PRELIMINARY PRELIMINARY GEOTECHNICAL Evaluation

FOR

PROPOSED RESIDENTIAL DEVELOPMENT
APN 220-210-49-00
NORTHEAST OF WOODWARD & MISSION
SAN MARCOS, CALIFORNIA

PREPARED FOR

Pacific Group
P.O. Box 9890
Rancho Santa Fe, Ca 92067

PREPARED BY

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PROJECT NO. 3575-SD MAY 16, 2019

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May 16, 2019
Project No. 3575-SD

Pacific Group
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Rancho Santa Fe, Ca 92067

Attention: Mr. Reza Shera

Subject: Preliminary Geotechnical Evaluation
APN 220-210-49-00
Northeast of Woodward & Mission
San Marcos, California

Dear Mr. Shera:

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.

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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 exploration consisting of surface field mapping by a Professional Geologist from our firm.
- Three (3) seismic refraction line "surveys" were completed to help evaluate subsurface conditions and excavation characteristics on the property.
- Excavation of seven (7) exploratory trenches onsite and collection of bulk soil samples for subsequent laboratory testing.
- Laboratory testing of the soil samples collected during the field investigation.
- Review and evaluation of site seismicity, and
- 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 project site is located northeast of Woodward Street and East Mission Road in the City of San Marcos, California (see Figure 1). The site is generally bounded to the west by Woodward Street, to the north by a natural slope, to the east by residential development, and to the south by a natural slope descending to East Mission Road. The site is located in a hillside setting with topography generally ascending from the north, west, and south site boundaries to the eastern boundary of the site. Site surface conditions generally consist of dense natural vegetation and outcrops of boulders. The site area appears vacant, with a local area which appears to have been cut to a level pad. Access is achieved via an unpaved road off Woodward

Street and wraps around the west side of the slope to a localized level pad. Fill areas appear to be attributed to the access road and level pad.

Total relief across the site is on the order of 100± feet, with surface drainage directed towards the southwest. Topographically, the high points on the property are located toward the northeast and northwest corners (elevation of roughly 695 feet msl), and the low point is toward the southwestern corner of the property. Steep natural gradients exist in the central, northeastern and northwestern portions of the property. These slope areas are locally as steep as 1:1 (horizontal to vertical) in gradient and expose Monzogranite (granitic) bedrock materials. Relatively small earthen fill slope areas (unmapped) were also noted along the access road and edges of the level pad.

2.2 Proposed Development

Based on review of the conceptual site plan for the project site prepared by Excel Engineering, unknown date, a multi-family residential development with a total of 22 buildings are proposed for the site. Fifteen buildings are 3-stories and seven buildings are 2-stories. Access to the development will be provided off Woodward Street and wrap around the hillside. Grading improvements generally consist of a 30 foot tall cut slope along the east property line and a 12 foot high fill slope inclined at a 2:1 horizontal:vertical (H:V) with an 8 foot tall concrete-masonry-unit retaining wall at the bottom of the fill slope. In addition, a 15 foot soil nail wall with a 25 foot tall cut slope above, accommodates the entry road to the development. The plan reviewed depicts several other smaller CMU retaining walls in addition to those stated above. Associated improvements are anticipated to consist of a water quality control basin and tree wells, wet and dry utilities, hardscape, structural pavement, and landscaping. A copy of the plan provided is used as the base for the Geotechnical Map (Figure 2) included with this report.

As site development planning progresses and plans become available, the plans should be provided to GeoTek for review and comment. Additional geotechnical field exploration, laboratory testing and engineering analyses may be necessary in order to provide specific earthwork recommendations and geotechnical design parameters for actual site development plans.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 Field Exploration

Our field exploration was conducted between April 26 and May 1, 2019 and consisted of a site reconnaissance, excavation of seven exploratory test pits, collection of loose bulk soil samples for subsequent laboratory testing, three seismic refraction surveys by a specialty consultant, and two field percolation test borings. A Professional Geologist from our firm field mapped the site from a geologic perspective, visually logged the test pits, and collected soil samples for laboratory analysis. Approximate locations of exploration locations are presented on the Geotechnical Map, Figure 2. A description of material encountered in the exploratory test pits and a copy of the seismic refraction report is included in Appendix A.

3.2 Percolation And Infiltration Testing

Borings PB-1 and PB-2, were advanced with a hand auger to practical refusal at a depth of approximately 12-inches below existing ground. Following completion of the borings, percolation testing was performed in borings PB-1, and PB-2 by a representative from our firm in general conformance with the City of San Marcos BMP Design Manual. The boreholes were presoaked overnight and the testing was performed on the following day. Percolation testing was performed by adding potable water to the borings, recording the initial depth to water and allowing the water to percolate for 30 minutes and the depth to water was measured. Water was generally added to each boring following each reading increment. In general, the percolation testing was performed for approximately 6 hours to allow rates to stabilize. Results of the final percolation increment were used to calculate an infiltration rate in inches per hour via the Porchet method.

For design of shallow infiltration basins, converting percolation rates to infiltration rates via the Porchet method is generally acceptable and appropriate, as this method factors out the sidewall component of the percolation results and represents the bottom conditions of a shallow basin (infiltration). Therefore, the percolation data for borings PB-1 and PB-2 were converted via the Porchet method. This method is consistent with the guidelines referenced in the City of San Marcos BMP Design Manual. Results of our infiltration analysis without a factor of safety are presented in the follow table for each of the test areas.

Location	Depth (inches)	Infiltration Rate (inches per hour)*
PB-1	12	0.68
PB-2	13	0.70

* Rate was converted to an infiltration rate via the Porchet method

Copies of the percolation data sheets and an infiltration conversion sheet are included in Appendix B.

The material exposed along the boring sidewalls and at the bottom of each test area were native soils. The tests performed and reported are indicative of the native soils. If the sidewalls and/or bottom of any of the proposed infiltration areas expose engineered fill, additional infiltration testing should be performed. Also, the granitic bedrock that underlies this site at shallow depth will likely resist infiltration stormwater disposal possibilities.

Over the lifetime of the storm water disposal areas, the percolation rates may be affected by silt build up and biological activities, as well as local variations in soil conditions. An appropriate factor of safety used to compute the design percolation rate should be considered at the discretion of the design engineer and acceptance of the plan reviewer.

3.3 Laboratory Testing

Laboratory testing was performed on bulk soil samples collected during the field explorations. The purpose of the laboratory testing was to evaluate their physical and chemical properties for use in the engineering design and analysis. Results of the laboratory testing program, along with a brief description and relevant information regarding testing procedures, are included 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 our subsurface exploration is presented in the following sections. Based on our field observations and review of published geologic maps the subject site area is locally underlain by sporadic undocumented fill materials, colluvium, and Cretaceous age crystalline bedrock.

4.2.1 Undocumented Fill

Some undocumented fill soils were locally observed in the vicinity of the level pad in the southern vicinity of the site and along the unpaved access road that wraps around the western side of the site. As observed, the undocumented fill generally consisted of silty sand with some cobbles and small boulders. Other areas of undocumented fill (unmapped) are also likely present on the site. Undocumented fill soils are not considered suitable for support of structural site improvements, but may be re-used as engineered fill if properly placed.

4.2.2 Colluvium (not mapped)

Colluvial soils are anticipated to be present in drainage swales or in localized areas on the site in relatively lower gradient slopes. Colluvial materials were not directly observed, however are anticipated to generally consist of clayey sands that are organic in nature. In general, the colluvium is likely thinner (i.e. bedrock is shallower) where slope gradients are steeper and in steep areas, might not be present at all.

4.2.3 Bedrock

The most recent regional geologic map showing the overall site geology (Kennedy, 2007), shows Mesozoic-aged meta-sedimentary bedrock at the surface across the site, however based on our site evaluation Cretaceous age plutonic bedrock tonalite-monzogranite (granite) was observed across the property, with outcrops and partially exposed core stones of bedrock materials. In the exploratory trenches, weathered to less weathered tonalite bedrock materials were encountered and observed to excavate primarily as red brown silty sand with gravel, cobble, and small boulder size fragments. The granitic bedrock materials were also observed on the adjacent surrounding properties, most notably to the east, north and south.

Anticipated bedrock excavation characteristics are discussed in a later section of this report. Descriptions of the bedrock materials as encountered in our exploratory trenches are shown on the exploratory trench logs included in Appendix A, and the results of the subsurface seismic refraction surveys are also 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 or possibly some minor surface run-off from immediately surrounding properties. Overall site area drainage is in a westerly direction, toward Woodward Street. 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 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. Rockfall potential should be further assessed when site development plans become available and during grading construction.

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 2016 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 fill materials, the upper lose and compressible materials should be removed for structural site areas. Removal depths in areas of existing undocumented fill, colluvium and highly weathered bedrock are estimated to be up to approximately 5 feet in areas of mapped existing fill. The lateral extent of removals beyond the outside edge of all settlement sensitive structures/foundations should be equivalent to that vertically removed. Depending on actual field conditions encountered during grading, locally deeper and/or shallower areas of removal may be necessary.

At a minimum, the cut portion(s) of any building pad areas in site bedrock or natural material(s) should be overexcavated a minimum of three (3) feet below finish pad grade or a minimum of two (2) feet below the bottom of the deepest proposed footing, whichever is deeper. Overexcavations should extend a minimum of five (5) feet outside the proposed building envelope(s). The intent of the recommended overexcavation is to support the improvements on engineered fill with relatively uniform engineering characteristics and decrease the potential for differential settlement.

Removal bottom gradients should be sloped towards the street to reduce potential long term perched groundwater conditions. The bottom of all removals should be scarified to a minimum depth of six (6) inches, 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 prior to scarification. The resultant voids from remedial grading/overexcavation should be filled with materials placed in general accordance with Section 5.2.4 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 Engineered Fill

Onsite materials are generally considered suitable for reuse as engineered fill provided they are free from vegetation, roots, debris, and rock/concrete or hard lumps greater than six (6) inches in maximum dimension. 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 inch 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.

5.2.5 Slopes

Proposed cut or fill slopes will need to be further evaluated when site development plans become more refined. Fill slopes constructed at gradients of 2:1 (horizontal:vertical), in accordance with industry standards, are anticipated to be both grossly and surficially stable. However, some fill soils derived from onsite materials may be sandy and have very low cohesion. These materials are erodible and the use of jute mesh or similar products designed to enhance surficial stability may be necessary until plant growth is established.

Cut slopes constructed at gradients of 2:1 (horizontal:vertical) or possibly steeper exposing bedrock materials are also anticipated to be grossly and surficially stable, but should be further evaluated based on anticipated and exposed structure (joints/fractures). Future landscaping on cut slopes exposing relatively unweathered bedrock materials could be difficult with the possibility of little plant growth due to the hard/dense nature of the bedrock. Replacement stability fills may be considered to enhance the plantability of the proposed cut slopes.

5.2.6 Excavation Characteristics

Excavations in the onsite undocumented fill, colluvium and weathered bedrock 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 and could locally require special techniques to excavate.

Seismic refraction data for the areas evaluated seems to generally indicate that excavations on the order of roughly 6 to 19 feet should be excavatable with conventional earthmoving equipment with some exceptions to core stones (See Appendix A). 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. A summary of bedrock hardness and recommendations is presented in Appendix A of this report (Subsurface Surveys, 2019).

Based on preliminary review of the current development plan for the project site and the anticipated depths of cut in localized areas, some blasting will likely be required to achieve the design grades or utility construction in some areas. The ultimate need for special excavation techniques and/or blasting will only be known during rough earthwork construction. In addition, materials generated from these special excavation techniques will likely be difficult to place in structural fills due to variations in size and wear on equipment. The need to trench for underground utilities should also be considered when rough grading is taking place for a project underlain by potentially very hard bedrock. Over-excavating streets to the depth of the anticipated deepest underground utility trench should be considered. Additional review and interpretation with respect to rock hardness is recommended by the project grading and utility construction contractors.

5.2.7 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 to 15 percent may be considered for the colluvium and undocumented fill materials requiring removal and re-compaction. Bedrock bulking could range from 0 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.

5.2.8 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 Stormwater Infiltration

Many factors control infiltration of surface waters into the subsurface, such as consistency of native soils and bedrock, geologic structure, fill consistency, material density differences, and existing groundwater conditions. Based on our review of the conceptual plan, a stormwater quality control basin is located above an approximate 5-foot fill slope. In consideration of the site setting located in a hillside topographic setting, the proposed stormwater quality control basin constructed as engineered fill over dense granitic bedrock (a significant density increase), and the adjacent residential property downslope, infiltration of stormwater into the subsurface is not recommended from a geotechnical perspective. Stormwater quality control basins should be constructed with an impermeable liner along the sides and bottom.

5.3.2 Foundation Design Criteria

Preliminary foundation design criteria, in general conformance with the 2016 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, 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)	One-Story – 12 Two-Story – 18 Three-Story – 24
Minimum Foundation Width (Inches)*	Supporting One Floor - 12 Supporting Two Floors - 15 Supporting Three Floors - 18
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	No. 3 rebar 24" on-center, each way, placed in the middle one-third of the slab thickness
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one (1) top and one (1) bottom
Presaturation of Subgrade Soil (percent of optimum moisture content)	Minimum 100% to a depth of 12 inches

*Code minimums per Table 1809.7 of the 2016 CBC should be complied with.

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 2000 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 2016 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 350 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 12 inches deep, should be utilized across large entrances, however, the base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

5.3.3 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 2016 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2016 CBC Section 1907.1

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.

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 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.

- Unsuitable soil removals along the property lines will likely be restricted due to adjacent improvements. Special considerations will be required for foundation elements in these areas. Such considerations may include deepening of foundations, reduced bearing capacity, or other measures. This issue should be further evaluated once site plans become available.

5.3.5 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.

5.3.6 Seismic Design Parameters

The site is located at approximately 33.1437 Latitude and -117.1593 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 "D" based on the anticipated depth of fill.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.019g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.398g
Site Coefficient for Site Class "D", F_a	1.092
Site Coefficient for Site Class "D", F_v	1.603
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	1.119g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	0.639g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.742g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.426g

5.3.7 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.

5.3.8 Soil Sulfate Content

The sulfate content was determined in the laboratory for a soil sample collected during the field investigation. The 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 2000 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	35
2:1	55

*Select backfill should consist of native or imported sand other approved materials with an SE>20 and 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.

Additional lateral forces can be induced on retaining walls during an earthquake. For level backfill and a Site Class "D", 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 2016 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.

5.4.2 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.

5.4.3 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.

5.4.4 Soil Nail Wall

At this time we understand that a soil nail wall is planned to accommodate the entry road into the site. We recommend that a wall designer and specialty contractor familiar with achievable strength parameters in the area be included in the development of parameters for the wall design. After preliminary design parameters have been determined, a verification test program should be developed and implemented to verify the adequacy of the soil nail wall design.

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. P0200419-SD) dated February 14, 2019 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

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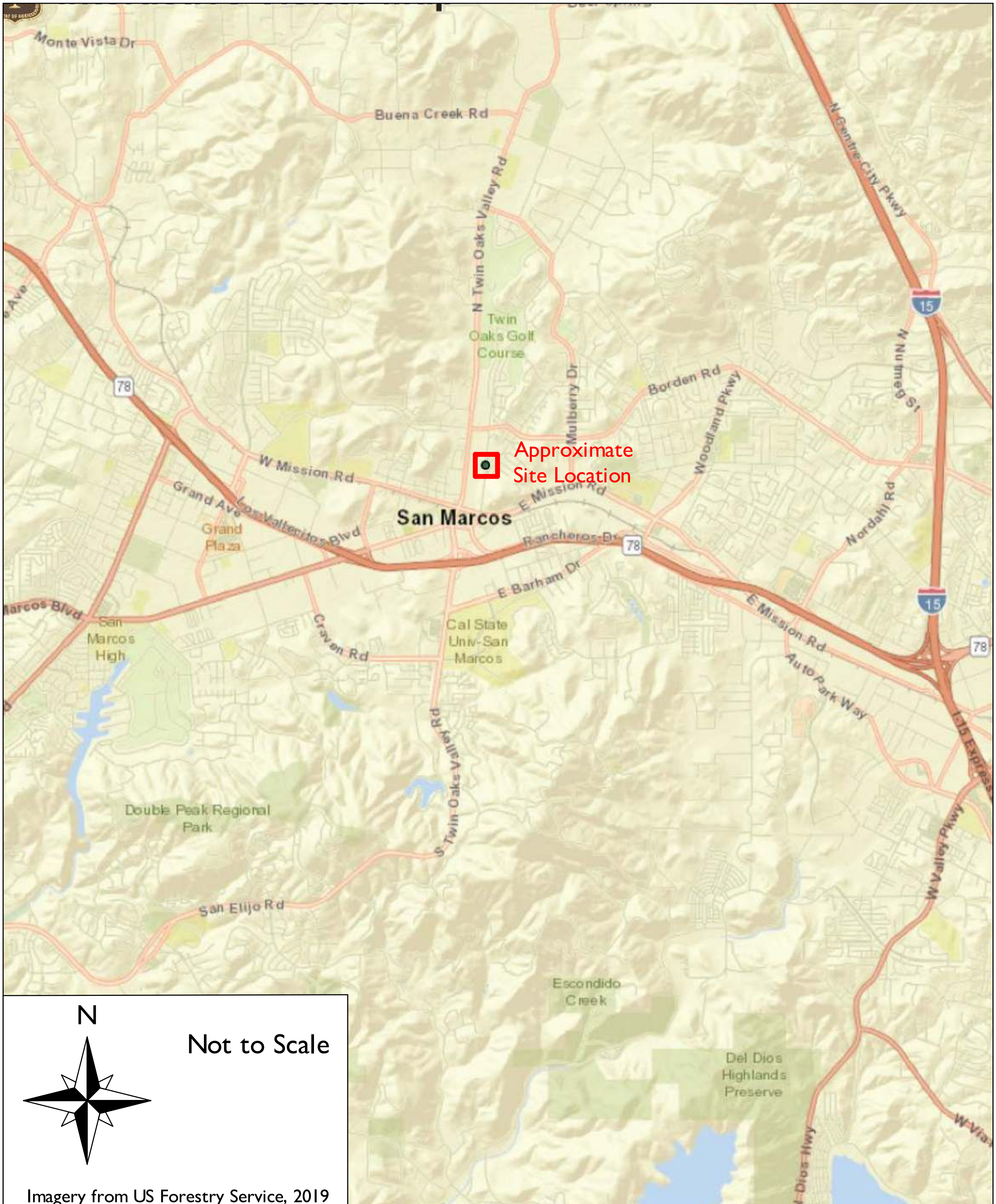


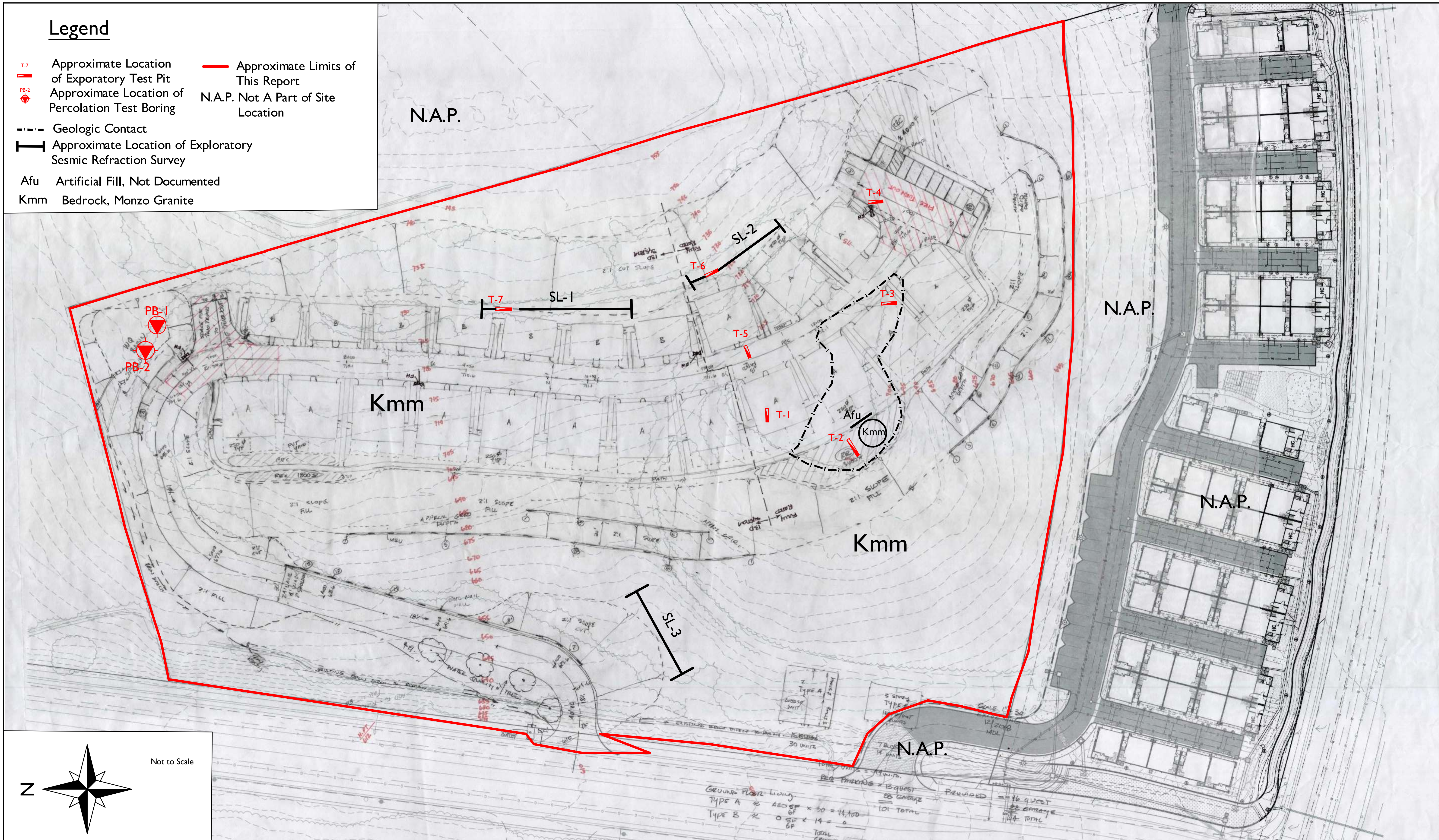
Pacific Group
APN 220-210-49-00
San Marcos, CA

PN: 3575-SD	DATE: April 2019

Figure 1

Site Location Map

GEOTEK
1384 Poinsettia Avenue, Suite A
Vista, California 92081



Pacific Group
Northest Corner of
Woodward St. & E. Mission Rd
San Marcos, California

PN: 3575-SD | DATE: May 2019

Figure 2

Geotechnical Map

GEOTEK
1384 Poinsettia Avenue, Suite A
Vista, California 92081

APPENDIX A

EXPLORATORY TRENCH LOGS

AND

SEISMIC REFRASCTION EVALUATION REPORT

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

Bulk Samples (Large)
These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

B –TRENCH LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings and trenches:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip

C: Contact line

………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
━━━	Thick solid line denotes end of boring

(Additional denotations and symbols are provided on the logs)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Granitic Bedrock			
					Weathered granite (poorly graded sand), medium dense, damp, coarse sand, angular small boulders in spoils, light grayish brown and orange brown, oxidized			
	Ring		BB-1					MD
					On N.W. side wall, large boulder, less weathered, jointed Practical Refusal			
5					**Trench Terminated at 4 Feet**			
					Practical Refusal No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol		Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				SM	**Artificial Fill Undocumented** Silty sand, loose, dry, orange-brown, fine to medium sand, rootlets, ~ 1/2 dozen fresh granite boulders			
			BB-1					MD, SR
					Granitic Bedrock			
5					**Trench Terminated at 4.5 Feet (Practical Refusal)** No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-3

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Artificial Fill Undocumented poorly graded coarse sand, dense, damp, light gray brown to orange brown, oxidized, some areas are weathered to a clayey sand			
					Granitic Bedrock			
5					**Trench Terminated at 5 Feet (Practical Refusal)** No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test
SR = Sulfate/Resisitivity Test; SH = Shear Test; CO = Consolidation test; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					<u>Artificial Fill Undocumented</u> Silty sand, loose, moist, light brown to dark brown, fine to medium sand, fill mottling			
					<u>Granitic Bedrock</u>			
5					**Trench Terminated at 5 Feet (Practical Refusal)** No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

<u>Sample type</u>: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

<u>Lab testing:</u>

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Granitic Bedrock Lightly weathered granite (poorly graded course sand), dense, damp, light grey brown Practicial Refusal			
5					**Trench Terminated at 3 Feet** Practical Refusal No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolidation test	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Granitic Bedrock Fresh granite, weathered pockets, poorly graded coarse sand, dense, pockets of oxidation, joints Practicial refusal			
					Trench Terminated at 3 Feet Practical Refusal No groundwater encountered Backfilled with soil cuttings			
5								
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | CO = Consolidation test | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Pacific Group	**DRILLER:**	Luna Construction	**LOGGED BY:**	CL
PROJECT NAME:	Woodward	**DRILL METHOD:**	Backhoe	**OPERATOR:**	
PROJECT NO.:	3575-SD	**HAMMER:**	NA	**RIG TYPE:**	CAT 420 E
LOCATION:	See Geotechnical Map	**ELEVATION:**	730'	**DATE:**	4/26/2019

TRENCH NO.: T-7

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Granitic Bedrock Weathered granite, silty sand, orange brown, moist, fine to medium sand, dense Becomes less weathered with depth Practical refusal			
5					**Trench Terminated at 3.5 Feet** Practical Refusal No groundwater encountered Backfilled with soil cuttings			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | CO = Consolidation test | MD = Maximum Density

Subsurface Surveys & Associates, Inc.
2075 Corte Del Nogal, Suite W   Carlsbad, CA 92011
Phone: (760) 476-0492   Fax: (760) 476-0493

GeoTek. Inc.
1384 Poinsettia Ave, Suite A
Vista, CA 92081

May 1, 2019

Attn: Chris Livesey

Re: Seismic Survey Summary Report
Woodward Project, San Marcos

Subsurface Surveys has completed a seismic refraction survey at the Woodward Project Site, in San Marcos, California. The purpose of the survey was to measure the compressional wave velocity of bedrock for rippability assessment and to provide cross sections showing thickness of the weathered zone and depth to the unweathered interface. This should be useful for planning cuts, grading, and other earthwork.

The field work was conducted on April 26, 2019. Three seismic lines were recorded at locations selected by GeoTek. A survey location map is provided on Figure 1 that shows the position and orientation of the traverses.

GEOLOGIC SETTING

A review of the "Geologic Map of the Oceanside 30'x60' Quadrangle", (Department of Conservation, 2005) indicates the local area is underlain by tonalite (Kt) of mid-Cretaceous age. Tonalite is also referred to as quartz diorite. This rock is a member of the Peninsular Ranges Batholith. Surface deposits are mostly loose soil and colluvium.

DATA ACQUISITION AND FIELD METHODS

Seismic refraction data were recorded with a Bison 9024 signal enhancement seismograph and 30 Hz geophones. The standard spread layout used 24 geophones with a 5-foot spacing. Each spread used five shotpoints, one off each end (5-foot offset) and three within the interior of the spread. Depth of investigation was approximately 25-30 feet.

Compressional wave energy was created by sledge hammer impacts on a metal plate. The signal enhancement feature of the seismograph allowed returns from repeated hits to be stacked, thus improving the signal. Vehicle traffic was not a factor during the survey. Each record was stored digitally on an internal hard disk and printed copies of each seismogram were made in the field on thermal paper.

Relative elevations of all shotpoints and geophones were determined by differential leveling with a hand level. Geophone 1 (distance = 0 ft.) at the beginning of each line was assigned a elevation value of 0.0 feet. This datum point served as the reference elevation for all other measurements.

Labeled wooden stakes were placed at the beginning and end of each spread and a Garmin handheld GPS receiver was used to record the latitude and longitude coordinates of the stakes. The coordinates were used to make the location map shown on Figure 1.

SEISMIC REFRACTION METHOD

The refraction method involves measuring the total time for compressional waves to travel from a shotpoint through the subsurface to a set of geophones placed linearly along the ground. Based on Snell's Law, when two or more layers are present with increasingly higher acoustic velocity, waves become critically refracted across the layer boundaries and begin traveling at the speed of the underlying layer. The advancing waves then generate new wavefronts back to the ground surface. The first surge of energy hitting the geophone is termed the "first arrival" and is depicted on the seismogram as a high angle deflection along each trace.

Recognition of direct wave arrivals (non-refracted) verses refracted waves is a key element of refraction interpretation. To assist this process, the first arrival times measured from the seismic records are plotted on graphs of time verses distance called Time-Distance graphs. An example T-D graph from Line 3 is shown on Figure 2. Based on changes in slope on the graphs, a preliminary layer number (i.e. 1, 2, 3) is assigned to each segment of the graph. The layer assignments together with time, distance and elevation data are input to a computer for additional processing.

DATA REDUCTION AND VELOCITY DETERMINATION

Processing and interpretation of this data set was accomplished with “SIPT2", an interactive inversion modeling program developed by James Scott for the U.S. Bureau of Mines. The inversion algorithm uses the delay time method to construct a first pass depth model. The model is then adjusted by an iterative ray tracing process that attempts to minimize the discrepancies between the total travel times calculated along ray paths and the observed travel times measured in the field.

This program calculates refractor velocity in two ways. First, apparent velocities from each shot are determined by the inverse slope of a best fit (least squares) line through datum-corrected travel times. True velocity is estimated from the apparent velocities by using the following equation:

$$Vt = 2(Vu \times Vd)/(Vu + Vd)$$

where
- Vt = true velocity
- Vu = apparent up dip velocity
- Vd = apparent down dip velocity

The second method uses a more sophisticated set of equations (the Hobson-Overton formula) developed by the Canadian Geological Survey. The final velocity assigned to the refractor is a weighted average of the results of the two methods. The weighting is based on the number of arrival times used in the computations.

SUMMARY OF RESULTS

Results from refraction analysis show a three layer solution beneath all lines (see Figures 4-6). Velocities posted on the cross sections represent averages as described in the previous section. Therefore, minor localized changes in velocity may occur along any profile. A description of the layers is provided below and a cross section summary is shown in Table 1.

Layer 1 - is mostly loose topsoil and colluvium. Thickness is generally less than 5 feet.

Layer 2 - is interpreted to be weathered bedrock. The velocity range is 3699-3885 ft/sec and is considered rippable with a D-9 Cat.

Layer 3 - represents hard unweathered bedrock.

Table 1. Cross Section Summary Velocity in (ft/sec), Depth in (feet)

Line	Velocity Layer 1	Velocity Layer 2	Velocity Layer 3	Depth Range Unweathered Interface
1	1397	3694	8341	8 - 19
2	1365	3669	7764	8 - 13
3	1269	3885	9057	6 - 16

Weathering tends to be gradational for most granitic rock types and usually produces a gradual increase in velocity with depth. Consequently, variation of $\pm$ 10% from the posted averages may occur between the top and bottom of Layer 2.

Large granitic boulders and core stones in the weathered layer are fairly common in this terrain where chemical and mechanical processes produce spheroidal weathering and exfoliation of the granitic basement rock. The result is remnant large dense spheroids.

Evidence of possible buried core stones were found at this site. Locations are highlighted on Lines 1 and 2 (see Figures 4 and 5).

Figure 4 presents a rippability chart (courtesy of Caterpillar Tractor Co.) for a D9R Ripper. Bar graphs show the relationship between seismic compressional wave velocity and ripper performance for various rock types in three categories: rippable, marginal, and non-rippable. Granite is listed as marginally rippable at approximately 6700 ft/sec and is considered non-rippable above 8000 ft/sec. This chart is provided only as a guide and should not be considered

absolute. Other geologic factors that may influence bedrock rippability at this site include changes in composition of the bedrock and the presence of fractures and joints.
All data acquired during this survey is considered confidential and is available for review by your staff at any time. We appreciate the opportunity to participate in this project.

Please call if there are any questions.

PAWalen

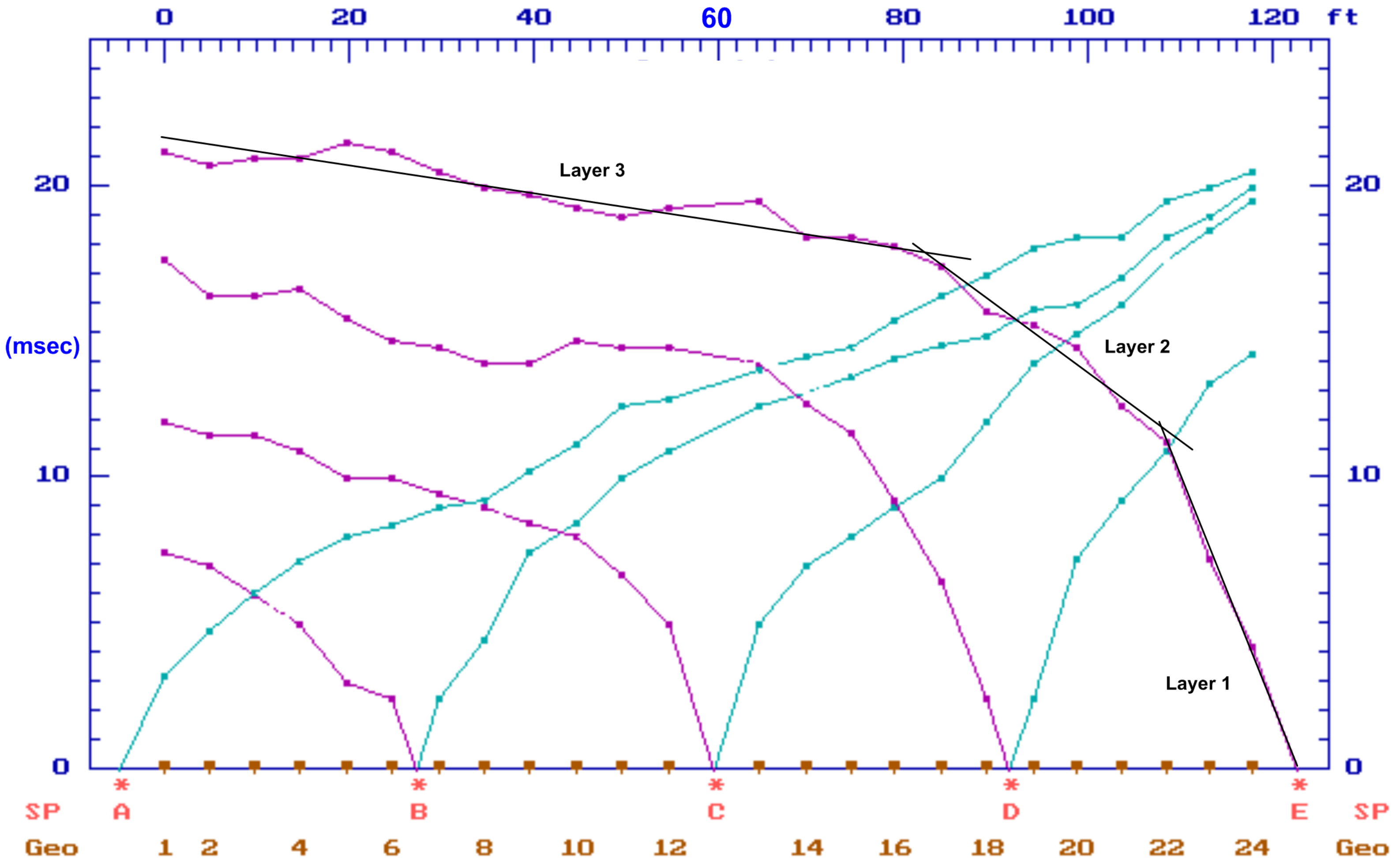
Phillip A. Walen
Senior Geophysicist
CA Registration No. GP917

Seismic Survey Location Map

Woodward Project Site -- San Marcos, CA

Figure 1

Example Time-Distance Graph -- Line 3



Seismic Refraction Survey 4/26/2019	Woodward Project Site
Geotek Project No. 3575-SD	San Marcos, California Figure 2

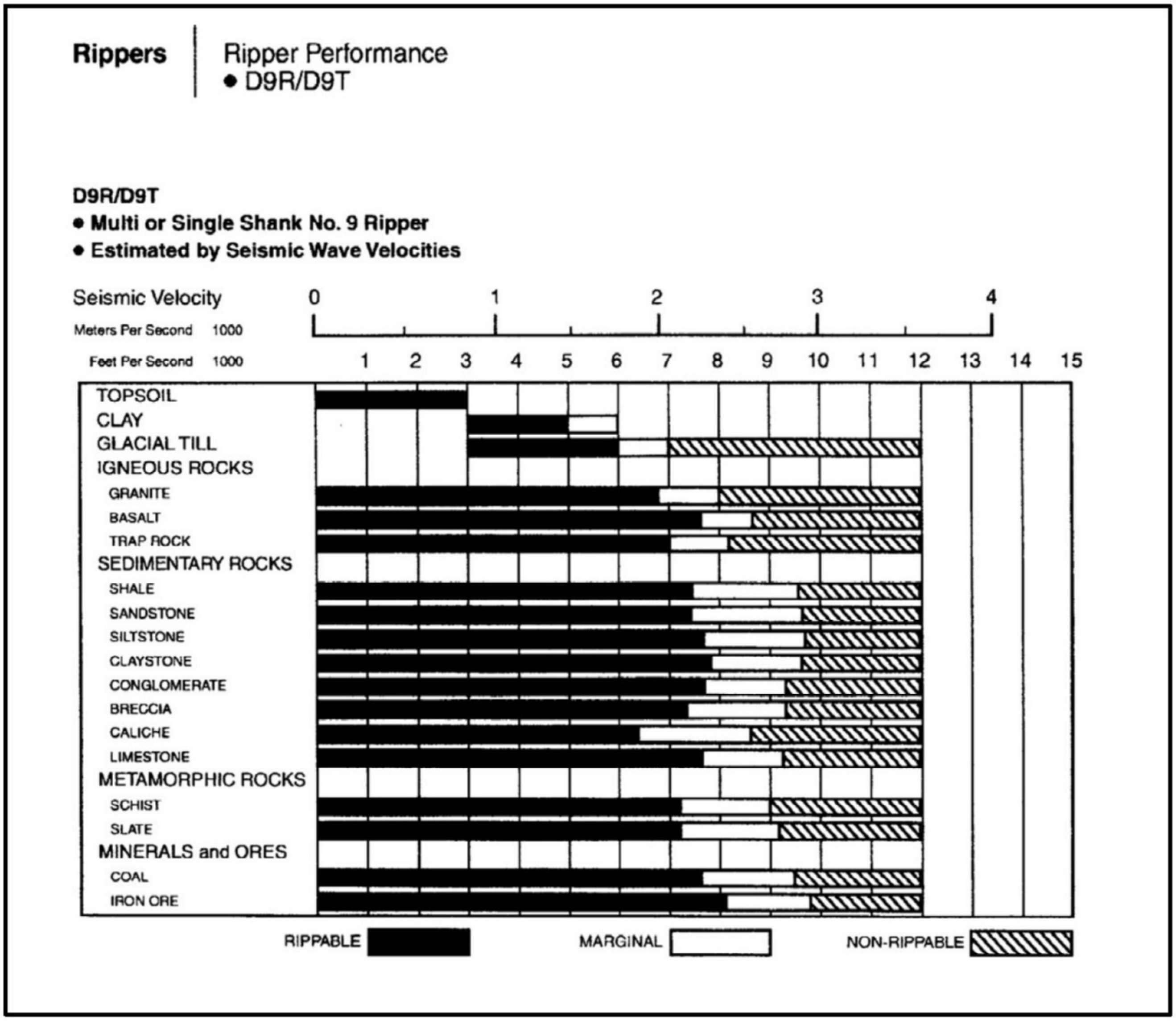
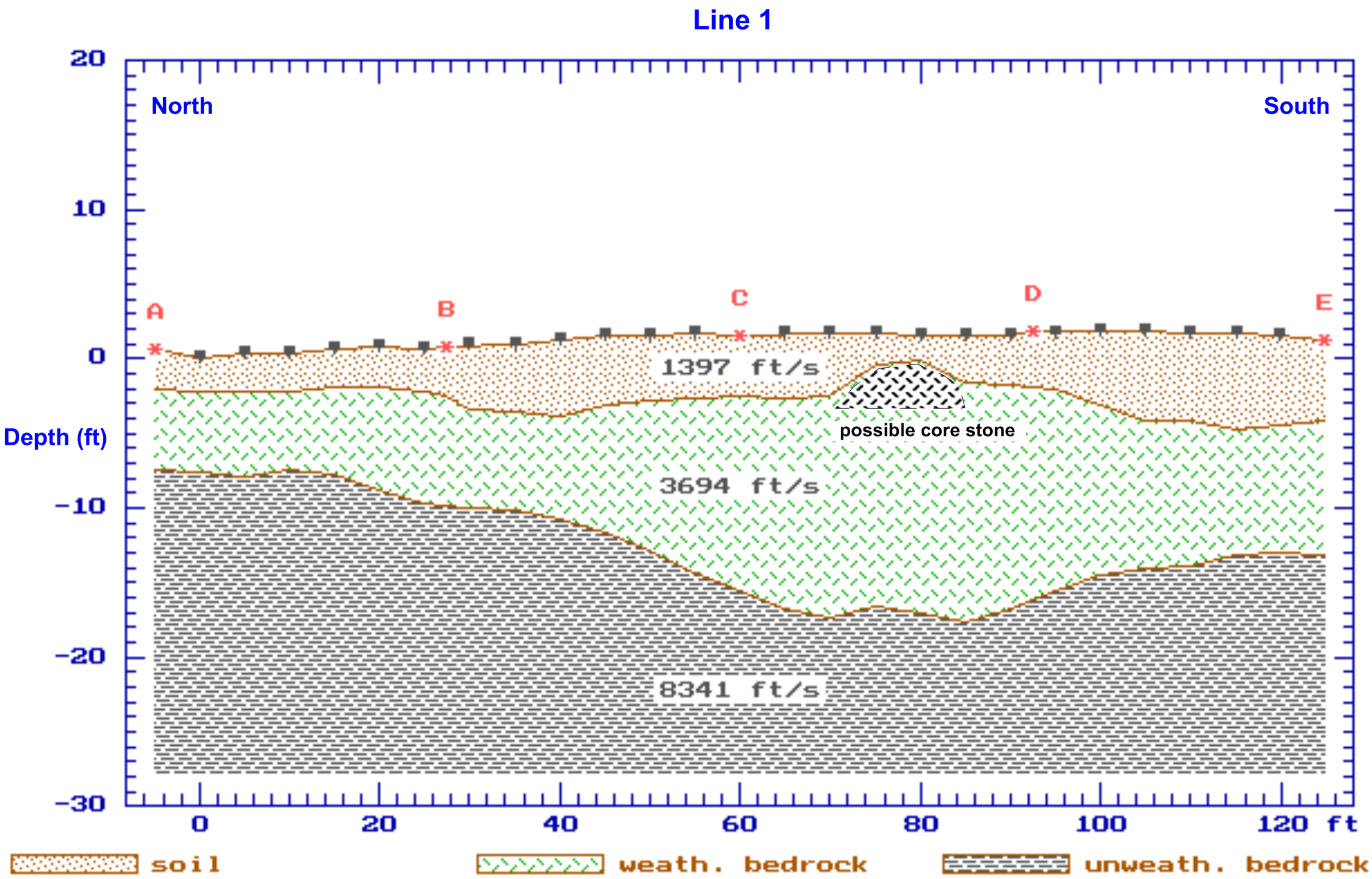
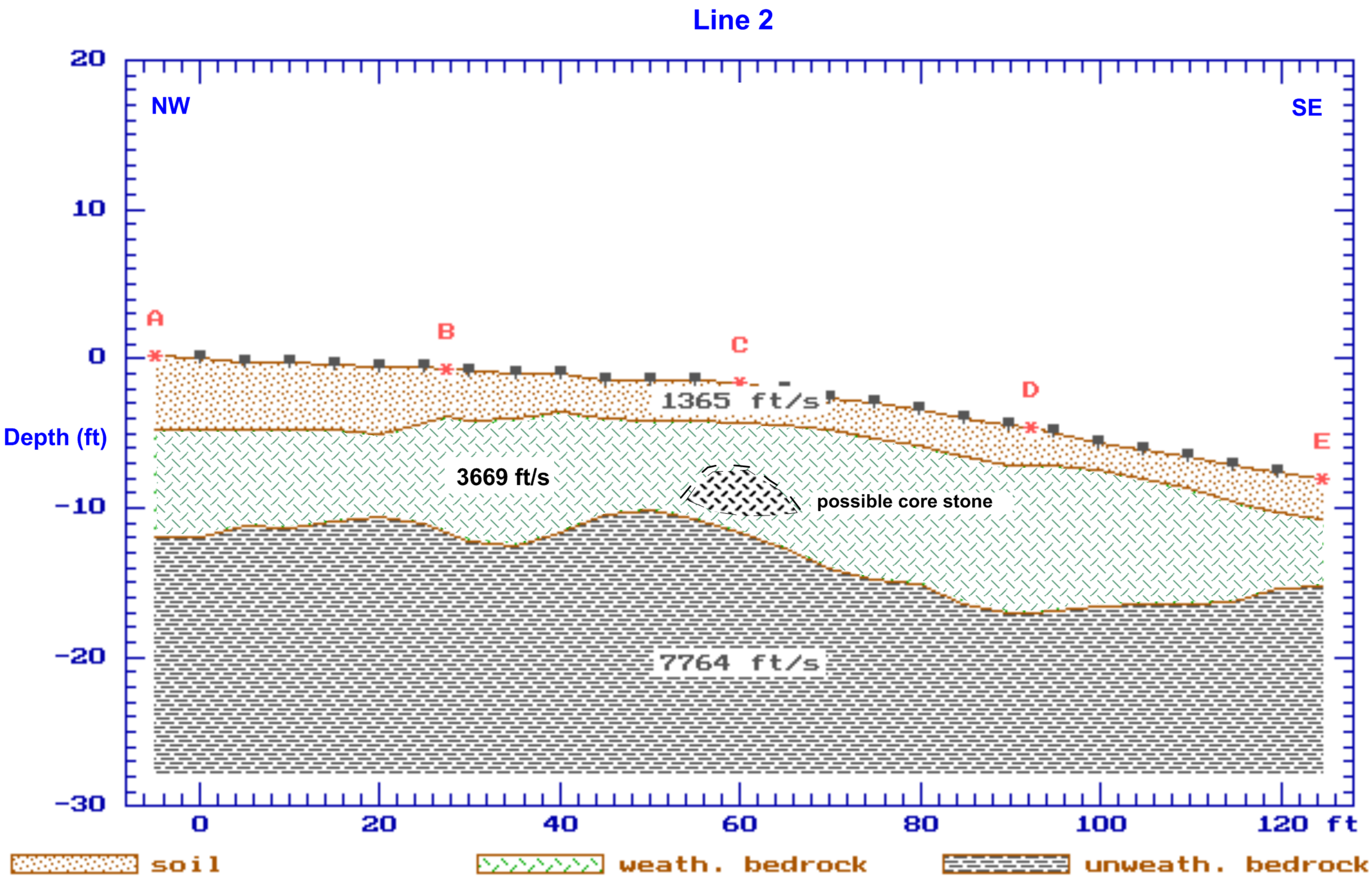

Rippers
Ripper Performance
• D9R/D9T
D9R/D9T
• Multi or Single Shank No. 9 Ripper
• Estimated by Seismic Wave Velocities
Seismic Velocity
Meters Per Second 1000
0 1 2 3 4
Feet Per Second 1000
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
TOPSOIL
CLAY
GLACIAL TILL
IGNEOUS ROCKS
GRANITE
BASALT
TRAP ROCK
SEDIMENTARY ROCKS
SHALE
SANDSTONE
SILTSTONE
CLAYSTONE
CONGLOMERATE
BRECCIA
CALICHE
LIMESTONE
METAMORPHIC ROCKS
SCHIST
SLATE
MINERALS and ORES
COAL
IRON ORE
RIPPABLE
MARGINAL
NON-RIPPABLE

Figure 3

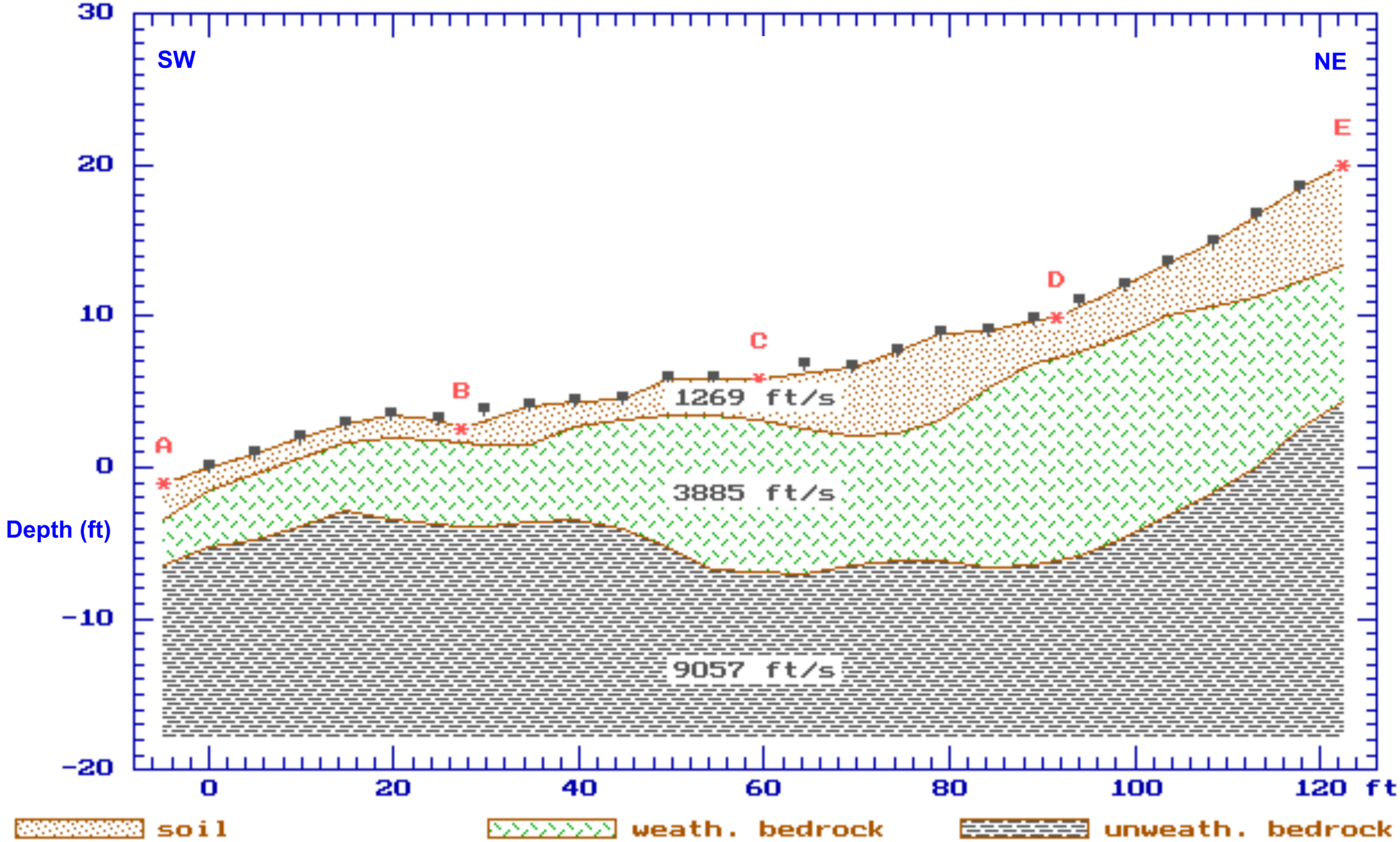


Seismic Refraction Survey 4/26/2019	Woodward Project Site
Geotek Project No. 3575-SD	San Marcos, California Figure 4



Seismic Refraction Survey 4/26/2019	Woodward Project Site
Geotek Project No. 3575-SD	San Marcos, California Figure 5

Line 3



Seismic Refraction Survey 4/26/2019	Woodward Project Site
Geotek Project No. 3575-SD	San Marcos, California

Figure 6

PERCOLATION DATA SHEET

Project: Woodward, **Job No.:** 3575-SD

Test Hole No.: PB-1 **Tested By:** SE, **Date:** 5/1/19.

Depth of Hole As Drilled: 12" **Before Test:** 12" **After Test:** 12".

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
1	10:01	30	12	1.875	5.625	3.75	
2	10:31	30	12	4	7.125	3.125	
3	11:02	30	12	3.375	6	2.625	
4	11:34	30	12	3.375	6.25	2.875	
5	12:09	30	12	3.375	6	2.625	
6	12:40	30	12	3.375	6.125	2.75	
7	13:11	30	12	3.625	6.625	3	
8	13:44	30	12	3.375	6.125	2.75	
9	14:17	30	12	3.625	6.625	3	

PERCOLATION DATA SHEET

Project: Woodward, **Job No.:** 3575-SD

Test Hole No.: PB-2 **Tested By:** SE, **Date:** 5/1/19.

Depth of Hole As Drilled: 13" **Before Test:** 13" **After Test:** 13"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Comments
1	10:10	30	13	7.125	0	7.125	
2	10:39	30	13	2.75	6.75	4	
3	11:15	30	13	3.75	7.875	4.125	
4	11:47	30	13	3.375	6.375	3	
5	12:19	30	13	3	6.125	3.125	
6	12:50	30	13	3.25	6.25	3	
7	13:21	30	13	3.25	6.375	3.125	
8	13:54	30	13	3.125	6.125	3	
9	14:28	30	13	3	6.25	3.25	

Client:	**Pacific Group**
Project:	**Woodward**
Project No:	**3575-SD**
Date:	**5/1/2019**

Boring No. **PB-1**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	6.625
Test Hole Radius, r =	1.8
Initial Depth to Water, D_O =	3.625
Total Test Hole Depth, D_T =	12

Equation - $$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$$

$H_O = D_T - D_O =$	8.38
$H_F = D_T - D_F =$	5.38
$\Delta H = \Delta D = H_O - H_F =$	3.00
$Havg = (H_O+H_F)/2 =$	6.88

I_t = 0.68 **Inches per Hour**

GEOTEK

Client:	**Pacific Group**
Project:	**Woodward**
Project No:	**3575-SD**
Date:	**5/1/2019**

Boring No. **PB-2**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	6.625
Test Hole Radius, r =	1.75
Initial Depth to Water, D_O =	3
Total Test Hole Depth, D_T =	13

Equation - $$I_t = \frac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$$

$H_O = D_T - D_O$ =	10
$H_F = D_T - D_F$ =	6.38
$\Delta H = \Delta D = H_O - H_F$ =	3.63
$H_{avg} = (H_O+H_F)/2$ =	8.19

I_t = 0.70 **Inches per Hour**

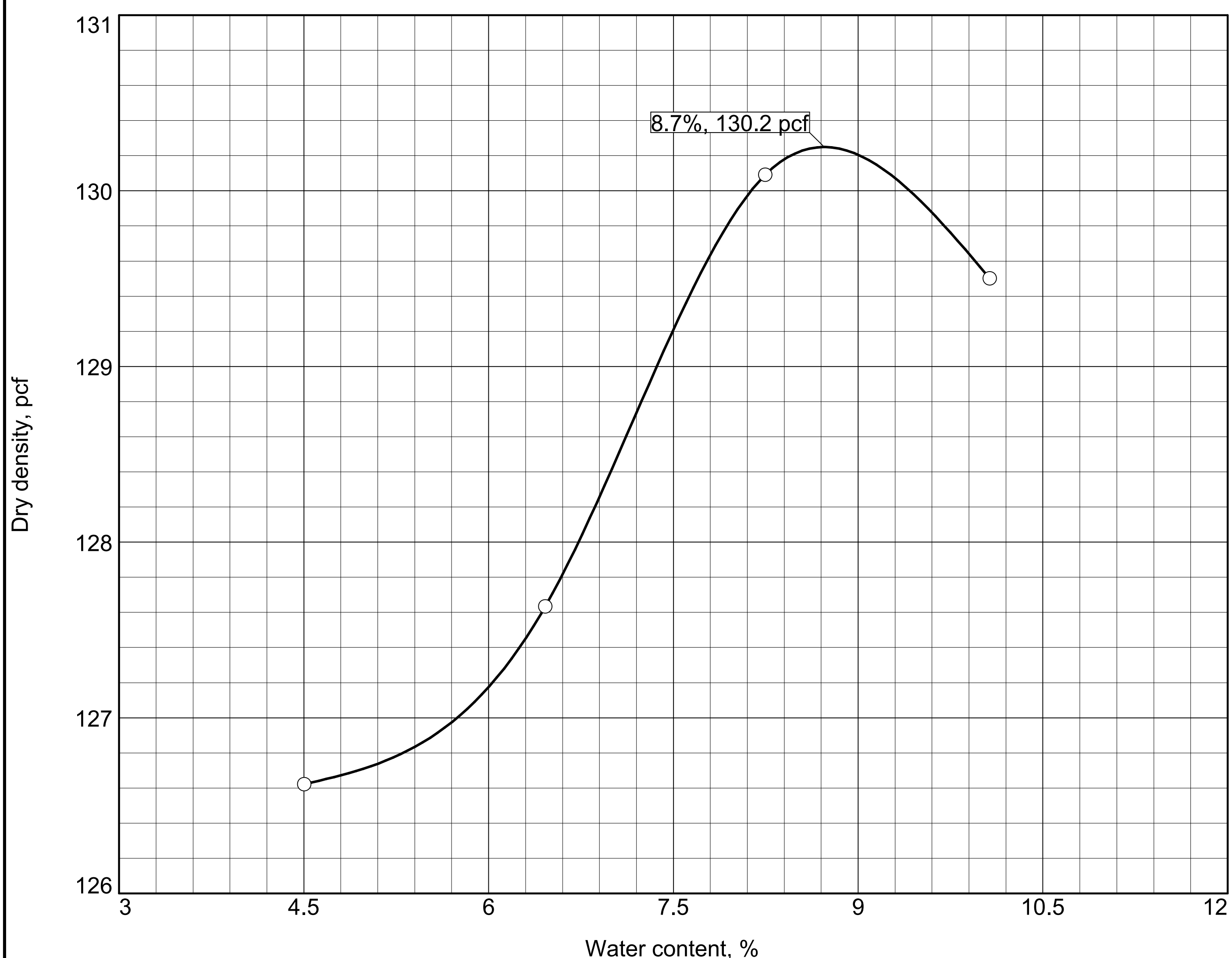
GEOTEK

APPENDIX B

RESULTS OF LABORATORY TESTING

GEOTEK

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
2-4'								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 130.2 pcf Optimum moisture = 8.7 %	Gravel Sand

Project No. 3575-SD **Client:** Pacific Group
Project: Woodward

○ **Location:** T-1 **Sample Number:** BB-1

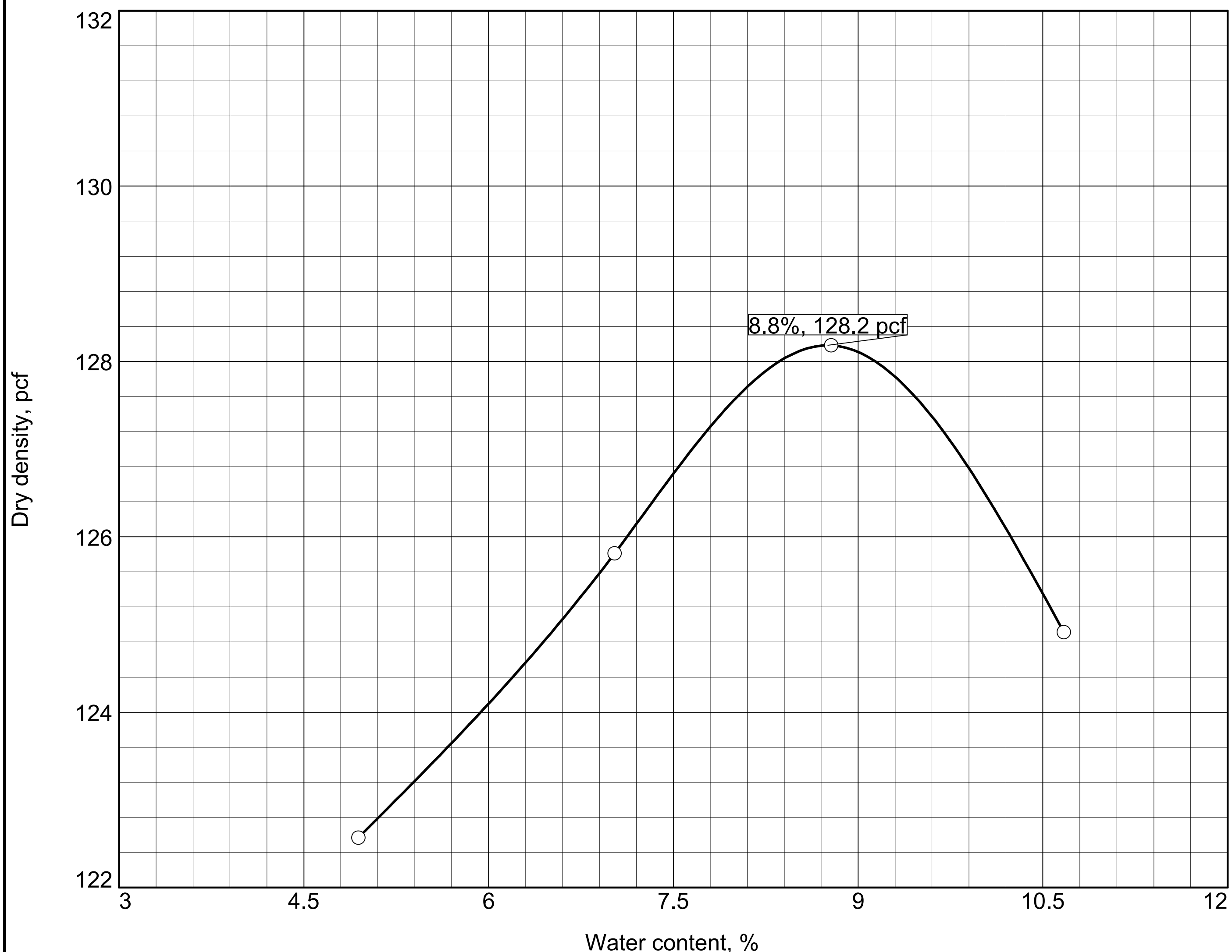
Remarks:

GEOTEK
Vista Office
1384 Poinsettia Ave Suite I Vista, CA 92081
Phone (760) 599-0509 Fax (760) 599-0593
www.geotekusa.com

Figure

Tested By: SE

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
3'								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 128.2 pcf Optimum moisture = 8.8 %	Dark Brown Silty Sand

Project No. 3575-SD **Client:** Pacific Group

Project: Woodward

○ **Location:** T-2 **Sample Number:** BB-1

Remarks:

GEOTEK
Vista Office
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Figure

Tested By: SE

Results Only Soil Testing for Woodward

May 6, 2019

Prepared for:
Chris Livesey
Geotek, Inc
1384 Poinsettia Ave, Suite A
Vista, CA, 92081
clivesey@geotekusa.com

Project X Job#: S190506A
Client Job or PO#: 3575-SD

Soil Analysis Lab Results

Client: Geotek, Inc
Job Name: Woodward
Client Job Number: 3575-SD
Project X Job Number: S190506A
May 6, 2019

	Method	ASTM G187		ASTM D516		ASTM D512B		SM 4500-NO3-E	SM 4500-NH3-C	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**Resistivity** As Rec'd	Minimum	**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
T-2 / BB-1	3.0	482,400	12,730	7.4	0.0007	5	0.0005	9	1.5	0.84	130	8.68

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Prepared by,

Nathan Jacob
Lab Technician

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
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REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

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, Chapters 18 and 33 of the Uniform Building Code 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 (1) 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 is 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 segment. 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 advise 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 slope, etc.) should be capped with a 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

It is anticipated that 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 inch) 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.

<u>Structural Rock Fills</u>

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inch 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 of 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 effect 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 contractors 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.

<u>Piping Potential and Filter Blankets:</u>

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 contractors 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 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 contractors 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 contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries our 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.

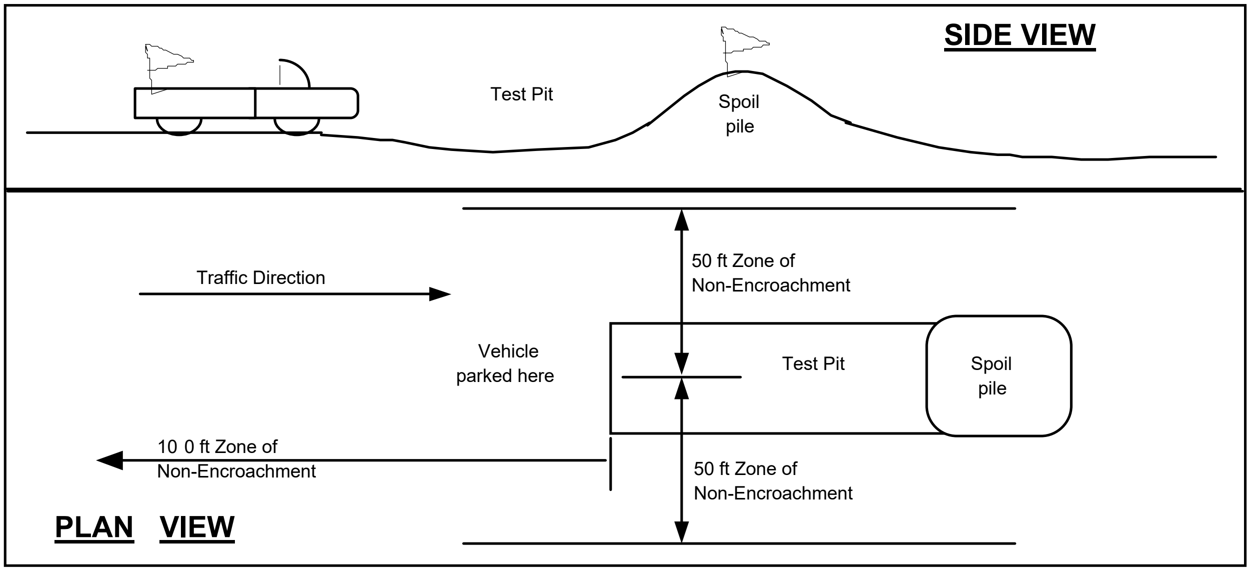
Test Pits Location, Orientation and Clearance

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, preferable 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 provide,
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 contractors representative will then be contacted in an effort to effect 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 effect 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 technicians 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.

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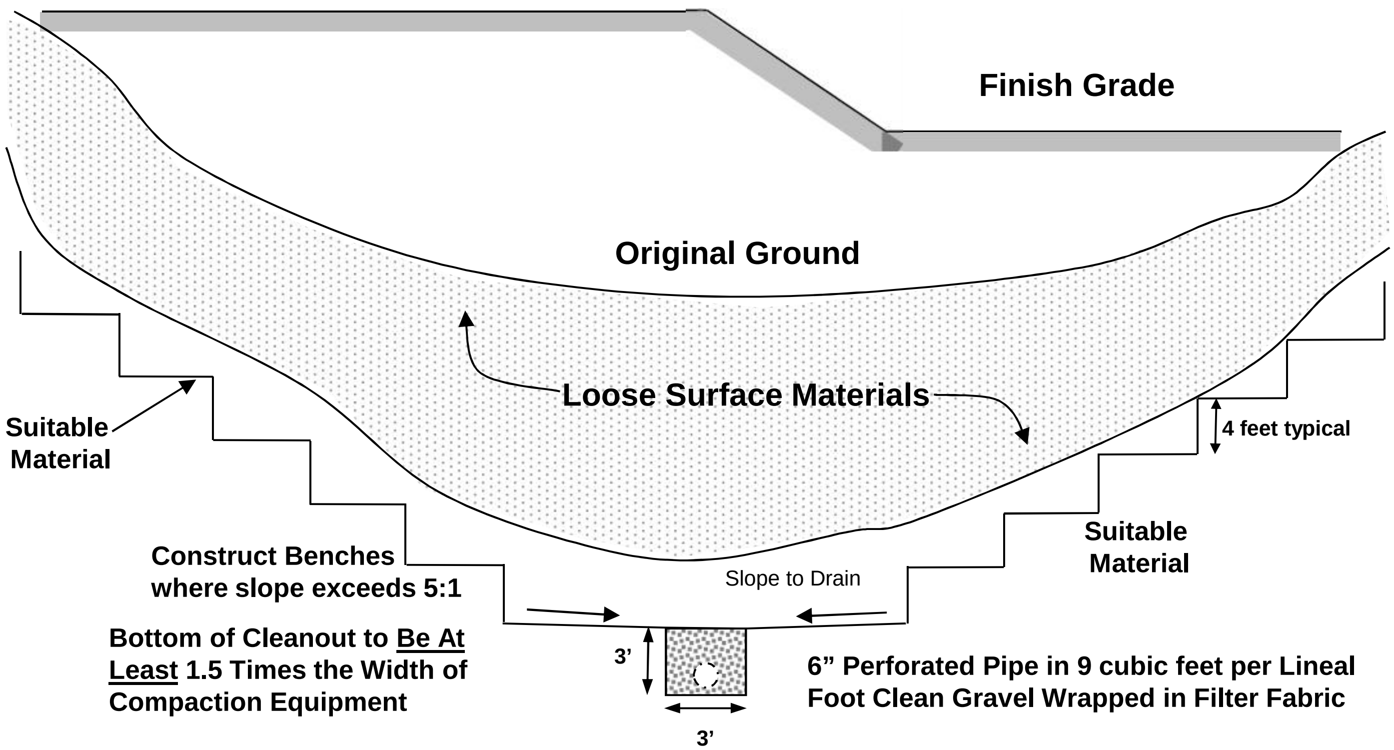
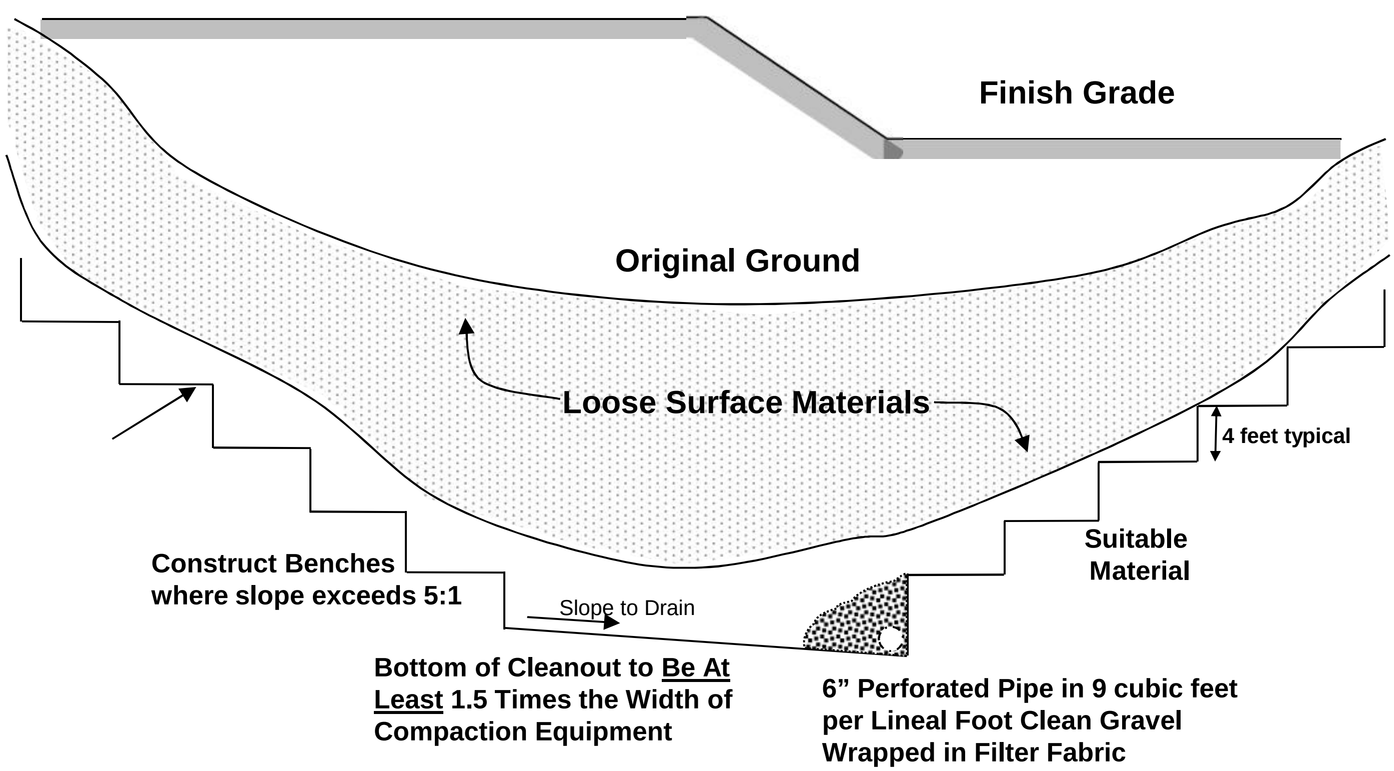
ALTERNATES





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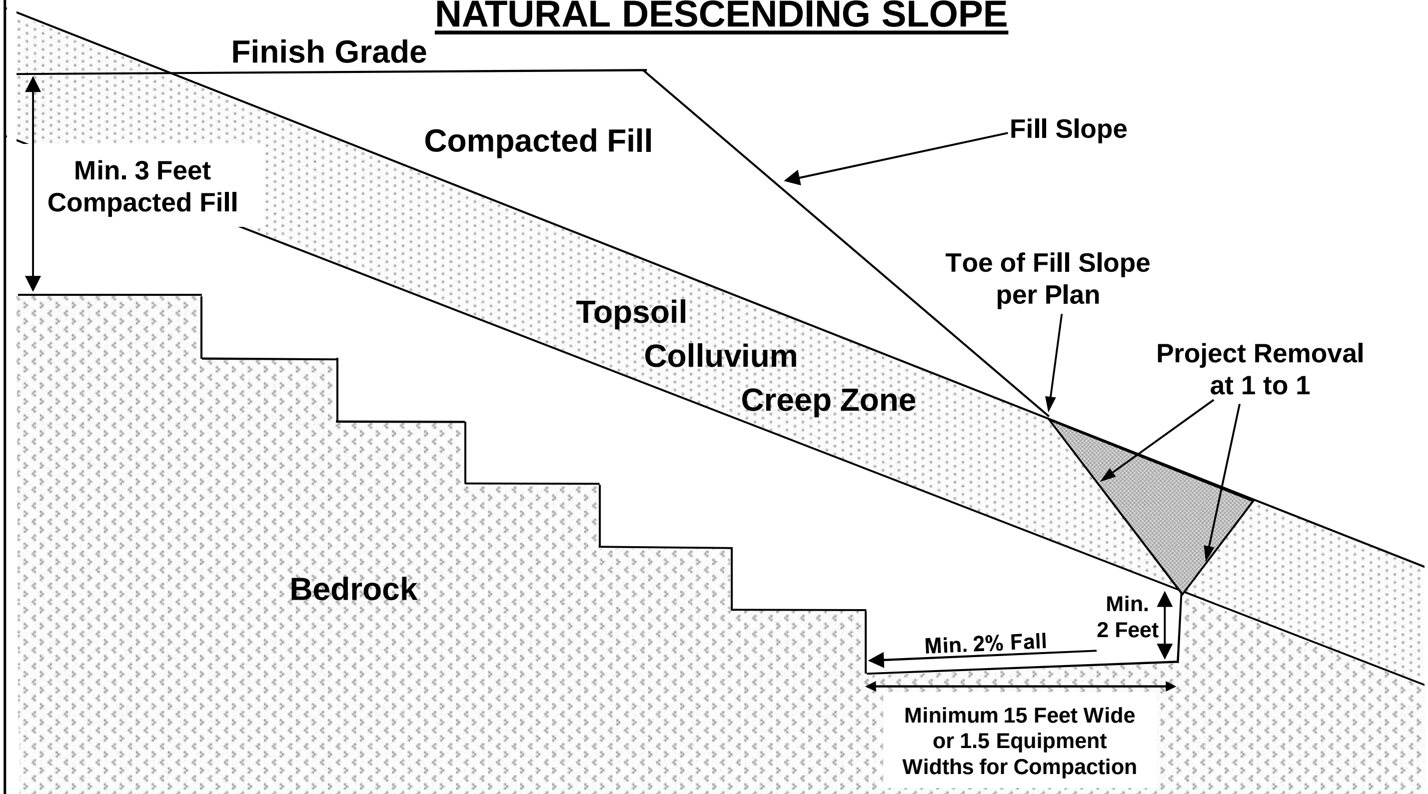
TYPICAL CANYON CLEANOUT

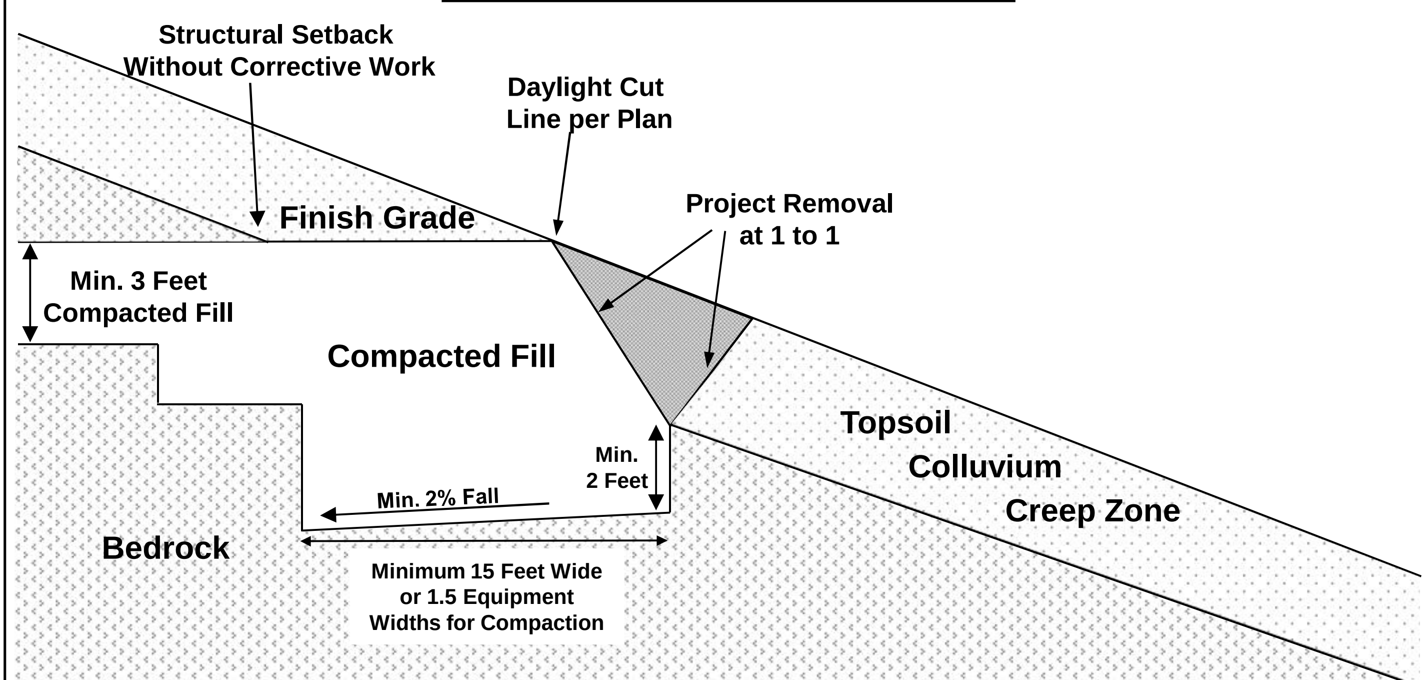
STANDARD GRADING GUIDELINES

PLATE G-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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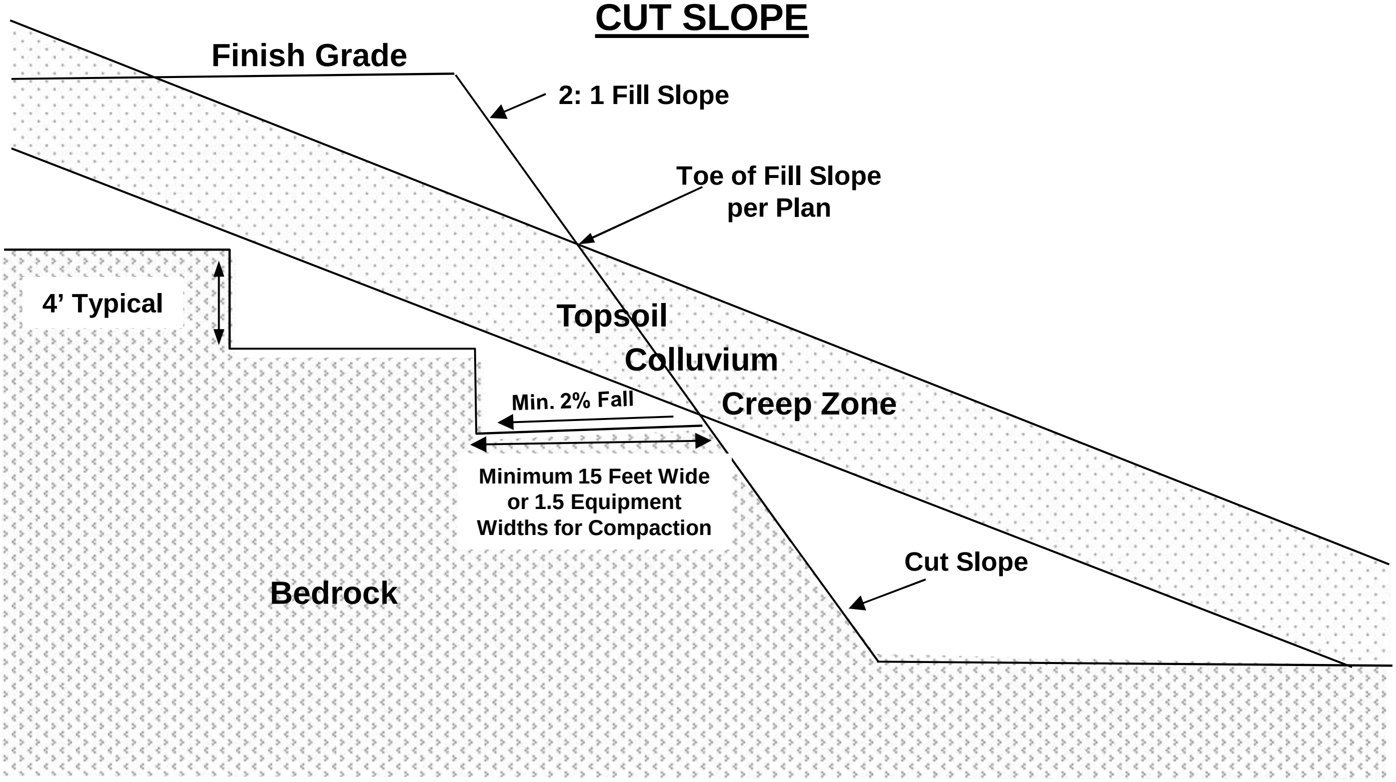
TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE G-2

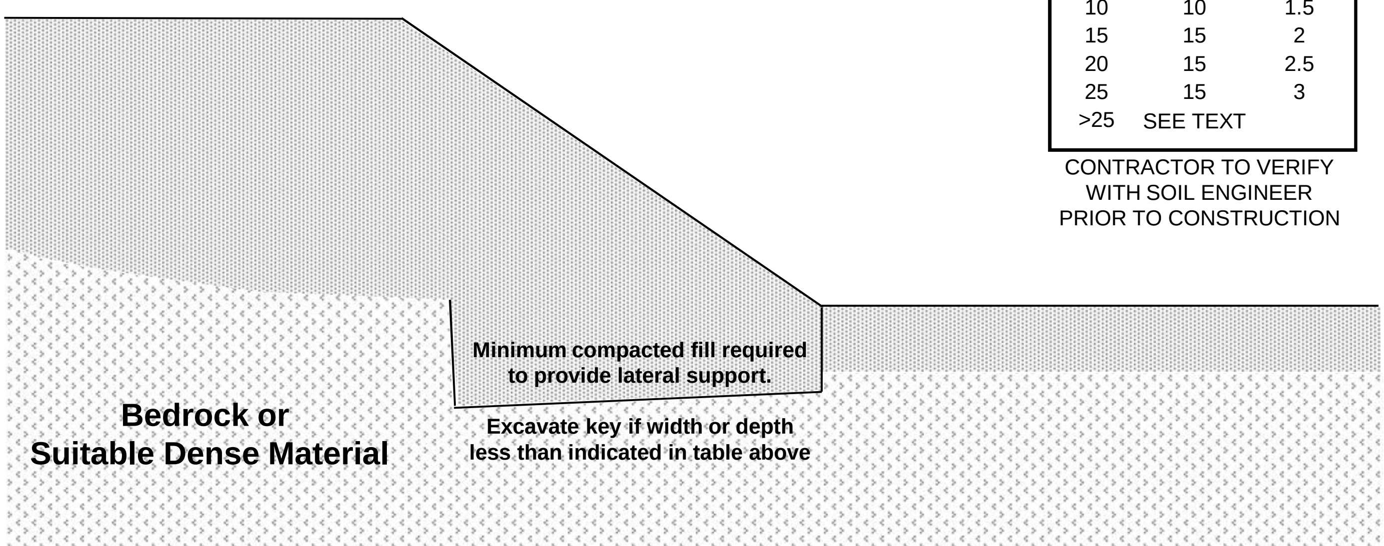
TYPICAL FILL SLOPE OVER CUT SLOPE



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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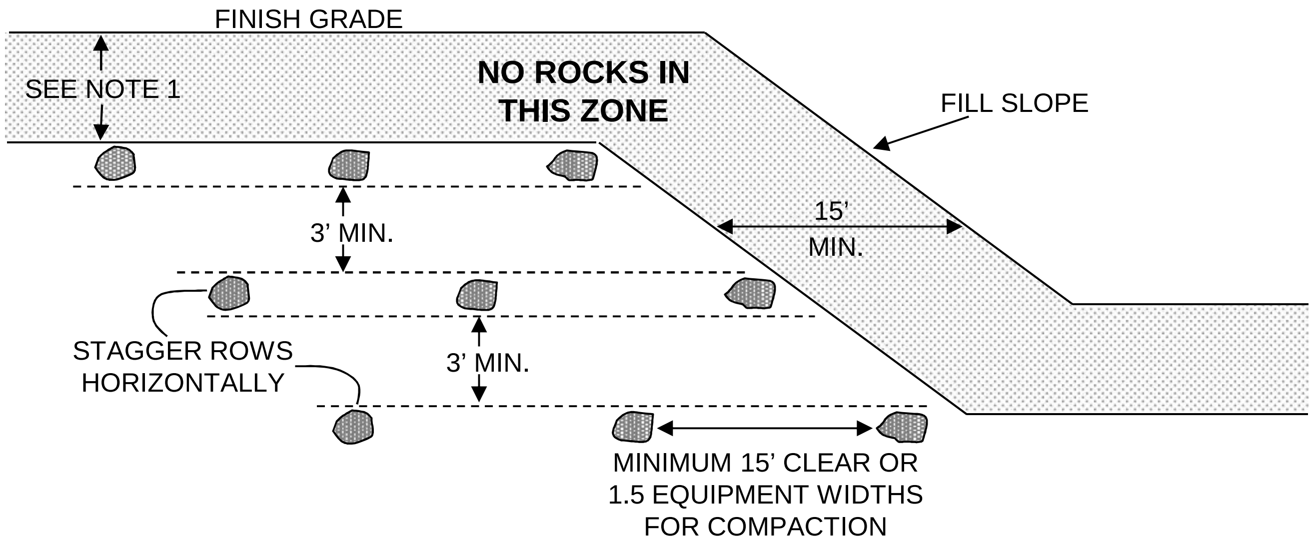
COMMON FILL SLOPE KEYS

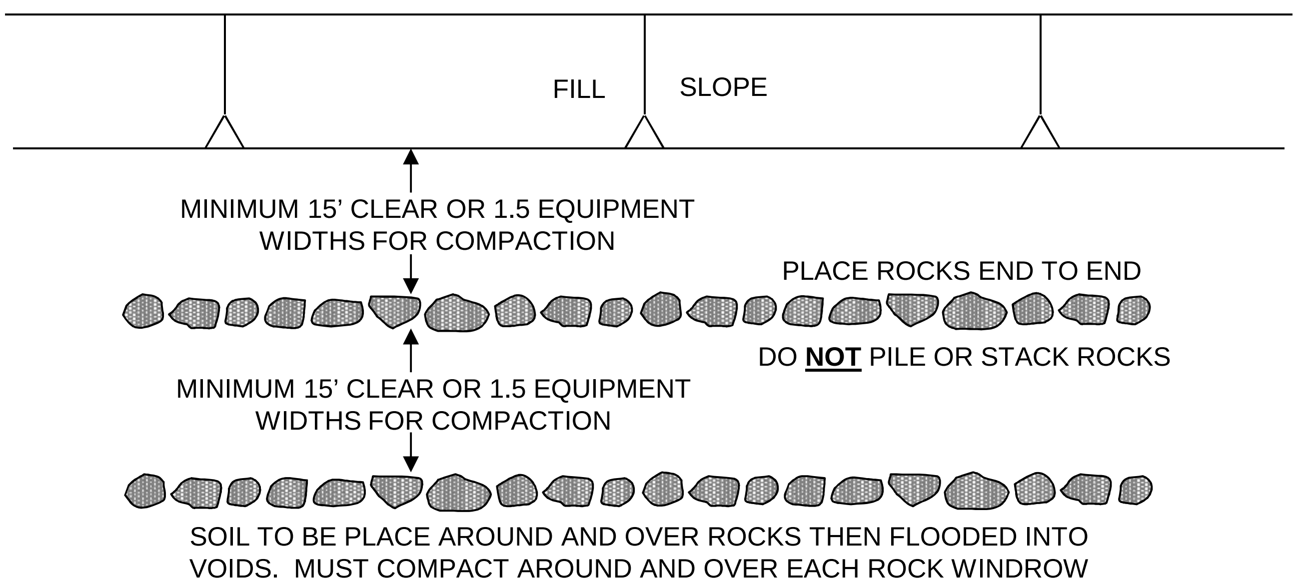
STANDARD GRADING GUIDELINES

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.

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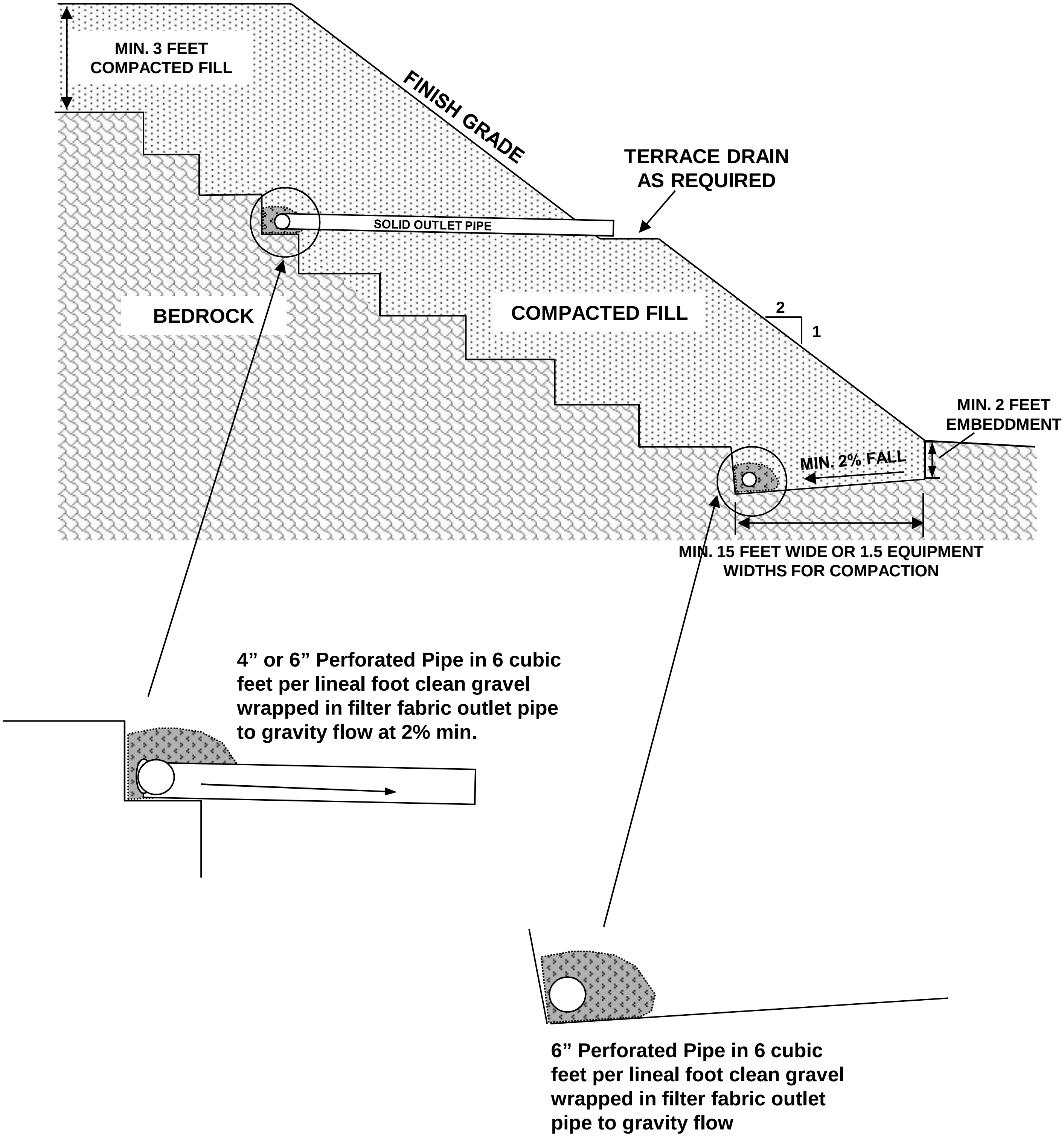
ROCK BURIAL DETAILS

STANDARD GRADING GUIDELINES

PLATE G-4



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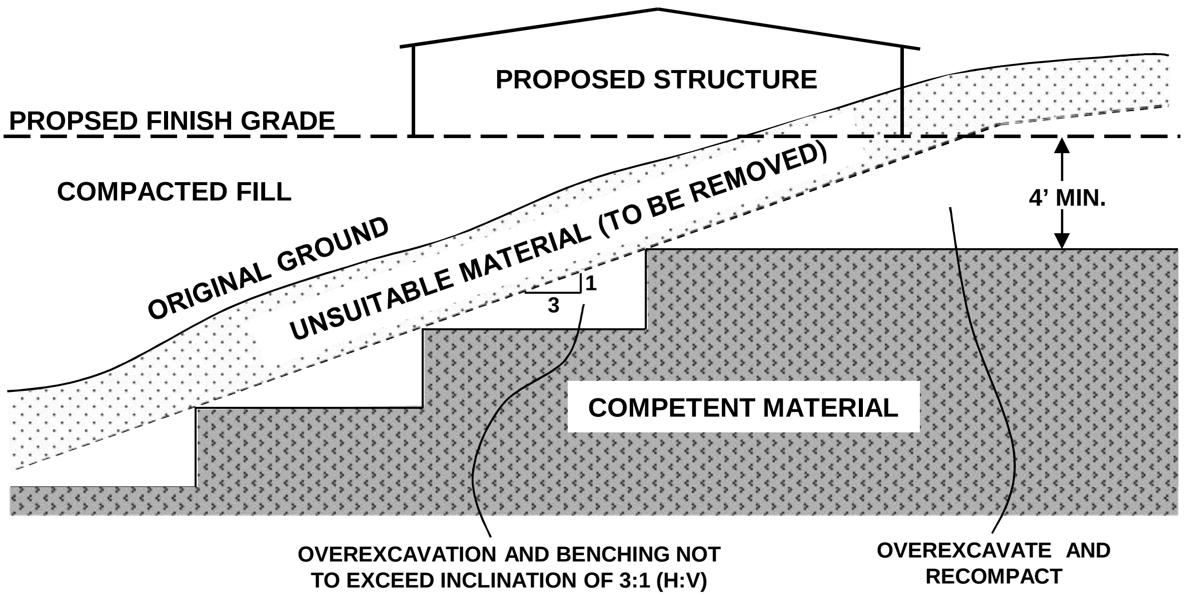
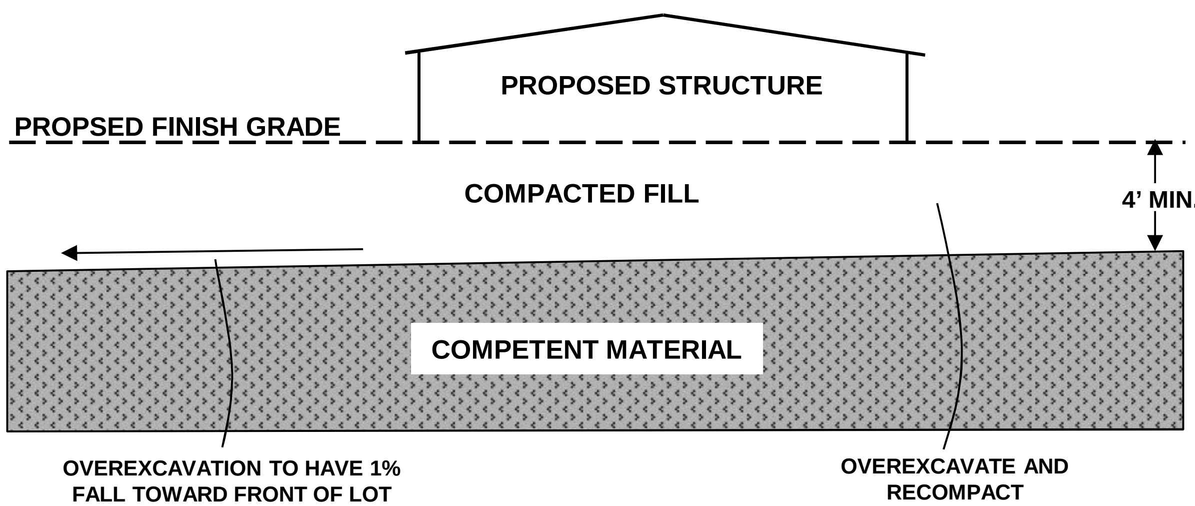
Typical Buttress and Stabilization Fill

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