered very low.

5.2.2 Ground Shaking

The site is situated in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relative close proximity to several active faults. Therefore, during the life of the proposed improvements, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the Southern California region. Design and construction in accordance with the current California Building Code (CBC) requirements is anticipated to address the issues related to potential ground shaking.

5.2.3 Liquefaction

The shallow bedrock conditions are generally not conducive for liquefaction. Furthermore the site is not located within a mapped California Geologic Survey liquefaction hazard zone. Therefore, the potential for liquefaction at the site is considered to be low.

5.2.4 Seiches and Tsunami

The site is elevated more than 590 feet above sea level and is located a substantial distance from a significant body of water within an enclosed basin. As such, the potential for hazards related to seiches and tsunami are considered very low.

5.3 LANDSLIDING AND SLOPE STABILITY

No conditions were observed during our investigation that would suggest the site is prone to landsliding. As such, the potential for landsliding is considered low provided that the site is rough graded in accordance with our recommendations. Minor slopes (less than 10 feet in height) that are constructed at a maximum gradient of 2 to 1 (H:V) are anticipated to be grossly stable under static and seismic conditions provided that grading is performed in accordance with the recommendations provided herein and that the slopes are maintained in accordance with the recommendations provided in Section 6.7 of this report.

Temporary slopes up to about 10 feet in height are anticipated for construction of subterranean portions of the buildings and for underground utilities. Temporary vertical cuts more than 5 feet in height may exhibit low factors of safety and therefore could be unstable. Potential instability would be dictated by the conditions of jointing in the Santiago Formation. However, essentially no data was obtained for jointing at the site and the anticipated conditions are based on data obtained from a nearby site. Additional subsurface exploration could be performed after the existing structures are demolished and allow for gathering of such data for further evaluation. In the absence of this data, cuts over 5 feet in height that are sloped steeper than 1.5 to 1 (H:V) may be unstable for temporary conditions. The condition can be mitigated through layback cuts or shoring as discussed in Section 6.1.9.

5.4 STATIC SETTLEMENT

The near-surface soils are anticipated to exhibit the potential for excessive settlement in their current state. This condition can be readily mitigated by removing and recompacting these soils within the influence of structural elements. Proposed grading could result in foundations being supported by both fill and bedrock materials. Foundations would be expected to undergo greater settlement where underlain by fill compared to bedrock where negligible settlement is anticipated. These conditions could result in high differential settlement although the total settlement would be less than 1 inch. The potential for excessive differential settlement due to transitions across bedrock and fill can be readily mitigated by either removing bedrock material below footings and replacing the material with compacted fill or founding all footings on bedrock.

Provided rough grading is performed in accordance with the recommendations provided in Section 6.1, total and differential static settlements are not anticipated to exceed 1 inch and ½-inch over 30 feet, respectively. The estimated magnitudes of static settlements are considered within tolerable limits for the proposed structures.

5.5 GROUNDWATER

Groundwater was not encountered within the site during this investigation to the maximum depth of 15 feet. No publications were found to provide historical groundwater data for the area. Our seismic refraction surveys provide some information regarding groundwater at greater depths. If groundwater were present in the materials below the site, the velocities would have reached a maximum of 6,000 feet per second at the groundwater surface. Since velocities exceeded 6,000 feet per second, we conclude there is no groundwater present to at least a depth of 35 to 40 feet below the current ground surface.

The surrounding areas are substantially developed at this time and future development of the site is not anticipated to result in a significant influx of water into the underlying ground. As such, we do not anticipate shallow groundwater conditions developing below the site in the future.

5.6 EXCAVATION AND MATERIAL CHARACTERISTICS

The residual soil, the artificial fill and the bedrock materials of the Santiago Formation are anticipated to be relatively easy to moderately difficult to excavate with conventional heavy earthmoving equipment. Excavations within the granitic bedrock materials will likely require difficult ripping with a single-shank CAT D-10 and/or blasting/jackhammering to remove. The estimated depth to the granitic bedrock unit, based on our exploratory drilling and interpretation of the seismic refraction survey data, are indicated on the Geotechnical Map, Plate 1. For consideration in assessing potential hard bedrock conditions, we have also prepared P-wave velocity contour maps. The maps provide the interpreted depth to both 4,500 ft./sec. and 7,000 ft./sec. velocities based on the Refraction Tomographic Models (See Plates 2 and 3, respectively). According to the rippability performance charts prepared by Caterpillar as well as the more conservative chart prepared by Caltrans, P-wave velocities less than 4,500 ft./sec. should be rippable with conventional heavy earthmoving equipment, while P-wave velocities over 7,000 ft./sec. will most likely require blasting.

Most of the site materials are near optimum moisture content. As such, preparation of site materials prior to placement as compacted fill is anticipated to require minor amounts of water.

Materials generated from cuts in the residual soils and Santiago Formation bedrock are generally considered suitable for reuse as fill provided they are cleared of deleterious debris. A considerable amount of oversize rock (over 12 inches in maximum dimension) will likely be generated from granitic bedrock cuts. The oversize rock will require special handling in a manner as described in Section 6.1.5 of this report.

5.7 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. Based on laboratory data and our experience with similar materials, we estimate the residual soils will shrink approximately 7 to 13 percent. Santiago bedrock materials are anticipated to bulk in the order of 0 to 10 percent. Granitic bedrock materials are anticipated to bulk in the order of 10 to possibly as much as 20 percent. Processing of the exposed bottoms is anticipated to result in negligible subsidence.

The estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occur during the grading process.

5.8 SOIL EXPANSION

Based on our laboratory test results and the USCS visual manual classification, the near-surface soils within the site are generally anticipated to possess a **Very Low** to **Low** expansion potential. Additional testing for soil expansion will be required subsequent to rough grading and prior to construction of foundations and other concrete work to confirm these conditions.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of San Marcos, California in addition to the recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of grading, we recommend a meeting be held between the developer, City Inspector, grading contractor, civil engineer and geotechnical consultant to discuss the proposed grading and construction logistics. We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site grading and foundation construction. This is to observe compliance with the design specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated. If conditions are encountered that appear to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

6.1.3 Site Clearing

All existing improvements, vegetation and other deleterious materials should be removed from the areas to be developed. Voids created by clearing should be left open for observation by the geotechnical consultant. Should any unusual soil conditions or subsurface structures (i.e. onsite sewage disposal systems such as seepage pits) be encountered during site clearing or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

6.1.4 Ground Preparation

Fill and Shallow Cut Areas: All existing fill, residual soil, and the upper 1 foot of the bedrock are considered unsuitable for support of proposed structural improvements. These materials should be removed through excavation to expose competent bedrock materials. Removal of unsuitable materials should extend laterally beyond the limits of proposed structural areas a distance equal to the depth of removal (1:1 projection). Based on our subsurface exploration, removals are anticipated to typically vary from about 1 to 5 feet below existing grade outside of the existing structures. The conditions below and around the existing structures were not investigated and we anticipate these areas contain various thickness of fill. The suitability of these materials or the soils underlying these fills is unknown. As such, removals greater than 5 feet in depth may be required to remove unsuitable fills or underlying residual soils.

All removals should be evaluated by the geotechnical consultant during grading to confirm the exposed conditions are as anticipated. Following removals, the exposed grade should first be scarified to a depth of 6 inches; moisture conditioned to at least 110 percent of the optimum moisture content, and then compacted to at least 90% of the laboratory standard.

Where removals are limited by existing structures or property lines, special grading techniques, such as slot cuttings, or other acceptable design criteria may be required. Under such conditions, specific recommendations should be provided by this firm during review of final grading plan.

Deep Cut Areas: Excavations for the subterranean parking at 339 Marcos Street and the excavation of the parking structure at 340 Marcos Street are anticipated to expose competent bedrock materials at proposed grades. Other deep cut areas will also likely expose competent bedrock materials at proposed grade. Once these areas are cut to grade, the need for further overexcavation should be evaluated by the project geotechnical consultant.

6.1.5 Fill Placement

In general, materials excavated from the site may be used as fill provided they are free of deleterious materials and particles greater than 12 inches in maximum dimension. *However, within the* upper *5 feet of building pads, the fill material should not contain particles greater than 6 inches in maximum dimension***.** Rock size criteria for fill materials to be placed within street undercut sections or within influence of future underground utilities should also be evaluated to avoid costly screening and/or importing of future trench backfill materials.

Fill materials should be placed in lifts no greater than approximately 8 inches in thickness. The fills should contain sufficient finer granular materials to eliminate nesting of rock fragments as recommended by the geotechnical consultant during grading. Each lift should be watered or air dried as necessary to achieve a uniform moisture content of 100 to 125 percent of optimum, and then compacted in place to at least 90 percent of the laboratory standard. The laboratory standard for maximum dry density and optimum moisture content for each soil type used should be determined in accordance with ASTM D 1557-07. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultant has approved the preceding lift. Lifts should be maintained relatively level and should not exceed a gradient of 20 to 1 (H:V). When placing fill on ground sloping steeper than 5:1 (H:V), vertical benches should be excavated into the adjacent slope.

6.1.6 Fill Slopes

Fill slopes, where necessary, should be constructed with a keyway having a minimum width of 10 feet and a minimum embedment of 2 feet into competent bedrock. The necessity of a fill key should be evaluated by the project geotechnical consultant during future plan reviews and/or during grading. A minimum fill width of approximately 5 feet should be maintained throughout fill slope construction to prevent sliver fills and cut/fill transitions within finished slopes.

Where practical, fill slopes should be constructed by over filling and trimming to a compacted core. The face of slopes that are not over-built should be backrolled with a sheepsfoot roller at least every 4 vertical feet of slope construction. The process should provide compacted fill to within 12 inches of the slope face. Finished slopes should be track-walked with a small dozer in order to compact the slope face. The slope face materials will tend to dry out prior to final face compaction. As such, the addition of water to the slope face will likely be required prior to compaction to achieve the required degree of compaction at the time of slope face compaction.

6.1.7 Cut Slopes

All cut slopes should be observed by an engineering geologist during rough grading to evaluate the competency of the slope.

6.1.8 Import Materials

If import materials are required to achieve the proposed finish grades, the proposed import soils should have an Expansion Index (EI) less than 50 (ASTM D4829) and a PI less than 15 (ASTM D 4318-08). Import sources should be indicated to the geotechnical consultant prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.1.9 Temporary Excavations

Temporary excavations with no adverse geologic conditions or surcharging of the excavations may be cut vertically up to a height of 5 feet within the onsite materials. Temporary excavations greater than 5 feet but no greater than 15 feet in height that are not surcharged should be laid back at a maximum gradient of 1.5:1 (H:V) or properly shored. Recommendations greater than 15 feet in height or any excavations that will be surcharged should be reviewed by the geotechnical consultant for specific recommendations. Additional investigation and evaluation of the bedrock conditions could be performed and may result in less restrictive recommendations.

The grading contractor should take appropriate measures when excavating adjacent existing improvements to avoid disturbing or compromising support of existing structures. The project geotechnical consultant should observe temporary cuts to confirm anticipated conditions and to provide alternate recommendations if conditions dictate. All excavations should conform to the requirements of CAL/OSHA.

6.2 SEISMIC DESIGN PARAMETERS

For design of the project in accordance with Chapter 16 of the 2013 CBC, the following table presents the seismic design factors:

TABLE 6.1
2013 CBC Seismic Design Parameters

Parameter	Value
Site Class	C
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.018
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.398
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.018
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	0.558
Design Spectral Response Acceleration, short periods, S_{DS}	0.679
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.372

MCE_R = Risk-Targeted Maximum Considered Earthquake

6.2.1 Soil Expansion

The recommendations presented herein are based on soils with a **LOW** expansion potential (EI<50 and PI<25). Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the existing expansion potential for the site. If site soils with significantly different expansion potentials are encountered, the recommendations contained herein may require modification

6.2.2 Settlement

Foundations should be designed for total and differential settlement of 1 inches and ½-inch over 30 feet, respectively.

6.2.3 Allowable Bearing Value - Soil

Provided site grading is performed as recommended herein, a bearing value of 3,000 psf may be used for continuous spread footings and isolated pad footings founded at a minimum depth of 12 inches below the lowest adjacent grade and having a minimum width of 12 inches and 24 inches square, respectively. The bearing value may be increased by 150 psf and 450 psf for each additional foot of increment in footing width and depth beyond the aforementioned minimum footing dimensions, respectively, up to a composite maximum value of 4,000 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.2.4 Allowable Bearing Value - Bedrock

Provided site grading is performed as recommended herein, a bearing value of 5,000 psf may be used for continuous spread footings and isolated pad footings founded at a minimum depth of 12 inches into bedrock and having a minimum width of 15 inches and 24 inches square, respectively. The bearing value may be increased by 500 psf and 1,000 psf for each additional foot of increment in footing width and depth beyond the aforementioned minimum footing dimensions, respectively, up to a composite maximum value of 7,500 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.2.5 Lateral Resistance

A passive earth pressure of 500 pounds per square foot per foot of depth expressed in terms of Equivalent Fluid Pressure (EFP) up to a maximum value of 1,500 pounds per square foot may be used to determine lateral bearing for footings. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.28 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. No increase in the coefficient of friction should be used when designing for wind and seismic forces.

The above values are based on footings placed directly against compacted fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90% relative compaction.

6.2.6 Footings and Slabs-on-grade

Exterior and interior continuous footings may be founded at the minimum depths indicated in Section 1809 of the 2013 CBC. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the lowest adjacent final grade. Exterior isolated pad footings intended for support of patio covers and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the lowest adjacent final grade.

Interior concrete slabs constructed on grade should be a minimum of 4 inches thick and should be reinforced with 4-inch by 4-inch, W2.9 X W2.9 reinforcing wire mesh or No. 3 bars spaced 12 inches on centers each way. Slabs to be used in garages and subject to vehicular loads should have a minimum thickness of 5 inches. Care should be taken to ensure the placement of reinforcement at mid-slab height. The structural engineer may recommend a greater slab thickness and reinforcement based on proposed use and loading conditions and such recommendations should govern if greater than the recommendations presented herein.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745-11, Class A. The membrane should be properly lapped, sealed, and underlain with at least 2 inches of sand having a SE no less than 30. This vapor retarder system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Where slabs will be subjected to high point loads, a subgrade modulus (Kv_1) of 250 pci, may be used for design of the slab. This modulus is based on a standard loaded area of 12 inches by 12 inches and should be adjusted for larger loading areas.

Special consideration should be given to slabs in areas to receive ceramic tile or other rigid, crack-sensitive floor coverings. Design and construction of such areas should mitigate hairline cracking as recommended by the structural engineer.

Block-outs should be provided around interior columns to permit relative movement and mitigate distress to the floor slabs due to differential settlement that will occur between column footings and adjacent floor subgrade soils as loads are applied.

Prior to placing concrete, subgrade soils below slab-on-grade areas should be thoroughly moistened to provide a moisture content that is at least 1 percentage point over the corresponding optimum moisture content to a minimum depth of 12 inches, subjected to the verification by a representative of the geotechnical consultant.

6.2.7 Foundation Observations

Foundation excavation should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.3 RETAINING AND SCREENING WALLS

6.3.1 General

The following preliminary design and construction recommendations are provided for general cantilever retaining and screen walls founded on conventional shallow footings. Final wall designs specific to the site development should be provided to project geotechnical consultant for review once completed. The structural engineer and architect should provide appropriate recommendations for sealing at all joints and applying moisture-proofing material on the back of the walls.

6.3.2 Allowable Bearing Value and Lateral Resistance

Provided site soils are prepared in accordance with Sections 6.1.4 of this report, a bearing value of 3,000 psf may be used for continuous spread footings and isolated pad footings founded at a minimum depth of 12 inches below the lowest adjacent grade and having a minimum width of 12 inches and 24 inches square, respectively. The bearing value may be increased by 150 psf and 450 psf for each additional foot of increment in footing width and depth beyond the aforementioned minimum footing dimensions, respectively, up to a composite maximum value of 4,000 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

A passive earth pressure of 500 pounds per square foot per foot of depth expressed in terms of Equivalent Fluid Pressure (EFP) up to a maximum value of 1,500 pounds per square foot may be used to determine lateral bearing for footings. The passive should be reduced where the top of a descending slope is located within a horizontal distance equal to 1.5 times the footing depth. The passive resistence value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.28 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. No increase in the coefficient of friction should be used when designing for wind and seismic forces.

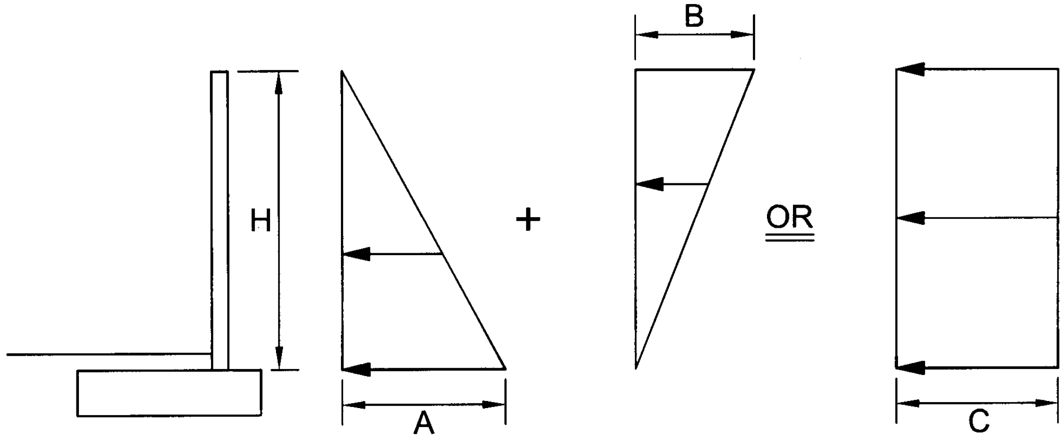
The above values are based on footings placed directly against compacted fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90% relative compaction.

6.3.3 Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Tables 6.2 through 6.4 below. Retaining walls supporting less than 6 feet of soils may be designed for static earth pressure only. Various backfill conditions may exist depending on the method of construction and local geotechnical conditions of the particular wall. Care should be taken when selecting the design values presented. Seismic earth pressures provided herein are based on the method provided by Seed &

Whitman (1970) using a PGA of 0.30g which is obtained by taking 40% of the S_{DS} value discussed in Section 6.2. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

STATIC & SEISMIC EARTH PRESSURES
Pressure Diagram



Static Component | Seismic Component | Total Force

TABLE 6.2

Cantilever Walls Backfilled with Compacted Fill
Pressure Values
Walls Up To 10 Feet High

Value	Level Backfill	2:1 Backfill
A	45H	68.0H
B	9.0H	9.0H
C	27.0H	38.5H

Note:
H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above.

TABLE 6.3

At-Rest Pressure Values
Walls Backfilled with Compacted Fill

Value	Level Backfill	2:1 Backfill
A	75H	113.0H
B	17H	17.0H
C	45H	65.0H
Note: H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above.		

TABLE 6.4

At-Rest Pressure Values
Walls Retaining In-place Earth Materials

Value	Level Backfill	2:1 Backfill
A	57H	86.0H
B	17H	17.0H
C	37H	51.5H
Note: H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above.		

6.3.4 Drainage and Moisture-Proofing

Retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch-diameter, ABS SDR-35 or PVC Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing and drainage outlet. Drainage gravel and piping should not be placed below outlets and weepholes. Filter fabric should consist of Mirafi 140N, or equal. Outlet pipes should be directed to positive drainage devices.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over onto the top of footing. A drainage blanket such as Mirafi Miradrain should be provided between the soil and the moisture-proofing materials. The drainage blanket should extend from the top of the gravel to within about 12 inches of finish grade. The top of footing should be finished smooth with a trowel to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

6.3.5 Footing Reinforcement

All continuous footings should be reinforced with a minimum of four No. 4 bars, two top and two bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

6.3.6 Footing Observations

Footing excavations should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level, and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.3.7 Wall Backfill

Onsite soils may be used for retaining wall backfill. The project geotechnical consultant should approve all backfill used for retaining walls. Soils should be moisture-conditioned to between 100 percent and 125 percent of the optimum moisture content, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. Hand-operated compaction equipment should be used to compact the backfill placed immediately adjacent the wall to avoid damage to the wall. Flooding or jetting of backfill material is not recommended.

6.4 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 5 feet in each direction. Flatwork more than 5 feet in width across the minimum dimension should be reinforced with 6” by 6”, W4 by W4 welded wire mesh or No 3 bars spaced 18 inches center to center in both directions. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 120 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete. The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to verify the recommended pre-moistening recommendations have been met.

Drainage from flatwork areas should be directed to local area drains or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. Flatwork adjacent the structure should slope at a minimum of 1% away from the building.

6.5 CONCRETE MIX DESIGN

Laboratory testing of on-site soils indicates soluble sulfate content less than 0.1%. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.6 POST GRADING CONSIDERATIONS

6.6.1 Site Drainage and Irrigation

Positive drainage devices, such as sloping concrete flatwork, graded swales or area drains, should be provided around the new construction to collect and direct all surface water to suitable discharge areas. In general, the site should be graded to conform to the requirements of 2013 CBC, Section 1804.3. However, the minimum slope away from the building may be reduced from 5% to 2% for soil and climatic reasons. No rain or excess water should be directed toward or allowed to pond against structures such as walls, foundations, flatwork, etc.

Excessive irrigation water can be detrimental to the performance of the proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within buildings that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of overwatering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match the water requirements of vegetation. Specific recommendations should be provided by a landscape architect or other knowledgeable professional.

6.6.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.7 of this report. Trench excavations must also conform to the requirements of Cal/OSHA.

Trench backfill materials and compaction criteria should conform to the requirements of the local municipalities. As a minimum, utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Trench backfill should be brought to moisture content slightly over optimum, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. The project geotechnical consultant should perform density testing, along with probing, to test compaction. Site conditions are generally not suitable for jetting of trench backfill and jetting should not be completed without prior approval from the project geotechnical consultant.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, imported clean sand having a SE of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly watered, and then compacted with a vibratory compactor. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

Where utility trenches intercept the building perimeter, consideration should be given to reducing the potential for water infiltration from the building exterior to the interior under slab area. This can be accomplished by the use of concrete, slurry, or other relatively impermeable material (subject to approval by the project geotechnical engineer) as trench backfill in this area. The “impermeable” material should be placed for the full depth of the trench and should be located at the foundation perimeter and extend at least 1 foot outside the building perimeter.

6.7 SLOPE MAINTENANCE

The long-term performance and stability of slopes can be greatly affected by maintenance. Initially, slopes should be provided with erosion resistance in the form of an herbaceous plant material, jute matting, polymer coating, or other suitable method as recommended by the landscape architect. Slopes should also be planted with deep-rooting, drought-tolerant, woody vegetation material as recommended by the landscape architect. The initial protection should be maintained until the woody material has become fully mature. Areas of slopes where vegetation becomes particularly distressed or dies should be replaced promptly. Watering of slopes should make judicious use of water by providing only that amount required to support the vegetation and adjusting the watering seasonally. Over watering must be avoided. Excessive drying of the soils is also detrimental to long-term slope performance and stability. The moisture content of soils should be maintained at a relatively uniform level. Rodent activity should be monitored and kept to a minimum. Excessive rodent burrowing can be detrimental to long-term slope performance and stability and should be repaired promptly. Drainage devices, such as V-ditches and backdrain outlet pipes installed on the slope face, should be periodically inspected to confirm they are clear and functional. Any accumulated debris should be removed promptly.

6.8 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

6.8.1 Subgrade Preparation

Prior to placement of pavement elements, the upper 12 inches of subgrade soils should be moisture-conditioned to at least 110 percent of the optimum moisture content and compacted to at least 90 percent of the laboratory standard. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding compacted soil or aggregate base materials.

6.8.2 Preliminary Pavement Designs

Existing near-surface soils are anticipated to have a range in R-value from low to high. Based on a typical R-value of 30 estimated to the site and traffic index, preliminary pavement structural sections are provided in the following table. The sections provided below are for planning purposes only. Actual R-value testing will be required following grading to determine the actual paving sections

Table 6.5
Preliminary Pavement Structural Sections

Location	Assumed T.I.	AC (inches)	PCC (inches)	AB (inches)
Interior Driveway	6.0	3.0	--	8.0
		--	6.5	--

AC – Asphalt Concrete; PCC – Portland Cement Concrete; AB – Aggregate Base

We recommend rigid pavements be constructed for all trash truck loading pads (areas in front of the trash enclosures). Trash Truck loading pads should be reinforced with No. 3 bars spaced at 12 inches each way. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints.

6.8.3 Pavement Materials

Aggregate base should be placed in lifts no greater than 6 inches in thickness, moistened to slightly over optimum moisture content, then compacted to at least 95 percent of the laboratory standard. The laboratory standard should be ASTM D1557-07. Aggregate base materials should be Crushed Aggregate Base or Crushed Miscellaneous Base conforming to Section 200-2 of the 2012 Standard Specification for Public Works Construction (Greenbook).

Paving asphalt should be PG 64-10 conforming to the requirements of Section 203-1 of the Greenbook. Asphalt concrete materials should conform to Section 203-6 and construction should conform to Section 302 of the Greenbook.

Portland Cement Concrete used to construct rigid pavements should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,000 psi at 28 days.

6.9 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus-Keefe & Associates, Inc*. be engaged to review any future development plans, including grading plans, foundation plans and proposed structural loads, prior to construction. This is to verify that the assumptions of this report are valid and that the preliminary conclusions and recommendations contained in this report have been properly interpreted and are incorporated into the project plans and specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our preliminary conclusions and recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly from the assumed development described herein, the project geotechnical consultant should review our preliminary design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report or subsequent design reports, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

7.0 LIMITATIONS

This report is based on the proposed development and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory testing for this investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil and bedrock materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

This report has been prepared for the exclusive use of **National CORE** and their project consultants in the planning and design of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report is subject to review by the controlling governmental agency.

Respectfully submitted,

ALBUS-KEEFE & ASSOCIATES, INC

Michael O. Spira
Principal Engineering Geologist
CEG 1976

David E. Albus
Principal Engineer
G.E. 2455

REFERENCES

Publications

C.D.M.G Open-File Report 96-02, 1996, Geologic Maps of the Northwest Part of San Diego County, California; Plate 1 Geologic Maps of the Oceanside, San Luis Rey, and San Marcos 7.5' Quadrangles.

C.D.M.G. Open-File Report 92-03, Preliminary Fault Activity Map of California, 1992.

C.D.M.G Open-File Report 86-8, Landslide Hazards in the Encinitas Quadrangle, San Diego County, California, 1986.

C.D.M.G. Open-File Report 92-03, Preliminary Fault Activity Map of California,1992.

California Geologic Survey, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California, 2008.

City of San Marcos General Plan, updated 2013.

Hart, E.W., Revised 1997, Fault-Rupture Hazard Zones in California, C.D.M.G. Special Publication 42.

Plans

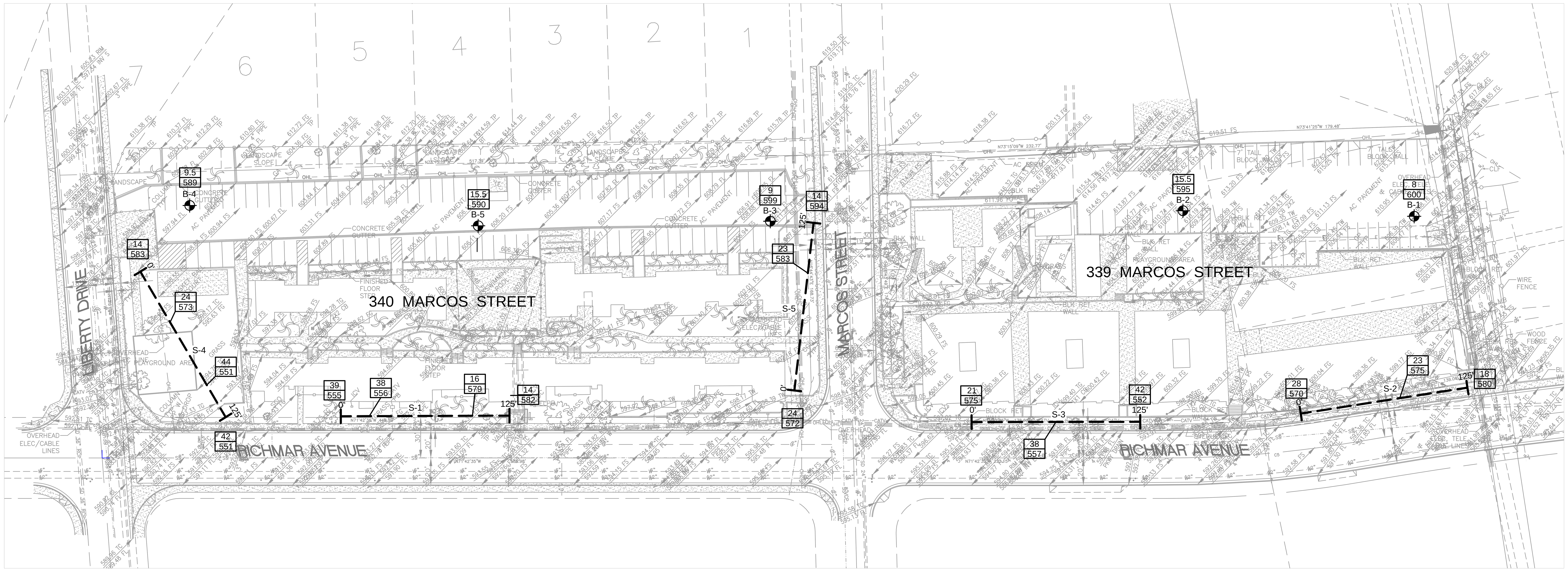
Topographic Survey, Villa Serena Apartments, 339-340 Marcos Street, San Marcos, California, prepared by DCI Engineering Inc., dated November 2013.

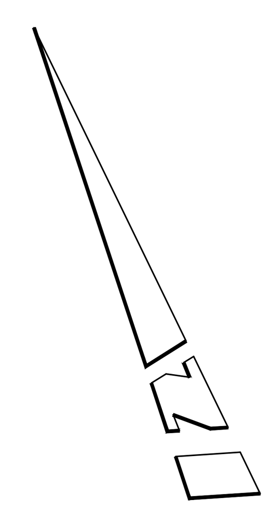
Conceptual Grading Plan, Phase 1, Villa Serena Apartments, 339-340 Marcos Street, San Marcos, California, prepared by DCI Engineering Inc., undated.

Conceptual Grading Plan, Phase 2, Villa Serena Apartments, 339-340 Marcos Street, San Marcos, California, prepared by DCI Engineering Inc., undated.

Architectural Plan, Phase 1, Villa Serena, San Marcos, California, prepared by ArchitectsDGa, dated February 25, 2015.

Architectural Plan, Phase 2, Villa Serena, San Marcos, California prepared by ArchitectsDGa, dated February 25, 2015.





40' 0' 40' 80' 120'

GRAPHIC SCALE: 1"=40'

EXPLANATION
(LOCATIONS APPROXIMATE)

- B-5 - Exploratory Boring
- 24 / 596 - Estimated Depth, in feet, to Granitic Bedrock, and corresponding Elevation
- 0' S-5 125' - Seismic Refraction Lines with Stations

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

Job No.: 2265.00	Date: 07/07/15	Plate: 1

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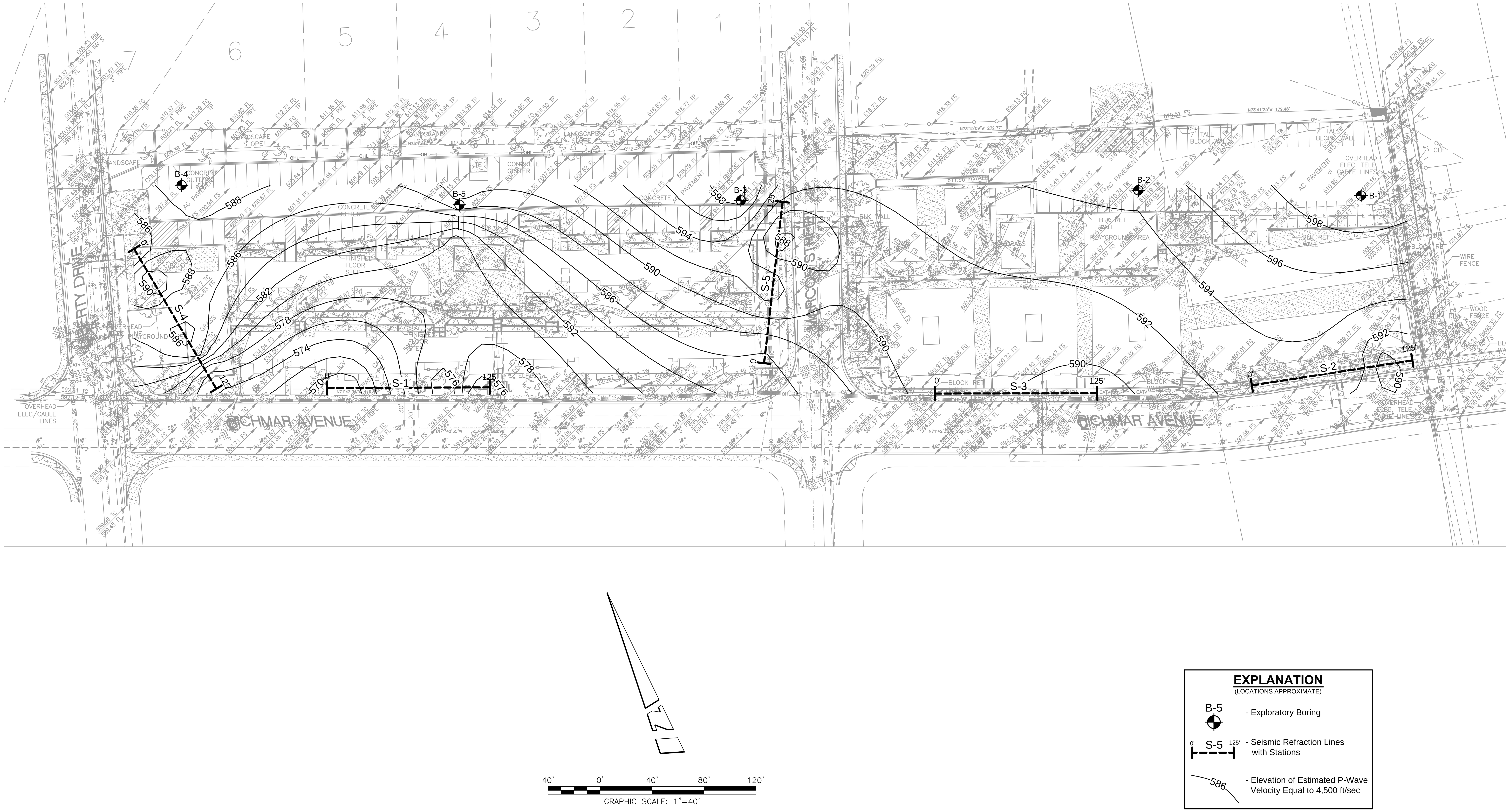
TOPOGRAPHIC SURVEY
VILLA SERENA APARTMENTS
339-340 MARCOS STREET
CITY OF SAN MARCOS,
IN THE COUNTY OF FRESNO, STATE OF CALIFORNIA.

DATE: NOVEMBER 2013
DRAWN BY: JMM
CHECKED BY: DRC
JOB NO: 1303.02
SHEET 3 OF 3



ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

P-WAVE CONTOUR MAP
(4,500 ft/sec)

Job No.: 2265.00	Date: 07/07/15	Plate: 2

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TOPOGRAPHIC SURVEY
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CITY OF SAN MARCOS,
IN THE COUNTY OF FRESNO, STATE OF CALIFORNIA.

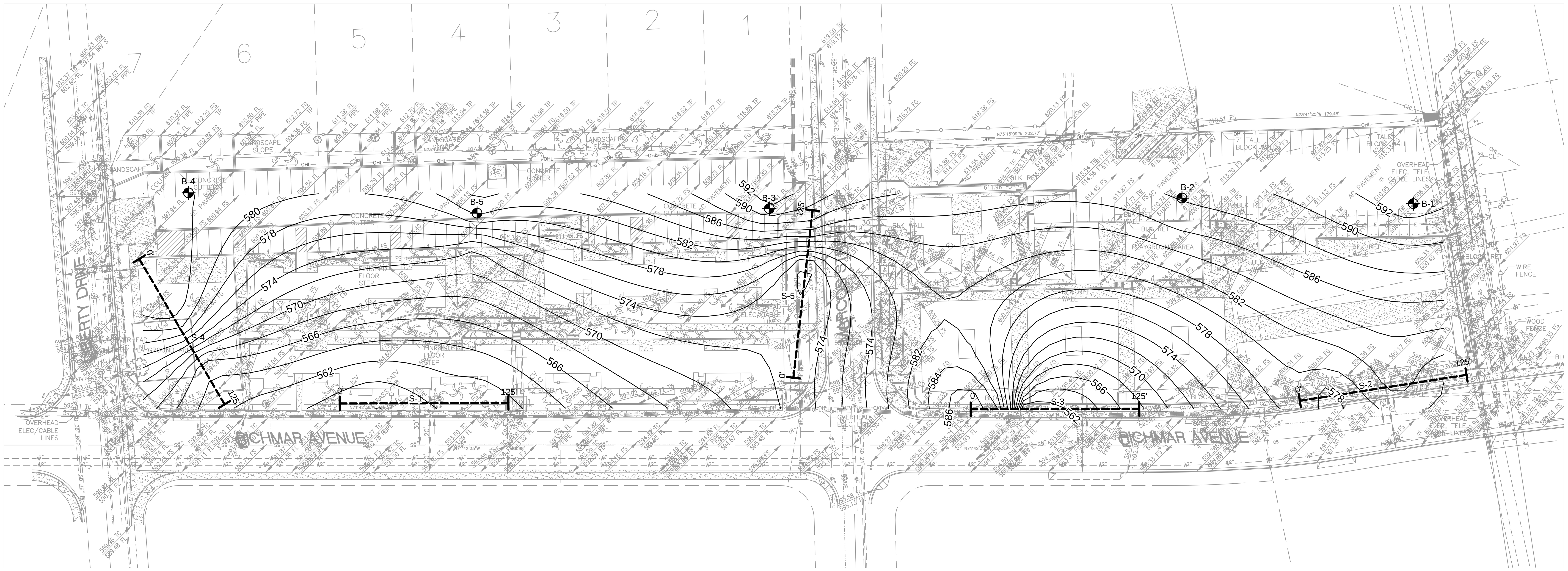
DATE: NOVEMBER 2013

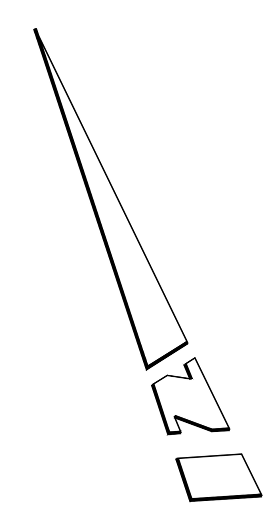
DRAWN BY: JMM

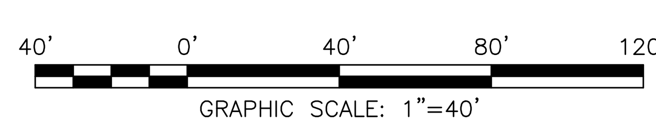
CHECKED BY: DRC

JOB NO: 1303.02

SHEET 3 OF 3







EXPLANATION
(LOCATIONS APPROXIMATE)

B-5 - Exploratory Boring

0' S-5 125' - Seismic Refraction Line with Stations

586 - Elevation of Estimated P-Wave Velocity Equal to 7,000 ft/sec

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DATE: NOVEMBER 2013

DRAWN BY: JMM

CHECKED BY: DRC

JOB NO: 1303.02

SHEET 3 OF 3

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

P-WAVE CONTOUR MAP
(7,000 ft/sec)

Job No.: 2265.00	Date: 07/07/15	Plate: 3

APPENDIX A

EXPLORATORY LOGS

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation		Location: Legend
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069		Elevation:
Job Number: 2265.00	Client: National Community Renaissance	Date:
Drill Method:	Driving Weight:	Logged By:

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		EXPLANATION							
		Solid lines separate geologic units and/or material types.							
5		Dashed lines indicate unknown depth of geologic unit change or material type change.							
		Solid black rectangle in Core column represents California Split Spoon sampler (2.5in ID, 3in OD).							
		Double triangle in core column represents SPT sampler.							
10		**Solid black rectangle** in Bulk column respresents large bag sample.							
		Other Laboratory Tests: Max = Maximum Dry Density/Optimum Moisture Content EI = Expansion Index SO4 = Soluble Sulfate Content DSR = Direct Shear, Remolded DS = Direct Shear, Undisturbed SA = Sieve Analysis (1" through #200 sieve) PSA = Particle Size Analysis (SA with Hydrometer) 200 = Percent Passing #200 Sieve Hydro = Hydrometer Only Consol = Consolidation SE = Sand Equivalent Rval = R-Value ATT = Atterberg Limits							
15									
20									

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation		Location: B-1
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069		Elevation: 611.0
Job Number: 2265.00	Client: National Community Renaissance	Date: 3/17/2014
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt = 5"							Hydro RVal
		RESIDUAL SOIL Clayey Sand (SC): Light reddish brown, moist, medium dense, fine to medium grained sand		9			11.1	115.7	
		@ 2', becomes dark reddish brown, more sand							
5		**BEDROCK - Santiago Formation (Tsa)** Sandstone : Light reddish brown, moist, moderately hard to hard, fine grained sand		50/ 5"			13.5	103.1	
				50/ 2"			13.5	Dist.	
		BEDROCK - Granitic Rock (Kgr) Refusal							

Total Depth 8 feet
Refusal at 8 feet
No Ground Water
Boring backfilled with soil cuttings and capped with Asphalt

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation	Location: B-2
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069	Elevation: 611.0
Job Number: 2265.00 — Client: National Community Renaissance	Date: 3/17/2014
Drill Method: Hollow-Stem Auger — Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt = 5.5"							
		RESIDUAL SOIL Sandy Clay (CL): Dark reddish brown, moist, medium stiff, fine to medium grained sand		30			9.8	119.1	
		BEDROCK - Santiago Formation (Tsa) Sandy Claystone / Sandstone : Light reddish brown to light gray, moist, moderately hard to hard, fine to medium grained sand, some pinhole pores and weathering		58			10.8	117.8	
5		@ 4', becomes fine to coarse grained sand, moderate pinhole pores, hard		92/ 7"			11.3	115.8	
		@ 6', becomes dark reddish brown to light gray, hard							
		Sandstone : Dark reddish brown, moist, hard, fine to coarse grained sand							
10		@ 10', some light gray to reddish brown Sandy Claystone, some gravel		50/ 6"			12.5	119.2	
		BEDROCK - Granitic Rock (Kgr) Light brown, damp, very hard, fine to coarse grained, refusal							

Total Depth 10.5 feet
Refusal at 10.5 feet
No Ground Water
Boring backfilled with soil cuttings and capped with asphalt

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation	Location: B-3
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069	Elevation: 603.5
Job Number: 2265.00 / Client: National Community Renaissance	Date: 3/17/2014
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt = 6"							Max EI SO4 DS Hydro ATT pH Resist
		RESIDUAL SOIL Sandy Clay (CL): Dark reddish brown, moist, very stiff, fine to medium grained sand		31			13.4	119	
		BEDROCK - Santiago Formation (Tsa) Clayey Sandstone : Dark reddish brown, moist, moderately hard, fine to coarse grained sand, some weathering		87/ 11"			13.3	115.9	
5		Sandy Claystone / Clayey Sandstone : Dark reddish brown to light gray, moist, moderately hard, fine to coarse grained sand		28			16.1	113.5	
10		**BEDROCK - Grantic Rock (Kgr)** Refusal							

Total Depth 9 feet
Refusal at 9 feet
No Ground Water
Boring backfilled with soil cuttings and capped with asphalt

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation		Location: B-4
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069		Elevation: 600.0
Job Number: 2265.00	Client: National Community Renaissance	Date: 3/17/2014
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		Asphalt = 3"							
		RESIDUAL SOIL Sandy Clay (CL): Dark reddish brown, moist, medium stiff, fine to medium grained sand							
		BEDROCK - Santiago Formation (Tsa) Clayey Sandstone : Light reddish brown to light gray, moist, moderately hard to hard, fine to coarse grained sand		33			10.9	121.9	
5				50/ 6"			10.5	117.6	
		@ 4', becomes hard, more sand, some gravel							
		Sandstone : Light brown to tan, damp to moist, hard, fine to coarse grained sand, some gravel, some decomposed granite		50/ 2"			4.5	Dist.	
10		**BEDROCK - Granitic Rock (Kgr)** Refusal							

Total Depth 9.5 feet
Refusal at 9.5 feet
No Ground Water
Boring backfilled with soil cuttings and capped with asphalt

EXPLORATION LOG

Project: Preliminary Geotechnical Investigation		Location: B-5
Address: Richmar Avenue and Marcos Street, San Marcos, CA 92069		Elevation: 600.4
Job Number: 2265.00	Client: National Community Renaissance	Date: 3/17/2014
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt = 3"							
		RESIDUAL SOIL Sandy Clay (CL): Dark reddish brown, moist, medium stiff, fine to medium grained sand, some pinhole pores		7			10.4	104.7	
5		**BEDROCK - Santiago Formation (Tsa)** Clayey Sandstone : Light reddish brown to light gray, moist, hard, fine to coarse grained sand, moderate weathering		95/ 8"			15.5	104.1	
		Sandstone with Some Clay : Dark reddish brown, moist, hard, fine to coarse grained sand		93/ 10"			10	113.1	
10		Clayey Sandstone : Light reddish brown to light gray, moist, hard, fine to coarse grained sand, some gravel		48			12	121.6	
15		**BEDROCK - Granitic Rock (Kgr)** Light brown, damp, very hard, fine to coarse grained, refusal		50/ 1"			2.4	Dist.	

Total Depth 15.5 feet
Refusal at 15.5 feet
No Ground Water
Boring backfilled with soil cuttings and capped with Asphalt

APPENDIX B

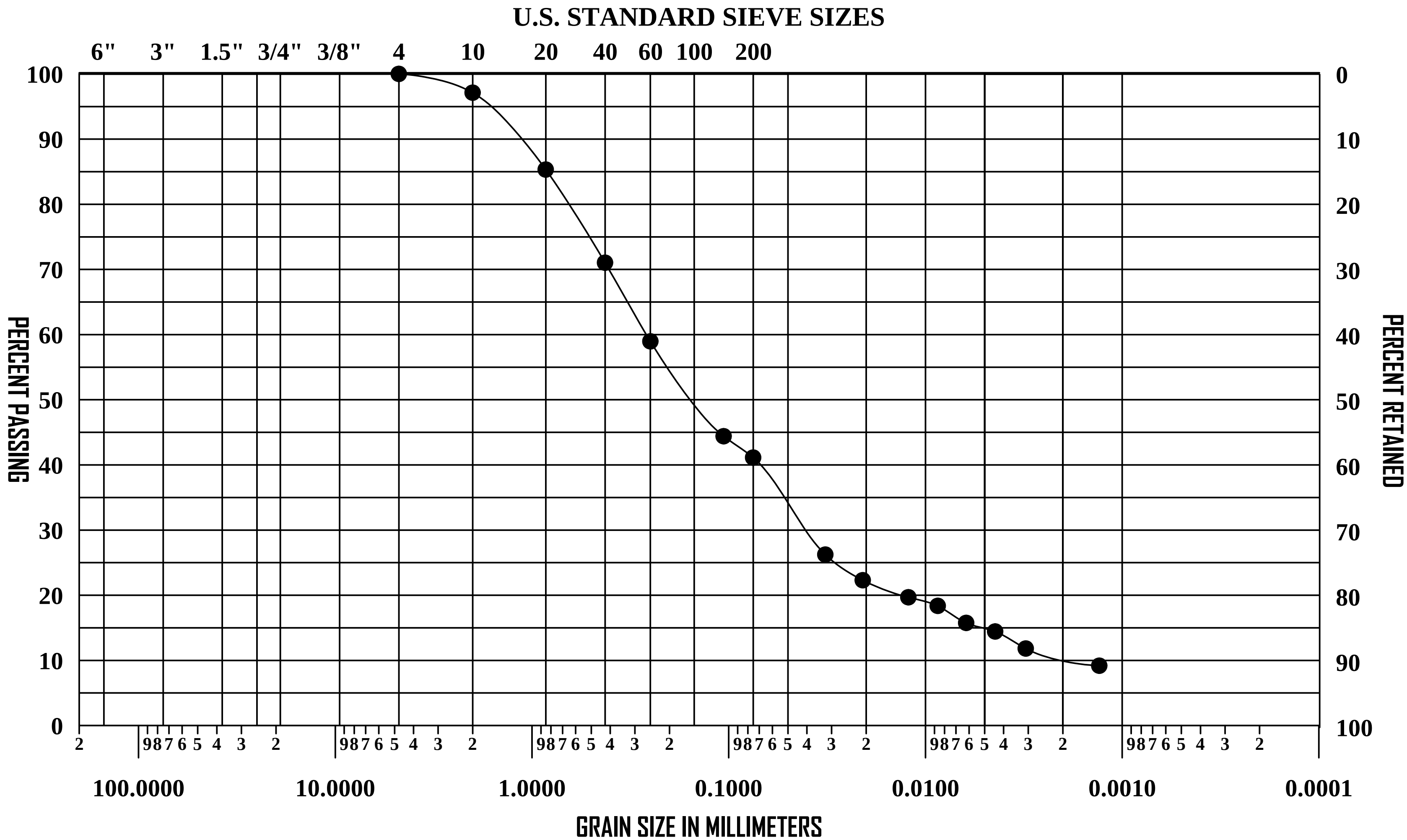
LABORATORY TEST PROGRAM

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GRAIN SIZE DISTRIBUTION

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	



LOCATION	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
B-1	0-5'	● — ●			Sandy Silt (ML)
		■ - - ■			
		▲ - - ▲			
		◆ - · - ◆			

Job No: 2265.00

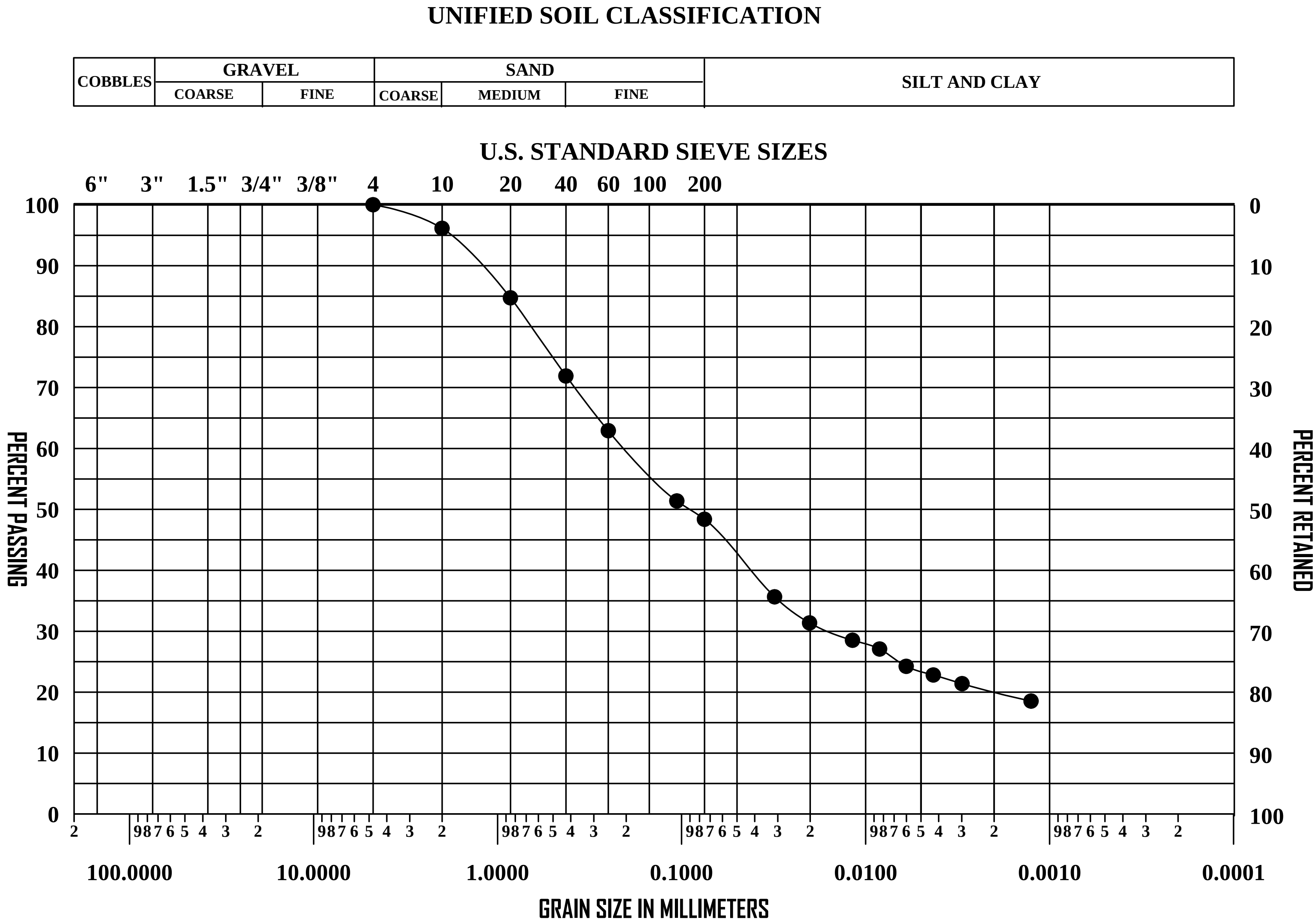
Plate No: B-1

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GRAIN SIZE DISTRIBUTION

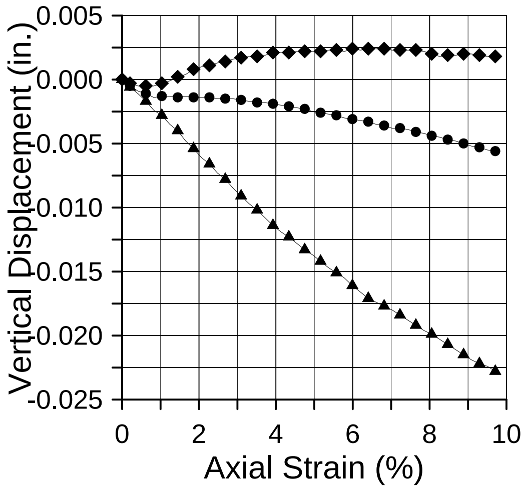
Job No: 2265.00

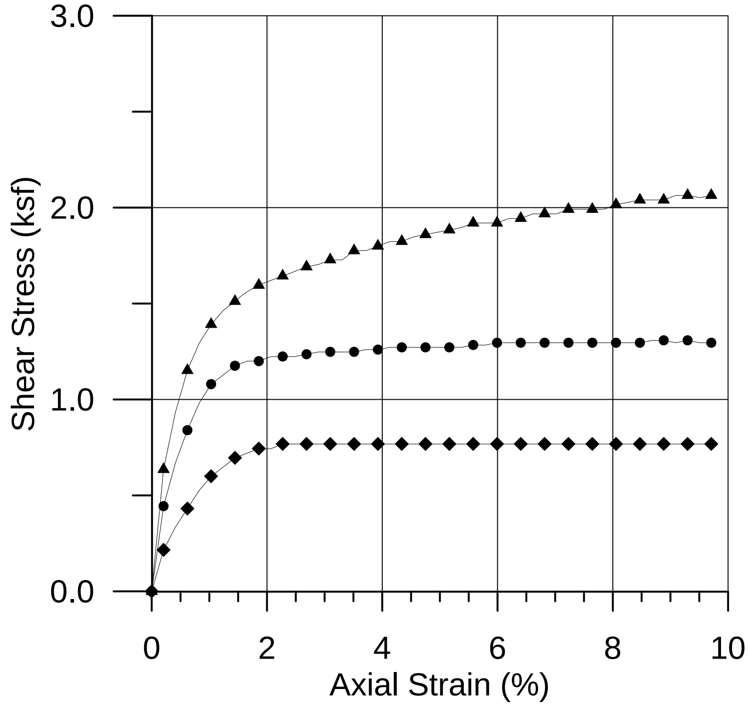
Plate No: B-2

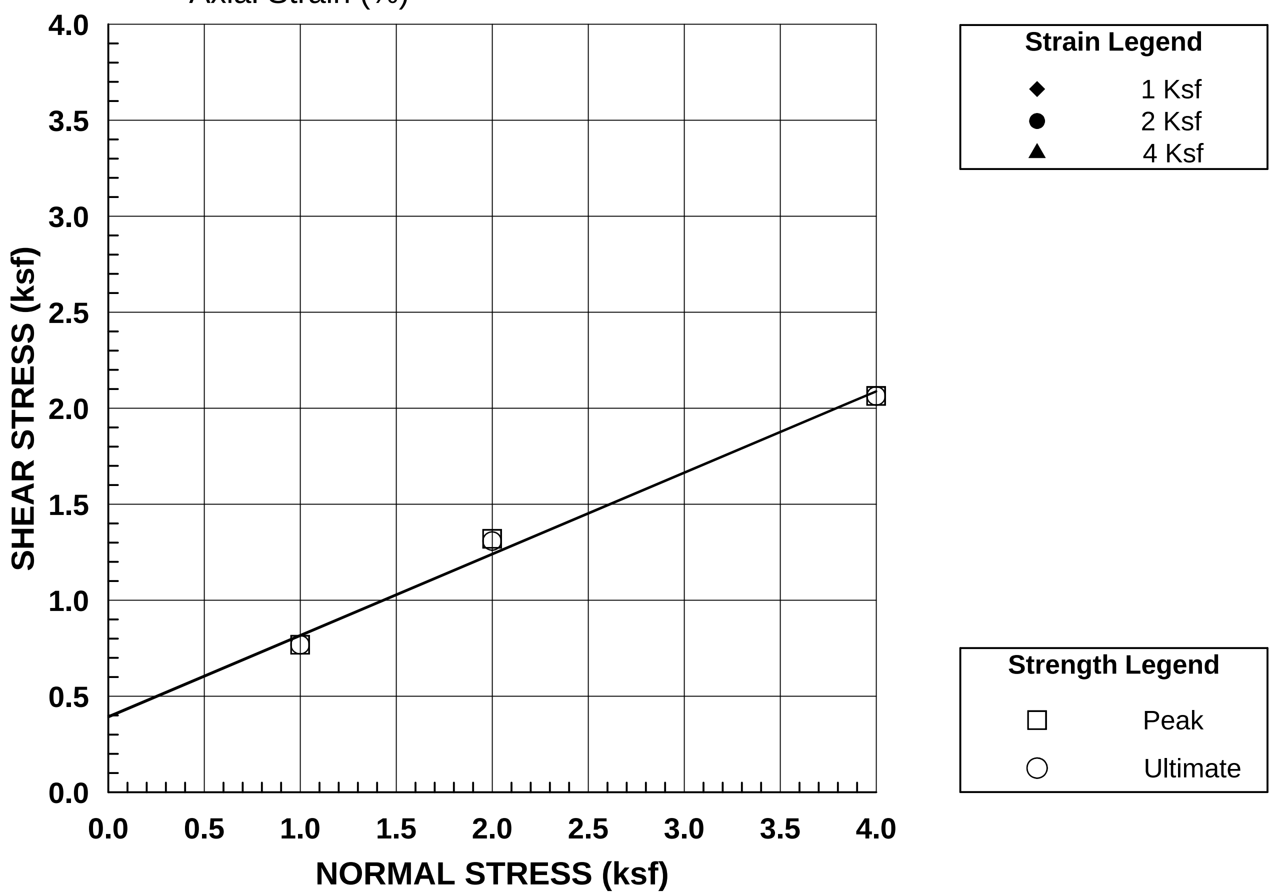


LOCATION	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
B-3	0-5'	●—●			Clayey Sand (SC)
		■- - - -■			
		▲— - - ▲			
		◆- · — ◆			

Specimen No.	1	2	3
Normal Stress (ksf)	1	2	4
Peak Stress (ksf)	0.768	1.32	2.064
Peak Displacement (in)	0.055	0.25	0.22
Ultimate Stress (ksf)	0.768	1.308	2.064
Ultimate Displacement (in)	0.245	0.245	0.245
Initial Dry Density	116.5	116.5	116.5
Initial Moisture Content (%)	9.443	9.443	9.443
Strain Rate (in/min)	0.010		







SAMPLE LOCATION	SAMPLE TYPE	SAMPLE DESCRIPTION
B-3 @ 0-5'	Remolded - Saturated	Sandy Clay (CL)

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

DIRECT SHEAR

Job No: 2265.00

Plate No: B-3

LABORATORY TESTING PROGRAM

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488-93). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented on the Boring Logs provided in Appendix A.

In-Situ Moisture Content and Dry Density

Moisture content and dry density of in-place soil materials were determined in representative strata. Test data are summarized on the Exploration Logs provided in Appendix A.

Maximum Dry Density and Optimum Moisture Content

Maximum dry density and optimum moisture content of onsite soils were determined for a selected sample in general accordance with Method A of ASTM D1557-07. Pertinent test values are given on Table B.

Grain Size/Hydrometer Analysis

Grain size and hydrometer analyses were performed on a selected sample to verify visual classifications performed in the field. The test was performed in accordance with ASTM D422-63. Test results are graphically presented on Plates B-1 to B-2.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one of our borings. The test was conducted in general conformance with ASTM D 3080-04. The sample was remolded to 90 percent of maximum dry density. Three specimens were prepared for the test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-3.

Expansion Potential

Expansion index testing was performed on a selected sample. The test was performed in conformance with ASTM D4829-08. The test result is presented on Table B.

Atterberg Limits

Atterberg limit tests (Liquid Limit, Plastic Limit and Plasticity Index) were performed on a selected sample to verify visual classifications. The tests were performed in general conformance with ASTM D 4318-08. Test results are presented on Table B.

Soluble Sulfate Content

A chemical analysis was performed on a selected soil sample to determine soluble sulfate content. The test was performed in accordance with California Test Method (CTM) 417. The test result is included in Table B.

Corrosion Analysis

Corrosion analyses, which include soluble sulfate content, minimum resistivity, and pH, were performed on a selected sample. The tests were performed in accordance with California Test Method (CTM) 417, CTM 643 and CTM 643, respectively. The test results are included in Table B.

R-value

R-value testing was performed for an existing surficial soil sample. This test was performed in general accordance with California Test Method No. 301. The test result is included in Table B.

TABLE B
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-1	0 - 5	Clayey Sand (SC)	R-Value:	56
B-3	0-5	Sandy Clay (CL)	Maximum Dry Density: Optimum Moisture Content: Expansion Index: Expansion Potential: Liquid Limit: Plasticity Index: Soluble Sulfate Content: Sulfate Exposure:	129.5 pcf 9.5% 42 Low 35.9 22.1 0.001% Negligible

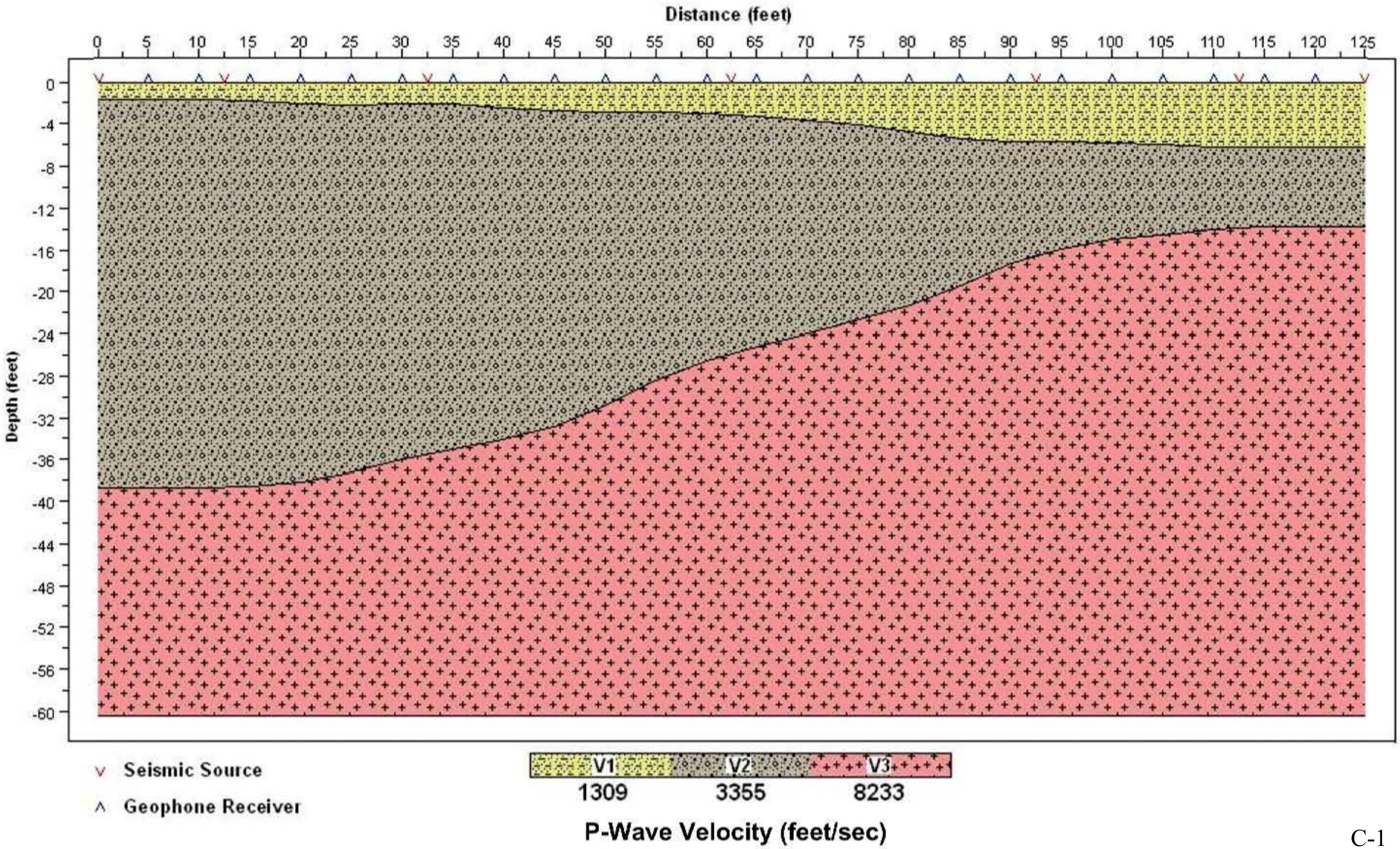
Note: Additional laboratory test results are provided on the boring logs provided in Appendix A

APPENDIX C

SEISMIC REFRACTION LINES

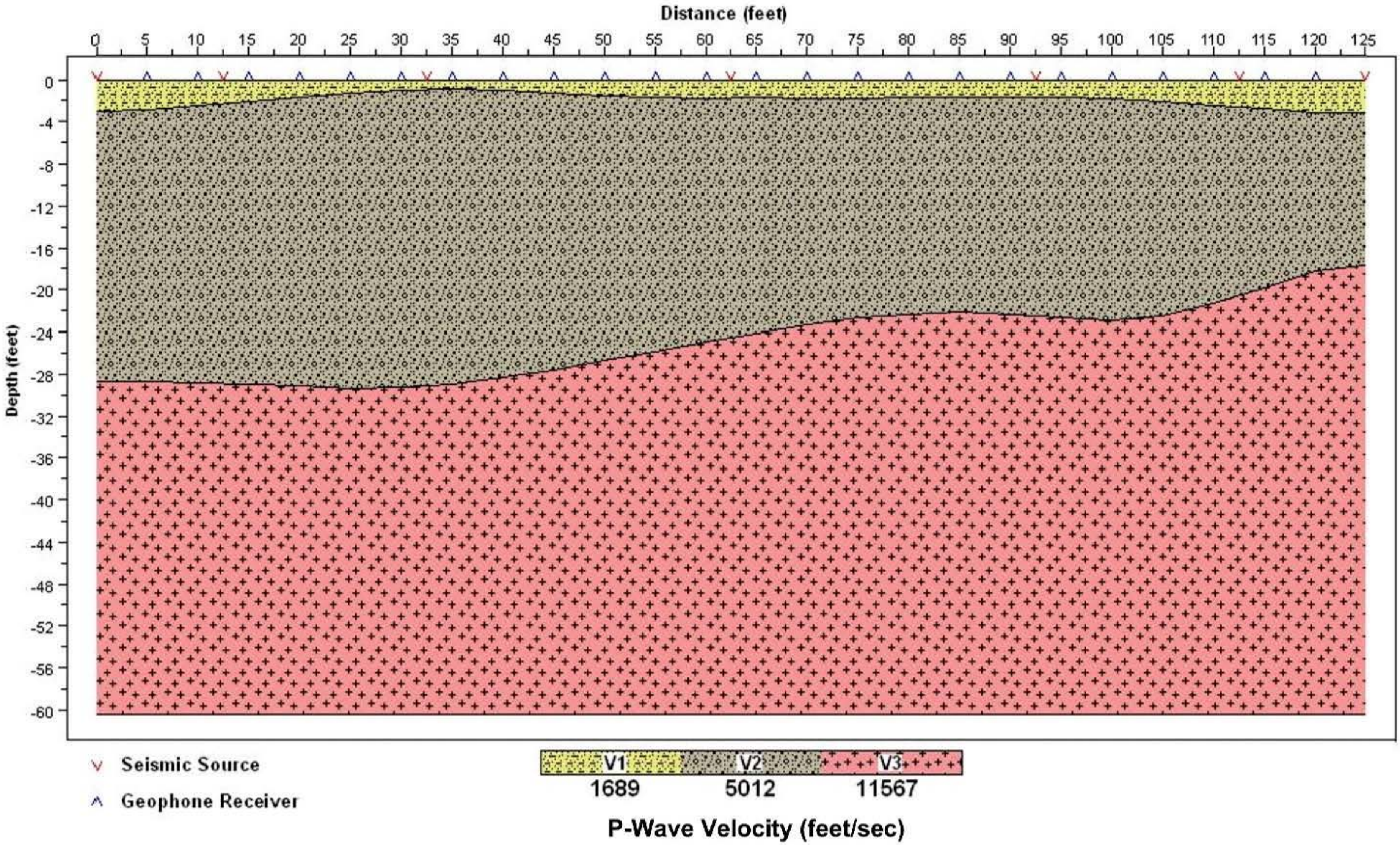
SEISMIC LINE S-1

South 73° East >



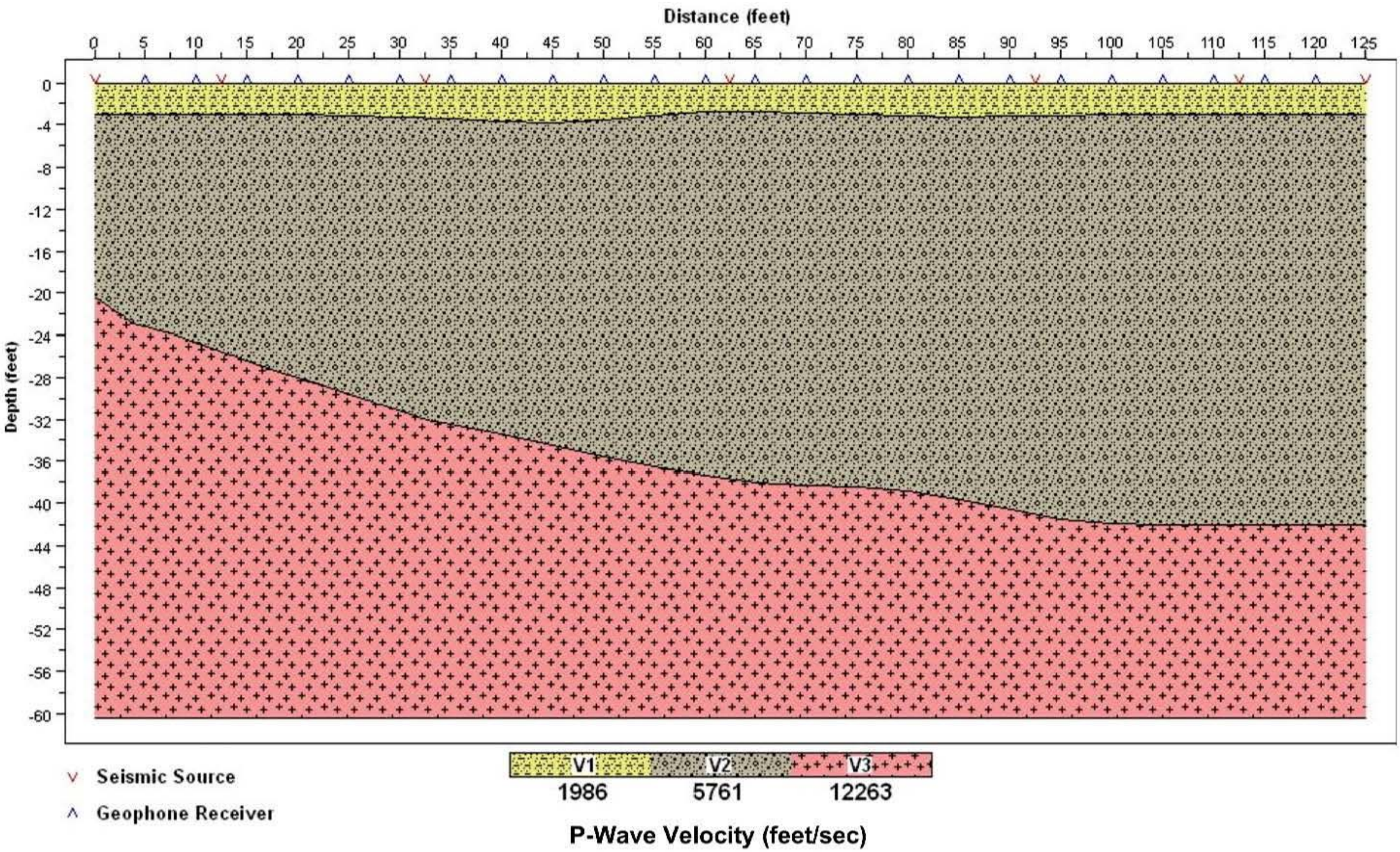
SEISMIC LINE S-2

South 80° East >



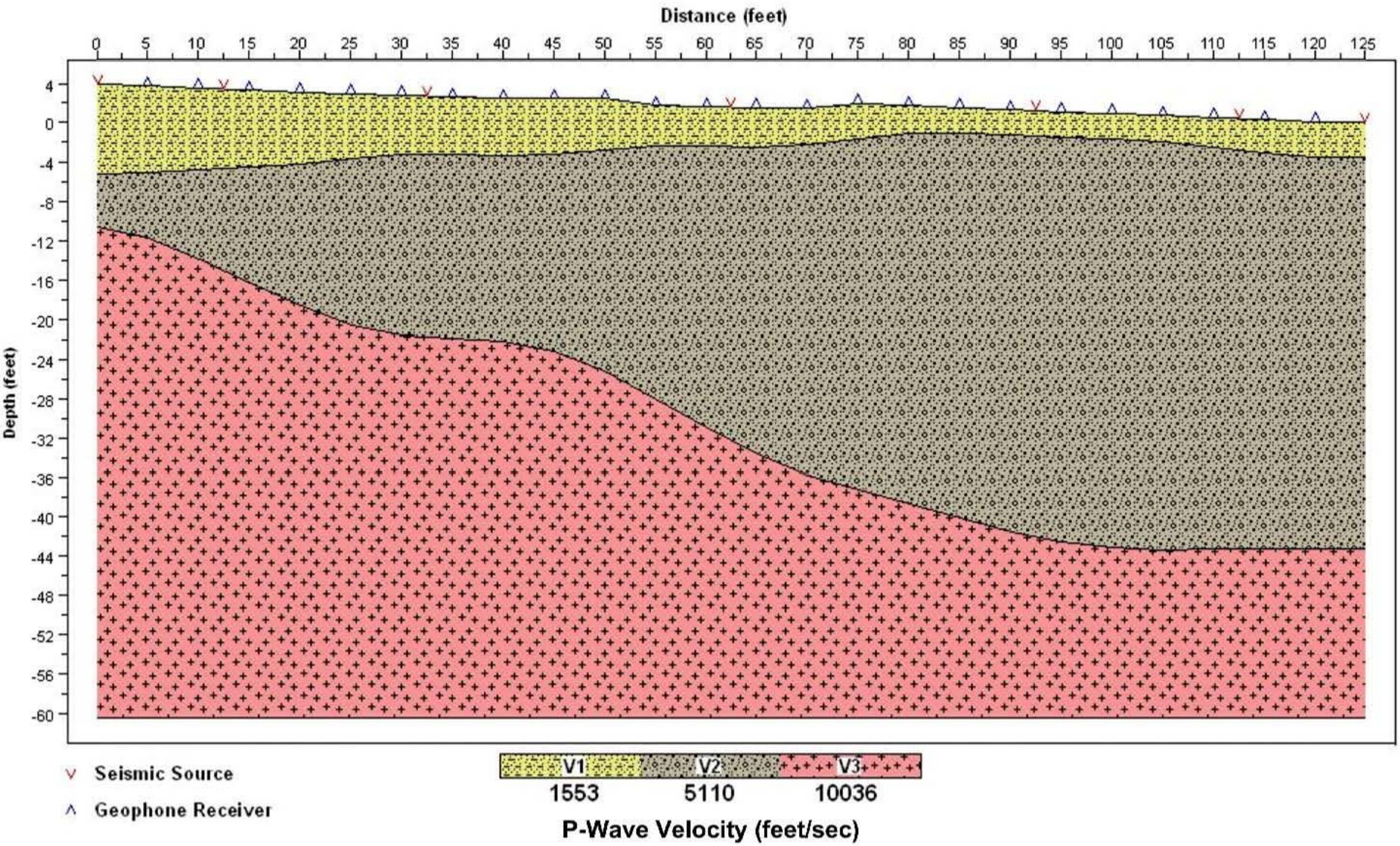
SEISMIC LINE S-3

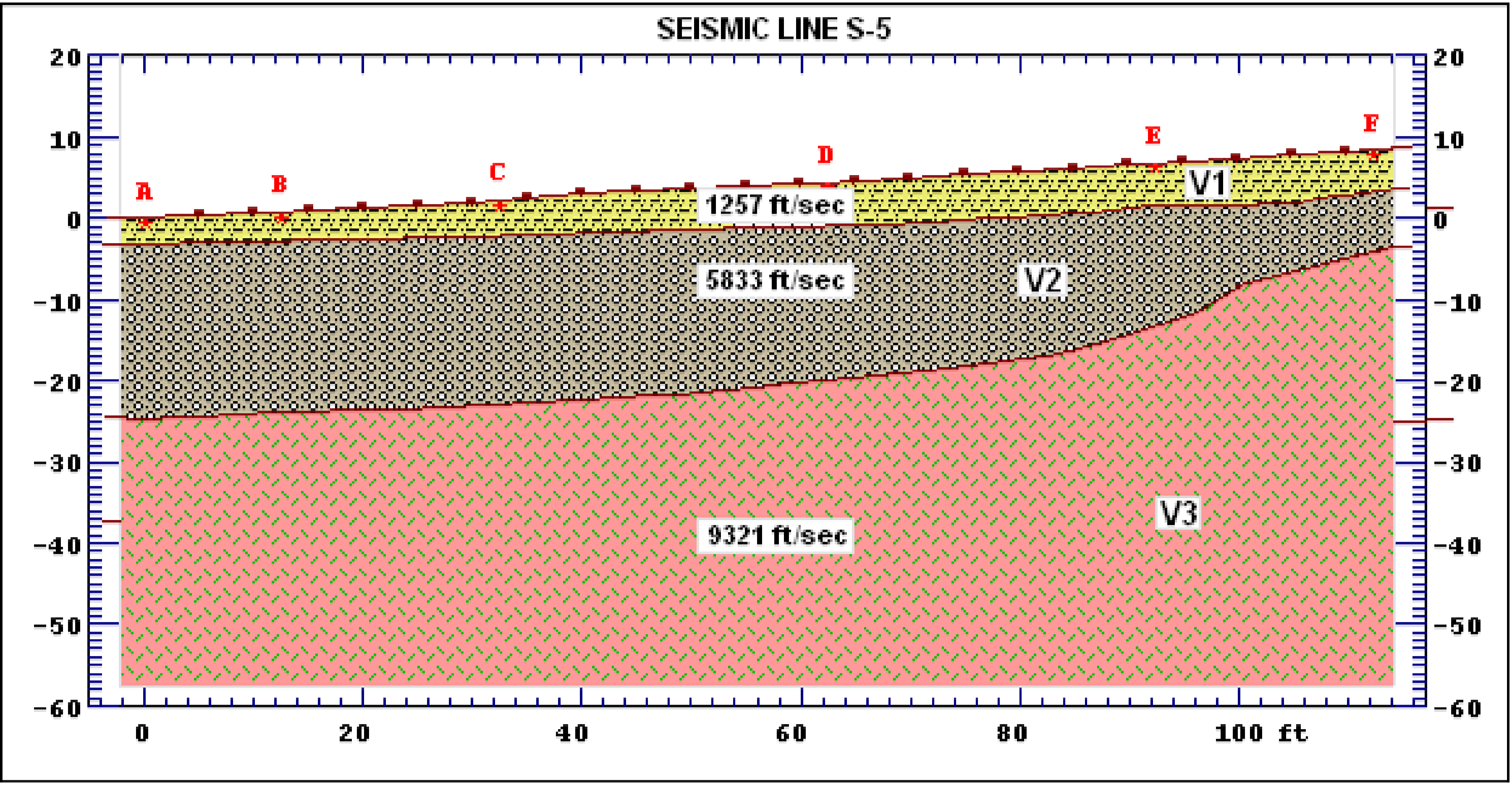
South 73° East >



SEISMIC LINE S-4

South 15° East >



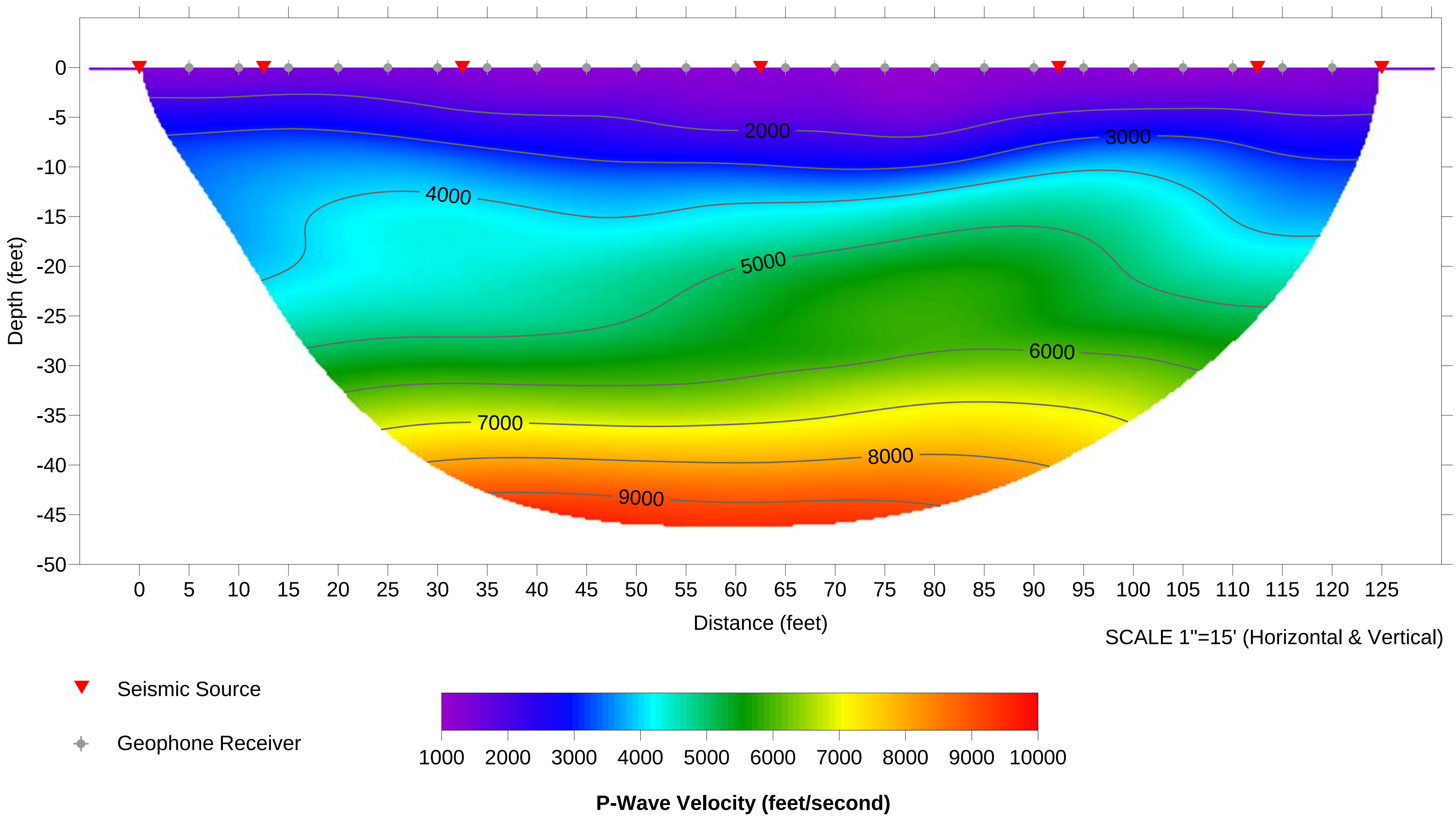


SEISMIC LINE S-5
A
B
C
D
E
F
V1
1257 ft/sec
5833 ft/sec
V2
V3
9321 ft/sec
20
10
0
-10
-20
-30
-40
-50
-60
0
20
40
60
80
100 ft

SEISMIC LINE S-1

South 73° East →

REFRACTION TOMOGRAPHIC MODEL

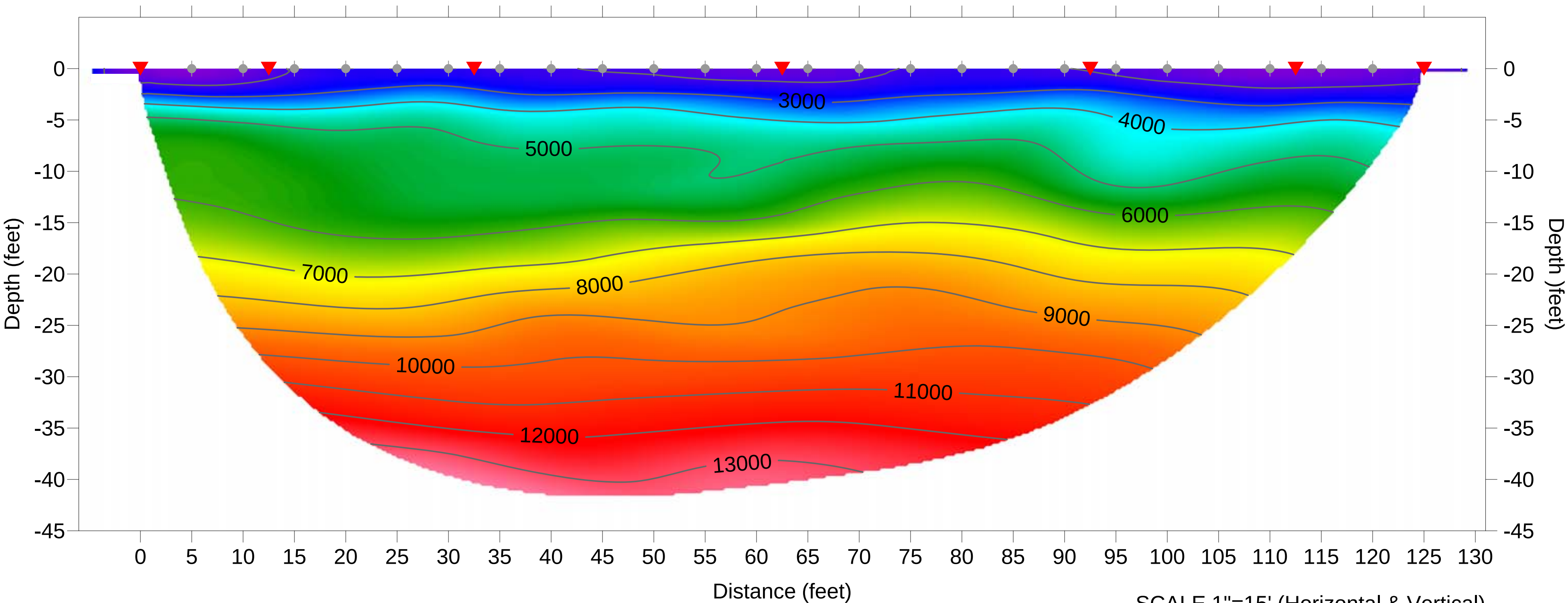


RMS error 1.9 %, Rayfract Version 3.31

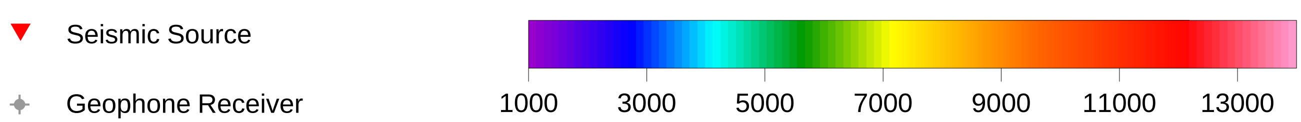
SEISMIC LINE S-2

South 80° East →

REFRACTION TOMOGRAPHIC MODEL



SCALE 1"=15' (Horizontal & Vertical)



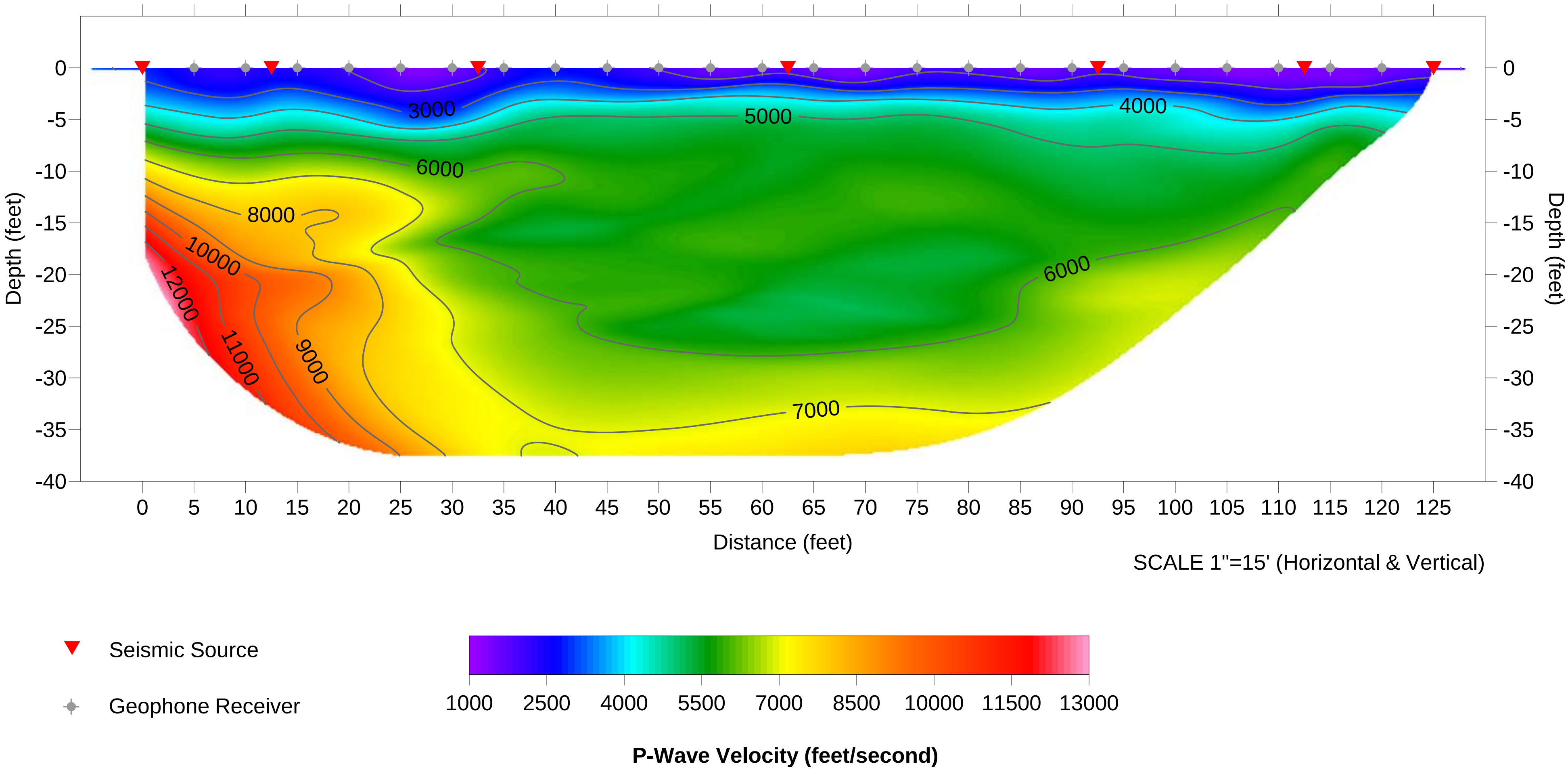
P-Wave Velocity (feet/second)

RMS error 1.8 %, Rayfract Version 3.31

SEISMIC LINE S-3

South 73° East →

REFRACTION TOMOGRAPHIC MODEL

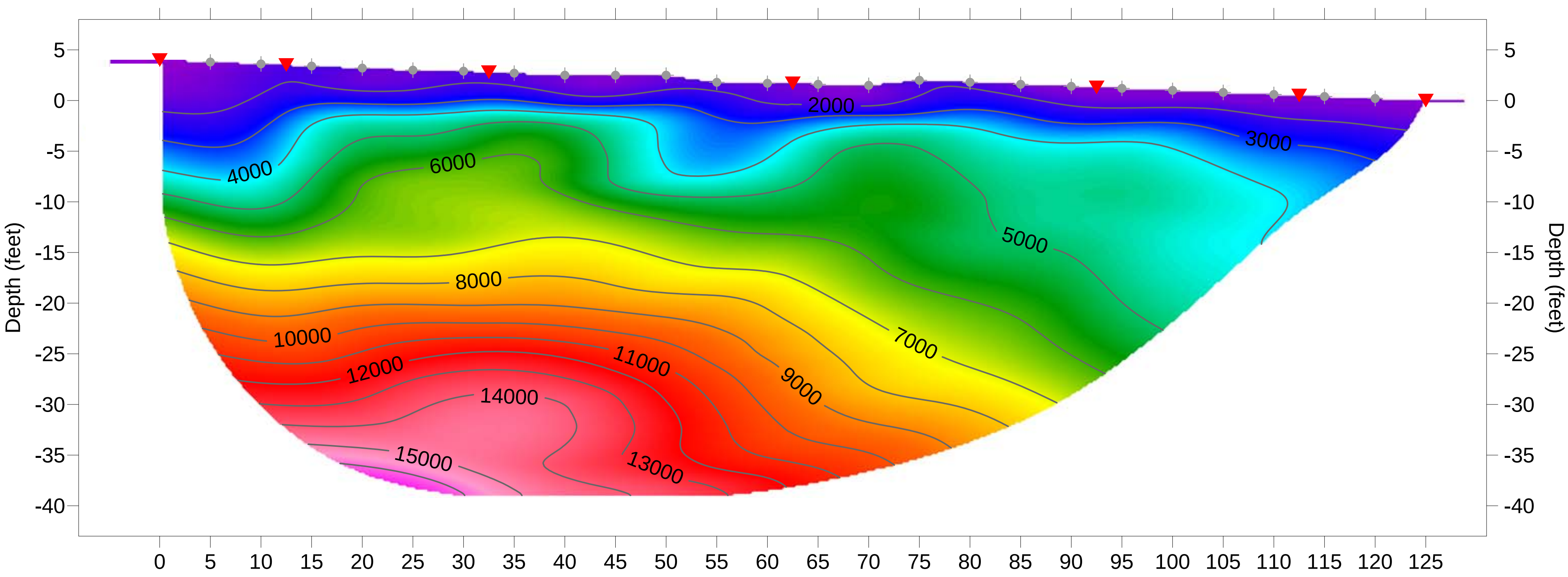


RMS error 2.3 %, Rayfract Version 3.31

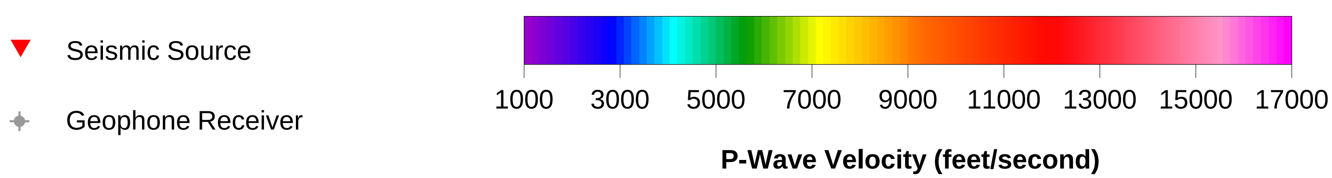
SEISMIC LINE S-4

South 15° East →

REFRACTION TOMOGRAPHIC MODEL



SCALE 1"=15' (Horizontal & Vertical)

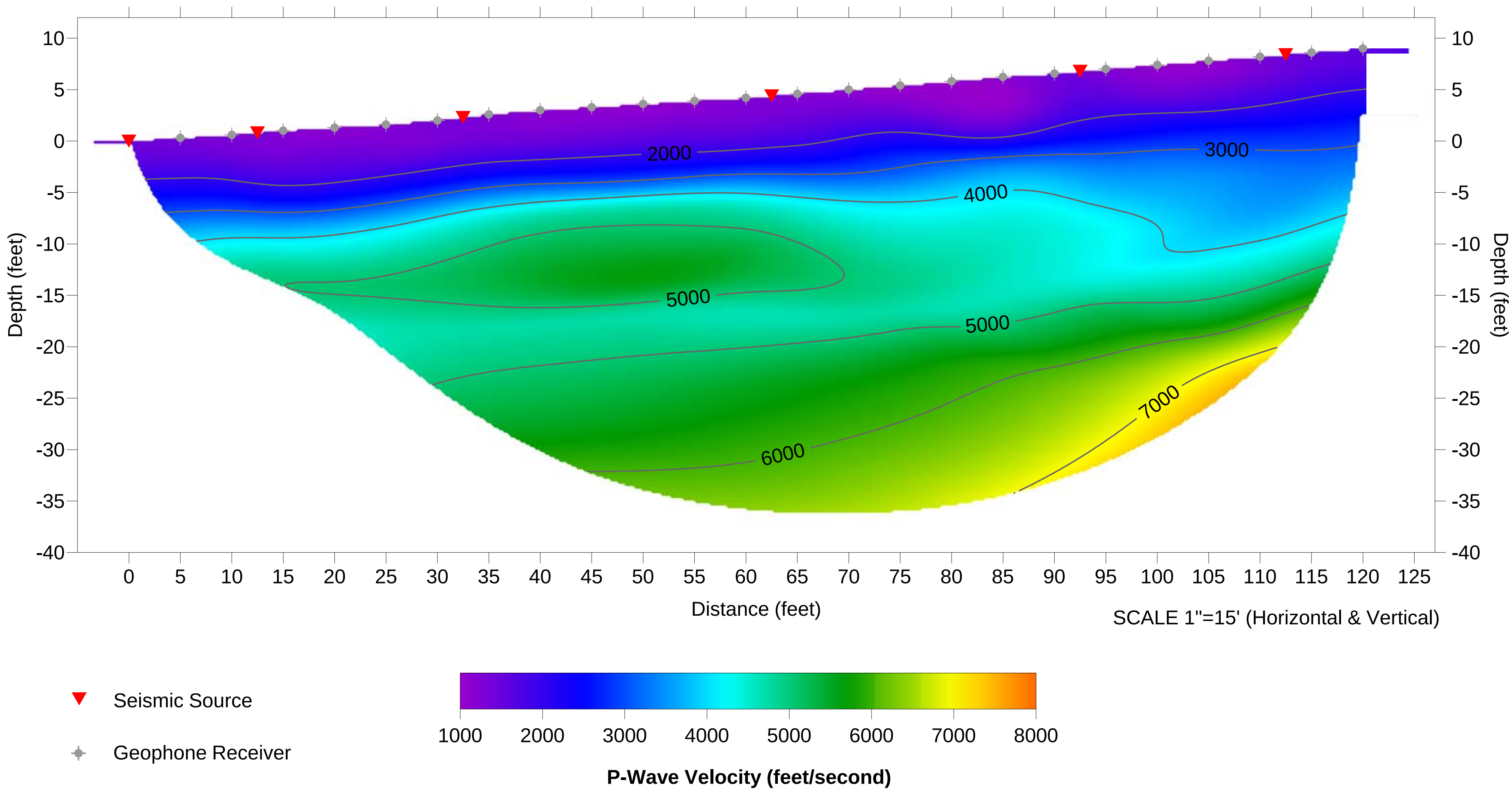


RMS error 2.8 %, Rayfract Version 3.31

SEISMIC LINE S-5

North 24° East →

REFRACTION TOMOGRAPHIC MODEL



RMS error 1.4 %, Rayfract Version 3.31