Appendix D

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

VISTA IRRIGATION DISTRICT - E RESERVOIR AND PUMP STATION

2330 Edgehill Road
Vista, California

Prepared By:
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Providing Professional Engineering Services Since 1959

GEOTECHNICAL INVESTIGATION

May 23, 2019

SCST No. 180433P4
Report No. 1R3

Neil Harper
Dudek
750 Second Street
Encinitas, CA 92024

Subject: GEOTECHNICAL INVESTIGATION
VISTA IRRIGATION DISTRICT
E RESERVOIR AND PUMP STATION
2330 EDGEHILL ROAD
VISTA, CALIFORNIA

Dear Mr. Harper:

SCST, LLC (SCST) is pleased to present our report describing the geotechnical investigation performed for the E Reservoir and Pump Station. We conducted the geotechnical investigation in general conformance with the scope of work presented in our proposal dated August 7, 2018. Based on the results of our investigation, we consider the planned development feasible from a geotechnical standpoint, provided the recommendations of this report are followed. If you have any questions, please call us at (619) 280-4321.

Respectfully submitted,
SCST, LLC

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EXECUTIVE SUMMARY

This report presents the results of the geotechnical investigation SCST, LLC (SCST) performed for the Vista Irrigation District (VID) E Reservoir Replacement and Pump Station project. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project.

We explored the subsurface conditions by drilling five borings to depths between about 9½ and 25½ feet below the existing ground surface using a truck-mounted drill rig equipped with a hollow-stem auger. An SCST geologist logged the borings and collected samples of the materials encountered for laboratory testing. SCST tested selected samples to evaluate pertinent soil classification and engineering properties to assist in developing geotechnical conclusions and recommendations. Additionally, we performed four seismic refraction traverses to evaluate rippability characteristics of the bedrock underlying the site.

The materials encountered in the borings consist of fill, colluvium, and igneous rock. The fill consists of loose to medium dense, silty and clayey sand with varying amounts of gravel and cobble and soft to medium stiff sandy clay. The colluvium consists of loose to medium dense clayey sand with varying amounts of gravel and cobble. The colluvium is underlain by igneous rock consisting of moderately soft to hard, weathered gabbro. Groundwater was not encountered in our borings.

The bottom of the planned reservoir may transition between fill and gabbro. The main geotechnical considerations affecting the planned structure and improvements are the presence of compressible fills and colluvium as well as difficult excavation conditions in gabbro. The contractor should expect to encounter hard gabbro. Special site preparation or foundation systems will be needed to reduce the potential for differential settlement.

1. INTRODUCTION

This report presents the results of the geotechnical investigation SCST, LLC (SCST) performed for the Vista Irrigation District (VID) E Reservoir Replacement and Pump Station project. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1 presents the site vicinity map.

2. SCOPE OF WORK

2.1 FIELD INVESTIGATION

We explored the subsurface conditions by drilling five borings to depths between about 9½ and 25½ feet below the existing ground surface using a truck-mounted drill rig equipped with a hollow-stem auger. Figure 2 shows the approximate locations of the borings. An SCST geologist logged the borings and collected samples of the materials encountered for laboratory testing. Logs of the borings are presented in Appendix I. Soils are classified according to the Unified Soil Classification System illustrated on Figure I-1.

Additionally, we performed four seismic refraction traverses to evaluate rippability characteristics of the bedrock underlying the site. Figure 2 presents the general locations of the seismic traverses. Appendix III presents the detailed results of the seismic refraction survey.

2.2 LABORATORY TESTING

Selected samples were tested to evaluate pertinent soil classification and engineering properties and to enable the development of geotechnical conclusions and recommendations. The laboratory tests consisted of in situ moisture and density, particle-size distribution, Atterberg limits, expansion index, corrosivity, R-value, and direct shear. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix II.

2.3 ANALYSIS AND REPORT

The results of the field and laboratory tests were evaluated to develop conclusions and recommendations regarding:

- Subsurface conditions beneath the site
- Potential geologic hazards
- Criteria for seismic design in accordance with the 2016 California Building Code (CBC)
- Site preparation and grading
- Excavation characteristics
- Appropriate alternatives for foundation support along with geotechnical engineering criteria for design of the foundations

- Retaining wall design
- Resistance to lateral loads
- Estimated foundation settlements
- Support for concrete slabs-on-grade
- Lateral pressures for the design of retaining walls
- Soil corrosivity

3. SITE DESCRIPTION

The site is located at 2330 Edgehill Road in Vista, California. The site is bounded by undeveloped land to the north, residential properties to the east and west, and Edgehill Road to the south. Improvements at the site consist of an existing 1.5-million-gallon reservoir, a pressure regulating station, and paved asphalt concrete (AC) roads with concrete curbs. Figure 1 presents a site vicinity map.

Topographically, the site slopes towards the southwest. Elevations vary between approximately 770 feet above mean sea level (MSL) on the unnamed access road located northeast of the reservoir to approximately 735 feet MSL near the existing pressure regulating station. Vegetation consists of trees, shrubs, and native plants.

4. PROPOSED DEVELOPMENT

A new reservoir will be constructed with the capacity of up to 4 million gallons depending on the site area, space planning, and space allowance for a new pump station. It is our understanding that the reservoir improvements will consist of asphalt pavement, steel security fence, and lighting. Minor grading of the existing slopes around the proposed reservoir may be recommended.

5. GEOLOGY AND SUBSURFACE CONDITIONS

Based on published geologic mapping (Kennedy and Tan, 2007), the geologic materials underlying the project site consist of undivided gabbro. Figure 2 presents a subsurface exploration map in the vicinity of the site. Per geologic mapping, the site is characterized by fill and colluvium underlain by weathered igneous rock. Descriptions of the materials as encountered in our borings are presented below. Figure 3 presents a regional geology map. Figure 4 presents a geologic cross-section of the site.

Fill (Qf): Fill was encountered in borings B-1, B-2, B-3, and B-5. The fill consists of loose to medium dense, silty and clayey sand with varying amounts of gravel and cobble and soft to medium stiff sandy clay. The fill extends to depths between ½ foot up to about 13 feet below the existing ground surface.

Colluvium (Qcol): Colluvium was encountered below the fill in borings B-1, B-2, B-3, and B-5 and at the surface in boring B-4. The colluvium consists of medium stiff sandy clay with trace gravel and loose to medium dense clayey sand with varying amounts of gravel and cobble. These materials were encountered to depths between about 2 to 5 feet and 13 to 19 feet below the existing ground surface.

Igneous Rock - Gabbro (Kgb): Generally, the igneous rock encountered is moderately weathered, moderately soft to hard, gabbro. Zones of hard rock and auger refusal were encountered in borings B-4 and B-5 at depths of about 9½ feet and 15½ feet, respectively.

Groundwater: Groundwater was not encountered during our investigation. Groundwater is anticipated to exist at a depth of greater than 10 feet below the existing ground surface. Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage. Because groundwater rise or seepage is difficult to predict, such conditions are typically mitigated if and when they occur.

6. GEOLOGIC HAZARDS

6.1 FAULTING AND SURFACE RUPTURE

The closest known active fault is the Newport-Inglewood-Rose Canyon Fault Zone located about 13.4 miles (21.6 kilometers) southwest of the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. No active faults are known to underlie or project toward the site. Therefore, the probability of fault rupture is considered low.

6.2 CBC SEISMIC DESIGN PARAMETERS

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site. The site coefficients and adjusted maximum considered earthquake spectral response accelerations in accordance with the 2016 California Building Code are presented below:

2016 California Building Code Seismic Design Parameters

Site Coordinates	
Latitude	**Longitude**
33.2121° (N)	-117.2011° (W)

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Site Coefficients, F_a	1.136
Site Coefficients, F_v	1.748
Mapped Spectral Response Acceleration at Short Period, S_s	1.910g
Mapped Spectral Response Acceleration at 1-Second Period, S_1	0.326g
Design Spectral Acceleration at Short Period, S_{DS}	0.689g
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.380g
Design Peak Ground Acceleration, PGA_M	0.437g

6.3 LIQUEFACTION AND DYNAMIC SETTLEMENT

Liquefaction occurs when loose, saturated, generally fine sands and silts are subjected to strong ground shaking. The soils lose shear strength and become liquid, potentially resulting in large total and differential ground surface settlements as well as possible lateral spreading during an earthquake. Due to the lack of shallow groundwater and given the relatively dense nature of the materials beneath the site, the potential for liquefaction and dynamic settlement to occur is considered negligible.

6.4 LANDSLIDES AND SLOPE STABILITY

Evidence of landslides or slope instabilities was not observed. The potential for landslides or slope instabilities to occur at the site is considered low.

6.5 TSUNAMIS, SEICHES, AND FLOODING

The site is not located within areas mapped as susceptible to tsunamis (California Emergency Management Agency, 2009). Therefore, damage due to tsunamis is considered negligible. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any confined bodies of water; therefore, the potential for a seiche to affect the site is low. The site is not located within a flood zone or dam inundation area (FIRM, 2012).

6.6 SUBSIDENCE

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered low.

6.7 HYDRO-CONSOLIDATION

Hydro-consolidation can occur in recently deposited (less than 10,000 years old) sediments that were deposited in a semi-arid environment. Examples of such sediments are aeolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore space between particle grains can re-adjust when inundated by groundwater, causing the material to consolidate. The relatively loose nature of the materials underlying the site may be susceptible to hydro-consolidation.

7. GEOPHYSICAL SURVEY SUMMARY

Four seismic refraction traverses were conducted along the proposed reservoir. Appendix III presents the results of the survey. Based on the results, it appears the study areas are underlain by low-velocity materials (e.g. fill and colluvium-low failure PSI) in the near surface and high-velocity igneous bedrock at depth (high failure PSI). Distinct vertical and lateral velocity variations are evident in the tomography models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the site. In addition, remnant boulder core stones appear to be present in the subsurface in some areas.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be recommended depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected in excavated materials.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree “hardness”. Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2011), as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate, and that rock characteristics such as fracture spacing and orientation play a significant role in evaluating rock quality or rippability. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator. A contractor with excavation experience in similarly difficult conditions should be consulted for expert advice on excavation methodology, equipment, and production rate.

For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

TABLE 1 – RIPPABILITY CLASSIFICATION

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Probably Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. Figures 5 through 8 present tomography profiles showing the relationship of elevation in regard to the depth of hard gabbro.

8. CONCLUSIONS

The bottom of the planned reservoir is likely to transition between gabbro and fill. The main geotechnical consideration affecting the planned structure and improvements are the presence of compressible fills and colluvium as well as difficult excavation conditions in gabbro. The contractor should expect to encounter hard gabbro. Special site preparation or foundation systems will be needed to reduce the potential for differential settlement.

9. RECOMMENDATIONS

9.1 SITE PREPARATION AND GRADING

9.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, and debris. Subsurface improvements that are to be abandoned should be removed, and the resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Pipeline abandonment can consist of capping or rerouting at the project perimeter and removal within the project perimeter. If appropriate, abandoned pipelines can be filled with grout or slurry as recommended by and observed by the geotechnical consultant.

9.1.2 Remedial Grading

The existing fill and colluvium are not considered suitable for support of the proposed improvements or reservoir. The existing fill and colluvium should be excavated to competent gabbro beneath the proposed foundations for the reservoir. Where necessary, concrete or a 2-sack sand/cement slurry mix can be placed between the formation and design bottom of footings to accommodate bearing on gabbro. Remedial grading beneath the pump station, as well as site improvements such as retaining walls, miscellaneous flatwork and walkways should consist of excavating to a minimum depth of 2 feet below the bottom of the lowest planned footing elevation or planned subgrade and replacing with suitable compacted fill materials. Horizontally, the excavations should extend at least 2 feet outside the planned hardscape and pavements, up to existing improvements, or to the limits of grading, whichever is less.

An SCST representative should observe conditions exposed in the bottom of the excavations to assess whether additional excavation is recommended.

9.1.3 Compacted Fill

Fill should be moisture conditioned to near optimum moisture content and compacted to at least 95% relative compaction. Prior to placing fill, the surface exposed at the bottom of excavations should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction.

Fill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. The maximum dry density and optimum moisture content for evaluating relative compaction should be evaluated in accordance with ASTM D1557. Utility trench backfill beneath structures, pavements, and hardscape should be compacted to at least 95% relative compaction. The top 12 inches of subgrade beneath pavements should be compacted to at least 95%.

9.1.4 Expansive Soil

To reduce the potential for expansive heave, soils with an expansion index greater than 50 should be sub-excavated 2 feet below the planned flatwork subgrade elevations. Granular, free-draining material with a sand equivalent of 20 or more that meets the gradation requirements from the Greenbook Specifications for Structural Backfill, with an expansion index of 50 or less, should be used as replacement fill. Based on our investigation, the on-site materials near the surface will meet this expansion index criteria. Clays and silts, when encountered, should not be used as fill materials.

9.1.5 Excavation Characteristics

It is anticipated that excavations in fill and colluvium can be achieved with conventional earthwork equipment in good working order. For anticipated excavation characteristics of gabbro, refer to the geophysical survey summary section of this report. Excavations in fill and colluvium may be locally unstable and may contain construction debris. Difficult drilling and excavation should be anticipated in zones of gabbro. Non-rippable gabbro exists on-site, and difficult excavation should be anticipated. Rock breakers, carbide/diamond-tipped equipment and/or blasting may be recommended to excavate less weathered rock. Localized “core stones” or large boulder inclusions may also be encountered. Excavations in rock may generate oversized material that will require extra effort to crush or haul off-site. Special handling may be recommended to excavate zones of hard rock, as auger refusal was encountered. Contract documents should specify that the contractor mobilize equipment capable of excavating and compacting the igneous rock.

9.1.6 Oversized Material

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 3 inches in largest dimension. Oversized material should be broken down to no greater than 3 inches in largest dimension for use in fill, used as landscape material, or disposed of off-site.

9.1.7 Temporary Excavations

Temporary slopes greater than 4 feet in the fill and colluvium should not be steeper than 1½:1 (horizontal: vertical) per Cal/OSHA type C soil classification and in the weathered gabbro should not be steeper than ¾:1 (horizontal: vertical) per Cal/OSHA type A soil classification. The faces of temporary slopes should be inspected daily by the contractor’s Competent Person before personnel are allowed to enter the excavation. Zones of potential instability, sloughing, or raveling should be brought to the attention of the Engineer and corrective action implemented before personnel begin working in the trench. Shoring is recommended for slopes steeper than those described above.

9.1.8 Temporary Shoring

For design of cantilevered shoring with level backfill, an active earth pressure equal to a fluid weighing 40 pounds per cubic foot (pcf) can be used. The surcharge loads from traffic and construction equipment adjacent to the shored excavation can be modeled by assuming an additional 2 feet of soil behind the shoring.

For design of soldier piles, an allowable passive pressure of 375 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 4,000 psf, can be used for soil above the groundwater level. An allowable passive pressure of 150 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 2,000 psf, can be used for soil below the groundwater level. Hydrostatic pressure should be applied below the groundwater level.

Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be recommended throughout. The soldier piles should be designed for the full-anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressure but can be limited to a maximum value of 400 psf.

Installation of soldier piles below groundwater (or dewatered soil) is recommended to have special construction techniques and equipment, such as temporary casing and/or drilling slurry to cope with groundwater and potential heavy caving. Other installation methods may be available. Contract documents should specify that the contractor mobilize equipment capable of installing piles below groundwater (or dewatered soil) to reduce the potential that claims for delays or extra work will arise.

Piles should be filled with concrete immediately after drilling. The concrete should be pumped to the bottom of the drilled holes using the tremie method. If casing is used, the casing should be removed as the concrete is placed, keeping the level of the concrete at least 5 feet above the bottom of the casing.

9.1.9 Temporary Dewatering

Temporary dewatering may be recommended to construct the proposed structure with a subterranean level. A specialty contractor should be retained to design and perform the dewatering. The design should incorporate measures to ensure the dewatering does not induce settlement of adjacent improvements. Generally, groundwater should be 3 feet or more below the planned temporary excavation bottom to provide a working surface.

9.1.10 Imported Soil

Imported soil should consist of predominately granular soil, free-draining material, free of organic matter and rocks greater than 3 inches. The imported soil should have a sand equivalent of 20 or more, an expansion index of 50 or less, and meet the gradation requirements from the Greenbook Specifications for Structural Backfill. If appropriate, imported soil should be inspected and tested by SCST prior to transport to the site.

9.1.11 Slopes

All permanent slopes should be constructed no steeper than 2:1 (horizontal:vertical). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (horizontal:vertical). It is our opinion that cut slopes constructed no steeper than 2:1 (horizontal:vertical) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

9.1.12 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from the structure and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop.

9.1.13 Grading Plan Review

SCST should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented and that no revised recommendations are needed due to changes in the development scheme.

9.2 FOUNDATIONS

9.2.1 Shallow Spread Footings

The planned tank can be supported on spread footings with bottom levels on competent gabbro. The fills beneath the proposed footings, as encountered, should be completely removed to gabbro. To accommodate bearing on gabbro, concrete or 2-sack sand/cement slurry can be placed between the formation and design bottom of footings. The planned pump station can be supported on spread footings with bottom levels on compacted fill.

Footings should extend at least 24 inches below the lowest adjacent finished grade. A minimum width of 24 inches is recommended for continuous footings. Isolated footings should be at least 24 inches wide. A bearing capacity of 2,500 psf can be used for footings bearing on compacted fill. For footings bearing on gabbro, 8,000 psf can be used. The bearing capacity can be increased by 500 psf for each foot of depth below the minimum and 250 psf for each foot of width beyond the minimum up to maximums of 5,000 psf for footings bearing on compacted fill and 10,000 psf for footings bearing on gabbro. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 7 feet exists between the lower outside footing edge and the face of the slope.

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. A friction factor of 0.35 can be used. Passive pressures can be computed using lateral pressure values of 375 and 425 psf per foot of depth, respectively for compacted fill and gabbro, below the ground surface for level ground conditions. Reductions for sloping ground should be made. The passive pressure can be increased by 1/3 when considering the total of loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

9.2.2 Mat Foundations

Mat foundations with bottom levels on gabbro may also be used to support the proposed tank. If this option is selected, the fills beneath the proposed mats, as encountered, should be completely removed to gabbro. To accommodate bearing on gabbro, concrete or 2-sack sand/cement slurry can be placed between the formation and design bottom of mats.

Mat foundations should have a minimum thickness of 12 inches with steel reinforcement top and bottom, both ways, and should have turned down edges embedded 6 inches below ground surface.

A modulus of subgrade reaction (k) of 250 pounds per cubic inch (pci) and an allowable bearing capacity of 8,000 pounds per square foot (psf) can be used. Above the groundwater level, the bearing value can be increased by ⅓ when considering the total of loads, including wind or seismic forces. Mats located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 7 feet exists between the lower outside footing edge and the face of the slope. Groundwater seepage should be anticipated.

9.2.3 Settlement Characteristics

Total foundation static settlements for conventional foundations are estimated to be less than 1 inch. Differential settlements are estimated to be less than ¾ inch over a distance of 50 feet. Static settlements should be completed shortly after structural loads are applied.

9.2.4 Foundation Excavation Observations

A representative from SCST should observe the foundation excavations prior to forming or placing reinforcing steel.

9.2.5 Foundation Plan Review

SCST should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

9.3 EXTERIOR FLATWORK

Exterior slabs not subjected to vehicular loads should be at least 5 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The design engineer should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

9.4 CONVENTIONAL RETAINING WALLS

9.4.1 Foundations

The recommendations provided in the foundation section of this report are also applicable to conventional retaining walls.

9.4.2 Lateral Earth Pressures

The active earth pressure for the design of unrestrained retaining walls with level backfill can be taken as equivalent to the pressure of a fluid weighing 40 pcf. The at-rest earth pressure for the design of restrained retaining walls with level backfills can be taken as equivalent to the pressure of a fluid weighing 60 pcf. These values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain expansive clay soils. An additional 20 pcf should be added to these values for walls with a 2:1 (horizontal:vertical) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. If any other surcharge loads are anticipated, SCST should be contacted for the necessary increase in soil pressure.

For any portion of the wall below the groundwater level, the active earth pressure for the design of unrestrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 20 pounds per cubic foot (pcf) plus full hydrostatic pressure. The at-rest earth pressure for the design of restrained earth retaining structures with level backfills can be taken as equivalent to the pressure of a fluid weighing 30 pcf plus full hydrostatic pressure. An additional 20 pcf should be added to these values for walls with a 2:1 (horizontal: vertical) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a backdrain to reduce the accumulation of hydrostatic pressures. Backdrains may consist of a 2-foot-wide zone of ¾-inch crushed rock. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided, or a perforated pipe should be installed at the base of the backdrain and sloped to discharge to a suitable storm drain facility. As an alternative, a geocomposite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project engineer should provide waterproofing specifications and details. Figure 9 presents typical conventional retaining wall backdrain details.

9.4.3 Seismic Earth Pressure

If recommended, the seismic earth pressure can be taken as equivalent to the pressure of a fluid weighing 20 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, static active earth pressure. The passive pressure and bearing capacity can be increased by ⅓ in evaluating the seismic stability of the wall.

9.4.4 Backfill

Wall backfill should consist of granular, free-draining material, with a sand equivalent of 20 or more, with an expansion index of 50 or less, that meets the gradation requirements from the Greenbook Specifications for Structural Backfill. Expansive or clayey soil should not be used. Additionally, fill within 3 feet from the back of the wall should not contain rocks greater than 3 inches in dimension. We anticipate that a portion of the on-site soils will be suitable for wall backfill. Backfill should be compacted to at least 90% relative compaction. Backfill should not be placed until walls have achieved adequate structural strength. Compaction of wall backfill will be necessary to minimize settlement of the backfill and overlying settlement sensitive improvements. However, some settlement should still be anticipated. Provisions should be made for some settlement of concrete slabs and pavements supported on backfill. Additionally, any utilities supported on backfill should be designed to tolerate differential settlement.

9.5 PIPELINES

9.5.1 Thrust Blocks

For level ground conditions, a passive earth pressure of 375 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 150 psf per foot should be used below groundwater level, if encountered.

9.5.2 Modulus of Soil Reaction

A modulus of soil reaction (E’) of 2,000 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

9.5.3 Pipe Bedding

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 30 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

9.5.4 Cutoff Walls

Where pipeline inclinations exceed 15 percent, cutoff walls are recommended in trench excavations. Additionally, we do not recommend that open graded rock be used for pipe bedding or backfill because of the potential for piping erosion. The recommended bedding is clean sand having a sand equivalent not less than 30. Alternatively, 2-sack sand-cement slurry can be used for the pipe bedding. If sand-cement slurry is used for pipe bedding to at least 1 foot over the top of the pipe, cutoff walls are not considered necessary. The need for cutoff walls should be further evaluated by the project civil engineer designing the pipeline.

9.5.5 Backfill

Excavated material that meets the conditions of the 2018 Greenbook Specifications and is free of organic debris and rocks greater than 3 inches in any dimension are generally expected to be suitable for use as backfill. Imported material should not contain rocks greater than 3 inches in any dimension or organic debris. Imported material should have an expansion index of 50 or less. SCST should observe and, if appropriate, test proposed imported materials before they are delivered to the site. Backfill should be placed in lifts 8 inches or less in loose thickness, moisture conditioned to optimum moisture content or slightly above, and compacted to at least 90% relative compaction. The top 12 inches of soil beneath pavement subgrade should be compacted to at least 95% relative compaction.

9.6 PAVEMENT SECTION RECOMMENDATIONS

The pavement support characteristics of the soils encountered during our investigation are considered low. An R-value of 33 was used for design of preliminary pavement sections. The actual R-value of the subgrade soils should be verified after grading and final pavement sections are provided. Based on an R-value of 33, the following pavement structural sections are recommended for the assumed Traffic Indexes.

Flexible Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
Parking / Bicycle Trail	4.5	3	7
Drive Lanes	6.0	4	9
Fire Lanes	7.5	5	12

The top 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. Soft or yielding areas should be removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the “Greenbook” and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. Materials and methods of construction should conform to good engineering practices.

9.7 SOIL CORROSIVITY

Representative samples of the on-site soils were tested to evaluate corrosion potential. The test results are presented in Appendix II. Based on the results of our laboratory testing, the on-site soils are not considered to be corrosive. According to the Caltrans Corrosion Guidelines (2018), a site is considered to be corrosive if the chloride concentration is 0.05 percent (500 ppm) or greater, sulfate concentration is 0.15 percent (1500 ppm) or greater, the pH is 5.5 or less, or the resistivity is less than 1,100 ohm-cm.

The project design engineer can use the sulfate results in conjunction with ACI 318 to specify the water/cement ratio, compressive strength, and cementitious material types for concrete exposed to soil. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

10. GEOTECHNICAL ENGINEERING DURING CONSTRUCTION

The geotechnical engineer should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of the geotechnical engineer during construction will enable an evaluation of the exposed conditions and modifications of the recommendations in this report or development of additional recommendations in a timely manner.

11. CLOSURE

SCST should be advised of any changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

12. REFERENCES

American Concrete Institute (ACI) (2011), Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary, August.

California Building Standards Commission (2016), 2016 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on the 2015 International Building Code, Effective Date: January 1, 2016.

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California Geological Survey (2002), Simplified Fault Activity Map of California", compiled by Jennings, C.W. and Saucedo, G. J., 1999, revised Toppozada, T. and Branum, D., 1999.

Caltrans (2015), Standard Specifications.

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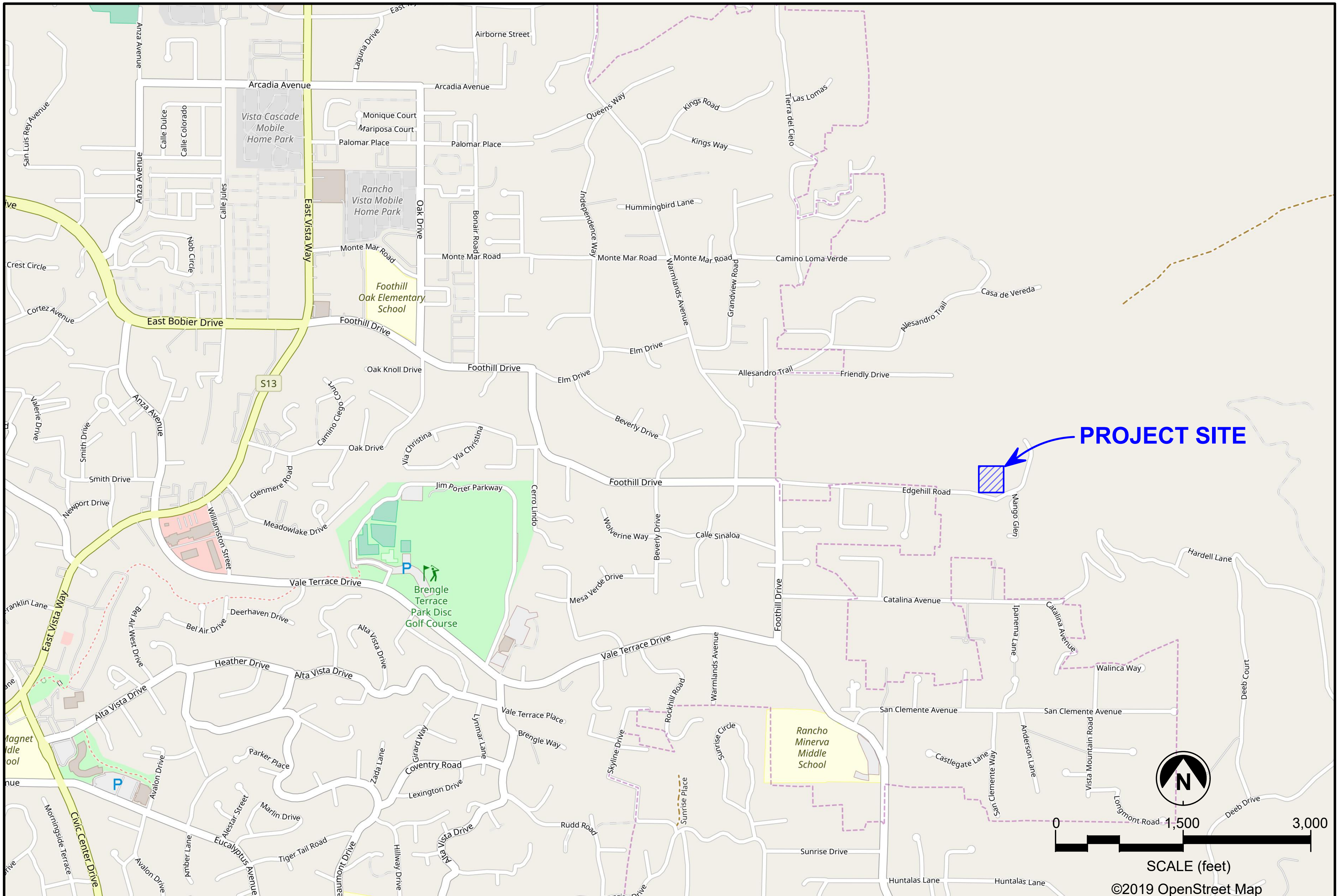
Jennings, C.W. and Bryant, W.A. (2010), Fault Activity Map of California, California Geologic Survey, Geologic Data Map No. 6.

Kennedy and Tan (2007), Geologic Map of the Oceanside 30'x60' Quadrangle, California.

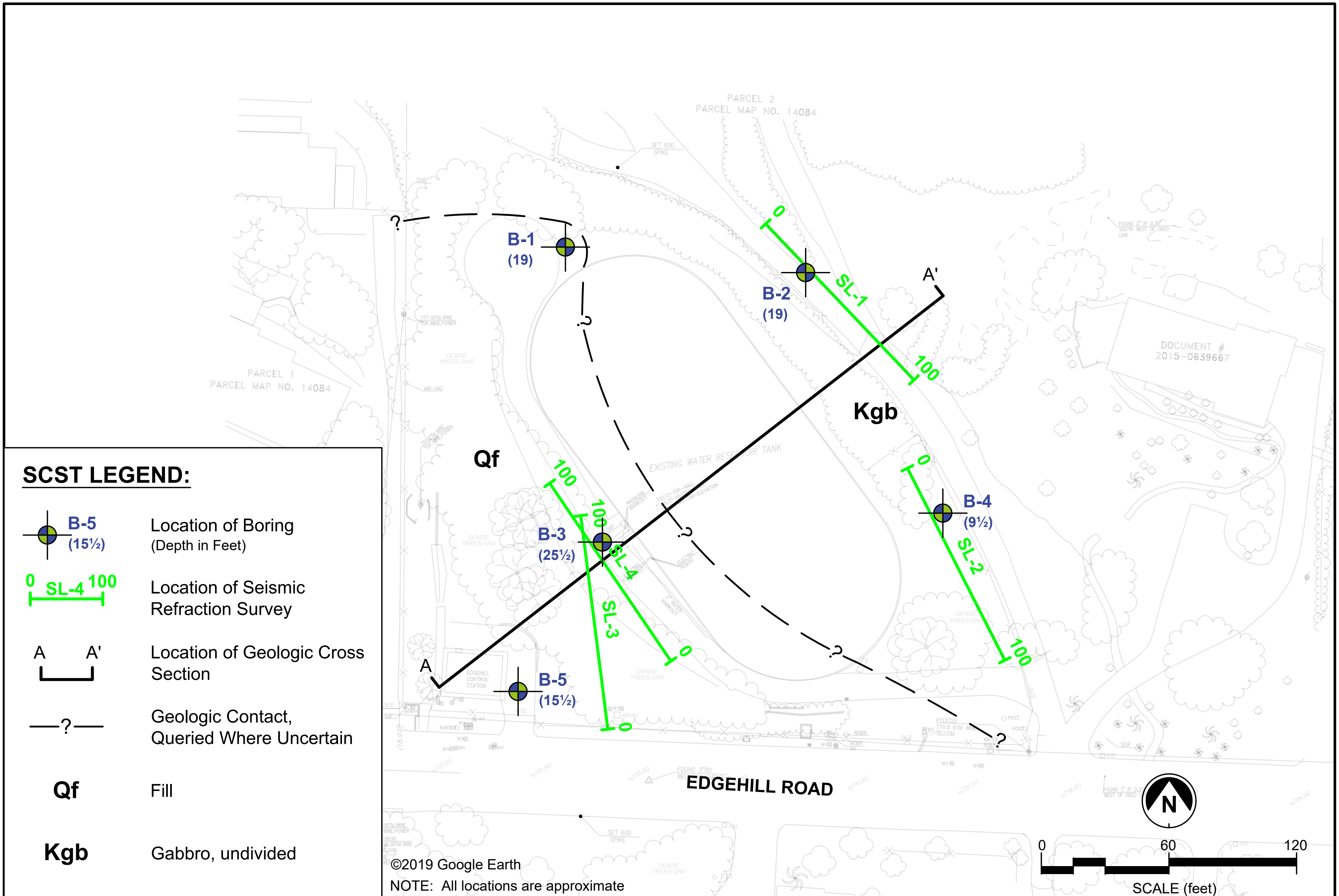
Public Works Standards, Inc. (2018), "Greenbook" Standard Specifications for Public Works Construction, 2018 Edition.

Vista Irrigation District (2018), Standard Specifications, revised May.



SCST, LLC	**SITE VICINITY MAP** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: NNW/DTC Job No.: 180433P4-1R3

Figure: 1



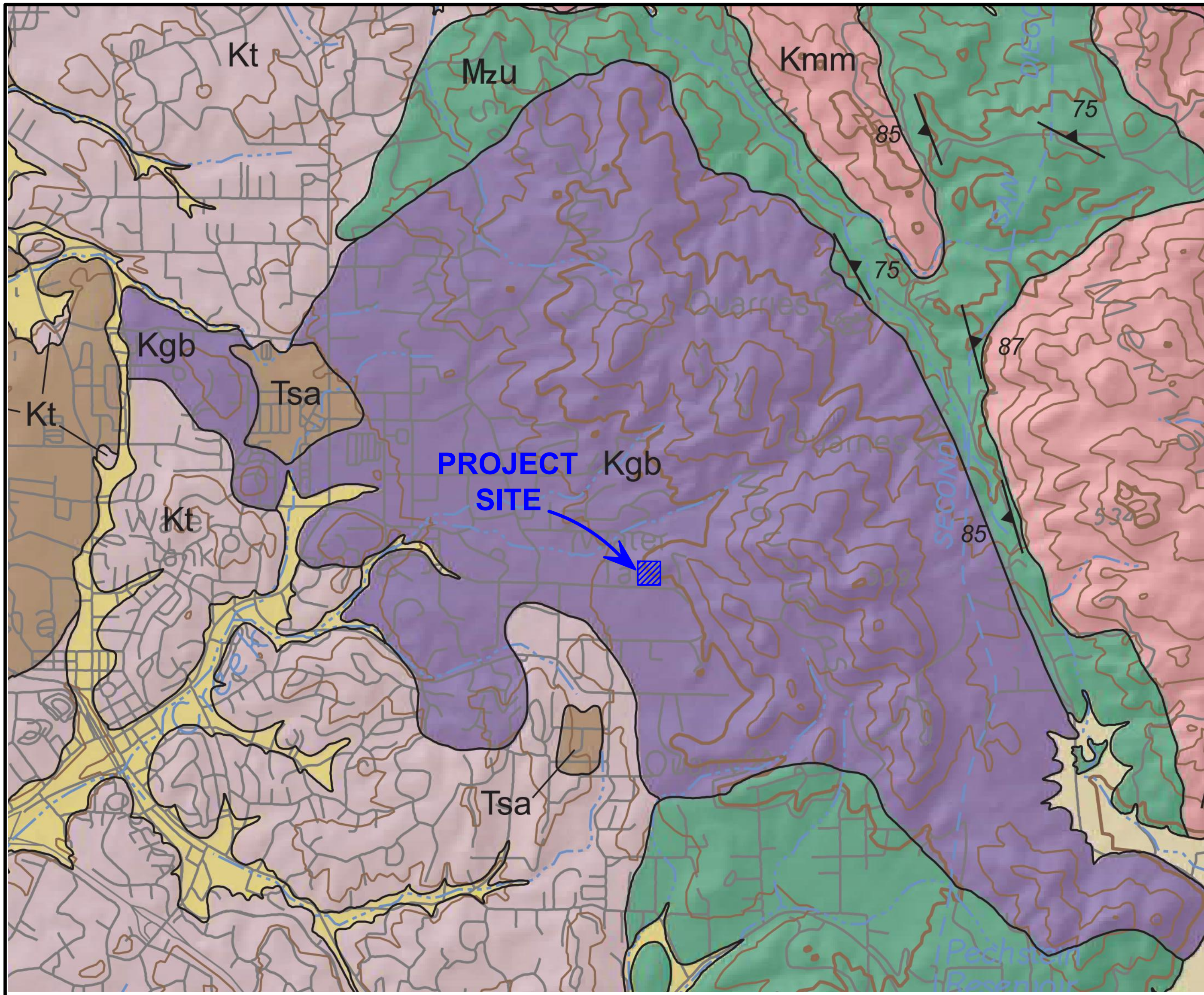
SCST, LLC

SUBSURFACE EXPLORATION MAP

VID, E Reservoir Replacement and Pump Station
Vista, California

Date: May, 2019
By: NNW/DTC
Job No.: 180433P4-1R3

Figure: 2



EXPLANATION:

Qa	Alluvial flood-plain deposits
Qya	Young alluvial flood-plain deposits
Tsa	Santiago Formation
Kt	Tonalite, undivided
Kgb	Gabbro, undivided
Kmm	Monzogranite of Merriam Mountain
Mzu	Metasedimentary and metavolcanic rocks, undivided

70 U D

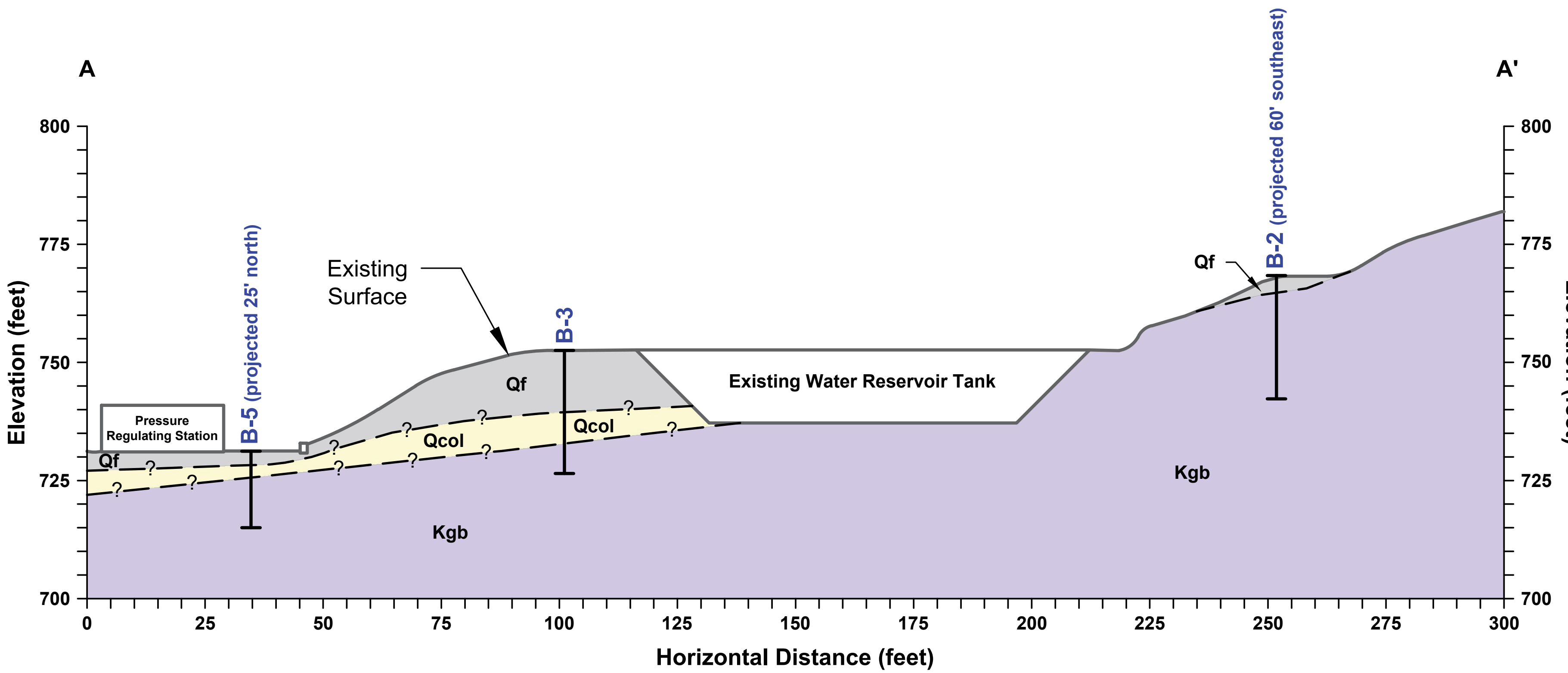
Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Strike and dip of metamorphic foliation
Inclined

0 4,000 8,000
SCALE (feet)

Reference:
Kennedy, M.P. and Tan, S.S. (2007), Geologic Map of the Oceanside 30' x 60' Quadrangle, California, California Geological Survey, Scale 1:100,000

SCST, LLC	**REGIONAL GEOLOGY MAP** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: NNW/DTC Job No.: 180433P4-1R3	Figure: 3



SCST LEGEND:

B-5 Approximate Location of Boring

—?— Approximate Geologic Contact, Queried Where Uncertain

Qf Fill

Qcol Colluvium

Kgb Gabbro, undivided

SCALE
1" = 35'

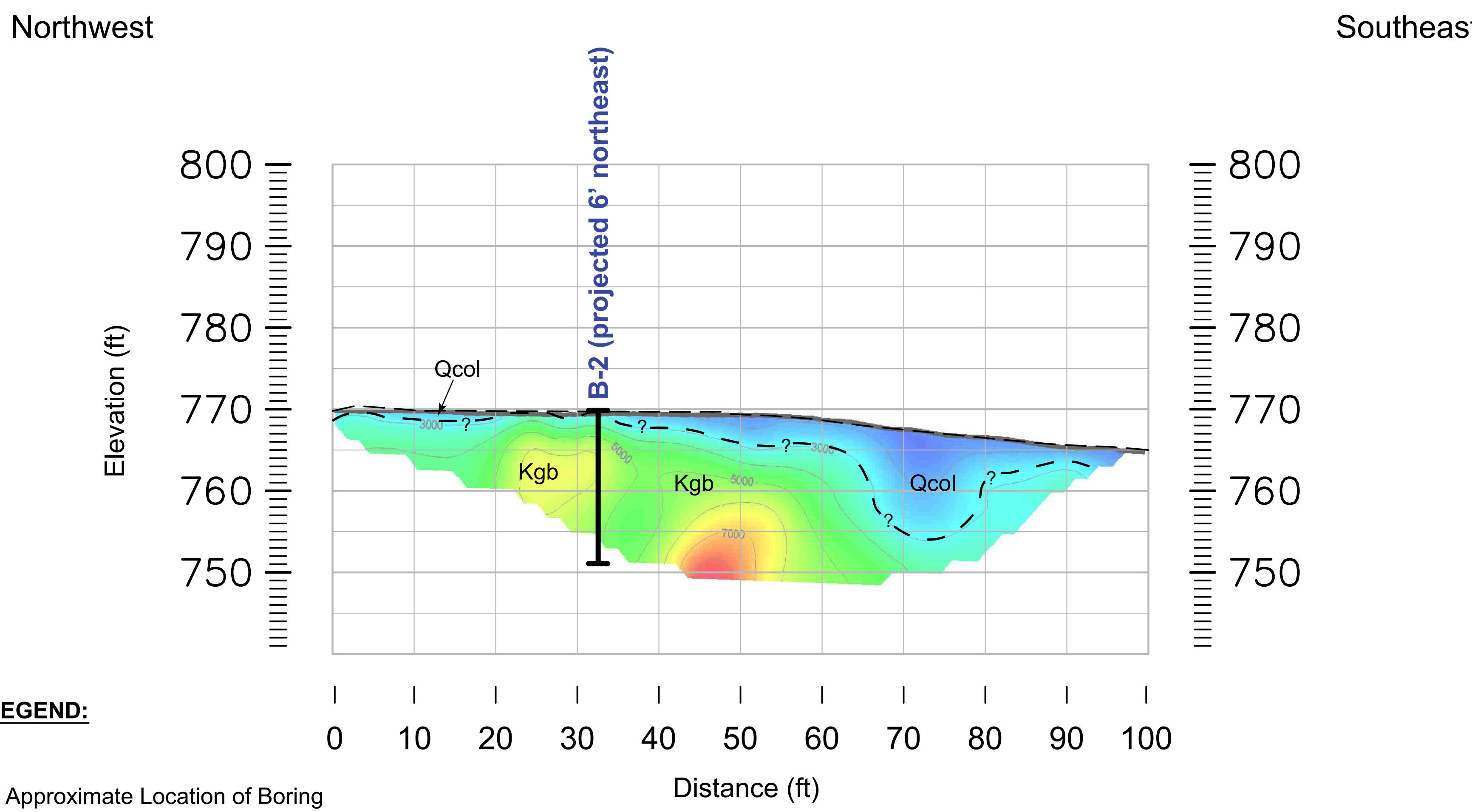
NOTE: All Locations are Approximate

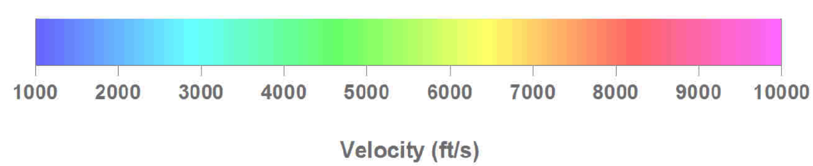
SCST, LLC

GEOLOGIC CROSS-SECTION
VID, E Reservoir Replacement and Pump Station
Vista, California

Date: May, 2019
By: NNW/DTC
Job No.: 180433P4-1R3

Figure: 4





SCST LEGEND:

B-2 Approximate Location of Boring

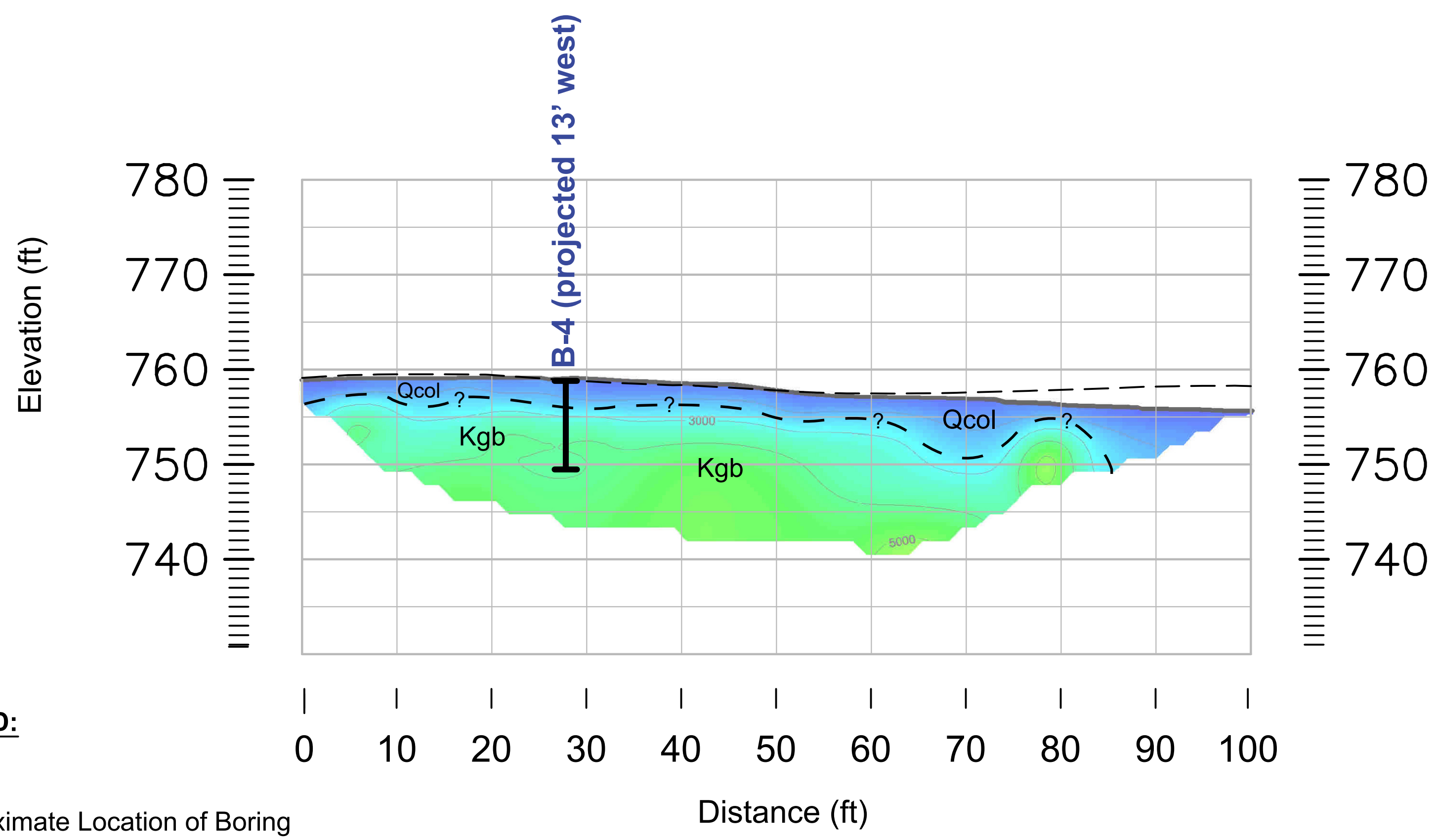
—?— Approximate Geologic Contact, Queried Where Uncertain

Qcol Colluvium

Kgb Gabbro, undivided

SCST, LLC	**TOMOGRAPHY PROFILE SL-1** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: EMW Job No.: 180433P4-1R3	Figure: 5

Northwest Southeast



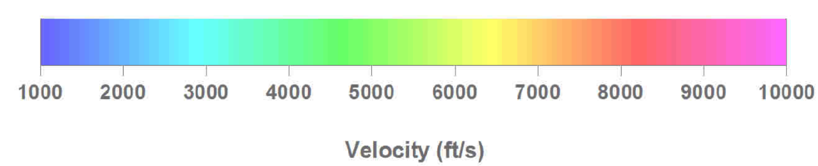
SCST LEGEND:

B-4 Approximate Location of Boring

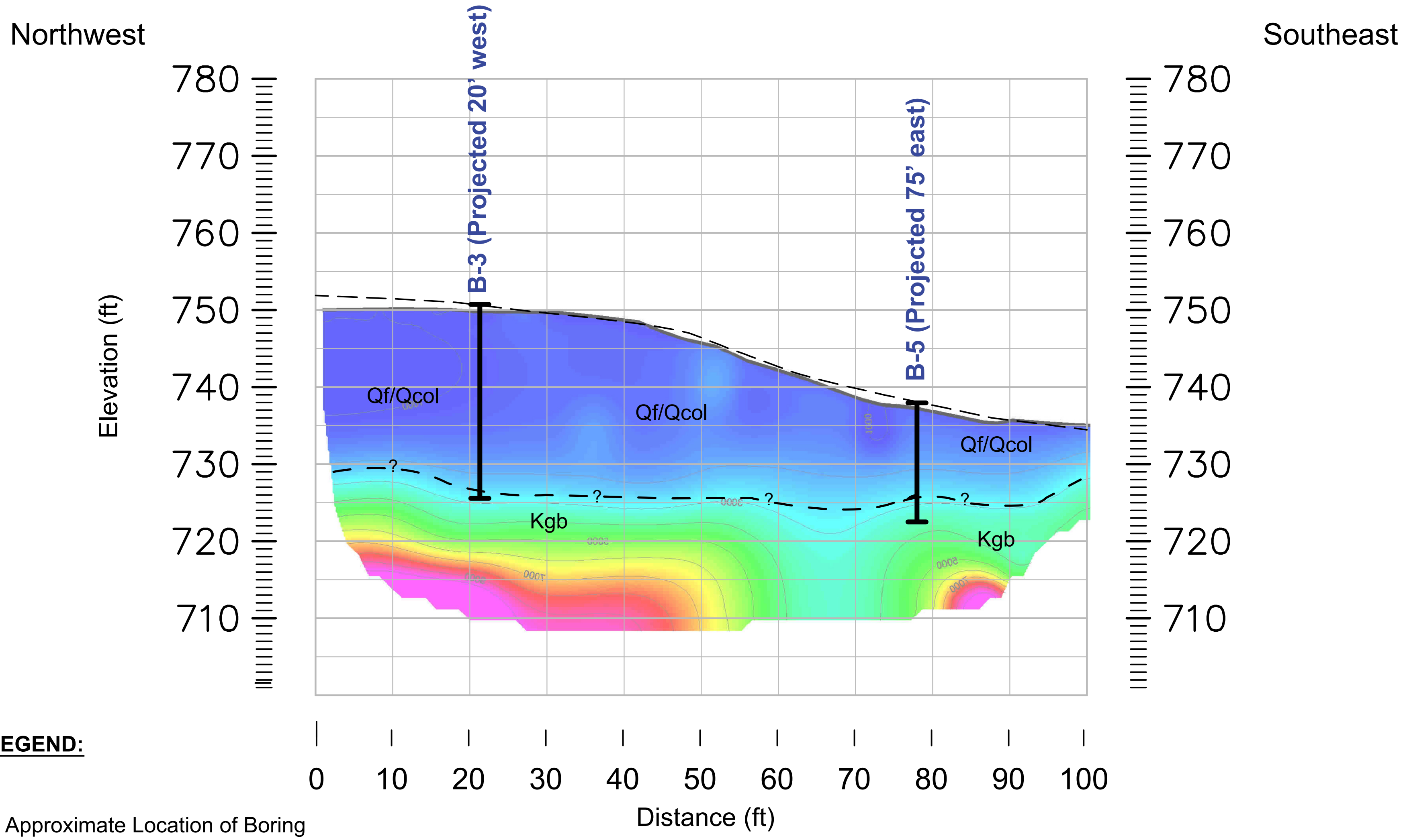
—?— Approximate Geologic Contact, Queried Where Uncertain

Qcol Colluvium

Kgb Gabbro, undivided



SCST, LLC	**TOMOGRAPHY PROFILE SL-2** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: EMW Job No.: 180433P4-1R3	Figure: 6



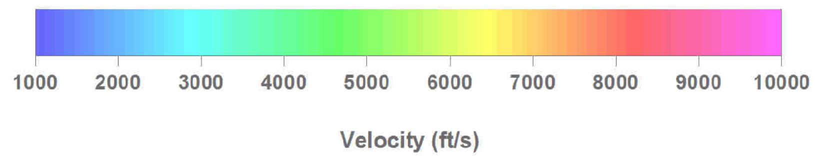
SCST LEGEND:

B-5 Approximate Location of Boring

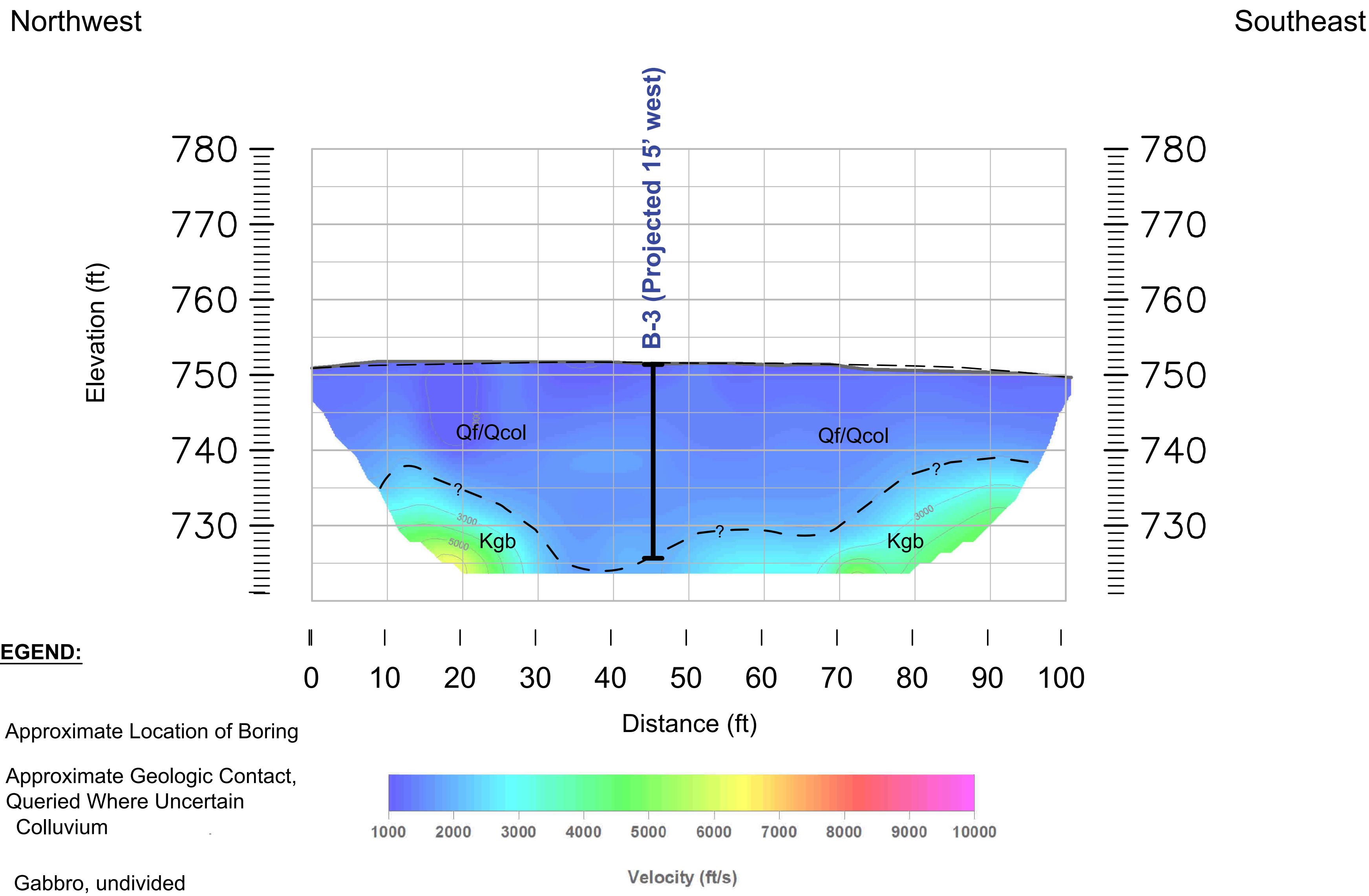
—?— Approximate Geologic Contact, Queried Where Uncertain

Qcol Colluvium

Kgb Gabbro, undivided



SCST, LLC	**TOMOGRAPHY PROFILE SL-3** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: EMW Job No.: 180433P4-1R3	Figure: 7



SCST LEGEND:

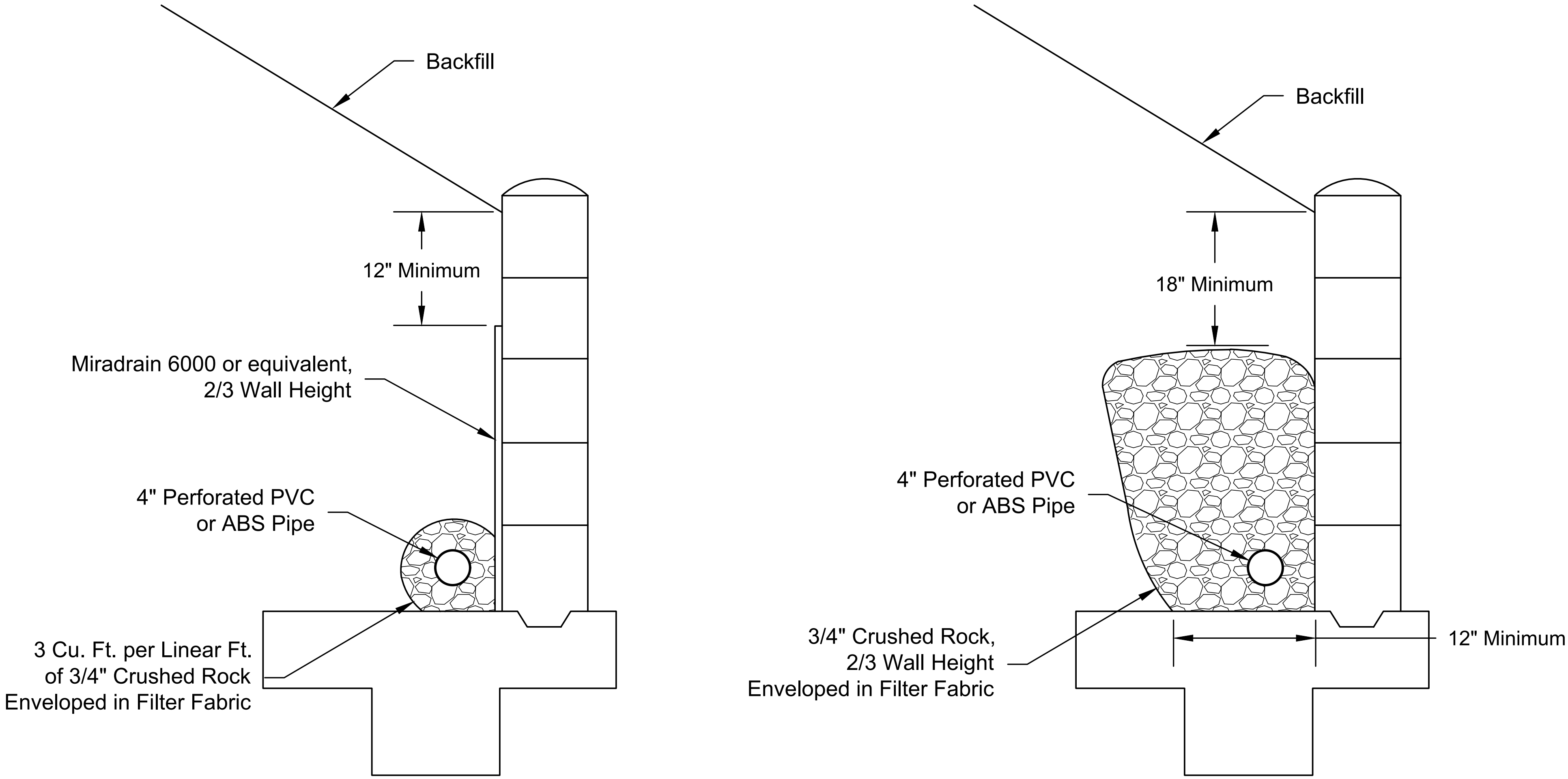
B-3 Approximate Location of Boring

—?— Approximate Geologic Contact, Queried Where Uncertain

Qcol Colluvium

Kgb Gabbro, undivided

SCST, LLC	**TOMOGRAPHY PROFILE SL-4** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: EMW Job No.: 180433P4-1R3	Figure: 8



NOT TO SCALE

NOTES:

1) Dampproof or waterproof back of wall following architect's specifications.
2) 4" minimum perforated pipe, SDR35 or equivalent, holes down, 1% fall to outlet. Provide solid outlet pipe at suitable locations.
3) Drain installation and outlet connection should be observed by the geotechnical consultant.

SCST, LLC	**TYPICAL RETAINING WALL BACKDRAIN DETAILS** VID, E Reservoir Replacement and Pump Station Vista, California	Date: May, 2019 By: NNW/DTC Job No.: 180433P4-1R3	Figure: 9

APPENDIX I
FIELD INVESTIGATION

Our field investigation consisted of drilling five borings to depths between about 9½ and 25½ feet below the existing ground surface using a truck-mounted drill rig equipped with a hollow-stem auger and hand tools. An SCST geologist logged the borings and collected samples of the materials encountered in the borings for laboratory testing. SCST tested selected samples from the borings to evaluate pertinent soil classification and engineering properties to assist in developing geotechnical conclusions and recommendations. Figure 2 presents the approximate locations of the borings. The field investigation was performed under the observation of an SCST geologist who also logged the borings and obtained samples of the materials encountered in the borings.

The soils are classified in accordance with the Unified Soil Classification System as illustrated on Figure I-1. Logs of the borings are presented on Figures I-2 through I-7.

SUBSURFACE EXPLORATION LEGEND

UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION		GROUP SYMBOL	TYPICAL NAMES
I. COARSE GRAINED, more than 50% of material is larger than No. 200 sieve size.			
GRAVELS More than half of coarse fraction is larger than No. 4 sieve size but smaller than 3".	CLEAN GRAVELS	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, poorly graded gravel-sand-silt mixtures.
		GC	Clayey gravels, poorly graded gravel-sand, clay mixtures.
SANDS More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS	SW	Well graded sand, gravelly sands, little or no fines.
		SP	Poorly graded sands, gravelly sands, little or no fines.
		SM	Silty sands, poorly graded sand and silty mixtures.
		SC	Clayey sands, poorly graded sand and clay mixtures.
II. FINE GRAINED, 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, sandy silt or clayey-silt-sand mixtures 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 or low plasticity.
	SILTS AND CLAYS (Liquid Limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity.
III. HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils.

SAMPLE SYMBOLS

- ☒ - Bulk Sample
- CAL - Modified California Sampler
- CK - Undisturbed Chunk sample
- MS - Maximum Size of Particle
- ST - Shelby Tube
- SPT - Standard Penetration Test sampler

GROUNDWATER SYMBOLS

- ▽ - Water level at time of excavation or as indicated
- - Water seepage at time of excavation or as indicated

LABORATORY TEST SYMBOLS

- AL - Atterberg Limits
- CON - Consolidation
- COR - Corrosivity Tests (Resistivity, pH, Chloride, Sulfate)
- DS - Direct Shear
- EI - Expansion Index
- MAX - Maximum Density
- RV - R-Value
- SA - Sieve Analysis

SCST, LLC	VID, E Reservoir Replacement and Pump Station Vista, California			
	By:	EMW	Date:	May, 2019
	Job Number:	180433P4-1R3	Figure:	I-1

LOG OF BORING B-1

Date Drilled: 1/24/2019
Equipment: CME-95 with 8-inch Hollow Stem Auger
Elevation (ft): Approximately 756

Logged by: EMW
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVEN	SAMPLES BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
0–2	SC	**FILL (Qf):** CLAYEY SAND, loose, brown, moist, fine to coarse grained, trace gravel, trace cobble.		X					
2–3.5	SC	**COLLUVIUM (Qcol):** CLAYEY SAND, medium dense, red brown, moist, fine to coarse grained.		X					SA AL COR
3.5		**GABBRO (Kgb):** Light brown, moist, weathered, moderately hard.							
5		Light brown to reddish brown, moderately hard to hard.	CAL		50/3"	>50			
10		Light brown.	CAL		50/2"	>50	7.4	107.9	
15		Light brown and gray.	CAL		50/1"	>50			
19			CAL		50/2"	>50			
19		**BORING TERMINATED AT 19 FEET**							

Depth scale (ft): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

SCST, LLC

VID, E Reservoir Replacement and Pump Station Vista, California			
By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	I-2

LOG OF BORING B-2

Date Drilled: 1/24/2019 | Logged by: EMW
Equipment: CME-95 with 8-inch Hollow Stem Auger | Reviewed by: AKN
Elevation (ft): Approximately 771 | Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
0–1.5	SC	**FILL (Qf):** CLAYEY SAND, loose, brown, moist, fine to coarse grained, trace gravel.		X (0.5–3)					
1.5–3.5	SC	**COLLUVIUM (Qcol):** CLAYEY SAND with GRAVEL, medium dense, reddish brown to red, moist, fine to coarse grained, trace cobble.							SA
3.5		**GABBRO (Kgb):** Light reddish brown to reddish brown, moist, weathered, moderately soft to moderately hard.							
5		Light orange brown and gray, moderately hard to hard.	CAL		50/5"	>50			
10		Light brown to grayish brown.	CAL		50/2"	>50	11.2	92.6	
15			CAL		50/3"	>50			
19			CAL		50/2"	>50			
19		**BORING TERMINATED AT 19 FEET**							

SCST, LLC

VID, E Reservoir Replacement and Pump Station
Vista, California

By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	I-3

LOG OF BORING B-3

Date Drilled: 1/24/2019
Equipment: CME-95 with 8-inch Hollow Stem Auger
Elevation (ft): Approximately 754

Logged by: EMW
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		3 inches of Asphalt Concrete							
1	SM	**FILL (Qf):** SILTY SAND, loose to medium dense, brown, moist, fine to medium grained, trace gravel.		X					
2									
3		Trace cobble.							
4		Some gravel, some cobble.							
5		Variably colored (light brown to brown, to red), dense, fine to coarse grained.							
6			CAL		35	30			
7									
8									
9									
10									
11			CAL		21	18			
	SC	CLAYEY SAND, medium dense, brown, moist, fine to coarse grained, trace gravel.							
12									
13	CL	**COLLUVIUM (Qcol):** SANDY CLAY, medium stiff, red brown, moist, fine to medium grained SAND, trace gravel.							
14									
15									
16			CAL		8	7			
17									
18									
19		**GABBRO (Kgb):** Light brown, moist, weathered, hard.	CAL		50/2"	>50			DS
20									

BORING CONTINUED ON I-5

SCST, LLC

VID, E Reservoir Replacement and Pump Station
Vista, California

By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	I-4

LOG OF BORING B-3 (continued)

Date Drilled: 1/24/2019
Equipment: CME-95 with 8-inch Hollow Stem Auger
Elevation (ft): Approximately 754

Logged by: EMW
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES DRIVEN	SAMPLES BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		GABBRO (Kgb): Light brown, moist, weathered, hard.							
21									
22									
23									
24									
25		Light brown to gray.	CAL		50/5"	>50			
26		**BORING TERMINATED AT 25½ FEET**							
27									
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									

LOG OF BORING B-4

Date Drilled: 1/24/2019
Equipment: CME-95 with 8-inch Hollow Stem Auger
Elevation (ft): Approximately 761

Logged by: EMW
Reviewed by: AKN
Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		2 to 3 inches of vegetation and associated top soil							
1	SC	**COLLUVIUM (Qcol):** CLAYEY SAND, loose to medium dense, reddish brown to red, moist, fine to medium grained, trace gravel, trace cobble.		X					
2									
3		**GABBRO (Kgb):** Light brown to gray, moist, weathered, moderately soft to moderately hard.							
4									
5		Hard.	CAL		50/2"	>50	5.0	106.6	DS
6									
7									
8									
9		Very hard.	SPT		50/1"	>50			
10		**AUGER REFUSAL AT 9½ FEET ON GABBRO ROCK**							
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									

SCST, LLC

VID, E Reservoir Replacement and Pump Station, Vista, California	
By: EMW	Date: May, 2019
Job Number: 180433P4-1R3	Figure: I-6

LOG OF BORING B-5

Date Drilled: 1/24/2019 | Logged by: EMW
Equipment: CME-95 with 8-inch Hollow Stem Auger | Reviewed by: AKN
Elevation (ft): Approximately 736 | Depth to Groundwater (ft): Not Encountered

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVEN	SAMPLES: BULK	DRIVING RESISTANCE (blows/ft of drive)	N_{60}	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)	LABORATORY TESTS
		3 inches of Asphalt Concrete							
1	CL	**FILL (Qf):** SANDY CLAY, soft, red brown, moist, fine to coarse grained SAND.		X					SA AL EI COR
2		Fine to coarse grained, trace gravel, trace cobble.		X					
3	SC	**COLLUVIUM (Qcol):** CLAYEY SAND, medium dense, light reddish brown, moist, fine to medium grained.							
4									
5		**GABBRO (Kgb):** Light brown to gray, moist, weathered, moderately hard.	CAL		50/4"	>50			
6									
7									
8									
9									
10			CAL		50/2"	>50			
11									
12									
13									
14									
15			CAL		50/1"	>50			
16		**AUGER REFUSAL AT 15½ FEET ON GABBRO ROCK**							
17									
18									
19									
20									

SCST, LLC

VID, E Reservoir Replacement and Pump Station
Vista, California

By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	I-7

APPENDIX II
LABORATORY TESTING

Laboratory tests were performed to provide geotechnical parameters for engineering analyses. The following tests were performed:

- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System.

- **PARTICLE-SIZE DISTRIBUTION:** The particle-size distribution was evaluated on selected soil samples in accordance with ASTM D422.

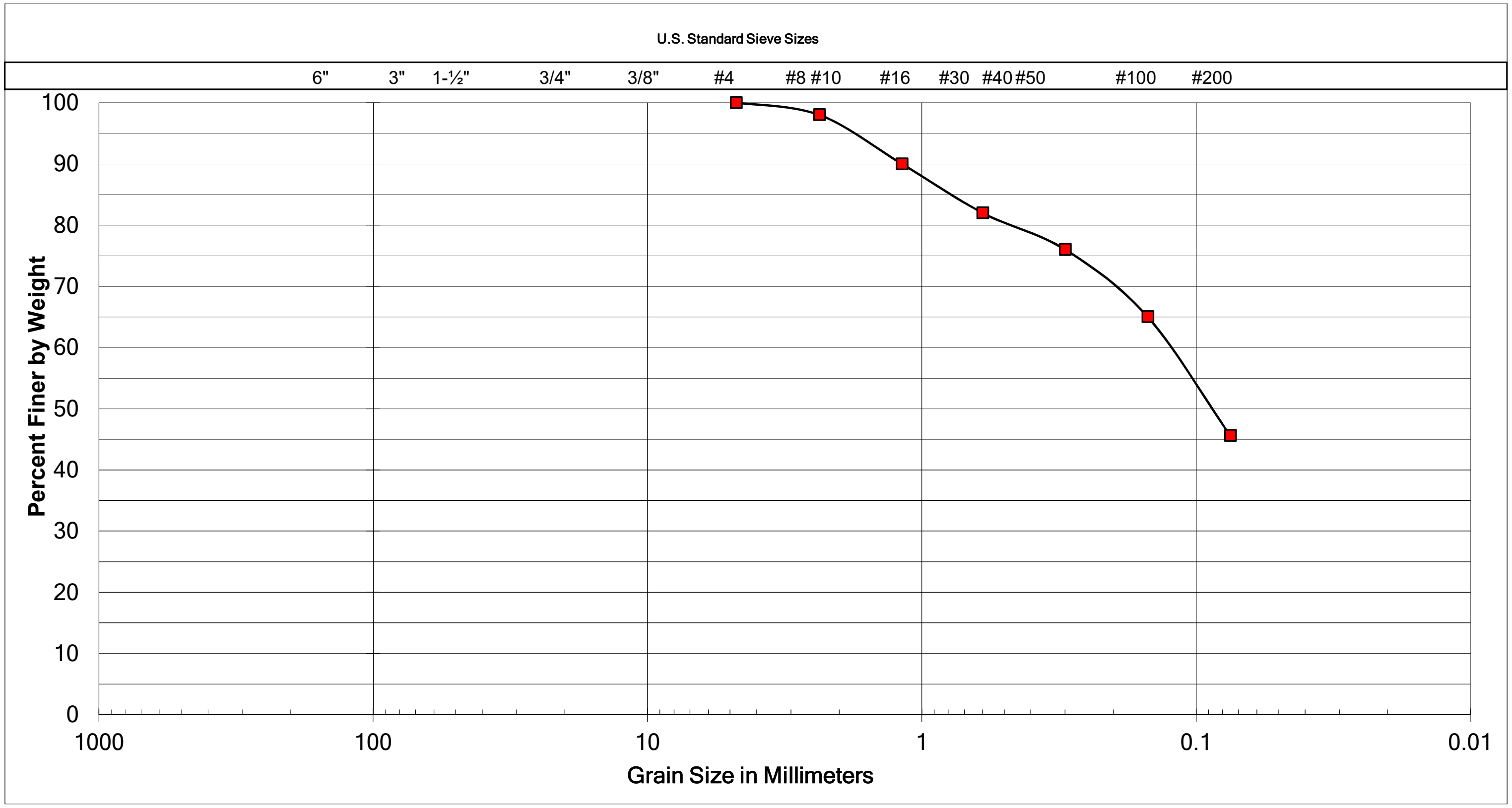
- **R-VALUE:** R-value tests were performed on selected soil samples in accordance with California Test Method 301.

- **EXPANSION INDEX:** The expansion index was evaluated on selected soil samples in accordance with ASTM D4829.

- **CORROSIVITY**: Corrosivity tests were performed on selected soil samples. The pH and minimum resistivity were evaluated in accordance with California Test 643. The total chloride ion content was evaluated in accordance with California Test 422. The soluble sulfate content was evaluated in accordance with California Test 417.

- **DIRECT SHEAR:** The direct shear was evaluated on selected soil samples in accordance with ASTM D3080.

Soil samples not tested are now stored in our laboratory for future reference and analysis, if needed. Unless notified to the contrary, samples will be disposed of 30 days from the date of this report.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

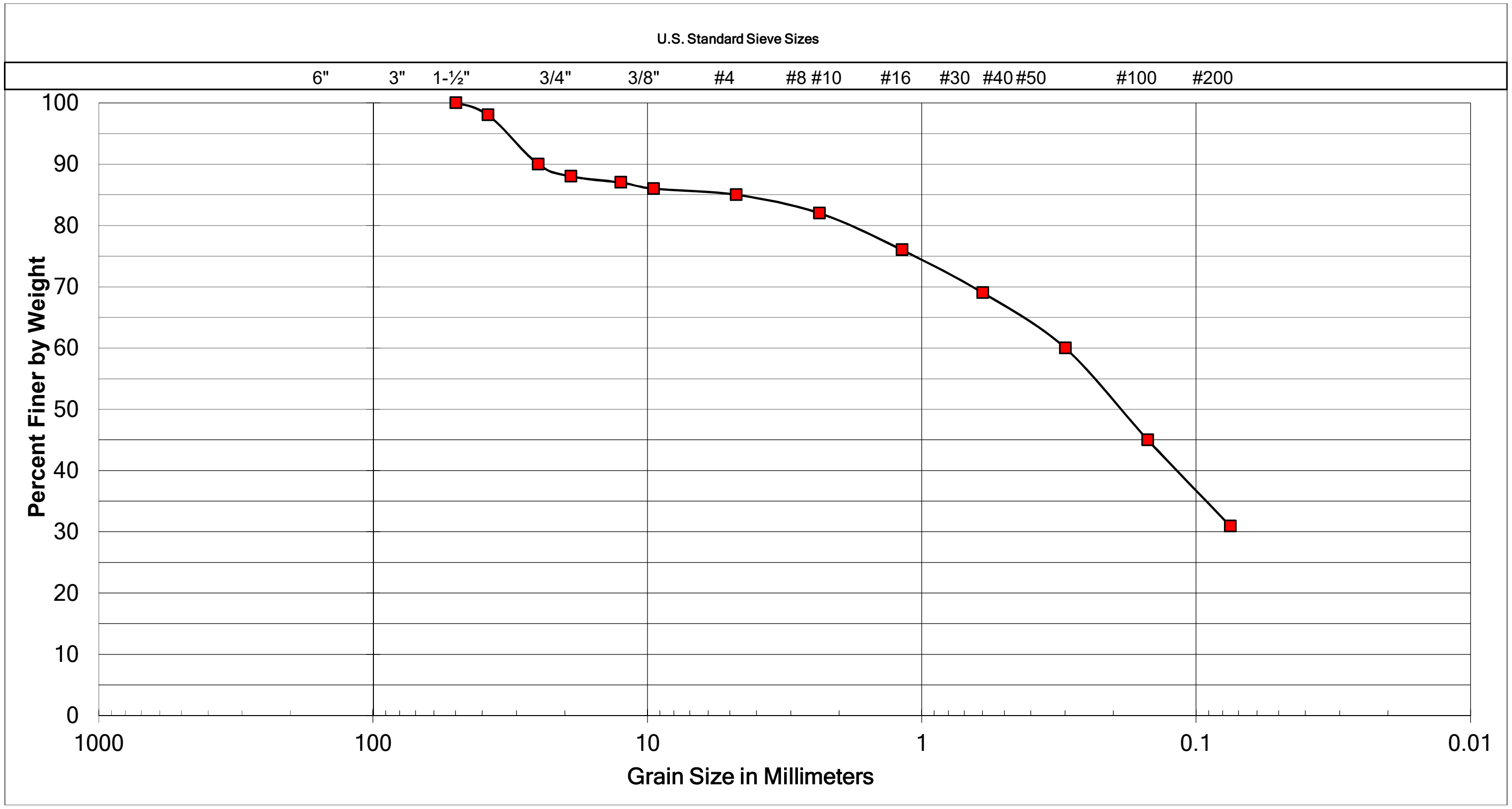
SAMPLE LOCATION
B-1 at 2 to 3 feet
SAMPLE NUMBER
37333

UNIFIED SOIL CLASSIFICATION:	SC
DESCRIPTION	CLAYEY SAND

ATTERBERG LIMITS	
LIQUID LIMIT	30
PLASTIC LIMIT	19
PLASTICITY INDEX	11

SCST, LLC

VID, E Reservoir Replacement and Pump Station Vista, California			
By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	II-1



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

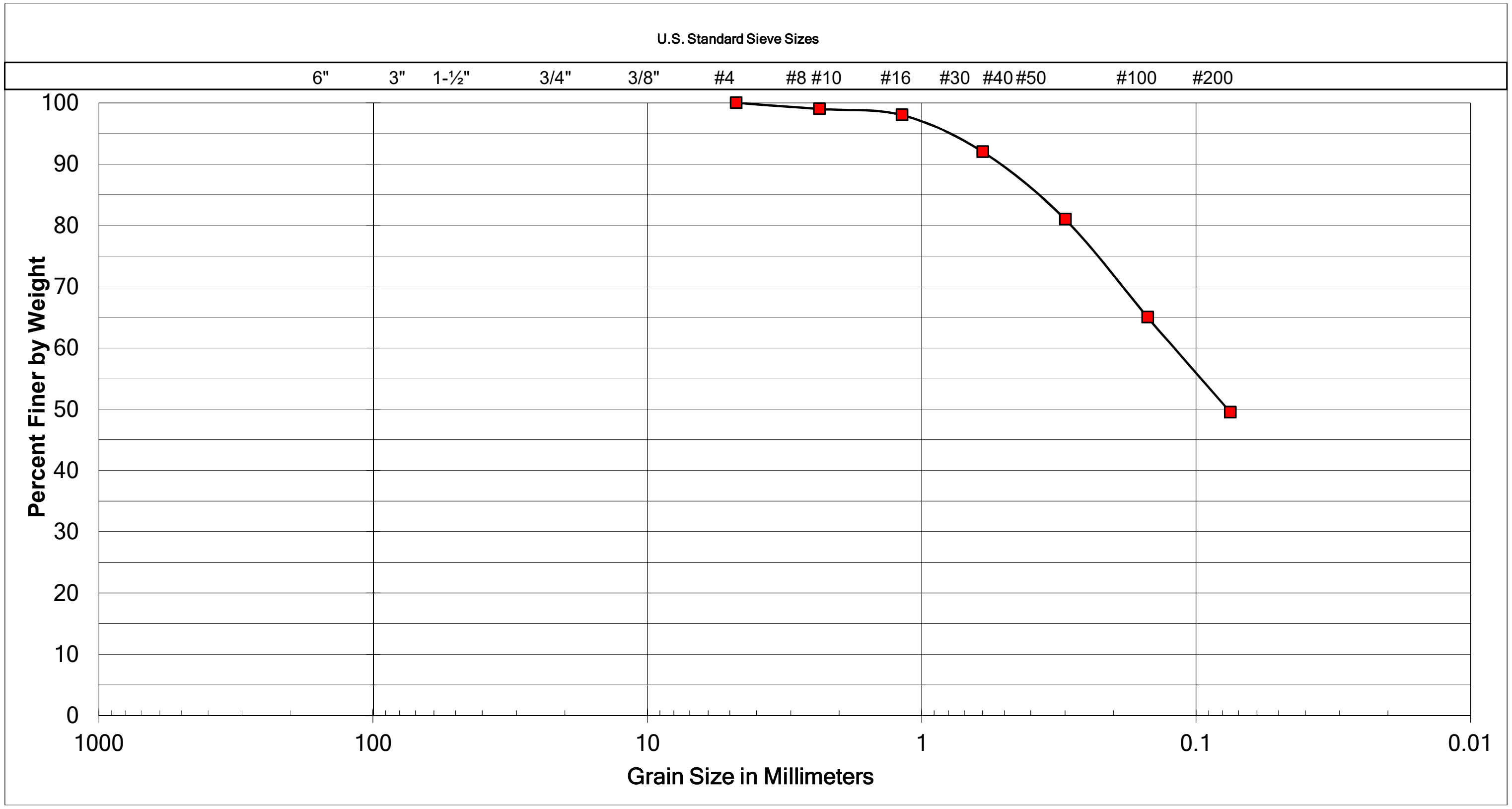
SAMPLE LOCATION
B-2 at ½ to 3 feet
SAMPLE NUMBER
37336

UNIFIED SOIL CLASSIFICATION:	SC
DESCRIPTION	CLAYEY SAND with GRAVEL

ATTERBERG LIMITS	
LIQUID LIMIT	--
PLASTIC LIMIT	--
PLASTICITY INDEX	--

SCST, LLC

VID, E Reservoir Replacement and Pump Station Vista, California			
By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	II-2



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
B-5 at 0 to 3 feet
SAMPLE NUMBER
37342

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION	SANDY CLAY

ATTERBERG LIMITS	
LIQUID LIMIT	32
PLASTIC LIMIT	21
PLASTICITY INDEX	11

SCST, LLC

VID, E Reservoir Replacement and Pump Station Vista, California			
By:	EMW	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	II-3

EXPANSION INDEX

ASTM D2489

SAMPLE	DESCRIPTION	EI
B-5 at 0 to 3 feet	CLAYEY SAND	33

Classification of Expansive Soil [1]

EXPANSIVE INDEX	POTENTIAL EXPANSION
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

1. ASTM - D4829

R-VALUE

CALIFORNIA TEST 301

SAMPLE	DESCRIPTION	R-VALUE
B-1 at 0 to 2 feet	CLAYEY SAND	33

RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

pH & Resistivity (Cal 643, ASTM G51)

Soluble Chlorides (Cal 422)

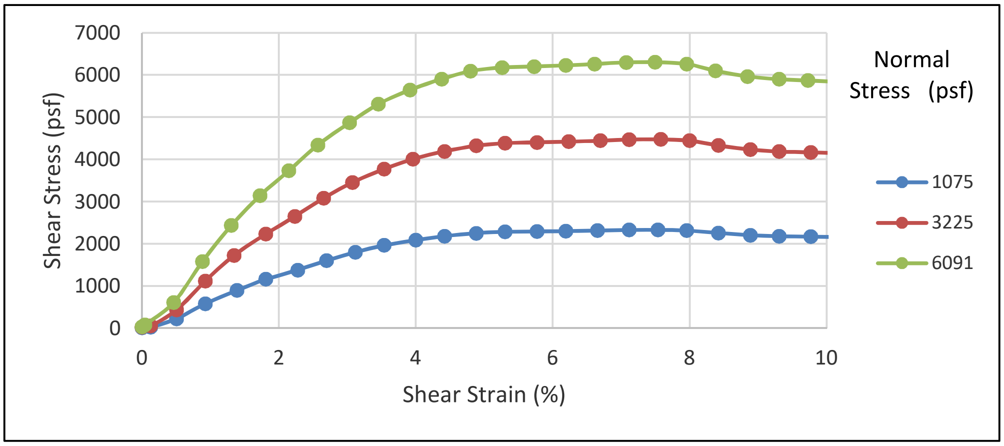
Soluble Sulfate (Cal 417)

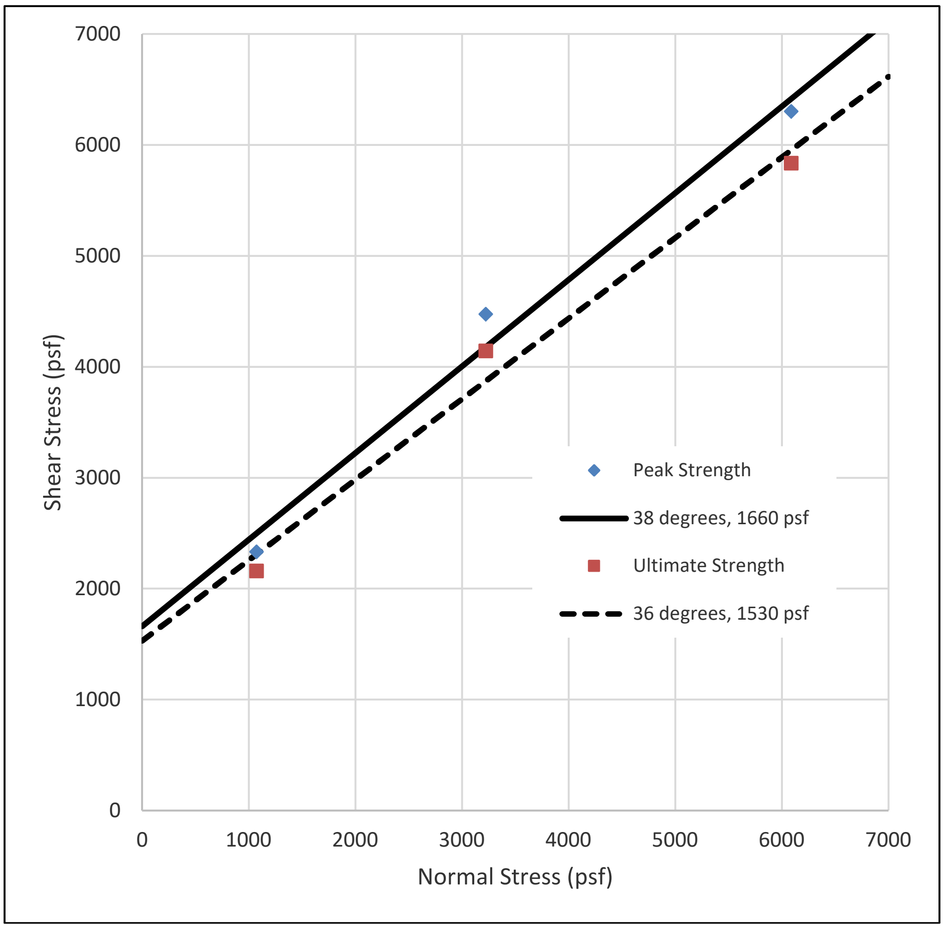
SAMPLE	RESISTIVITY (Ω-cm)	pH	CHLORIDE (%)	SULFATE (%)
B-1 at 2 to 3 feet	2130	7.02	0.002	0.004
B-5 at 0 to 3 feet	1160	7.25	0.006	0.016

WATER-SOLUBLE SULFATE (SO_4^{2-}) EXPOSURE

Modified from ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1

Water-soluble sulfate (SO_4^{2-}) in soil, percent by weight	Exposure Severity	Exposure Class	Cement Type (ASTM C150)	Max. w/cm	Min. f_c'
$SO_4^{2-} < 0.10$	Not applicable	S0	No type restriction	N/A	2,500
$0.10 \le SO_4^{2-} < 0.20$	Moderate	S1	II	0.50	4,000
$0.20 \le SO_4^{2-} < 2.00$	Severe	S2	V	0.45	4,500
$SO_4^{2-} > 2.00$	Very Severe	S3	V plus pozzolan or slag cement	0.45	4,500





SAMPLE ID: B-3 at 19½ to 20 feet

GABBRO (CLAYEY SAND)

NOTES: In Situ

Strain Rate: 0.003 in/min

Sample was consolidated and drained

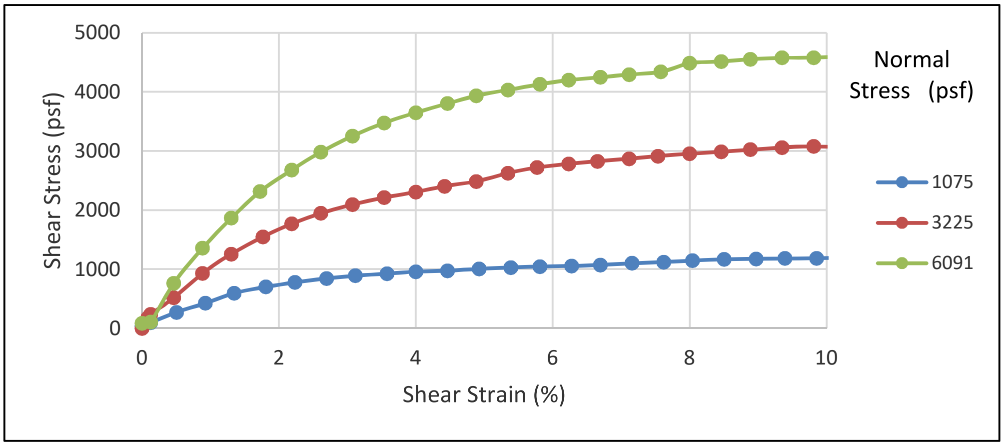
	Peak	Ultimate
Φ	38 °	36 °
c	1660 psf	1530 psf

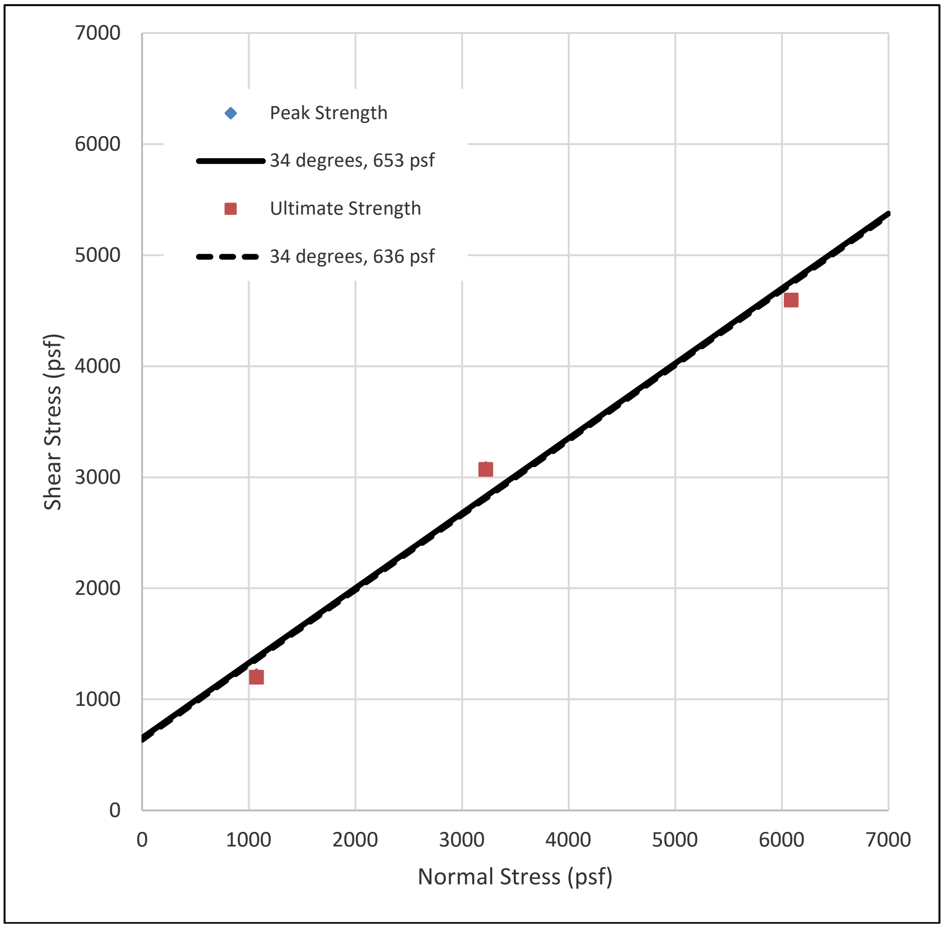
	Initial	Final
γ_d	117.5 pcf	117.5 pcf
w_c	17.0 %	18.2 %
Saturation	100 %	100 %

SCST, LLC

VID, E Reservoir Replacement and Pump Station
Vista, California

By:	DRB	Date:	May, 2019
Job Number:	180433P4-1R3	Figure:	II-5





SAMPLE ID: B-4 at 5½ to 6 feet

GABBRO (CLAYEY SAND)

NOTES: In Situ

Strain Rate: 0.003 in/min

Sample was consolidated and drained

	Peak	Ultimate
Φ	34 °	34 °
c	653 psf	636 psf

	Initial	Final
γ_d	120.8 pcf	120.8 pcf
w_c	3.5 %	15.3 %
Saturation	24 %	100 %

SCST, LLC	VID, E Reservoir Replacement and Pump Station Vista, California			
	By:	DRB	Date:	May, 2019
	Job Number:	180433P4-1R3	Figure:	II-6

APPENDIX III
SEISMIC REFRACTION SURVEY

SEISMIC REFRACTION SURVEY
EDGEHILL ROAD
VISTA, CALIFORNIA

PREPARED FOR:
SCST, LLC
6280 Riverdale Street
San Diego, CA 92120

PREPARED BY:
Southwest Geophysics, LLC
6280 Riverdale Street Suite 200
San Diego, CA 92120

February 26, 2019
Project No. 119042b

SOUTHWEST GEOPHYSICS LLC
YOUR SUBSURFACE SOLUTION

February 26, 2019
Project No. 119042b

Mr. Andrew K. Neuhaus, C.E.G.
SCST, LLC
6280 Riverdale Street
San Diego, CA 92120

Subject: Seismic Refraction Survey
Edgehill Road
Vista, California

Dear Mr. Neuhaus:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Edgehill Road project located in Vista, California. Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. Our field services were conducted on February 1, 2019. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, LLC

Eric R. Carlson
Project Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/29/20
STATE OF CALIFORNIA

HV/ERC/hv

Distribution: Addressee (electronic)

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Edgehill Road project located in Vista, California (Figure 1). Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. Our field services were conducted on February 1, 2019. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of four seismic P-wave refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE DESCRIPTION

The project site is located just northeast of the intersection of Edgehill Road and Audrey Place in Vista, California. Vegetation in the area consists of scattered brush and small trees, and cacti. Several remnant granitic rock boulders were observed in the study area. Figures 2 and 3 depict the general site conditions in the areas of the seismic traverses.

4. SURVEY METHODOLOGY AND ANALYSIS

As previously indicated, the primary purpose of our services was to characterize the subsurface conditions at preselected locations through the collection of seismic P-wave refraction data. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves (compression waves) generated at the surface are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse. The refraction method requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by buried boulders, fractures, dikes, etc. can result in the misinterpretation of the subsurface conditions.

Four seismic P-wave traverses, SL-1 through SL-4, were conducted at the site. The location of the profiles, which were generally selected by your office, and the line lengths are depicted on Figure 2. Multiple shot points (signal generator locations) were conducted at the ends, midpoint, and intermediate points along the lines. The P-wave signal (shot) was generated using a 20-pound hammer and an aluminum plate.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2011) as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability.

The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in narrow trenching operations, should be anticipated.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

5. RESULTS

Figures 4a through 4d present the results from the P-wave refraction survey. Based on the velocity models generated from our P-wave analysis, it appears the study areas are underlain by low velocity materials (e.g., colluvium and topsoil) in the near surface and granitic rock with varying degrees of weathering at depth. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the study areas. In addition, pockets or zones of relatively “hard” rock appear to be present in the subsurface.

Based on the P-wave refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. Furthermore, blasting may be required depending on the excavation depth, location, equipment used, and desired rate of production. A contractor with excavation experience in similar conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

6. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

7. SELECTED REFERENCES

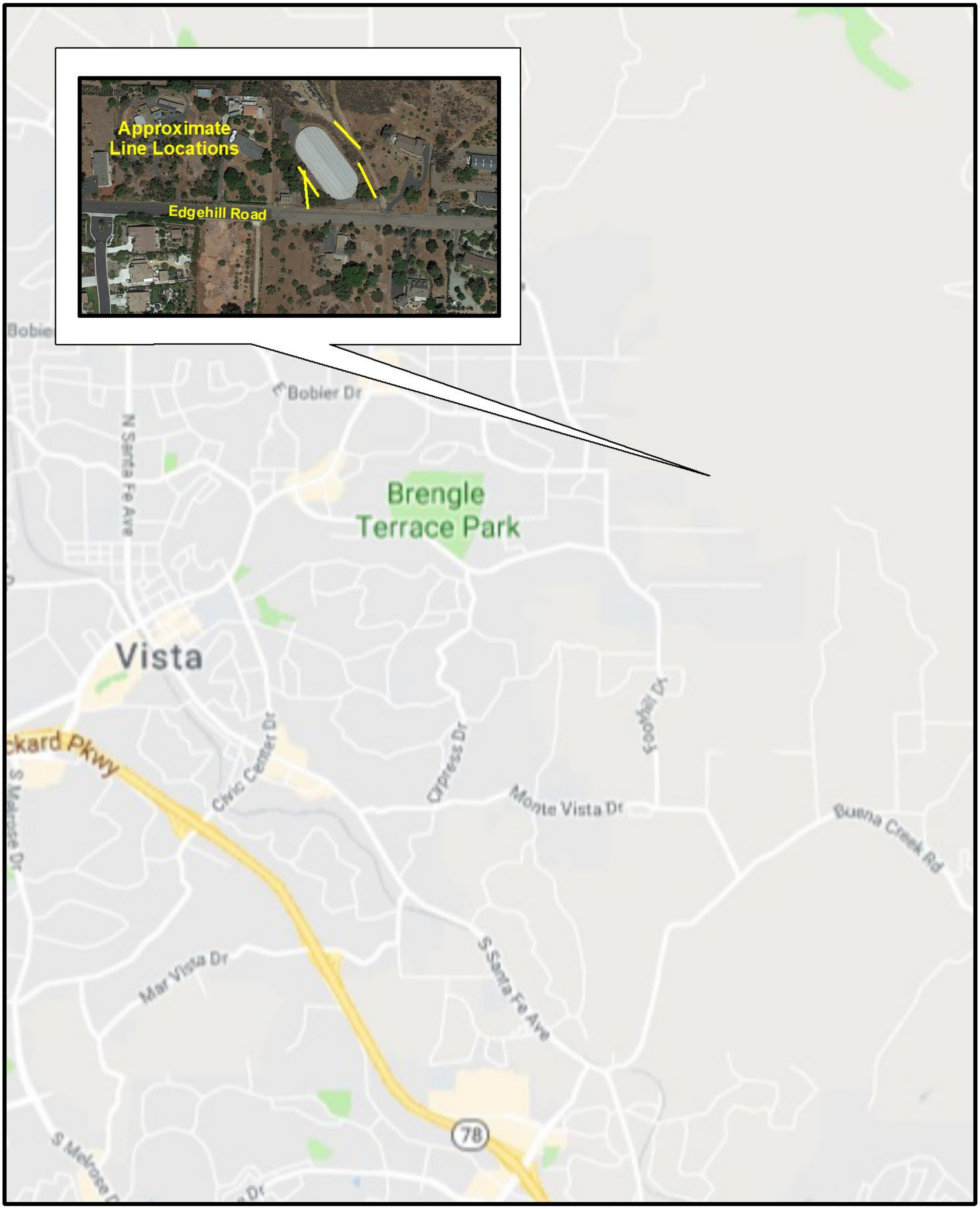
Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

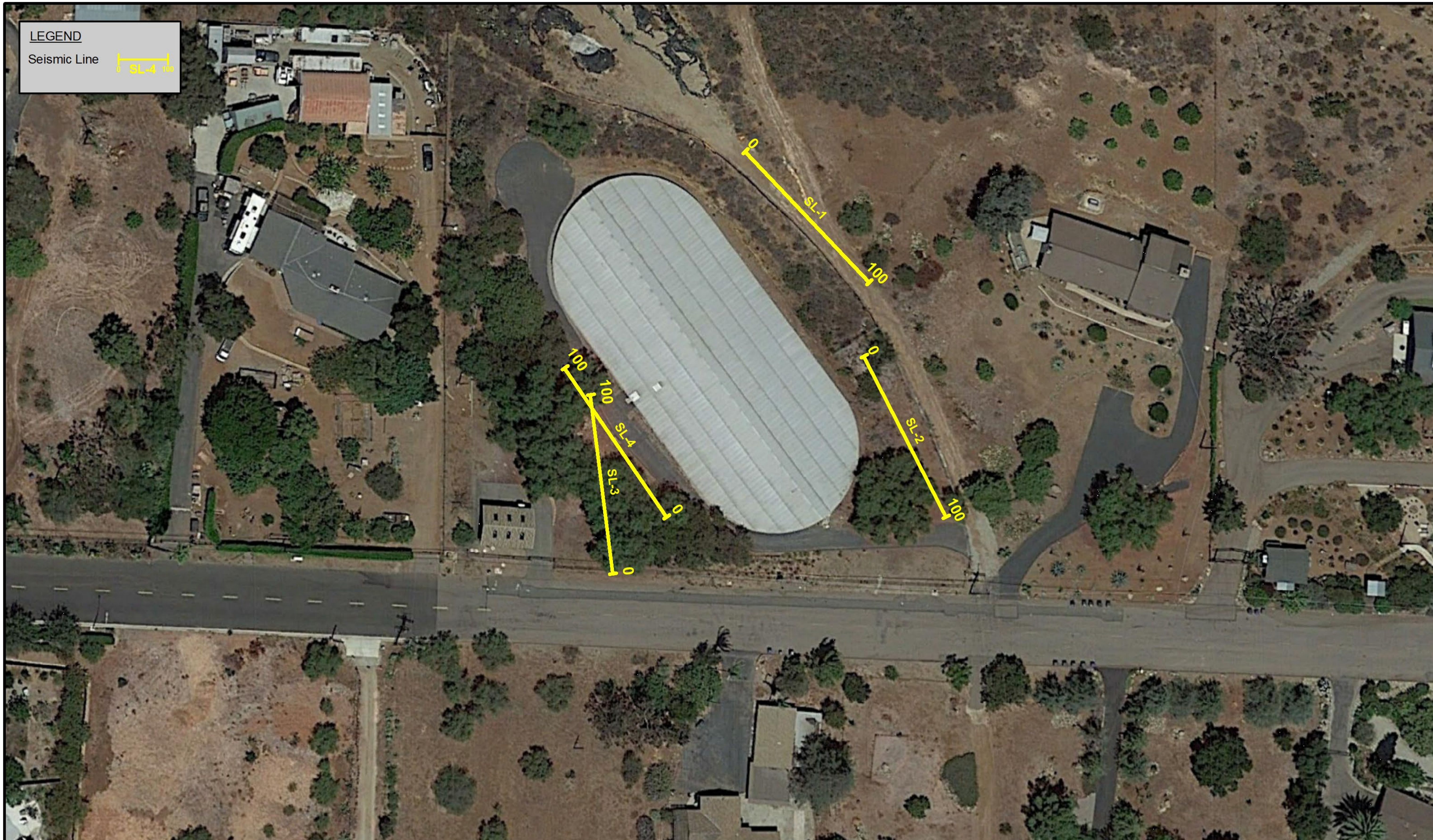
Optim, Inc., 2008, SeisOpt Pro, V-5.0.

Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.

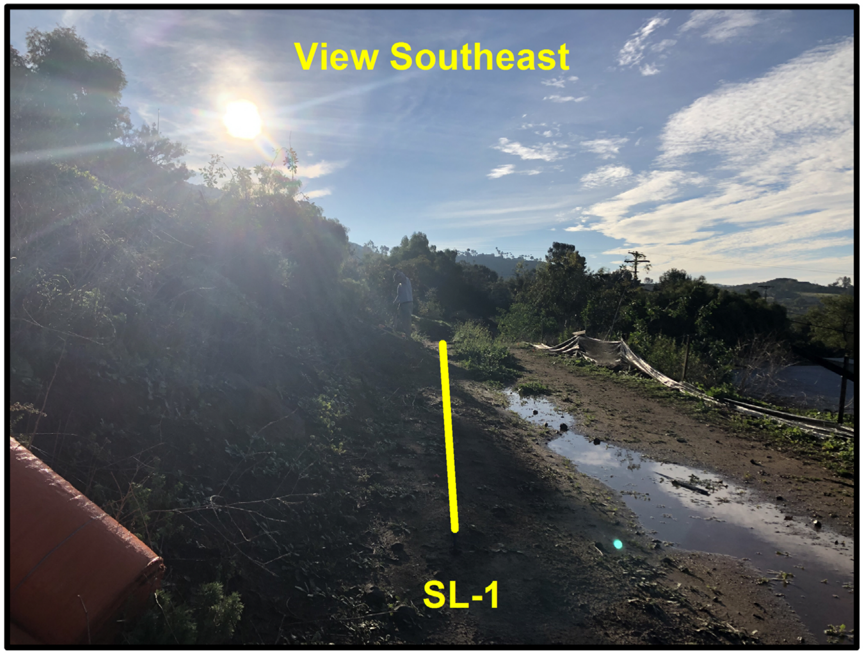


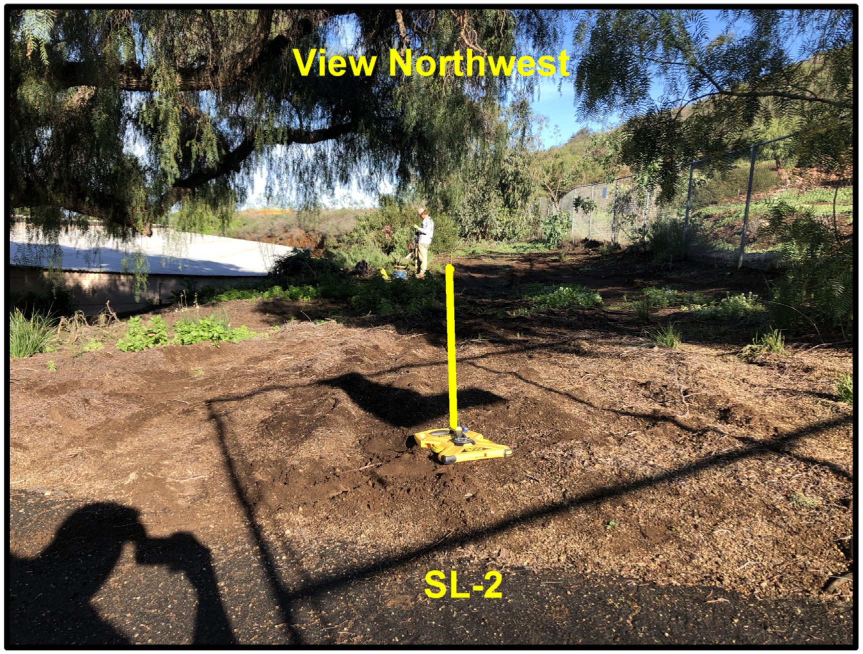
SITE LOCATION MAP	N	Edgehill Road Vista, California	SouthWest Geophysics LLC Figure 1
		Project No.: 119042b / Date: 02/19	

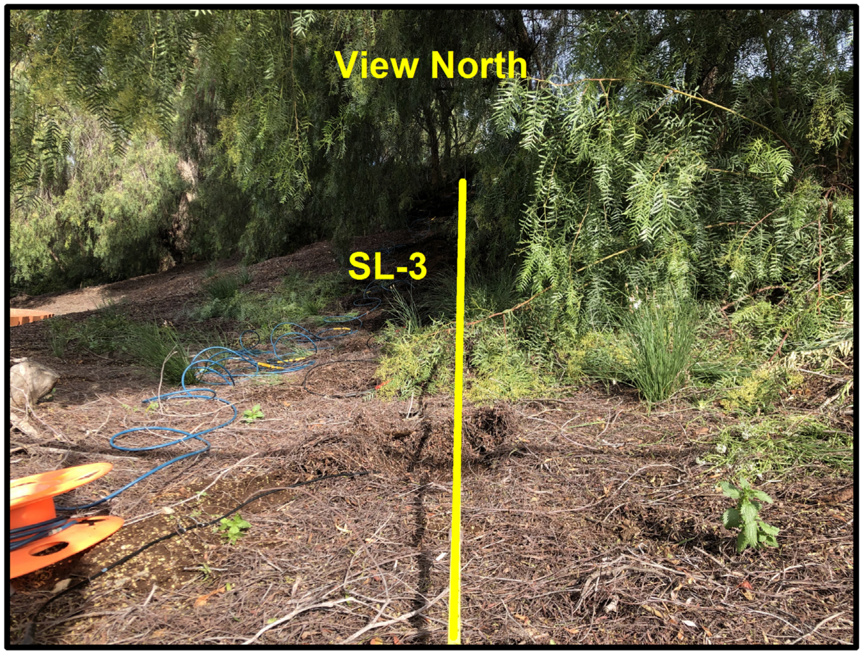


LINE LOCATION MAP	N	Edgehill Road Vista, California Project No.: 19042b Date: 02/19	SOUTHWEST GEOPHYSICS INC. Figure 2

0 50 100
approximate scale in feet

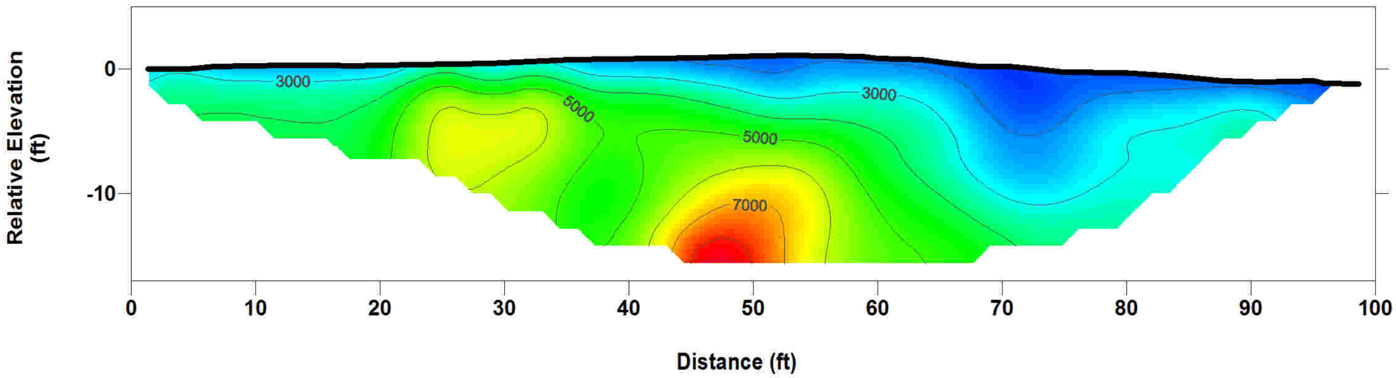


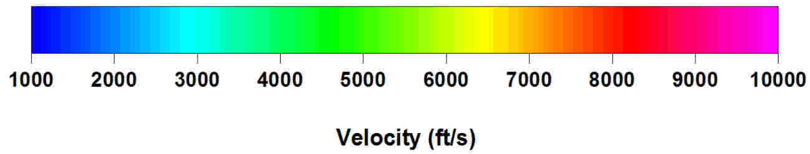




SITE PHOTOGRAPHS	Edgehill Road Vista, California	SOUTHWEST GEOPHYSICS LLC Figure 3
	Project No.: 119042b / Date: 02/19	

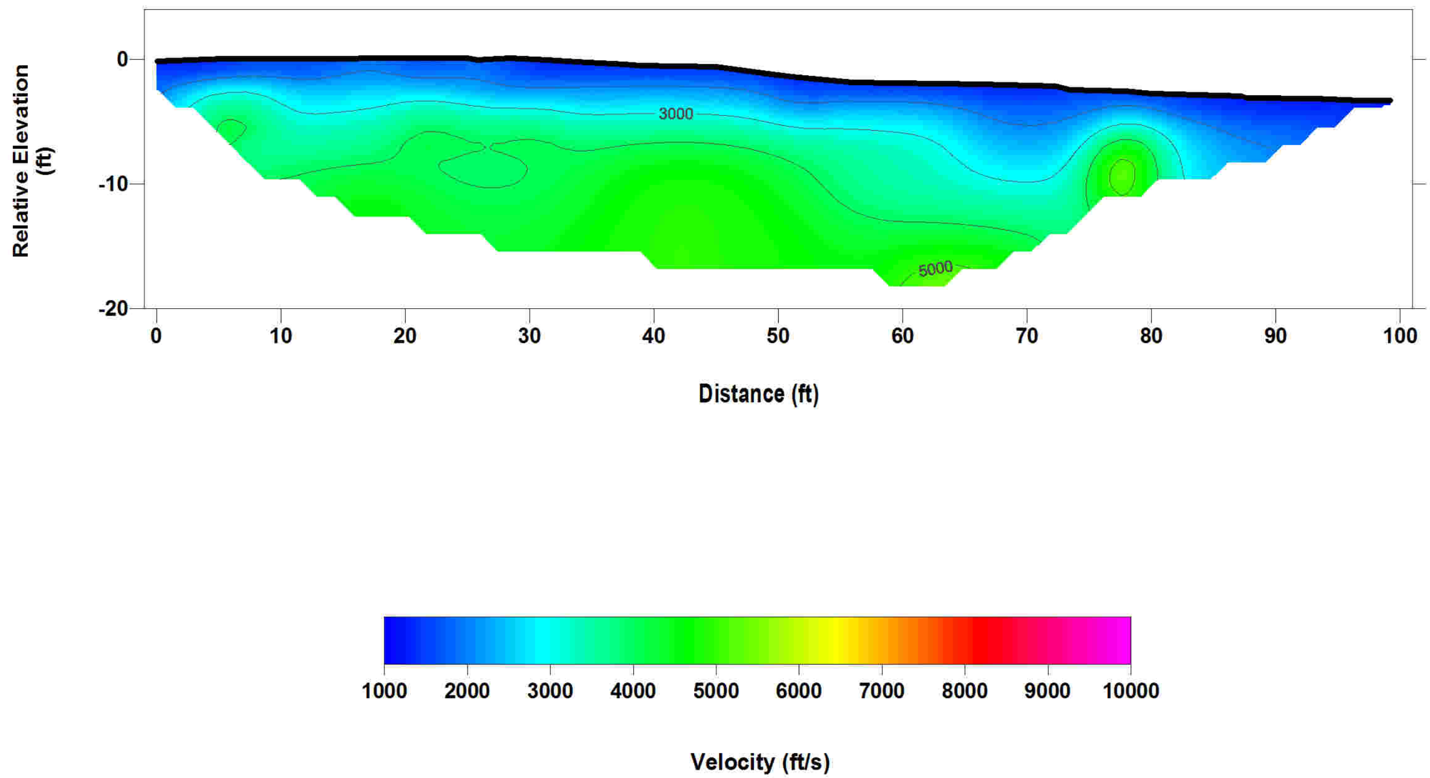
TOMOGRAPHY MODEL





P-WAVE PROFILE SL-1	Edgehill Road Vista, California	SOUTHWEST GEOPHYSICS	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 119042b	Date: 02/19	Figure 4a

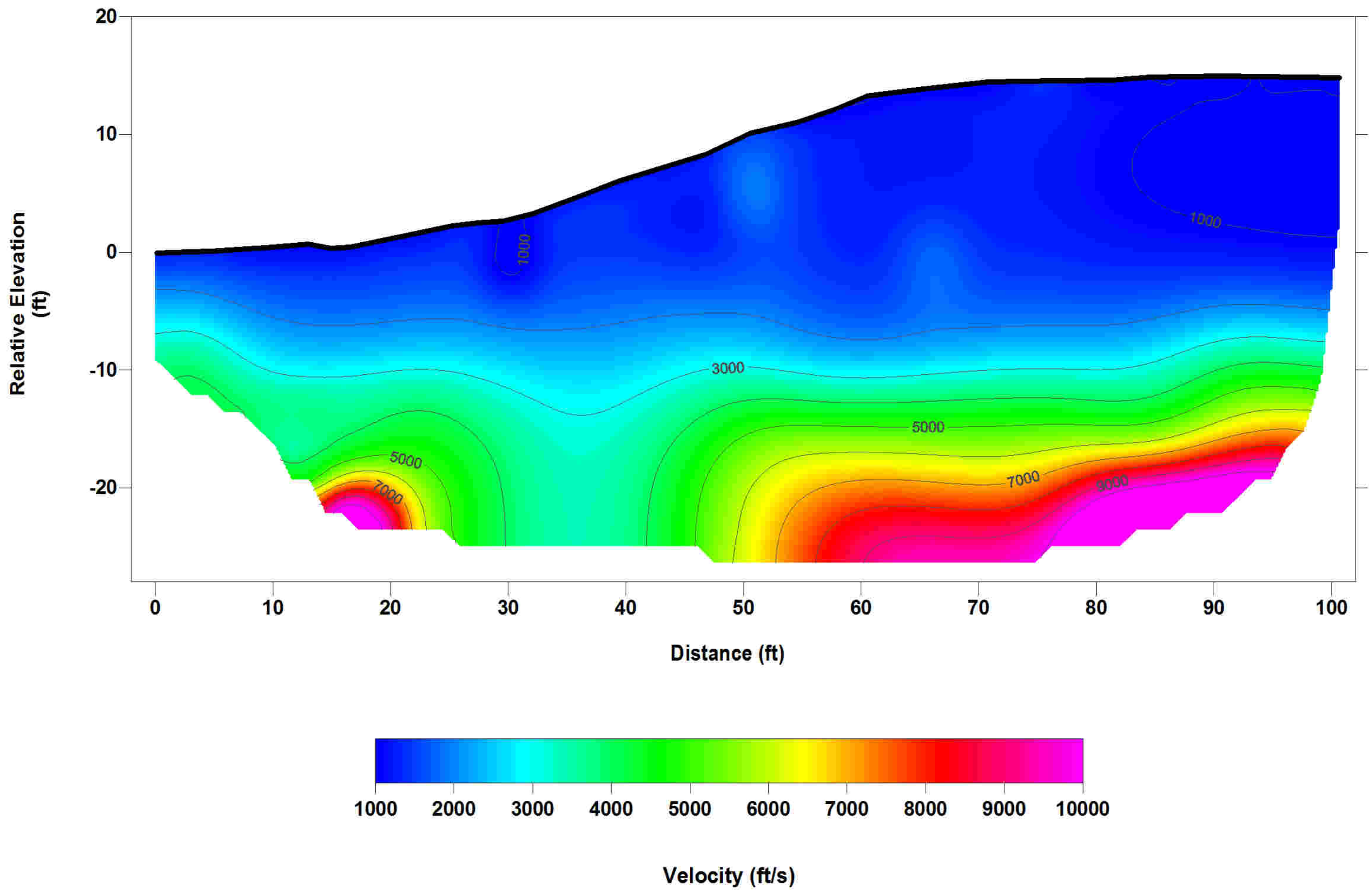
TOMOGRAPHY MODEL



P-WAVE PROFILE SL-2	Edgehill Road Vista, California	SOUTHWEST GEOPHYSICS Figure 4b
	Project No.: 119042b — Date: 02/19	

Note: Contour Interval = 1,000 feet per second

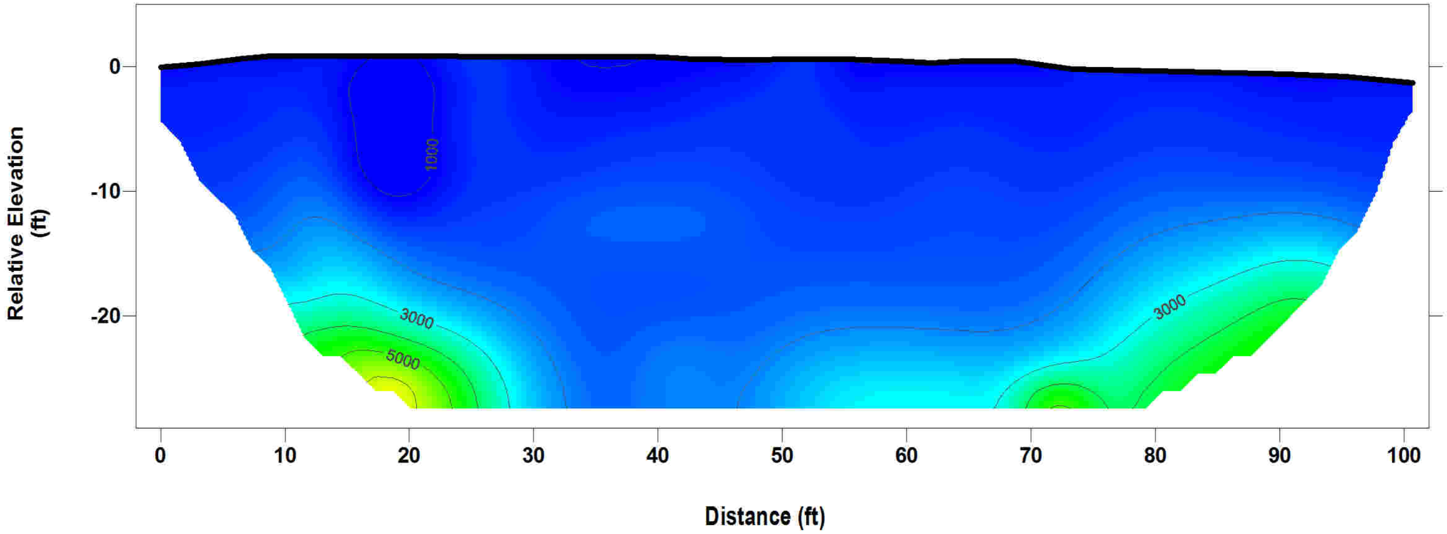
TOMOGRAPHY MODEL

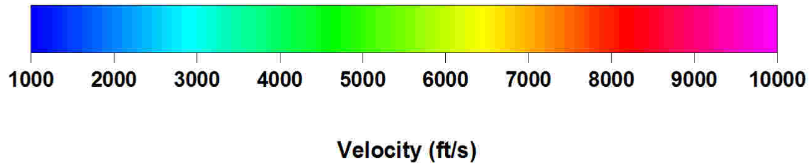


P-WAVE PROFILE SL-3	Edgehill Road Vista, California	SOUTHWEST GEOPHYSICS Figure 4c
	Project No.: 119042b — Date: 02/19	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL





P-WAVE PROFILE SL-4	Edgehill Road Vista, California	SOUTHWEST GEOPHYSICS Figure 4d	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 119042b / Date: 02/19		