ATTACHMENT G

UDSP EIR 2025 ADDENDUM - APPENDIX B - GEOTECHNICAL REPORT

APPENDIX B

Geotechnical Report and Update Letter

29 January 2025

Carmel Enterprise, LLC
5550 Carmel Mountain Road, Suite 204
San Diego, CA 92130
Attn: Mr. Gary Levitt, Manager

Job No. 20-12714.2

Subject: **Geotechnical Report Update**
Carmel Enterprise Cube Smart Storage Building
337 East Carmel Street, Parcel 2, APN 220-201-90
San Marcos, California

Dear Mr. Levitt:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has prepared this update to our previous geotechnical investigation for the site, the results of which were presented in our report dated January 04, 2021, for a previous owner (Seabreeze Properties, LLC).

The subject project will be constructed in the northern half portion of a site previously referred to as Campus Pointe. The purpose of this update is to address the currently proposed preliminary civil plans and provide supplemental/revised recommendations as warranted. It is our understanding that the proposed development will consist of two structures. The main structure will be located near the center of the parcel and will consist of a 3 or 4-story building. The secondary building is to be located along the south property line, which will consist of a single-story. Our scope of work for this update included a review of our previous work at the site and the preliminary civil plans.

In general, it is our professional opinion that the design and construction recommendations presented in our previously issued report dated January 04, 2021 conform to the current ASCE 7-16 and 2022 CBC, which remain applicable for the currently proposed construction. As detailed building plans are developed, they should be provided to us for review and supplemental recommendations may be provided if warranted.

Should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 20-12714.2** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Richard A. Cerros, P.E.
R.C.E. 94223

LICENSED PROFESSIONAL ENGINEER
RICHARD A. CERROS
C 94223
EXP. 12/31/26
CIVIL
STATE OF CALIFORNIA

REPORT OF UPDATE GEOTECHNICAL INVESTIGATION
Proposed Campus Pointe Affordable Housing Project
337 East Carmel Street
San Marcos, California

JOB NO. 20-12714
04 January 2021

Prepared for:

Sea Breeze Properties, LLC

Geotechnical Exploration, Inc.
SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

04 January 2021

Sea Breeze Properties, LLC
5550 Carmel Mountain Road, Suite 204
San Diego, CA 92130
Attn: Mr. Gary Levitt

Project No. 20-12714

Subject: **Update Geotechnical Investigation**
Proposed Campus Pointe Affordable Housing Project
337 East Carmel Street
San Marcos, California

Dear Mr. Levitt:

In accordance with your request ***Geotechnical Exploration, Inc***. has performed an update geotechnical investigation for the subject project in San Marcos, California. This update is based on our review of preliminary plans provided us, a previous geotechnical investigation at the site by Robert Prater Associates dated March 12, 1999 (see Appendix A) and a previous seismic refraction survey by Southwest Geophysics, Inc. dated May 16, 2014 (see Appendix B).

If the conclusions and recommendations presented in this report are incorporated into the design and construction of the proposed project, it is our opinion that the site is suitable for the project.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 20-12714** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Wm. D. Hespeler, G.E. 396
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
WM. DAVID HESPELER
No. 396
Exp. 3/31/22
GEOTECHNICAL
STATE OF CALIFORNIA

7420 TRADE STREET ● SAN DIEGO, CA. 92121 ● (858) 549-7222 ● FAX: (858) 549-1604 ● EMAIL: geotech@gei-sd.com

Table of Contents

REPORT OF UPDATE GEOTECHNICAL INVESTIGATION
Proposed Campus Pointe Affordable Housing Project
337 East Carmel Street
San Marcos, California

JOB NO. 20-12714

The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject proposed affordable housing project.

I. PROJECT SUMMARY AND SCOPE OF WORK

Based on our review of preliminary plans provided to us, the project will include 5-story residential structures over one garage level in the northern portion of the site and 3-story residential structures in the southern portion of the site. Based on a preliminary grading study provided us, maximum cuts and fills will be on the order of 4 feet and 18 feet deep, respectively, and require on the order of 50,500 cubic yards of imported fill material.

Based on the preceding, our scope of work included review of preliminary plans provided us, a previous geotechnical investigation at the site by Robert Prater Associates dated March 12, 1999 (see Appendix A) and a previous seismic refraction survey by Southwest Geophysics, Inc. dated May 16, 2014 (see Appendix B). In addition, we performed a brief site visit on December 22, 2020. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for the project earthwork, seismic design and building foundations, slab on-grade floors, basement/retaining walls and pavements.

II. SITE DESCRIPTION

The subject property is located on the south side of East Carmel Street just east of the Sprinter tracks in the City of San Marcos (See Vicinity Map, Figure I). The property is rectangular in shape measuring about 475 by 570 feet in plan dimensions. acres. Based on our recent site visit on December 22, 2020, the site is essentially the same as described in the previous 1999 investigation report.

Elevations across the property range from a high of approximately 622 feet Above Mean Sea Level (AMSL) along the south side of the property to a low of approximately 590 feet AMSL along the northwestern side of the property.

III. SOIL DESCRIPTION

The materials encountered during the previous 1999 investigation consisted of artificial fill, alluvium and colluvium in the lower lying portions of the site that were underlain by granitic rock materials and topsoils over granitic rock materials in the higher portions of the site. The majority of the materials encountered at the site had a low potential for expansion while some of the more weathered materials had a medium expansion potential. Refer to Appendices A and B for details.

IV. GROUNDWATER

Perched groundwater was encountered in the lower lying portion of the site during the 1999 investigation. In our opinion the presence of perched groundwater during construction will depend on the time of year and rainfall events prior to construction. In our opinion, the possible presence of perched groundwater during mass grading of the site will be possible and may result in some extra work related to temporary dewatering. We do not anticipate that groundwater will have any significant impact on the post construction performance of the project. It must be noted, however, that fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, and other possible factors that may not have been evident at the time of the previous field investigation.

It should be kept in mind that grading operations can change surface drainage patterns and/or reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The appearance of such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

V. SEISMIC CONSIDERATIONS

The San Diego area, as most of California, is located in a seismically active region. The San Diego area has been referred to as the eastern edge of the Southern California Continental Borderland, an extension of the Peninsular Ranges Geomorphic Province. The borderland is part of a broad tectonic boundary between the North American and Pacific Plates. The plate boundary is dominated by a complex system of active major strike-slip (right lateral), northwest trending faults extending from the San Andreas fault, about 70 miles east, to the San Clemente fault, about 50 miles west of the San Diego metropolitan area.

Based on our review of some available published information there are no faults known to pass through the site. The prominent fault zones generally considered having the most potential for earthquake damage in the vicinity of the site are the active Rose Canyon and Coronado Bank fault zones mapped approximately 15 and 25 miles southwest of the site, respectively, and the active Elsinore and San Jacinto fault zones mapped approximately 17 and 41 miles northeast of the site, respectively.

Although research on earthquake prediction has greatly increased in recent years, geologists and seismologists have not yet reached the point where they can predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed structures may be subject to the effects of at least one moderate to major earthquake during their design life. During such an earthquake, the danger from fault offset through the site is remote, but relatively strong ground shaking is likely to occur.

VI. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on our review of preliminary plans provided us, the data in a previous geotechnical investigation at the site by Robert Prater Associates dated March 12, 1999 (see Appendix A) and a previous seismic refraction survey by Southwest Geophysics, Inc. dated May 16, 2014 (see Appendix B) and our experience with similar soils and formational materials. In addition, we performed a brief site visit on December 22, 2020. The data obtained and the analyses performed were for the purpose of providing updated design and construction criteria for the project earthwork, seismic design and building foundations, slab on-grade floors, basement/retaining walls and pavements

From a geotechnical engineering standpoint, it is our opinion that the site is suitable for construction of the proposed residential structures and associated improvements provided the conclusions and recommendations presented in this report are incorporated into its design and construction. The primary feature of concern is the mixed foundation conditions that will vary from dense to very dense granitic materials at relatively shallow depth versus significant thicknesses of fill soils.

In order to minimize the potential for excessive differential settlement resulting from the mixed foundation conditions it will be necessary to compact the fills underlying the proposed structures to a higher than normal degree of compaction. Detailed earthwork and foundation recommendations are presented in the following paragraphs. The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project.

> *If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the City Engineer in writing of such change prior to the recommencement of grading and/or foundation installation work and comply with the governing agency's requirements for a change to the Geotechnical Consultant of Record for the project.*

A. Preparation of Soils for Site Development

1. *Clearing and Stripping:* The site should be cleared of the existing structures, pavements, utilities to be abandoned, and any miscellaneous debris that may

be present at the time of construction and stripped of all vegetation. The cleared and stripped materials should be properly disposed of off-site.

2. *Excavation:* See Appendices A and B for information on excavation characteristics.

3. *Removal and Recompaction of Existing Fill, Alluvium. Colluvium and Topsoils:* We recommend that all existing fill soils, alluvium, colluvium and topsoils on the entire site be removed and recompacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12. All fill soils, however, within and 10 feet beyond the building limits should be compacted to a minimum degree of compaction of 95 percent based upon ASTM D1557-12. We also recommend that a minimum fill thickness of 5 feet be provided in the building areas. The higher-than-normal degree of compaction is to minimize the potential for excessive differential settlement due to the anticipated mixed conditions of compacted fills and weathered granitic materials across the site.

4. *Subgrade Preparation:* After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive fill and/or building improvements should be scarified to a depth of 8 inches, moisture conditioned to at least 2 percent above the laboratory optimum, and compacted to the requirements for structural fill.

5. *Material for Fill:* All existing on-site soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. The fill should not, however, contain rocks or lumps more than 3 inch in greatest dimension and no more than 10 percent of the fill should be larger than ¼-inch. The required imported fill material should be a granular, low expansion potential soil and

not contain rocks or lumps more than 3 inches in greatest dimension and no more than 10 percent of the fill should be larger than ¼-inch. Imported fill should also have a minimum laboratory maximum density of 126 pounds per cubic foot as determined by ASTM D1557-12. All materials for use as fill should be approved by our representative prior to importing and filling.

6. *Fill Compaction:* All fill should in general be compacted to a minimum degree of compaction of 90 percent at a moisture content at least 2 percent above the laboratory optimum based upon ASTM D1557-12. Structural fill within and extending at least 10 feet beyond the building limits, however, should be compacted to a minimum degree of compaction of 95 percent. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture.

7. *Permanent Slopes:* We recommend that any required permanent cut and fill slopes be constructed to an inclination no steeper than 2.0:1.0 (horizontal to vertical). The project plans and specifications should contain all necessary design features and construction requirements to prevent erosion of the on-site soils both during and after construction. Slopes and other exposed ground surfaces should be appropriately planted with a protective groundcover.

   Fill slopes should be constructed to assure that the recommended minimum degree of compaction is attained out to the finished slope face. This may be accomplished by "backrolling" with a sheepsfoot roller or other suitable equipment as the fill is raised. Placement of fill near the tops of slopes should be carried out in such a manner as to assure that loose, uncompacted soils are not sloughed over the tops and allowed to accumulate on the slope face.

8. *Temporary Slopes:* In our opinion, temporary slopes up to 12 feet in height cut into the weathered granitic materials should be safe at an inclination of 0.75 horizontal to 1 vertical.

9. *Trench Backfill:* All backfill soils placed in utility trenches should be compacted by mechanical means to a minimum degree of compaction of 90 percent. Backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized. In pavement areas, that portion of the trench backfill within the pavement section should conform to the material and compaction requirements of the adjacent pavement section.

   Our experience has shown that even shallow, narrow trenches, such as for irrigation and electrical lines, that are not properly compacted can result in problems, particularly with respect to shallow groundwater accumulation and migration.

10. *Surface Drainage:* Positive surface gradients should be provided adjacent to the structures and roof gutters and downspouts should be installed so as to direct water away from foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed anywhere on the site. Appropriate erosion control measures should be taken at all times during and after construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas.

B. ***Seismic Design***

11. *Seismic Design Criteria:* Site-specific seismic design criteria for the proposed structure is presented in the following table in accordance with the 2019 CBC, which incorporates by reference ASCE 7-16 for seismic design. We have determined the mapped spectral acceleration values for the site, based on a latitude of 33.1374 degrees and longitude of -117.1559 degrees, utilizing a third-party tool provided by the USGS, which provides a solution for ASCE 7-16 (2019 CBC) utilizing digitized files for the Spectral Acceleration maps. We have assigned a Site Soil Classification of C.

TABLE I
Mapped Spectral Acceleration Values and Design Parameters

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}
0.895g	0.329g	1.2	1.5	1.074g	0.494g	0.716g	0.329g

C. ***Foundation Recommendations***

12. *Footings:* We recommend that the proposed structures be supported on conventional, individual-spread and/or continuous footing foundations bearing on 95 percent well compacted fill soil. All footings should be founded at least 2 feet below the lowest adjacent finished grade.

At the recommended depth, footings may be designed for allowable bearing pressures of 5,000 pounds per square foot (psf) for combined dead and live loads and 6,700 psf for all loads, including wind or seismic. The footings should, however, have a minimum width of 18 inches.

13. *General Criteria for All Footings*: Footings located adjacent to the tops of slopes should be extended sufficiently deep so as to provide at least 10 feet of horizontal cover or 1½ times the width of the footing, whichever is greater, between the slope face and outside edge of the footing at the footing bearing level. Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.5 to 1.0 plane projected upward from the bottom edge of the adjacent utility trench.

All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of four No. 5 reinforcing bars be provided in the footings (two near the top and two near the bottom). A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

14. *Lateral Loads:* Lateral load resistance for the structures supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 350 pounds per cubic foot (pcf) acting against the foundations

may be used in design provided the footings are poured neat against the adjacent undisturbed compacted fill or formational materials. These lateral resistance values assume a level surface in front of the footing for a minimum distance of three times the embedment depth of the footing and any shear keys.

15. *Settlement*. For footings designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlements should not exceed 1 inch and that post-construction differential settlements should be less than ¼-inch in 25 feet.

D. Concrete Slab-on-grade Criteria

16. *Minimum Floor Slab Thickness and Reinforcement:* Based on our experience, we have found that, for various reasons, floor slabs occasionally crack, causing brittle surfaces such as ceramic tiles to become damaged. Therefore, we recommend that all slabs-on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur.

    16.1 Interior floor slabs should be a minimum of 5 inches actual thickness and be reinforced with at least No. 4 bars on 18-inch centers, both ways, placed at midheight in the slab. Actual floor slab reinforcement should be provided by the project structural engineer. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc***. representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

16.2 Following placement of any concrete floor slabs, sufficient drying time must be allowed prior to placement of floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

17. *Slab Moisture Protection and Vapor Barrier Membrane:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested minimum protection criteria. Actual recommendations should be provided by the architect and waterproofing consultants.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-97 (2009) Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs;

ASTM E154-88 (2005) Standard Test Methods for Water Vapor Retarders Used in Contact with Earth; ASTM E96-95 Standard Test Methods for Water Vapor Transmission of Materials; ASTM E1643-98 (2009) Standard Practice for Installation of Water Vapor Retarders Used in Contact Under Concrete Slabs; and ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

17.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and sub-paragraphs 7.1.1-7.1.5) should be less than 0.01 perms (grains/square foot/hour in Hg) and comply with the ASTM E1745 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643. The basis of design is 15-mil StegoWrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the slab be poured directly on the vapor barrier which is placed directly on the prepared subgrade soil; no gravel or sand layers are used.

17.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped and sealed with mastic or the manufacturer's recommended tape or sealing products. In actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement.

17.3 Vapor retarders/barriers do not provide full waterproofing for structures constructed below free water surfaces. They are intended to help reduce or prevent vapor transmission and/or capillary migration through the soil and through the concrete slabs. Waterproofing systems must be designed and properly constructed if full waterproofing is desired. The owner and project designers should be consulted to determine the specific level of protection required.

17.4 Following placement of concrete floor slabs, sufficient drying time must be allowed prior to placement of any floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

18. *Exterior Slabs:* As a minimum for protection of on-site improvements, we recommend that all exterior pedestrian concrete slabs be 4 inches thick, founded on properly compacted and tested fill, be reinforced with No. 3 bars at 18-inch centers, both ways, at the center of the slab, and contain adequate isolation and control joints.

The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing.

19. *Retaining/Basement Walls*: Retaining/basement walls must be designed to resist lateral earth pressures and any additional lateral pressures caused by surcharge loads on the adjoining retained surface. We recommend that unrestrained (cantilever) walls be designed for an equivalent fluid pressure of 35 pcf for level backfill, and 50 pcf for 2.0:1.0 sloping backfill. We recommend that restrained walls (i.e., basement walls and any retaining walls with angle points that restrain them from rotation) with level backfill be designed for an equivalent fluid pressure of 35 pcf plus an additional uniform pressure of 8H psf where H equals the wall height in feet. For 2.0:1.0 sloping backfill we recommend that restrained walls be designed for an equivalent fluid pressure of 50 pcf plus an additional uniform pressure of 10H psf. Wherever walls will be subjected to surcharge loads, they should also be designed for an additional uniform lateral pressure equal to one-third the anticipated surcharge pressure in the case of unrestrained walls and an additional one-half the anticipated surcharge pressure in the case of restrained walls.

The preceding design pressures assume that the walls are backfilled with low expansion potential materials (Expansion Index less than 50) and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that drainage be provided by a composite drainage material such as J-Drain 200/220 and J-Drain SWD or equivalent; no gravel or pipes are used with the J-Drain system. The drain material should terminate 12 inches below the finish surface where the surface is covered by slabs or 18 inches below the finish surface in landscape areas.

For seismic design of unrestrained walls, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 12 pcf. For restrained walls (i.e., basement walls or any walls with angle points that restrain them from rotation) we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 17 pcf added to the active static fluid pressure utilizing an equivalent fluid weight of 35 or 50 pcf.

Backfill placed behind the walls should be compacted to a minimum degree of compaction of 90 percent using light compaction equipment. If heavy equipment is used, the walls should be appropriately temporarily braced.

E. Pavements

20. *Parking Garage Floor:* We recommend that concrete parking garage floors subject only to automobile and light truck traffic be 6 inches thick. The upper 6 inches of the pavement subgrade soils should be compacted to a minimum degree of compaction of 95 percent just prior to paving. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 2000 Edition, for Class 560-C-3250.

    In order to control shrinkage cracking, we recommend that the garage floor slabs be reinforced with No. 4 bars at 18-inch spacing both ways at the center of the slabs.

21. *Asphalt Concrete Pavements:* We anticipate that pavement sections for the proposed development should consist of 2.5 inches of asphalt concrete on 4 to 6 inches of aggregate base for parking stalls and minor traffic channels (Traffic

Index of 4.0) and 3 inches of asphalt concrete on 7 to 9 inches of aggregate base for major automobile traffic channels and areas subject to occasional heavy truck traffic such as trash and fire trucks and moving vans (TI of 5.5). The final thickness of the base sections should be determined during construction based on R-value tests on the materials exposed at the rough subgrade level.

Asphalt concrete should consist of Type III-C2-PG 64-10) conforming to the Standard Specifications for Public Works Construction, 2000 Edition (Standard Specifications), Section 400-4 and be placed in accordance with Section 302-5. Aggregate base should conform to the requirements for Crushed Aggregate Base or Crushed Miscellaneous Base in Section 200-2 of the Standard Specifications. The upper 6 inches of the pavement subgrade soil as well as the aggregate base layer should be compacted to a minimum degree of compaction of 95 percent. Preparation of the subgrade and placement of the asphalt concrete and base materials should be performed under the observation of our representative.

F. General Recommendations

22. *Project Start Up Notification:* In order to minimize any work delays during site development, this firm should be contacted 24 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or re-designing foundation structures at any locations, any formwork or steel reinforcement in the affected

footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

VII. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be retained to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in this "*Report of Update Geotechnical Investigation*" for the project. In addition, the compaction of any fill soils placed during site grading work must be observed and tested by the soil engineer. It is the responsibility of the grading contractor to comply with the requirements on the grading plans and the local grading ordinance. All retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

VIII. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our document review, as well as our experience with similar soils and formational materials located in this area of San Diego County. Of necessity, we must assume a certain degree of continuity between exploratory excavations. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time grading operations begin or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the City of San Marcos. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the grading and structural plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 20-12714** will expedite a reply to your inquiries.

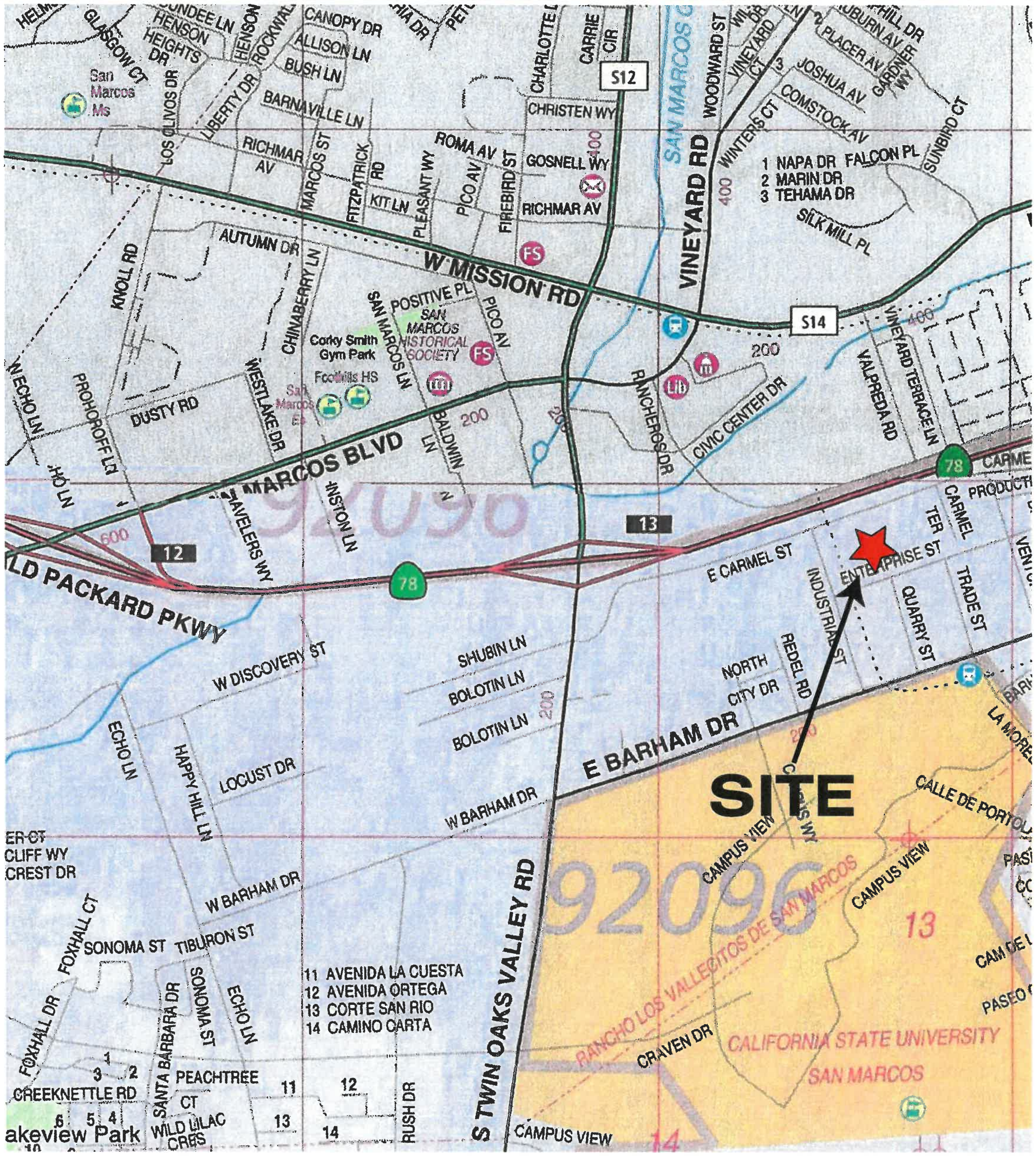
Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Wm. D. Hespeler, G.E. 396
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
WM. DAVID HESPELER
No. 396
Exp. 3/31/22
GEOTECHNICAL
STATE OF CALIFORNIA

VICINITY MAP



Thomas Bros Guide San Diego County pg 1128-J1

Proposed Campus Pointe
Affordable Housing Project
337 East Carmel Street
San Marcos, CA.

Figure No. I
Job No. 20-12714

APPENDIX A

Geotechnical Investigation Report by Robert Prater Associates
dated March 12, 1999

GEOTECHNICAL INVESTIGATION

FOR

LIGHT INDUSTRIAL BUILDING
APN 220-201-01, 02
SAN MARCOS, CALIFORNIA

MARCH 1999

March 12, 1999
546-6, 99-61

Elkins Zirpolo Inc.
2150 First Avenue
San Diego, California 92101

Attention: Mr. Alex Zirpolo

Re: Geotechnical Investigation
Light Industrial Building
APN 220-201-01,02
San Marcos, California

Gentlemen:

In accordance with your request we have performed a geotechnical investigation for the subject building. The accompanying report presents the results of our field investigation, laboratory tests, and engineering analysis. The soil, foundation, and geologic conditions are discussed and recommendations for the geotechnical engineering aspects of the construction are presented.

If you have any questions concerning our findings, please call.

Very truly yours,

ROBERT PRATER ASSOCIATES

Wm. D. Hespeler, G.E.

REGISTERED PROFESSIONAL ENGINEER
WM. DAVID HESPELER
No. 396
Exp. 3/31/2002
GEOTECHNICAL
STATE OF CALIFORNIA

James J. Barton, C.E.G.

REGISTERED GEOLOGIST
JAMES JOSEPH BARTON
No. 1301
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

WDH/JJB:jb
Copies: Addressee (6)
Construction Testing and Engineering Inc., Attn: Mr. Marius Sinca (4)

GEOTECHNICAL INVESTIGATION

For

LIGHT INDUSTRIAL BUILDING
APN 220-201-01, 02
San Marcos, California

To

ELKINS ZIRPOLO INC.
2150 First Avenue
San Diego, California 92101

MARCH 1999

TABLE OF CONTENTS

TABLE OF CONTENTS (con't.)

GEOTECHNICAL INVESTIGATION

FOR

LIGHT INDUSTRIAL BUILDING
APN 220-201-01, 02
SAN MARCOS, CALIFORNIA

INTRODUCTION

In this report we present the results of our geotechnical investigation for the subject project located at 337 East Carmel Street in San Marcos, California (see Vicinity Map, Figure 1.) The purpose of this investigation was to evaluate the subsurface soil and geologic conditions at the site and to provide recommendations concerning the soil, foundation and geologic engineering aspects of the project.

It is our understanding that a concrete tilt up industrial building with a plan area of about 100,000 square feet with a mezzanine will be constructed at the site. We anticipate that maximum combined dead plus live column and wall loads will be on the order of 70 kips and 4 kips per lineal foot, respectively. In addition, retaining walls up to a maximum height of about 10 feet will be constructed along the south side of the project. A 72-inch RCP storm drain pipe will be installed along the east and north property boundaries. Paved parking and drives will also be provided on-site. Grading will consist of cuts and fills of less than about 13 feet in depth.

SCOPE

Our scope of work was performed in accordance with our proposal dated February 15, 1999, as well as verbal authorization for additional seismic refraction exploration. This investigation included a site reconnaissance, subsurface exploration, laboratory testing, engineering analysis of the field and laboratory data, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for site earthwork, building foundations, slab-on-grade floors, retaining walls, and pavements.

SITE CONDITIONS

A. Surface

The property is located in the foothills of the Peninsular Ranges Geomorphic Province. The site is generally characterized by a moderately sloping, north facing hillside. An east-west trending drainage tributary is present near the base of the hillside crossing the center of the property. An existing concrete box culvert drains to the drainage channel near the center of the eastern side of the site. The northwest third of the drainage channel is lined with rip rap. At the time of our investigation, a steady stream was present along the channel. In addition, a dry, shallow earth lined drainage ditch is present adjacent and parallel to the existing Carmel Street along the northern margin of the site. Elevations across the site range from a high of approximately 622 feet above mean sea level along the south side of the property to a low of approximately 590 feet along the northwest side of the property.

The property is currently occupied by a single-story, wood-frame residence and associated appurtenances including several wood and/or metal sheds. The eastern property line is bordered by a concrete retaining wall that covers approximately two-thirds of the eastern boundary, extending north from near Enterprise Street to an existing residential property. The retaining wall extends up to a maximum height of about 5 feet to an existing asphalt concrete pavement. The property is bordered by a fence and Enterprise Street along the south side of the property. A fence and an adjacent commercial development border the west side of the property. Carmel Street and a drainage ditch are present along the north side of the property.

B. Subsurface

A subsurface investigation was performed at the site using a Case 580 D backhoe to explore and sample the subsurface soils. Twelve exploratory test pits were excavated on February 24, 1999 to a maximum depth of 11-1/2 feet at the approximate locations shown on the Site Plan and Geologic Map, Figure 2. In addition, four seismic refraction traverses were performed to aid in the investigation of the subsurface conditions on-site. Logs of the test pits and details regarding the field investigation as well as the results of the seismic refraction traverses are presented in Appendix A. Details of the laboratory testing and the laboratory test results are presented in Appendix B.

Based on the results of our subsurface exploration and geologic reconnaissance, the site is underlain essentially by four soil and geologic units. From the youngest to oldest, they are: 1) artificial fill, 2) topsoil/residuum, 3) alluvium and/or colluvium, and 4) Cretaceous age granitic rock. A discussion and general description of each unit, with the geologic map symbol in parentheses, is presented below.

1) **Artificial Fill (Qaf):** Undocumented fill soils were mapped along the edge of the property near Enterprise Street and along portions of the west side of the property. In addition, existing fill soils comprised of loose clayey sand with scattered concrete, brick and glass were encountered in Test Pits 1 and 7 to depths of about 3 and 4 feet, respectively. The fill soils appear to be scattered across the site and are considered potentially compressible.

2) **Topsoil/Residuum (not mapped):** The natural hillside and low lying granitic area adjacent to the drainage channel are, in general, mantled by a topsoil/residuum horizon comprised of loose to medium dense clayey sand to depths of less than about 2 feet. Some stiff, sandy clay residual soils are also present on-site as encountered in Test Pit 8 at 1 to 2 feet in depth. These soils are formed in-place through weathering of the underlying parent formational material and generally reflect the composition of the underlying geologic unit. The topsoil/residuum is considered potentially compressible. Based on our laboratory test results and past experience with similar soils, it is our opinion that the sandy surface soils possess a low expansion potential. The more clayey residual materials, however, possess a medium expansion potential. The aerial extent of the more clayey residual soil appears to be limited to the north and northwest portion of the site.

3) **Alluvium and/or Colluvium (Qal/Qcol):** The drainage area is underlain by alluvium and/or colluvium comprised predominantly of very loose to loose clayey sand. Some scattered lenses of firm sandy clay were encountered in Test Pit 6 below a depth of about 4 feet. In addition, loose, poorly graded sand was encountered at depths of 8 to 10-1/2 feet in Test Pit 7. The alluvial soils are water borne sediments associated with the natural as well as the

existing storm drain system of the area. The colluvial soils are generally mass wasting of the more highly weathered parent material along the toe of the hillside as well as the upper portion of the drainage area. The alluvial and/or colluvial soils encountered on-site range in thickness from about 5 to 10-1/2 feet. Based on our subsurface exploration and past experience with similar materials the alluvial and/or colluvial soils on-site are considered potentially compressible.

4) **Granitic Rocks (Kgr):** A basement complex of Cretaceous age granitic rocks underlie the entire site at variable depths. Natural weathering processes have produced a mantle of decomposed granitic rock material of variable thickness over an intermediate zone of fractured granitic rock which is underlain by unweathered crystalline rock. The decomposed granitic rock materials encountered in the test pits were generally comprised of dense to very dense clayey and silty sand. Some dense sandy silt was encountered underlying the alluvial soils in Test Pits 6 and 7 at depths of 7 and 10-1/2 feet, respectively. Refusal was met at depths of 2 feet in Test Pit 11 and at 8 feet in Test Pit 12.

In order to further evaluate the rippability within the proposed cut areas, including the proposed 72-inch storm drain along the northeast side of the site, four seismic refraction traverses were performed with a Nimbus ES-125 engineering seismograph. The results of the seismic refraction traverses are presented in Table A-1.

Based on the results of the seismic traverses, the subsurface underlying the granitic rock area may be divided in three general layers. The upper layer is a soil cover that ranges from approximately 2 to 8 feet in thickness and consists generally of highly decomposed granitic rock materials with a compressional wave (p-wave) velocity ranging from approximately 1,050 to 1,500 feet per second (fps). The underlying intermediate layer consists generally of mildly decomposed and fractured granitic rock with a p-wave velocity ranging from approximately 3,800 to 5,700 fps. No intermediate layer was encountered in Traverse 4 in the north direction. The seismic data indicates that the thickness of the intermediate layer ranges from about 2 to 30 feet. The third layer consists of massive unweathered crystalline granitic rock with p-wave velocities of 6,000 fps or greater.

The seismic refraction data indicates that there are some sloping boundaries between the granitic rock layers. Based on our past experience with similar conditions there are also probably irregularities in the contact between the layers beneath each traverse.

The test pit logs, seismic refraction data and related information depict subsurface conditions only at the specific locations shown on the site plan and geologic map and on the particular dates designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these test pit and traverse locations. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes.

C. Ground Water

Free ground water was encountered in Test Pits 4, 6 and 7 adjacent to the existing creek at depths of 9½, 8, and 9 feet respectively. It must be noted, however, that fluctuations in the level of ground water may occur due to variations in ground surface topography, subsurface stratification, rainfall, and other possible factors which may not have been evident at the time of our field investigation.

D. Seismic Considerations

The San Diego area, as most of California, is located in a seismically active region. The San Diego area has been referred to as the eastern edge of the Southern California Continental Borderland, an extension of the Peninsular Ranges Geomorphic Province. The borderland is part of a broad tectonic boundary between the North American and Pacific Plates. The plate boundary is dominated by a complex system of active major strike-slip (right lateral), northwest trending faults extending from the San Andreas fault, about 70 miles east, to the San Clemente fault, about 50 miles west of the San Diego metropolitan area.

During recent history the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes have been experienced in historic time within the San Diego area. Since the period of instrumentally recorded earthquakes, the San Diego area has experienced scattered "micro seismicity" with Richter magnitudes generally less than 4.0. During June 1985 a series of small earthquakes occurred beneath San Diego Bay; three of these earthquakes had recorded magnitudes of 4.0 to 4.2. In addition, the Oceanside earthquake of July 13, 1986 resulted in a magnitude of 5.3 (Hauksson, 1988) located approximately 26 miles offshore of the City of Oceanside.

Based on a review of some available published information including the County of San Diego Faults and Epicenters Map, there are no faults known to pass through the site. The prominent fault zones generally considered to have the most potential for earthquake damage in the vicinity of the site are the active Elsinore and San Jacinto fault zones mapped approximately 17 and 41 miles northeast of the site, respectively, and the active Rose Canyon and Coronado Bank fault zones mapped approximately 15 and 25 miles southwest of the site, respectively.

Although research on earthquake prediction has greatly increased in recent years, geologists and seismologists have not yet reached the point where they can predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed building will be subject to the effects of at least one moderate to major earthquake during its design life. During such an earthquake, the danger from fault offset through the site is remote, but relatively strong ground shaking is likely to occur.

Strong ground shaking not only can cause structures to shake, but it also has the potential for including other phenomena that can indirectly cause substantial ground movements or other hazards resulting in damage to structures. These phenomena include seismically induced waves such as tsunamis and seiches, inundation due to dam or embankment failure, landsliding, soil liquefaction, lateral spreading, differential compaction, and ground cracking. Available information indicates that the location of and geotechnical conditions at the site are not conducive of these phenomena after the removal and recompaction of alluvial/colluvial soils, as recommended in the following section.

CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering standpoint, it is our opinion that the site is suitable for construction of the proposed building provided the conclusions and recommendations presented in this report are incorporated into the design and construction of the project.

The primary features of concern at the site are: 1) the presence of loose compressible natural and/or fill soils, and 2) the rippability of the granitic rock underlying the site. In order to minimize the possibility of damage to the proposed building and other improvements due to excessive settlements resulting from compression of the loose natural and/or fill soils it will be necessary to remove and recompact these soils. With regard to the rippability of the granitic rock materials, difficult ripping and some probable blasting should be anticipated for the deeper cuts.

Detailed earthwork and foundation recommendations are presented in the following paragraphs. The opinions, conclusions, and recommendations presented in this report are contingent upon Robert Prater Associates being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations.

A. Earthwork

1. Clearing and Stripping

The site should be cleared of all obstructions including buried foundations, abandoned utilities, septic systems (if present) and any miscellaneous trash or debris that may be present at the time of construction. After clearing, the ground surface should be stripped of surface vegetation as well as associated root systems. Holes resulting from the removal of buried obstructions that extend below the proposed finished site grades should be cleared and backfilled with suitable material compacted to the requirements given under Item A.6, "Compaction." Prior to any filling operations, the cleared and stripped materials should be disposed of off-site.

2. Treatment of Existing Fills and Loose Natural Soils

In order to provide suitable foundation support for the proposed building and other improvements, we recommend that all existing fill material and loose natural soils (including topsoils, residuum, and alluvium/colluvium) that remain after the necessary site excavations have been made, be removed and recompacted. The recompaction work should consist of a) removing all existing fill material and loose natural soils down to the underlying granitic materials, b) scarifying, moisture conditioning, and compacting the exposed natural subgrade soils, and c) replacing the materials as compacted structural fill. To provide support for the property line retaining wall along the western boundary, it will be necessary to extend the removal and recompaction 8 feet beyond the wall/property line. Alternatively, the wall footings could be deepened to extend through the alluvium/colluvium to bear on the underlying granitic materials. The areal extent and depth required to remove the fills and loose natural soils should be determined by our representative during the excavation work based on his examination of the soils being exposed. Any unsuitable materials (such as oversize rubble and/or organic matter) should be selectively removed as directed by our representative and disposed of off-site. In particular, the existing rip rap covering the drainage channel at the north end should be removed from the site.

In the area adjacent to the existing wall along the eastern property boundary, the removal and recompaction will require slot cutting perpendicular to the wall to minimize the possibility of undermining and damage to the wall. Details regarding the slot cutting will be determined at the time of construction based on the existing conditions exposed.

3. Excavation

Based on the results of our exploratory test pits, seismic refraction traverses, and our experience with similar materials, it is our opinion that the natural soils including the highly decomposed granitic rock can be excavated utilizing ordinary heavy earthmoving equipment. It should be noted, however, that heavy earthmoving equipment may have difficulty in removing alluvial/colluvial materials in areas with groundwater at the time of construction.

In areas underlain by granitic rock we anticipate a significant amount of difficult ripping and/or blasting will be required to reach the presently proposed grades. Data prepared by the Caterpillar Tractor Company for the D-9 tractor with a single No. 9 hydraulic ripper indicates that weathered rock materials can be ripped to depths at which seismic velocities approach 7,000 fps. Experience in the San Diego area, however, indicates that this velocity usually requires blasting and a more reasonable rippable velocity would be on the order of 5,500 fps. Granitic rock with velocities of 5,700 fps and greater was encountered in the southern portion of the site at depths as shallow as 4-1/2 feet. Based on our seismic refraction traverses it appears that some of the intermediate velocity material may consist of fractured crystalline granitic rock in which difficult ripping may be encountered. The fractured intermediate zone is also likely to produce abundant oversize rock.

For trenching operations, the rippability figures should be adjusted downward. Velocities as low as 3,500 fps may indicate difficult ripping depending on the degree of fracturing and/or weathering of the rock. Fractured rock and even small boulders can be very troublesome in a narrow trench. For example, decomposed granite is easier to dig than fractured granite even when the velocities are similar. In general, however, based on a machine comparable to a Kohering 505, most materials with velocities of approximately 3,800 fps or less should be rippable, over 4,300 fps non-rippable, and marginal in between. In a narrow trench, a condition of many boulders can be almost as troublesome as solid rock. Accordingly, the above figures should be used with discretion. It should also be noted that excavation in the intermediate layer material will be difficult if not impossible with ordinary light backhoe equipment. Contractors should not, however, be relieved of making their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

4. Subgrade Preparation

After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soil in those areas to receive fill, building improvements and/or pavements should be scarified to a depth of 8 inches, moisture conditioned, and compacted to the requirements of Item A.6, "Compaction." In areas where dense undisturbed decomposed granitic rock is exposed at the subgrade surface, the subgrade need not be scarified and compacted.

5. Material for Fill

All on-site soils with an organic content of less than 3 percent by volume are in general suitable for reuse as fill. Fill material should not contain rocks or lumps over 6 inches in greatest dimension, not more than 15 percent larger than 2-1/2 inches, and no more than 25 percent larger than 1/4-inch. Oversize rock encountered in the excavations more than 6 inches in greatest dimension should be selectively removed and either disposed of off-site, used for non-fill purposes (such as landscaping), or buried in non-building areas. In general, this can be accomplished by accumulating all the oversize rock, spreading in a single layer in the excavation, flooding on-site decomposed granite with a minimum sand equivalent of 30 into the voids, and thoroughly track-rolling the rock/sand mass. The rock/sand mass should be at least 5 feet below finish subgrade or deeper than the deepest utility, whichever is greater. Detailed recommendations regarding the placement of oversize rock on-site should be developed at the time of construction, depending on the actual conditions encountered. Any potentially expansive clayey soils (expansion index greater than 20) removed from the required excavations should only be reused as fill below a depth of 12 inches below the finished site grades.

Imported fill material should be a low-expansion potential, granular soil with a plasticity index of 12 or less and an R(Resistance)-Value of 30 or better. In general, all imported fill material should have an Expansion Index (ASTM D 4829-95) of 50 or less. The upper 12 inches from the subgrade level in the building area, however, should have an expansion index of 20 or less. In addition, the import fill should not contain rocks or lumps over 6 inches in greatest dimension, not more than 15 percent larger than 2-1/2 inches, and no more than 25 percent larger than 1/4-inch.

It should be noted that depending on the time of the year that construction is undertaken, the soils removed from some of the excavations may be in a saturated or near-saturated condition. If this occurs it will be necessary to allow the materials to dry sufficiently to allow for proper compaction before they can be reused as fill or backfill. Consideration should be given to install the new 72-inch storm drain prior to the removal and recompaction of the loose alluvial soils in the existing drainage area.

6. Compaction

All structural fills should be compacted to a minimum degree of compaction of 90 percent based on ASTM Test Designation D 1557-91. The upper 6 inches of subgrade soil beneath pavements should be compacted to a minimum degree of compaction of 95 percent just prior to placement of the aggregate base layer. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: 1) aerating the fill if it is too wet, or 2) moistening the fill with water if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture.

7. Temporary Construction Slopes

Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary cut-slopes for construction of the proposed retaining walls should be safe against mass instability at an inclination of 1-1/2 (horizontal) to 1 (vertical) in areas underlain by existing fill soils and/or loose natural soils and 3/4 to 1 in areas underlain by decomposed granitic rock. In areas where fractured and/or massive crystalline rock is exposed, the temporary slope could be near vertical with a height not exceeding 10 feet. Some localized sloughing or raveling of the soils exposed on the slopes, however, may occur. Since the stability of temporary construction slopes will depend largely on the contractor's activities and safety precautions (storage and equipment loadings near the tops of cut-slopes, surface drainage provisions, etc.) it should be the contractor's responsibility to establish and maintain all temporary construction slopes at a safe inclination appropriate to his methods of operation.

In light of the existing fill and loose natural soils adjacent to the southeast corner of the site as well as the proposed property line wall along the western portion of the site, it may be necessary to provide temporary shoring. Details regarding temporary shoring can be provided when additional information is available regarding any limitations on temporary slopes outside of the right of way along the south and west sides of the property.

8. Permanent Slopes

Based on our subsurface investigation work, laboratory test results, and engineering analysis, we recommend that permanent cut and fill slopes be constructed to an inclination no steeper than 2 (horizontal) to 1 (vertical).

Fill slopes should be constructed so as to assure that the recommended minimum degree of compaction is attained out to the finished slope face. Construction of the outer edges of the fills should in general be accomplished by operation of the compaction equipment parallel and up to the edge of the fill with the grading surface sloping down and away from the slope edge. We recommend that a sheepsfoot roller or segmented wheel compactor be used to compact the soils at the outer edge of fills adjacent to slopes. The slope face should be thoroughly backrolled with a sheepsfoot roller in two-foot vertical increments as the fill is raised. In addition, placement of fill near the tops of slopes should be carried out in such a manner as to assure that loose, uncompacted soils are not sloughed over the tops and allowed to accumulate on the slope face.

The on-site sandy soils will be susceptible to erosion. Therefore, the project plans and specifications should contain all necessary design features and construction requirements to prevent erosion of the on-site soils both during and after construction. Slopes and other exposed ground surfaces should be appropriately planted with a protective ground cover.

It should be the grading contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It should be made the contractor's responsibility to maintain slopes in their as-graded form until all slopes, berms and associated drainage devices are in satisfactory compliance with the project plans and specifications.

9. Trench Backfill

Pipeline trenches should be backfilled with compacted fill. Backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means. In pavement areas, that portion of the trench backfill within the pavement section should conform to the material and compaction requirements of the adjacent pavement section.

Our experience has shown that backfills for even shallow, narrow trenches, such as for irrigation and electrical lines, which are not properly compacted can result in problems, particularly with respect to shallow ground water accumulation and migration.

Based on our past experience with similar materials, excavation in the granitic materials may generate fragmented, oversize material. It is our opinion that these materials may be difficult to compact in trenches with light compaction equipment. In addition, if these materials are used they should be broken down as necessary to meet the size requirements presented in Item A.5.

10. Drainage

Positive surface gradients should be provided adjacent to the building, and roof gutters and downspouts should be installed so as to direct water away from foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed, especially adjacent to the building or on pavements.

11. Subsurface Drainage

In light of the existing ground water and the recommendations for removal and recompaction of the loose alluvial soils, we recommend that a subsurface drain be installed below the existing drainage area on-site. The approximate location of the subsurface drain should generally be along the existing alignment of the channel. The subdrain should be connected to the proposed 72-inch storm drain at the northwest end of the site. Details of the subsurface drain installation are illustrated on Figure 3.

Based on our subsurface exploration, it appears that the ground water may be trapped along the southeast end. Accordingly, we recommend that consideration be given to performing the remedial grading work along the northwest end of the channel and the partial installation of the subdrain prior to the remedial grading and the extension of the subdrain along the southeast end of the site.

12. Construction Observation

Variations in soil and geologic conditions are possible and may be encountered during construction. In order to permit correlation between the preliminary soil and geologic data and the actual conditions encountered during construction and so as to aid in evaluating conformance with the plans and specifications as originally contemplated, it is essential that we be retained to perform on-site review during the course of construction.

All earthwork should be performed under the observation of our representative to aid in proper site preparation, selection of satisfactory fill materials, as well as placement and compaction of the fills. Sufficient notification prior to earthwork operations is essential to make certain that the work will be properly observed.

B. Foundations

1. Footings

We recommend that the proposed building be supported on conventional, individual-spread and/or continuous footing foundations bearing on undisturbed natural soil and/or well-compacted fill material. All footings should be founded at least 18 inches below the lowest adjacent finished grade. Footings located adjacent to the tops of slopes should be extended sufficiently deep so as to provide at least 8 feet of horizontal cover or 1-1/2 times the width of the footing, whichever is greater, between the slope face and outside edge of the footing at the footing bearing level. Footings located adjacent to utility trenches, including the new 72-inch storm drain, should have their bearing surfaces situated below an imaginary 1-1/2 to 1 plane projected upward from the bottom edge of the adjacent utility trench.

At the recommended depths footings may be designed for allowable bearing pressures of 3,500 pounds per square foot (psf) for combined dead and live loads and 4,700 psf for all loads, including wind or seismic. The footings should, however, have a minimum width of 12 inches. All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of two No. 4 top and two No. 4 bottom reinforcing bars be provided in the footings. In order for us to offer an opinion whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

2. Slabs-On-Grade

Concrete slabs-on-grade may be supported directly on low-expansion potential compacted fill soil and/or firm undisturbed low-expansion potential natural soil as recommended in Item A.5. Slab reinforcing as well as slab thickness should be designed in accordance with the anticipated use of and loading on the slab. As a minimum, however, we recommend that the slabs have a thickness of 5 inches and be reinforced with No. 4 reinforcing bars at 24-inches on center both ways to minimize hairline cracking of the slabs due to concrete shrinkage.

In areas where moisture-sensitive floor coverings are to be utilized and in other areas where floor dampness would be undesirable, we recommend that an impermeable membrane be provided beneath the slabs. The membrane should be covered with 2 inches of sand (minimum sand equivalent of 50) to protect it during construction. The sand should be lightly moistened just prior to placing the concrete. In addition, if the subgrade on which the membrane will be placed is rocky, we recommend a 2-inch sand layer be placed below the membrane.

3. Retaining Walls

Retaining walls must be designed to resist lateral earth pressures and any additional lateral pressures caused by surcharge loads on the adjoining retained surface. We recommend that unrestrained (cantilever) walls with level backfill be designed for an equivalent fluid pressure of 35 pounds per cubic foot (pcf). We recommend that restrained walls with level backfill be designed for an equivalent fluid pressure of 35 pcf plus an additional uniform lateral pressure of 8H pounds per square foot where H = the height of backfill above the top of the wall footing in feet. Unrestrained walls with up to 2 (horizontal) to 1 (vertical) sloping backfills should be designed for an equivalent fluid pressure of 50 pcf. Wherever walls will be subjected to surcharge loads, they should also be designed for an additional uniform lateral pressure equal to one-third the anticipated surcharge pressure in the case of unrestrained walls and one-half the anticipated surcharge pressure in the case of restrained walls.

The preceding design pressures assume that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. Adequate drainage may be provided by means of weepholes with permeable filter material installed behind the walls or by means of a system of subdrains. If gravel is used, it should be completely wrapped in a suitable filter fabric such as Mirafi 140N or equivalent.

Backfill placed behind the walls should consist of sandy materials and be compacted to a minimum degree of compaction of 90 percent using light compaction equipment. If heavy equipment is used, the walls should be appropriately temporarily braced.

Retaining walls should be supported on footing foundations designed in accordance with the recommendations presented previously under Item B.1., "Footings." Lateral load resistance for the walls can be developed in accordance with the recommendations presented under Item B.4., "Lateral Loads."

4. Lateral Loads

Lateral load resistance for structures supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot acting against the foundations may be used in design provided the footings are poured neat against the adjacent undisturbed native soils and/or compacted fill materials. Passive resistance against unreworked alluvium/colluvium, however, should be reduced to 150 pcf. These lateral resistance values assume a level surface in front of the footing for a minimum distance of 3 times the embedment depth of the footing and any shear keys and are based on a factor of safety of 1.5.

C. Corrosion Potential

Laboratory pH, resistivity and sulfate tests were performed by D-TEK on a sample representative of the on-site soils to evaluate their corrosion potential on metal pipes as well as degradation of concrete from sulfates. Details regarding the tests and the test results are included in Appendix B.

Based on criteria developed by the State of California, Department of Public Works, Division of Highways and presented in Test Method No. Calif. 643-C, we have utilized the pH and resistivity data to estimate a service life of 35 years for 16 gauge metal piping. Based on this estimate, it is our opinion that the on-site materials have a mild to moderate potential for corrosion attack on metal piping.

The sulphate content test indicates 89 parts per million (ppm). Based on Table 19-A-3 of the Uniform Building Code, 1994 edition, this value indicates a low potential for sulfate attack on concrete. Table 19-A-3 indicates the use of Type II cement is appropriate at the site. It should be noted that the source of proposed import soils is currently unknown. At the time of construction we recommend that additional corrosivity testing be done on the imported soil.

D. Concrete Pavements

It is our understanding that the south, east, and west sides of the property will include concrete pavements. We recommend that the PCC thickness be 4-1/2 inches in areas subject only to automobile and very light truck (such as pick-ups) traffic. Areas subject to up to 27 heavy two-axle trucks per week should have a PCC thickness of 7-1/2 inches. The upper 8 inches of the subgrade below the slab should be compacted to a minimum degree of compaction of 95 percent just prior to paving. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 1994 Edition, for Class 560-C-3250.

In order to control shrinkage cracking, we recommend that saw-cut, weakened plan joints be provided at about 15-foot centers both ways. The pavement slab should be saw-cut no more than 24 hours after the placement of the concrete. The depth of the joint should be 1/4 of the slab thickness and its width should not exceed 0.02 feet.

E. Asphalt Concrete Pavements

Based on the results of our exploratory test pits and laboratory tests, as well as an assumed R-value of 30, we anticipate that pavement sections for the proposed development will be on the order of 2 inches of asphalt concrete on 5 ½ inches of aggregate base for parking stalls and minor traffic channels, 2-1/2 inches on 6-1/2 inches for major automobile traffic channels and pavement areas subject to no more than 4 heavy trucks per week, and 3 inches on 8-1/2 inches for pavements subject to up to 27 heavy two-axle trucks per week, such as truck access drives and truck loading areas. Actual pavement section recommendations should be based on R(Resistance)-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the mass grading operations.

Asphalt concrete should consist of Type III-B3-AR-4000 conforming to the Standard Specifications for Public Works Construction, 1994 Edition (Standard Specifications), Section 400-4 and be placed in accordance with Section 302-5. Aggregate base should conform to the requirements for Crushed Aggregate Base or Crushed Miscellaneous Base in Section 200-2 of the Standard Specifications. The upper 6 inches of the pavement subgrade soil as well as the aggregate base layer should be compacted to a minimum degree of compaction of 95 percent. Preparation of the subgrade and placement of the asphalt concrete and base materials should be performed under the observation of our representative.

F. Limitations

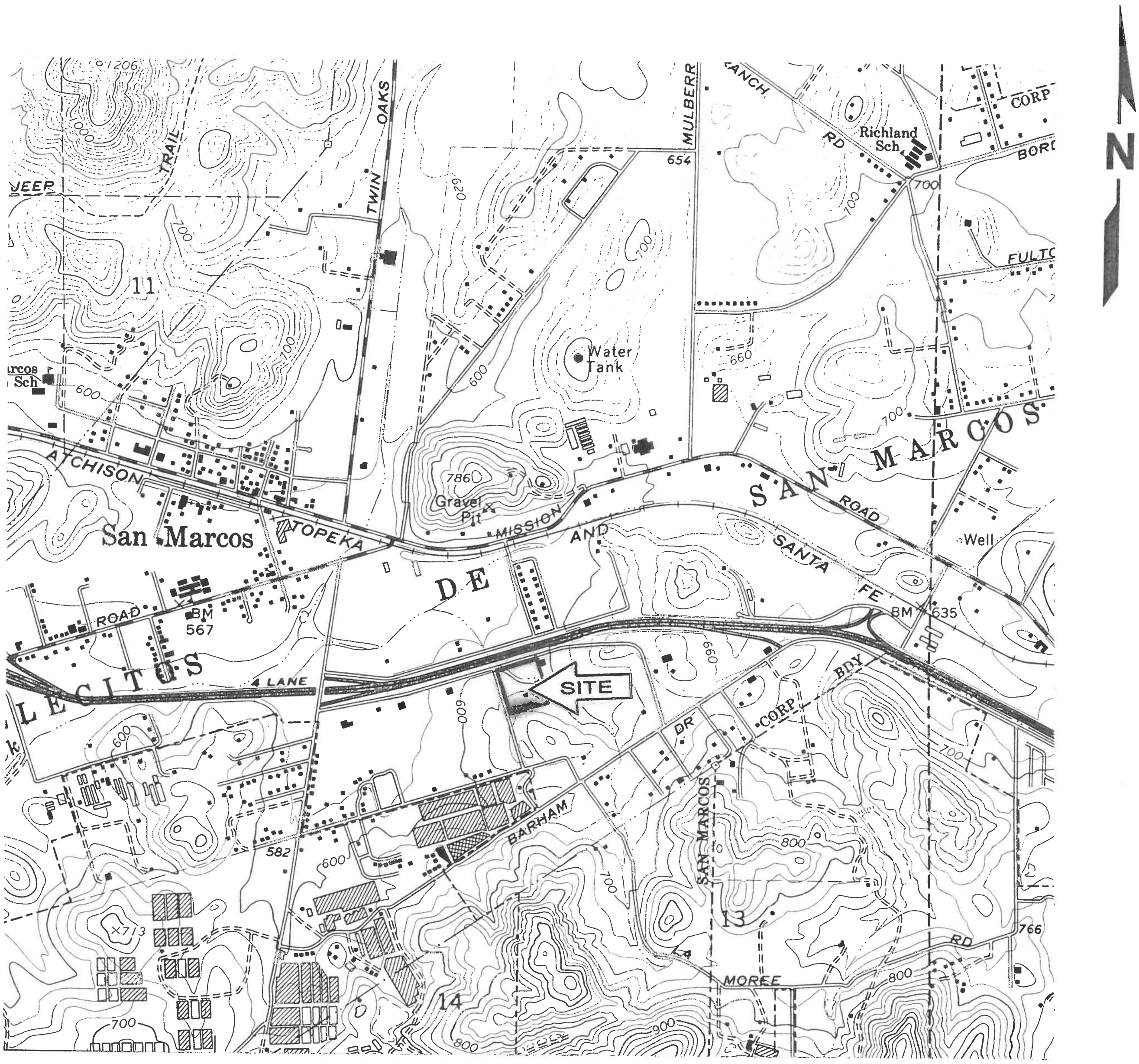
The recommendations presented in this report are specifically for the proposed construction of the light industrial building at 323 E. Carmel Street in San Marcos, California. Our office should be notified of any changes in the proposed development for further recommendations, if necessary, based on our review. As grading and foundation plans are developed we should be retained to review them for conformance to our recommendations. We also recommend that our office review any other plans which may affect the geotechnical conditions on-site such as landscaping, irrigation, plumbing, or other similar type plans. We should also be retained to review any future development plans including building additions in order to develop specific recommendations for proposed construction. Additional subsurface exploration could be required.

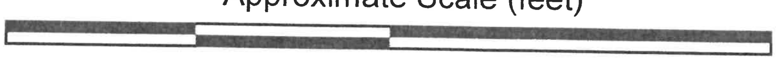
The conclusions and recommendations presented in this report are based on our evaluation of the subsurface materials encountered on-site, our understanding of the proposed development, and our general experience in the geotechnical field. If significant variations in the geotechnical conditions are encountered during construction our office should be consulted for further recommendations.

The satisfactory performance of the site is also dependent on proper maintenance. Proper maintenance includes, but is not limited to, providing and maintaining good drainage away from structures and slopes, establishing good vegetation cover on slopes, and avoiding excess irrigation.

Significant variations in geotechnical conditions may occur with the passage of time due to natural processes or the works of man on this or adjacent properties. In addition, changes in the state of the practice may occur as a result of legislation or the broadening of knowledge. Accordingly, the conclusions and recommendations presented in this report should be reviewed and updated, if necessary, after a period of two years.

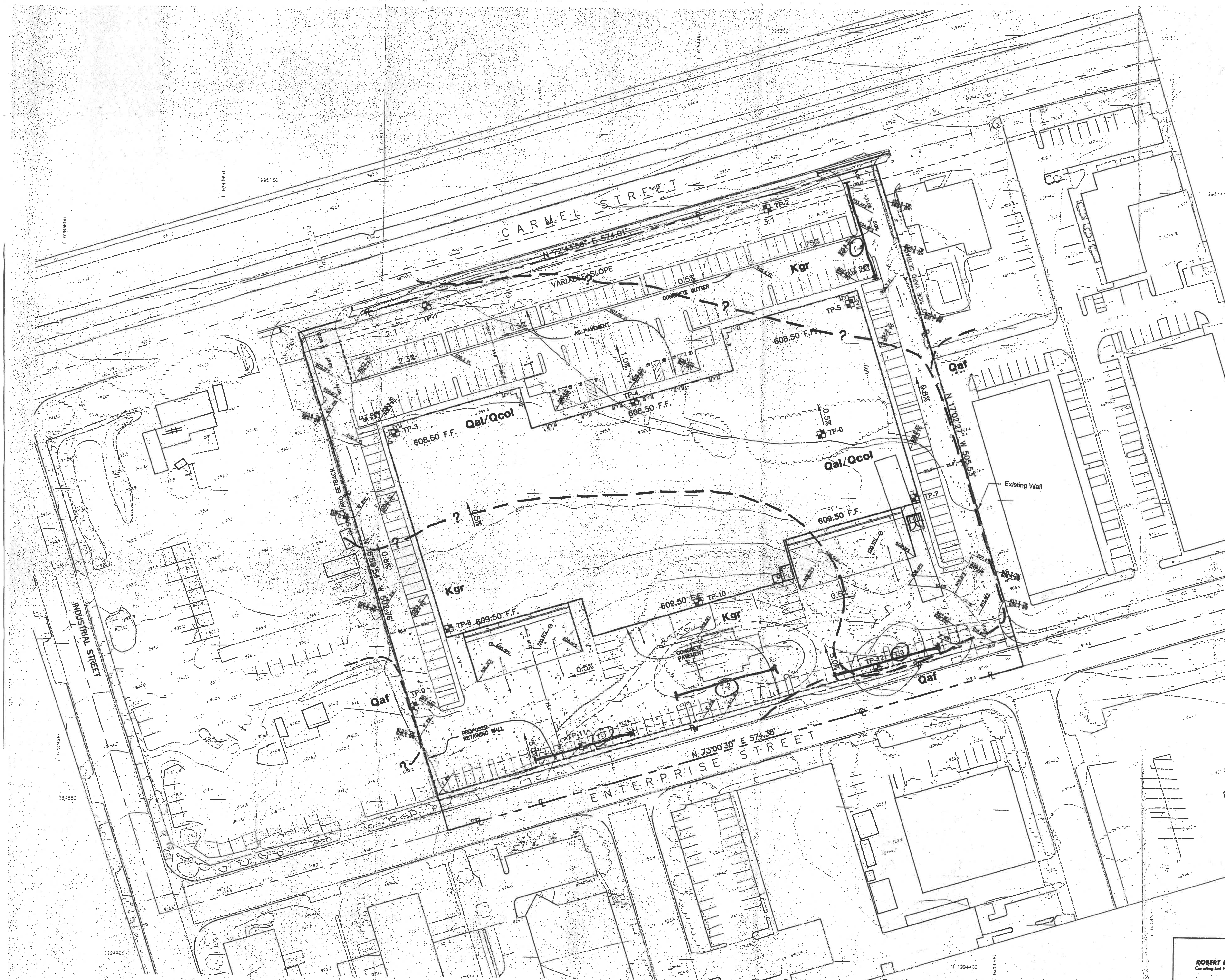
Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either express or implied.





Base: USGS topographic map titled "San Marcos, California," dated 1968

ROBERT PRATER ASSOCIATES *Consulting Soil, Foundation, & Geological Engineers*	VICINITY MAP		
	LIGHT INDUSTRIAL San Marcos, California		
	PROJECT NO. 546-6	DATE March 1999	FIGURE 1



LEGEND

Qaf	Artificial Fill
Qal/Qcol	Undifferentiated alluvium and/or colluvium
Kgr	Granitic Rock
?	Indicates approximate location of geologic contact; queried where questionable
TP-1	Indicates approximate location of exploratory test pit
T-1	Indicates approximate location and orientation of seismic refraction traverse

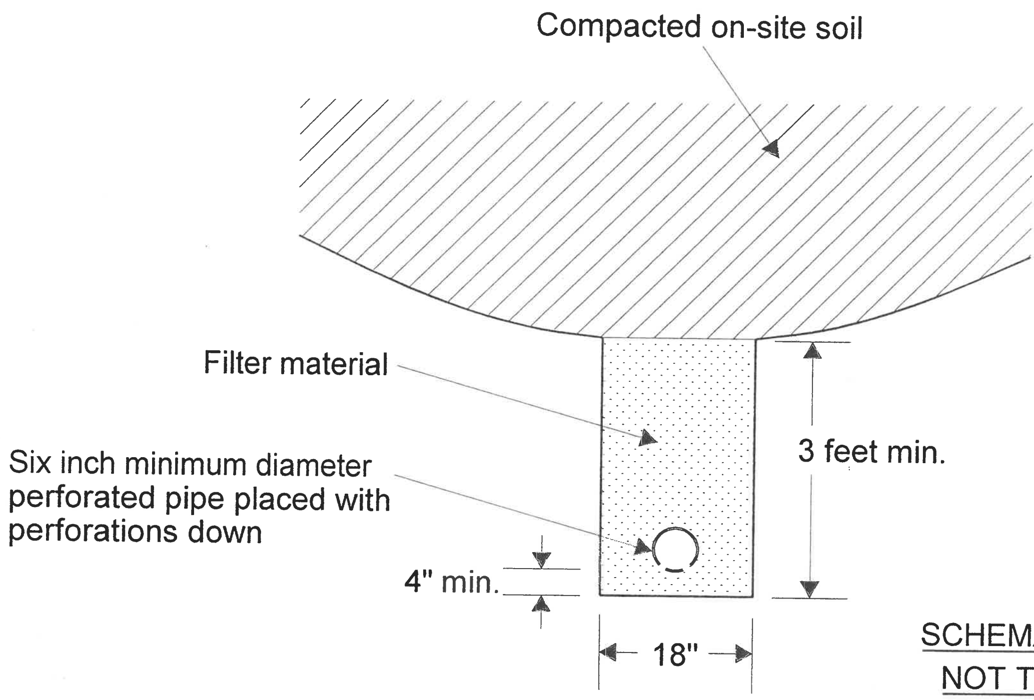
Approximate Scale (feet)

0 50 100 200

Base: An undated, untitled grading plan provided by Construction, Testing & Engineering, Inc.

ROBERT PRATER ASSOCIATES
Consulting Soil Foundation & Geological Engineers

SITE PLAN AND GEOLOGIC MAP		
LIGHT INDUSTRIAL San Marcos, California		
PROJECT NO.	DATE	Figure 2
546-6	March 1999	



SCHEMATIC ONLY
NOT TO SCALE

NOTES:

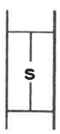
1. Filter material shall be comprised of washed concrete sand conforming to ASTM C-33.
2. Perforated drain pipe shall consist of ABS or PVC material construction. For filling heights up to 35 feet SDR 35 pipe may be used.
3. Perforated drain pipe to have a minimum drainage gradient of 0.5 percent.
4. Upstream end of subdrain pipe shall be capped and subdrain trench covered by at least 10 feet of soil.
5. Subdrain pipe shall be connected to the 72-inch RCP storm drain.
6. Location and details of subsurface drain installation are subject to revision by the soil engineer in the field during construction.

ROBERT PRATER ASSOCIATES *Consulting Soil, Foundation, & Geological Engineers*	SUBSURFACE DRAIN DETAIL		
	LIGHT INDUSTRIAL San Marcos, California		
	PROJECT NO. 546-6	DATE March 1999	FIGURE 3

APPENDIX A
FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using a Case 580D backhoe. Twelve exploratory test pits were excavated on February 24, 1999, and four seismic refraction traverses were performed on March 3, 1999, at the approximate locations shown on the Site Plan and Geologic Map, Figure 2. The soils encountered in the test pits were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2487). Logs of the test pits as well as a key for soil classification are included as part of this appendix. In addition, results of the seismic refraction traverses are presented in Table A-1. The test pit and seismic refraction traverse locations shown on the site plan and geologic map were estimated from an undated, untitled grading plan provided by Construction Testing & Engineering, Inc.

Representative samples were obtained from the exploratory test pits at selected depths appropriate to the investigation. All samples were returned to our laboratory for evaluation and testing. Test pit log notations for jar and sack samples taken from test pit spoils are indicated below.



Indicates sack sample taken from test pit spoils

"X" Indicates jar sample taken from test pit sidewall

The test pit logs and seismic refraction traverses show our interpretation of the subsurface conditions on the dates and at the locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little of no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, non-plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sand, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS 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, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

DEFINITION OF TERMS

	U.S. STANDARD SERIES SIEVE: 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS: 3/4"	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH**	BLOWS/FOOT*
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

**Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

KEY TO EXPLORATORY TEST PIT LOGS
Unified Soil Classification System (ASTM D-2487)

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	FIGURE A-1
546-6	March 1999	

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 590' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (fill)		dark brown-brown	loose	SC		S				
					1	X		12		
Scattered glass and brick fragments near contact FILL					2					
CLAYEY-SILTY SAND (decomposed granitic rock)		grayish brown	very dense	SC-SM	3	X				
Bottom of Test Pit @ 3-1/2 feet					4					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO. 1
546-6	March 1999	

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 596' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (topsoil and/or residuum)		brown	loose	SC	1	X		8		
CLAYEY SAND (decomposed granitic rock)		grayish brown	dense	SC	2	X				
Bottom of Test Pit @ 3 feet					3					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	2

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 595' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (colluvium)		brown	loose-medium dense	SC	1	S				
					2	X				
						X				
Scattered roots					3					
					4	X				
SILTY SAND (decomposed granitic rock)		gray-grayish brown	dense	SM	5					
Bottom of Test Pit @ 6 feet					6					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO. 3
546-6	March 1999	

EQUIPMENT	Case 580D Backhoe	SURFACE ELEVATION 594' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER	9-1/2' (see note)	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (alluvium)		dark brown-brown	loose	SC	0–2	S				
					9	X		16		
	▽				9-1/2					
SILTY SAND (decomposed granitic rock)		grayish brown	dense	SM	10	X				
Bottom of Test Pit @ 10 feet										

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	4

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 599' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION											
DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (topsoil)		dark brown	loose	SC		X		7		
CLAYEY SAND (decomposed granitic rock)		brown-reddish brown	dense-very dense	SC	1 2	X				
Bottom of Test Pit @ 3 feet					3					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO. 5
546-6	March 1999	

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 598' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER 8' (see note)	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (alluvium)		dark brown/ grayish brown	loose-medium dense	SC	1					
					2	X		10		
					3					
Scattered lenses of sandy clay					4	X		14		
					5					
					6					
SANDY SILT (highly decomposed granitic rock)	▽	grayish brown	med. dense-dense	ML	7	X X				
Bottom of Test Pit @ 8 feet					8					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	6

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 603' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER 9' (see note)	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND with scattered wood and concrete (fill)		dark brown-brown	loose	SC	0–4					
					2	X		11		
FILL										
CLAYEY SAND (alluvium)		brown	loose	SC	4–8					
					6	X		14		
POORLY GRADED SAND (alluvium)	▽	brown	loose	SP	8–10.5					
SANDY SILT (highly decomposed granitic rock)		olive/gray-brown	medium dense-dense	ML	10.5–11.5	X		26		
Bottom of Test Pit @ 11-1/2 feet										

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	7

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 604' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (topsoil)		dark brown	loose	SC	1	X				
SANDY CLAY (residuum)		brown	stiff	CL	2	X		14		
CLAYEY SAND (decomposed granitic rock)		reddish/ yellowish brown	dense	SC	3	X				
Bottom of Test Pit @ 3 feet										

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	8

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 609' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (topsoil)		dark brown-black	loose	SC	1	S X		14		
					2					
CLAYEY SAND (decomposed granitic rock)		reddish/ yellowish brown	dense	SC	3	X				
Bottom of Test Pit @ 4 feet					4					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	9

EQUIPMENT	Case 580D Backhoe	SURFACE ELEVATION	615' (approx.)	LOGGED BY	JB
DEPTH TO GROUNDWATER	None	Test pit was excavated with a 24-inch bucket on 2/24/99			

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (topsoil)		dark brown	loose	SC						
CLAYEY-SILTY SAND (decomposed granitic rock)		grayish brown	dense-very dense	SC-SM	1	X				
					2	X				
					3					
					4					
					5					
					6					
Bottom of Test Pit @ 7 feet					7					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	10

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 622' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND CLASSIFICATION					DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
DESCRIPTION AND REMARKS	SYM-BOL	COLOR	CONSIST.	SOIL TYPE						
CLAYEY SAND with gravel (fill) FILL		dark brown	loose	SC	1					
SILTY SAND (decomposed granitic rock)		grayish brown	dense	SM	2	X				
Bottom of Test Pit @ 2-1/2 feet Met refusal on granitic rock					3					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

ROBERT PRATER ASSOCIATES

Consulting Soil, Foundation, & Geological Engineers

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	11

EQUIPMENT Case 580D Backhoe	SURFACE ELEVATION 616' (approx.)	LOGGED BY JB
DEPTH TO GROUNDWATER None	Test pit was excavated with a 24-inch bucket on 2/24/99	

DESCRIPTION AND REMARKS	SYMBOL	COLOR	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLER	PENETRATION RESISTANCE (BLOWS/FT.)	WATER CONTENT(%)	SHEAR STRENGTH BY TORVANE (KSF)	DRY UNIT WEIGHT (PCF)
CLAYEY SAND (colluvium) Porous		reddish brown-brown	loose-medium dense	SC	1					
					2	S				
					3	X		11		
					4					
					5					
					6					
CLAYEY SAND (decomposed granitic rock)		grayish brown	dense-very dense	SC	7	X				
Bottom of Test Pit @ 8 feet Met refusal on granitic rock					8					

Note: The stratification lines represent the approximate boundary between material types and the transition may be gradual.

EXPLORATORY TEST PIT LOG

LIGHT INDUSTRIAL
San Marcos, California

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

PROJECT NO.	DATE	TEST PIT NO.
546-6	March 1999	12

TABLE A-1

SEISMIC TRAVERSE RESULTS

Traverse	Seismic Velocity (feet per second)	Depth Interpretation (feet)
T-1E	1,500	0 - 4
	5,700	4-30
	20,000	>30
T-1W	1,050	0 - 2
	4,700	2 - 29
	20,000	>29
T-2E	2,500	0 - 6
	3,800	6-24
	8,000	>24
T-2W	1,300	0 - 5
	5,500	5-23
	7,000 (assumed)	>23
T-3E	1,250	0 - 8
	5,700	8-21
	7,000 (assumed)	>21
T-3W	1,300	0 - 7
	5,050	7-24
	7,000 (assumed)	>24
T-4S	1,500	0 - 7
	4,300	7-27
	12,000	>27
T-4N	1,350	0 - 7
	6,200	>7

Notes:

1) Traverses denoted by line number corresponding to designation on Figure 2. Seismic measurements for each traverse were run in opposite directions. The letter following each traverse number indicates the compass heading of the run (i.e., north, south, east, west).

2) Assumed seismic velocities used to estimate the minimum depth to crystalline rock when high velocity layer not encountered.

APPENDIX B
LABORATORY TESTING

The natural water content and dry unit weight was determined on selected samples and is recorded on the test pit logs at the appropriate sample depths.

Nine No. 200 sieve tests were performed on selected samples of the subsurface soils to aid in classifying the soils according to the Unified Soil Classification System. The results of these tests are presented in Table B-1.

One laboratory compaction test (ASTM D 1557-91) was performed on a representative bulk sample of the on-site soils. The results of the test are presented on Figure B-1.

One laboratory direct shear test was performed on a sample remolded to approximately 90 percent of the laboratory maximum density. The sample was sheared at a constant rate under various surcharge pressures; failure was taken at the peak shear stress. The results of the test are presented on Figure B-2.

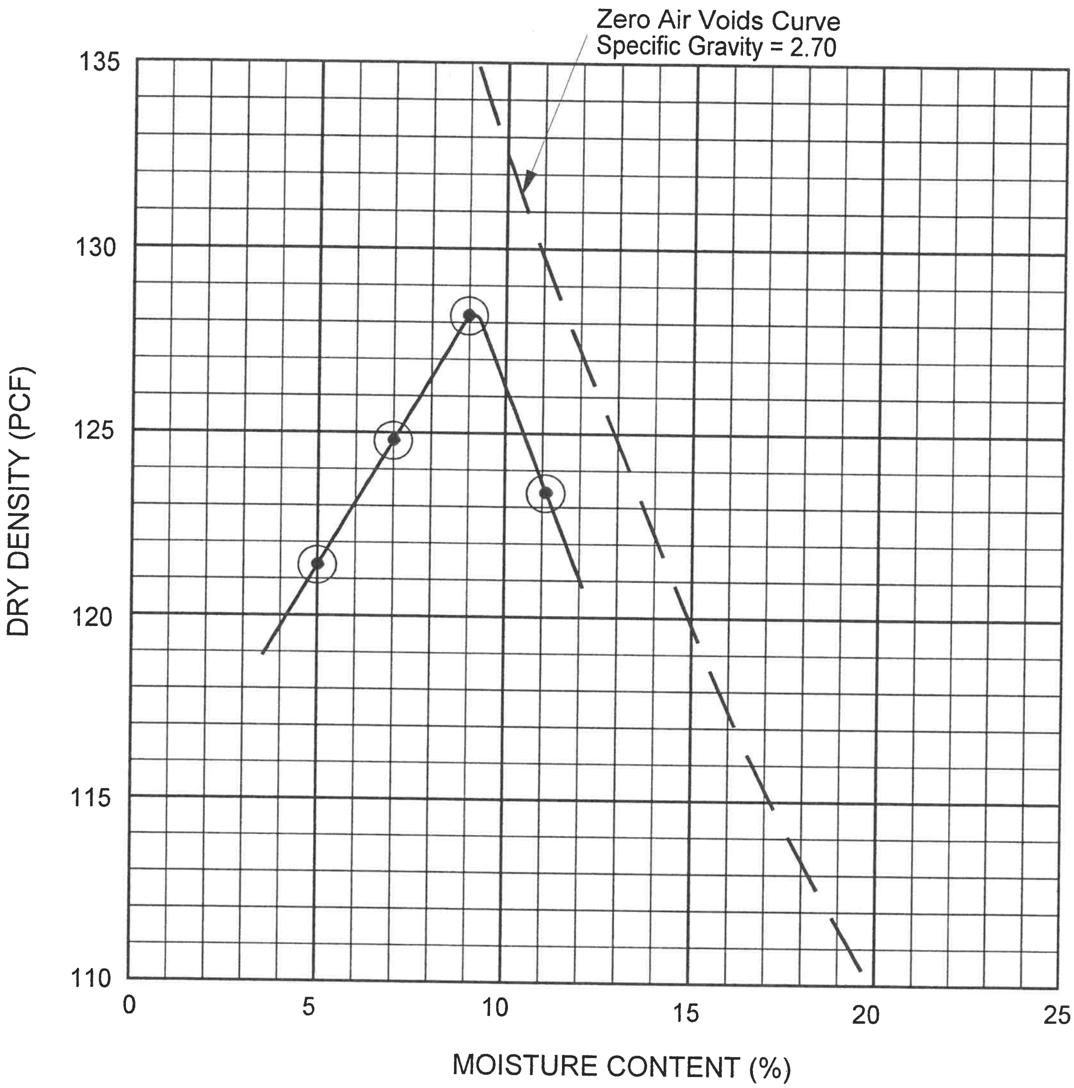
Laboratory pH, resistivity and sulfate tests were performed on a selected sample of the on-site soils to aid in evaluation of the corrosivity of these soils. The testing was performed by D-TEK. The test results are presented at the end of this appendix.

TABLE B-1

RESULTS OF NO. 200 SIEVE TESTS

Exploratory Boring No.	Sample Depth (Feet)	Sample Description	Percent Passing No. 200 Sieve
1	1 ½	CLAYEY SAND (SC), dark brown	38
4	8 ½	CLAYEY SAND (SC), dark brown	39
5	½	CLAYEY SAND (SC), dark brown	40
6	4	SANDY CLAY (CL), grayish brown	52
7	6	CLAYEY SAND (SC), brown	46
7	10 ½	SANDY SILT (ML), olive brown	78
8	2	SANDY CLAY (CL), brown	58
9	1	CLAYEY SAND (SC), dark brown	34
12	0 - 4	CLAYEY SAND (SC), brown	35

BORING NO.	DEPTH (FT.)	SAMPLE DESCRIPTION	SPECIFIC GRAVITY
12	0 - 4	CLAYEY SAND (SC), brown	---



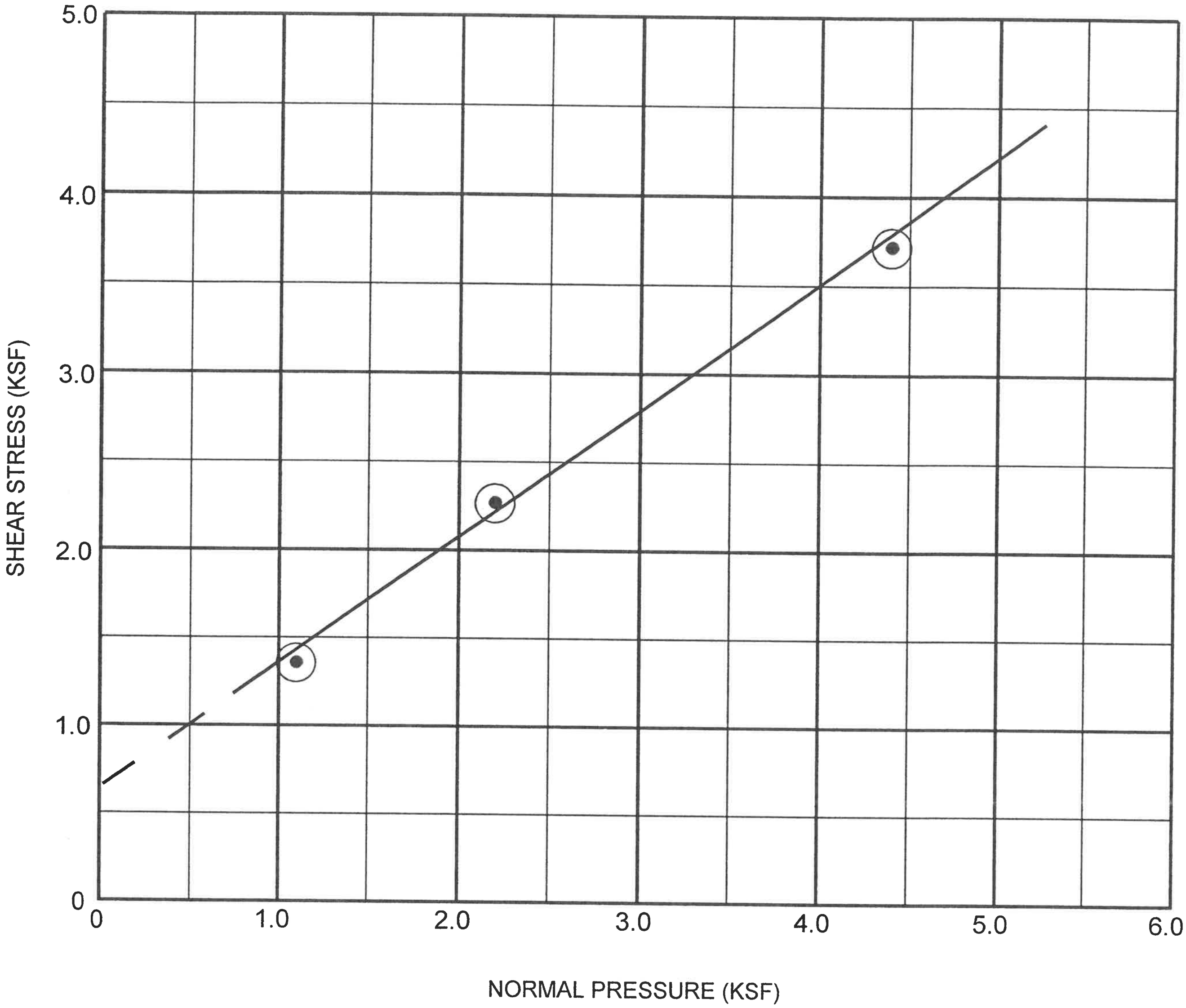
MAXIMUM DRY DENSITY (PCF)	128.2
OPTIMUM WATER CONTENT (%)	9.0
TEST DESIGNATION	ASTM D 1557-91

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geologic Engineers

COMPACTION TEST RESULTS

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	FIGURE B-1
546-6	March 1999	



SAMPLE DATA	
BORING NO.: 12	DEPTH (FT.) : 0-4
DESCRIPTION: CLAYEY SAND (SC), brown	

TEST RESULTS
APPARENT COHESION: 0.65 (ksf)
APPARENT ANGLE OF INTERNAL FRICTION: 36 °

TEST DATA			
TEST NUMBER	1	2	3
NORMAL PRESSURE (KSF)	1.10	2.20	4.40
SHEAR STRENGTH (KSF)	1.36	2.27	3.72
INITIAL H2O CONTENT (%)	7.9	8.2	8.2
FINAL H2O CONTENT (%)	- - - -	- - - -	- - - -
INITIAL DRY DENSITY (PCF)	115.0	114.5	113.9
FINAL DRY DENSITY (PCF)	- - - -	- - - -	- - - -
STRAIN RATE: 0.01 inches per minute			

Note: Test was performed on a sample remolded to approximately 90 percent of laboratory maximum density.

ROBERT PRATER ASSOCIATES
Consulting Soil, Foundation, & Geological Engineers

DIRECT SHEAR TEST RESULTS

LIGHT INDUSTRIAL
San Marcos, California

PROJECT NO.	DATE	FIGURE B-2
546-6	March 1999	

D-TEK Analytical Laboratories, Inc.
9020 Kenamar Drive, Suite 205
San Diego, CA 92121
WWW.PAGEKEEPER.COM/CA/DTEK
(619) 566-4540 FAX (619) 566-4542

ROBERT PRATER & ASSOCIATES
10505 Roselle Street
San Diego, CA 92121

Attn: Mr. Jim Barton

Date of Report:	3/5/99
Sampling Date:	2/24/99
Date Sample Received:	3/2/99
Date Analyzed:	March 2, 4 and 5, 1999
Analyzed By:	EA and JV
Sample Type:	Soil
Project Name:	Light Industrial/546-6
Log Number:	99-0468

The sample(s) were analyzed with EPA methodology or equivalent methods as specified on the attached "Analyses Results" report. The symbol for "less than" indicates a value below the reportable detection limit.

The results of these analyses and the quality control data are enclosed.

Ellen Atienza
Operations Manager

D-TEK Analytical Laboratories, Inc.
9020 Kenamar Drive, Suite 205
San Diego, CA 92121
WWW.PAGEKEEPER.COM/CA/DTEK
(619) 566-4540 FAX (619) 566-4542

ROBERT PRATER & ASSOCIATES
10505 Roselle Street
San Diego, CA 92121

Attn: Mr. Jim Barton

Date of Report:	3/5/99
Sampling Date:	2/24/99
Date Sample Received:	3/2/99
Date Analyzed:	March 2, 4 and 5, 1999
Analyzed By:	EA and JV
Sample Type:	Soil
Project Name:	Light Industrial/546-6
Log Number:	99-0468

ANALYSES RESULTS

Analysis	Prep/Analysis Method	Units	Log Number : Sample ID:	99-0468 EB-12 @ 0-4
pH	EPA 9045 C			6.61
Resistivity	Cal Test 643	Ohm-cm		12500
Sulfate	EPA 9038/TEX-620-J	mg/kg		88.4

Ellen Atienza
Operations Manager

D-TEK Analytical Laboratories, Inc.

9020 Kenamar Drive, Suite 205
San Diego, CA 92121
WWW.PAGEKEEPER.COM/CA/DTEK
(619) 566-4540 FAX (619) 566-4542

QUALITY CONTROL DATA REPORT

Date: 3/5/99

Attn: Mr. Jim Barton

Log Numbers: 99-0468

Date Analyzed: March 2, 4, and 5, 1999

Analysis	Prep/Analysis Method	LCS % Recovery	Spike % Recovery	Duplicate RPD %
pH	EPA 9045 C	100		0
Sulfate	EPA 9038/ TEX-620-J	101	108	1

Ellen Atienza
Operations Manager

QUALITY CONTROL TERMINOLOGY

- **LCS** - Laboratory Control Sample. Reported as % Recovery of an Independent Standard carried through all sample preparation procedures to verify method performance. Acceptable range Is 80 % -120 % recovery.
- **Spike**- Environmental sample is matrix spiked with method compounds and % recovery of concentration spiked into sample is calculated. Reported as
- **% Recovery**. Acceptable range for "Normal Matrix Sample" is 75 % - 125% recovery.
- **% Recovery** = (Spike Sample Result - Sample Result) *100/Spike Concentration.
- **RPD (Relative % Difference)** = (Spiked Sample - Spike Duplicate) * 100 / Average Result.

D-TEK Analytical Laboratories, Inc.
9020 Kenamar Drive, Suite 205
SAN DIEGO, CA 92121
(619) 566-4540, FAX (619) 566-4542

CHAIN OF CUSTODY

DATE: 3/2/99 PAGE 1 OF 1

D-TEK LOG #: ______ TO #: ______

CUSTOMER INFORMATION

COMPANY: ROBERT PRATER ASSOC. CONTACT PERSON: JIM BARTON
PROJECT MANAGER:
ADDRESS: 10505 ROSELLE STREET CITY: SD ZIP: 92121
PHONE: 619 453 5605
FAX: 619 453 7420

PROJECT INFORMATION

PROJECT NAME/NUMBER: 546-6 LIGHT INDUSTRIAL

BILLING INFORMATION

BILL TO: SAME
ADDRESS:
PHONE:
P.O.#:

ANALYSIS REQUEST

3-DAY

D-TEK LOG #	SAMPLE # / SAMPLE IDENTIFICATION	SAMPLE DATE	SAMPLE TIME	SAMPLE MATRIX	CONTAIN TYPE	# OF CONTAINERS	pH	RESISTIVITY	SULPHATE									
94-0468	EB-12 @ 0-4	2/24	10:00 am	SOIL	BAG		/	/	/									

SAMPLE INTEGRITY

MEETS HOLDING TIME ?	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N
CORRECT CONTAINERS ?	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N

1. RELINQUISHED BY: DATE/TIME 3/2/99 7:15	2. RELINQUISHED BY: DATE/TIME	3. RELINQUISHED BY: DATE/TIME	SAMPLE RECEIPT	SPECIAL INSTRUCTIONS
SIGNATURE: Jim Barton	SIGNATURE:	SIGNATURE:	RECEIVED IN ICE? YES/NO	3-day Turn around
PRINTED NAME: JIM BARTON	PRINTED NAME:	PRINTED NAME:	TAPE SEAL INTACT Y/N NA	
COMPANY: RPA	COMPANY:	COMPANY:	PRESERVATIVE YES/NO NA	
1. RECEIVED BY: DATE/TIME 3/2/99 1135	**2. RECEIVED BY: DATE/TIME**	**3. RECEIVED BY: DATE/TIME**	PRECAUTIONS:	
SIGNATURE: Jennifer Brewhl	SIGNATURE:	SIGNATURE:		
PRINTED NAME: JENNIFER BREWHL	PRINTED NAME:	PRINTED NAME:		
COMPANY: D-TEK	COMPANY:	COMPANY:		TAT

-- ALL SAMPLES ARE SUBJECT TO TERMS AND CONDITIONS ON REVERSE SIDE --

APPENDIX B

Seismic Refraction Survey by Southwest Geophysics dated May 16, 2014

SEISMIC REFRACTION SURVEY
CAMPUS POINTE
SAN MARCOS, CALIFORNIA

PREPARED FOR:

Richard and Richard Construction Company, Inc.
234 Venture Street, Suite 100
San Marcos, CA 92078

PREPARED BY:

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

May 16, 2014
Project No. 114175

May 16, 2014
Project No. 114175

Mr. Bryan Toscani
Richard and Richard Construction Company, Inc.
234 Venture Street, Suite 100
San Marcos, CA 92078

Subject: Seismic Refraction Survey
Campus Pointe
San Marcos, California

Dear Mr. Toscani:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the proposed grading at the subject project located in San Marcos, California. Specifically, our survey consisted of performing 10 seismic refraction traverses within the limits of the proposed property. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability and depth to bedrock 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 related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Edward Verdugo, G.I.T.
Senior Staff Geologist/Geophysicist

Patrick Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/16
STATE OF CALIFORNIA

ERV/PFL/HV/hv

Distribution: (1) Addressee (electronic)

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to the proposed grading at the subject project located in San Marcos, California (Figure 1). Specifically, our survey consisted of performing 10 seismic refraction traverses within the limits of the proposed property. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability and depth to bedrock 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 10 seismic refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results, conclusions and recommendations.

3. SITE DESCRIPTION

The project site is located to the south of Interstate 78 and to the northwest of the intersection of Enterprise Street and Venture Street in San Marcos, California (Figure 1). A residence and various outbuildings including a large chicken coup are present onsite. Vegetation in the project area includes trees, bushes, and minor amounts of brush. A large ravine runs southeast to northwest along a portion of the property. Figures 2 and 3 depict the general site conditions in the area of the lines.

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 subsurface velocity profiles of the areas 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 geophones and recorded with a 24-channel Geometrics StrataView 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.

Ten seismic lines (SL-1 through SL-10) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint for a total of five shot points along each line. 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 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, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assume 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 rippability. These characteristics may also vary with location and depth. 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 (Caterpillar, 2011). 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. RESULTS

As previously indicated, 10 seismic traverses were conducted as part of our study. 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.

Figures 4a through 4j present the velocity models generated from our study. The approximate locations of the seismic refraction traverses are shown on the Line Location Map (Figure 2).

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil overlying granitic bedrock with varying degrees of weathering. Figures 4a through 4j provide the velocity models calculated from SeisOpt Pro. Distinct vertical and lateral velocity variations are evident in the models. These inhomogeneities are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. It is also evident in the tomography models that the depth to bedrock is highly variable across the site.

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

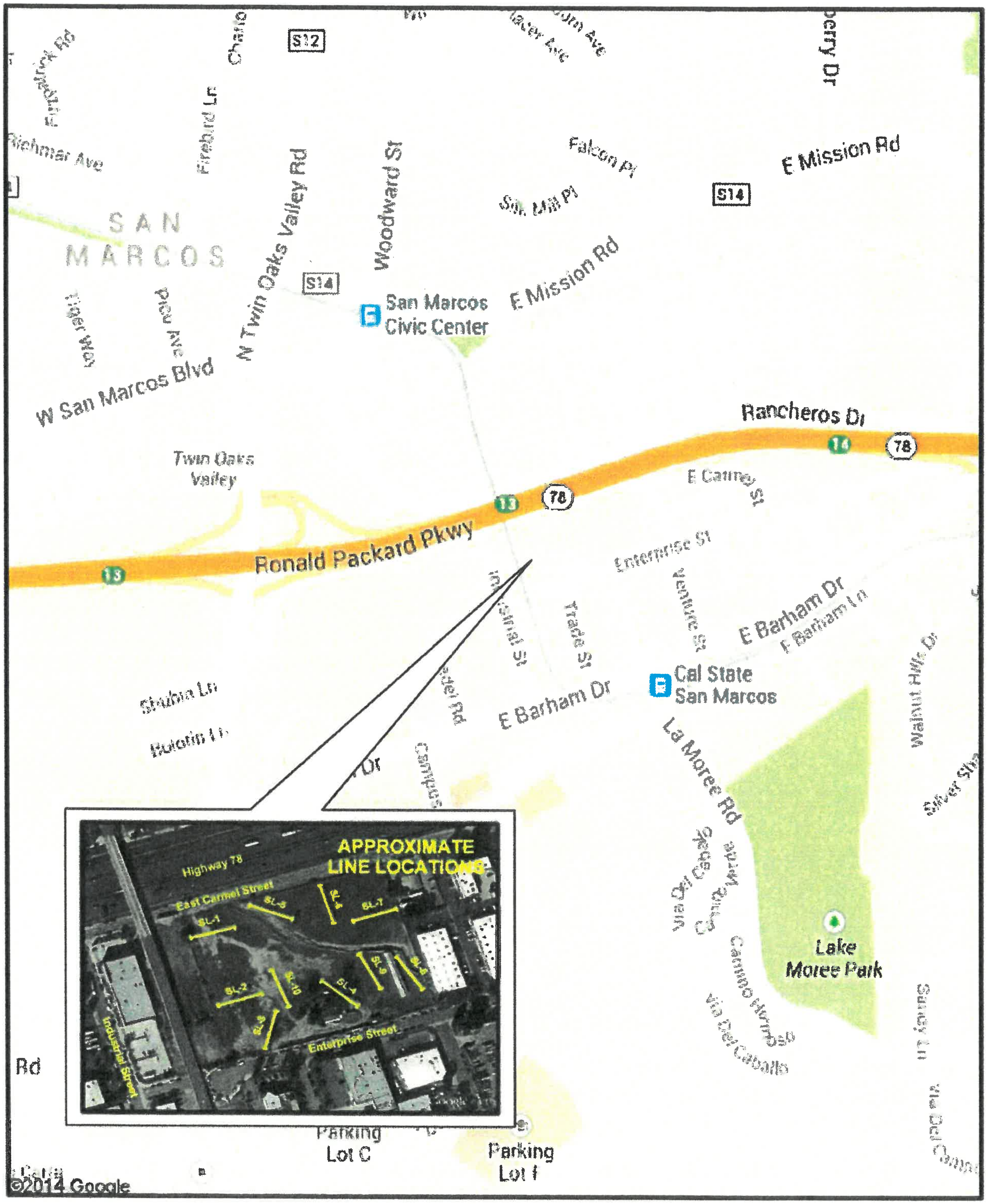
Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

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

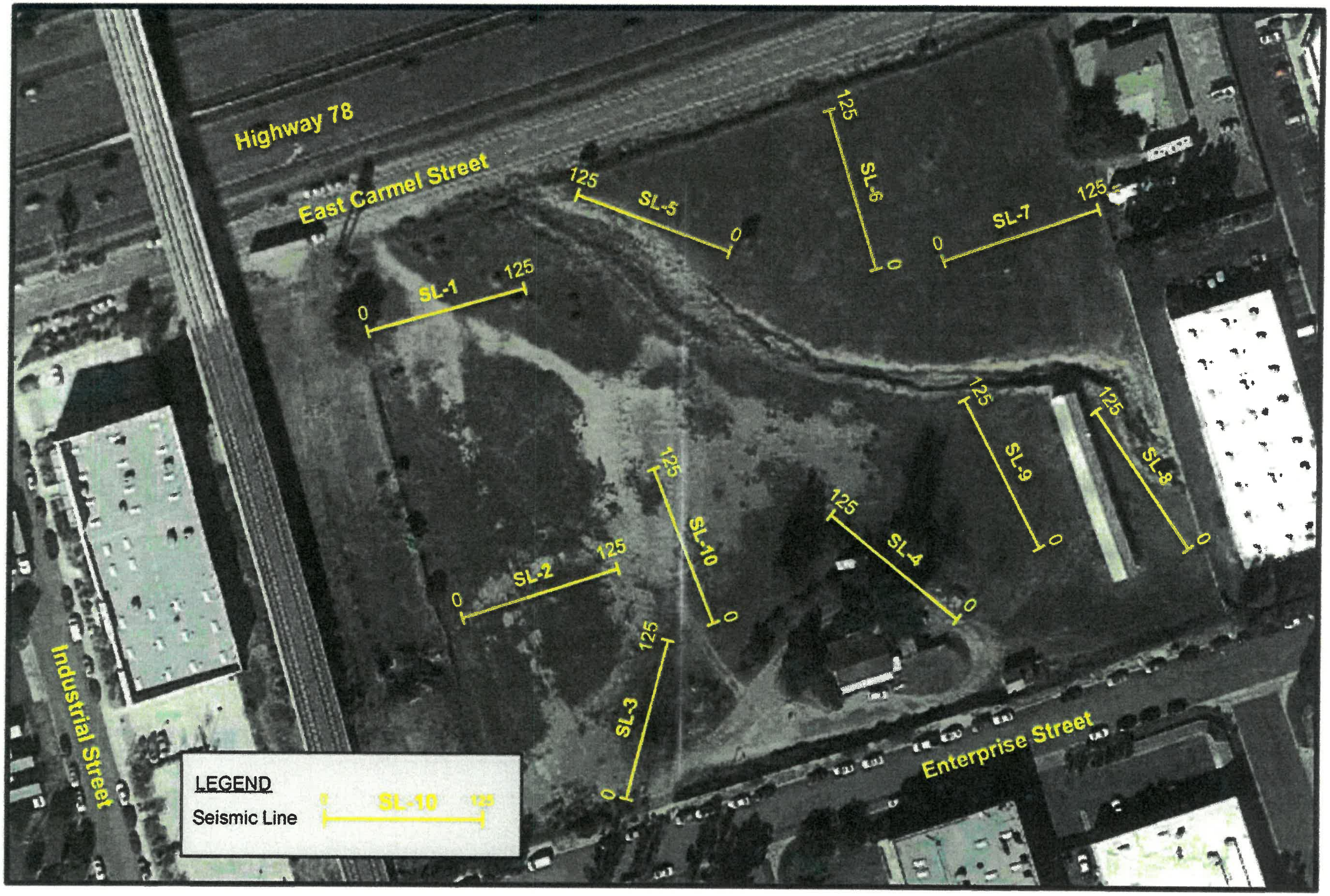
Optim, Inc., 2008, SeisOpt Pro, V-5.0.

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

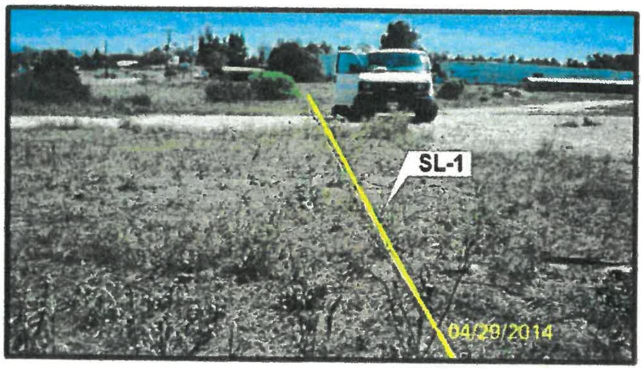
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.

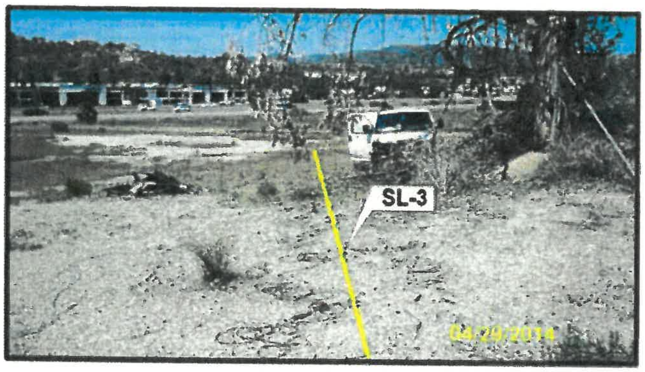


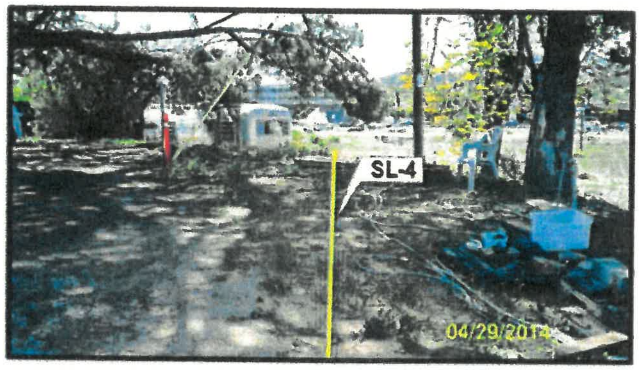
LINE LOCATION MAP	N	Campus Pointe San Marcos, California	SOUTHWEST GEOPHYSICS INC
		Project No.: 114175 Date: 05/14	Figure 1

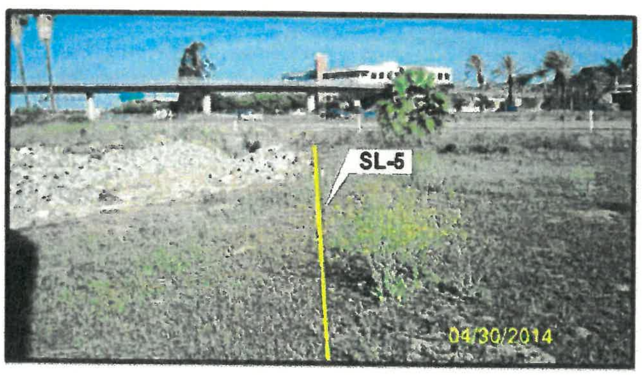


LINE LOCATION MAP	N	Campus Pointe San Marcos, California Project No.: 114175 Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 2	0 100 200 approximate scale in feet

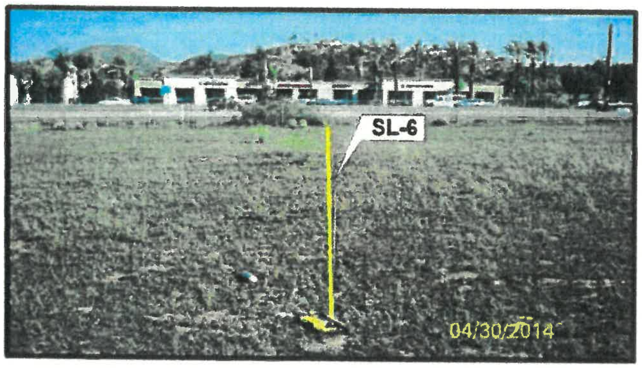


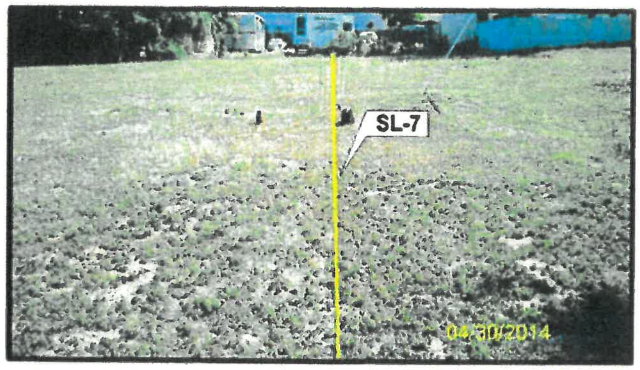


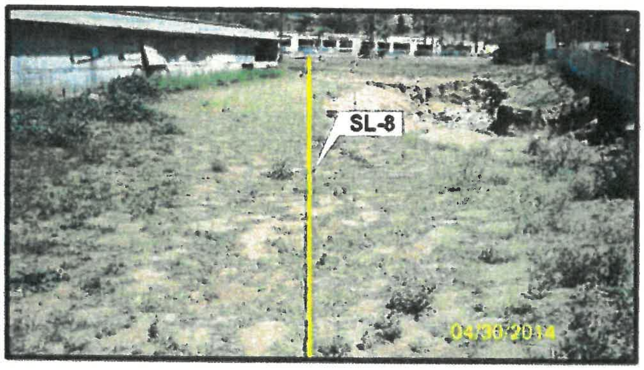


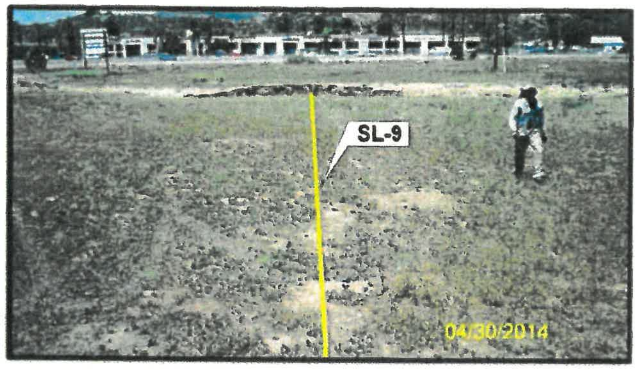


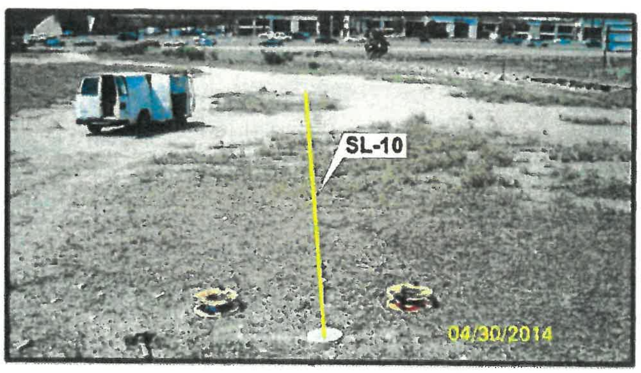
SITE PHOTOGRAPHS	Campus Pointe San Marcos, California	SOUTHWEST GEOPHYSICS INC
	Project No.: 114175 Date: 05/14	Figure 3a







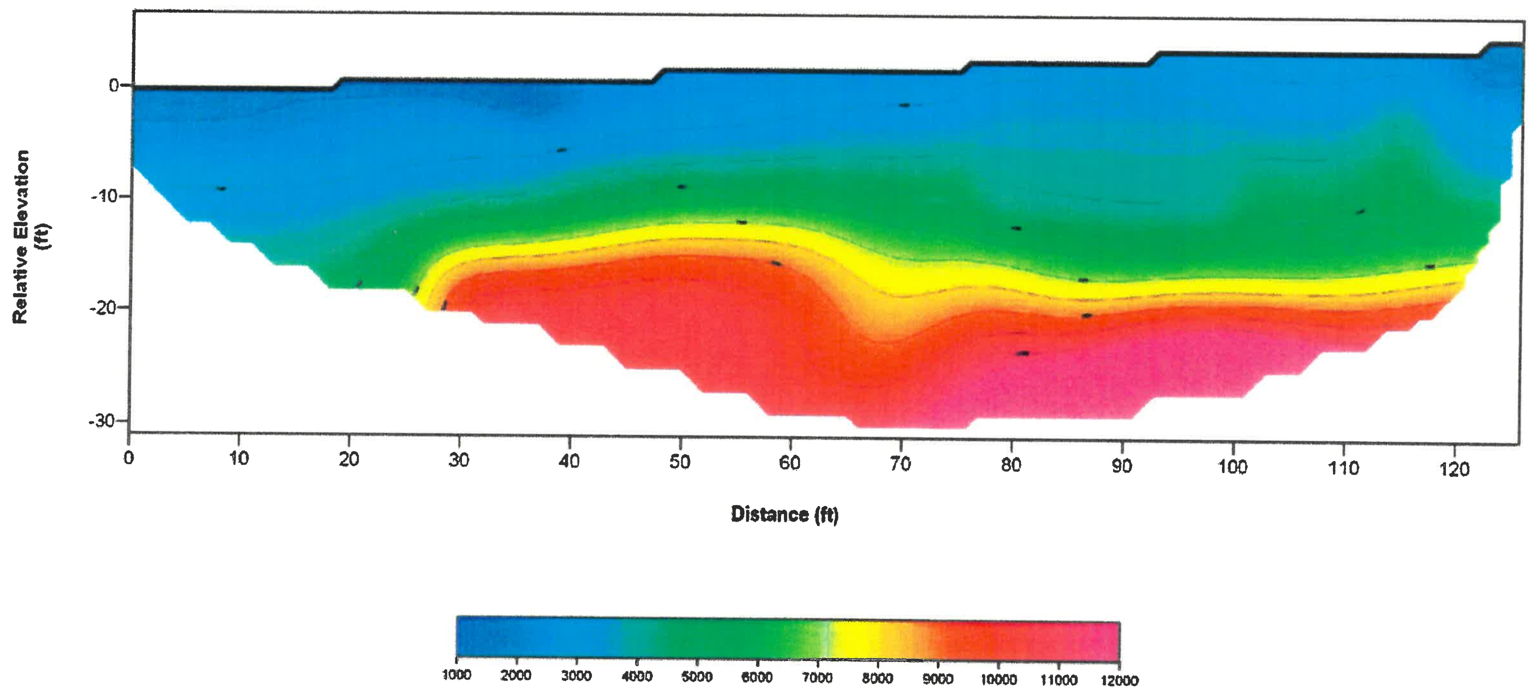




SITE PHOTOGRAPHS	Campus Pointe San Marcos, California		SOUTHWEST GEOPHYSICS INC.
	Project No.: 114175	Date: 05/14	Figure 3b

TOMOGRAPHY MODEL

SL-1

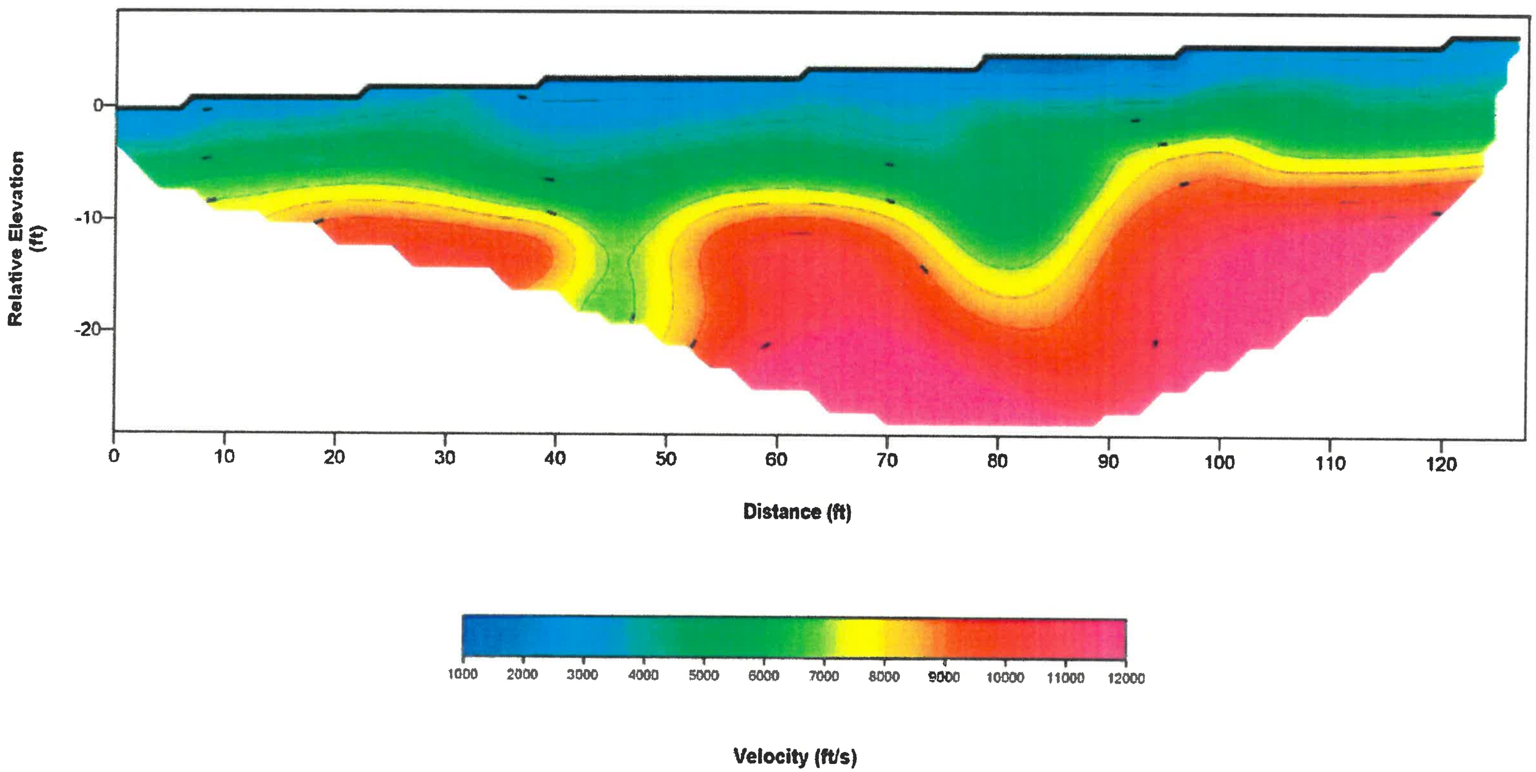


SEISMIC PROFILE	Campus Pointe San Marcos, California	SOUTHWEST GEOPHYSICS INC Figure 4a
	Project No : 114175 — Date: 05/14	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

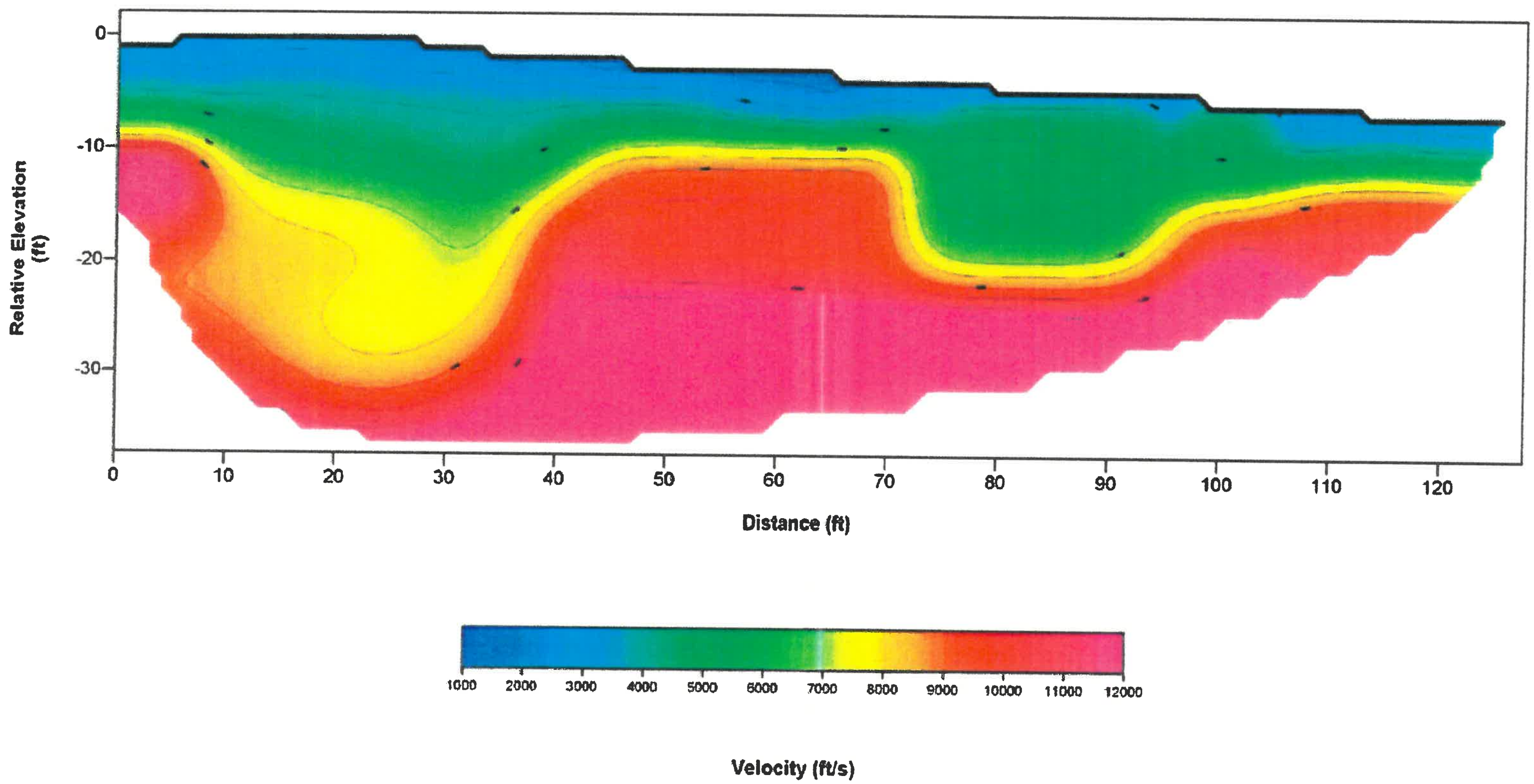
SL-2



SEISMIC PROFILE	Campus Pointe San Marcos, California Project No.: 114175 / Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 4b	**Note: Contour Interval = 1,000 feet per second**

TOMOGRAPHY MODEL

SL-3

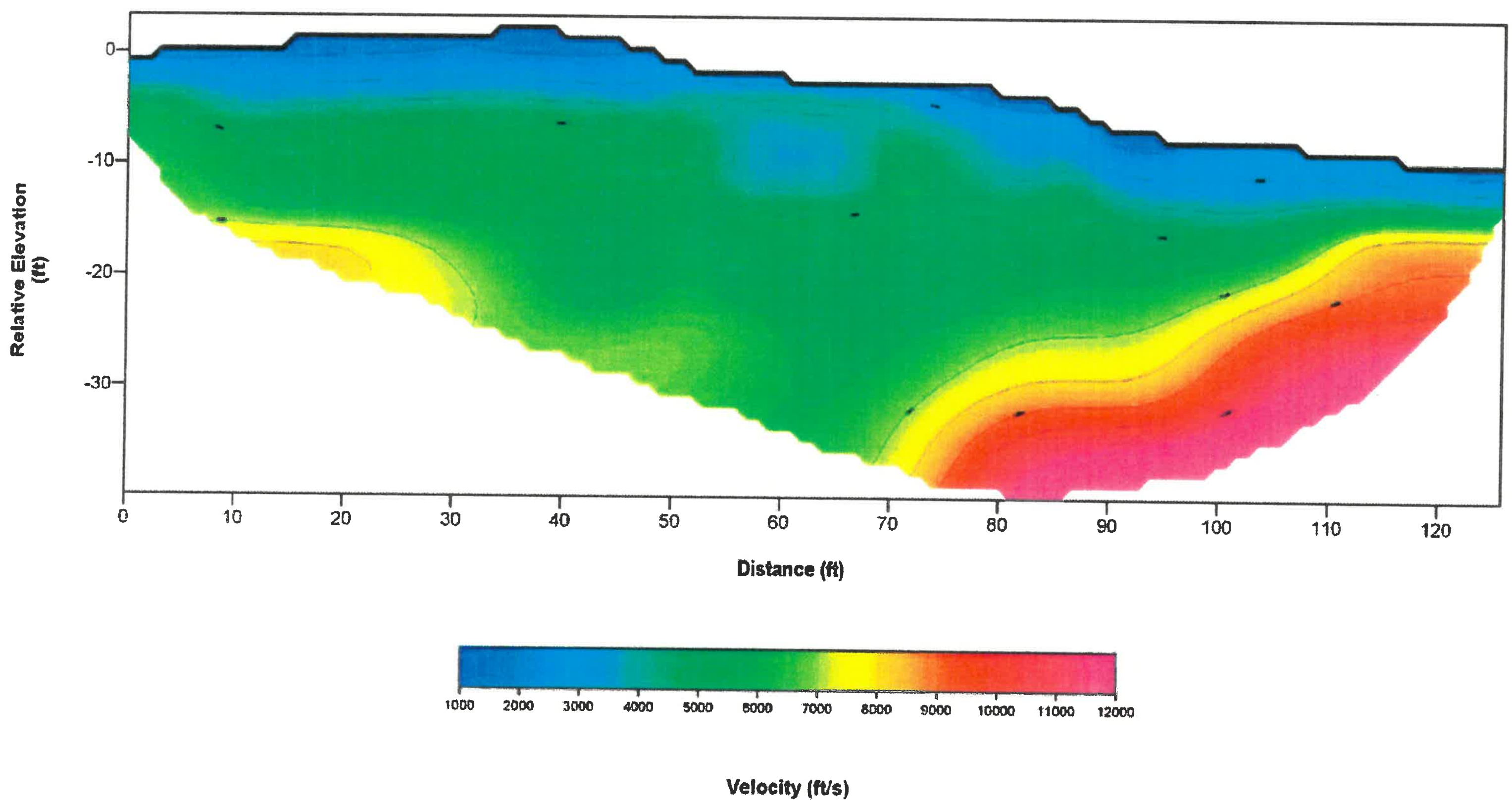


SEISMIC PROFILE	Campus Pointe San Marcos, California	SOUTHWEST GEOPHYSICS INC. Figure 4c
	Project No.: 114175 / Date: 05/14	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

SL-4

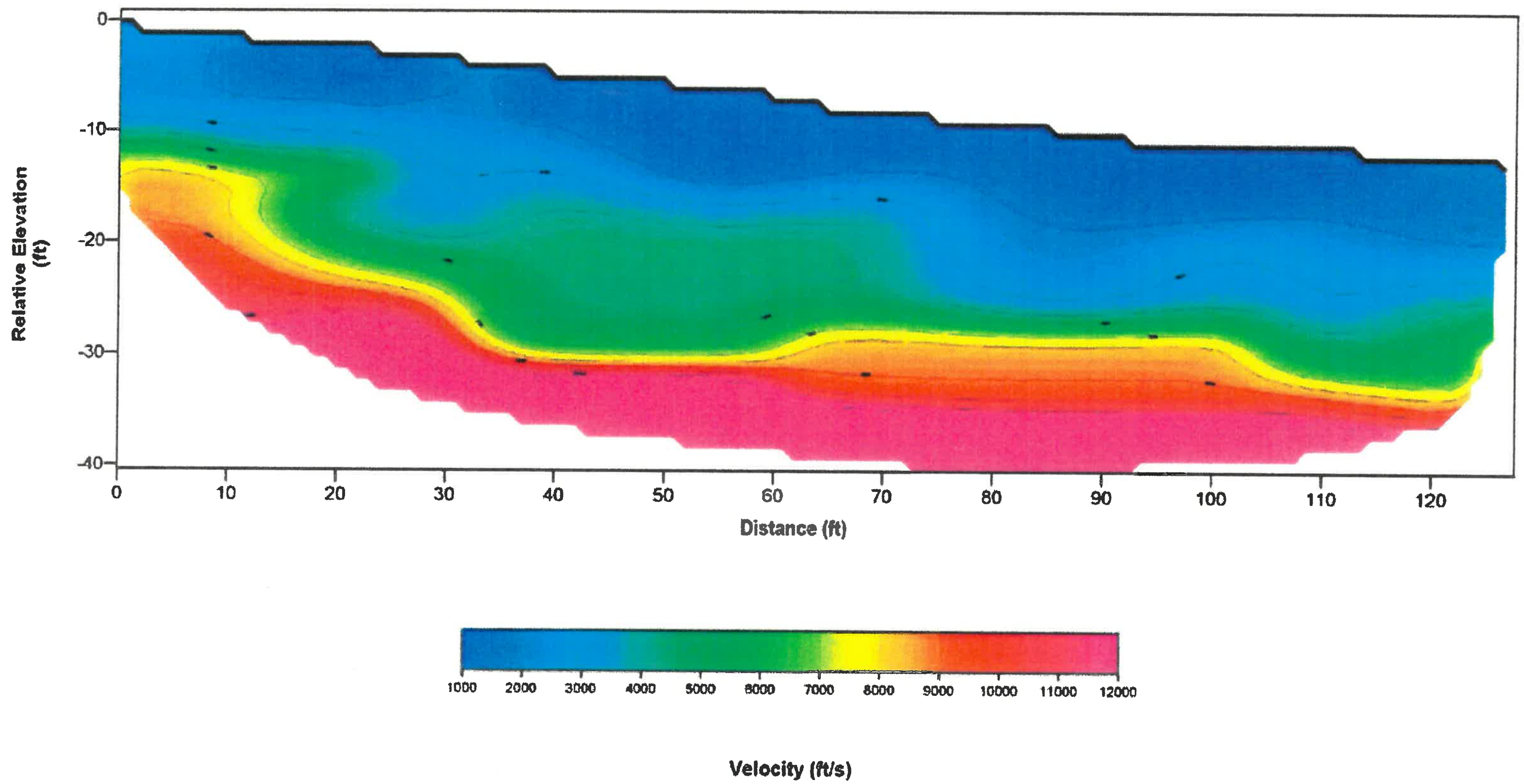


SEISMIC PROFILE	Campus Pointe San Marcos, California	SOUTHWEST GEOPHYSICS INC. Figure 4d
	Project No: 114175 / Date: 05/14	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

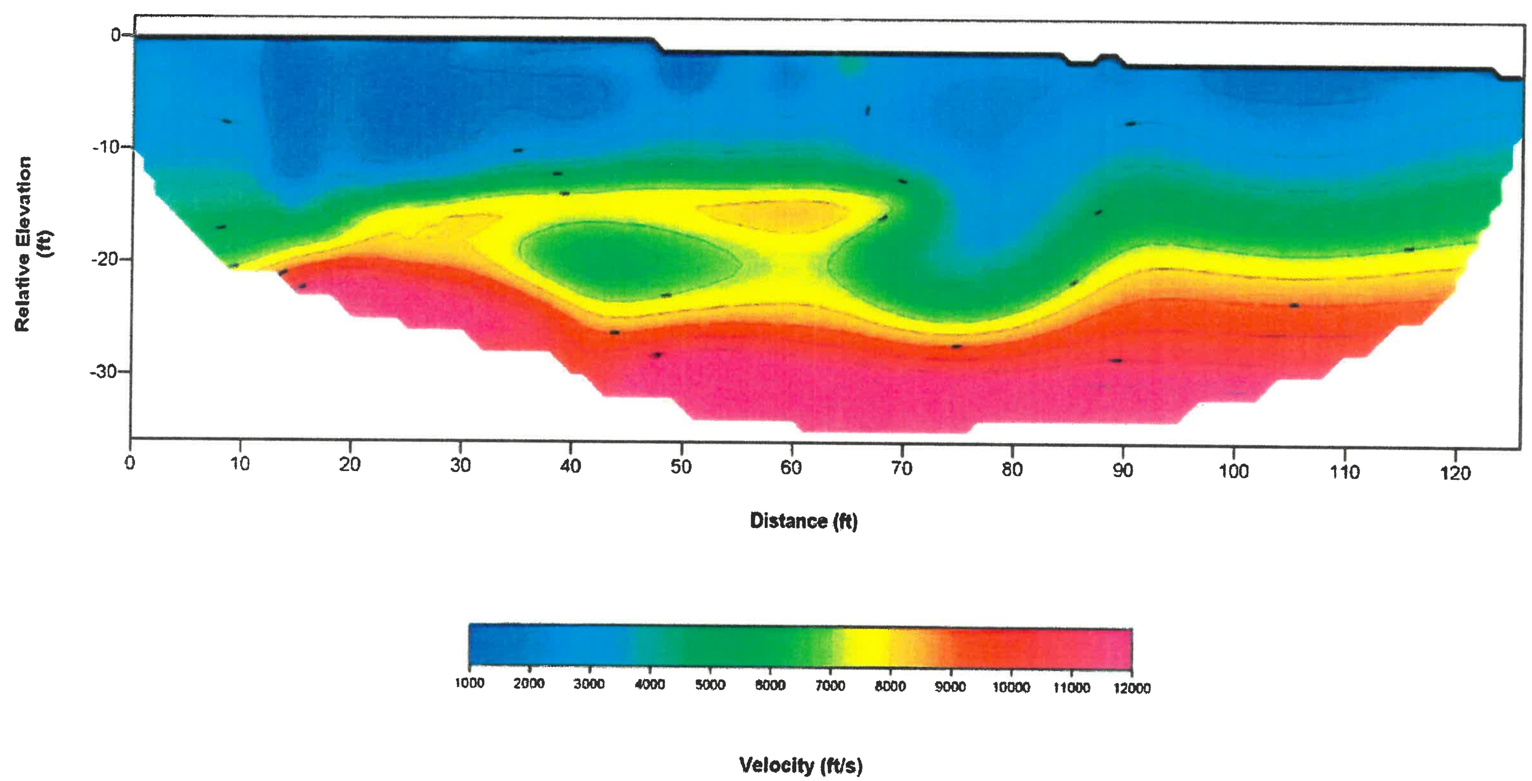
SL-5



SEISMIC PROFILE	Campus Pointe San Marcos, California Project No.: 114175 — Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 4e	Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

SL-6

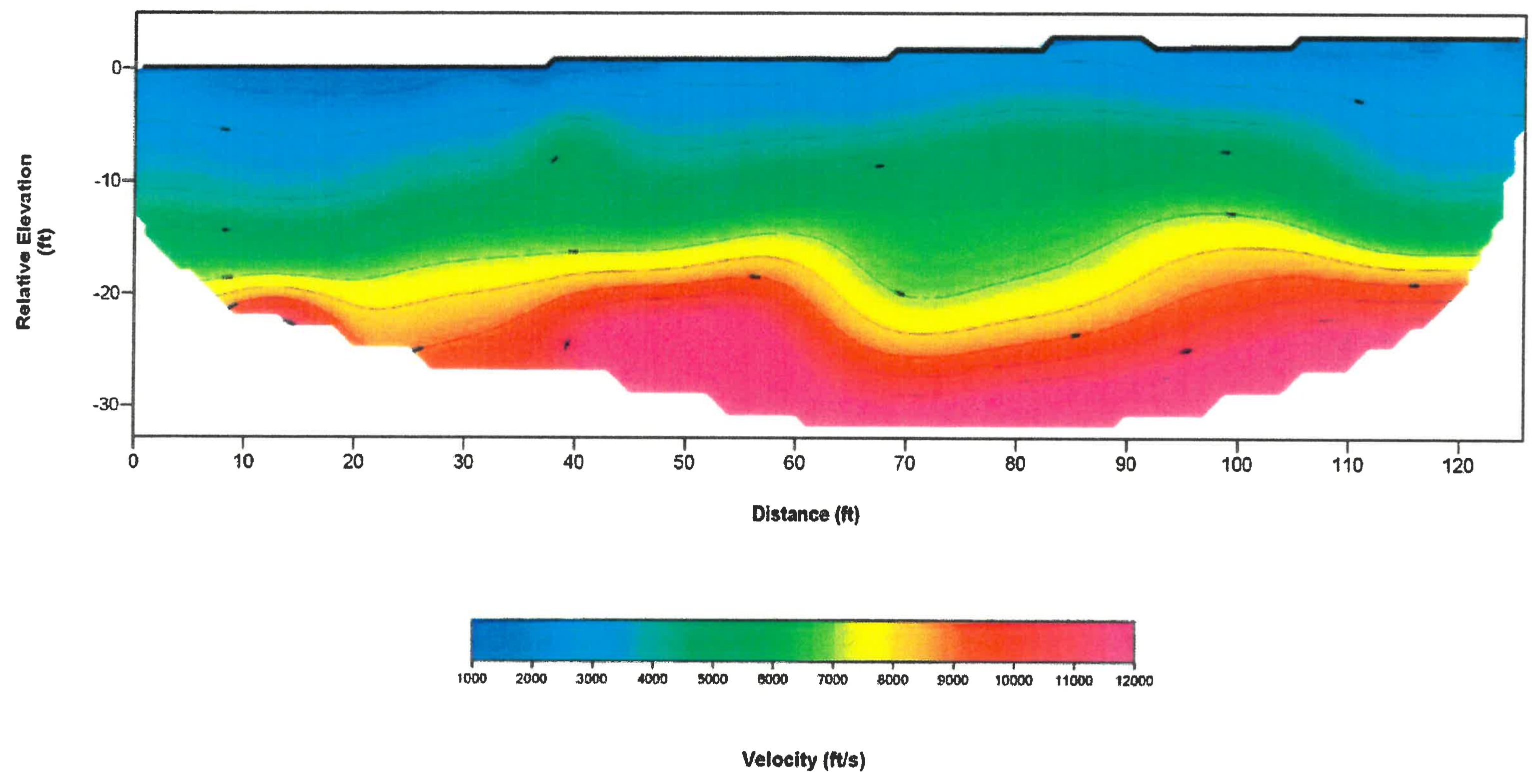


SEISMIC PROFILE	Campus Pointe San Marcos, California Project No : 114175 · Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 4f

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

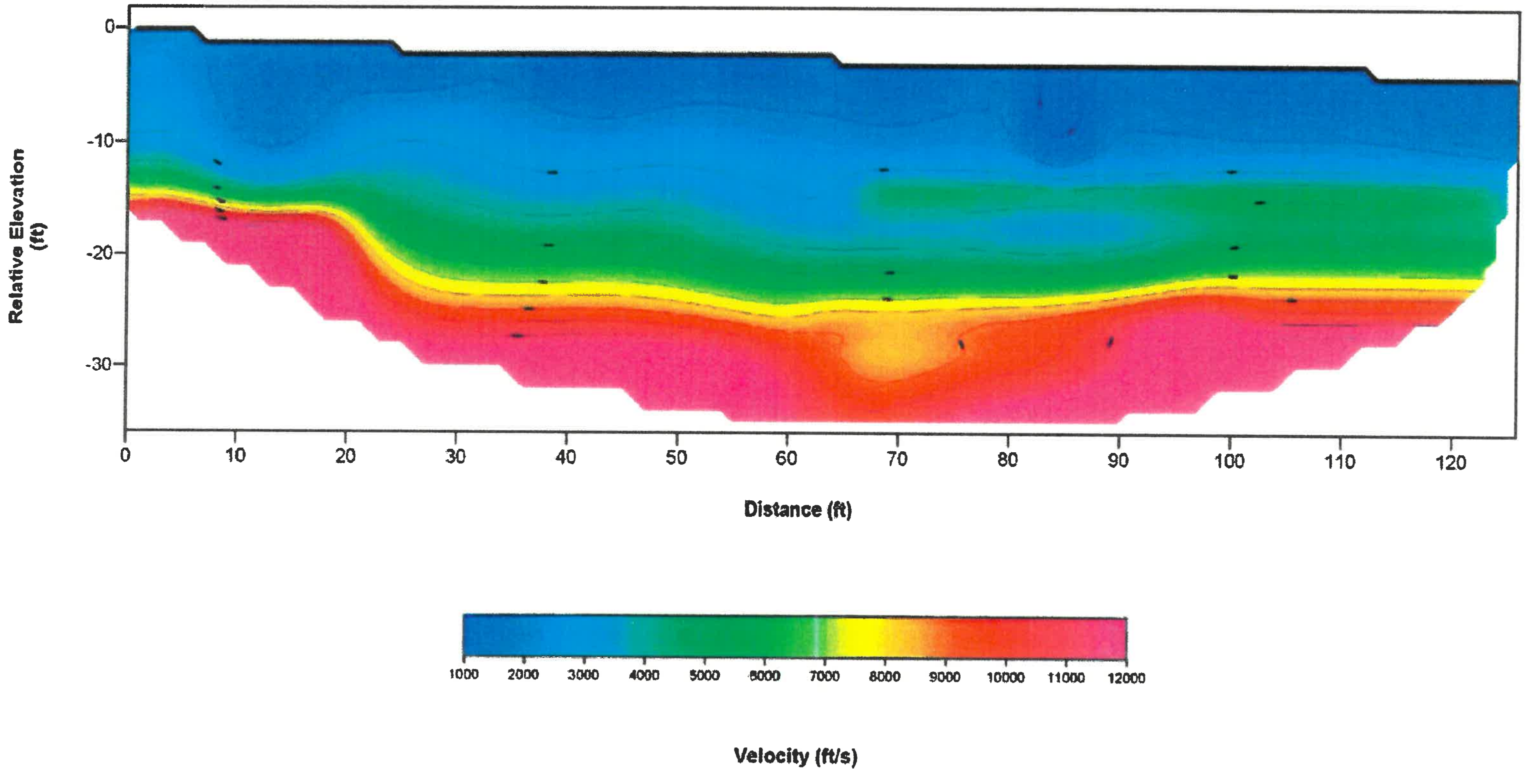
SL-7



SEISMIC PROFILE	Campus Pointe San Marcos, California		SOUTHWEST GEOPHYSICS INC Figure 4g	Note: Contour Interval = 1,000 feet per second
	Project No.: 114175	Date: 05/14		

TOMOGRAPHY MODEL

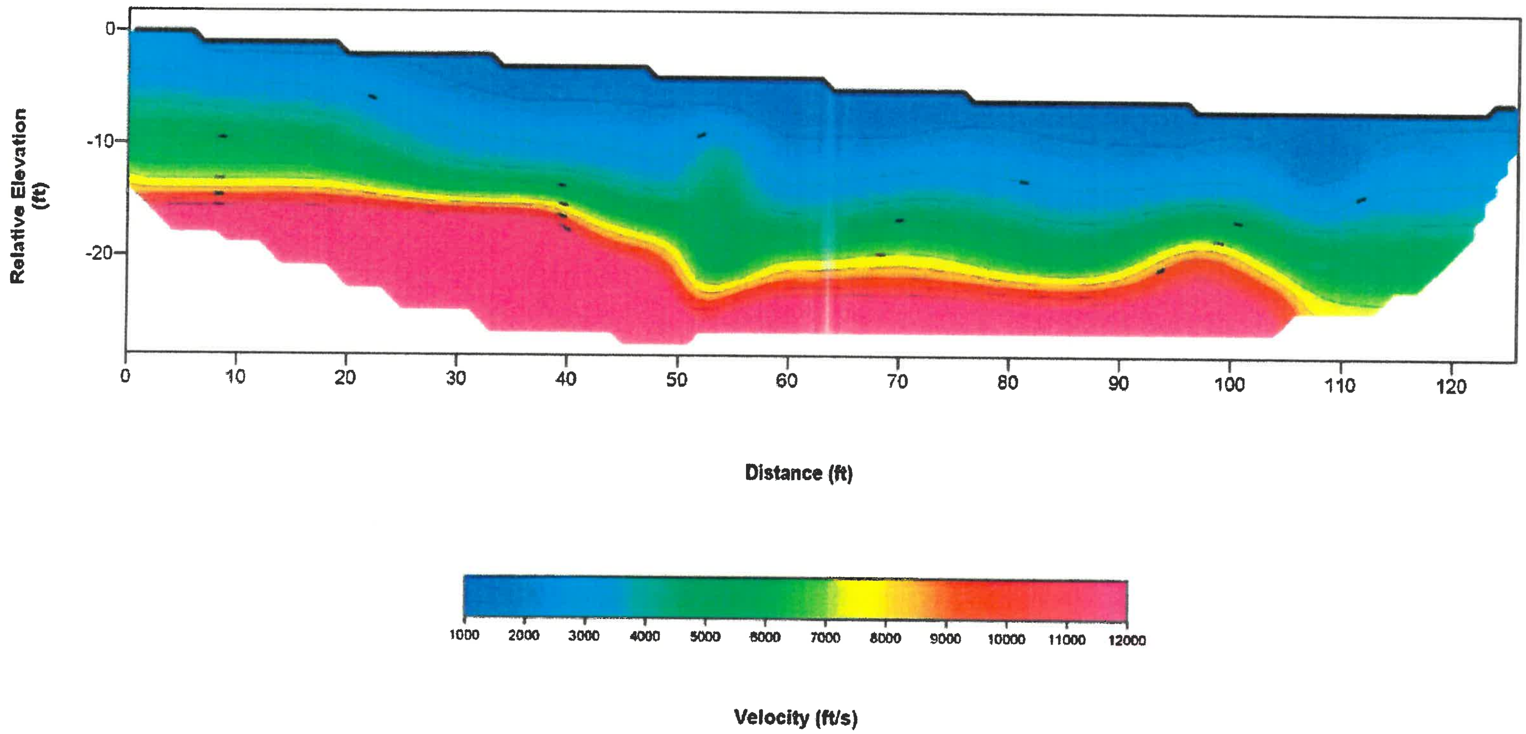
SL-8



SEISMIC PROFILE	Campus Pointe San Marcos, California Project No: 114175 Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 4h	**Note: Contour Interval = 1,000 feet per second**

TOMOGRAPHY MODEL

SL-9

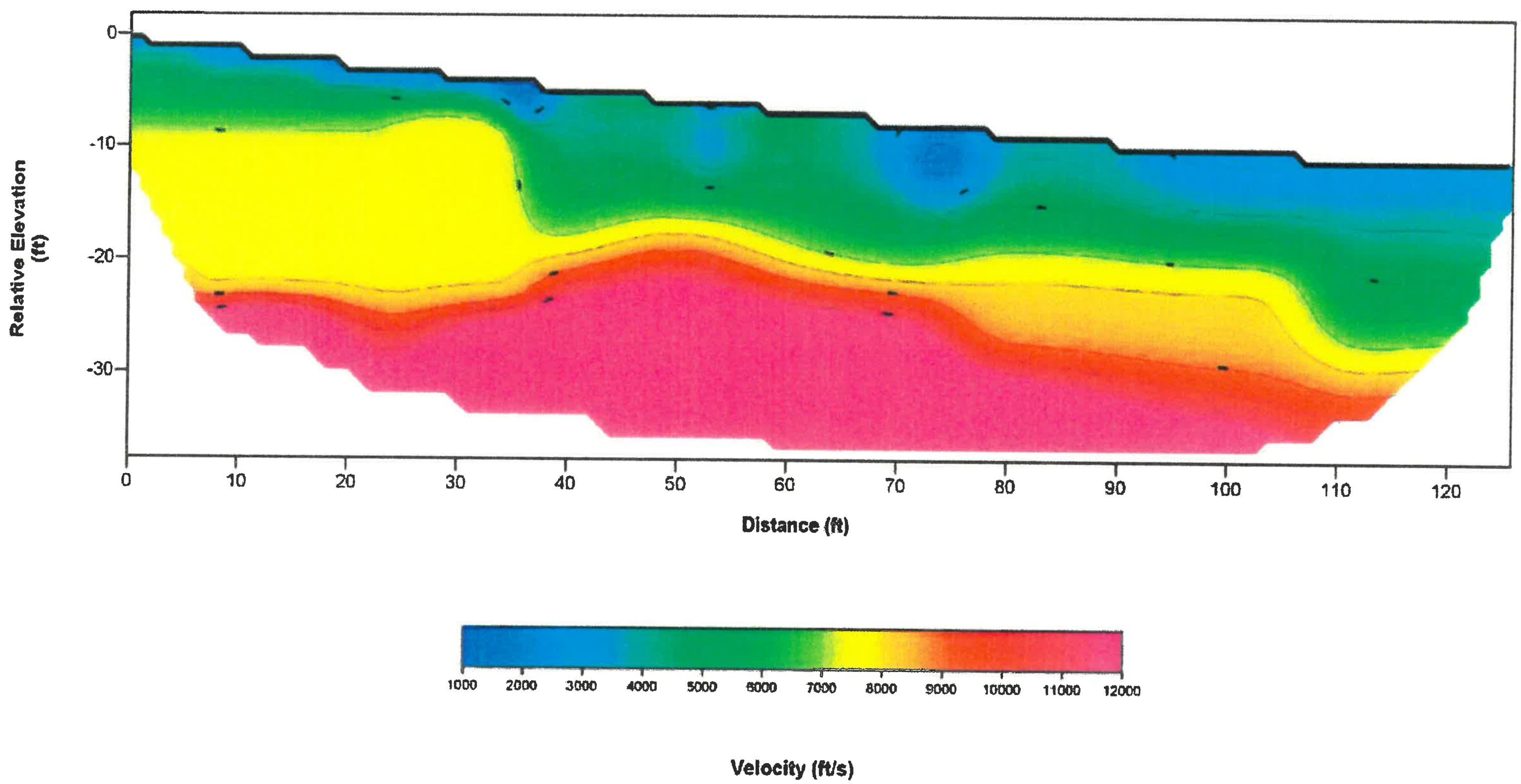


SEISMIC PROFILE	Campus Pointe San Marcos, California		SOUTHWEST GEOPHYSICS INC. Figure 4i
	Project No : 114175	Date: 05/14	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL

SL-10



SEISMIC PROFILE	Campus Pointe San Marcos, California Project No : 114175 — Date: 05/14	SOUTHWEST GEOPHYSICS INC. Figure 4j

Note: Contour Interval = 1,000 feet per second