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GEOTECHNICAL INVESTIGATION
PROPOSED SINGLE FAMILY RESIDENCE
13074 POLVERA AVENUE
SAN DIEGO, CALIFORNIA

PREPARED FOR:

MR. ALEX VARDY
13074 POLVERA AVENUE
SAN DIEGO, CALIFORNIA 92260

PREPARED BY:

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April 26, 2018

SCST Project No. 180169N
Report No. 1

Mr. Alex Vardy
13074 Polvera Avenue
San Diego, California 92260

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED SINGLE FAMILY RESIDENCE
13074 POLVERA AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Vardy:

SCST, Inc. is pleased to present our report describing the geotechnical investigation performed for the subject project. We conducted the geotechnical investigation in general conformance with the scope of work presented in our agreement dated January 22, 2018. Based on the results of our investigation, we consider the planned construction feasible from a geotechnical standpoint provided the recommendations of this report are followed. If you have questions, please call us at (619) 280-4321.

Respectfully submitted,
SCST, INC.

REGISTERED PROFESSIONAL GEOLOGIST
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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

SECTION **PAGE**

ATTACHMENTS

FIGURES

APPENDICES

1. INTRODUCTION

This report presents the results of the geotechnical investigation SCST, Inc. (SCST) performed for the single family residential development located at 13074 Polvera Avenue in the city of San Diego, California. We understand that the project will consist of the design and construction of a single-family residence, with two detached garages, and associated hardscape areas. Figure 1 presents a site vicinity map. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project.

2. SCOPE OF WORK

2.1 FIELD INVESTIGATION

We explored the subsurface conditions by excavating 5 test pits within the proposed improvement area up to 10 feet below the existing ground surface using a backhoe. Additionally, a subcontracted Registered Geophysicist performed five seismic refraction traverses in planned cut areas to evaluate rippability characteristics of the bedrock underlying the site. Figure 2 presents the approximate locations of the test pits and seismic refraction traverses. An SCST geologist logged the test pits and collected samples of the materials encountered for laboratory testing. Appendix I presents logs of the test pits. Soils are classified according to the Unified Soil Classification System illustrated on Figure I-1. Appendix II presents the results of the seismic refraction survey.

2.2 LABORATORY TESTING

Selected samples obtained from the test pits were tested to evaluate pertinent soil classification and engineering properties and enable development of geotechnical conclusions and recommendations. The laboratory tests consisted of grain size distribution, expansion index and corrosivity testing. Appendix III presents the results of the laboratory tests and brief explanations of the test procedures.

2.3 ANALYSIS AND REPORT PREPARATION

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
- Temporary excavations and shoring
- Excavation characteristics
- Foundation support, potential foundation settlement, resistance to lateral loads and lateral earth pressures for retaining wall design
- Estimated foundation settlements
- Support for concrete slabs-on-grade
- Lateral pressures for the design of retaining walls
- Soil corrosivity

3. SITE DESCRIPTION

The site consists of vacant lot located at 13074 Polvera Avenue located in the community of Rancho Bernardo in San Diego, California. The site is located on a slope that overall descends towards the north-northeast. The site is bordered by residential properties on the east, south, and west. Topographically, the site has an elevation difference of about 30 feet across the property. The site currently supports native vegetation.

4. PROPOSED IMPROVEMENTS

The proposed improvements will consist of a single-family residence with two detached garages, a paved driveway, and associated hardscape areas. We anticipate that site grading will consist of cuts and fills up to about 5 feet thick. Any planned fill slopes will be constructed at 2:1 (horizontal:vertical). Cut slopes will generally be constructed at 1½:1 (horizontal:vertical).

5. GEOLOGY AND SUBSURFACE CONDITIONS

The materials encountered in the test pits consist of residual soil, weathered and unweathered granitic rock. Descriptions of the materials are presented below. Figure 4 presents the site-specific geology.

Residual Soil – The residual soil consists of loose silty sand. The soil extends to depths up to 10 inches below the existing ground surface where explored. Deeper residual soil may be encountered in areas not explored.

Granitic Rock – Granitic rock underlies the residual soil. Weathered and decomposed rock extends to depths between 5 to 10 feet below the existing ground surface where explored. Thicker layers of weathered granitic rock may be encountered in areas not explored. The unweathered granitic rock underlies the weathered material and consists of hard, unrippable rock.

Groundwater - Groundwater was not encountered in the test pits. However, 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 faults are the Rose Canyon Fault (Oceanside section) fault located 18.1 miles (29.2 kilometers) west of the site and the Elsinore (Julian) fault located 20.4 miles (32.9 kilometers) east 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 CBC are presented below:

Site Coordinates: Latitude 33.05021°
Longitude -117.04984°
Site Class: B
Site Coefficients, F_a = 1.000
F_v = 1.000
Mapped Spectral Response Acceleration at Short Periods, S_s = 0.973g
Mapped Spectral Response Acceleration at 1-Second Period, S_1 = 0.376g
Design Spectral Acceleration at Short Period, S_{DS} = 0.649g
Design Spectral Acceleration at 1-Second Period, S_{D1} = 0.250g
Site Peak Ground Acceleration, PGA_M = 0.361g

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 relatively dense nature of the underlying bedrock materials beneath the site, the potential for liquefaction and dynamic settlement to occur is considered negligible.

6.4 TSUNAMIS, SEICHES AND FLOODING

The site is not located within a mapped area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); therefore, damage due to tsunamis is considered low. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any lakes or 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 (County of San Diego, 2012).

6.5 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.

6.6 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 negligible.

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 eolian 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 dense materials underlying the site are not considered susceptible to hydro-consolidation.

7. CONCLUSIONS

The main geotechnical considerations affecting the planned development are the presence of potentially compressible residual soils and difficult excavations in the granitic rock. Remedial grading will need to be performed to reduce the potential for distress to the planned improvements. Remedial grading recommendations are provided in Section 8.1.2 of this report. Non-rippable granitic rock exists at the site that will require rock-breaking equipment and/or blasting and special handling. Contract documents should specify that the contractor mobilize equipment capable of excavating and breaking the granitic rock.

8. RECOMMENDATIONS

8.1 SITE PREPARATION AND GRADING

8.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, topsoil, 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.

8.1.2 Remedial Grading

To reduce the potential for settlement, the residual soils and any existing fills should be excavated in their entirety beneath settlement sensitive improvements and new fills. Horizontally, remedial excavations should extend at least 5 feet outside the planned perimeter foundations and at least 2 feet outside the planned hardscape/pavements. An SCST representative should observe conditions exposed in the bottom of the excavation to determine if additional excavation is required.

8.1.3 Cut/Fill Transitions

The planned structure should not be underlain by cut/fill transitions or transitions from shallow fill to deep fill. Where such transitions are encountered at finished pad elevation, the granitic rock should be over-excavated and replaced with compacted fill to provide a relatively uniform thickness of compacted fill beneath the entire structure and reduce the potential for differential settlement. The over-excavation depth should be at least 3 feet below the planned finished pad elevation, at least 2 feet below the deepest planned footing bottom elevation, or to a depth of H/2, whichever is deeper, where H is the greatest depth of fill beneath the structure. Horizontally, the over-excavation should extend at least 5 feet outside the planned footing perimeter or up to existing improvements, whichever is less. Where practical, the bottom of excavations should be sloped toward the fill portion of the site and away from its center. An SCST representative should observe the conditions exposed in the bottom of excavations to determine if additional excavation is required.

8.1.4 Keyways and Benching

Keyways should be established at the base of fill slopes. The entire keyway should expose competent granitic rock. The keyway should be at least 15 feet wide at the bottom and sloped towards the slope at an inclination of about 2%. The keyway may need to be wider to accommodate compaction equipment. Final keyway recommendations will depend on the final grading plans. Fill should be benched into sloping ground inclined steeper than 5:1 (horizontal:vertical). The benches should expose competent material as evaluated by the geotechnical consultant.

8.1.5 Excavation Characteristics

It is anticipated that excavations in residual soil and intensely weathered granitic rock in the upper 5 to 10 feet can generally be achieved with conventional earthwork equipment in good working order. However, non-rippable granitic rock exists on site, and difficult excavation should be anticipated. Rock breakers, carbide/diamond tipped equipment and/or blasting may be required to excavate less weathered rock. Localized “floaters” or large boulder inclusions may also be encountered. Excavations in rock may generate oversized material that will require extra effort to crush or haul offsite. Contract documents should specify that the contractor mobilize equipment capable of excavating and compacting the granitic rock.

8.1.6 Temporary Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations should be laid back no steeper than 1:1 (horizontal:vertical) in fill or residual soil or ½:1 (horizontal:vertical) in granitic rock. The faces of temporary slopes should be inspected daily by the contractor’s Competent Person before personnel are allowed to enter the excavation. Any 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 excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. SCST should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces. Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (horizontal:vertical) downward from the outside bottom edge of existing structures or improvements will require shoring. A shoring system consisting of soldier piles and lagging can be used.

8.1.7 Temporary Shoring

For design of cantilevered shoring, an active soil pressure equal to a fluid weighing 35 pcf can be used for level retained ground or 55 pcf for 2:1 (horizontal:vertical) sloping ground. The surcharge loads on shoring from traffic and construction equipment adjacent to the 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 350 psf per foot of embedment over twice the pile diameter up to a maximum of 5,000 psf can be used. Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be required 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.

8.1.8 Temporary Dewatering

Groundwater seepage may occur locally and should be anticipated in excavations. Dewatering can be accomplished by sloping the excavation bottom to a sump and pumping from the sump. A layer of gravel about 6 inches thick placed in the bottom of the excavation will facilitate groundwater flow and can be used as a working platform.

8.1.9 Compacted Fill

Excavated material, except for vegetation, debris and rocks greater than 6 inches can be used as compacted fill. Due to the varying thicknesses of residual soil and bedrock, concrete slabs, retaining wall footings, and building foundations should be underlain by at least 2 feet of compacted fill material with an expansion index of 50 or less. We expect that most of the onsite materials will meet the expansion index criteria and can be used as compacted fill. Fill should be moisture conditioned to near optimum moisture content and compacted to at least 90% 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 determined in accordance with ASTM D 1557. Utility trench backfill beneath structures, pavements and hardscape should be compacted to at least 90% relative compaction. The top 12 inches of subgrade beneath pavements should be compacted to at least 95%.

8.1.10 Utility Over-Excavation

Utility alignments underlain by hard rock may be over-excavated and replaced with compacted fill to facilitate trench excavations. The depth of over-excavation should be based on the anticipated trench excavations.

8.1.11 Oversized Material

Excavations may generate oversized material. Oversized material is defined as rocks greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in compacted fill, used as landscape material, or disposed offsite.

8.1.12 Bulking and Shrinking Factors

For earthwork estimating purposes, excavated residual soils placed as fill is estimated to shrink about 5% to 10% in volume. Excavated granitic rock is estimated to bulk about 5% to 15%.

8.1.13 Imported Soil

Imported soil should consist of predominately granular soil free of organic matter and rocks greater than 6 inches. Imported soil should have an expansion index of 20 or less and should be inspected and, if appropriate, tested by SCST prior to transport to the site.

8.1.14 Slopes

Permanent fill slopes should be constructed no steeper than 2:1 (horizontal:vertical). Permanent cut slopes in competent granitic rock should be constructed no steeper than 1½:1 (horizontal:vertical). Faces of fill slopes should be compacted either by rolling with a sheep-foot roller or other suitable compaction equipment, or by overfilling and cutting back to design grade. An engineering geologist should observe all cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. All slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the tops of slopes. Slopes should be protected or planted with vegetation that will reduce the potential for erosion.

8.1.15 Surface Drainage

Final surface grades around improvements should be designed to collect and direct surface water away from the improvement and toward appropriate drainage facilities. The ground around the improvement should be graded so that surface water flows rapidly away from the improvement without ponding. In general, we recommend that the ground adjacent to the improvement 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 improvement. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed improvements. 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.

8.1.16 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.

8.2 FOUNDATIONS

8.2.1 Conventional Footings

The planned buildings can be supported on shallow spread footings with bottom levels on compacted fill. Footings should extend at least 24 inches below lowest adjacent finished grade. Continuous footings should be at least 12 inches wide. Isolated or retaining wall footings should be at least 24 inches wide. An allowable bearing capacity of 2,500 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 a maximum of 5,000 psf. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. 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. An allowable coefficient of friction of 0.35 can be used. Passive pressure can be computed using an allowable lateral pressure of 350 psf per foot of depth below the ground surface for level ground conditions. Reductions for sloping ground should be made. The passive pressure can be increased by ⅓ when considering the total of all 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.

8.2.2 Settlement Characteristics

Total foundation settlements are estimated to be less than 1 inch. Differential settlements between adjacent columns and across continuous footings are estimated to be less than ¾ inch over a distance of 40 feet. Settlements should be completed shortly after structural loads are applied.

8.2.3 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.

8.2.4 Foundation Excavation Observations

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

8.3 SLABS-ON-GRADE

8.3.1 Interior Slab-on-Grade

The project structural engineer should design the interior concrete slabs-on-grade floor. However, we recommend that building slabs be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. Moisture protection should be installed beneath slabs where moisture sensitive floor coverings will be used. The project architect should review the tolerable moisture transmission rate of the proposed floor covering and specify an appropriate moisture protection system. Typically, a plastic vapor barrier is used. Minimum 10-mil plastic is recommended. The plastic should comply with ASTM E1745. The vapor barrier installation should comply with ASTM E1643. Current construction practice typically includes placement of a 2-inch thick sand cushion between the bottom of the concrete slab and the vapor barrier. This cushion can provide some protection to the vapor barrier during construction and may assist in reducing the potential for edge curling in the slab during curing. However, the sand layer also provides a source of moisture to the underside of the slab that can increase the time required to reduce vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The slab can be placed directly on the vapor barrier.

8.3.2 Exterior Slabs-on-Grade

Exterior slabs should be at least 4 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 project architect 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.

8.4 CONVENTIONAL RETAINING WALLS

8.4.1 Foundations

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

8.4.2 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 35 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 55 pcf. These values assume a granular and drained backfill condition. 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.

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 architect should provide waterproofing specifications and details. Figure 7 shows typical conventional retaining wall backdrain details.

8.4.3 Seismic Earth Pressure

If required, 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 determining the seismic stability of the wall.

8.4.4 Backfill

Wall backfill should consist of granular, free-draining material. Expansive or clayey soil should not be used. Additionally, backfill 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 onsite 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.

8.5 MECHANICALLY STABILIZED EARTH RETAINING WALLS

The following soil parameters can be used for design of mechanically stabilized earth (MSE) retaining walls.

MSE Wall Design Parameters

Soil Parameter	Reinforced Soil	Retained Soil	Foundation Soil
Internal Friction Angle	35°	32°	32°
Cohesion	0	0	0
Moist Unit Weight	125 pcf	125 pcf	125 pcf

The reinforced soil should consist of granular, free-draining material with an expansion index of 20 or less. The bottom of MSE walls should extend to such a depth that a total of 5 feet exists between the bottom of the wall and the face of the slope. Figure 8 presents a typical MSE retaining wall backdrain detail. MSE retaining walls may experience lateral movement over time. The wall engineer should review the configuration of proposed improvements adjacent to the wall and provide measures to help reduce the potential for distress to these improvements from lateral movement.

8.6 PIPELINES

8.6.1 Thrust Blocks

For level ground conditions, a passive earth pressure of 350 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.

8.6.2 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.

8.6.3 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 onsite 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.

8.7 PAVEMENT SECTION RECOMMENDATIONS

The pavement support characteristics of the soils encountered during our investigation are considered moderate to good. An R-value of 40 was assumed for design of preliminary pavement sections. The actual R-value of the subgrade soils should be determined after grading and final pavement sections be provided. Based on an R-value of 40 the following pavement structural sections are recommended.

Flexible Pavement Sections

Location	Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
Driveway	5.0	3	4

Portland Cement Concrete Pavement Sections

Location	Traffic Index	Full-Depth JPCP* (inches)	Aggregate Base (inches)
Driveway	5.0	6	4

***Jointed Plain Concrete Pavement**

The top 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content and compacted to at least 95% relative compaction. All 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. All materials and methods of construction should conform to good engineering practices and the minimum local standards.

Deepened curbs or vertical cutoff membranes consisting of 30 mil HDPE or PVC should be installed at the edges of pavements adjacent to storm water infiltration facilities to reduce the potential for water-related distress to pavements. The curb/membrane should extend below the aggregate base section.

8.8 SOIL CORROSIVITY

Representative samples of the onsite soils was tested to evaluate corrosion potential. The test results are presented in Appendix III. 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.

9. 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.

10. 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 test pit 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.

11. REFERENCES

American Concrete Institute (ACI) (2012), 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.

California Emergency Management Agency, California Geological Survey, University of Southern California (Cal EMA) (2009), Tsunami Inundation Map for Emergency Planning, June 1.

Caltrans (2010), Standard Specifications.

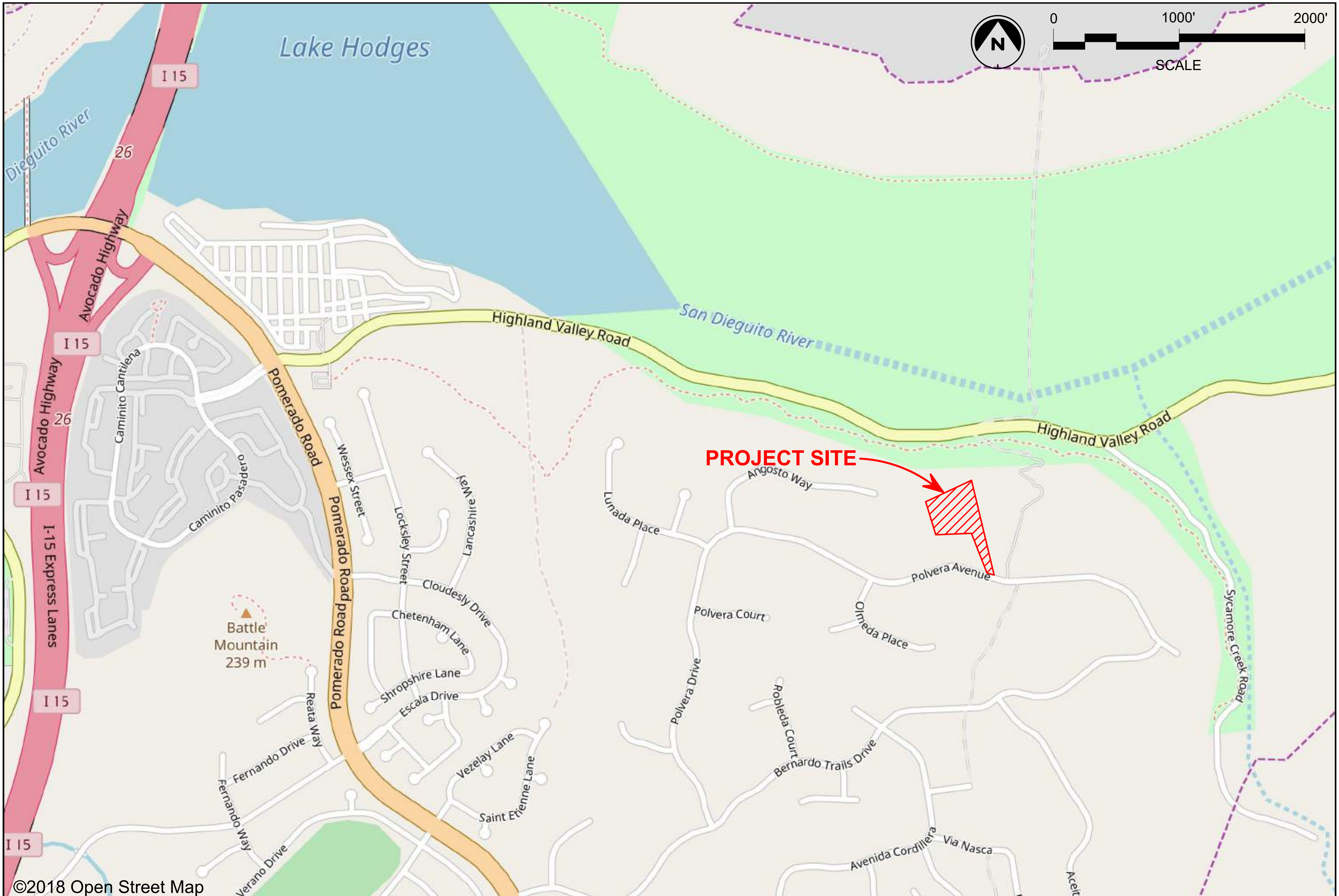
Caltrans (2014), Pervious Pavement Design Guidance, August.

County of San Diego (2012), SanGIS Interactive Map.

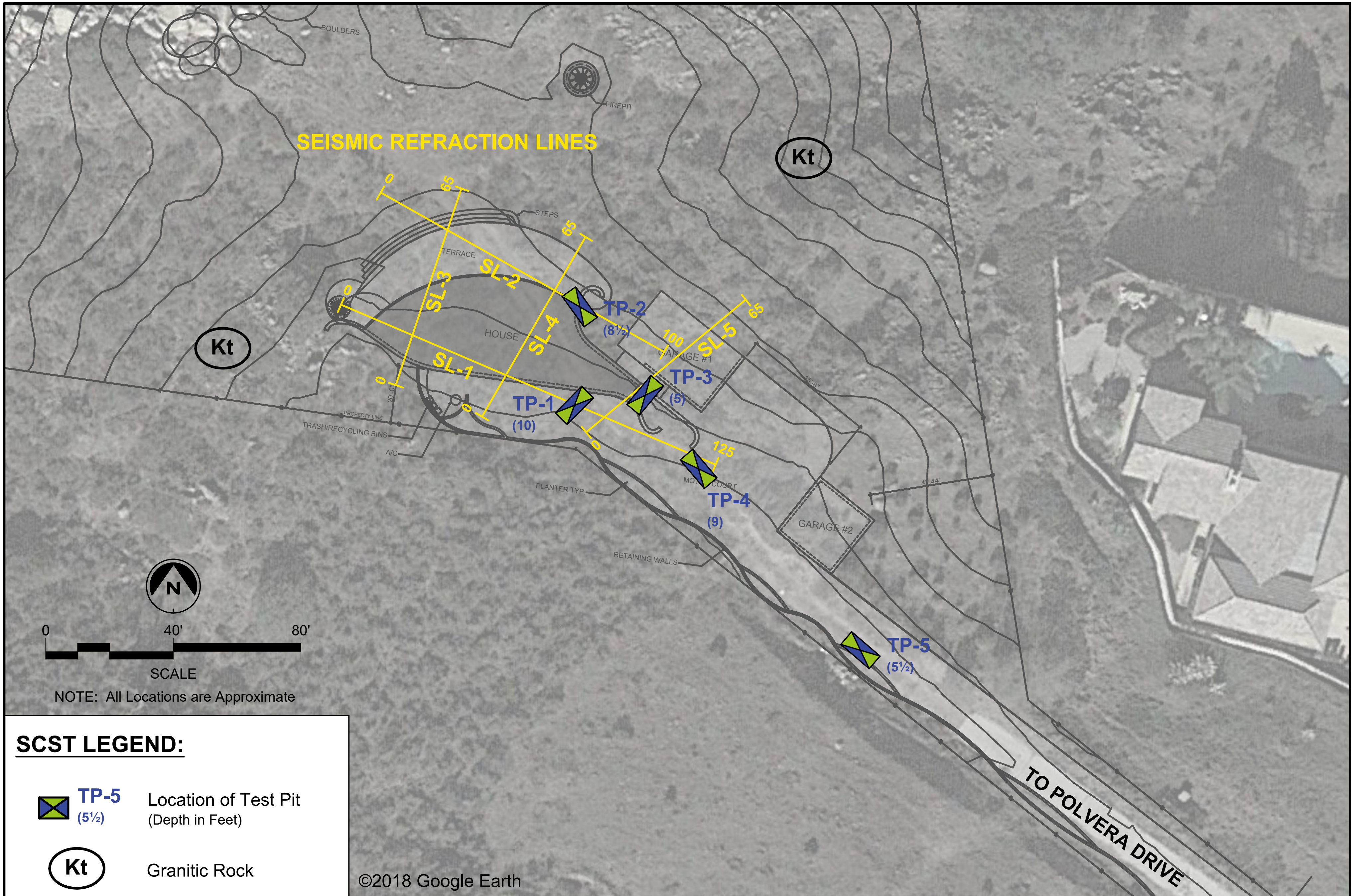
Gary Maxwell Testing (2006), Percolation Test Data, Rancho Nuevo, Via Tesoro, Alpine 91901, January 2.

Kennedy, M.P. and Tan, S.S. (2008), Geologic Map of the San Diego 30’ x 60’ Quadrangle, California, California Geological Survey.

Public Works Standards, Inc. (2015), The “Greenbook,” Standard Specifications for Public Works Construction, 2015 Edition.



SCST, Inc.	**SITE VICINITY MAP** 13074 Polvera Avenue San Diego, California	Date: April, 2018 By: PFL Job No.: 180169N-1	Figure: 1

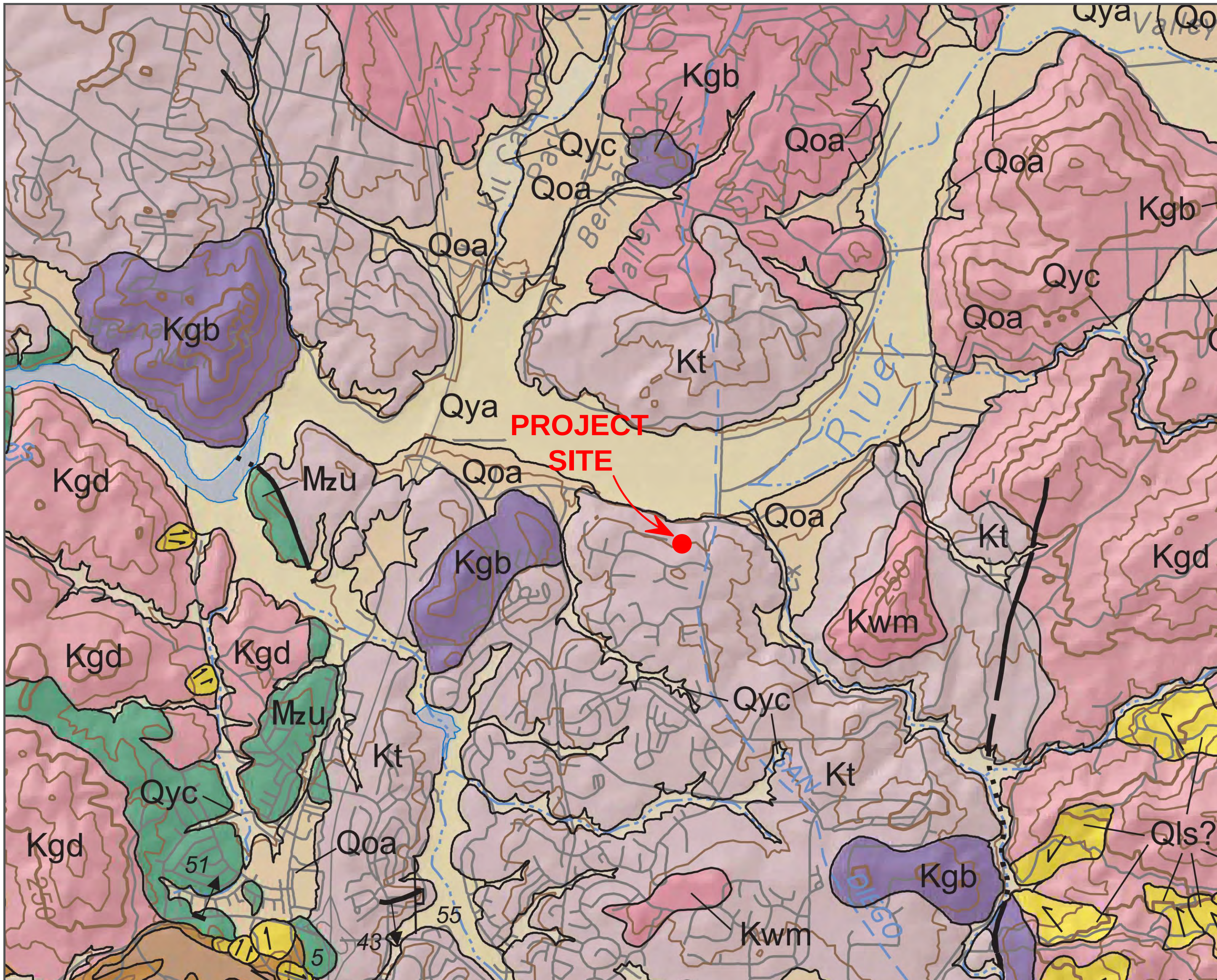


SCST, Inc.

SUBSURFACE EXPLORATION MAP
13074 Polvera Avenue
San Diego, California

Date: April, 2018
By: PFL/DTC
Job No.: 180169N-1

Figure:
2



EXPLANATION:

Qls	Landslide deposits, undicided (Holocene and Pleistocene)
Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qyc	Young colluvial deposits (Holocene and late Pleistocene)
Qoa	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)
Kgd	Granodiorite, undivided (mid-Cretaceous)
Kt	Tonalite, undivided (mid-Cretaceous)
Kgb	Gabbro, undivided (mid-Cretaceous)
Kwm	Granodiorite of Woodson Mountain (mid-Cretaceous)
Mzu	Metasedimentary and metavolcanic rocks, undicided (Mesozoic)

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.

Qls?

Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

Strike and **dip** of sedimentary joints Vertical

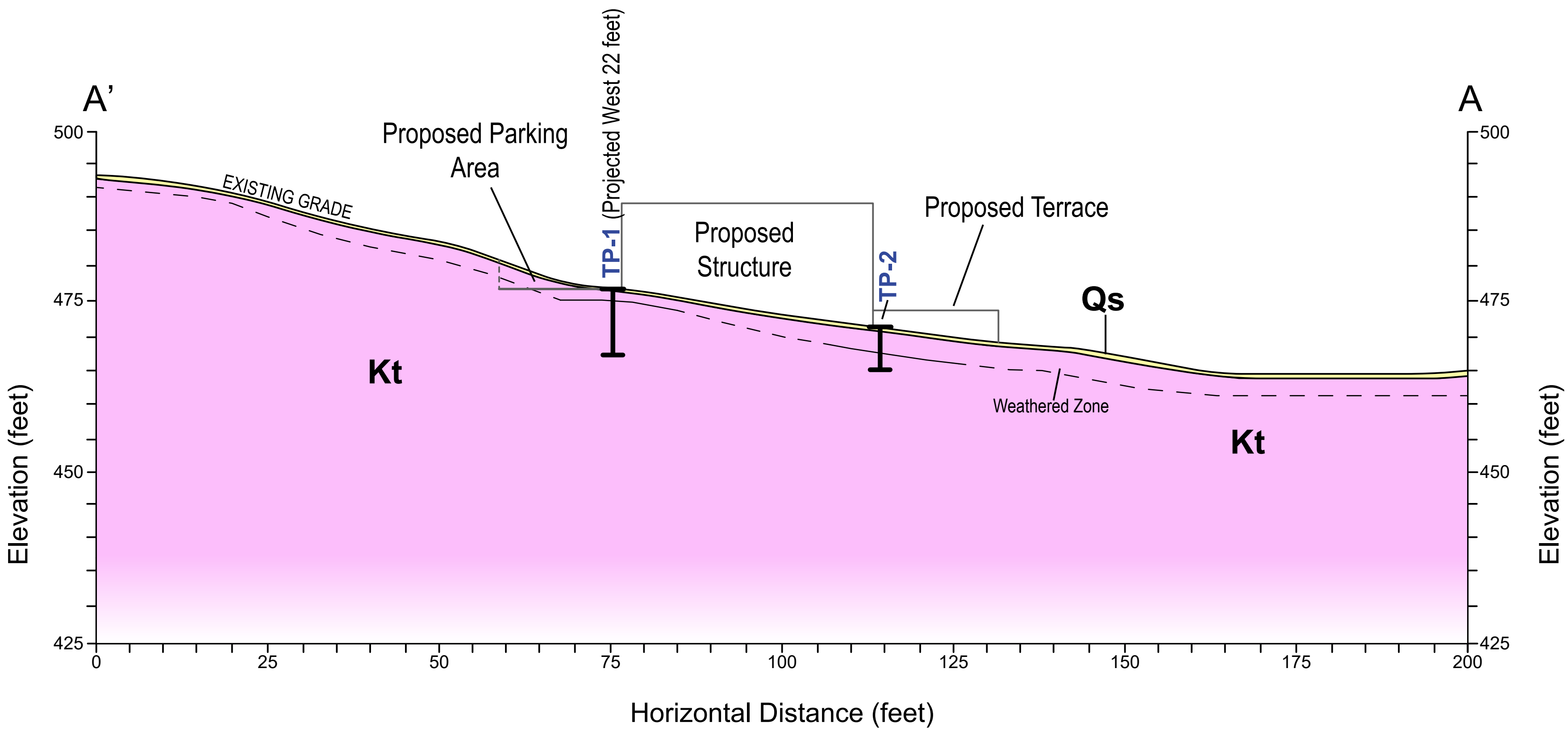
N

0 4000' 8000'

SCALE

Reference:
Geologic Map of the Oceanside 30' x 60' Quadrangle
By Kennedy, M.P. and Tan, S.S. (2008)

SCST, Inc.	**REGIONAL GEOLOGY** 13074 Polvera Avenue San Diego, California	Date: April, 2018 By: PFL Job No.: 180169N-1	Figure: 3



SCST LEGEND:

TP-2 Approximate Location of Test Pit

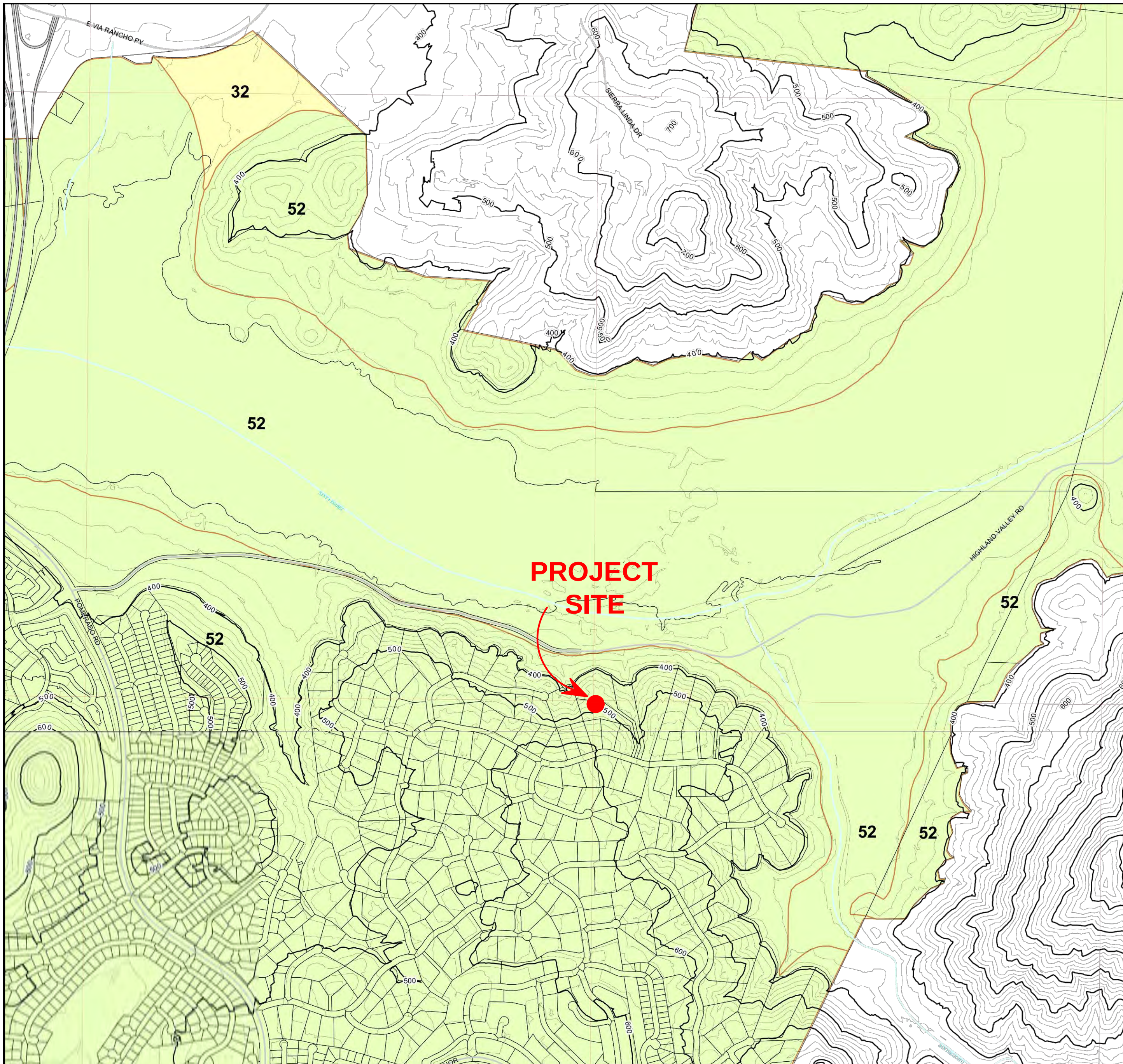
Approximate Geologic Contact
(Dashed where uncertain)

Qs Residual Soil

Kt Granitic Rock

SCALE
1" = 25'

SCST, Inc.	**GEOLOGIC CROSS SECTION** 13074 Polvera Avenue San Diego, California	Date: April, 2018 By: EMW Job No.: 180169N-1	Figure: 4



LEGEND:

32	**LIQUEFACTION:** Low Potential -- fluctuating groundwater minor drainages
52	**OTHER TERRAIN:** Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk

N

0 1500' 3000'

SCALE

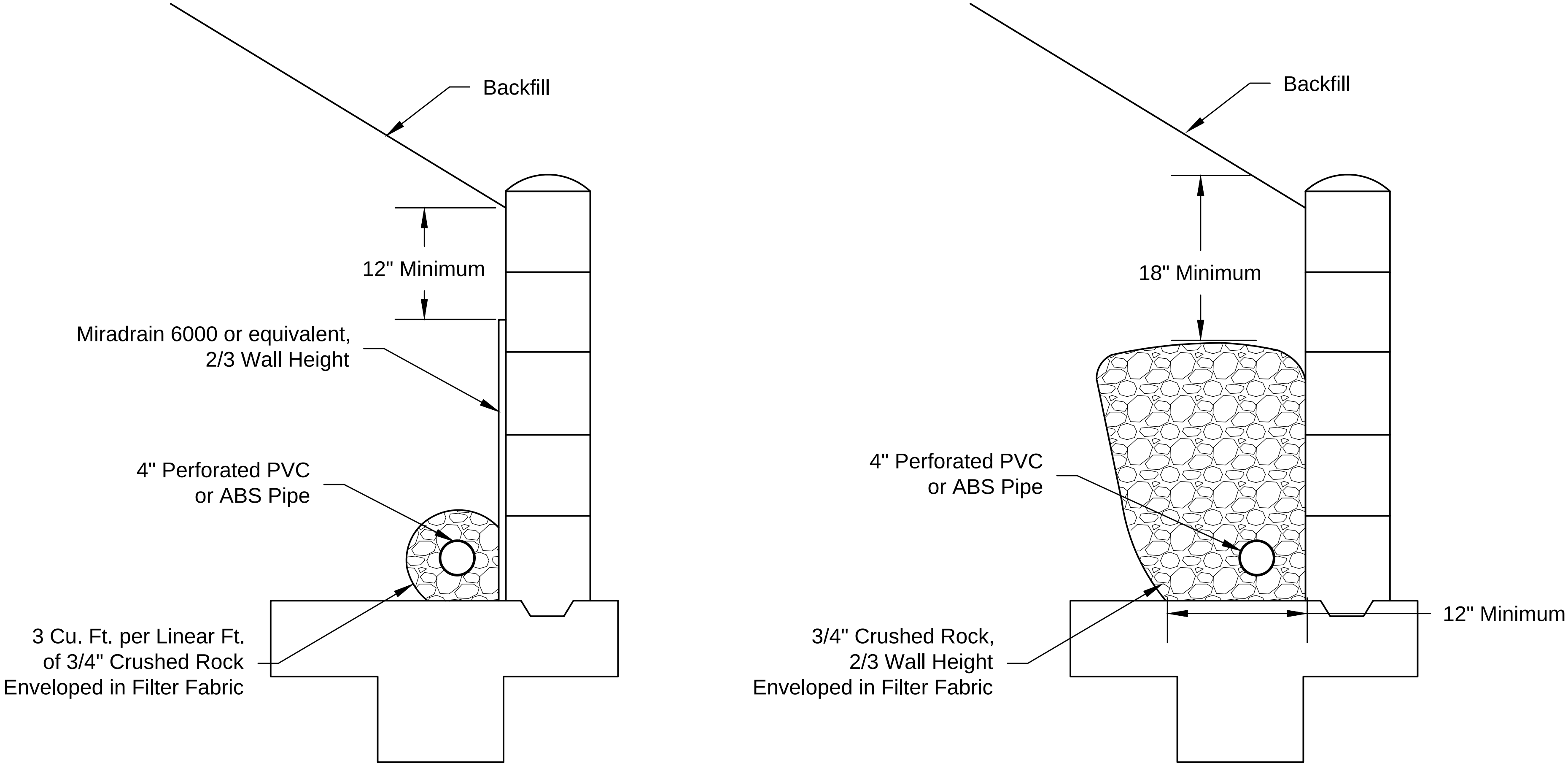
Reference:
City of San Diego Seismic Safety Study

SCST, Inc.

SEISMIC SAFETY STUDY MAP
13074 Polvera Avenue
San Diego, California

Date: April, 2018
By: PFL
Job No.: 180169N-1

Figure: 5

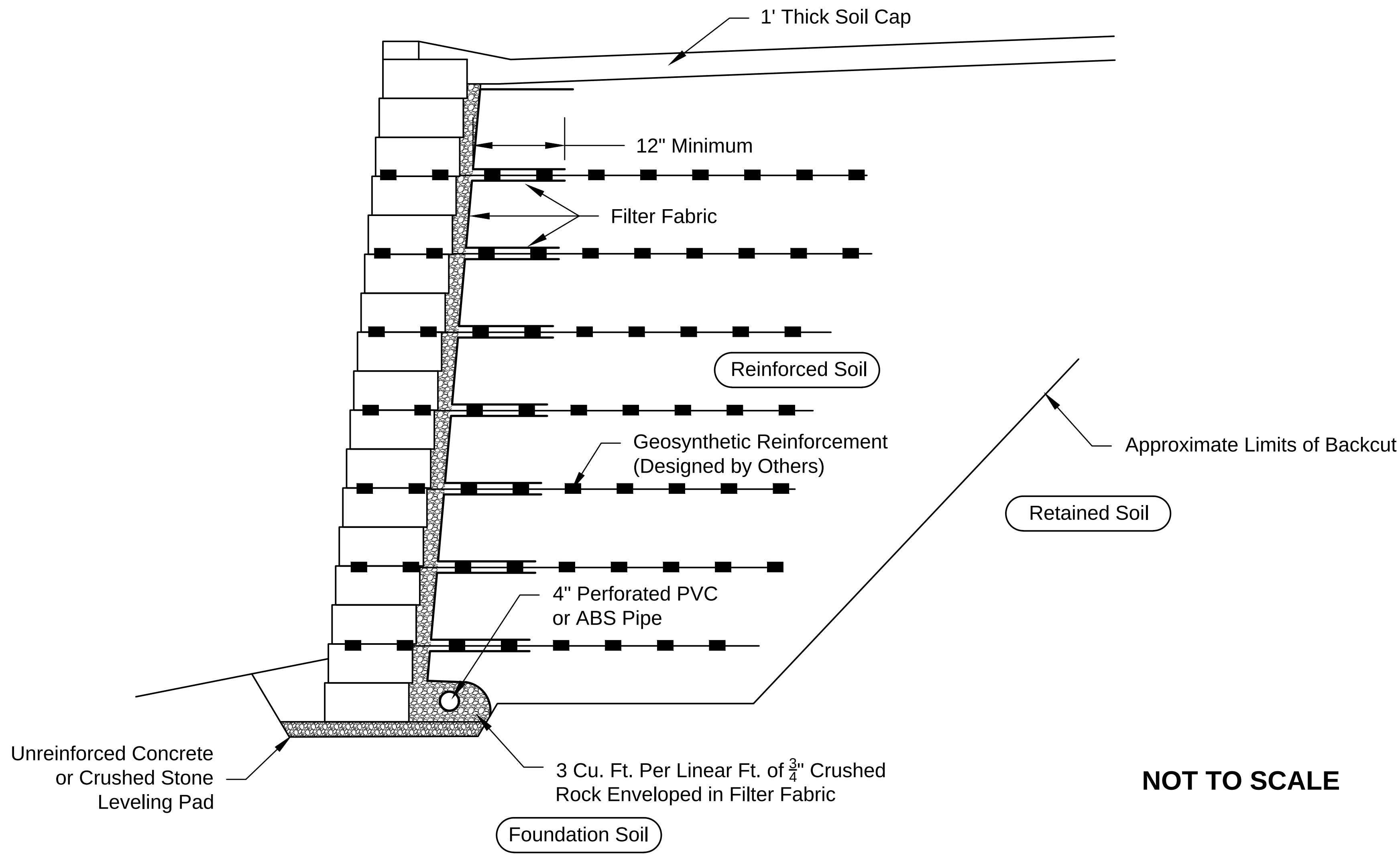


NOT TO SCALE

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, Inc.	**TYPICAL RETAINING WALL BACKDRAIN DETAILS** 13074 Polvera Avenue San Diego, California	Date: April, 2018 By: EMW Job No.: 180169N-1	Figure: 6



NOTES:

1) Backcut as recommended by the geotechnical report or field evaluation
2) Additional drain at excavation backcut may be recommended base on conditions obsewrved during construction.
3) Filter fabric should be installed between crushed rock and soil. Filter fabric should consist of Mirafi 140N or equivalent. Filter fabric should be overlapped approximately 6 inches.
4) Perforated pipe should outlet through a solid pipe to an appropriate gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

SCST, Inc.	**TYPICAL MSE RETAINING WALL DETAIL** 13074 Polvera Avenue San Diego, California	Date: April, 2018 By: EMW Job No.: 180169N-1	Figure: 7

APPENDIX I
FIELD INVESTIGATION

Our field investigation consisted of a visual reconnaissance of the site and excavating five (5) test pits on April 5, 2018 at the subject site to approximate depths between 5 and 10 feet below the existing ground surface using a backhoe. Additionally, a subcontracted Registered Geophysicist performed five (5) seismic refraction surveys that traverses the planned building areas to evaluate rippability characteristics. Figure 2 presents the approximate locations of the test pits. The field investigation was performed under the observation of an SCST geologist who also logged the test pits and obtained samples of the materials encountered. The soils are classified in accordance with the Unified Soil Classification System as illustrated on Figure I-1. Logs of the test pits are presented on Figures I-2 through I-6. The results of the seismic refraction survey are presented in Appendix II.

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, Inc.	Proposed Single-Family Residence - Vardy House San Diego, California			
	By:	EMW	Date:	April, 2018
	Job Number:	180169N-1	Figure:	I-1

LOG OF TEST PIT TP-1

Date Excavated: 4/5/2018	Logged by: EMW
Equipment: Backhoe	Reviewed by: DAS/IC
Elevation (ft): Approximately 483	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
0.5	SM	**RESIDUAL SOIL:** SILTY SAND, loose, brown, moist, fine grained, trace gravel, trace cobble. Roots encountered.							
1.0 – 2.0		**GRANITIC ROCK (Kt)**: medium dense, red brown to yellow brown, moist, weathered.		X					SA EI Corr
2.0 – 4.0		Becomes gray to tan.		X					
4.5 – 7.5									
8.0		Becomes very dense.							
8.5 – 9.5									
10		**REFUSAL @ 10 FEET ON UNWEATHERED TONALITE**							

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	I-2

LOG OF TEST PIT TP-2

Date Excavated: 4/5/2018
Equipment: Backhoe
Elevation (ft): Approximately 478

Logged by: EMW
Reviewed by: DAS/IC
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
0–0.5	SM	**RESIDUAL SOIL:** SILTY SAND, loose, brown, moist, fine grained, trace gravel, trace cobble. Roots encountered.							
0.5–3.0		**GRANITIC ROCK (Kt)**: medium dense, red brown to yellow brown, moist, weathered.							
3.0–8.5		Becomes gray to tan, dense.							
8.5		**REFUSAL @ 8½ FEET ON UNWEATHERED TONALITE**							

Depth scale (ft): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	I-3

LOG OF TEST PIT TP-3

Date Excavated: 4/5/2018
Equipment: Backhoe
Elevation (ft): Approximately 480

Logged by: EMW
Reviewed by: DAS/IC
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
0–0.5	SM	**RESIDUAL SOIL:** SILTY SAND, loose, brown, moist, fine grained, trace gravel, trace cobble. Roots encountered.							
0.75–3.0		**GRANITIC ROCK (Kt)**: medium dense, red brown to yellow brown, moist, weathered.		X					
3.0		Becomes gray to tan, dense.		X					SA
4.0		Becomes gray, very dense.		X					
5.0		**REFUSAL @ 5 FEET ON UNWEATHERED TONALITE**							

Depth scale (ft): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	I-4

LOG OF TEST PIT TP-4

Date Excavated: 4/5/2018
Equipment: Backhoe
Elevation (ft): Approximately 484

Logged by: EMW
Reviewed by: DAS/IC
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
0.5	SM	**RESIDUAL SOIL:** SILTY SAND, loose, brown, moist, fine grained, trace gravel, trace cobble. Roots encountered.							
1.0		**GRANITIC ROCK (Kt)**: medium dense, red brown to yellow brown, moist, weathered.							
1.5		Becomes gray to tan, dense.							
2.0									
2.5									
3.0									
3.5									
4.0									
4.5									
5.0									
5.5									
6.0									
6.5									
7.0									
7.5									
8.0									
8.5									
9.0		**REFUSAL @ 9 FEET ON UNWEATHERED TONALITE**							
9.5									
10									

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	I-5

LOG OF TEST PIT TP-5

Date Excavated: 4/5/2018
Equipment: Backhoe
Elevation (ft): Approximately 486

Logged by: EMW
Reviewed by: DAS/IC
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
0.5	SM	**RESIDUAL SOIL:** SILTY SAND, loose, brown, moist, fine grained, trace gravel, trace cobble. Roots encountered.							
1.0		**GRANITIC ROCK (Kt):** medium dense, red brown to yellow brown, moist, weathered.		X					SA
1.5									
2.0									
2.5									
3.0		Becomes yellow brown.							
3.5									
4.0									
4.5									
5.0		Becomes gray, very dense.							
5.5		**REFUSAL @ 5½ FEET ON UNWEATHERED TONALITE**							
6.0									
6.5									
7.0									
7.5									
8.0									
8.5									
9.0									
9.5									
10									

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	I-6

APPENDIX II
SEISMIC REFRACTION SURVEY

SEISMIC REFRACTION SURVEY
VARDY RESIDENCE
SAN DIEGO, CALIFORNIA

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

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

April 10, 2018
Project No. 118139

SOUTHWEST GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

April 10, 2018
Project No. 118139

Mr. Doug Skinner
SCST, Inc.
6280 Riverdale Street
San Diego, CA 92120

Subject: Seismic Refraction Survey
Vardy Residence
San Diego, California

Dear Mr. Skinner:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Vardy Residence project located at 13074 Polvera Avenue in San Diego, California. Specifically, our survey consisted of performing five 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. 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, INC.

Aaron Puente
Project Geologist/Geophysicist

Hans van de Vrugt
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

ATP/HV/hv
Distribution: Addressee (electronic)

8057 Raytheon Road, Suite 9 • San Diego • California 92111 • Telephone 858-527-0849 • Fax 858-225-0114

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the Vardy Residence project located at 13074 Polvera Avenue in San Diego, California (Figure 1). Specifically, our survey consisted of performing five 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. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of five 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 generally located east of the Interstate 15 and Pomerado Road intersection in a residential estate of San Diego, California (Figure 1). Specifically, the property is located along the north side of Polvera Avenue, just east of the intersection of Polvera Avenue and Olmeda Place. Access to the undeveloped lot is down a steep paved driveway. The seismic lines were conducted along a north facing slope containing tall annual grass and brush. Several scattered granitic outcrops and boulders are present on and near the property. Figures 2 and 3 depict the general site conditions in the area of the seismic traverses.

Based on our discussions with you it is our understanding the project involves the construction of a private single family residence. The data provided herein may be used in the formulation of design and construction parameters.

4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the rippability characteristics of the subsurface materials and to develop a subsurface velocity profile of the area surveyed. 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 generated at the surface, using a hammer and plate, 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.

Five seismic lines (SL-1 through SL-5) were conducted in the study area. The general locations and lengths of the lines were selected by the property owner. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory 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 core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the spread.

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 determining rock quality or rippability.

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, Possible 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.

5. DATA ANALYSIS

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.

6. RESULTS AND CONCLUSIONS

As previously indicated, five seismic traverses were conducted as part of our study. The resulting P-wave velocity models are presented in Figures 4a through 4e. Based on the results it appears that the study area is underlain by low velocity materials (e.g., colluvium and topsoil) in the near surface and granitic rock 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 area.

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 required depending on the excavation depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

7. 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, Inc. 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.

8. SELECTED REFERENCES

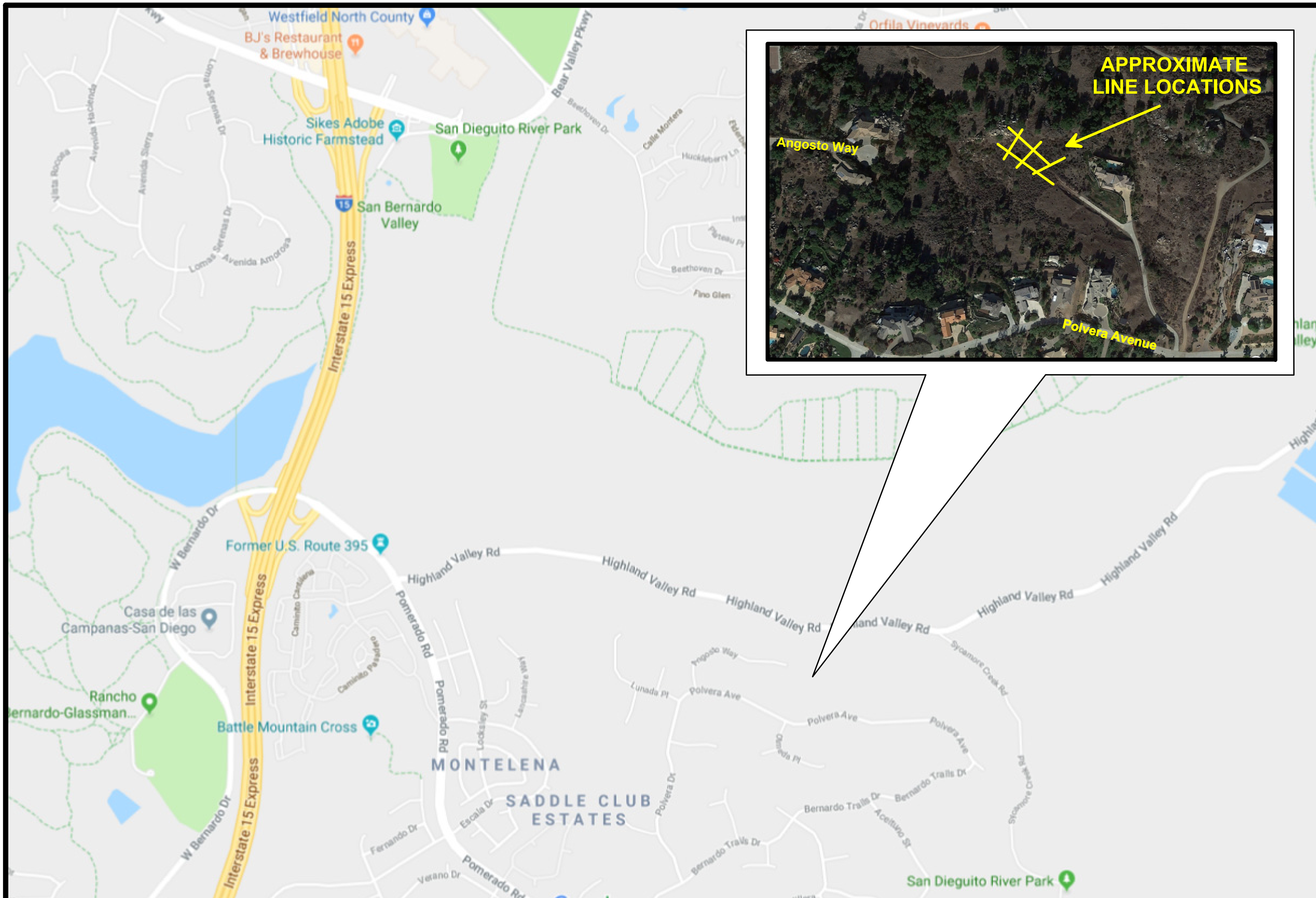
Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

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

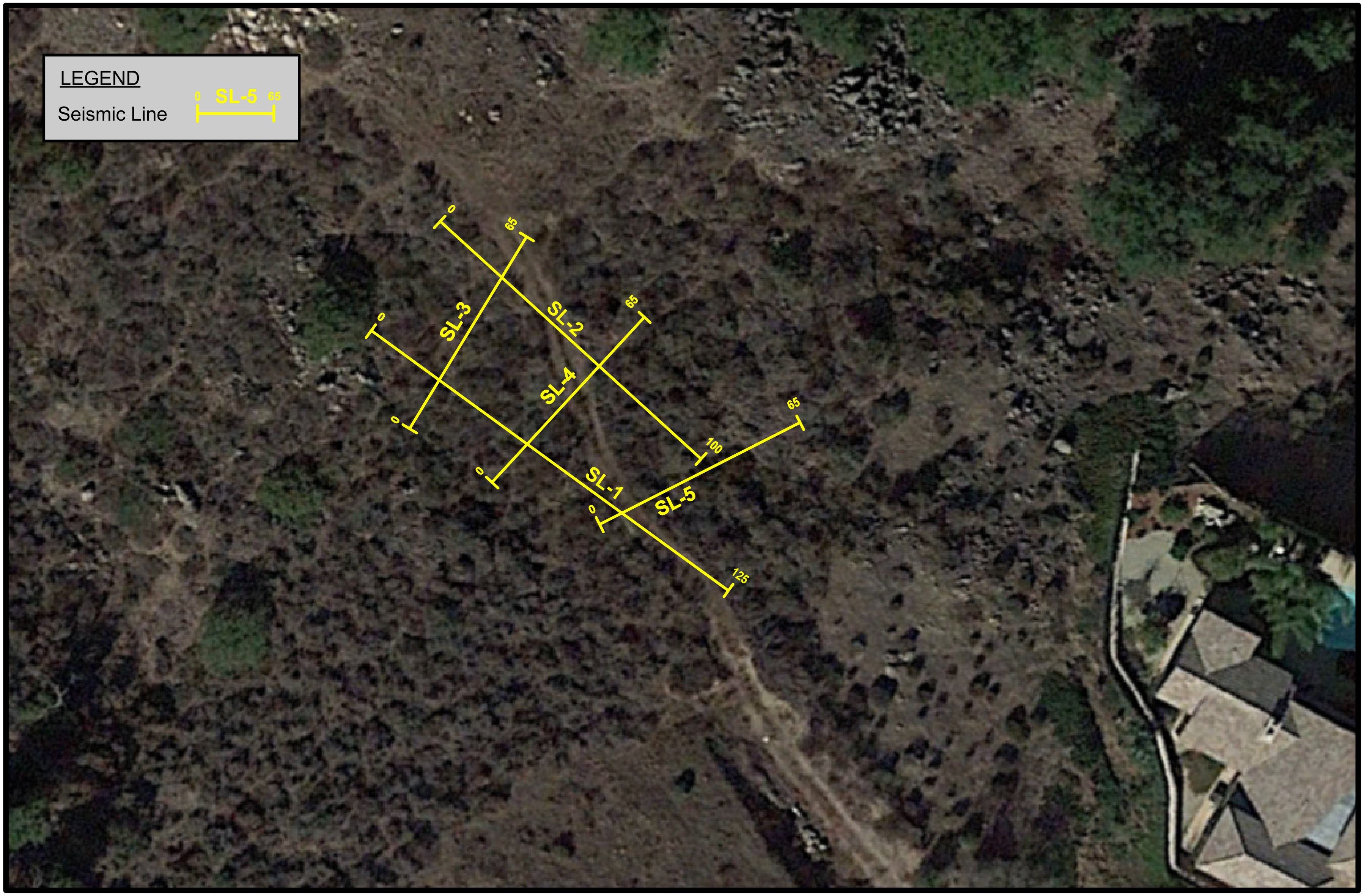
Optim, Inc., 2008, SeisOpt Pro, V-5.0.

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	Vardy Residence San Diego, California Project No.: 118139 Date: 4/18	SOUTHWEST GEOPHYSICS INC. Figure 1



LINE LOCATION MAP

N

Vardy Residence
San Diego, California

Project No.: 118139

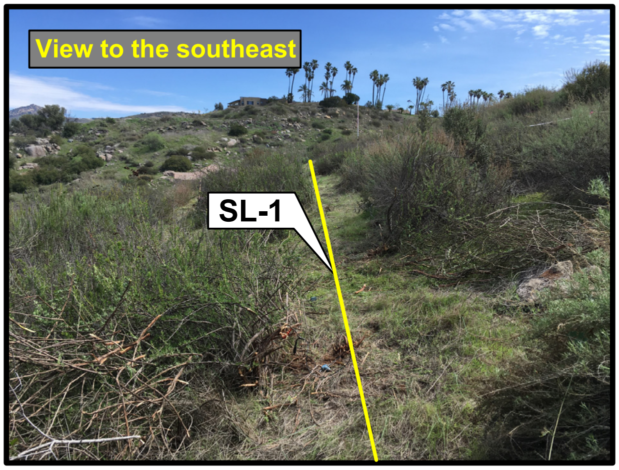
Date: 4/18

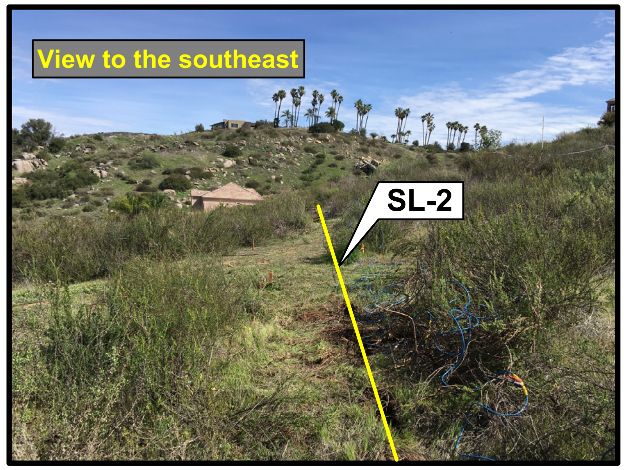
SOUTHWEST GEOPHYSICS INC.

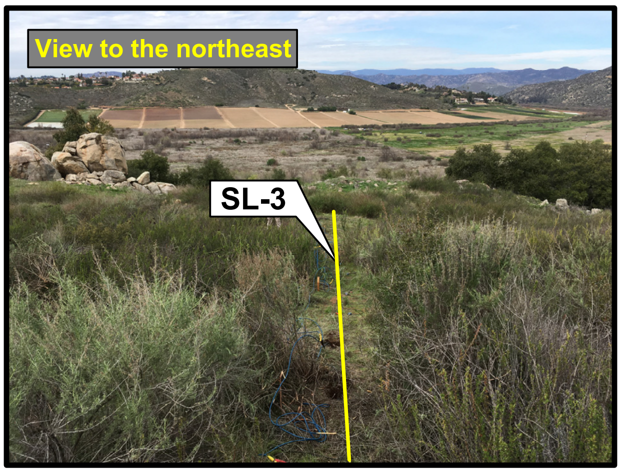
Figure 2

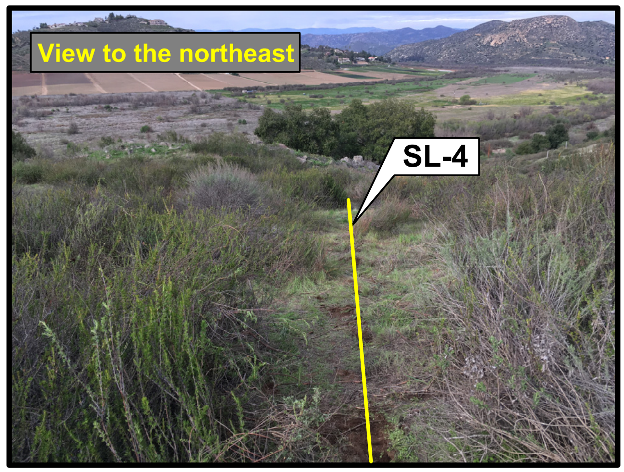
0 40 80

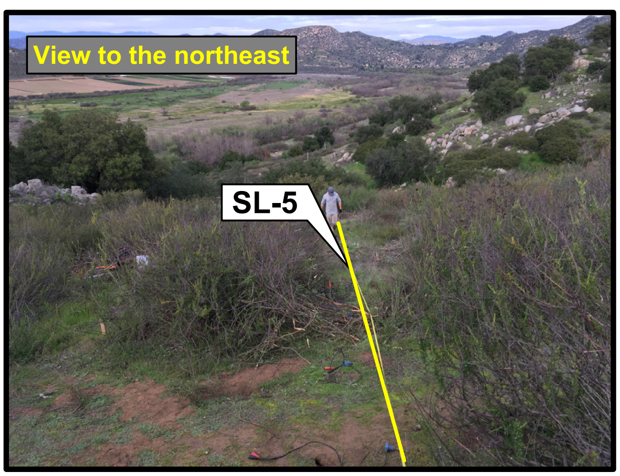
approximate scale in feet











SITE PHOTOGRAPHS

Vardy Residence
San Diego, California

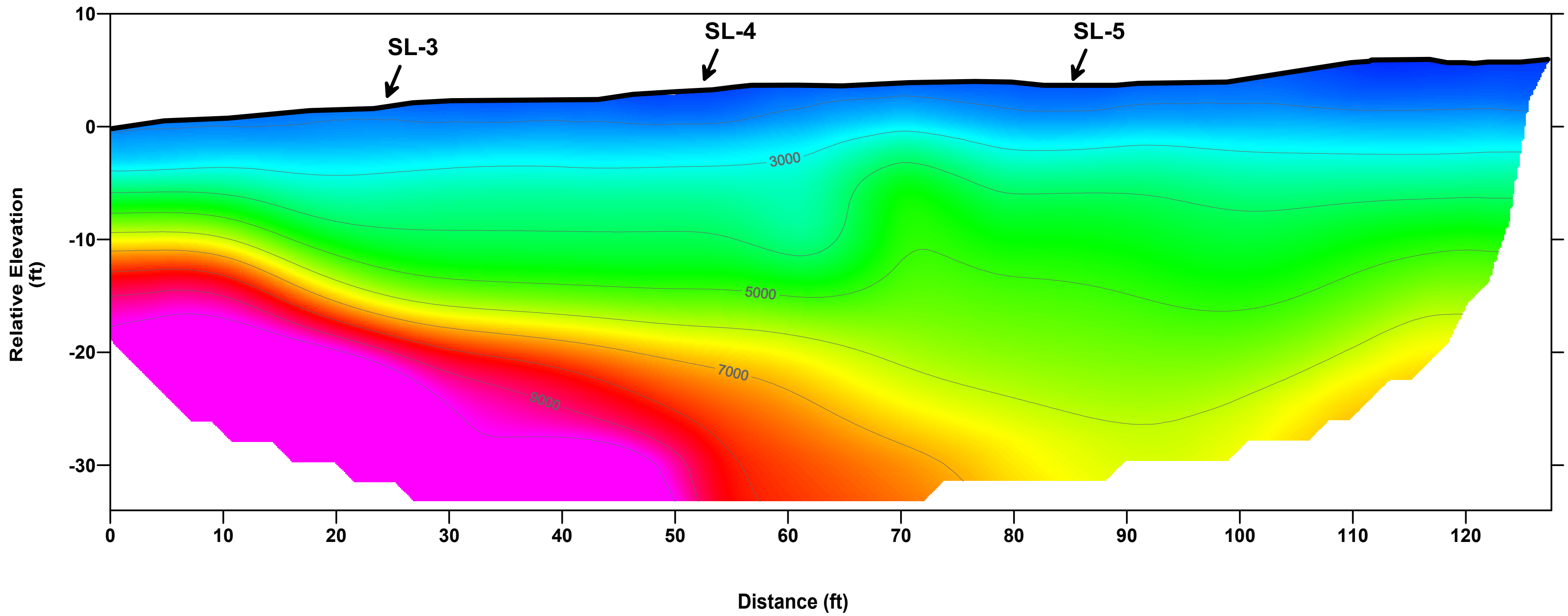
Project No.: 118139

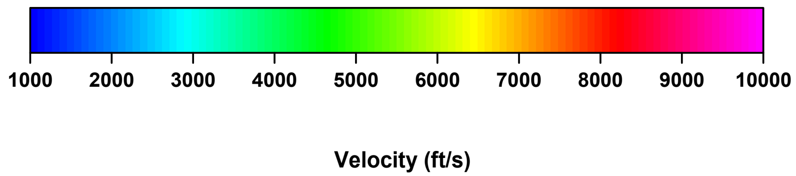
Date: 4/18

SOUTHWEST GEOPHYSICS INC.

Figure 3

TOMOGRAPHY MODEL

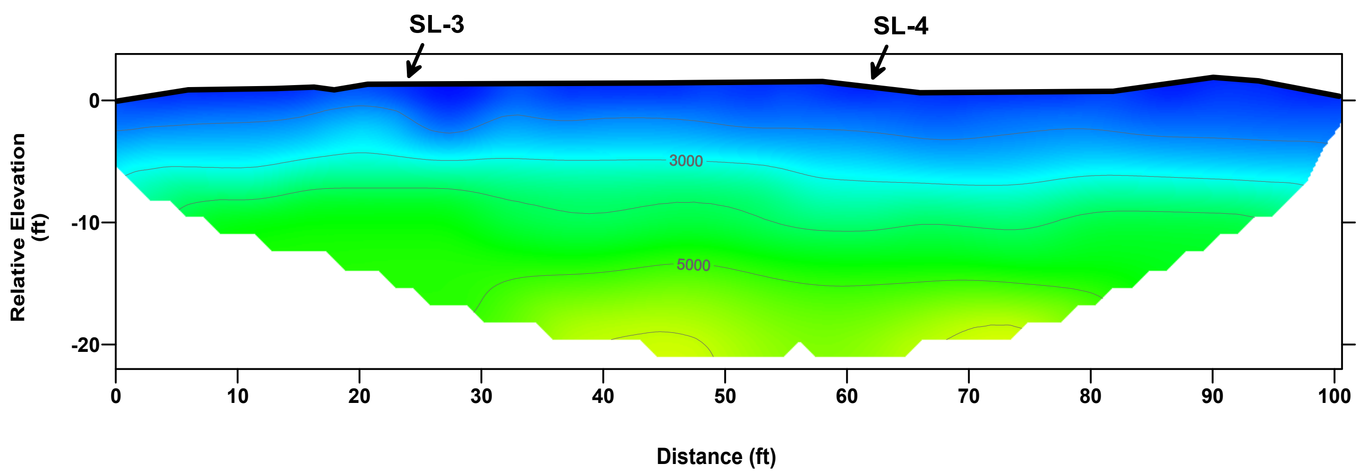


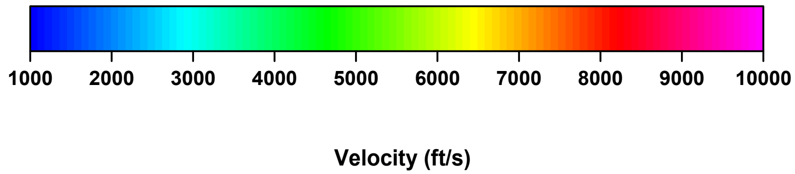


P-WAVE PROFILE SL-1	Vardy Residence San Diego, California Project No.: 118139 · Date: 04/18	SOUTHWEST GEOPHYSICS INC. Figure 4a

Note: Contour Interval = 1,000 feet per second

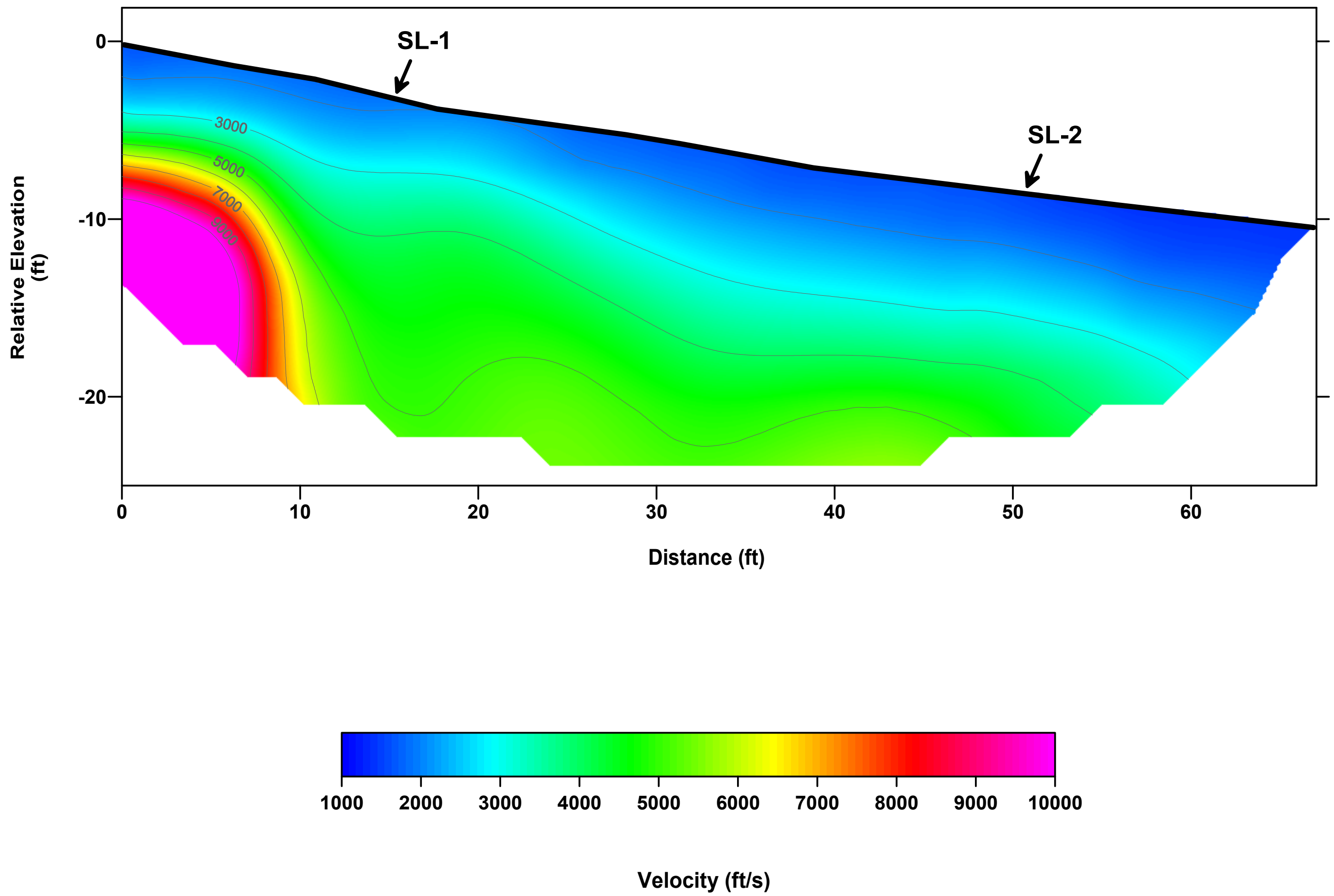
TOMOGRAPHY MODEL





P-WAVE PROFILE **SL-2**	Vardy Residence San Diego, California Project No.: 118139 — Date: 04/18	SOUTHWEST GEOPHYSICS INC. Figure 4b	**Note: Contour Interval = 1,000 feet per second**

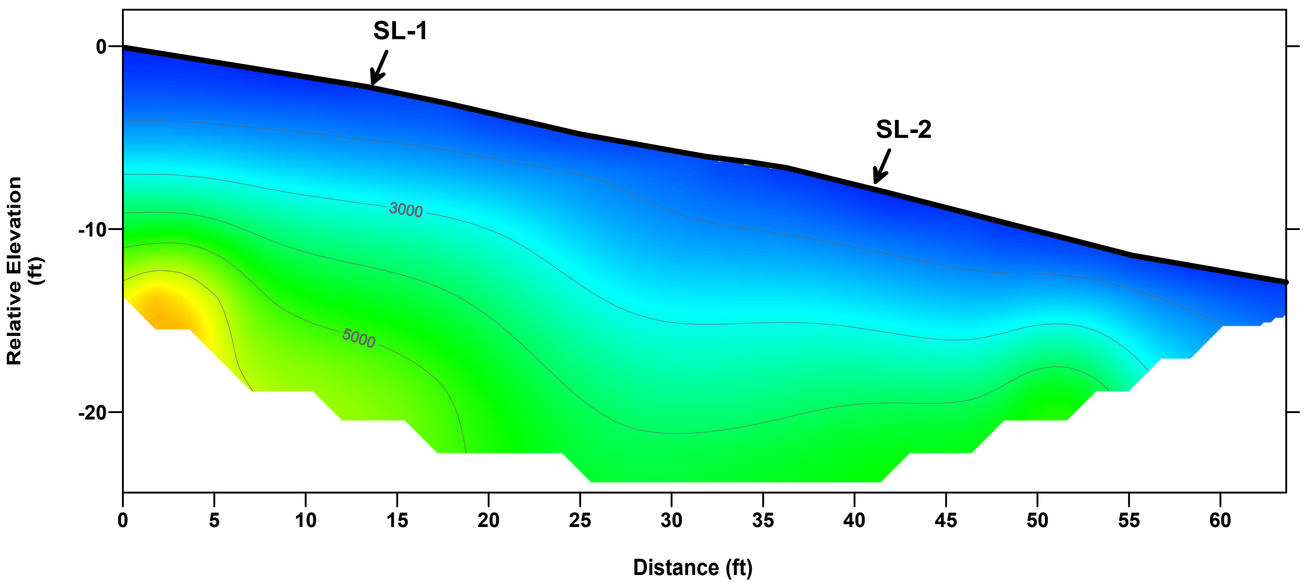
TOMOGRAPHY MODEL

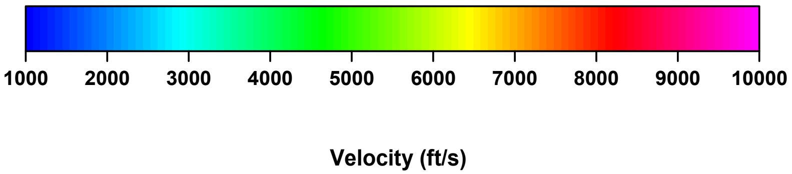


P-WAVE PROFILE SL-3	Vardy Residence San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 4c
	Project No.: 118139 — Date: 04/18	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

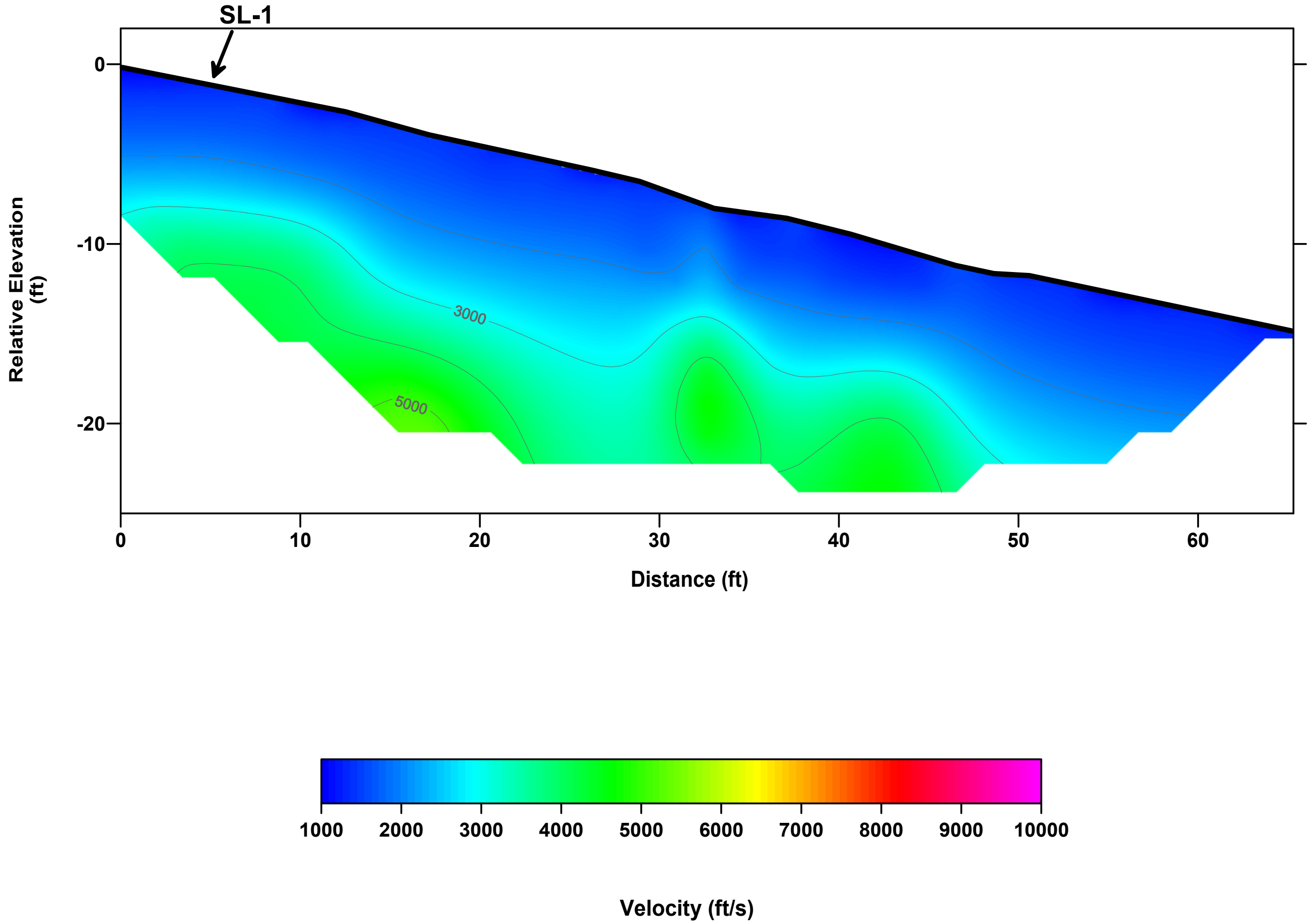




P-WAVE PROFILE SL-4	Vardy Residence San Diego, California		SOUTHWEST GEOPHYSICS INC. Figure 4d
	Project No.: 118139	Date: 04/18	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



P-WAVE PROFILE SL-5	Vardy Residence San Diego, California Project No.: 118139 — Date: 04/18	SOUTHWEST GEOPHYSICS INC. Figure 4e	**Note: Contour Interval = 1,000 feet per second**

APPENDIX III
LABORATORY TESTING

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

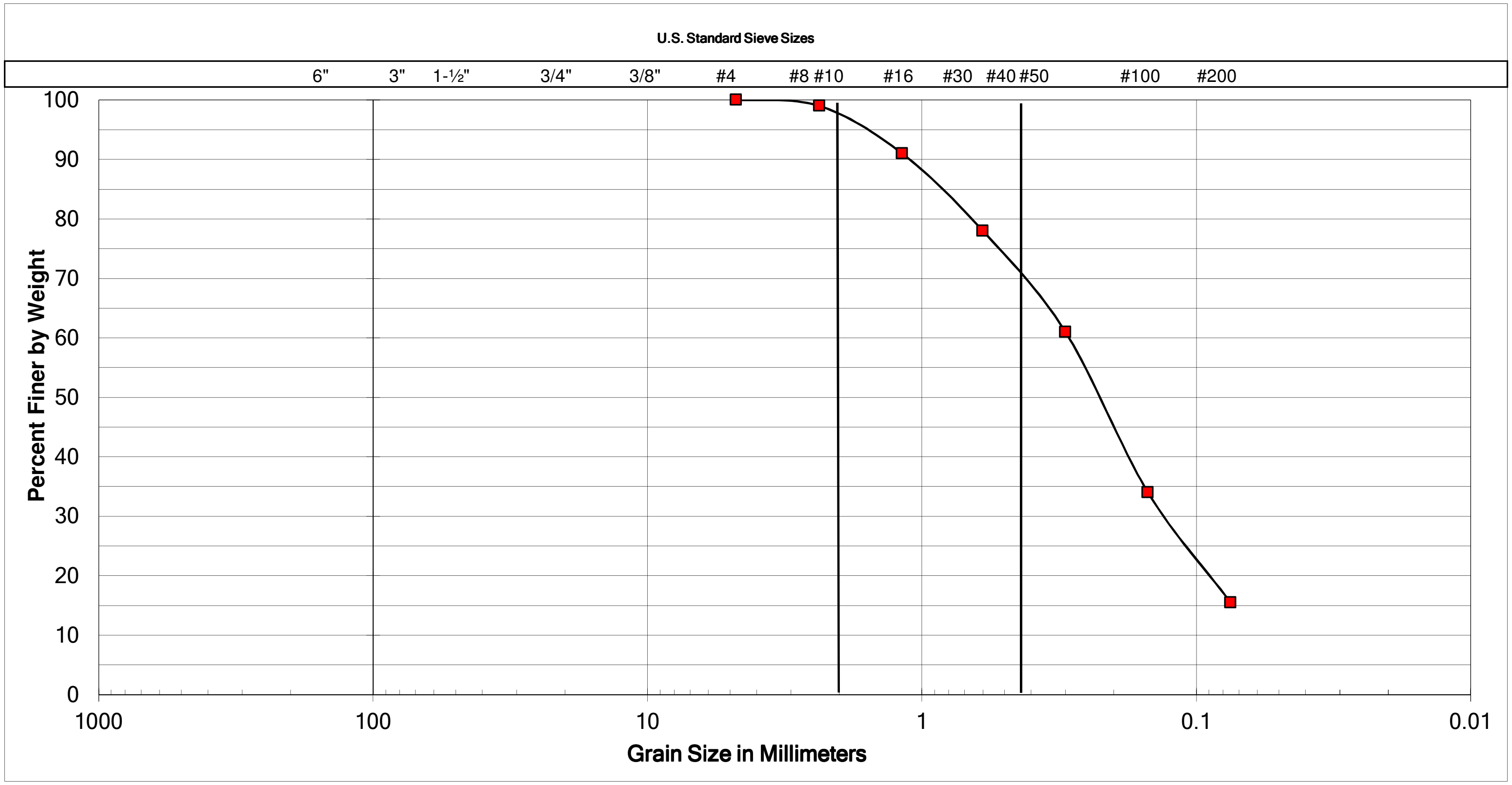
- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System.

- **GRAIN SIZE DISTRIBUTION:** The grain size distribution was determined on three samples in accordance with ASTM D422. Figures III-1 through III-3 present the test results.

- **EXPANSION INDEX:** The expansion index was determined on three samples in accordance with ASTM D4829. Figure III-4 presents the test results.

- **CORROSIVITY**: Corrosivity tests were performed on three samples. The pH and minimum resistivity were determined in general accordance with California Test 643. The soluble sulfate content was determined in accordance with California Test 417. The total chloride ion content was determined in accordance with California Test 422. Figure III-4 presents the test results.

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



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

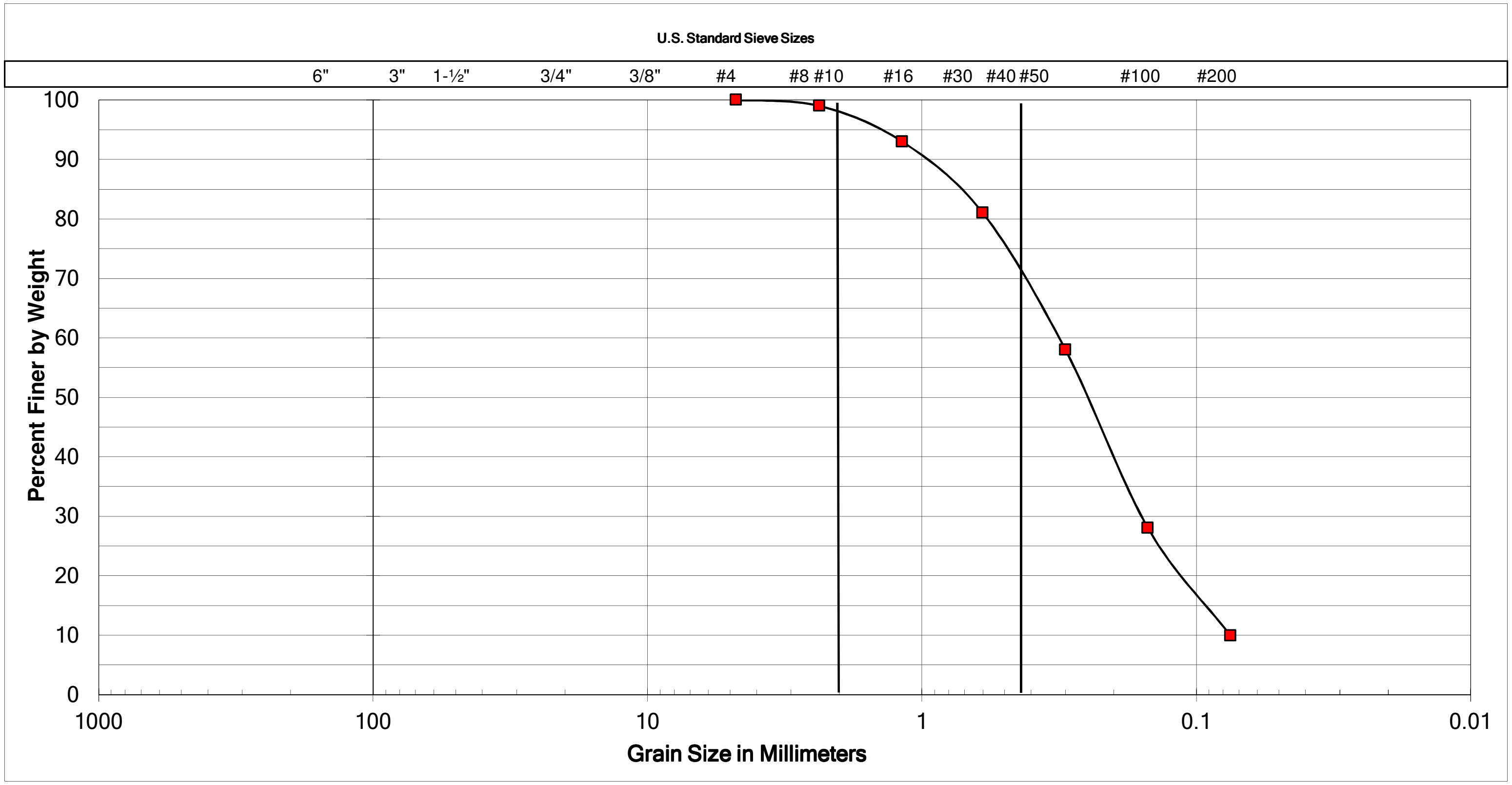
SAMPLE LOCATION
TP-1 at 1 to 2 feet

UNIFIED SOIL CLASSIFICATION:	SM
DESCRIPTION	SILTY SAND

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

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N	Figure:	III-1



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

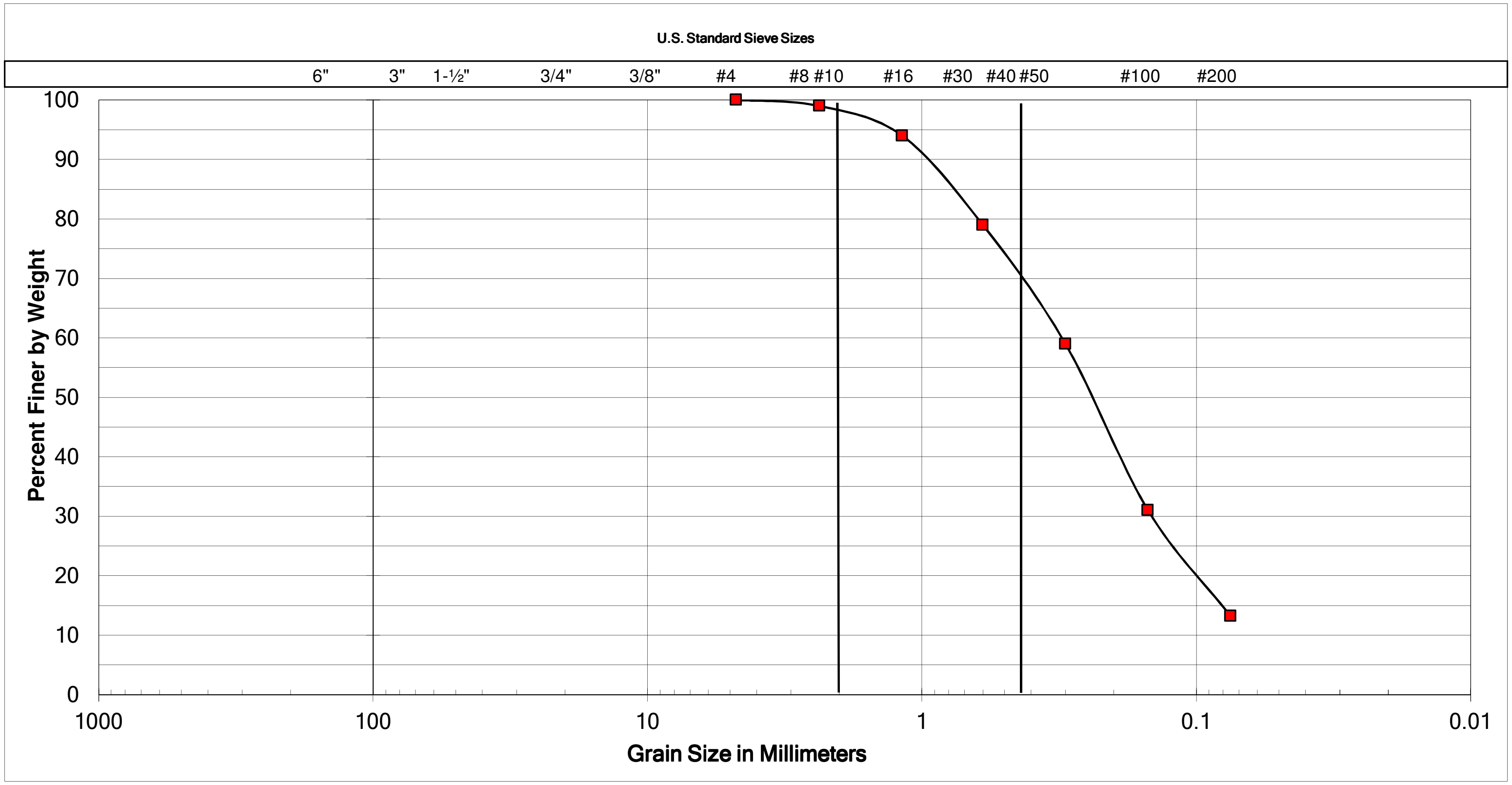
SAMPLE LOCATION
TP-3 at 3 to 5 feet

UNIFIED SOIL CLASSIFICATION:	SM
DESCRIPTION	SILTY SAND

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

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N	Figure:	III-2



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

SAMPLE LOCATION
TP-5 at 1 to 5 feet

UNIFIED SOIL CLASSIFICATION:	SM
DESCRIPTION	SILTY SAND

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

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N	Figure:	III-3

EXPANSION INDEX

ASTM D2489

SAMPLE	DESCRIPTION	EI
TP-1 at 1 to 2 feet	SILTY SAND	2

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

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 (%)
TP-1 at 1 to 2 feet	6840	6.02	0.010	0.000

Sulphate Exposure Classes [2]

CLASS	SEVERITY	WATER-SOLUBLE SULFATE (SO_4) IN SOIL, PERCENT BY MASS
S0	Not applicable	$SO_4 < 0.10$
S1	Moderate	$0.10 \leq SO_4 < 0.20$
S2	Severe	$0.20 \leq SO_4 \leq 2.00$
S3	Very Severe	$SO_4 > 2.00$

2. ACI 318, Table 19.3.1.1

SCST, Inc.

Proposed Single-Family Residence - Vardy House San Diego, California			
By:	EMW	Date:	April, 2018
Job Number:	180169N-1	Figure:	III-4