PRELIMINARY GEOTECHNICAL EVALUATION

FOR

EASTERN PORTION OF 225 N. LAS POSAS ROAD
SAN MARCOS, CALIFORNIA

PREPARED FOR

Las Posas, LLC
c/o Avenue Secured Capital Group
2555 Locus Street
San Diego, California 92069

PREPARED BY

GEOTEK, INC.
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PROJECT NO. 3865-SD **AUGUST 10, 2023**

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Project No. 3865-SD

Las Posas, LLC
c/o Avenue Secured Capital Group
2555 Locust Street
San Diego, CA 92106

Attention: Dan Tate

Subject: Preliminary Geotechnical Evaluation
Eastern Portion of 225 N. Las Posas Rd.
San Marcos, CA 92069

References: See Page 5

Dear Mr. Tate

We are pleased to provide herein the results of our preliminary geotechnical evaluation for the subject project located in the City of San Marcos, California. This report presents the results of our evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development. The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Christopher D. Livesey
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Project Engineer

Distribution: (1) Addressee via email

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions on the site. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site.
- Site reconnaissance by a field professional from our firm.
- Compilation of this geotechnical report which presents our findings of pertinent site geotechnical conditions and geotechnical recommendations for site development.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 Site Description

The subject site is the eastern portion of Assessor's Parcel Number (APN) 219-162-62-00, which is located on Armorlite Drive, in the City of San Marcos, California, see figure 1 (Site Location Map). The Site Location Map differentiates the site vs the legal address of 225 North Las Posas Road. This geotechnical due diligence is focused only on the site and not the entirety of 225 North Las Posas Road.

The site is currently vacant land and bounded by a railroad easement to the north (NCTD), by Armorlite Drive to the south, by a multifamily residential property to the east (Palomar Station Apartments at 1257 Armorlite Drive), and George's Burgers Restaurant (217 North Las Posas Road) and AT&T Facility Building (225 North Las Posas Road) to the west.

Based on a review of Google Earth and our site reconnaissance, topography is characterized by a topographic knoll in the north-center of the subject site sloping outward. A natural slope grades to the south at an approximate 13:1 slope. Cut slopes approximately eight feet tall at an approximate gradient of 2:1 descending to the west and north (225 N Las Posas and NCTD, respectively). The descending grade to the east is supported by an offsite concrete masonry retaining wall approximately 2-3 feet. Elevations range from 575 near the center of the knoll to 562 feet along Armorlite Drive. Vegetation consists mostly of waist-high brush and shrubs.

Several granitic boulders outcrop the surface in the north and scattered animal burrows are found throughout the site.

2.2 Proposed Development

Based on the Site Plan prepared by Latitude 33, GeoTek anticipates that the proposed development will consist of four levels of wood-framed apartment homes above a concrete podium facilitating vehicular parking and commercial space for a total of five stories.

Based on preliminary discussions regarding potential hard rock, site grades for the commercial building along the project frontage are designed with a finished floor of 563.50 feet to approximately match Armorlite Drive grades. The parking lot podium structure is designed with a finish floor of 576 feet. Designed grades result in minor cuts and fills (2 feet) in the north and cuts of approximately 4 feet in the south.

As site development planning progresses and plans become available, the plans should be provided to GeoTek for review and comment.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 Previous Evaluation by Alta California Geotechnical

Alta California Geotechnical Inc. (Alta) prepared a preliminary geotechnical investigation on March 31, 2021, for the subject site. Relevant aspects of the report include:

- A John Deere 310 backhoe was utilized to excavate nine test pits in an effort to explore the subsurface conditions.

- Two seismic refraction survey lines were performed, SL-1 and SL-2, 250 and 150 feet in length, respectively.

- A summary of the seismic refraction results was presented, with rippability recommendations and graphical data included. Variability in the rippability of the subsurface material was expected across the subject site, with non-rippable rock for SL-1 and SL-2 interpreted to be at depths of 5 to 20 feet and 3 to 20 feet, respectively.

- Two infiltration tests were conducted on site using shallow percolation test methods in general accordance with County of San Diego standards. The tests were conducted at the the bottom of the test pits (4 and 4.5 feet below the existing surface) in approximately 1-foot deep hand dug borings. During the test, the handdug borings were filled with water and the level was measured every 30 minutes until the readings stabilized. The data was then adjusted to provide an infiltration rate utilizing the Porchet Method.

Summary of Infiltration Testing (No Factor of Safety)		
Test Designation	P-1	P-2
Approximate Depth of Test	5 feet	5.5 feet
Final Time Interval	30 minutes	30 minutes
Radius of Test Hole	4 inches	4 inches
Tested Infiltration Rate	0.1 Inches/hr	0.4 Inches/hr

3.2 Field Exploration

A site visit was performed on December 14, 2022, by a GeoTek representative. The site was traversed to observe and document surface conditions. Weathered granitics was observed at the surface. Weathering profile of the granitics consisted of a "decomposed granite" type soil and is profiled in cut slopes descending from the site to the AT&T parking lot and NCTD railroad tracks. Exposed granite outcrops as boulders or "floaters" are at the surface in the northern portion of the site. Site conditions were consistent with those described by Alta.

3.3 Laboratory Testing

Laboratory testing was performed by Alta and is presented in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 Regional Setting

The subject property is located in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends roughly 975 miles from the north and northeasterly adjacent the Transverse Ranges geomorphic province to the tip of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zones trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province. No faults are shown in the immediate site vicinity on the map reviewed for the area.

4.2 EARTH MATERIALS

A brief description of the earth materials encountered during subsurface exploration is presented in the following sections. Based on our field observations and review of published geologic maps and previous studies the subject site area is locally underlain by a layer of colluvium over Cretaceous age Tonalite (granitic) bedrock. Alta described the surficial material as alluvium, however considering the topography of the site GeoTek interprets the material as colluvium. Alluvial soils may be present in the topographic lows of the site. The colluvial and alluvial soils are not considered to be competent to support structural foundations in their current state, thus differentiation of alluvial and colluvial soils is not considered to be necessary.

4.2.1 Colluvium and Alluvium, undifferentiated (map symbol Qal)

Colluvium/alluvium was locally observed on the surface of the site and dispersed throughout and generally consisted of brown to tannish brown to orange brown silty sand. Colluvium/alluvium material is not considered suitable for support of structural site improvements, but may be re-used as engineered fill, if properly placed.

4.2.2 Tonalite (map symbol Kt)

Based off the most recent regional geologic map showing the overall site geology (Kennedy, 2007) and our site reconnaissance, we observed Cretaceous-age tonalite (granitic) bedrock at the surface across the site, with outcrops and partially exposed core stones of bedrock materials or boulders. Anticipated bedrock excavation characteristics are discussed in a later section of this report. The results of the subsurface seismic refraction surveys are included in Appendix A.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during our site visit. If encountered during earthwork construction, surface water on this site is the result of precipitation. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater is not anticipated to be within 50 feet of the ground surface at the subject site and is not anticipated to be a factor in site development. Localized perched groundwater could be present but is also not anticipated to be a factor in site development.

4.4 EARTHQUAKE HAZARDS

4.4.1 Surface Fault Rupture

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone or a Special Studies Zone (Bryant and Hart, 2007). No faults are identified on the readily available geologic maps that were reviewed by this firm for the immediate study area.

4.4.2 Liquefaction/Seismic Settlement

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The liquefaction potential and seismic settlement potential on this site is considered to be negligible, due to anticipated medium dense consistency and thickness of less than 10 feet of anticipated fills, shallow bedrock and absence of a shallow groundwater table.

4.4.3 Other Seismic Hazards

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazards such as seiche and tsunami is considered to be remote due to site elevation and distance from an open body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Development of the site appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated in the design and construction phases of the development. The following sections present general recommendations for currently anticipated site development plans.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of San Marcos, the 2022 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix C outline general procedures and do not anticipate all site specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

5.2.2 Site Clearing and Preparation

Site preparation should start with removal of deleterious materials and vegetation. These materials should be disposed of properly off site. Any existing underground improvements, utilities and trench backfill should also be removed or be further evaluated as part of site development operations.

5.2.3 Remedial Grading

Prior to placement of engineered fill materials, the upper loose and compressible materials should be removed in preparation for areas to receive structural improvements and engineered fills. Removal depths in areas of existing colluvium and highly weathered bedrock are estimated to be an average of approximately 2 to 3 feet in most areas. In areas not explored, deeper colluvium may exist and should anticipate deeper removals of 5 feet. The lateral extent beyond the outside edge of all settlement sensitive structures/foundations should be equivalent to that vertically removed.

Removal bottom gradients should be sloped towards the street to reduce potential long term perched groundwater conditions. The bottom of all removals should be brought to at or above optimum moisture content, and then compacted to minimum project standards prior to fill placement. The remedial excavation bottoms should be observed by a GeoTek representative.

The resultant voids from remedial grading/overexcavation should be filled with materials placed in general accordance with Section 5.2.6 Engineered Fill of this report.

Depending on final proposed grades relative to the less weathered bedrock materials, additional excavation may be required to enable utility trench excavation with typically employed equipment. If this is desired, such excavation would extend to the bottom of the deepest proposed utility trench bedding zone.

5.2.4 Cut Building Pads

Foundations bearing completely on competent granitic bedrock may remain as a cut condition, however, foundation and underground utility excavations may be difficult with conventional light duty excavation equipment.

5.2.5 Over-Excavation

At a minimum, the cut portion(s) of any cut/fill transition building pad areas in bedrock should be overexcavated a minimum of three feet below finish pad grade or a minimum of two (2) feet below the bottom of the deepest proposed footing or underground utility servicing the building. whichever is deeper.

5.2.6 Engineered Fill

Onsite materials are generally considered suitable for reuse as engineered fill provided, they are free from vegetation, roots, debris and oversized material (rock or hard lumps) greater than 6 inches. Oversized materials generated from grading operations should be anticipated and can be utilized onsite provided they are incorporated into fills as structural rock fills (see section 5.2.7) or other methods presented in appendix C. The earthwork contractor should have the proposed excavated materials to be used as engineered fill at this project approved by the soils engineer prior to placement.

Engineered fill materials should be moisture conditioned to at or above optimum moisture content and compacted in horizontal lifts not exceeding 8 inches in loose thickness to a minimum relative compaction of 90% as determined in accordance with laboratory test procedure ASTM D 1557.

If fill is being placed on slopes steeper than 5:1 (h:v), the fill should be properly benched into the existing slopes and a sufficient size keyway shall be constructed in accordance with grading guidelines presented in Appendix C.

5.2.7 Structural Rock Fills

It should be anticipated that hard rock will be encountered during rough grading. Remedial removals are anticipated to be relatively shallow. As a result, the site is not anticipated to be a good candidate for incorporating oversized rock into engineered fills. However, if the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated burial or windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within five feet of finished grade, they may affect foundation design.
2. Rock fills are required to be placed in horizontal layers that should not exceed two feet in thickness, or the maximum rock size present, which ever is less. All rocks exceeding two feet should be broken down to a smaller size, or disposed of in isolated burial pits. Localized larger rock up to 4 feet in largest dimension may be placed in rock fill as individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill. Loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer. The portion of the rock above grade is covered with a second lift.
3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

5.2.8 Compaction Procedures of Rock Fills

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.
    a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.
    b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.
    c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). (Water should be applied before and during spreading.)
2. Rock fill should be generously watered (sluiced)
    a) Water should be applied by water trucks to the:
        i) dump piles,
        ii) front face of the lift being placed and,
        iii) surface of the fill prior to compaction.
    b) No material should be placed without adequate water.
    c) The number of water trucks and water supply should be sufficient to provide constant water.
    d) Rock fill placement should be suspended when water trucks are unavailable:
        i) for more than 5 minutes straight, or,
        ii) for more than 10 minutes/hour.

3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber tired compactors may be required.
   a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
   b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.
4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:
   a) the general segregation of rock size,
   b) for any unfilled spaces between the large blocks, and
   c) the matrix compaction and moisture content.
5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
   a) A lift should be constructed by the methods proposed, as proposed.
6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractor's procedures.

A minimum horizontal distance of 7 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 7 feet should be built of general fill materials, not structural rock fill material.

5.2.9 Slope Construction

An engineering geologist should observe all cut slopes. Cut slopes should expose competent bedrock. If adverse structure or unsuitable materials are exposed and identified in the cut slopes, stabilization fills may be recommended.

Where fill is to be placed against sloping ground with gradients of 5:1 (h:v) or steeper, the sloping ground surface should be benched to provide horizontal surfaces for fill placement. A keyway should be constructed at the toe of the fill slope areas into dense natural material and in accordance with Plate G-3, Appendix C.

The base of the keyways and benches should be sloped back into the hillside at a gradient of at least two percent. The base of the benches should be evaluated by a representative of GeoTek prior to processing. Upon approval, the exposed materials should be moistened to at least the optimum moisture content and densified to a relative compaction of at least 90 percent (ASTM D1557). Details showing slope construction are presented in Appendix C.

Fill slopes should be overfilled during construction and then cut back to expose compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slope to provide a dense, erosion resistant surface.

Back drains should be installed in the keyways in accordance with the recommendations outlined in Appendix C.

5.2.10 Excavation Characteristics

Excavations in the onsite colluvium/alluvium materials can generally be accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. Less weathered bedrock materials are also likely to be encountered at depth or at the surface and could locally require special techniques to excavate.

In certain conditions it may be more cost effective to rip a small quantity of hard rock (given equipment wear and tear), than fracturing hard rock by blasting techniques in a mass grade condition. In general, rock displaying high seismic velocities can be more readily excavated in an open cut than in a trench condition. Contractors who perform work in a hard rock environment can better assess their ability to excavate potential hard rock materials.

Seismic refraction data for the areas evaluated seems to generally indicate that excavations on the order of roughly 5 to 20 feet should be excavatable with conventional earthmoving equipment. Shallow hard rock should be anticipated in the central and northern portions of the site. Hard rock material is anticipated to descend to greater depths (i.e. 20 feet) towards Armorlite and the AT&T property. Core stones and boulders were also noted in the seismic refraction data, and also observed on the site, which could necessitate special excavation techniques if encountered during site earthwork. Additional review and interpretation with respect to rock hardness is recommended by the project grading and utility construction contractors.

5.2.11 Shrinkage and Bulking

Several factors will impact earthwork balancing on the site, including bedrock bulking, undocumented fill and colluvium shrinkage, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage and bulking are largely dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor ranging from 5 percent may be considered for the colluvial/alluvial materials requiring removal and re-compaction. Bedrock bulking could range from 10 to 20 percent. Subsidence should not be a factor on the subject site if removals are completed down to the recommended depths to expose bedrock materials. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction.

5.2.12 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 inclinations for short durations during construction, and where cuts do not exceed 10 feet in height. Temporary cuts to a maximum height of 4 feet can be excavated vertically.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

Utility trench backfill should be compacted to at least 90% relative compaction of the maximum dry density as determined per ASTM D 1557. Under-slab trenches should also be compacted to project specifications.

Onsite materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than 6± inches are removed.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Preliminary foundation design criteria, in general conformance with the 2022 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Based on our visual classification of materials encountered onsite and laboratory testing, soils near subgrade may be classified as "very low" expansive (EI≤20) per ASTM D 4829. Additional laboratory testing should be performed at the completion of site grading to verify the expansion potential and plasticity index, if necessary, of the subgrade soils.

The following criteria for design of foundations are preliminary. Additional laboratory testing of the samples obtained during grading should be performed and final recommendations should be based on as-graded soil conditions.

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALLY REINFORCED FOUNDATIONS	
DESIGN PARAMETER	**"Very Low" Expansion Potential (0≤EI≤20)**
Foundation Embedment Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent finished grade)	Five Story – 24
Minimum Foundation Width (Inches)	Five Story - 18
Minimum Slab Thickness (actual)	5 inches
Minimum Slab Reinforcing (parking floor)	No. 3 rebar 24" on-center, each way, placed in the middle one-third of the slab thickness
Minimum Footing Reinforcement	Four No. 4 Reinforcing Bars, two (1) top and two (1) bottom
Presaturation of Subgrade Soil (percent of optimum moisture content)	Minimum 100% to a depth of 12 inches

It should be noted that the above recommendations are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

The following recommendations should be implemented into the design:

- An allowable bearing capacity of 2500 pounds-per-square-foot (psf) may be used for design of continuous and perimeter footings that meet the depth and width requirements in the table above. This value may be increased by 400 pounds per square foot for each additional 12 inches in depth and 200 pounds per square foot for each additional 12 inches in width to a maximum value of 3000 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads).

- Based on our experience in the area, structural foundations may be designed in accordance with 2022 CBC, and to withstand a total settlement of 1 inch and maximum differential settlement of one-half of the total settlement over a horizontal distance of 40 feet. These values assume that seismic settlement potential is not a significant constraint.

- The passive earth pressure may be computed as an equivalent fluid having a density of 300 psf per foot of depth, to a maximum earth pressure of 3000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

- A grade beam, a minimum of 12 inches wide and the depth of the grade beam should be at the same elevation as the bottom of the adjoining footings.

5.3.2 Underslab Moisture Membrane

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2022 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2022 CBC Section 1907.1.1.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the vapor retarder placed atop the underlying aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture that thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6-mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10-mil membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e. thickness, composition, strength and permeability) to achieve the desired performance level.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek does not practice in the field of moisture vapor transmission evaluation/migration, since that practice is not a geotechnical discipline. Therefore, we recommend that a qualified person, such as the flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the building be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person (or persons) should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate. In addition, the recommendations in this report and our services in general are not intended to address mold prevention; since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

5.3.3 Miscellaneous Foundation Recommendations

- To reduce moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.

- Spoils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

- We recommend that control joints be placed in two directions spaced the numeric equivalent roughly 24 times the thickness of the slab in inches (e.g. a 4 inch slab would have control joints at 96 inch [8 feet] centers). These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

5.3.4 Foundation Set Backs

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least 7 feet and need not exceed 40 feet.

- The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem. This applies to the existing retaining walls along the perimeter, if they are to remain.

- The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 projection upward from the bottom of the nearest excavation.

5.3.5 Seismic Design Parameters

The site is located at approximately 33.14709 latitude and -117.18820 longitude. Site spectral accelerations (Ss and S1), for 0.2 and 1.0 second periods for a risk targeted two (2) percent probability of exceedance in 50 years (MCER) were determined using the web interface provided by SEAOC/OSHPD (https://seismicmaps.org) to access the USGS Seismic Design Parameters. We have selected a Site Class "C" based on the shallow depth of fill overlying granitic bedrock.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.03g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.33g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	1.12g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	0.48g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.75g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.32g

5.3.6 Soil Corrosivity

The soil resistivity at this site was tested in the laboratory on samples collected during the field investigation. The results of the testing indicate that the onsite soils are not considered corrosive with current standards used by corrosion engineers. These characteristics are considered typical of soils commonly found in this area of southern California. We recommend that a corrosion engineer be consulted to provide recommendations for proper protection of buried metal at this site.

5.3.7 Soil Sulfate Content

Results indicate that the water soluble sulfate range is less than 0.1 percent by weight, which is considered "not applicable" (negligible) as per Table 4.2.1 of ACI 318-11. Based upon the test results, no special mix design is required by Code to resist sulfate attack. As a minimum, additional testing should be completed subsequent to rough grading in order to confirm these initial results.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of 10 feet. Additional review and recommendations should be requested for higher walls.

Retaining wall foundations embedded a minimum of 18 inches into engineered fill or dense formational materials should be designed using an allowable bearing capacity of 2500 psf. An increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 350 psf per foot of depth, to a maximum earth pressure of 3000 psf. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in the table below for specific slope gradients of retained materials.

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Select Backfill*
Level	40
2:1	70

*Select backfill should consist of native or imported sand or other approved materials with an EI≤20.

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions or adverse geologic conditions.

5.4.2 Seismic Induced Incremental Addition

Additional lateral forces can be induced on retaining walls during an earthquake. For level backfill and a Site Class "C", the minimum earthquake-induced force (F_{eq}) should be $13H^2$ (lbs/linear foot of wall) for cantilever walls. This force can be assumed to act at a distance of 0.6H above the base of the wall, where "H" is the height of the retaining wall measured from the base of the footing (in feet). The 2022 CBC only requires the additional earthquake induced lateral force be considered on retaining walls in excess of six (6) feet in height;

however, the additional force may be applied in design of lesser walls at the discretion of the wall designer.

5.4.3 Wall Backfill and Drainage

Wall backfill should include a minimum one (1) foot wide section of ¾ to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. If the walls are designed using the "select" backfill design parameters, then the "select" materials shall be placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the wall. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs.

The backfill materials should be placed in lifts no greater than 8-inches in thickness and compacted to a minimum of 90% of the maximum dry density as determined in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC, or approved equivalent) in a minimum of one (1) cubic foot per lineal foot of 3/8 to one (1) inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and be directed (via a solid outlet pipe) to an appropriate disposal area.

Walls from two (2) to four (4) feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may collect in front of the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

5.4.4 Restrained Retaining Walls

Any retaining wall that will be restrained prior to placing backfill or walls that have male or reentrant corners should be designed for at-rest soil conditions using an equivalent fluid pressure of 60 pcf (select backfill), plus any applicable surcharge loading. For areas having male or reentrant corners, the restrained wall design should extend a minimum distance equal to twice the height of the wall laterally from the corner, or as otherwise determined by the structural engineer.

5.4.5 Preliminary Pavement Design

Traffic indices have not been provided during this stage of site planning. In addition, site conditions have not been graded to a final design to evaluate specific pavement subgrade conditions. Therefore, the minimum structural sections provided below are based on a preliminary laboratory R-Value of 25 and the assumed traffic indices.

PRELIMINARY ASPHALT PAVEMENT STRUCTURAL SECTION			
Design Criteria	Traffic Index (TI)	Asphaltic Concrete (AC) Thickness (inches)	Aggregate Base (AB) Thickness (inches)
Driveway Aisle or Parking Lot	5.0	3.0	6.0

Actual structural pavement design is to be determined by the geotechnical engineer's testing (R-Value) of the exposed subgrade. Thus, the actual R-Value of the subgrade soils can only be determined at the completion of grading for street subgrades and the above values are subject to change based laboratory testing of the as-graded soils near subgrade elevations.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the ASTM D1557 laboratory maximum dry density as determined by ASTM D1557 test procedures

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the City of San Marcos specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

5.4.6 Portland Cement Concrete (PCC)

For driveways constructed with Portland Cement Concrete (PCC), the following recommended minimum PCC pavement section is provided for these areas:

Driveway into the parking structure or other structural surface pavement

5.5 Inches Portland Cement Concrete (PCC) over
6 Inches Aggregate Base (AB) over
12-inches subgrade compacted to 95% per ASTM D 1557

For the PCC options, it is recommended concrete having a minimum 28-day flexural strength (or modulus of rupture (MOR)) of 650 psi be used. A "pavement"-type concrete mix (not a "slab"-type) concrete mix should be use. Air-entrainment (5 ± 2 percent) of the concrete should be provided. Sulfate resistant concrete is not required. A maximum joint spacing of 12 feet is also recommended. Reinforcement of the concrete should be provided as recommended by the structural engineer.

5.4.7 Exterior Concrete Slabs and Sidewalks

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness with 6" x 6" – W1.4/W1.4 welded wire fabric, placed in the middle of slab. It is recommended that control joints be placed in two directions spaced the numeric equivalent roughly 24 times the thickness of the slab in inches (e.g., a 4-inch slab would have control joints at 96 inch [8 feet] centers). These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices typically utilized in construction.

Presaturation of flatwork subgrade should be verified to be a minimum of 100% of the soils optimum moisture to a depth of 12 inches for soils having a "very low" expansive index potential.

5.5 POST CONSTRUCTION CONSIDERATIONS

5.5.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be warranted and advisable. We could discuss these issues, if desired, when plans are made available.

5.5.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

5.6 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications, retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. Additional recommendations may be necessary based on these reviews. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement, and collect soil samples for laboratory testing when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Observe and test the fill for field density and relative compaction.

- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

6. LIMITATIONS

The scope of our evaluation is limited to the area explored that is shown on the Geotechnical Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our proposal (Proposal No. P-1200622-SDCO2) dated April 25, 2023 and geotechnical engineering standards normally used on similar projects in this region.

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusions and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

7. SELECTED REFERENCES

Alta California Geotechnical Inc., 2021, "Preliminary Geotechnical Investigation, Las Posas Road and Armorlite Drive, City of San Marcos, California," Proposal No. 1-0371, dated March 31, 2021.

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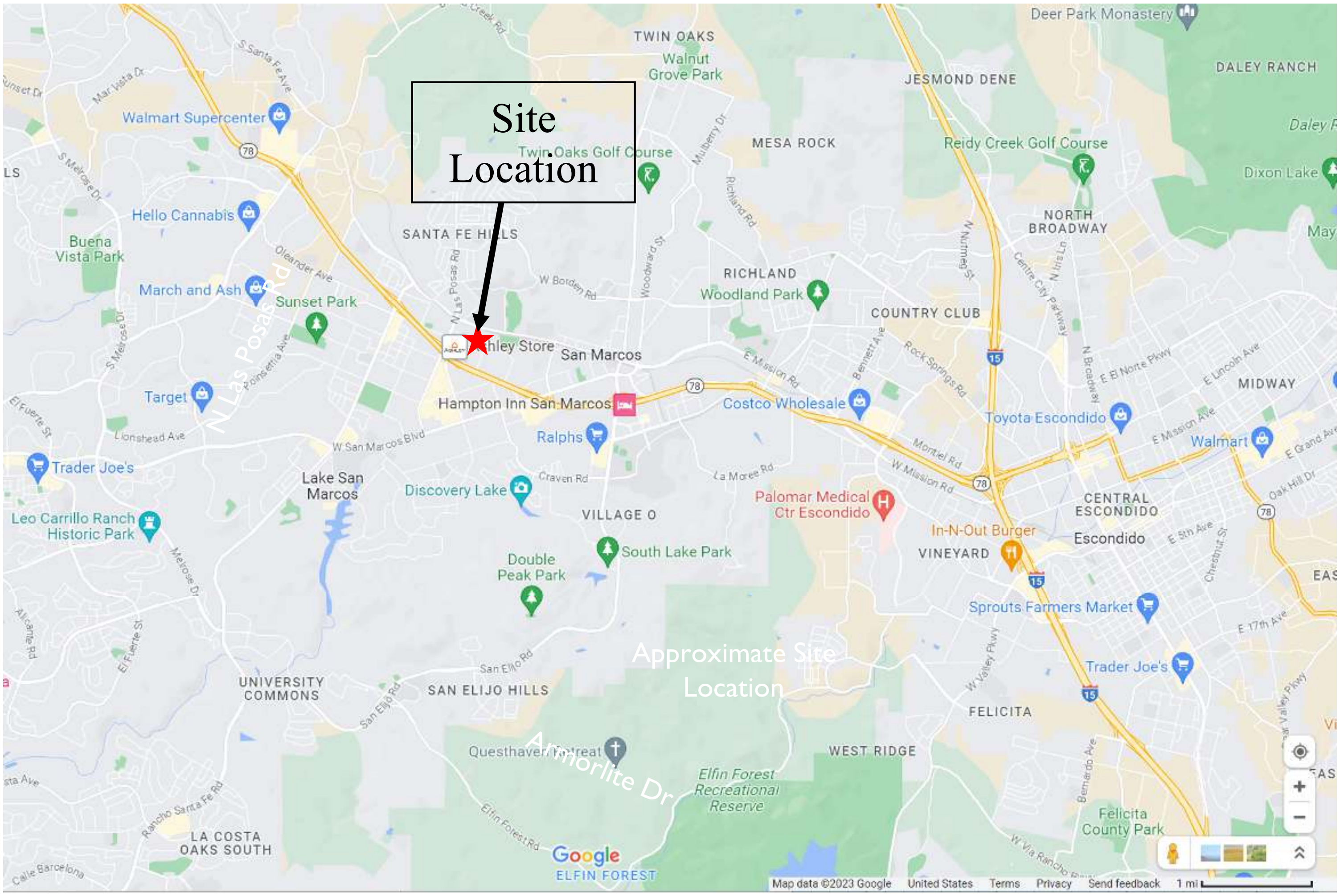


GEOTEK

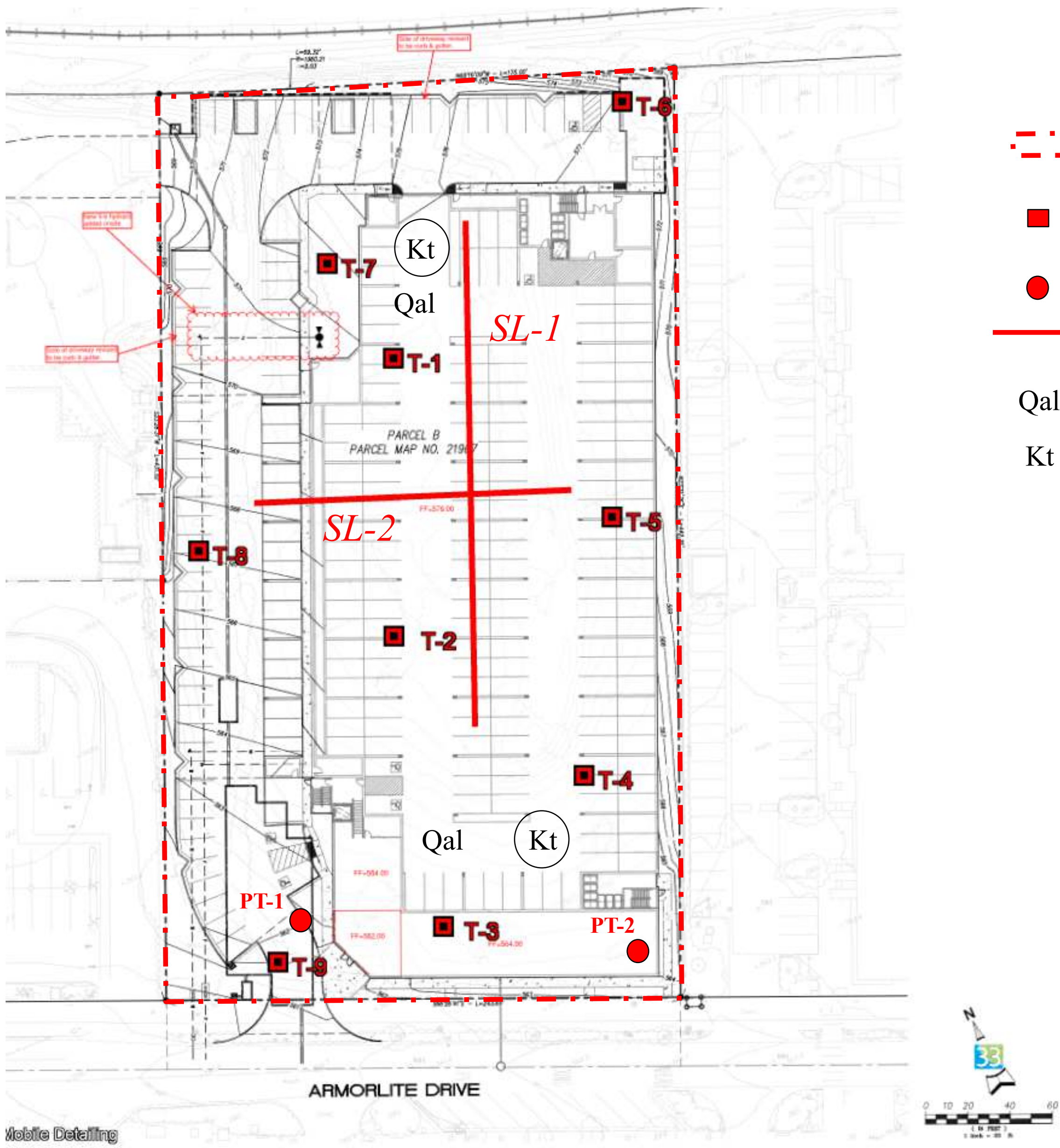
GEOTECHNICAL | ENVIRONMENTAL | MATERIALS

1384 Poinsettia Avenue, Suite A, Vista, CA 92081
(760) 599-0509 (phone) / (760) 599-0593 (FAX)

FIGURE 1

SITE LOCATION
225 N LAS POSAS RD.
SAN MARCOS, CALIFORNIA

Project No.:	Report Date:	Drawn By:
3865-SD	05/2023	CDL



Explanation

		Approximate Site Boundary
	T-9	Approximate Location of Test Pit
	PT-2	Approximate Location of Percolation Test
	PSL-2	Approximate Location of Seismic Refraction
Qal		Colluvium/Alluvium, undifferentiated
Kt		Granitics, Circled Where Buried

Armorlite

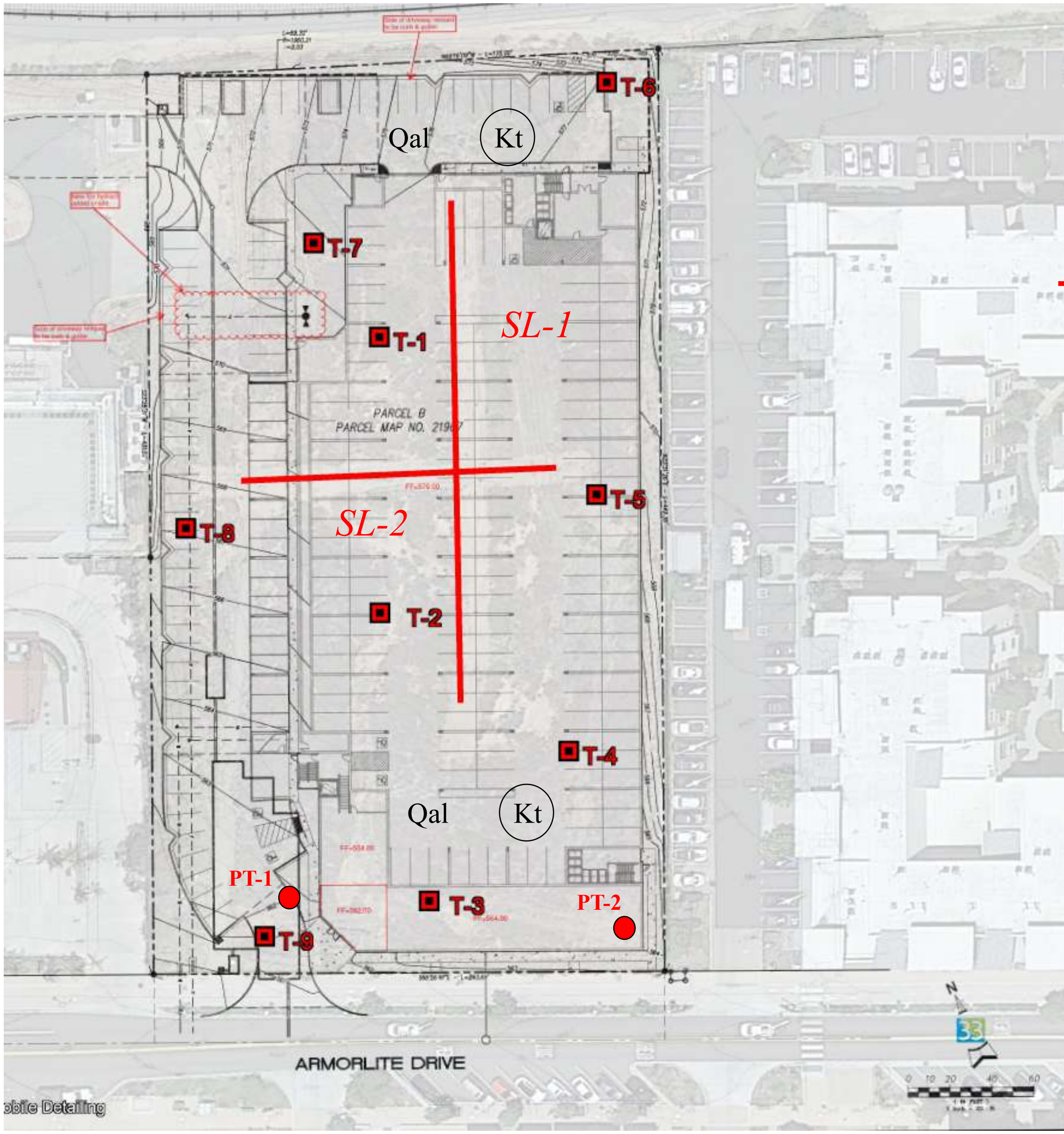
Figure 2

Geotechnical Map

PN: 3865-SD

May 2023

GEOTEK



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Armorlite

Geotechnical Map

Figure 2

PN: 3865-SD

May 2023

GEOTEK

APPENDIX A

TEST PITS

AND

SEISMIC REFRACTION EVALUATION REPORT

GEOTEK

Project No.	1-0371
Date Excavated	February 12th, 2021
Logged by	JC
Equipment	JD 310

TABLE I

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
TP-1	0.0-1.0	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, dark brown, slightly moist, medium dense, few fine to coarse gravel <3”, some roots
	1.0-3.4		**TONALITE** (Kt): BEDROCK, fine to medium grained, slightly orange tan, dry, hard, slightly to moderately weathered. @1.8ft. grayish tan, slightly weathered.
			TOTAL DEPTH 3.4 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-2	0.0-2.5	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, brown, moist, medium dense, some roots. @0.4ft. few fine to coarse gravel <3”, few boulders <2.0ft.
	2.5-4.0		**TONALITE** (Kt): BEDROCK, fine to coarse grained, grayish tan, dry, hard, moderately weathered. @3.4ft. slightly orange tan, slightly weathered.
			TOTAL DEPTH 4.0 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-3	0.0-1.5	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, brown, moist, medium dense, some roots. @0.9ft. slightly orange brown, trace pores.
	1.5-3.3		**TONALITE** (Kt): BEDROCK, fine to coarse grained, grayish tan, dry, hard, slightly weathered.
			TOTAL DEPTH 3.3 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-4	0.0-1.3	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, tannish brown, moist, medium dense, some roots. @0.7ft. slightly orange brown, trace pores.
	1.3-3.7		**TONALITE** (Kt): BEDROCK, fine to coarse grained, orange tan, dry, hard, slightly weathered.
			TOTAL DEPTH 3.7 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-5	0.0-3.5	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, tan, moist, medium dense, some roots. @1.3ft. slightly orange tan, some pores. @2.2ft. some fine to coarse gravel <3”.
	3.5-5.2		**TONALITE** (Kt): BEDROCK, fine to coarse grained, orange tan, slightly moist, hard, moderately weathered. @4.4ft. grayish tan, slightly weathered.
			TOTAL DEPTH 5.2 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-6	0.0-1.8	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, brown, moist, medium dense, some roots. @0.5ft. trace pores.
	1.8-3.0		**TONALITE** (Kt): BEDROCK, fine to coarse grained, reddish brown, dry, hard, moderately weathered. @2.7ft. tannish gray, very hard.
			TOTAL DEPTH 3.0 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-7	0.0-1.9	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, brown, moist, medium dense, some roots. @010ft. reddish brown, trace pores.
	1.9-3.5		**TONALITE** (Kt): BEDROCK, fine to coarse grained, reddish brown, dry, hard, moderately weathered. @2.4ft. tan, slightly weathered.
			TOTAL DEPTH 3.5 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-8	0.0-3.1	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, dark brown, moist, loose, some roots. @0.5ft. medium to coarse grained, dry, medium dense, trace pores. @1.2ft. orange tan. @2.8ft. trace clay.
	3.1-4.7	SM	**TONALITE** (Kt): BEDROCK, fine to coarse grained, reddish brown, dry, hard, moderately weathered. @3.6ft. slightly weathered.
			TOTAL DEPTH 4.7 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
TP-9	0.0-2.5	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, tannish brown, moist, medium dense, some roots. @0.7ft. slightly orange brown, trace pores.
	2.5-3.8		**TONALITE** (Kt): BEDROCK, fine to coarse grained, orange tan, dry, hard, slightly weathered.
			TOTAL DEPTH 3.8 FEET NO GROUNDWATER ENCOUNTERED SLIGHT CAVING ABOVE 4.0 FEET

Test Pit No.	Depth (ft.)	USCS	Description
PT-1	0.0-1.4	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, tannish brown, moist, medium dense, some roots. @0.7ft. slightly orange brown, trace pores.
	1.4-4.0		**TONALITE** (Kt): BEDROCK, fine to coarse grained, orange tan, dry, hard, slightly weathered.
			TOTAL DEPTH 4.0 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Test Pit No.	Depth (ft.)	USCS	Description
PT-2	0.0-2.1	SM	**ALLUVIUM**(Qal): SILTY SAND, very fine to fine grained, tannish brown, moist, medium dense, some roots. @1.5ft. slightly orange brown, few fine gravel <3/4”, trace pores.
	2.1-4.5		**TONALITE** (Kt): BEDROCK, fine to coarse grained, orange tan, dry, hard, slightly weathered. TOTAL DEPTH 4.5 FEET NO GROUNDWATER ENCOUNTERED NO CAVING OBSERVED

Categorization of Infiltration Feasibility Condition			Form I-8
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: An infiltration testing evaluation was performed at the site, by borehole method to obtain percolation rates that were converted to infiltration rates via Porchet method. Site specific testing resulted in rates of 0.1 and 0.4 inches per hour. GeoTek, Inc. "Preliminary Geotechnical Evaluation, Eastern Portion of 225 N. Las Posas Road, San Marcos, California 92069," Project No. 3685-SD, dated May 30, 2023. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		
Provide basis: Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Form I-8 Page 2 of 4</th></tr>
<tr><th>Criteri
a</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	No

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

Form I-8 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

An infiltration testing evaluation was performed at the site, by borehole method to obtain percolation rates that were converted to infiltration rates via Porchet method.
Site specific testing resulted in rates of 0.1 and 0.4 inches per hour.

GeoTek, Inc. "Preliminary Geotechnical Evaluation, Eastern Portion of 225 N. Las Posas Road, San Marcos, California 92069," Project No. 3685-SD, dated May 30, 2023.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

Based on the underlying geology, granitic rock is shallow and will create a groundwater mounding affect.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

<table>
<tr><th colspan="4">Form I-8 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
Based on the underlying geology, granitic rock is shallow and will create a shallow ground water condition. A review of Geotracker.com did not reveal environmental concerns immediately adjacent to the property.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Although GeoTek does not practice in water rights consultation, infiltration of surface waters into the subsurface does not appear to violate downstream water rights.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result *</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings

Factor of Safety and Design Infiltration Rate Worksheet					Form I-9
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25	2	0.5
		Predominant soil texture	0.25	2	0.5
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.5
B	Design	Level of pretreatment/ expected sediment loads	0.5	To Be Determined by Civil Engineer	
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{total} = S_A \times S_B$					
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)				0.1 inches per hour	
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$					
Supporting Data					
Briefly describe infiltration test and provide reference to test forms: An infiltration testing evaluation was performed at the site, by borehole method to obtain percolation rates that were converted to infiltration rates via Porchet method. GeoTek, Inc. "Preliminary Geotechnical Evaluation, Eastern Portion of 225 N. Las Posas Road, San Marcos, California 92069," Project No. 3685-SD, dated May 30, 2023.					

SEISMIC REFRACTION STUDY

1244 ARMORLITE DRIVE

San Marcos, California

PREPARED FOR:

Alta California Geotechnical Inc.
170 North Maple Street, Suite 108
Corona, California 92880

PREPARED BY:

Atlas Technical Consultants
6280 Riverdale Street
San Diego, California 92120

March 30, 2021

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March 30, 2021

Atlas No. 121096SWG
Report No. 1

MR. JAMES COYNE
ALTA CALIFORNIA GEOTECHNICAL INC.
170 NORTH MAPLE STREET, SUITE 108
CORONA, CALIFORNIA 92880

Subject: **Seismic Refraction Study**
1244 Armorlite Drive
San Marcos, California

Dear Mr. Coyne:

In accordance with your authorization, Atlas Technical Consultants has performed a seismic refraction study pertaining to the 1244 Armorlite Drive project located in San Marcos, California. Specifically, our evaluation consisted of performing two seismic P-wave refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas studied, and to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on March 12, 2021. This data report presents our methodology, equipment used, analysis, and results.

If you have any questions, please call us at (619) 280-4321.

Respectfully submitted,
Atlas Technical Consultants LLC

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TMB:KJA:MDE:PFL:ds

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CONTENTS

TABLE

FIGURES

1. INTRODUCTION

In accordance with your authorization, Atlas Technical Consultants has performed a seismic refraction study pertaining to the 1244 Armorlite Drive project located in San Marcos, California. Specifically, our evaluation consisted of performing two seismic P-wave refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas studied, and to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on March 12, 2021. This data report presents our methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

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

3. SITE AND PROJECT DESCRIPTION

The project site is located in an undeveloped lot located at 1244 Armorlite Drive in San Marcos, California (Figure 1). The seismic traverses were performed at locations provided at your specified locations. The area is an undeveloped lot with a perimeter fence and thick, waist high vegetation. Figures 2 and 3 depict the general site conditions in the area of the seismic traverses.

Based on our discussions with you, it is our understanding that the results of our study may be used in the formulation of design and construction parameters for the project.

4. STUDY METHODOLOGY

A seismic P-wave (compression wave) refraction study was conducted at the project to develop subsurface velocity profiles of the areas studied, and to assess the depth to bedrock and apparent rippability of the subsurface materials. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Two seismic traverses (SL-1 and SL-2) were conducted in the study area. The general location and length of the lines were determined by surface conditions, site access, and your requested depth of investigation. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions, or boulders can also result in the misinterpretation of the subsurface conditions. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth of the length of the spread.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2018), as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristic, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

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

Table 1 – Rippability Classification

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

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. DATA ANALYSIS

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

6. RESULTS AND CONCLUSIONS

As previously indicated, two seismic traverses were performed at your specified locations. Figures 4a and 4b present the P-wave velocity models generated from our analysis. The results from our seismic study revealed distinct layers/zones in the near surface that might represent soil overlying granitic bedrock with varying degrees of weathering. 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 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 similarly 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. Atlas 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

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

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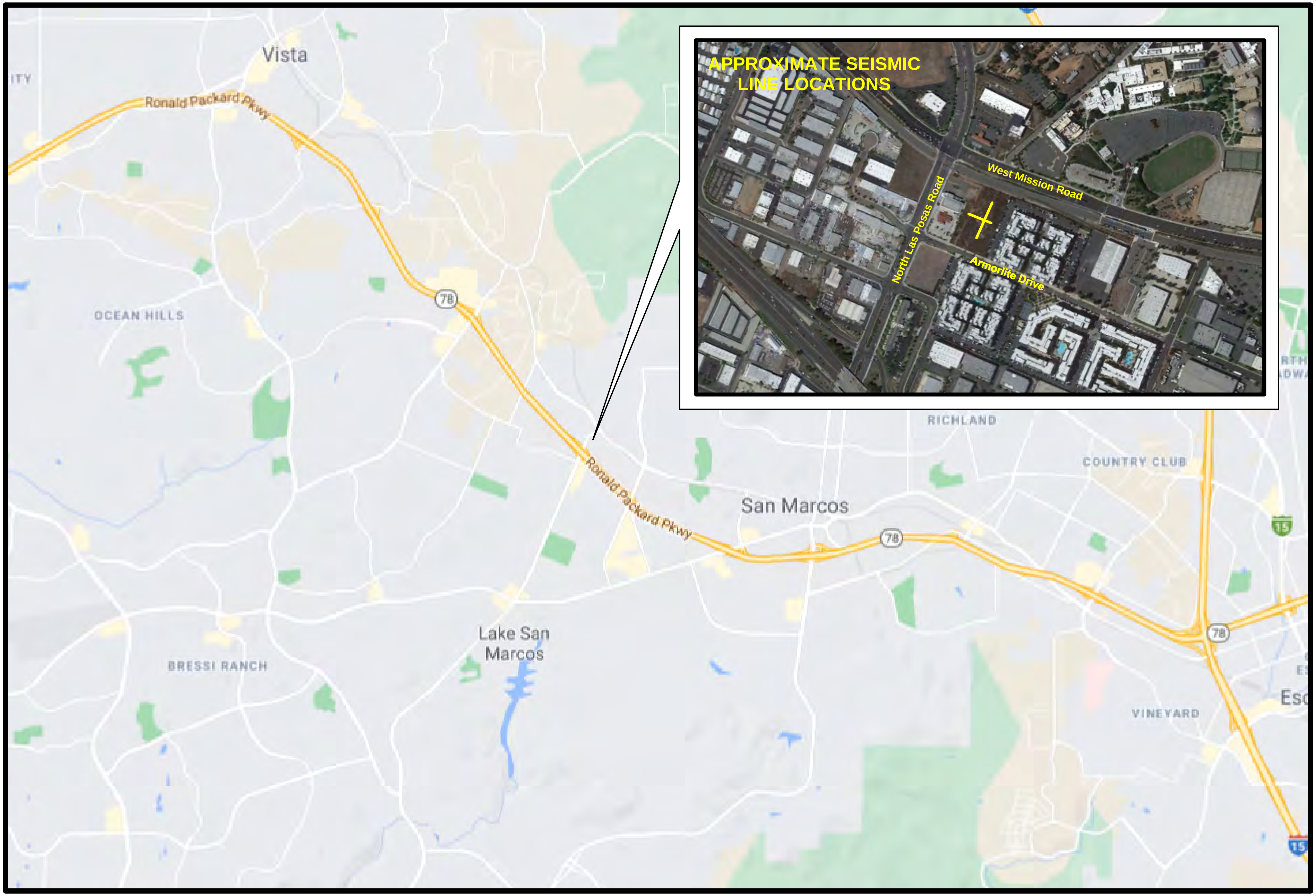
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.



SITE LOCATION MAP

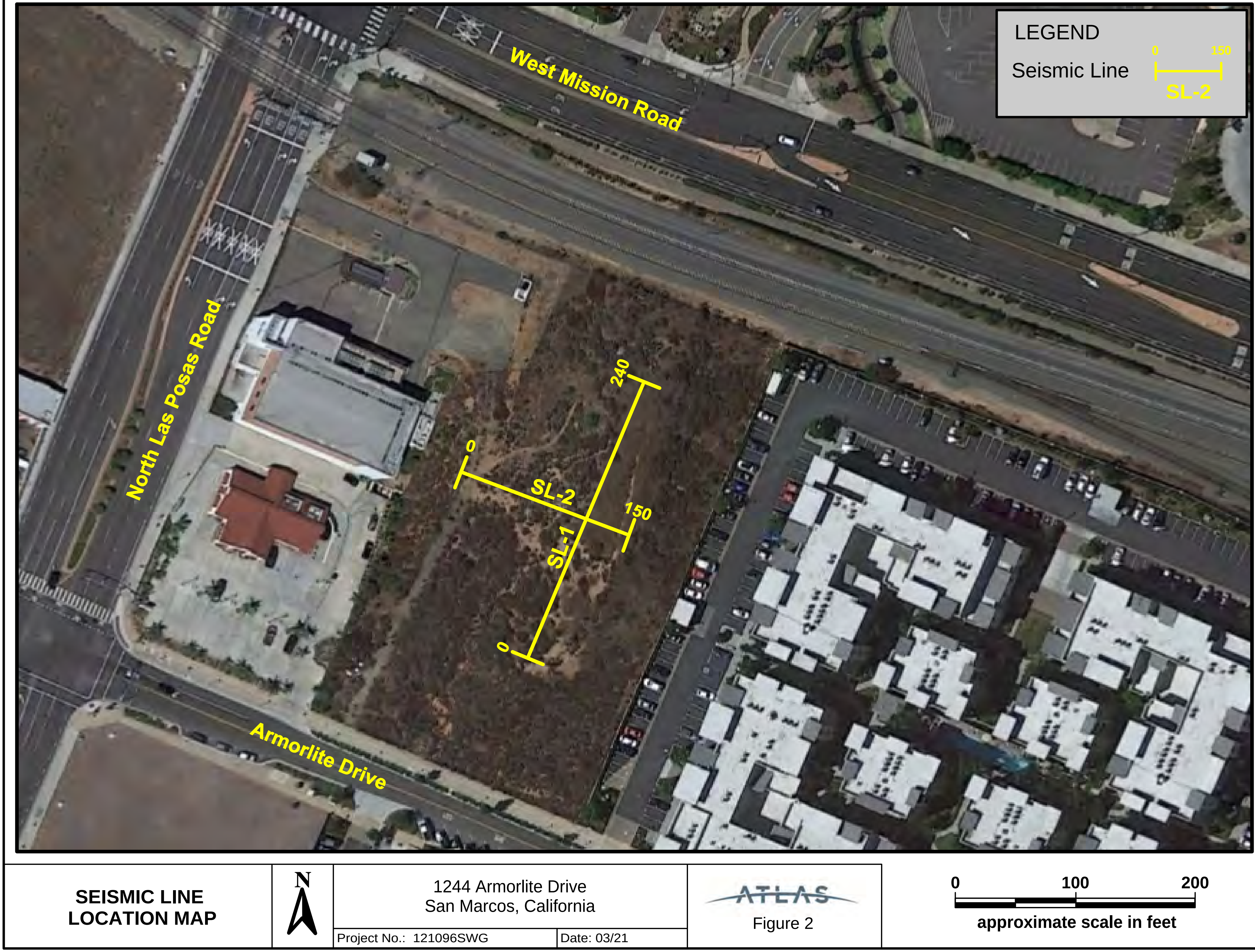
N

1244 Armorlite Drive
San Marcos, California

Project No.: 121096SWG | Date: 03/21

ATLAS

Figure 1



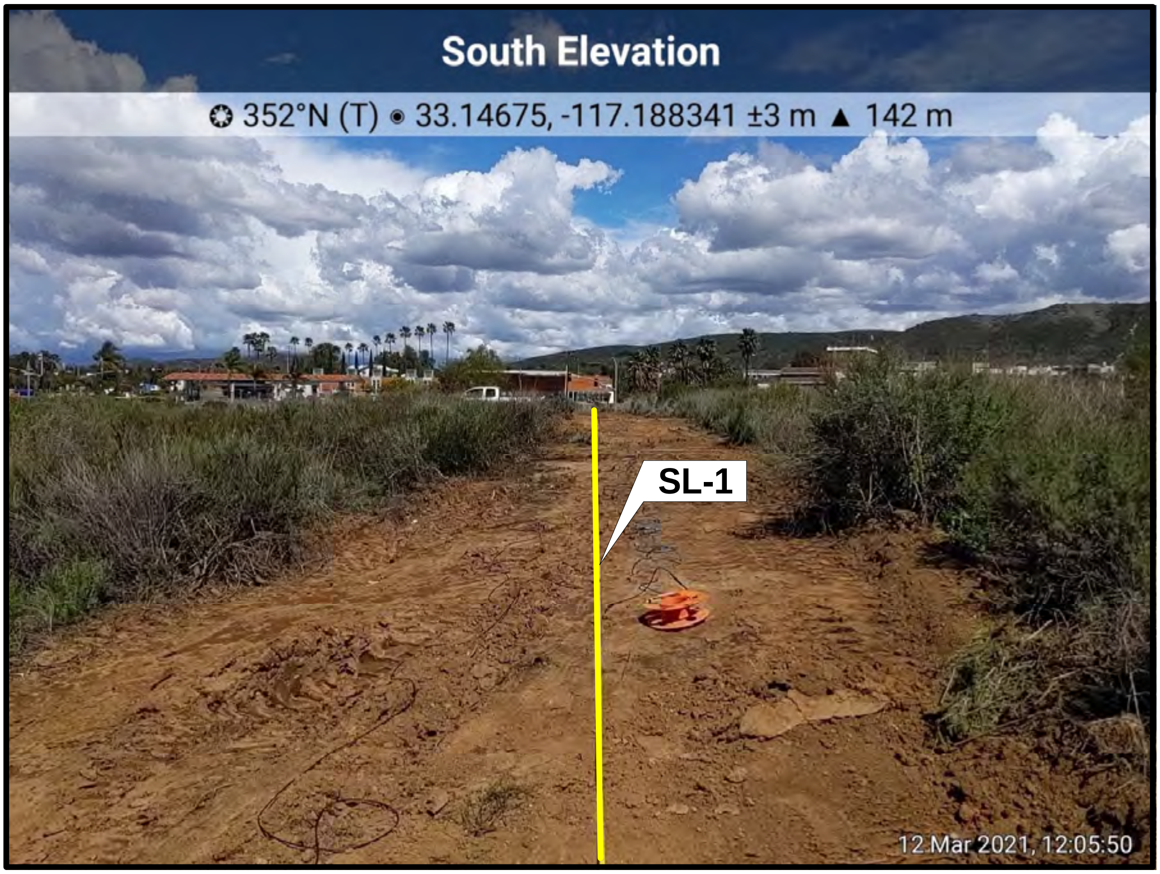
SEISMIC LINE LOCATION MAP

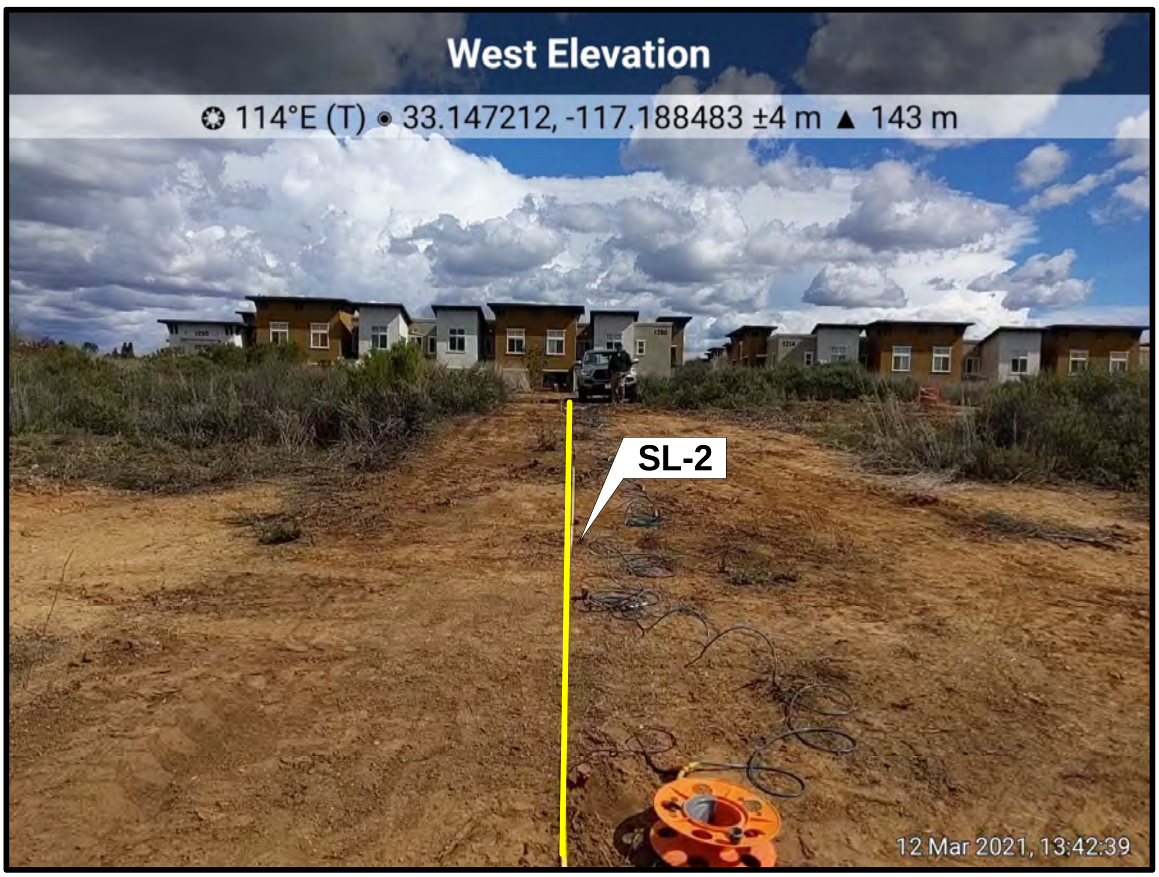
1244 Armorlite Drive
San Marcos, California

Project No.: 121096SWG | Date: 03/21

ATLAS

Figure 2

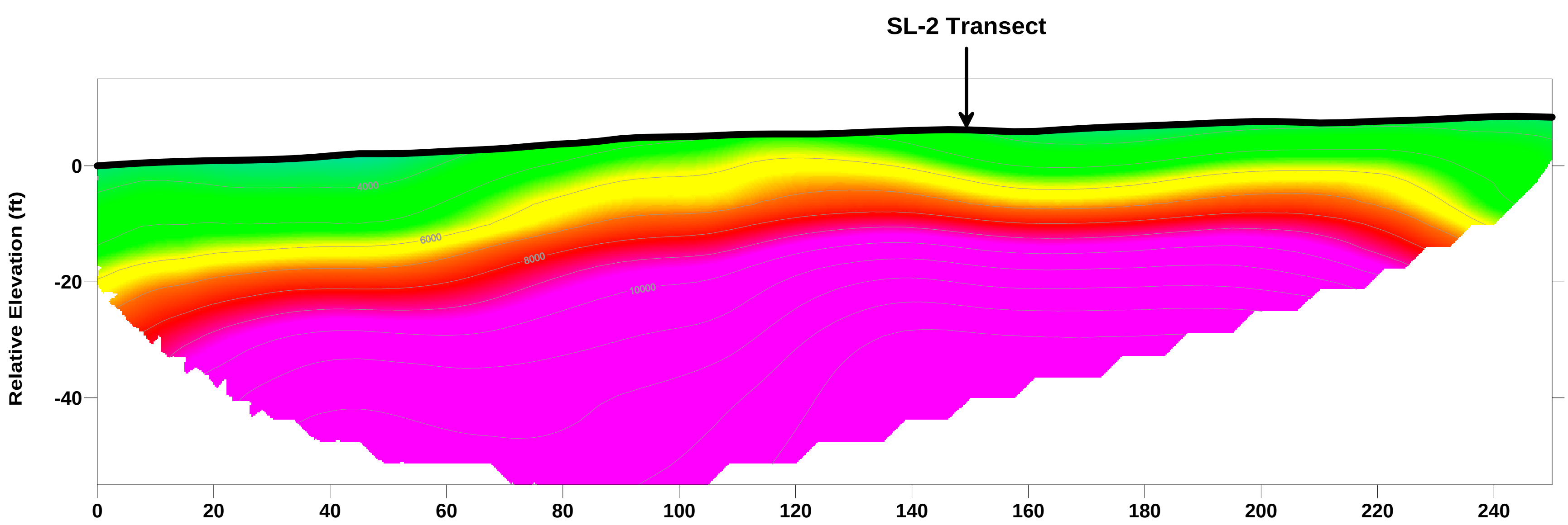


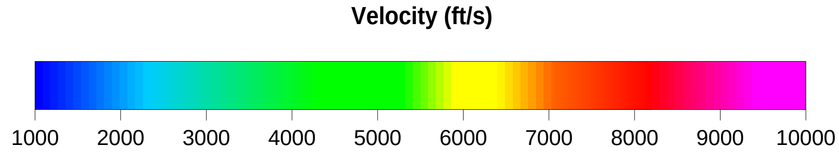


SITE PHOTOGRAPHS	1244 Armorlite Drive San Marcos, California	ATLAS Figure 3
	Project No.: 121096SWG Date: 03/21	

TOMOGRAPHY MODEL

SL-1

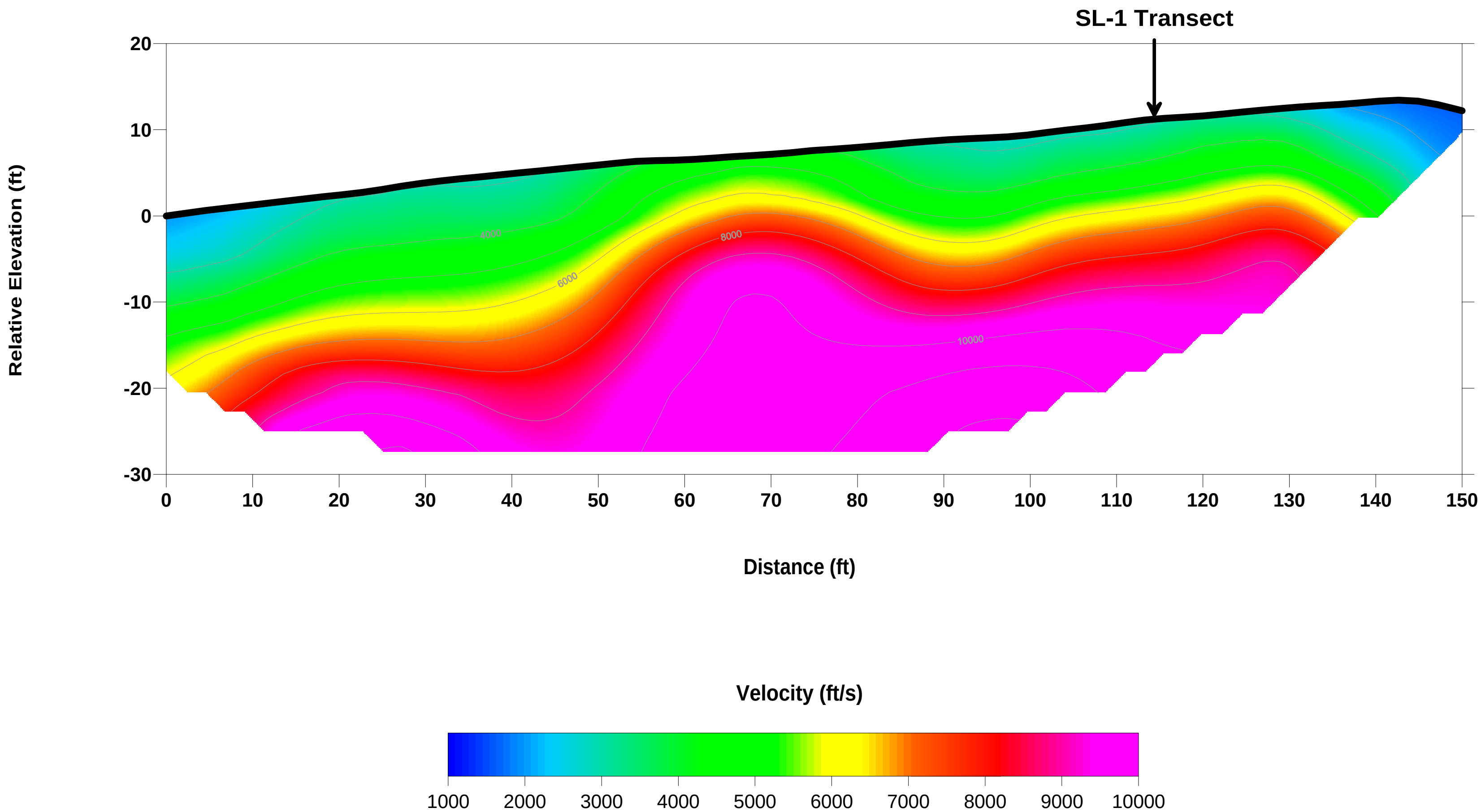




SEISMIC PROFILE (SL-1)	1244 Armorlite Drive San Marcos, California	ATLAS Figure 4a	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 121096SWG / Date: 03/21		

TOMOGRAPHY MODEL

SL-2



SEISMIC PROFILE (SL-2)	1244 Armorlite Drive San Marcos, California	ATLAS Figure 4b	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 121096SWG / Date: 03/21		

APPENDIX B

RESULTS OF LABORATORY TESTING

LABORATORY TESTING

The following laboratory tests were performed on representative samples in accordance with the applicable latest standards or methods from the ASTM, California Building Code (CBC) and California Department of Transportation.

Classification

Soils were classified with respect to the Unified Soil Classification System (USCS) in accordance with ASTM D-2487 and D-2488.

Particle Size Analysis

Modified hydrometer testing was conducted to aid in classification of the soil. The results of the particle size analysis are presented in Table 7-1 and Table C.

Expansion Index Tests

One (1) expansion index test was performed to evaluate the expansion potential of typical on-site soil. Testing was carried out in general conformance with ASTM Test Method D-4829. The results are presented in Table C.

Chemical Analyses

Chemical testing was performed on one select sample. The results of this test (sulfate content, resistivity, chloride content and pH) is presented on Table C.

TABLE C
SUMMARY OF LABORATORY TEST DATA
P.N. 1-0371

Boring/Pit No.	Depth (Feet)	Soil Description	Group Symbol - Unified Soil Classification System	Maximum Dry Density		Direct Shear	Grain Size Analysis				Expansion Index	Sulfate Content (%)	Consolidation	Other Tests Remarks
				Maximum Density (pcf)	Optimum Moisture (%)		Gravel (% + No. 4 Screen)	% Sand	%Silt (0.074 to 0.005mm)	% Clay (-0.005 mm)				
T-3	2	Bedrock (Kt)	-	-	-	-	0	56	29	15	1	ND	-	Min. Resistivity: 7,600 OHM-CM Chloride: 20ppm PH: 6.37

Alta California Geotechnical, Inc.

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

GEOTEK

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, the California Building Code, CBC (2022) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor’s overall level of efforts during grading. The contractor’s personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor’s responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed (see Plates G-1, G-2 and G-3) unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Subdrainage

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to schematic diagrams G-1 and G-5, and be acceptable to our representative.
2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.
3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.
4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.
5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.
6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.
7. Consideration should be given to having subdrains located by the project surveyors.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
   a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
   b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
   a) They are not placed in concentrated pockets;
   b) There is a sufficient percentage of fine-grained material to surround the rocks;
   c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal (see Plate G-4). On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If

significant oversize materials are encountered during construction, these guidelines should be requested.

6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

Keyways, Buttress and Stabilization Fills

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates G-2, G-3). As the fill is elevated, it should be benched through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates G-1, G-2, and G-3).
2. Fill over cut slopes should be constructed in the following manner:
    a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
    b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction), and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
    c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate G-3 for schematic details.)
3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate G-2.

4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate G-2.
5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate G-3 for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate G-5 shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions, and need to maintain a minimum fill width and provide working room for the equipment.
2. On longer slopes, backcuts and keyways should be excavated in maximum 250 feet long segments. The specific configurations will be determined during construction.
3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent, whichever is greater.
4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate G-5.
5. Benching of backcuts during fill placement is required.

Lot Capping

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advice based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.
2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g. lots above stabilization fills, along daylight lines, above natural slopes, etc.) should be capped with a minimum three foot thick compacted fill blanket.
3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

ROCK PLACEMENT AND ROCK FILL GUIDELINES

If large quantities of oversize material would be generated during grading, it's likely that such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

Limited Larger Rock

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inches) should be placed in windrows.
   a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.

   b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).
   c) The maximum rock size allowed in windrows is four feet
2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate G-4).
3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.
4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
   a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
   b) The over size rock trenches should be no closer together than 15 feet from any slope face.
   c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
   d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

Structural Rock Fills

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may affect foundation design.
2. Rock fills are required to be placed in horizontal layers that should **not exceed two feet in thickness, or the maximum rock size present, which ever is less**. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:
   a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill ,
   b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,
   c) the portion of the rock above grade is covered with a second lift.
3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

Compaction Procedures

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.
    a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.
    b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.
    c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). (Water should be applied before and during spreading.)

2. Rock fill should be generously watered (sluiced)
    a) Water should be applied by water trucks to the:
        i) dump piles,
        ii) front face of the lift being placed and,
        iii) surface of the fill prior to compaction.
    b) No material should be placed without adequate water.
    c) The number of water trucks and water supply should be sufficient to provide constant water.
    d) Rock fill placement should be suspended when water trucks are unavailable:
        i) for more than 5 minutes straight, or,
        ii) for more than 10 minutes/hour.

3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber tired compactors may be required.
    a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
    b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.

4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:
    a) the general segregation of rock size,
    b) for any unfilled spaces between the large blocks, and
    c) the matrix compaction and moisture content.

5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
    a) A lift should be constructed by the methods proposed, as proposed

6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractor's procedures.

7. A minimum horizontal distance of 15 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 15 feet should be built of conventional fill materials.

Piping Potential and Filter Blankets

Where conventional fill is placed over rock fill, the potential for piping (migration) of the fine grained material from the conventional fill into rock fills will need to be addressed.

The potential for particle migration is related to the grain size comparisons of the materials present and in contact with each other. Provided that 15 percent of the finer soil is larger than the effective pore size of the coarse soil, then particle migration is substantially mitigated. This can be accomplished with a well-graded matrix material for the rock fill and a zone of fill similar to the matrix above it. The specific gradation of the fill materials placed during grading must be known to evaluate the need for any type of filter that may be necessary to cap the rock fills. This, unfortunately, can only be accurately determined during construction.

In the event that poorly graded matrix is used in the rock fills, properly graded filter blankets 2 to 3 feet thick separating rock fills and conventional fill may be needed. As an alternative, use of two layers of filter fabric (Mirafi 700 x or equivalent) could be employed on top of the rock fill. In order to mitigate excess puncturing, the surface of the rock fill should be well broken down and smoothed prior to placing the filter fabric. The first layer of the fabric may then be placed and covered with relatively permeable fill material (with respect to overlying material) 1 to 2 feet thick. The relative permeable material should be compacted to fill standards. The second layer of fabric should be placed and conventional fill placement continued.

Subdrainage

Rock fill areas should be tied to a subdrainage system. If conventional fill is placed that separates the rock from the main canyon subdrain, then a secondary system should be installed. A system consisting of an adequately graded base (3 to 4 percent to the lower side) with a collector system and outlets may suffice.

Additionally, at approximately every 25 foot vertical interval, a collector system with outlets should be placed at the interface of the rock fill and the conventional fill blanketing a fill slope.

Monitoring

Depending upon the depth of the rock fill and other factors, monitoring for settlement of the fill areas may be needed following completion of grading. Typically, if rock fill depths exceed 40 feet, monitoring would be recommend prior to construction of any settlement sensitive improvements. Delays of 3 to 6 months or longer can be expected prior to the start of construction.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractor's responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.

2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractor's procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractor's attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

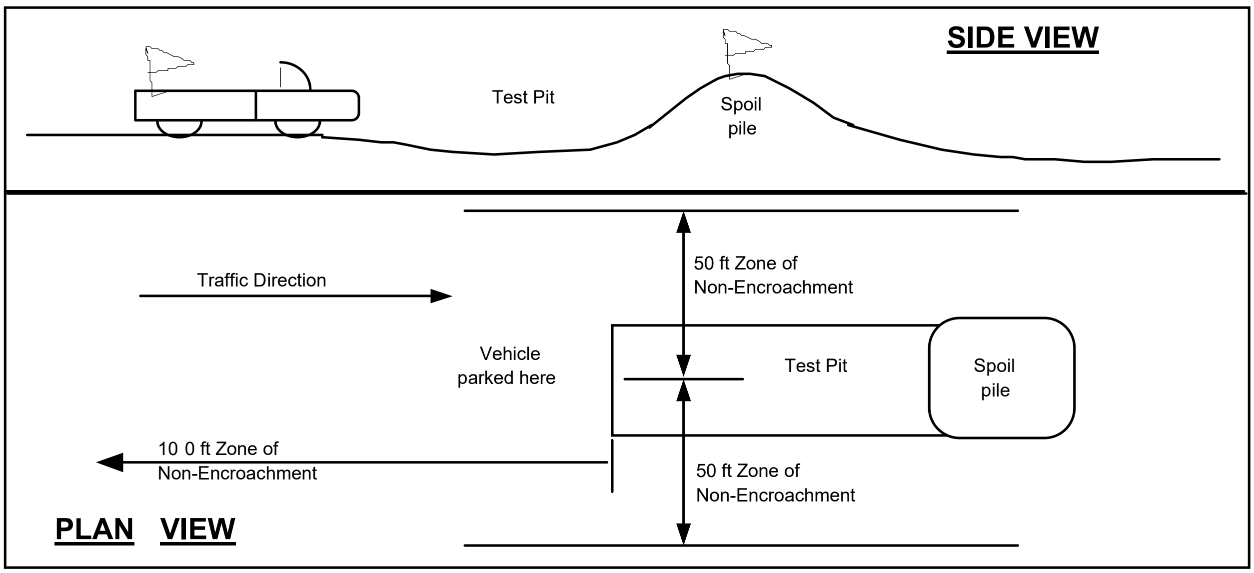
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractor’s representative will then be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

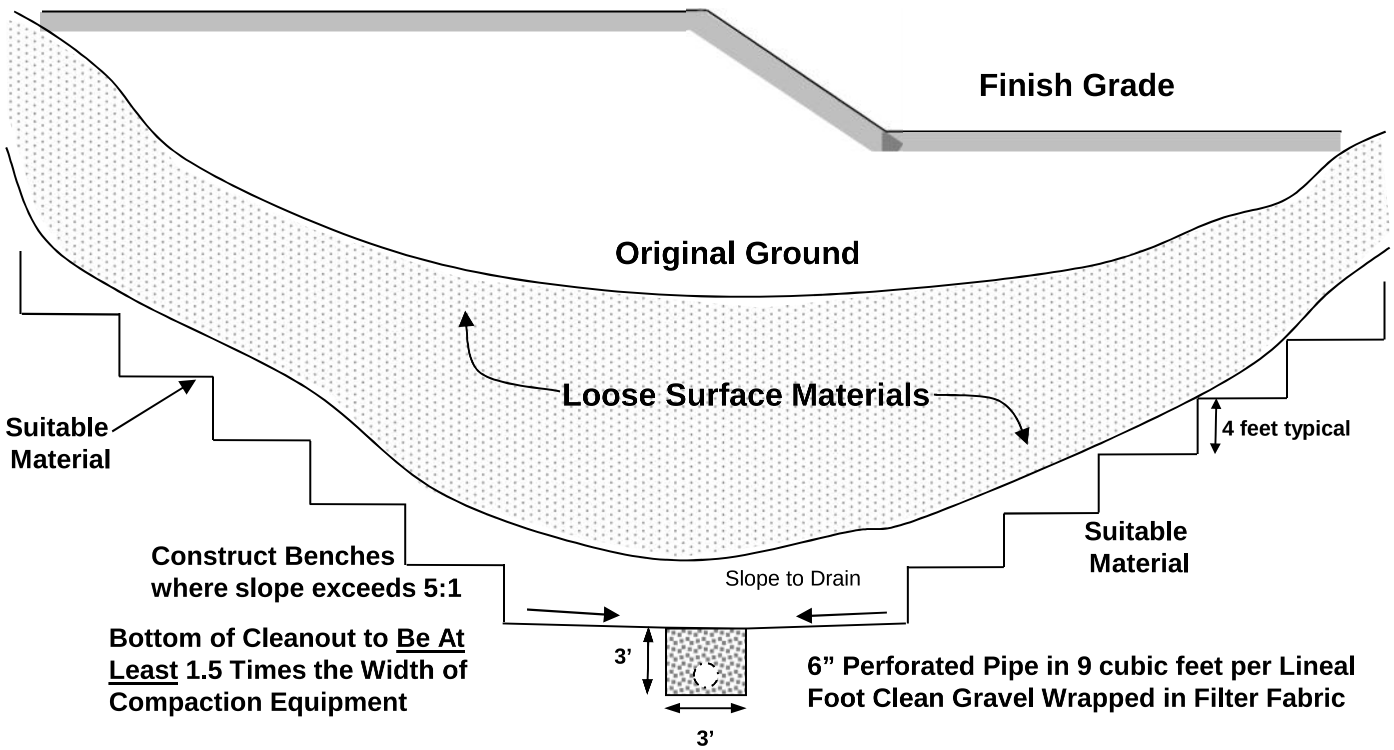
In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor’s representative will then be contacted in an effort to affect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

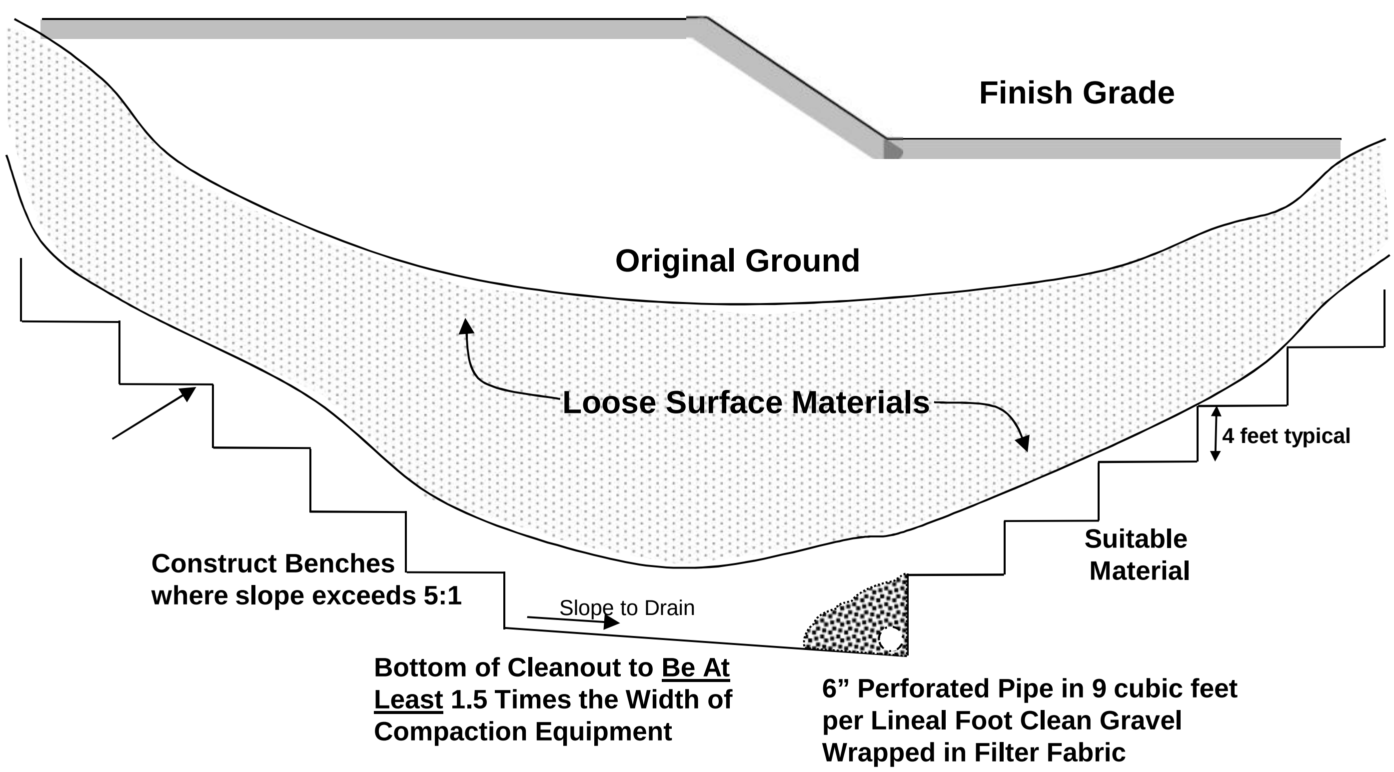
In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technician’s attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

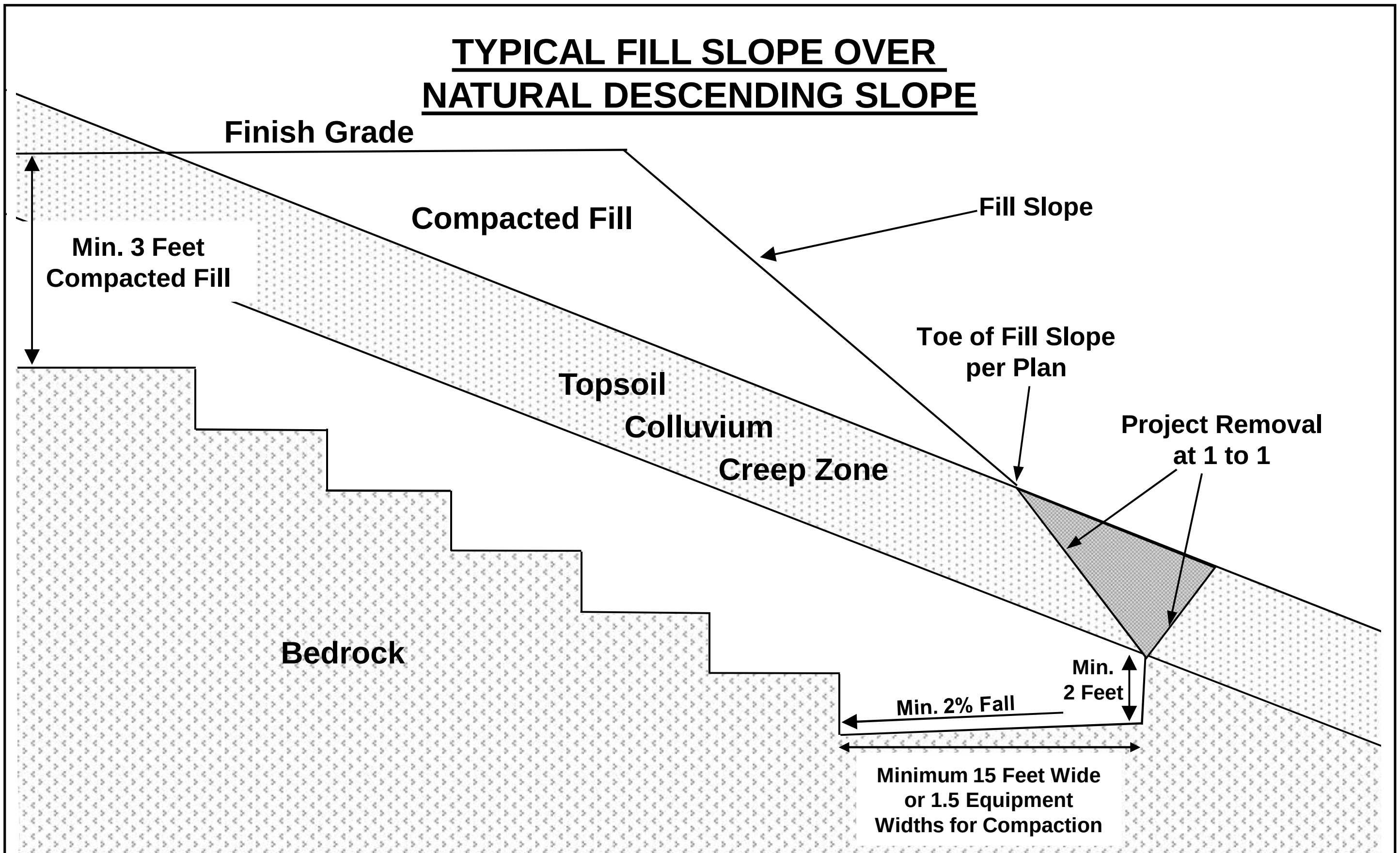
The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

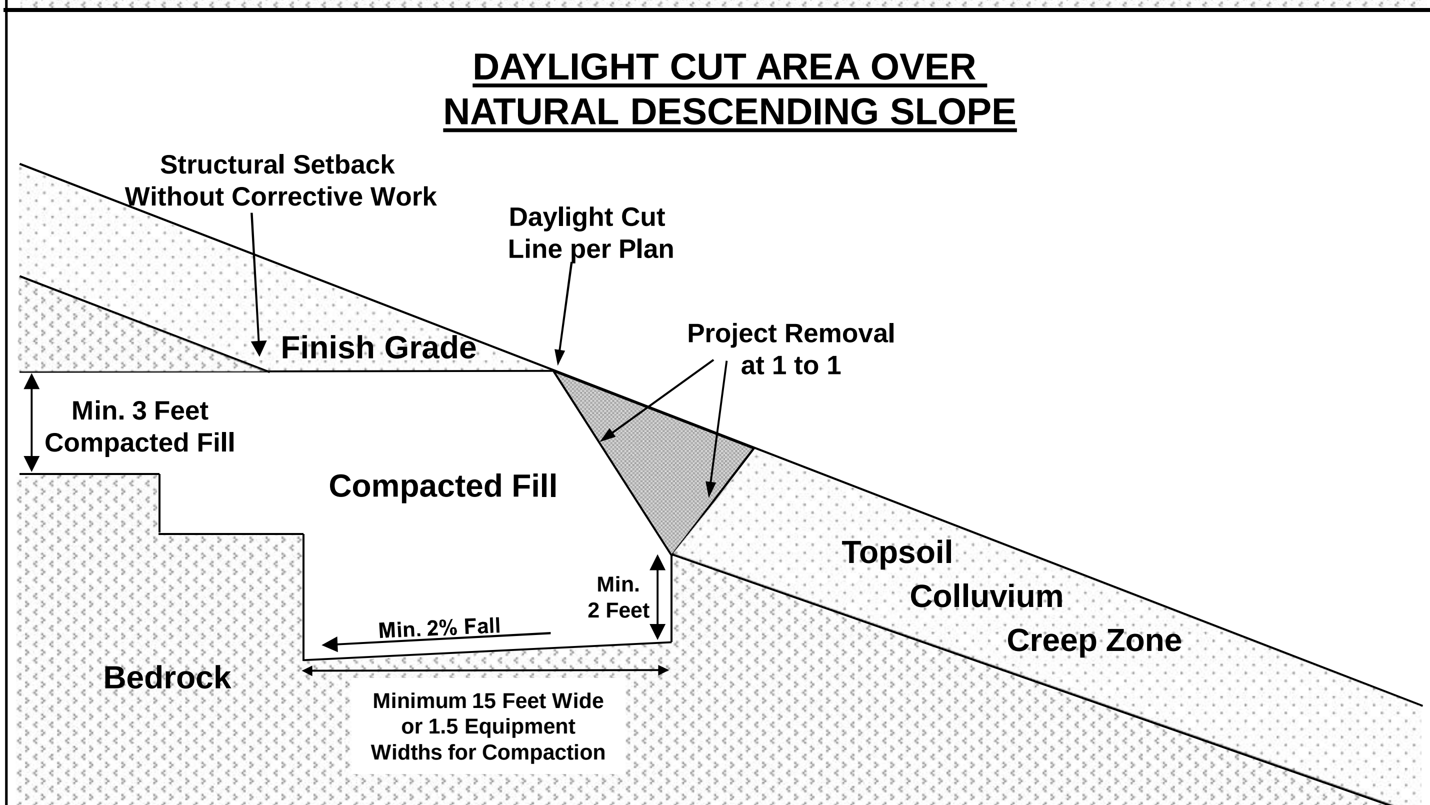
ALTERNATES



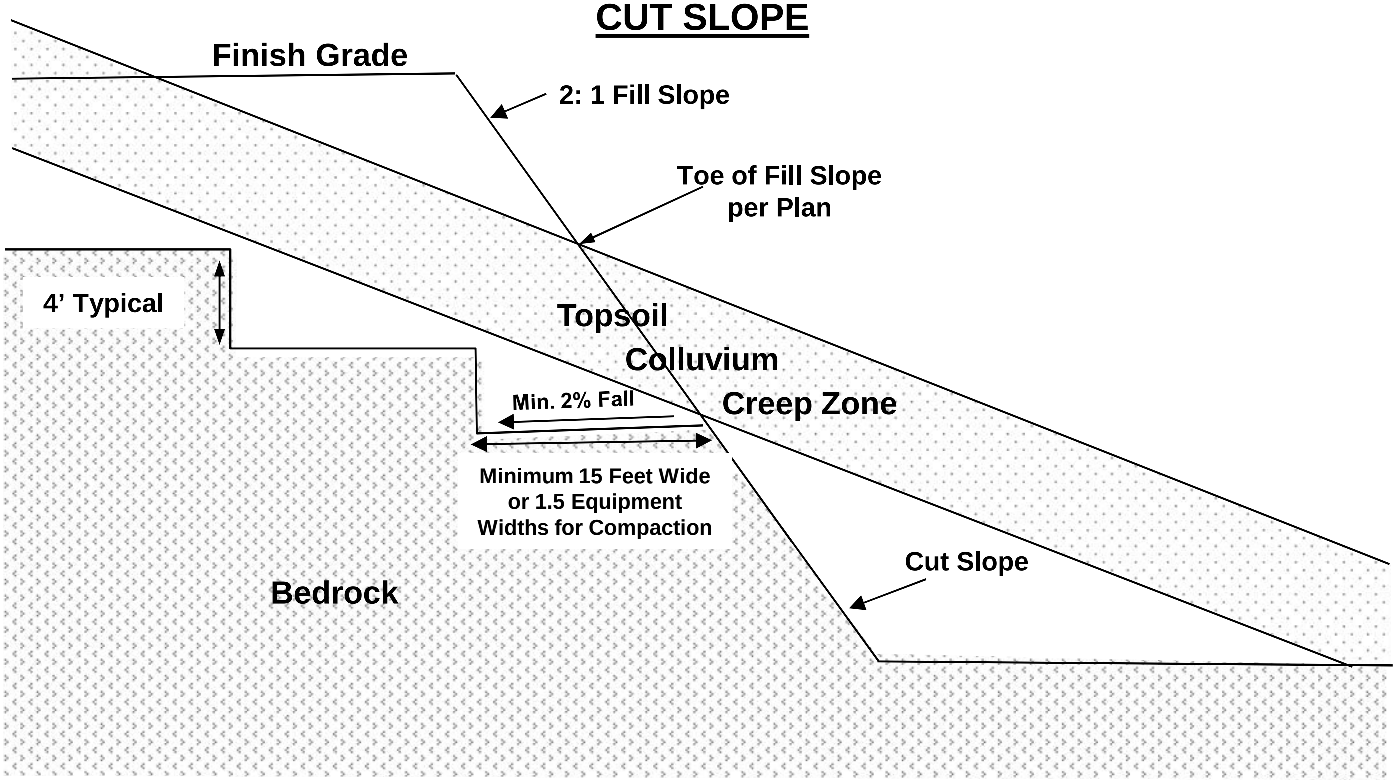


GEOTEK	1384 Poinsettia Avenue, Suite A Vista, California 92083	TYPICAL CANYON CLEANOUT	STANDARD GRADING GUIDELINES PLATE G-1


TYPICAL FILL SLOPE OVER
NATURAL DESCENDING SLOPE
Finish Grade
Compacted Fill
Fill Slope
Min. 3 Feet
Compacted Fill
Toe of Fill Slope
per Plan
Topsoil
Colluvium
Creep Zone
Project Removal
at 1 to 1
Bedrock
Min.
2 Feet
Min. 2% Fall
Minimum 15 Feet Wide
or 1.5 Equipment
Widths for Compaction


DAYLIGHT CUT AREA OVER
NATURAL DESCENDING SLOPE
Structural Setback
Without Corrective Work
Daylight Cut
Line per Plan
Finish Grade
Project Removal
at 1 to 1
Min. 3 Feet
Compacted Fill
Compacted Fill
Topsoil
Colluvium
Creep Zone
Min.
2 Feet
Min. 2% Fall
Bedrock
Minimum 15 Feet Wide
or 1.5 Equipment
Widths for Compaction

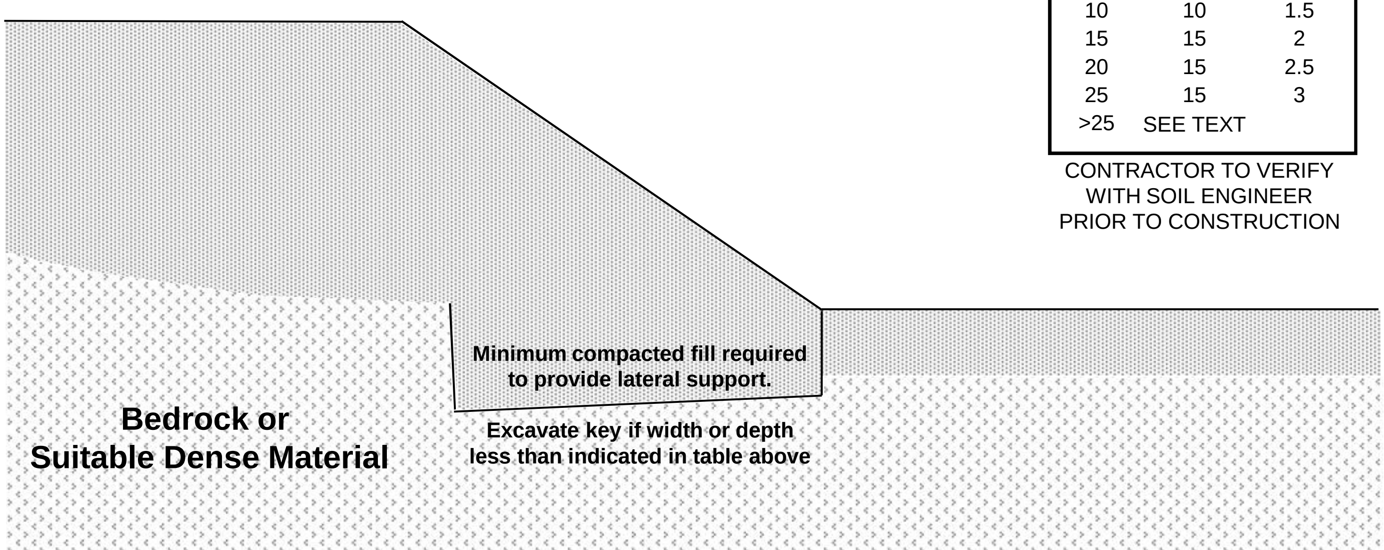
TYPICAL FILL SLOPE OVER CUT SLOPE



TYPICAL FILL SLOPE

SLOPE HEIGHT	MIN. KEY WIDTH	MIN. KEY DEPTH
5	7	1
10	10	1.5
15	15	2
20	15	2.5
25	15	3
>25	SEE TEXT	

CONTRACTOR TO VERIFY WITH SOIL ENGINEER PRIOR TO CONSTRUCTION



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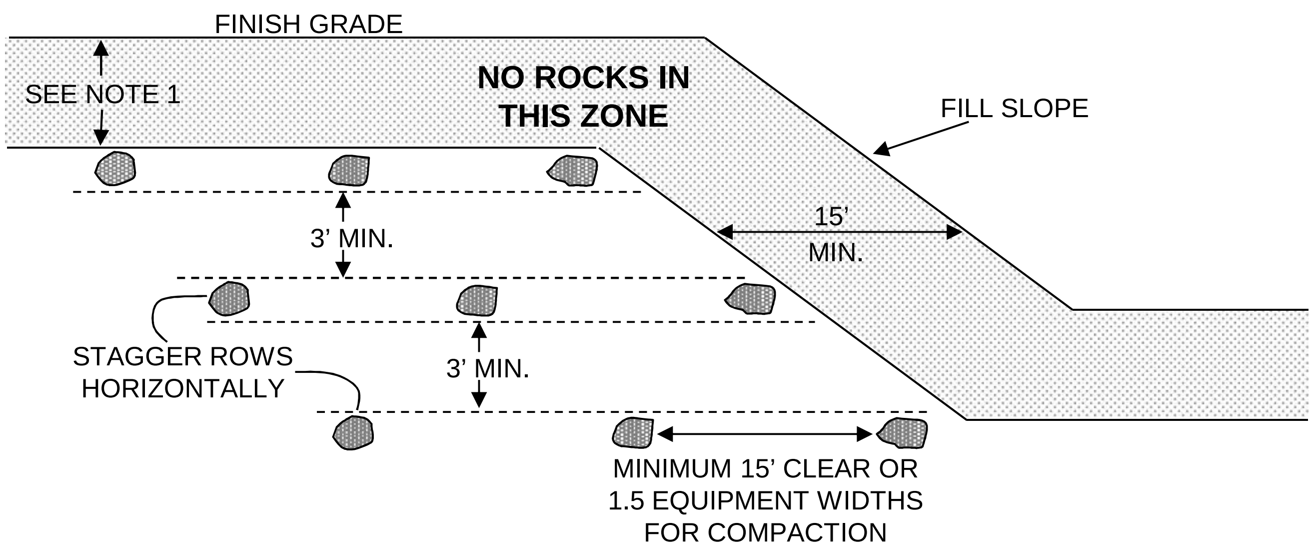
COMMON FILL SLOPE KEYS

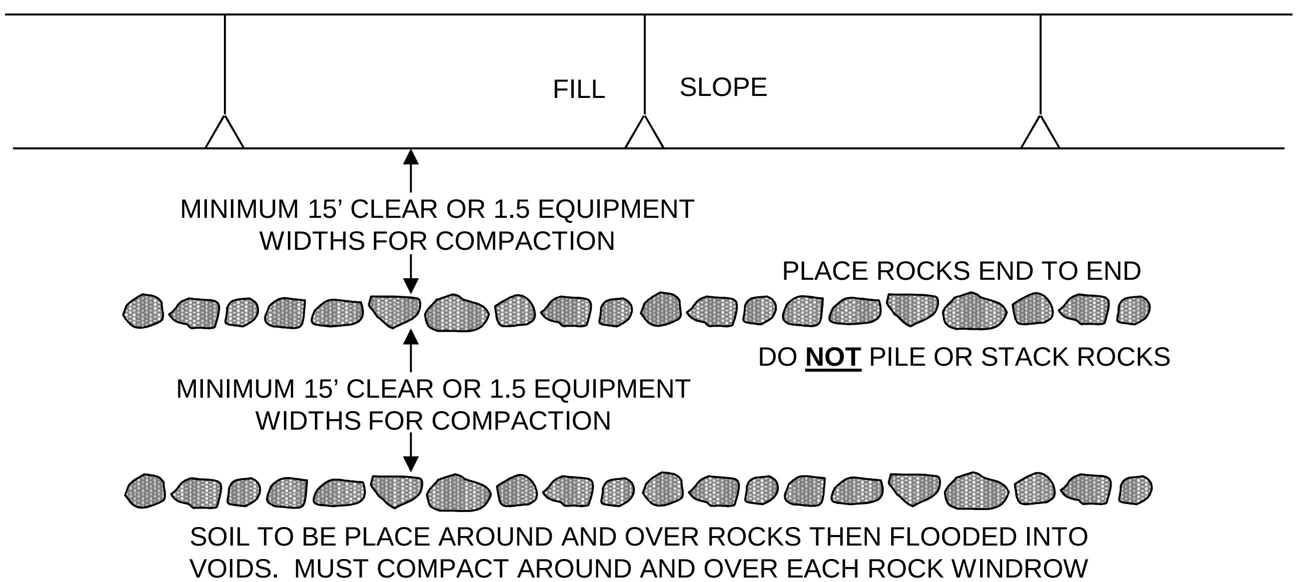
STANDARD GRADING GUIDELINES

PLATE G-3

CROSS SECTIONAL VIEW



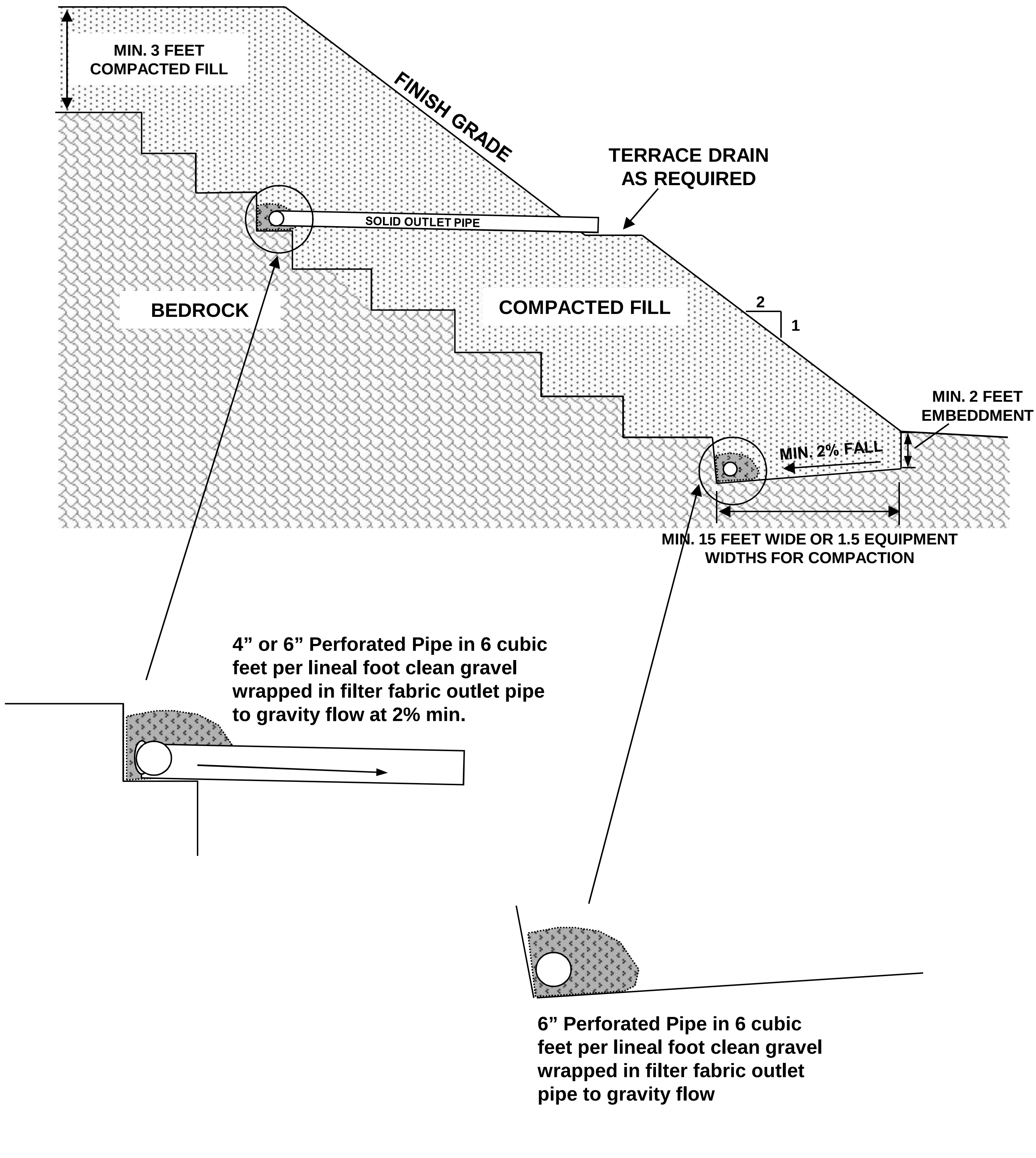
PLAN VIEW



NOTES:

1) SOIL FILL OVER WINDROW SHOULE BE 7 FEET OR PER JURISDUICTIONAL STANDARDS AND SUFFICIENT FOR FUTURE EXCAVATIONS TO AVOID ROCKS
2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET MINIMUM DIAMETER
3) SOIL AROUND WINDROWS TO BE SANDY MATERIAL SUBJECT TO SOIL ENGINEER ACCEPTANCE
4) SPACING AND CLEARANCES MUST BE SUFFICIENT TO ALLOW FOR PROPER COMPACTION
5) INDIVDUAL LARGE ROCKS MAY BE BURIED IN PITS.

GEOTEK 1384 Poinsettia Avenue, Suite A Vista, California 92081-8505	**ROCK BURIAL DETAILS**	**STANDARD GRADING GUIDELINES** **PLATE G-4**



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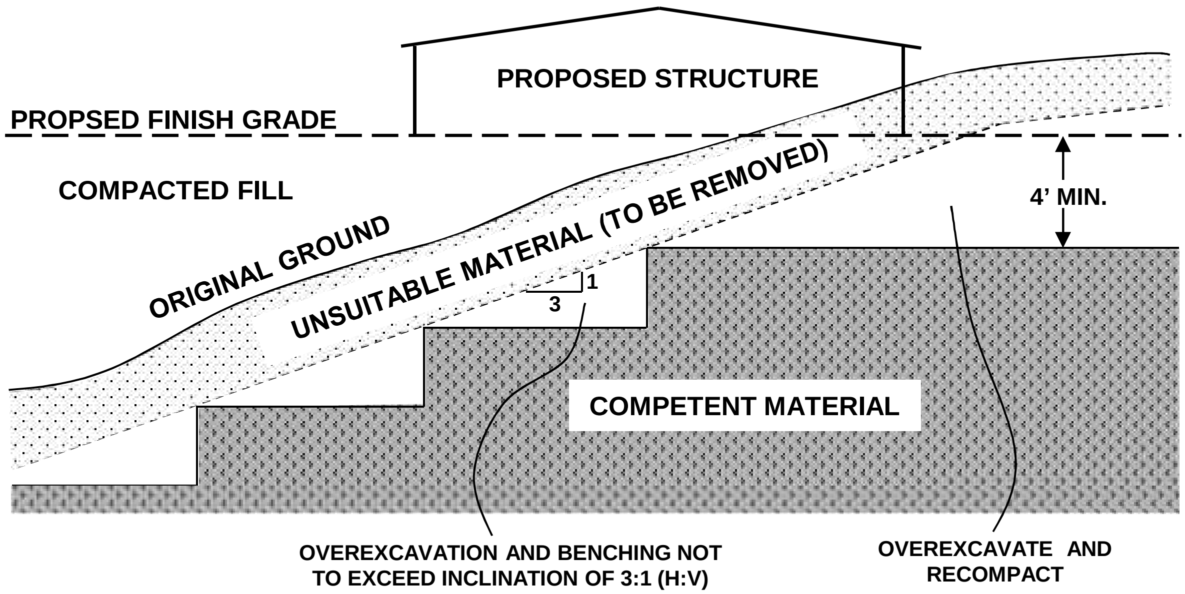
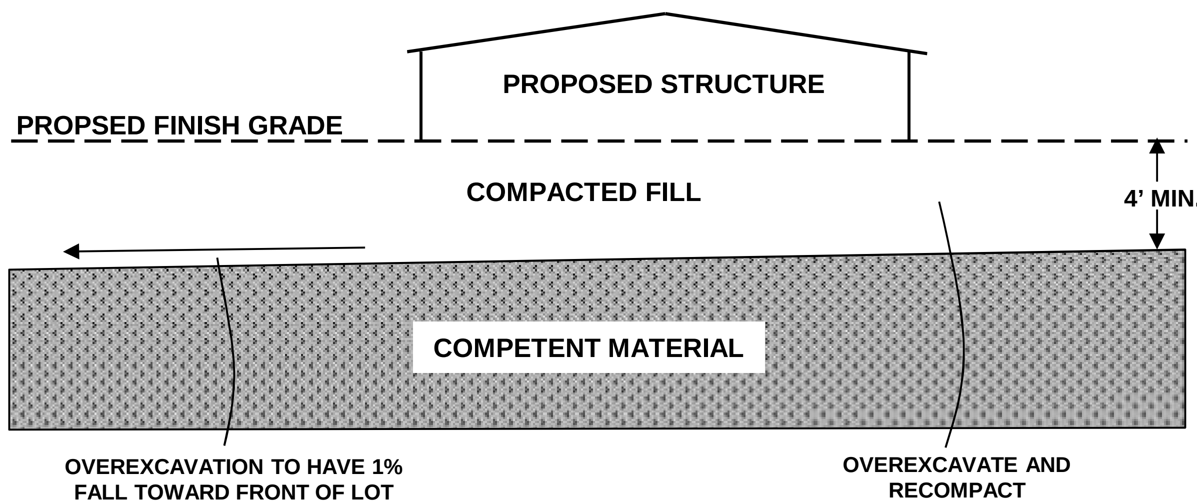
Typical Buttress and Stabilization Fill

PLATE G-5

TRANSITION LOT



UNDERCUT LOT



Notes:
1. Removed/overexcavated soils should be recompacted in accordance with recommendations included in the text of the report.
2. Location of cut/fill transition should verified in the field during site grading.

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TRANSITION & UNDERCUT LOTS

STANDARD GRADING GUIDELINES

PLATE G-6