Appendix F: Geology and Soils Report

GEOTECHNICAL EVALUATION
FOR
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
APNS 224-260-23, -46, AND -47
CITY OF ESCONDIDO, CALIFORNIA

PREPARED FOR

NUTMEG SOUTH, LLC
10632 MEADE AVENUE
ORANGE, CALIFORNIA 90869

PREPARED BY

GEOTEK, INC.
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PROJECT NO. 3539-SD JUNE 15, 2018

GEOTEK

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June 15, 2018
Project No. 3539-SD

Nutmeg South, LLC
10632 Meade Avenue
Orange, California 90869

Attention: Mr. John Martin

Subject: Geotechnical Evaluation
Proposed Multi-Family Residential Development
APNs 224-260-23, -46, and -47
Escondido, California

Dear Mr. Martin:

We are pleased to provide the results of our geotechnical evaluation for the subject property located in the city of Escondido, California. This report presents a discussion of our evaluation and provides geotechnical recommendations for earthwork, foundation design, and construction.

In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations presented in this report are incorporated into the design and construction phases of the project.

The opportunity to be of service is sincerely appreciated. If you 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 for the proposed improvements. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- A site reconnaissance,
- Evaluation of rock hardness on-site and along Centre City Parkway via seismic refraction surveys, performed by a subconsultant,
- Two field percolation test borings with testing to evaluate initial infiltration rate,
- Advancement of five hollow-stem auger geotechnical borings on-site,
- Advancement of seven hollow-stem auger geotechnical borings along the proposed sewer mainline within the right of way of Centre City Parkway,
- Collection of soil and bedrock samples,
- Laboratory testing of select soil and bedrock samples collected from our exploration program,
- Review and evaluation of site seismicity, and;
- Compilation of this geotechnical report which presents our findings, conclusions, and recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future proposed development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report might need to be updated based upon our review of the final site development plans. These plans should be provided to GeoTek, Inc. (GeoTek) for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The subject project site is located west of the intersection of Nutmeg Avenue and N. Centre City Parkway in the city of Escondido, California (see Figure 1). The irregular-shaped property is comprised of approximately 6.87 acres. The site is primarily vacant, undeveloped land with natural chaparral, shrubs, bush, grass and in the southern vicinity of the site a cluster of trees.

The site is bounded by Centre City Parkway to the east, Interstate 15 to the west, a California Department of Transportation (Caltrans) right-of-way easement to the south, and an undeveloped hillside to the north. Nutmeg Avenue bisects the northern 1/3 of the project from the southern 2/3. Topographically, the site lies in a depression, with ascending hillsides to the Caltrans and Centre City Parkway easements and an ascending hillside that comprises the northern 1/3 of the site. Topographic contours range between 940 and 878 feet above mean seal level (msl). Surface water generally sheet flows to the southwest corner of the site with the aide of minor natural drainage swales and gullies.

2.2 PROPOSED DEVELOPMENT

Based on a conceptual plan designed by Summa Architecture, 2018 (Figure 2) and information provided to us by the client, it is our understanding the proposed improvements include a multi-family residential development consisting of 31 buildings, hosting 140 homes, a recreation area, utilities, associated roadways, and landscaping.

Specific structural loading was not provided to us. Therefore, this report has been prepared based on the proposed residential structures will be of wood-frame construction, two to three stories and incorporate concrete slab-on-grade floors and conventional shallow foundations.

Based on our review of project documents prepared by an unknown author, plotted with a date of November 6, 2017, the site grading will involve moderate cuts up to 15 feet in height and placements of fill up to 30 feet in thickness in order to achieve design grades. Fill slopes of up to approximately 25 feet in height and cut slopes of up to 34 feet in height are proposed. Slopes will have a maximum gradient of 2:1 (H:V).

We understand that stormwater at the site may be managed via relatively shallow infiltration systems to be located within future common areas and a detention basin located in the south end of the development. Depths of these systems were unknown at the time of our evaluation.

For this evaluation, we conducted infiltration tests at two locations (northeast and south portions of the site) at five feet deep.

In addition, to provide sewer service to the proposed residential development, a sewer mainline is designed to be constructed within the right-of-way of Centre City Parkway. The sewer line is anticipated to extend from the southern end of project site to approximately the intersection of Centre City Parkway and West Country Club Lane. The sewer line is planned to be about 5 to 15 feet deep (Excel Engineering, 2017).

If site development differs from the information presented in this report, the recommendations would be subject to further review and evaluation by GeoTek. Final site development plans should be reviewed by GeoTek when they become available.

3. FIELD EXPLORATION, INFILTRATION, AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Our geotechnical field exploration was conducted on April 19, 2018 and May 18, 2018. A geologist from GeoTek logged twelve exploratory borings advanced by a conventional CME-75 truck mount hollow-stem auger drilling rig equipped with an automatic trip hammer. Five borings (B-1 through B-5) were advanced on site. Seven borings (B-6 through B-12) were advanced along and offset from the proposed sewer alignment within Centre City Parkway. The approximate locations of our exploratory borings are presented on Plates 1 through 4, Geotechnical Exploration Maps and logs of the exploratory borings are included in Appendix B. Samples of soils and bedrock encountered in the borings were returned to the laboratory for testing and evaluation.

In addition to the geotechnical borings, a seismic refraction survey was conducted on May 18, 2018 which involved recording of three seismic lines. Two seismic lines were performed on site and one seismic line was performed offsite along the shoulder of Centre City Parkway. The survey was performed by a subconsultant (Subsurface Surveys & Associates, Inc.). The seismic survey summary report is included in Appendix A.

3.2 INFILTRATION TEST INFORMATION

Two test borings (Borings I-1 and I-2) were excavated to a maximum depth of approximately five feet below the existing ground surface. Following drilling to the desired depth, the boring bottom and side walls were scarified and cleaned of potential drilling fines adhered to the boring walls. The test hole was then filled with water to pre-soak the test hole. Following overnight pre-soaking, the test holes were filled with water and the drop in water level as the water percolated into the underlying soil was recorded every 30 minutes. The test was continued for a minimum of twelve readings and the final reading was used in the calculation of the infiltration rate. The field data was converted to an infiltration rate via the Porchet method with an applied factor of safety of 2.

INFILTRATION TEST RESULTS	
Test No.	Infiltration Rate (inches per hour)
I-1	0.49
I-2	0.14

Copies of the percolation data sheets and infiltration conversion sheets (Porchet Method) are included in Appendix B.

Over the lifetime of the storm water disposal areas, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. If appropriate by others, we recommend that additional factors of safety be applied.

3.3 LABORATORY TESTING

Laboratory testing was performed on selected bulk and relatively undisturbed soil samples collected during our field exploration. The purpose of the laboratory testing was to confirm the field classification of the soils encountered and to evaluate their physical 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 C or on the boring logs included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. Basically, it extends from the point of contact with the Transverse Ranges geomorphic province, southerly 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 zone trend northwest-southeast and are found in the near middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

No faults are shown presently in the immediate site vicinity on the maps reviewed for the area. More specific to the subject property, the site is located in an area geologically mapped to be underlain by granitic bedrock (Monzogranite and Metavolcanic varities) (Kennedy and Tan, 2007).

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered at the subject site is presented in the following sections. Based on our field exploration and observations, the site is generally underlain by undocumented fill, topsoil, colluvium, younger and older alluvium and granitic bedrock (Monzogranite and Metavolcanic varieties).

4.2.1 Undocumented Artificial Fill (Afu)

Undocumented fill material was encountered in seven of the exploratory borings (B-1, B-2, B-6, B-7, B-8, B-11, and B-12). In general, these materials typically consist of brown medium dense silty sand with minor units of clayey sand and silty clay.

4.2.2 Quaternary Colluvium (Qcol)

Colluvial materials were not encountered in our exploratory borings, however, colluvium may exist at the base of steep slopes along the ascending slope located in the northern vicinity of the project. In general, colluvium material consists of unconsolidated, loose material transported in short distances from its origin and may contain abundant vegetation.

4.2.3 Quaternary Young Alluvium (Qal)

Young alluvium was encountered in four of the exploratory borings (B-2, B-3, B-4, and B-5). In general, the alluvial materials typically consist of light brown to brown, loose to medium dense, fine to medium grained, poorly graded sand to silty sand.

4.2.4 Quaternary Older Alluvium (Qoa)

Older alluvium was encountered in five of the exploratory borings (B-1, B-2, B-4, B-5, and B-7). In general, the older alluvial materials typically consist of light brown to brown and mottled brown-orange brown-reddish brown, dense to very dense, fine to medium grained, well cemented silty sand.

4.2.5 Cretaceous Granitic Bedrock (Kmm and Kjd)

Granitic bedrock (Monzogranite and Metavolcanic, undifferentiated) was encountered in all of the exploratory borings with the exception of B-5. Shallow refusal was encountered in all borings with the exception of B-5 and ranged between 2 and 20 feet below the ground surface (bgs). The depth of average refusal is approximately 12 feet bgs. The granitic material ranged from a weathered clay matrix to an unaltered crystalline matrix. With regards to rippability, please refer to both section 5.2.6 “Excavation Characteristics” and Appendix A of this report.

The approximate locations of the above described geologic units within the project area are shown on the attached Geotechnical Exploration Maps, Plates 1 through 4. Detailed description of the subsurface site conditions is provided in the borings logs, Appendix B.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed on the site during our subsurface exploration. If encountered during the earthwork construction, surface water on this site is the result of precipitation. Overall site area drainage is generally by sheet flow in a southwesterly direction. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Perched groundwater was encountered in boring B-5 at a depth of approximately 8 feet bgs within the young alluvial soils overlying the more dense older alluvial soils. Perched groundwater is anticipated to occur near the interface of material density differences, such as young alluvium overlying older alluvium and alluvium overlying granitic bedrock. Perched groundwater may travel along the undulatory near surface granitic bedrock and pond in low points and subject to seasonal storm events.

A groundwater table was not encountered in our exploratory excavations. According to the State Water Resources Control Board database (http://www.water.ca.gov/waterdatalibrary/) no groundwater data is available in the vicinity or pertinent to the site, therefore groundwater is not considered to be a design constraint toward the project, if it exists at all.

4.4 FAULTING AND SEISMICITY

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 presently known to exist at this site nor is the site situated within an "Alquist-Priolo" Earthquake Fault Zone. The nearest zoned fault is the Elsinore fault, located approximately thirteen miles to the northeast.

4.4.1 Seismic Design Parameters

The site is located at approximately 33.1662 Latitude and –117.1064 Longitude. Site spectral accelerations (Ss and S1), for 0.2 and 1.0 second periods for a Class "C" site, were determined from the USGS Website, Earthquake Hazards Program, U.S. Seismic Design Maps for Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Response Accelerations for the Conterminous 48 States by Latitude/Longitude.

The results are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.065g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.414g
Site Coefficient for Site Class "C", F_a	1.000
Site Coefficient for Site Class "C", F_v	1.386
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.065g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.574g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.710g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.382g
Peak Ground Acceleration adjusted for Site Class Effects, PGA_M	0.401g

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

4.5 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

Liquefaction is not considered to be a hazard at the subject site due to the current and historical absence of a groundwater table and the shallow presence of dense older alluvium and granitic bedrock.

The site rough grading is anticipated to remove all unsuitable materials and replace them with engineered compacted fill. Therefore, seismically-induced settlement of surficial sandy sediments is anticipated to be nil.

4.6 LANDSLIDES

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

4.7 ROCKFALLS

The natural terrain of a granitic hillside inherently exhibits core stone or boulders resulting from differential weathering of granitic material. As the surrounding weathered granitic material is eroded from around core stones, the core stones propagate out of the slope face creating apparent boulder outcrops which may or may not be well rooted or seated. Localized outcrops of core stones were observed within and upslope of the proposed development in the northern portion of the project. In addition, the proposed cut slope may expose core stones at design grades.

Based on our site investigation, the core stones observed located upslope of the development were found to be seated into the hillside. Provided that potential core stones exposed during grading of cut slopes are removed or evaluated to be reasonably stable, the potential for rockfalls is considered low.

4.8 OTHER SEISMIC HAZARDS

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

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations, and those provided by this firm at a later date, are properly incorporated into the design and construction phases of development. Site development and grading plans should be reviewed by GeoTek when they become available.

5.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Escondido, the 2016 California Building Code (CBC), California Department of Transportation (where applicable), and recommendations contained in this report. The Grading Guidelines included in Appendix D 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 D.

5.2.1 Site Clearing and Demolition

In areas of planned grading and improvements, the site should be cleared of vegetation, roots, and debris. These materials should be properly disposed of off-site. Voids resulting from site clearing should be replaced with engineered fill materials with expansion characteristics similar to the on-site soils.

5.2.2 Removals

All of the undocumented fill, colluvium and younger alluvial materials should be removed below proposed structural improvements. A geologist of this firm should observe the bottom of all excavations.

In order to provide a reasonably uniform bearing surface and to accommodate construction, bedrock below the proposed footings and floor-slabs supporting the residential structures should be overexcavated to a minimum depth of five (5) feet below proposed grade on all proposed cut lots. Transition (i.e. cut/fill) lots should be overexcavated a minimum of five (5) feet below proposed grades or to a depth of 1/3 the maximum fill thickness whichever is deeper. The horizontal extent of removals/overexcavations should extend at least five (5) feet outside the perimeter of the footings and floor-slabs, or a distance equal to the depth of overexcavation below the bottom of the structural elements, whichever is greater.

A minimum of 24 inches of engineered fill should be provided below the bottom of the proposed footings.

5.2.2.1 Pavement and Hardscape Areas

A minimum of 24 inches of engineered fill should be provided below asphaltic concrete pavement and Portland cement concrete hardscape areas. The horizontal extent of removals should extend at least two (2) feet beyond the edge, where possible.

5.2.2.2 Preparation of Areas to Receive Engineered Fill

A representative of this firm should observe the bottom of all excavations. Upon completion of the excavation, the exposed materials and all soils in areas to receive engineered fill should be scarified to a depth of approximately six (6) inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent (ASTM D 1557).

5.2.3 Engineered Fills

The on-site materials are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. The undercut areas should be brought to the final subgrade elevations with fill materials that are placed in lifts, eight (8) inch or less in loose thickness, moisture conditioned to at least the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM Test Method D 1557.

Where fill is to be placed against slopes inclined at inclinations of 5 horizontal to 1 vertical or steeper, the sloping ground surface should be benched to remove loose and disturbed surface soil and bedrock to assure that the new fill is placed in direct contact with competent, undisturbed soil or bedrock, and to provide horizontal surfaces for fill placement. The upper one (1) foot of structural pavement subgrade should be compacted to 95 percent.

5.2.4 Slope Construction

In accordance to industry standards, cut and fill slopes up to a maximum height of 30 feet and to a maximum gradient of 2:1 (horizontal to vertical) are considered to be stable. Preliminary plans appear to indicate that some site slopes may be taller than 30 feet. The stability of these tall slopes should be evaluated once final grading plans for the site are available.

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

Fill slopes should be constructed with a 15-foot wide keyway at the toe of the fill slopes and extend at least five (5) feet vertically into dense natural material. The base of the keyways and benches for fill over cut slopes should be sloped back into the hillside at a gradient of at least 2 percent. The base of the benches should be evaluated by a geologist from this firm prior to processing. Backdrains should be installed in the keyways in accordance with the recommendations outlined in Appendix D. Upon approval, the exposed soils should be moistened to at least the optimum moisture content and densified to a relative compaction of at least 90 percent (ASTM D 1557). Details showing slope construction are presented in Appendix D.

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

5.2.5 Oversized Materials

Oversized materials (larger than six (6) inches in dimension) are anticipated to be generated during rough grading. Placement of such materials will require special handing. Oversized materials may be placed in accordance to methods outlined in Appendix D "Grading Guidelines" of this report. The preferred location of oversized rock placement is within open spaces and landscaped areas below 3 feet of pad grade and 5 feet horizontally from the slope face. Oversized material placed below a building pad should not be placed within 10 feet of pad grade to provide a clean and neat-cut environment for foundation excavations. Oversized material should not be placed within the utility backfill zone or within two feet of the utility invert to provide a safe excavation environment during utility excavation and construction, as well as, provide the ability for the excavated cuttings to be reused as utility trench backfill. Alternatively, the rocks should be reduced in size or removed from the site.

Additional recommendations may be necessary based on exposed conditions during earthwork construction.

5.2.6 Excavation Characteristics

Excavation in the on-site soils (alluvium, colluvium, and undocumented fill) is expected to be feasible utilizing heavy-duty grading equipment well operated and in good operating condition. Some oversized materials may be present in old fills.

The following paragraphs briefly summarize the data as it relates to the rock hardness and material excavation characteristics. This summary includes data from the CME-75, 8-inch diameter, hollow-stem truck mounted auger (see Appendix B) and the seismic refraction survey (see Appendix A). Two general areas of concern are addressed 1) the design cuts located in the northeastern vicinity of the site and 2) the offsite proposed sewer alignment along Centre City Parkway. In general, the methods available to assess rock hardness and related excavation characteristic all have various limitations. In certain conditions it may be more cost effective to rip a small quantity of hard rock (given equipment wear and tear), than fracturing hard rock by blasting techniques in a mass grade condition. In general, rock displaying high seismic velocities can be more readily excavated in an open cut than in a trench condition. Contractors who perform work in a hard rock environment can better asses their ability to excavate potential hard rock materials.

Northeastern Cuts

Based on the results of the seismic refraction survey, onsite bedrock materials in the northwest corner of the site are considered rippable with a Caterpillar D-9R/T to depths ranging from 0 to 13 feet bgs, SR-1 further suggests rippable materials (<3200fps) to depths varying from 10 up to about 25 feet with a rather variable non-rippable (9700fps) unweathered bedrock surface below those depths. SR-2 indicates the upper 7 to 8 feet is relatively soft having velocity of ~1300fps underlain by higher velocity (5090fps) extending to depthe of ~25 to 33 feet, again with non-rippable (9700fps) unweathered bedrock surface below those depths. Our experience suggests that while, as indicated by Caterpillar publications materials with reasonably high velocities are rippable, the costs associated with grading materials exceeding velocities greater the ~4500fps typically increase and materials faster than 6000fps may be more effectively excavated using blasting in mass excavation. Results of this survey are provided in Appendix A. Once the site is approved and environmental clearances obtained additional exploration in this area could better delineate conditions.

Center City Parkway Sewer Alignment

The offsite proposed sewer alignment was evaluated with one seismic line, SR-3 and seven borings spaced from roughly 360 to 600 feet apart advanced with a conventional CME-75, 8-inch diameter, hollow-stem truck mounted auger. Borings were extended to depths of 20 feet or to practical refusal. Interpretation of the data is presented on Plates 2 through 4. In general it appears that trenching with typical large excavators (Cat 225 or equivalent) from Sta 1+00 to about 10+50 is feasible. From 10+50 to 20+00 it appears the trench would be wholly excavated in bedrock with potentially very hard rock near the bottom. From 20+00 on, the majority of the trench appears to be into very hard rock. The very hard rock is considered likely to require non-typical trenching methods (e.g. line blasting, rock trencher, etc.) The excavation feasibilities presented may vary erratically beyond +/- 50 feet horizontally from explorations. It should further be noted that seismic line SR-3 performed between approximate stations 25+00 and 26+00 was offset from the proposed alignment, and given the shallow bedrock, velocities might not be presented as accurate and actual higher velocities (harder rock) is highly possible. When reviewing SR-3 data it should be noted that the planned sewer alignment lies in an existing cut area from prior construction thus the shallow slower velocity material represented on SR-3 appears to have been at least partially removed by prior road construction at the sewer alignment.

Rippability is generally correlated to conventional heavy-duty earth moving equipment such as a Caterpillar dozer series D-8 to D-11 equipped for ripping, however correlation of velocities to rippability with conventional trench excavation equipment (e.g. Caterpillar 345 to 385 excavators) is limited. Below these depths, specialized rock trenching equipment is highly probable to be required during utility construction. We strongly recommend a qualified utility construction company specializing in a hard rock environment be consulted and used for construction.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the on-site materials should be stable at 1:1 (horizontal:vertical) inclinations for cuts less than five (5) feet in height.

Although we did not provide grading logistics as part of our scope of work or specifically address such efforts in this report, in general, it may be prudent to perform rough grading in a manor as to provide an area of deep fill for disposal of oversized rock generated during blasting of bedrock or oversized material generated from rippable areas of bedrock. This might include stockpiling of soil from removal areas and placement of rock fill/oversized rock in deeper fill. Stockpiled materials could then be used for capping. In addition, over-excavate for utility trenches in areas of bedrock with the heavy earthwork equipment during rough grading operations for ease of utility construction.

5.2.7 Shrinkage and Subsidence

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

Shrinkage is primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 15 percent may be considered for the undocumented fill, topsoil and alluvial soil. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction. Subsidence is not considered to be a factor with the underlying materials within the vicinity of the proposed construction.

Bulking of the bedrock cuts may occur during grading. A bulking factor of 5 to 20 percent may be considered for the bedrock placed as compacted fill. Generally, the degree of bulking will increase with depth of the cut as the granitic rock becomes more dense & less weathered.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2016 CBC, are presented below. Based on our laboratory test results, the on-site materials have "very low" to "medium" expansion potential ($0 \leq EI < 90$) in accordance with ASTM D 4829. Typical design criteria for the site based upon expansion potentials are tabulated below. These are minimal recommendations and are not intended to supersede the design by the project structural engineer.

The foundation elements for the proposed structures should bear entirely in engineered fill soils. Foundations should be designed in accordance with the 2016 California Building Code (CBC).

Expansion index and soluble sulfate testing of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions.

A summary of our foundation design recommendations is presented in the following table:

GEOTECHNICAL RECOMMENDATIONS FOR FOUNDATION DESIGN			
Design Parameter	"Very Low" Expansion Potential 0≤EI≤20	"Low" Expansion Potential 21≤EI≤50	"Medium" Expansion Potential 51≤EI≤90
Foundation Depth or Minimum Perimeter Beam Depth (inches below the lowest adjacent grade)	One- & Two-story – 12 Three-story – 18	One- & Two-story – 12 Three-story – 18	One- & Two-story – 18 Three-story – 18
Minimum Foundation Width (inches)*	One- & Two-story – 12 Three-story – 15	One- & Two-story – 12 Three-story – 15	One- & Two-story – 12 Three-story – 15
Minimum Slab Thickness (inches)	4 – Actual	4 - Actual	4 – Actual
Sand Blanket and Moisture Retardant membrane below On-Grade Building Slabs	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**	2 inches of sand** overlying moisture vapor retardant membrane overlying 2 inches of sand**
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric, placed in the middle of slab	6" x 6" – W2.9/W2.9 welded wire fabric, placed in the middle of slab	No. 3 rebar 18 inches on-center, each way, placed in middle of slab
Minimum Footing Reinforcement for Continuous Footings, Grade Beams and Retaining Wall Footings	Two No. 4 reinforcing bars, one placed near the top and one near the bottom	Two No. 4 reinforcing bars, one placed near the top and one near the bottom	Four No. 4 reinforcing bars, two placed near the top and two near the bottom
Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)	Minimum of 100% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum of 110% of the optimum moisture content to a depth of at least 12 inches prior to placing concrete	Minimum of 120% of the optimum moisture content to a depth of at least 18 inches prior to placing concrete

* Code minimums per Table 1809.7 of the 2016 CBC
** Sand should have a sand equivalent of at least 30

As shown by the preliminary site development plans, significant grading with cuts up to 34 feet and fill placements up to 34 feet are anticipated to reach design grades. For preliminary purposes and assuming that relatively granular materials will support structural foundations, an allowable bearing capacity of 1,800 pounds per square foot (psf) may be used for design of continuous footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This value may be increased by 400 psf for each additional 12 inches of embedment depth and by 200 psf for each additional 12 inches in width to a maximum of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering short-term wind and seismic loads.

For footings designed in accordance with the recommendations presented in this report, we would anticipate a maximum static settlement of less than one (1) inch and a maximum differential static settlement of less than ½-inch in a 40-foot span.

The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.40 may be used with dead load forces. The upper one foot of soil below the adjacent grade should not be used in calculating passive pressure. When combining passive and frictional resistance, the passive pressure component should be reduced by one-third.

5.3.2 Underslab Moisture Retarders

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 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

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 aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a six (6) mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

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. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarders 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 recommends that a qualified person, such as the flooring contractor, structural engineer, and/or architect be consulted to evaluate the general and specific moisture vapor transmission paths and associated potential impact.

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 areas of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

5.3.3 Miscellaneous Foundation Recommendations

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

- Soils 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.

- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If soils to be used as backfill have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

5.3.4 Foundation Set Backs

Minimum setbacks for all foundations should comply with the 2016 CBC or City of Escondido requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The bottom of all footings for new 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 footing.
- The top outside edge of all building, retaining wall and screen wall 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 seven (7) feet and need not exceed 40 feet.
- The outside edge of all footings should be set back a minimum of H/2 (where H is the slope height) from the face of any ascending slope. The setback should be at least five (5) feet and need not exceed 15 feet.

5.3.5 Retaining and Garden Wall Design and Construction

5.3.5.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete vertical retaining walls to a maximum height of up to six (6) feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be embedded a minimum of 12 inches into engineered fill or bedrock. Retaining wall foundations should be designed in accordance with Sections 5.3.1 of this report. Allowable bearing could be increase in bedrock cut areas. Structural needs may govern and should be evaluated by the project structural engineer.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 5.3.4.3 in this report.

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the earth retention structure, may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at top, such as typical basement walls) should be designed using the at-rest condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent building or traffic loading, should be considered in the design of the earth retention structures. Loads applied within a 1:1 (h:v) projection from the surcharge on the stem of the earth retention structure should be considered in the design.

Final selection of the appropriate design parameters should be made by the designer of the earth retention structures.

5.3.5.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six (6) feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES		
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure (pcf) On-Site Granular Alluvial Materials*	Equivalent Fluid Pressure (pcf) On-Site Bedrock Materials**
Level	43	38
2:1	70	55

*The design pressures assume the wall backfill will consist of on-site granular alluvial materials with a friction angle of at least 30 degrees and an expansion index less than 20. Backfill zone includes area between back of the wall to a plane (1:1 h:v) up from bottom of the wall foundation (on the backside of the wall) to the ground surface.

**The design pressures assume the wall backfill will consist of on-site remolded bedrock materials with a friction angle of at least 34 degrees and an expansion index less than 20. Backfill zone includes area between back of the wall to a plane (1:1 h:v) up from bottom of the wall foundation (on the backside of the wall) to the ground surface.

Values of earth pressures should be confirmed when the wall backfill materials are generated by conducting direct shear testing on the backfill.

5.3.5.3 Restrained Retaining Walls

Retaining walls that will be restrained at the top that support level backfill or that have reentrant or male corners, should be designed for an equivalent at-rest fluid pressure of 65 pcf for walls backfilled with native granular alluvial materials or 60 pcf for walls backfilled with native bedrock materials, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.3.5.4 Retaining Wall Backfill and Drainage

The backfill materials should exhibit an expansion index less than 20 and be placed in lifts no greater than eight (8) inches in thickness and compacted to a minimum of 90% relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained. The wall backfill should also include a minimum one (1) foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. A four (4)-inch diameter perforated collector subdrain pipe (Schedule 40, SDR 35, or approved equivalent) should be embedded into the gravel near the base of the wall and connected to a suitable outlet. The upper 24 inches should consist of compacted on-site materials. The rock should be separated from the earth with Mirafi 140N filter fabric (or an approved equivalent). Where filter fabric in sections is required, the fabric should overlap each other a minimum of 12 inches. The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs.

As an alternative to the drain, rock and fabric, Miradrain 2000 or approved equivalent may be used behind the retaining wall. The Miradrain 2000 should extend from the subdrain near the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the Miradrain 2000.

5.3.5.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are verified by compression testing of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

5.3.6 Soil Corrosivity

There is a wide range of soil characteristics that can be used to determine a site to be corrosive. Herein, Caltrans definition for a corrosive environment is used. Although the site development is not under the jurisdiction of Caltrans a majority of the offsite improvements are, therefore it is our professional opinion Caltrans methodology can be used as a preliminary indicator for characterizing soils onsite with regards to corrosivity. The California Department of Transportation (Caltrans) defines a corrosive environment in terms of resistivity, pH, and soluble salt content of the soil and/or water. Resistivity serves as an indicator for the possible presence of soluble salts. Caltrans considers a site to be corrosive if one or more of the following conditions exists:

- Resistivity value is less than 1,100 ohm-cm indicates;
- Chloride concentration is 500 ppm or greater;

- Sulfate concentration is 1,500 ppm (0.0015%)or greater; or
- pH is 5.5 or less;

The results of testing indicate that the on-site soils are not considered to be defined as a corrosive environment by Caltrans standards. Sample B-2 did exhibit a resistivity value less than 1,110 ohms-cm, however we interpret this as an outlier when compared to the other samples. The table below summarizes the results of corrosion testing performed on representative samples of the materials anticipated to be exposed near final surface conditions.

SUMMARY OF CORROSION TEST RESULTS					
Sample Location	Depth (Feet)	Resistivity (ohms-cm)	Sulfates (wt%)	Chlorides (ppm)	pH
B-1	0-5	3,819	0.0120	120	9.00
B-2	0-5	804	0.0180	144	8.46
B-8	1-5	5,427	0.0060	75	7.93
B-12	10	22,110	0.0150	90	7.30

However, we recommend that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site. Additional testing should be performed during site grading to assess the corrosivity of the as-graded soils.

5.3.7 Soil Sulfate Content

The sulfate content was determined in the laboratory for four soil samples. The results indicate that the water soluble sulfate result is less than 0.1 percent by weight, which is considered "not applicable" (negligible) as per Table 4.2.1 of ACI 318. Based on the test results and Table 4.3.1 of ACI 318, special concrete mix design is not necessary. However, additional soluble sulfate testing should be performed during site grading to assess the sulfate levels within the as-graded soils.

5.3.8 Pavement Design

Our field observations and R-Value test results indicate that the materials in the project area are mostly granular with an R-value of 50 or better. Based on a design R-value of 50 for pavement subgrade soils and using Traffic Indices (TI) of 4.5 and 6.0 generally required by the City of Escondido for residential sites and a TI of 8.0 for the pavement of the sewer trench along Centre City Parkway, the following preliminary structural sections are recommended for project:

PRELIMINARY STRUCTURAL SECTIONS		
Traffic Index	Asphaltic Concrete (AC) Thickness (Inches)	Aggregate Base (AB) Thickness (Inches)
4.5	3*	6*
6.0	3*	6*
8.0	4*	8*

*Minimum thickness per City of Escondido Design Standards (2014)

Traffic Indices (TIs) used in our pavement design are considered reasonable values for the proposed residential street areas and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving will result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper five feet) after final grading has been completed.

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

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

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557). Rock fragments over six inches in one dimensions should not be placed within the upper 12 inches of the subgrade. If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

5.3.9 Import Soils

Import soils should have expansion characteristics similar to the on-site soils. GeoTek also recommends that the proposed import soils be tested for expansion and sulfate potential. If imported soils are planned to be used for foundation support and/or wall backfill, these materials should also be tested in direct shear. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.3.10 Concrete Flatwork

5.3.10.1 Exterior Concrete Slabs, Sidewalks, and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four (4) inch minimum thickness. No specific reinforcement is required from a geotechnical perspective. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. "Very low" expansive subgrade soils below exterior slabs, sidewalks, driveways, etc. at the subject site should be pre-saturated to a minimum of 100% of optimum moisture content to a depth of at least 12 inches. "Low" and "medium" expansive subgrade soils should be pre-saturated to at least 110% of optimum moisture content to a minimum depth of 12 inches and to 120% of optimum to a minimum depth of 18 inches, respectively.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Escondido specifications, and under the observation and testing of GeoTek and a City inspector, if necessary.

5.3.10.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 0.125-inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent upon a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two orthogonal directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. We suggest that the same standards of care be applied to these features as to the structures themselves.

5.4 POST CONSTRUCTION CONSIDERATIONS

5.4.1 Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground, near-surface perched water, or seeps may result if irrigation water is excessively or improperly applied. All irrigation systems should be adjusted to provide the minimum water needed to sustain landscaping and prevent excessive drying of the soils. Generally significant runoff during an irrigation cycle indicates excessive irrigation, while soils which dry to a depth of more than several inches between irrigation cycles indicate inadequate irrigation. Adjustments should be made for changes in the climate and rainfall. Irrigation should stop when sufficient water is provided by precipitation.

It is important to avoid repeated wetting and drying of the slope surface, which may cause the soil to crack, loosen and/or slowly move laterally (creep) downslope. Landscaping and irrigation will reduce repeated wetting and drying of the slopes.

It is important to maintain uniform soil moisture conditions adjacent to the structure to reduce soil expansion and shrinkage that can cause cracking to the structure. Irrigation should be utilized to prevent the soils from drying to a depth more than several inches.

Broken, leaking or plugged sprinklers or irrigation lines should be repaired immediately. Frequent inspections of the irrigation systems should be performed.

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 care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be necessary and advisable.

5.4.2 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 adjacent to the footings. Soil areas within 10 feet of the proposed structure should slope at a minimum of five (5) percent away from the building, if possible unless the area is paved. Paved areas are to be sloped at two (2) percent away from the structure. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas 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.

5.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that specifications and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to observe and document proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek representatives 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 where necessary.
- Observe the fill for uniformity during placement, including utility trench backfill. Also, perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

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. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of our evaluation is limited to the boundaries of the subject project. This review does not and should in no way be construed to encompass any areas beyond the specific area of the 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 No. P-0204618 dated February 20, 2018 and geotechnical engineering standards normally used on similar projects in this locality at the present.

7. LIMITATIONS

Our findings are based on site conditions observed and the stated sources. GeoTek has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report.

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 of any kind is expressed or implied. Standards of care/practice are subject to change with time.

8. SELECTED REFERENCES

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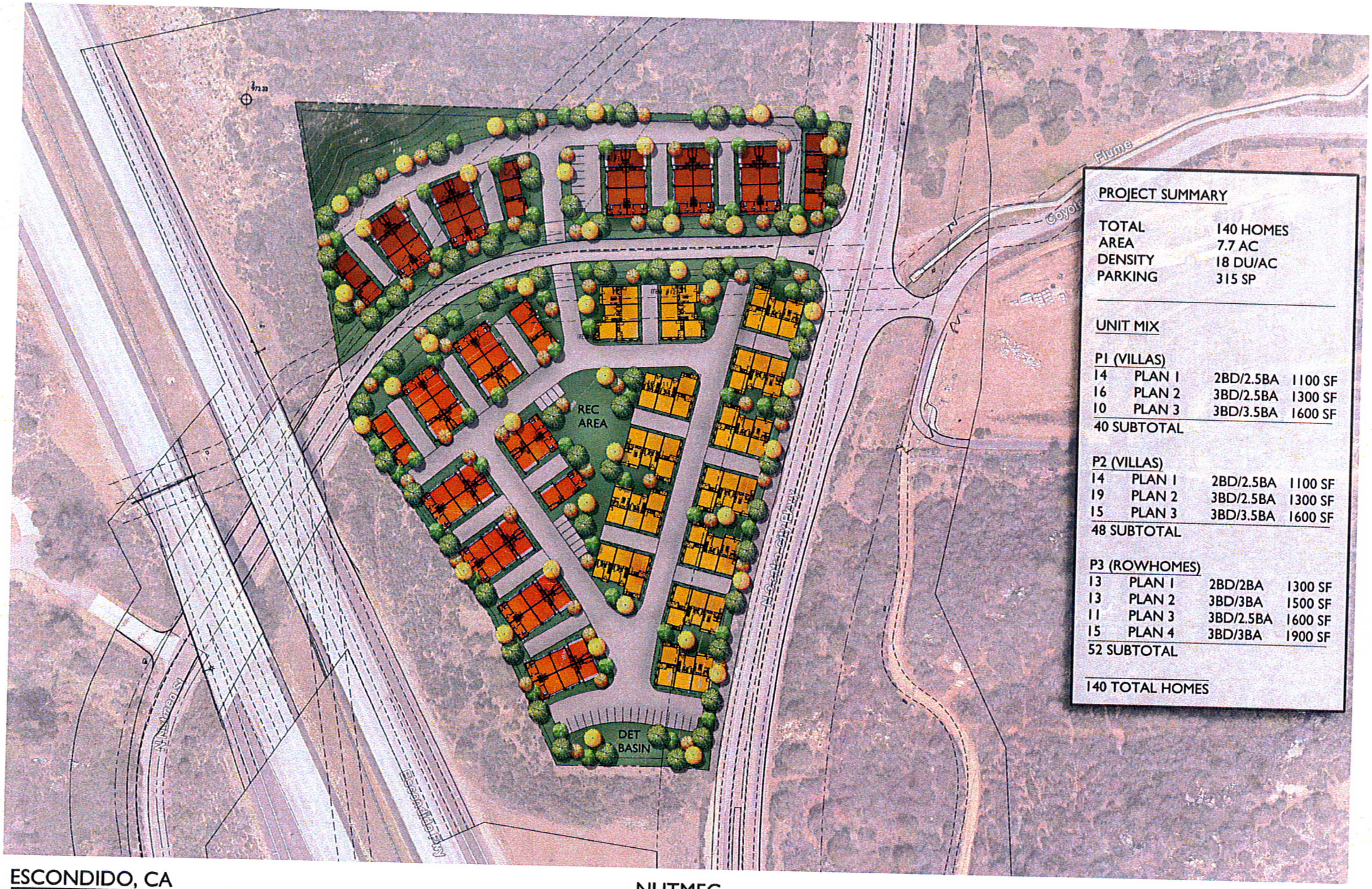
Imagery from Google Earth Pro, 2018, Imagery date November 8, 2016

0 ft 8305 ft

Nutmeg South, LLC Assessor's Parcel Number (APN) 224-260-23, -46, AND -47 Escondido, California GeoTek Project No. 3539-SD	NORTH	**Figure 1** Site Location Map	GEOTEK



ESCONDIDO, CA

JMI REAL ESTATE
10632 MEADS AVE
ORANGE, CA 92869

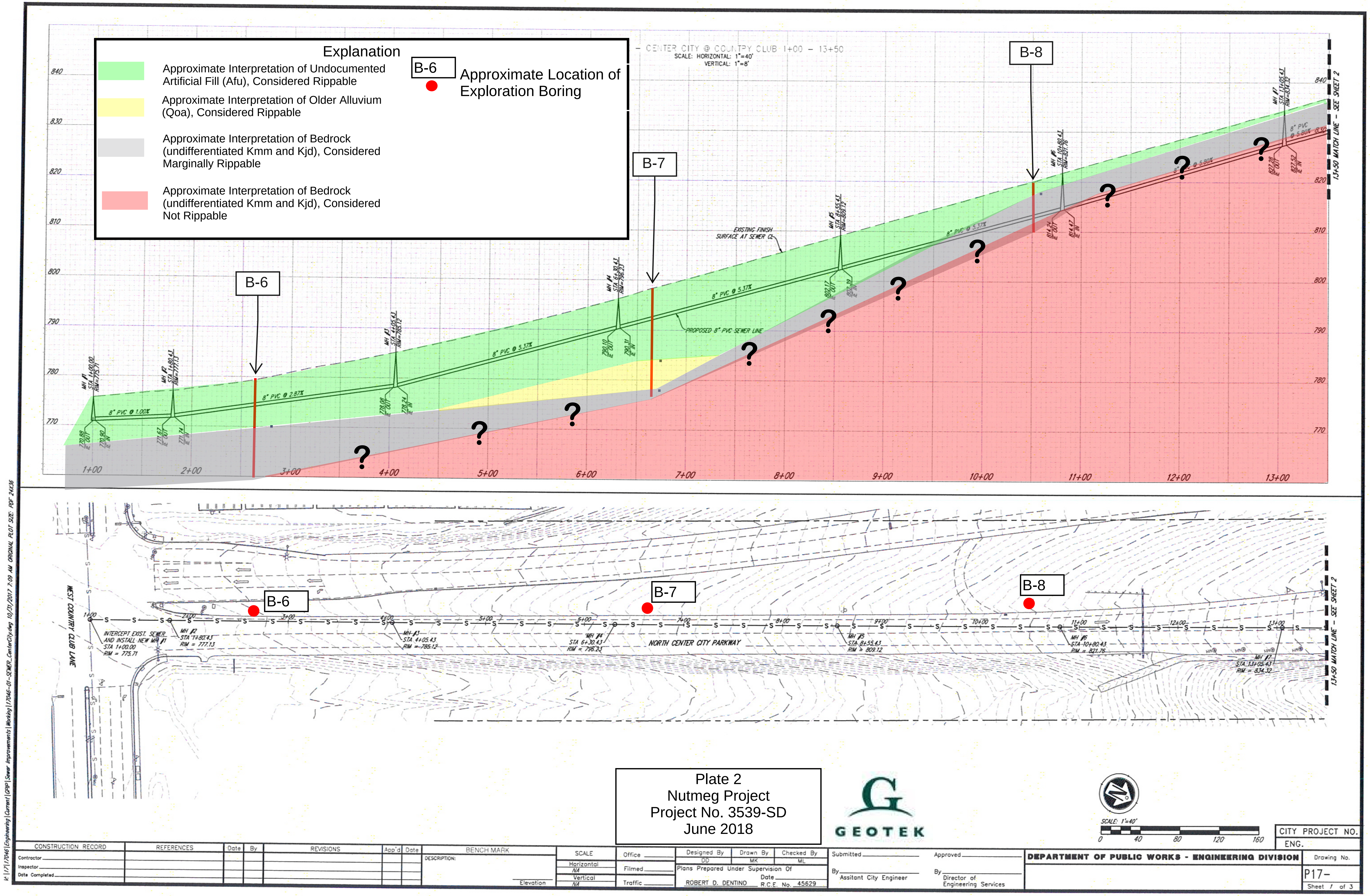
NUTMEG
JANUARY 11, 2018

SCALE: 0 40 80 120

SITE PLAN

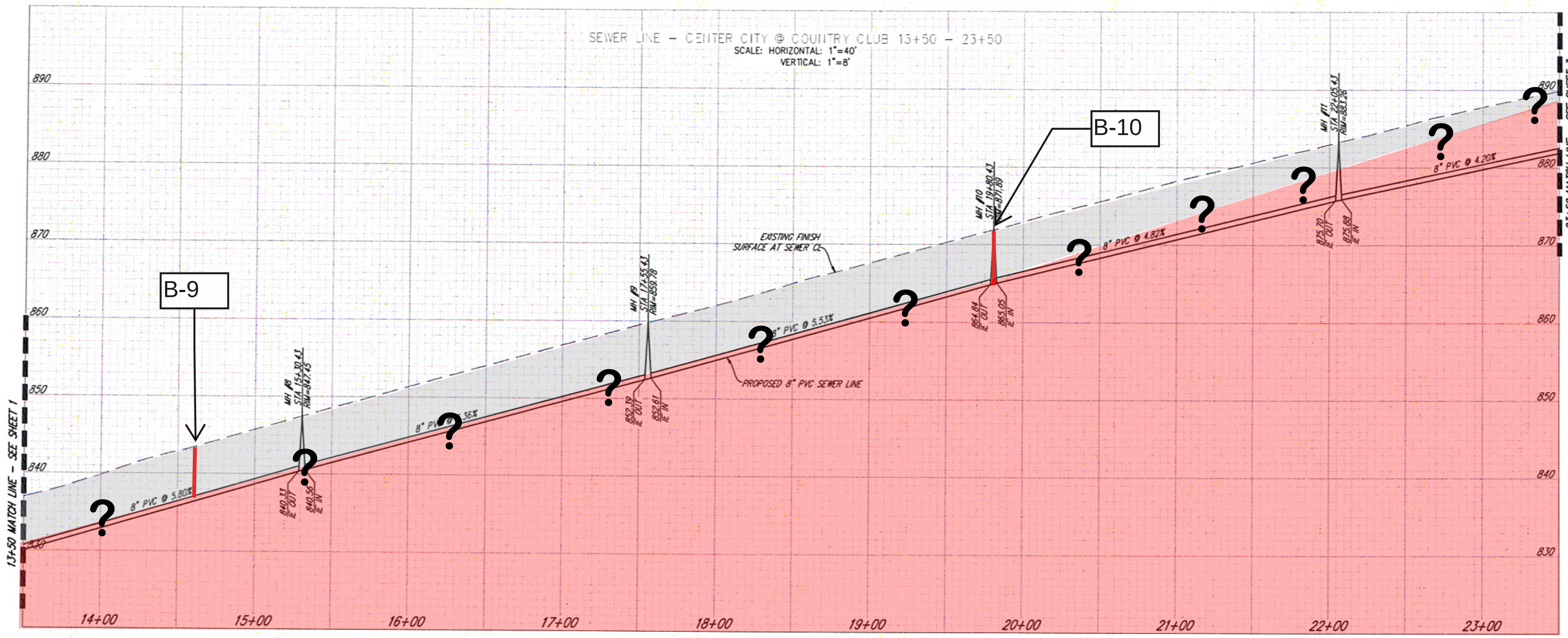
SUMMA ARCHITECTURE
5256 S. Mission Road, Ste 404
Bonsall, CA 92003
760.724.1198

Figure 2


Explanation
Approximate Interpretation of Undocumented Artificial Fill (Afu), Considered Rippable
Approximate Interpretation of Older Alluvium (Qoa), Considered Rippable
Approximate Interpretation of Bedrock (undifferentiated Kmm and Kjd), Considered Marginally Rippable
Approximate Interpretation of Bedrock (undifferentiated Kmm and Kjd), Considered Not Rippable
B-6
Approximate Location of Exploration Boring
B-6
B-7
B-8
?
8" PVC @ 1.00%
8" PVC @ 2.87%
8" PVC @ 5.37%
PROPOSED 8" PVC SEWER LINE
EXISTING FINISH SURFACE AT SEWER CL
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
11+00
12+00
13+00
770
780
790
800
810
820
830
840
13+50 MATCH LINE - SEE SHEET 2
WEST COUNTRY CLUB LANE
NORTH CENTER CITY PARKWAY
INTERCEPT EXIST. SEWER AND INSTALL NEW MH #1 STA 1+00.00 RIM = 775.71
MH #2 STA 1+80.43 RIM = 777.13
MH #3 STA 4+05.43 RIM = 785.12
MH #4 STA 6+30.43 RIM = 796.23
MH #5 STA 8+55.43 RIM = 809.12
MH #6 STA 10+80.43 RIM = 821.76
MH #7 STA 13+05.43 RIM = 834.32
Plate 2
Nutmeg Project
Project No. 3539-SD
June 2018
GEOTEK
SCALE: 1"=40'
0
40
80
120
160
CITY PROJECT NO.
ENG.
DEPARTMENT OF PUBLIC WORKS - ENGINEERING DIVISION
Drawing No.
P17-
Sheet 1 of 3
CONSTRUCTION RECORD
REFERENCES
REVISIONS
BENCH MARK
SCALE
Designed By DD
Drawn By MK
Checked By ML
Plans Prepared Under Supervision Of
ROBERT D. DENTINO
R.C.E. No. 45629
Assitant City Engineer
Director of Engineering Services

Explanation

See Plate 1 of 3



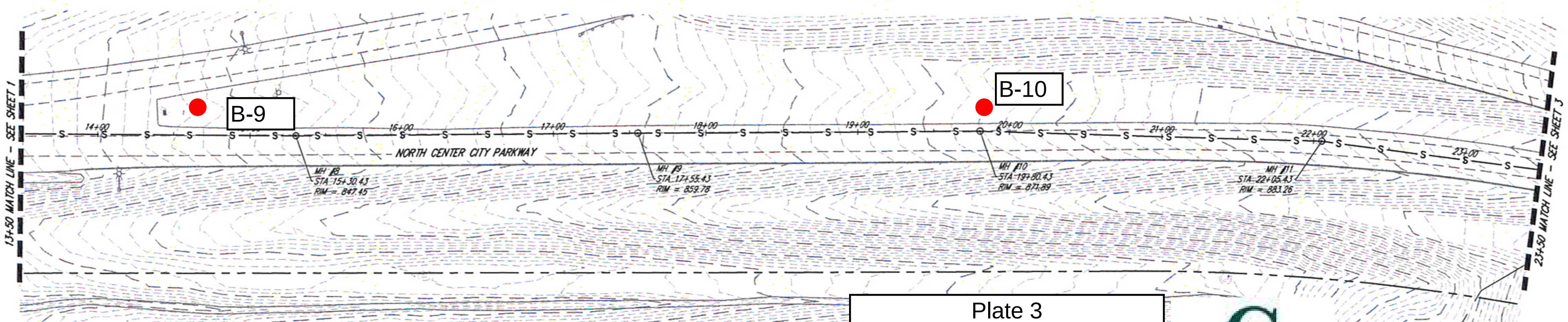


Plate 3
Nutmeg Project
Project No. 3539-SD
June 2018

GEOTEK

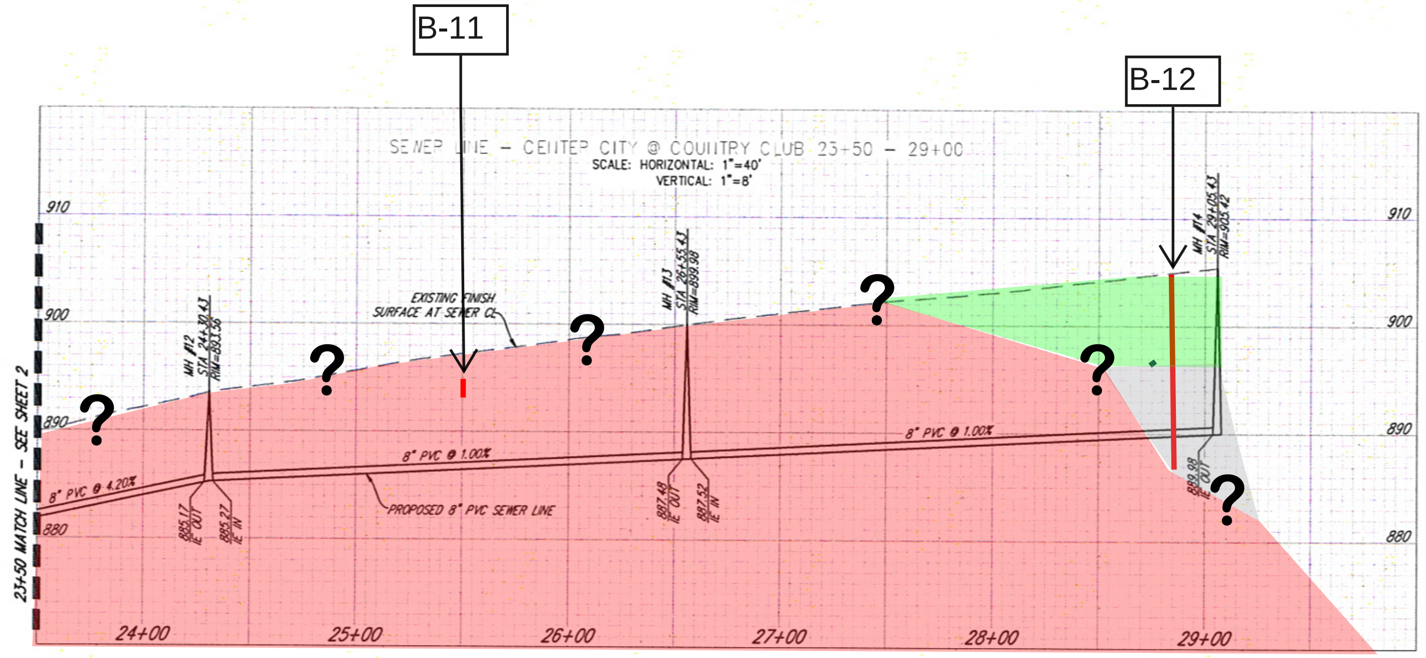
SCALE: 1"=40'
0 40 80 120 160

CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE					
Contractor / Inspector / Date Completed							DESCRIPTION: Elevation	Horizontal NA / Vertical NA	Office / Filmed / Traffic	Designed By DD / Drawn By MK / Checked By ML / Plans Prepared Under Supervision Of ROBERT D. DENTINO / Date / R.C.E. No. 45629	Submitted By Assitant City Engineer	Approved By Director of Engineering Services	DEPARTMENT OF PUBLIC WORKS - ENGINEERING DIVISION

CITY PROJECT NO.
ENG.
Drawing No.
P17-
Sheet 2 of 3

Explanation

See Plate 1 of 3



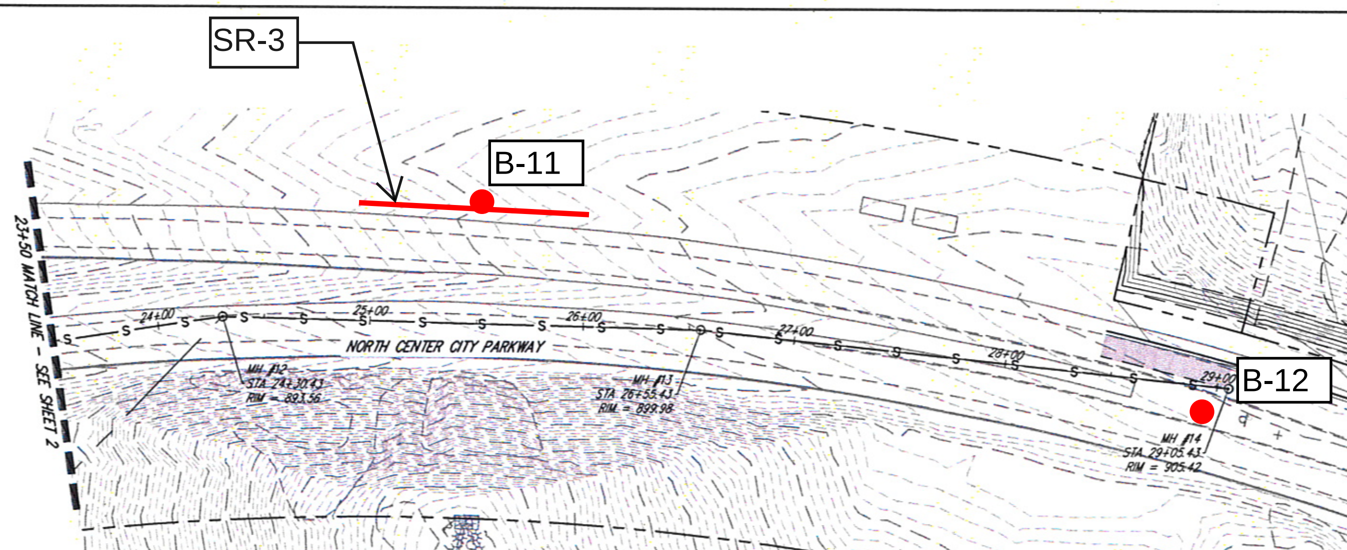


Plate 4
Nutmeg Project
Project No. 3539-SD
June 2018

GEOTEK

SCALE: 1"=40'
0 40 80 120 160

CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE	Office	Designed By DD	Drawn By MK	Checked By ML	Submitted	Approved	DEPARTMENT OF PUBLIC WORKS - ENGINEERING DIVISION	CITY PROJECT NO. ENG.
Contractor / Inspector / Date Completed							DESCRIPTION: Elevation	Horizontal NA / Vertical NA	Filmed / Traffic	Plans Prepared Under Supervision Of ROBERT D. DENTINO Date R.C.E. No. 45629			By Assitant City Engineer	By Director of Engineering Services		Drawing No. P17– Sheet 3 of 3

APPENDIX A

SEISMIC SURVEY SUMMARY REPORT

Nutmeg South, LLC
Escondido, California
Project No. 3539-SD

GEOTEK

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 29, 2018

Attn: Chris Livesey

Re: Seismic Survey Summary Report
Nutmeg Project, Escondido

This report covers the results of a seismic refraction survey performed at the Nutmeg Project Site, in Escondido, 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 May 18, 2018. Three seismic lines were recorded at locations selected by GeoTek. Survey location maps are provided on Figures 1 and 2 that show 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 monzogranitic rock of mid-Cretaceous age, described as massive, medium-to-coarse grain, biotite granitic rock. Surface deposits are mostly 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 on I-15 was a factor during the survey. Recording had to wait for gaps in traffic, primarily large trucks. Each record was stored digitally on an internal hard disk and printed copies of each seismogram were made in the field on thermal paper. Example field records are shown of Figure 3.

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 maps shown on Figures 1 and 2.

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 1 is shown on Figure 4. 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:

$$V_t = 2(V_u \times V_d)/(V_u + V_d)$$

where V_t = true velocity
V_u = apparent up dip velocity
V_d = 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 5-7). 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. Thickness is generally less than 10 feet.

Layer 2 - is interpreted to be weathered bedrock. The velocity range is 3199-5090 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	1250	3199	9700	13 - 25
2	1204	5090	11967	20 - 33
3	1509	4005	9488	4 - 13

Weathering tends to be gradational for most 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 were observed along the ground surface at various locations across the site. Core stones (in a weathered rock matrix) and loose boulders 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.

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 Line Location Map 1

Nutmeg Street -- Escondido, California

Figure 1

Seismic Line Location Map 2

N Centre City Parkway -- Escondido, California

Figure 2

Example Seismic Field Records

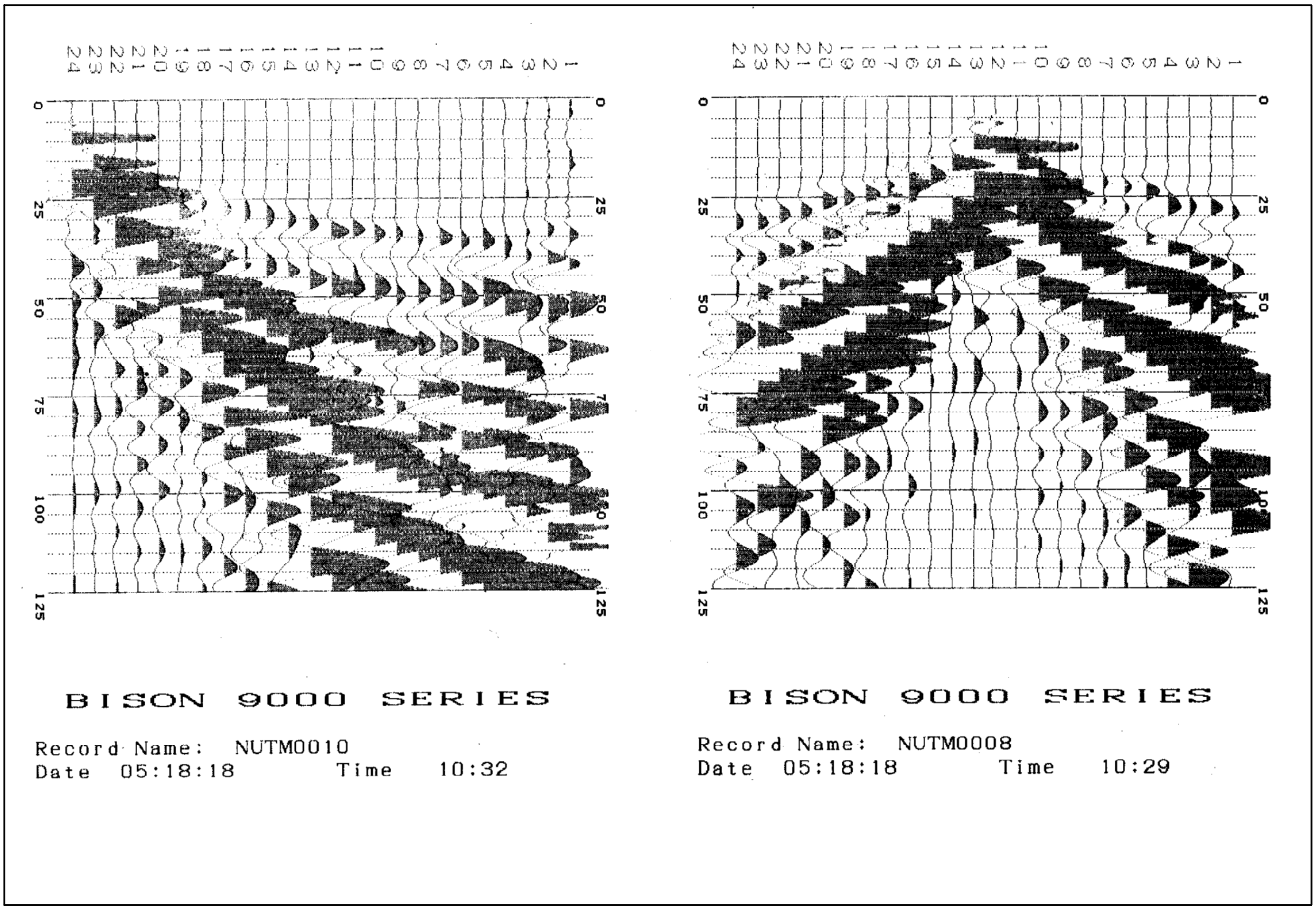
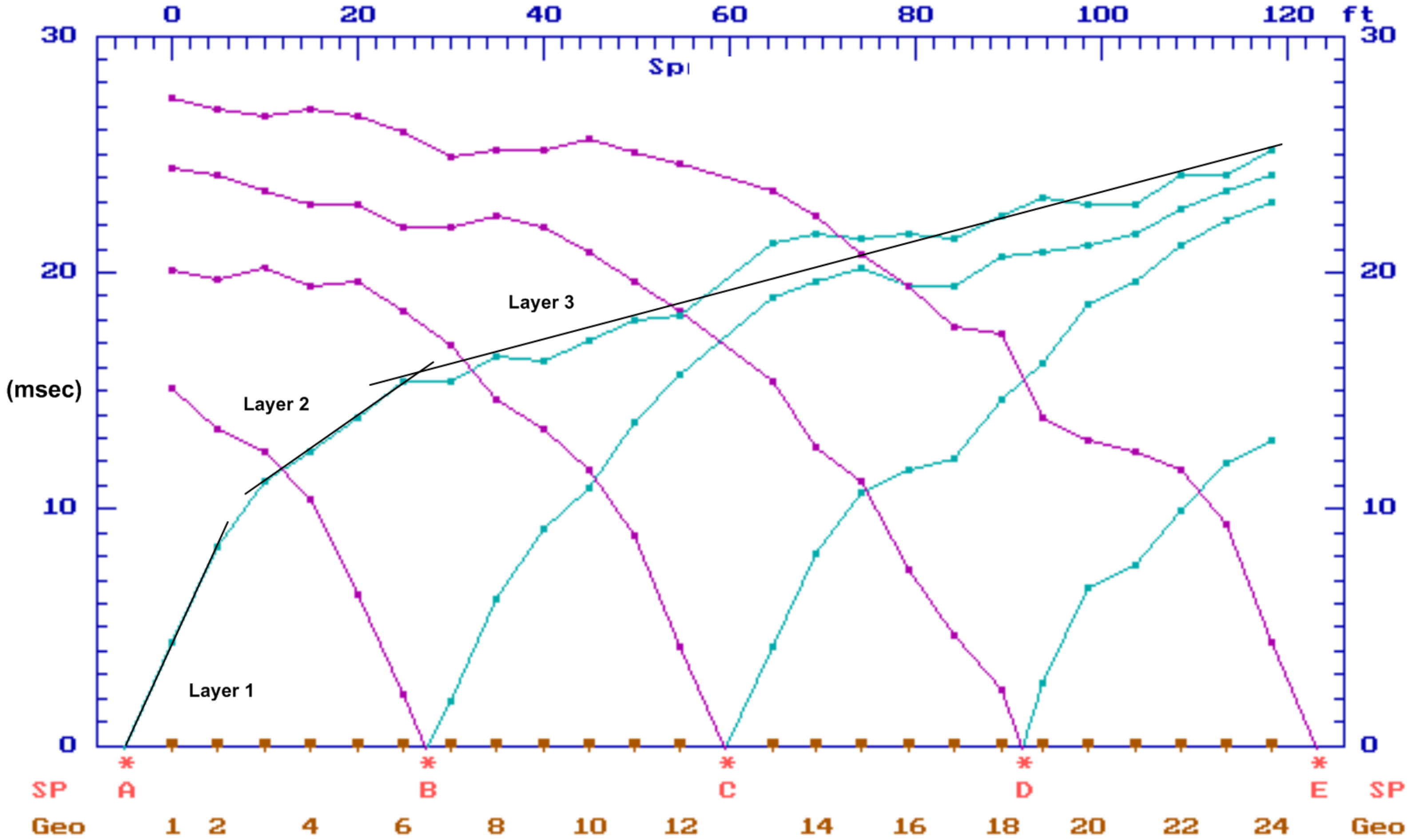


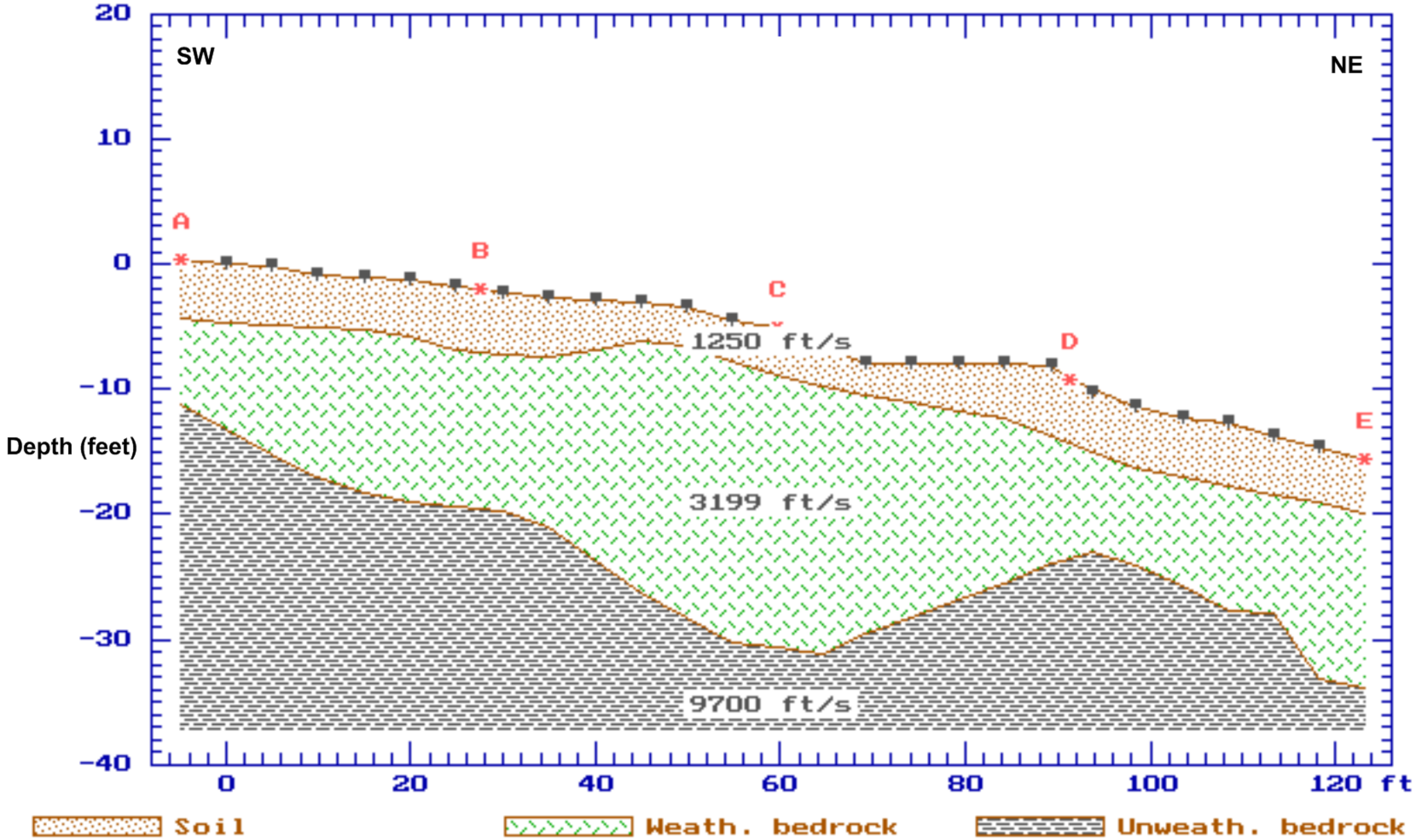
Figure 3

Example Time-Distance Graph -- Line 1

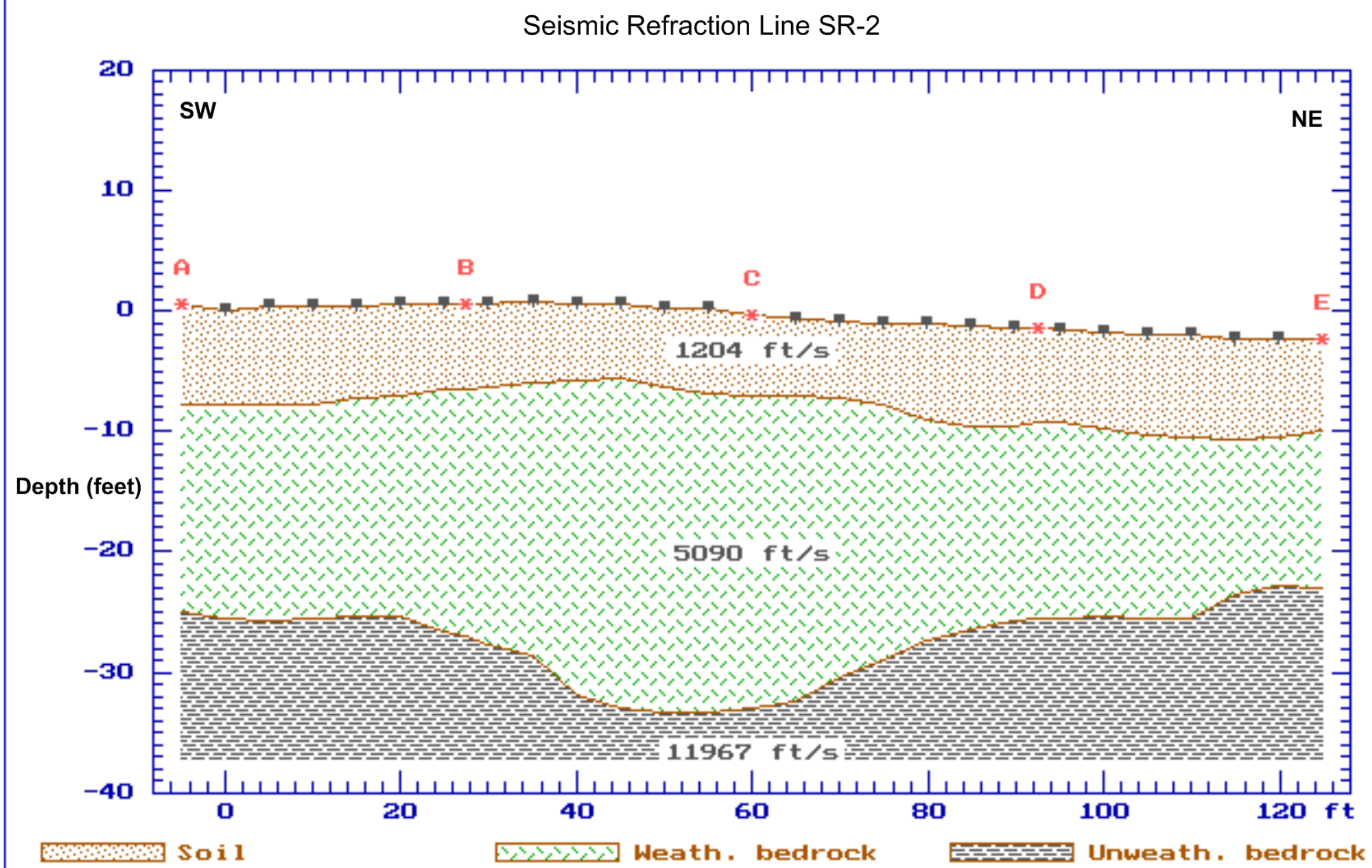


Seismic Refraction Survey 5/18/18	Nutmeg Project
GeoTek Project No. 3539-SD	Escondido, California Figure 4

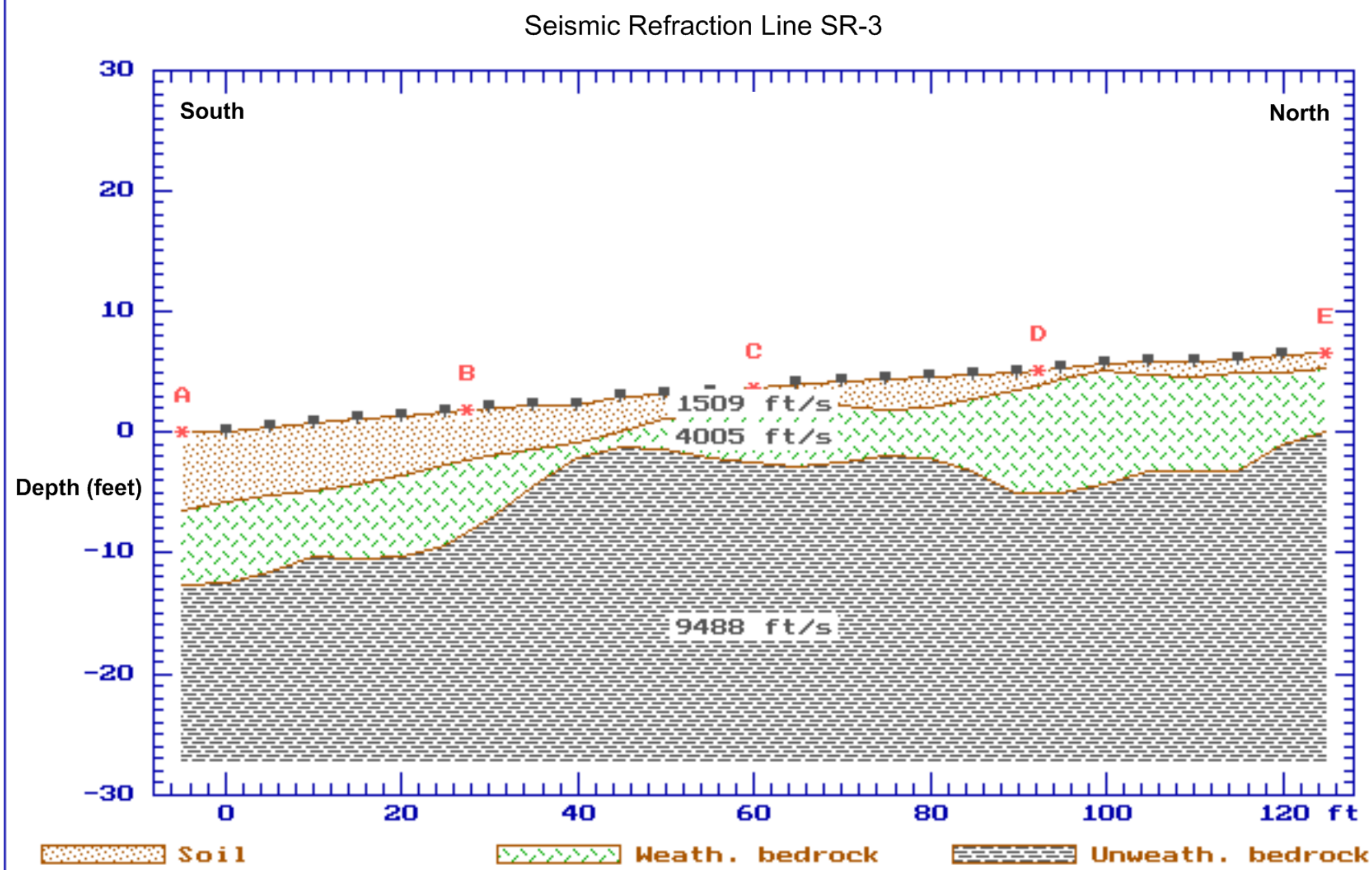
Seismic Refraction Line SR-1



Seismic Refraction Survey 5/18/18	Nutmeg Project
GeoTek Project No. 3539-SD	Escondido, California Figure 5



Seismic Refraction Survey 5/18/18	Nutmeg Project
GeoTek Project No. 3539-SD	Escondido, California Figure 6



Seismic Refraction Survey 5/18/18	Nutmeg Project
GeoTek Project No. 3539-SD	Escondido, California Figure 7

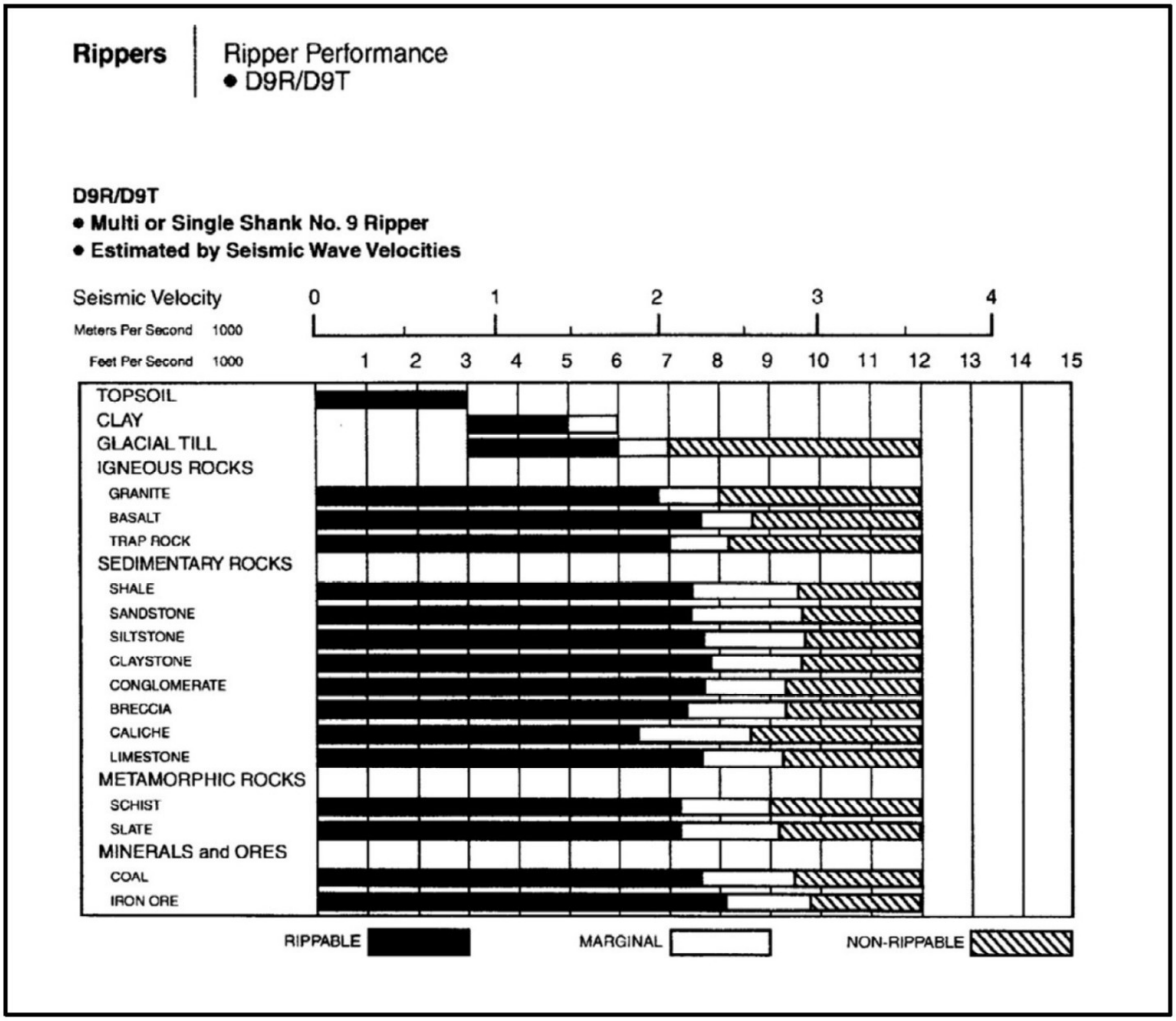

Rippers
Ripper Performance
• D9R/D9T
D9R/D9T
• Multi or Single Shank No. 9 Ripper
• Estimated by Seismic Wave Velocities
Seismic Velocity
Meters Per Second 1000
Feet Per Second 1000
0 1 2 3 4
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
TOPSOIL
CLAY
GLACIAL TILL
IGNEOUS ROCKS
GRANITE
BASALT
TRAP ROCK
SEDIMENTARY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

Figure 8

APPENDIX B

LOGS OF EXPLORATORY BORINGS
AND INFILTRATION WORKSHEETS

Nutmeg South, LLC
Escondido, California
Project No. 3539-SD

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.

Bulk Samples (Small)
These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING LOG LEGEND

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

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 the boring

(Additional denotations and symbols are provided on the log of Exploratory Borings)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	N/A
PROJECT NO.:	3539-SD	**HAMMER:**	Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895 ft msl	**DATE:**	4/19/2018

BORING NO.: *B-1*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Undocumented Artificial Fill (Afu)			
			BB-1	SM	Silty SAND, Light Brown, Damp, Fine SAND			SH, MD EI
				GP	Gravel/Cobble			
			R-1	SM	Silty SAND, Dark Brown, Moist, Fine SAND			
					No Recovery move 5' Northeast, Resample, No Recovery			
5	Ring	10 9 12 12 15 21		SP	No Recovery (very loose SAND)			
					Quaternary Older Alluvium (Qoa)			
10	SPT	14 25 43	S-1	SM	Silty SAND, Very Dense, Light Brown, Damp, Fine SAND Well Cemented			
					Cretaceous Granitic Material (kmm)			
					TD 13" Refusal of Auger Advancement No Groundwater Encountered During Drilling Backfilled with Soil Cuttings			
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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	
PROJECT NO.:	3539-SD	**HAMMER:**	Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895 ft msl	**DATE:**	4/19/2018

BORING NO.: *B-2*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
0	Large Bulk		BB-1	ML	**Undocumented Artificial Fill (AFU)** Sandy SILT, Medium Stiff, Light Brown, Damp, Fine SAND			SH, MD EI
5	SPT	6 6 8	S-1	SM	**Quaternary Young Alluvium (Qal)** Small Sample Recovered Sluff of upper sample remained in sampler, Light Brown Silty SAND, Crushed, Manufactured GRAVEL, Cuttings are Dark Brown, Silty SAND			
10	Ring	11 21 50/54	R-1	SM	**Quaternary Older Alluvium (Qoa)** Silty SAND, Very Dense, Mottled Brown & Reddish Brown, Moist, Cemented	13.1	122.3	
15	SPT	35 50/44	S-2	N/A	**Cretaceous Granitic Material (kmm)** Auger is suspected to be on ROCK, Drill to 15.5', Sample No Recovery			
	SPT	42 54/3	S-3		Decomposed Granite (Well Graded SAND), Very Dense, Light Brown, Damp, Fine to Course, Light Weathering Minerals, Generally Unaltered		CF	
20					TD 18.5' Refusal of Auger Advancement No Groundwater Encountered During Drilling Backfilled with Soil Cuttings			
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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	N/A
PROJECT NO.:	3539-SD	**HAMMER:**	Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895 ft msl	**DATE:**	4/19/2018

BORING NO.: *B-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
					Quaternary Alluvium (Qal)			
				SM	Silty SAND, Loose, Brown, Moist, Fine SAND			
5	SPT	8 10 11	S-1		Silty SAND, Medium Dense, Brown, Moist, Fine SAND, 31% Fine SAND			SA
					Cretaceous Granitic Material (kmm)			
10	Ring	37 30/40	R-1	N/A	Decomposed GRANITE, (Well Graded SAND) Very Dense, Orange Brown, Angular Grains, Light Weathering, Minerals Unaltered	4.6	125.9	SH
					TD 11' Refusal of Angar Advancement No Groundwater Encountered during Drilling Backfilled with Soil Cuttings			
15	SPT							
20	Ring							
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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	N/A
PROJECT NO.:	3539-SD	**HAMMER:**	Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895 ft msl	**DATE:**	4/19/2018

BORING NO.: *B-4*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Quaternary Young Alluvium (Qal)			
			BB-1	SP-SM	Poorly Graded SAND with SILT, Loose, Light Brown, Damp, Fine SAND			
5								
	Ring	25 50/44	R-1	SM	**Quaternary Older Alluvium (Qoa)** Silty SAND, Very Dense, Mottled Pale White and Orange Brown, Fine SAND, WELL CEMENTED	4.6	125.7	
					TD 8' Refusal of Auger Advancement (on Cretaceous Granitic Material, kmm) No Groundwater Encountered during Drilling 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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	
PROJECT NO.:	3539-SD	**HAMMER:**	Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895 ft msl	**DATE:**	4/19/2018

BORING NO.: *B-5*

Depth (ft)	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
				Quaternary Young Alluvium (Qal)			
		BB-1	SM	Silty SAND, Loose, Light Brown, Moist, Fine to Medium SAND, Rootlets, Porous			EI
5	4 4 3	R-1		Dark Brown, Porous	13.3	110.6	CO
				Perched Water			
10	11 18 21	S-1	SM	**Quaternary Older Alluvium (Qoa)** Silty SAND, Dense, Brown, Moist, Fine to Medium SAND			
15	16 26 49	R-2		Silty SAND Very Dense, Mottled Light & Red Brown with Greenish Gray, Moist, Fine to Medium SAND, CEMENTED			CO
20	15 24 41	S-2		Silty SAND, Very Dense, Mottled Light & Medium Brown, Very Moist, ~ 25% Fine SAND			SA
				TD 21.5' Perched Groundwater @ 8' Backfilled with Soil Cuttings			
25							
30							

Laboratory Testing: Water Content (%), Dry Density (pcf), Others

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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	780' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-6*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
0–5	Large Bulk		SM		**Undocumented Artificial Fill (Afu)** Asphalt-Concrete, Serverly Degraded over Silty SAND Gravel, Dense, Brown, Moist, Fine to Course SAND, Fine SAND, Small Boulders/Cobbles			MD RV
5	SPT	12 5 4	S-1	SP-SM	Poorly Graded SAND with Silt, Loose, Brown, Moist, Medium to Course SAND, Easy Drilling			
10	SPT	9 36 50/3"	R-1		**Cretaceous Granite Material (kmm)** Decomposed GRANITE, (Clayey SAND) Very Dense, Orange Brown, Moist, Clay Development, Coarse Angular Grains, Drilling becomes more difficult	9.2	125.7	
15	SPT	50/2"	S-2		Decomposed Granite, (Poorly Graded SAND with Silt) Very Dense, Light Brown, Moist, Medium to Coarse SAND, Lightly Weathered			
20	SPT	50/3"	S-3		GRANITE, (Poorly Graded SAND) Very Dense, Light Gray, Damp, Fine to Medium SAND, No Mineral Alteration, Fresh Weathering			
					TD 20' Refusal of Auger Advancement No Groundwater Encountered during Drilling Backfilled with Bentonite Grout			
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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	6" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	798' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-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
					Undocumented Artificial Fill (Afu)			
				SM	Silty SAND, Dense, Light Brown, Damp, Fine to Medium SAND, Fine GRAVEL, Some Small Boulders			
5	SPT	8 10 12	S-1		Silty SAND, Medium Dense, Brown, Moist, Fine SAND, Some Angular Fine GRAVEL			
10	Ring	10 12 13	R-1		Silty SAND, Medium Dense, Reddish Brown, Moist, Fine SAND, Fine Angular GRAVEL	8.2	112.7	
					Quaternary Older Alluvium (Qoa)			
15	SPT	18 29 50/5	S-2	SM	Silty SAND, Very Dense, Reddish Brown, Damp, Fine SAND			
20	Ring	50/5"	R-2		**Decomposed Granite (kmm)** Decomposed Granite, (Clayey SAND) Very Dense, Reddish Brown, Moist, Severly Weathered, Minerals Altered to CLAY, Course Anglular SAND, Transition in Upper Ring			
					TD 20.5' Refusal of Auger Advancement No Groundwater Encountered during Drilling Backfilled with Bentonite Grout			
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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	820' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-8*

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 (Afu)			
	Large Bulk				Silty SAND with Gravel, Very Dense, Reddish Brown, Damp, Fine SAND			
					Cretaceous Granitic Material (kmm)			
5	SPT	18 50/2	S-1		Decomposed GRANITE, (Clayey GRAVEL) Reddish Brown, Very Dense, Very Hard Drilling			RV MD
					Slow/Auger Advancement			
10	SPT	50/0	S-2		No Recovery, Crystalized ROCK, Non-Weathered			
					TD 10' Refusal of Auger Advancement No Groundwater Encountered during Drilling Backfilled with Bentonite Grout			
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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	843 + MSL	**DATE:**	5/18/2018

BORING NO.: *B-9*

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
					Cretaceous Granitic Material (kjd) (Porrly Graded SAND) Granite, Very Dense, Light Gray, Damp, Fine to Medium SAND			
5	Ring	50/3	S-1		GRANITE, (Poorly Graded SAND) Very Dense, Light Gray, Moist, Fine to Medium SAND, Non-Weathered Very Dense Auger Refusal on Crystalized ROCK, Rig Chatter			
					TD 6.5' Refusal of Auger Advancement No Groundwater Encountered During Drilling Backfilled with Bentonite Grout			
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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	872' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-10*

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
					Cretaceou Granitic Material (kjd)			
					GRANITE, Very Dense, Light Gray, Damp, Fine to Medium SAND, Hard Slow Drilling			
5	SPT	50/5"	S-1		GRANITE, (Poorly Graded SAND) Very Dense, Light Gray, Damp, Fine to Medium SAND, Low Recovery, Fresh GRANITE			
					Jumping Auger, Possiable Top of Blasting Surface, Angular Cobbes in Cuttings Refusal			
					TD 7' Refulsal of Auger Advancement No Groundwater Encountered during Drilling 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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	895' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-11*

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
					Undocumented Artificial Fill (Afu)			
				SM	Silty SAND, Dense, Light Brown, Moist, Fine SAND, Cobbles			
					TD 2' Refusal of Auger Advancement (on Granitic Material (kjd)) Auger Chatter and Jumping Teeth Chipped Off No Groundwater Encountered During Drilling 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:	Nutmeg South, LLC	**DRILLER:**	BAJA Exploration	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	8" Hollow-Stem Auger	**OPERATOR:**	Randy
PROJECT NO.:	3539-SD	**HAMMER:**	140 lb Automatic Hammer	**RIG TYPE:**	CME-75
LOCATION:	Escondido, CA	**ELEVATION:**	905' + MSL	**DATE:**	5/18/2018

BORING NO.: *B-12*

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
					Undocumented Artifical Fill (Afu)			
	Large Bulk			SM	ASPHALT CONCRETE 3" Over Gravelly Silty SAND, Medium Dense, Light Brown to Gray, Moist, Fine to Course SAND, Fine GRAVEL ROCK			RV
5	SPT	5 7 8	S-1	SC	Clayey SAND, Medium Dense, Reddish Brown, Moist, Coarse SAND, Fill Mottling Color Changing in Cuttings to More Reddish			
					Cretaceous Granitic Material (kmm)			
10	Ring	18 50/5"	R-1		Decomposed GRANITE (Silty SAND), Very Dense, Light Orange Brown, Damp, Fine SAND Shore: Poorly Graded SAND, Light Brown, Damp, Medium to Coarse SAND Drilling Encounters Denser Material	6.8	120.4	
15	SPT	50/5"	S-2		Poorly Graded SAND, Very Dense, Light Orange Brown, Damp, Fine to Course SAND, No Mineral Alteration			
					TD 18' Refusal of Auger Advancement No Groundwater Encountered Backfilled with Bentonite Grout			
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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drilling	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	Hand Auger	**OPERATOR:**	N/A
PROJECT NO.:	3539-SD	**HAMMER:**	N/A	**RIG TYPE:**	Hand Auger
LOCATION:	Escondido, CA	**ELEVATION:**	890' + MSL	**DATE:**	4/19/2018

BORING NO.: *I-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
					Poorly Graded SAND, Loose, Light Orange Brown, Dry Course SAND			
					TD: 57.6 inches Converted to Percolation Test Boring 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:	Nutmeg South, LLC	**DRILLER:**	Pacific Drillign	**LOGGED BY:**	CDL
PROJECT NAME:	Nutmeg	**DRILL METHOD:**	Hang Auger	**OPERATOR:**	N/A
PROJECT NO.:	3539-SD	**HAMMER:**	N/A	**RIG TYPE:**	Hand Auger
LOCATION:	Escondido, CA	**ELEVATION:**	885' + MSL	**DATE:**	4/19/2018

BORING NO.: *I-2*

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
					Silty SAND, Tan Brown, Damp, Fine SAND			
5					TD: 58.8 inches Pre-Soak Boring Converted to Percolation Test Boring 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

APPENDIX C

LABORATORY TESTING RESULTS

Nutmeg South, LLC
Escondido, California
Project No. 3539-SD

GEOTEK

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Gradation Analysis

Date:	5/2/2018		
W.O.:	3539-SD	sample ID	B-3
Client:	Nutmeg South	depth	5
Technician:	DI		

Sieve Size	Particle Diameter		Wt. Retained	Wt. Passing	% Passing	Specs
	in.	mm.				
3	3.00	76.2		150	100.0%	
2	2.00	50.8		150	100.0%	
1 1/2	1.50	37.5		150	100.0%	
1	1.00	25.4		150	100.0%	
3/4	0.742	18.85		150	100.0%	
1/2	0.500	12.7		150	100.0%	
3/8	0.371	9.423		150	100.0%	
1/4	0.250	6.350		150	100.0%	
#4	0.185	4.699		150	100.0%	
#8	0.093	2.362		150	100.0%	
#10	0.0787	2.000		150	100.0%	
#16	0.0460	1.168		150	100.0%	
#20	0.0331	0.840		150	100.0%	
#30	0.0232	0.589		150	100.0%	
#40	0.0165	0.420		150	100.0%	
#50	0.0116	0.295		150	100.0%	
#60	0.0085	0.265		150	100.0%	
#100	0.0058	0.147		150	100.0%	
#200	0.0029	0.074	103	47	31.3%	
#270	0.0021	0.053		47	31.3%	
Pan				47	31.3%	
Total						

Dry Weight 150

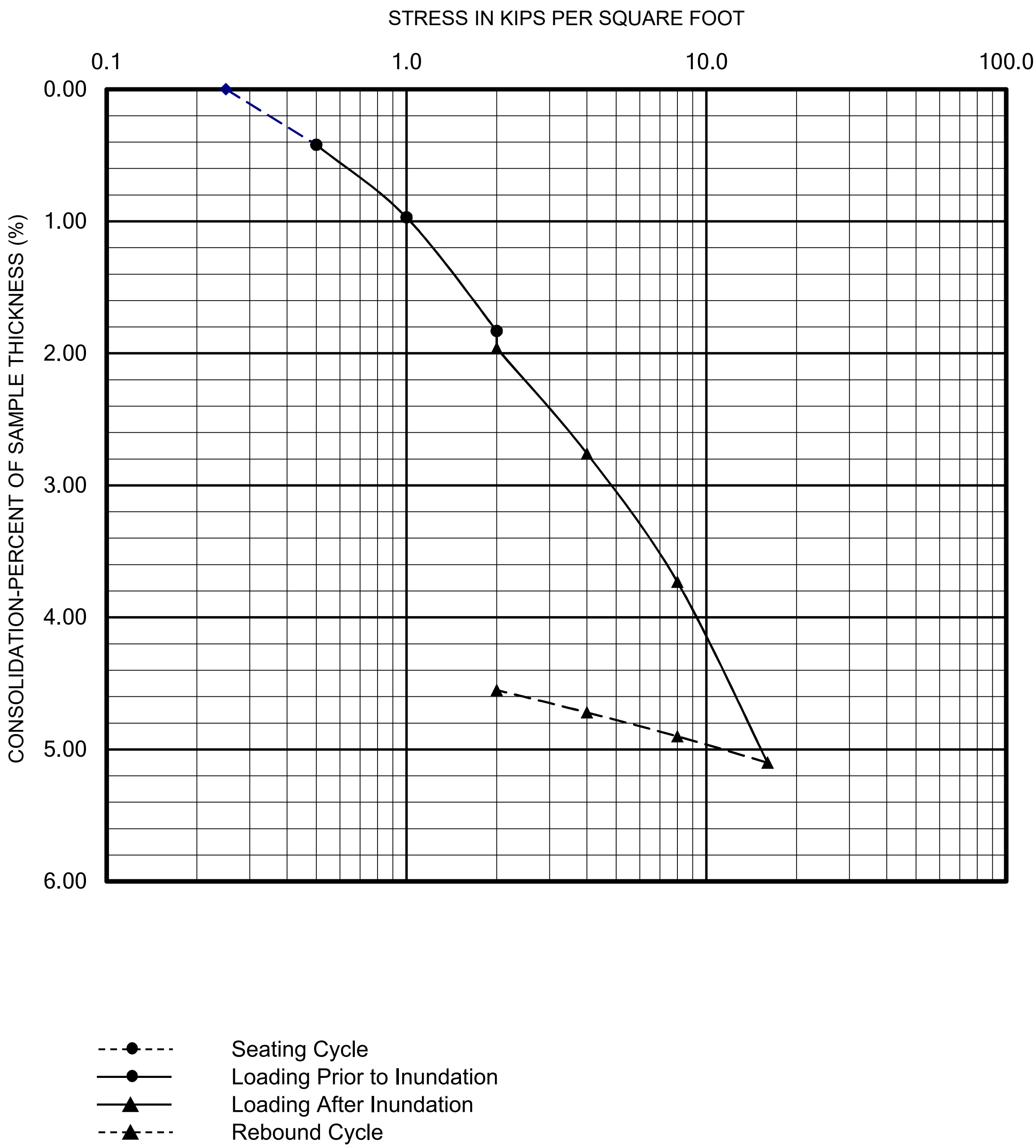
GEOTEK

Gradation Analysis

Date:	5/2/2018		
W.O.:	3539-SD	sample ID	B-5
Client:	Nutmeg South	depth	20
Technician:	DI		

Sieve Size	Particle Diameter		Wt. Retained	Wt. Passing	% Passing	Specs
	in.	mm.				
3	3.00	76.2		150	100.0%	
2	2.00	50.8		150	100.0%	
1 1/2	1.50	37.5		150	100.0%	
1	1.00	25.4		150	100.0%	
3/4	0.742	18.85		150	100.0%	
1/2	0.500	12.7		150	100.0%	
3/8	0.371	9.423		150	100.0%	
1/4	0.250	6.350		150	100.0%	
#4	0.185	4.699		150	100.0%	
#8	0.093	2.362		150	100.0%	
#10	0.0787	2.000		150	100.0%	
#16	0.0460	1.168		150	100.0%	
#20	0.0331	0.840		150	100.0%	
#30	0.0232	0.589		150	100.0%	
#40	0.0165	0.420		150	100.0%	
#50	0.0116	0.295		150	100.0%	
#60	0.0085	0.265		150	100.0%	
#100	0.0058	0.147		150	100.0%	
#200	0.0029	0.074	113.3	36.7	24.5%	
#270	0.0021	0.053		36.7	24.5%	
Pan				36.7	24.5%	
Total						

Dry Weight 150



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

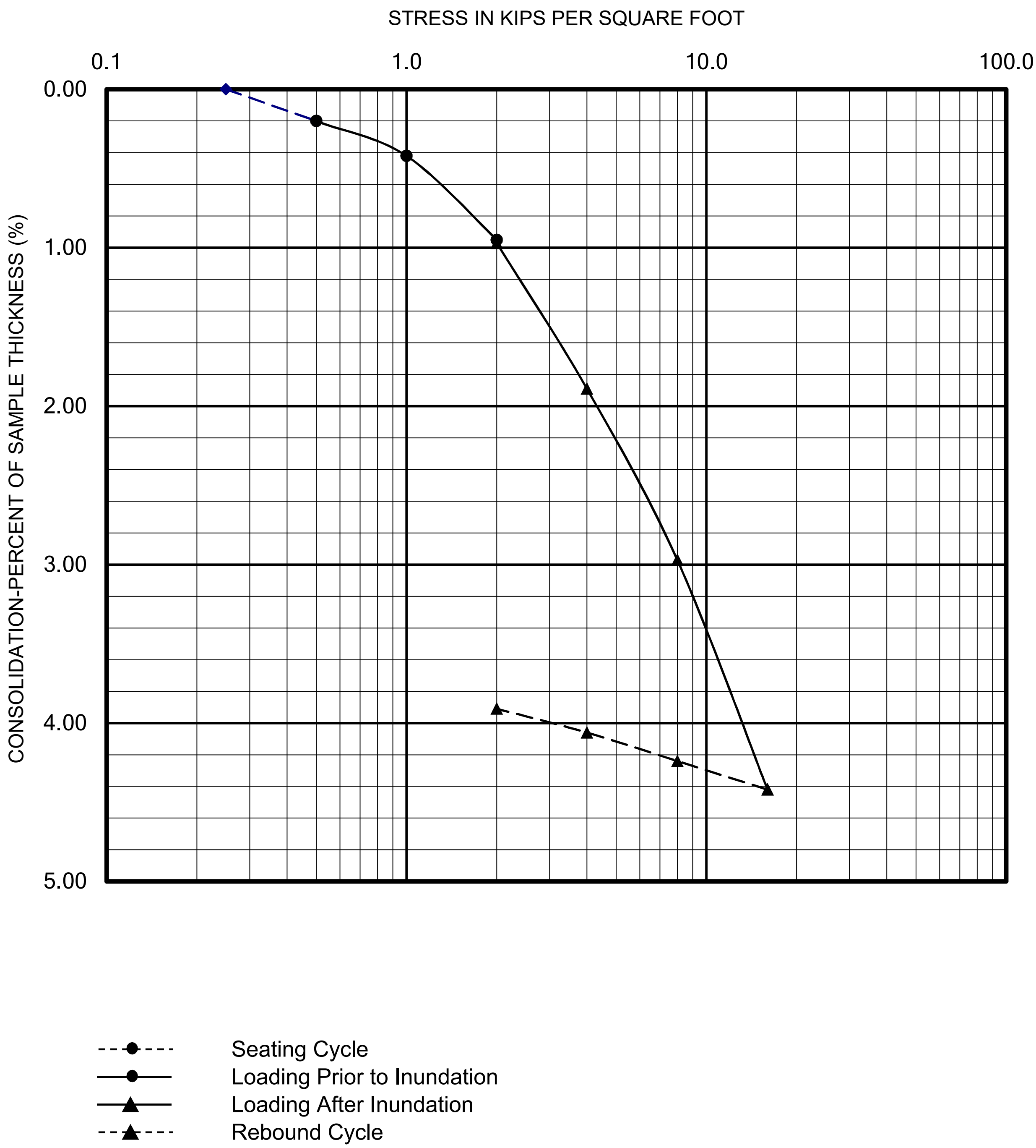
GEOTEK

CHECKED BY:	Lab: DI	
PROJECT NO.: 3539-SD	Date: 4/27	

CONSOLIDATION REPORT

Sample:
B-5 @ 5

Plate C-1



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

GEOTEK

CHECKED BY:	Lab: DI
PROJECT NO.: 3539-SD	Date: 4/27

CONSOLIDATION REPORT

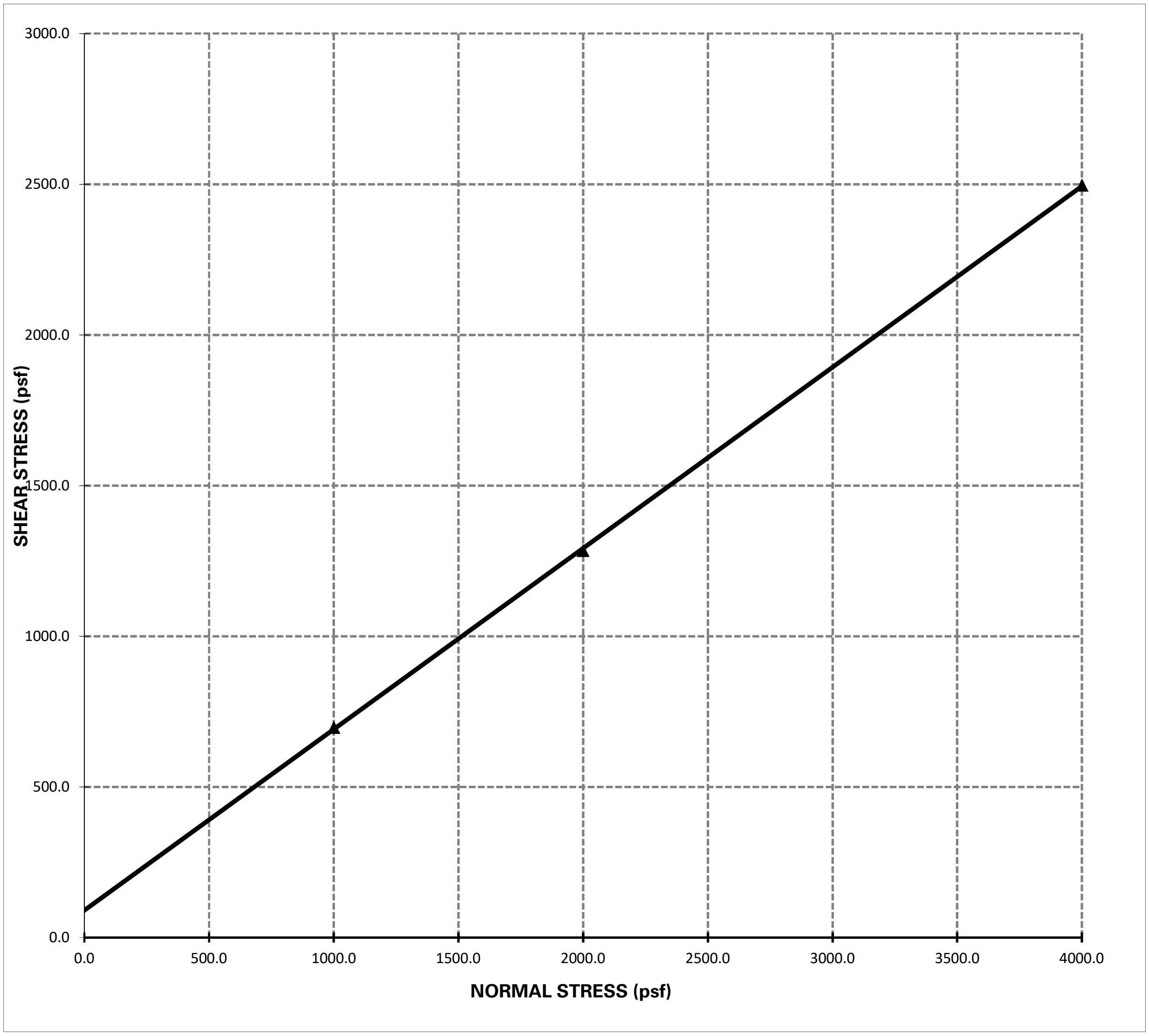
Sample:
B-5 @ 15

Plate C-1

DIRECT SHEAR TEST

Project Name:	Nutmeg South	***Sample Location:***	B-1 @ 0 - 5
Project Number:	3539-SD	***Date Tested:***	5/7/2018

PEAK VALUE



Shear Strength: **Φ = 31.0 °** ; **C = 90.00 psf**

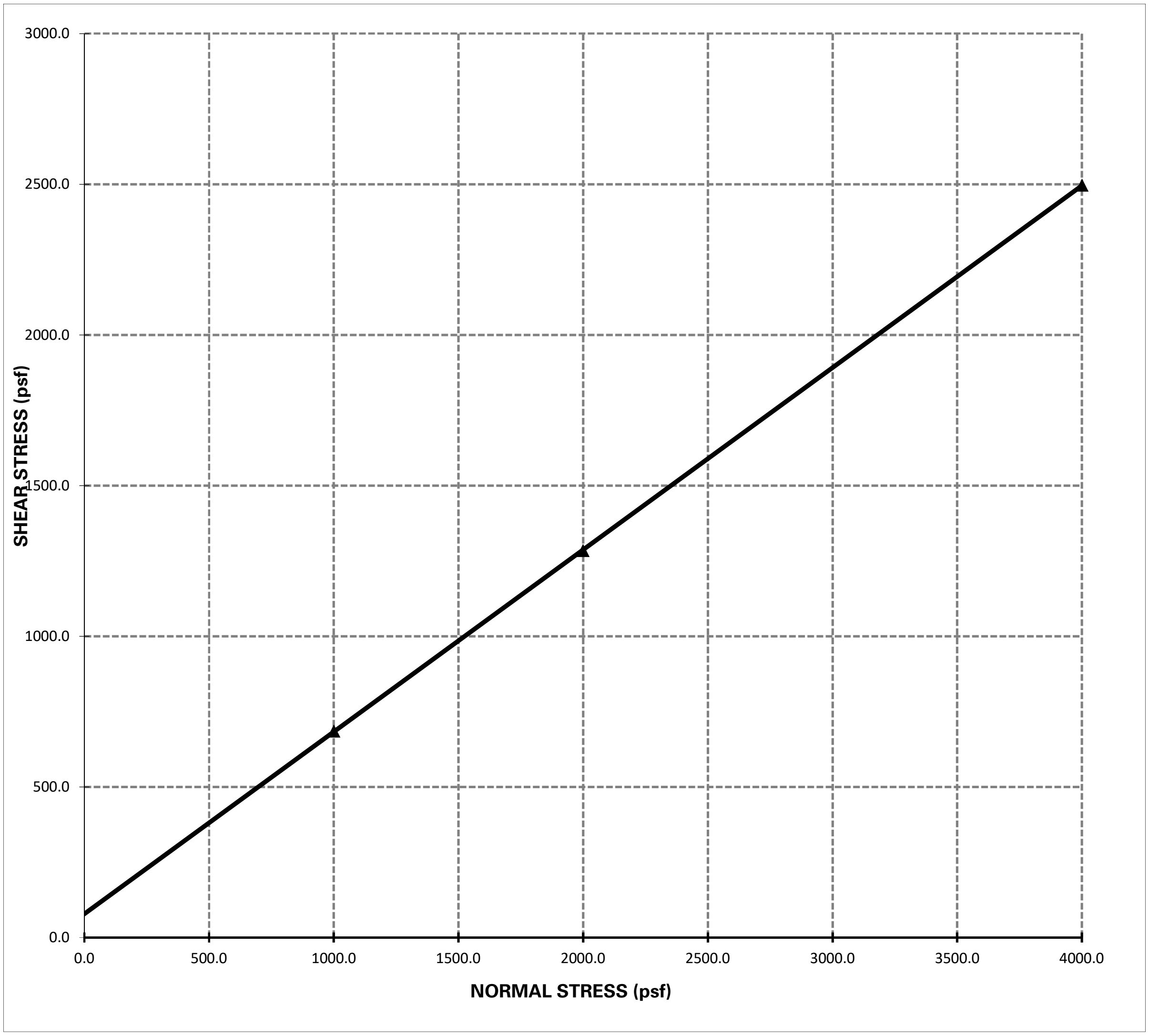
Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name:	Nutmeg South	***Sample Location:***	B-1 @ 0 - 5
Project Number:	3539-SD	***Date Tested:***	5/7/2018



Shear Strength: Φ = **31.1 °** ; **C = 78.00 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

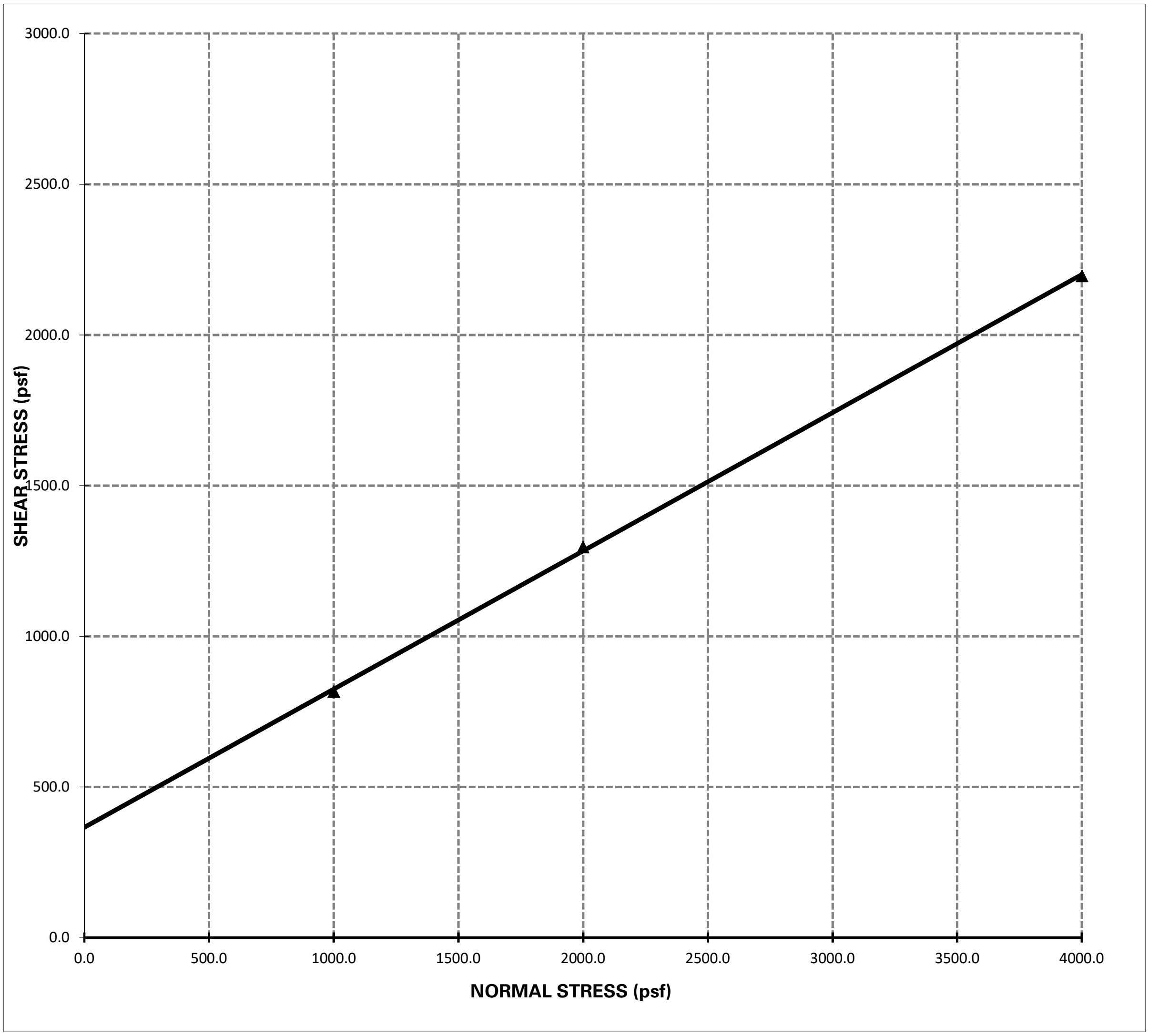
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name:	Nutmeg South	***Sample Location:***	B-2 @ 0 - 5
Project Number:	3539-SD	***Date Tested:***	5/7/2018

PEAK VALUE



Shear Strength: Φ = **24.6 °** **C = 366.00 psf**

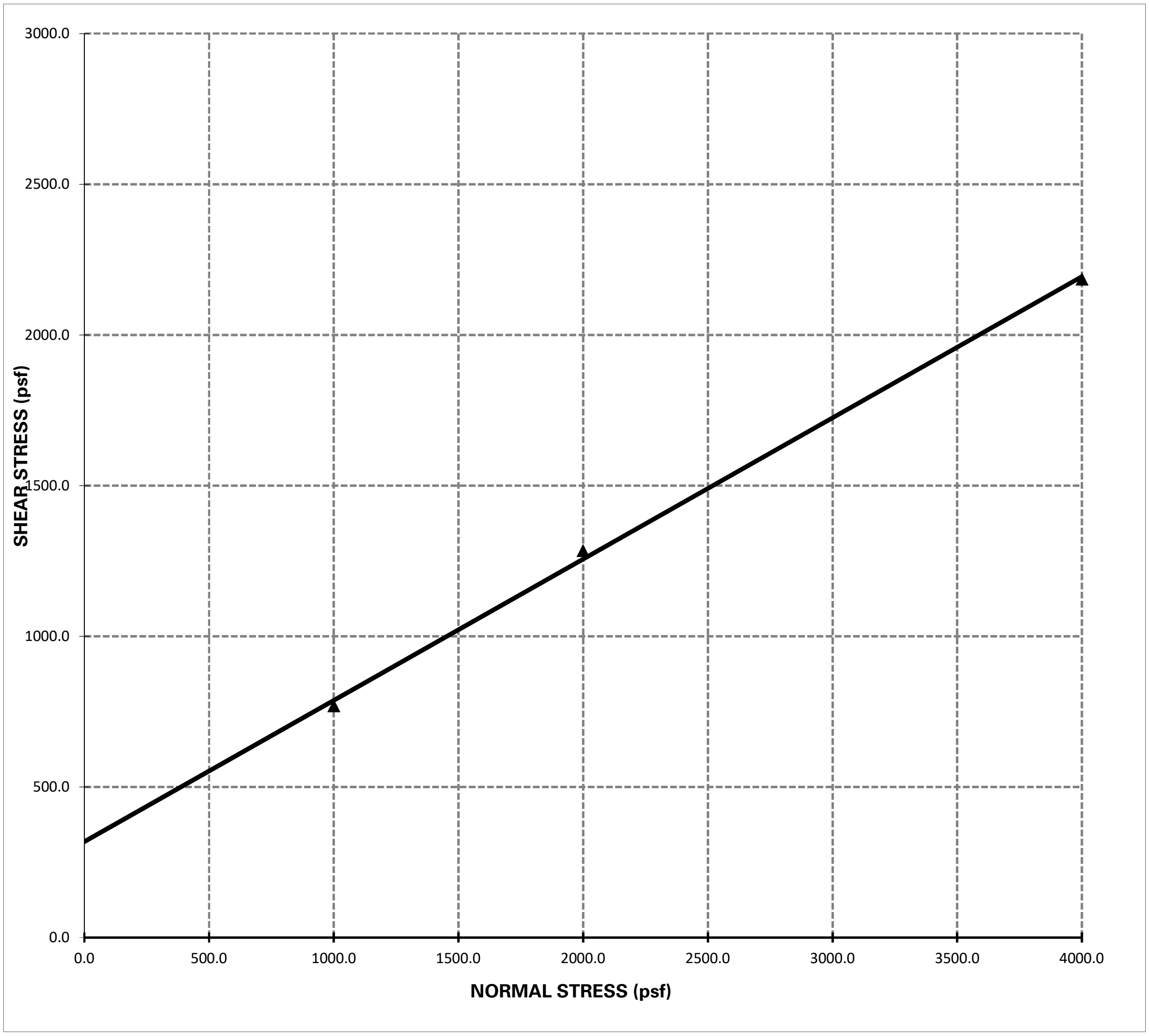
Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name:	Nutmeg South	***Sample Location:***	B-2 @ 0 - 5
Project Number:	3539-SD	***Date Tested:***	5/7/2018



Shear Strength: Φ = **25.1 °** **C = 318.00 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

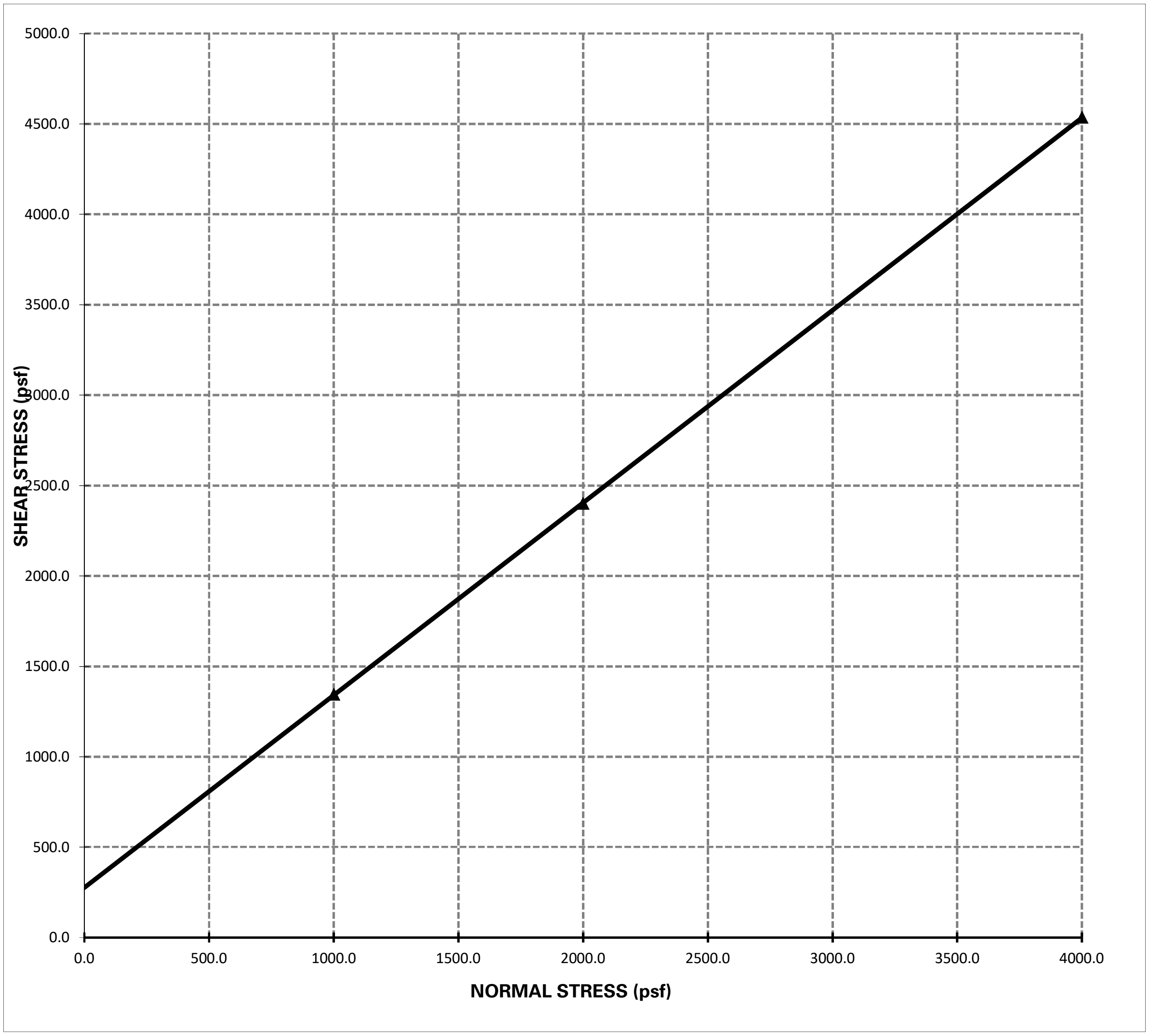
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

Project Name:	Nutmeg	***Sample Location:***	B-3 @ 10
Project Number:	3539-SD	***Date Tested:***	5/2/2018

PEAK VALUE



Shear Strength: $\Phi =$ **46.8 °** **C = 276.00 psf**

Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.

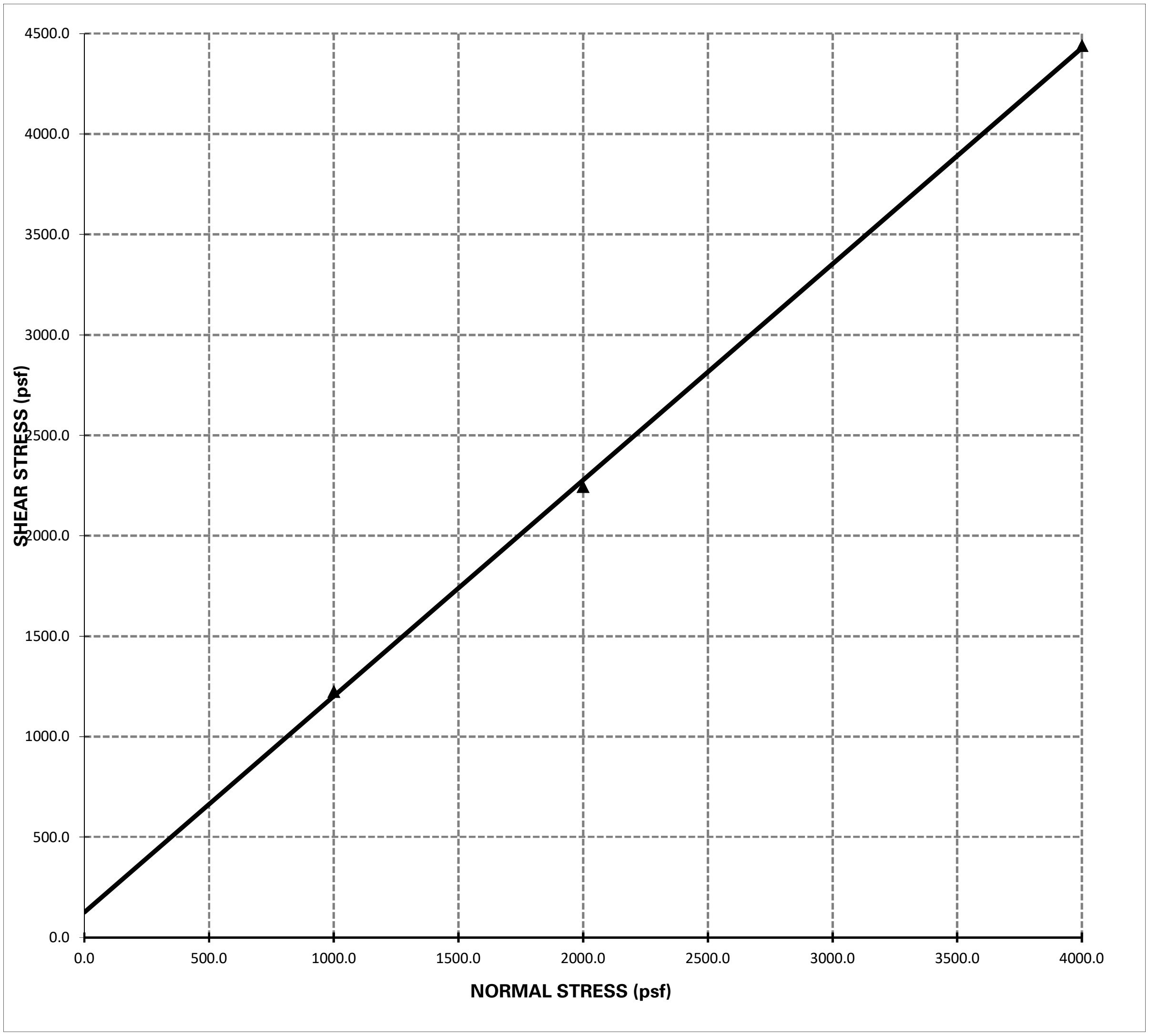
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.035 in/min.

DIRECT SHEAR TEST

GEOTEK

Project Name:	Nutmeg	***Sample Location:***	B-3 @ 10
Project Number:	3539-SD	***Date Tested:***	5/2/2018



Shear Strength: **Φ = 47.1 °** , **C = 126.00 psf**

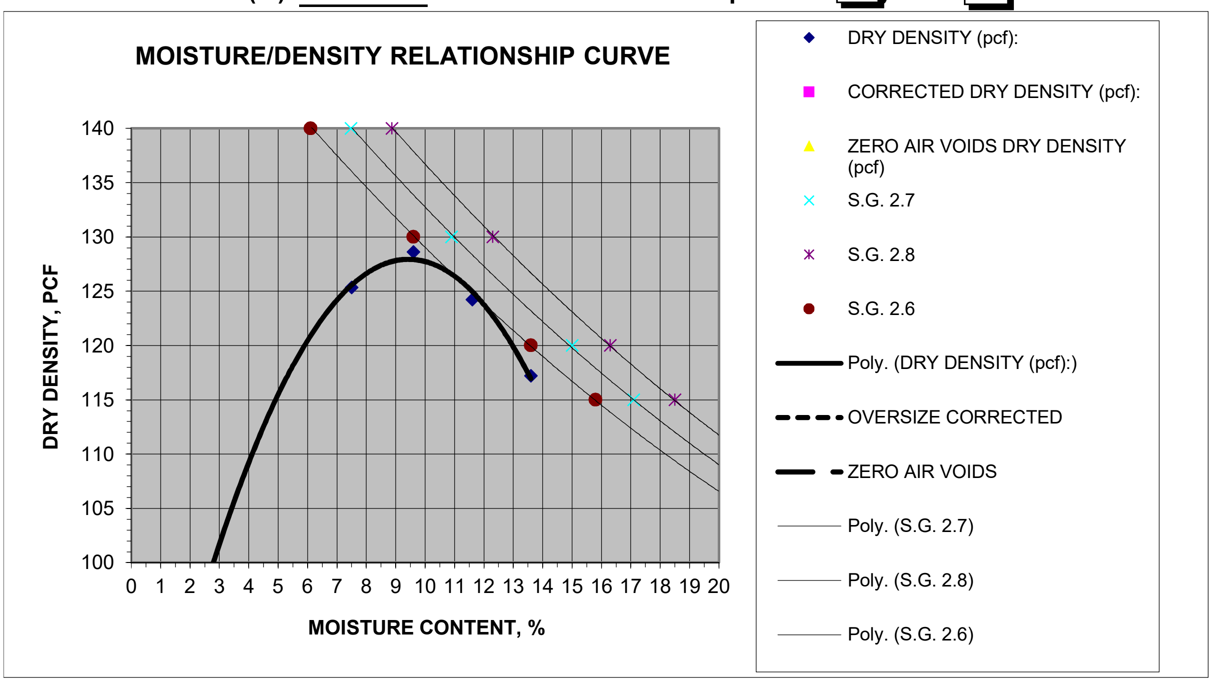
Notes: 1 - The soil specimens sheared were "undisturbed" ring samples.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.035 in/min.

MOISTURE/DENSITY RELATIONSHIP

Client:	Nutmeg South	**Job No.:**	3539-SD
Project:	Nutmeg	**Lab No.:**	Corona
Location:	0		
Material Type:	Brown Clayey F - M Sand		
Material Supplier:			
Material Source:			
Sample Location:	B-1 @ 0 - 5		
Sampled By:	CL	**Date Sampled:**	20-Apr-18
Received By:	DA	**Date Received:**	27-Apr-18
Tested By:	DA	**Date Tested:**	2-May-18
Reviewed By:		**Date Reviewed:**	

Test Procedure: ASTM 1557 **Method:** A

Oversized Material (%): 8.8 **Correction Required:** [ ] yes [x] no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	128.5	**@ Optimum Moisture, %**	9.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:

AASHTO Soils Classification:

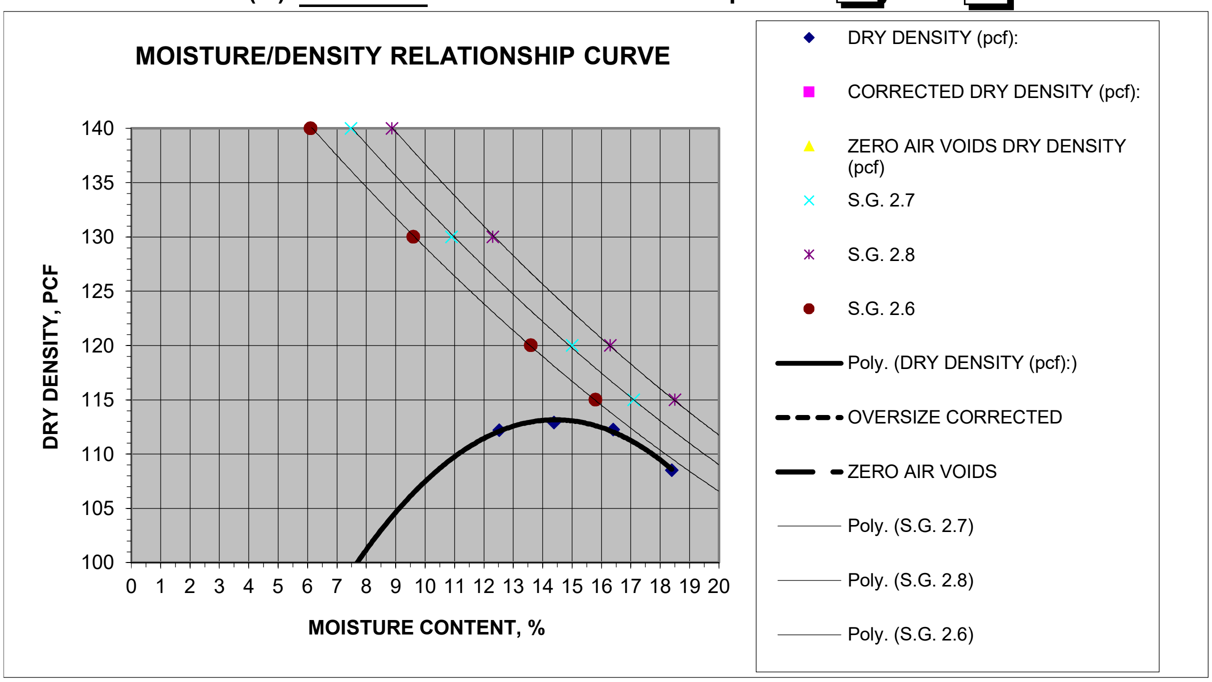
GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client: Nutmeg South **Job No.:** 3539-SD
Project: Nutmeg **Lab No.:** Corona
Location: 0
Material Type: Olive Brown Silty Clay
Material Supplier:
Material Source:
Sample Location: B-2@ 0 - 5

Sampled By: CL **Date Sampled:** 20-Apr-18
Received By: DA **Date Received:** 27-Apr-18
Tested By: DA **Date Tested:** 2-May-18
Reviewed By: **Date Reviewed:**

Test Procedure: ASTM 1557 **Method:** A
Oversized Material (%): 8.8 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	113.0	**@ Optimum Moisture, %**	15.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:
AASHTO Soils Classification:

MOISTURE/DENSITY RELATIONSHIP

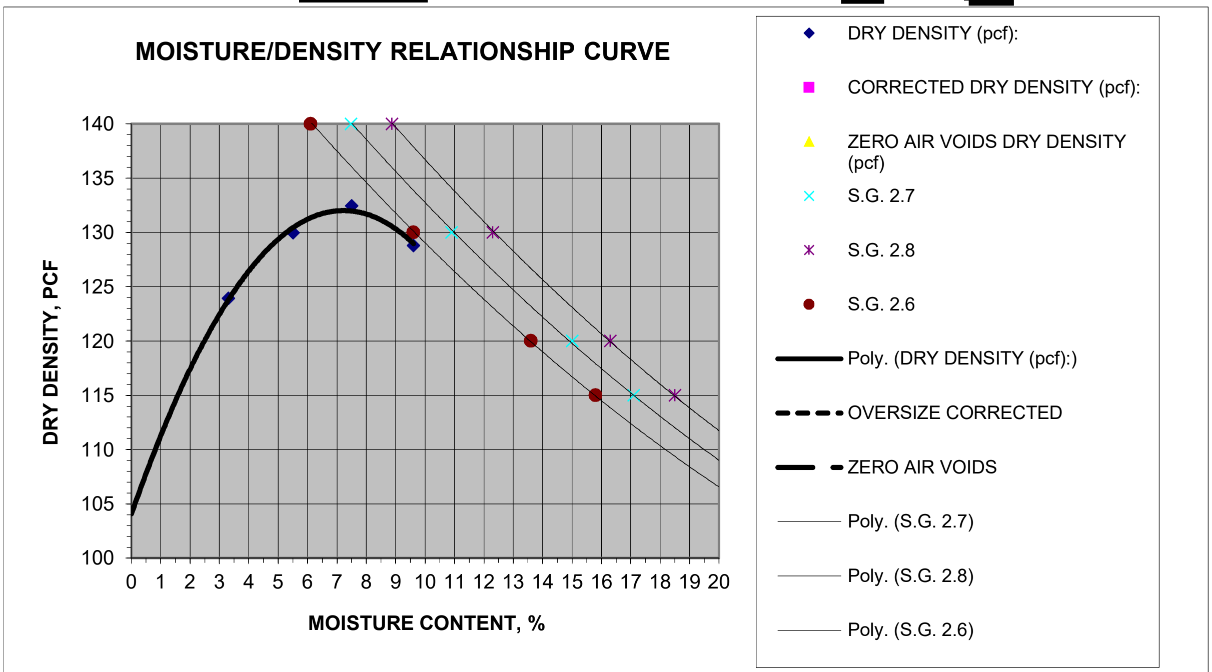
Client: Nutmeg South, L.L.C.
Project: Nutmeg
Location: 0
Material Type: Gray Brown F - C Sand w/ Silt
Material Supplier:
Material Source:
Sample Location: B-6 @ 1 - 5

Job No.: 3539-SD
Lab No.: Corona

Sampled By: CDL
Received By: DLI
Tested By: DLI
Reviewed By:

Date Sampled: 18-May-18
Date Received: 22-May-18
Date Tested: 25-May-18
Date Reviewed: 25-May-18

Test Procedure: ASTM 1557 **Method:** A
Oversized Material (%): 12.2 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	132.5	**@ Optimum Moisture, %**	7.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

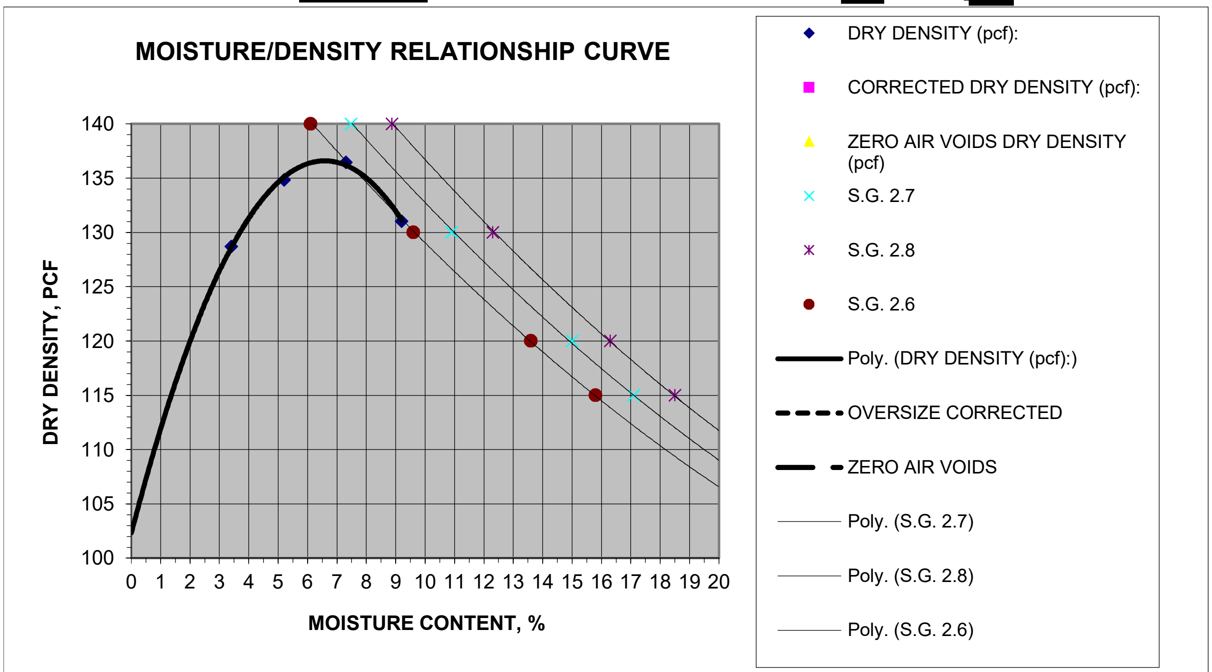
Unified Soils Classification:
AASHTO Soils Classification:

MOISTURE/DENSITY RELATIONSHIP

Client: Nutmeg South, L.L.C. **Job No.:** 3539-SD
Project: Nutmeg **Lab No.:** Corona
Location: 0
Material Type: Brown Silty F - C Sand
Material Supplier:
Material Source:
Sample Location: B-8 @ 1 - 5

Sampled By: CDL **Date Sampled:** 18-May-18
Received By: DLI **Date Received:** 22-May-18
Tested By: DLI **Date Tested:** 25-May-18
Reviewed By: **Date Reviewed:** 25-May-18

Test Procedure: ASTM 1557 **Method:** A
Oversized Material (%): 13.2 **Correction Required:** ☐ yes ☒ no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	137.0	**@ Optimum Moisture, %**	7.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:
AASHTO Soils Classification:

● ANALYSIS
● DESIGN

LaBelle • Marvin

PROFESSIONAL PAVEMENT ENGINEERING

A CALIFORNIA CORPORATION

● SOILS, ASPHALT TECHNOLOGY

June 1, 2018

Ms. Anna Scott
GeoTek Inc.
1548 North Maple Street
Corona, California 92880

Project No. 43678

Attention Ms. Scott:
Laboratory testing of the bulk soil samples delivered to our laboratory on 5/30/2018 has been completed.

Reference:	W.O. # 3539-SD
Project:	Nutmeg South LLC
Samples:	B-6 @1-5'
	B-12 @1-5'
	B-8 @1-5'

Data sheets and graphical presentations are transmitted herewith for your use and information. Any untested portion of the samples will be retained for a period of sixty (60) days prior to disposal. The opportunity to be of service is appreciated, and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:jw
Enclosures

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LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 43678

DATE: 6/1/2018

BORING NO. B-6 @ 1-5'

Nutmeg South LLC

W.O.# 3539-SD

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	20	39	15
Initial Test Water, %	7.7	9.5	7.3
Compact Gage Pressure,psi	350	105	350
Exudation Pressure, psi	389	120	618
Height Sample, Inches	2.45	2.52	2.42
Gross Weight Mold, grams	3103	3116	2913
Tare Weight Mold, grams	1955	1950	1775
Sample Wet Weight, grams	1148	1166	1138
Expansion, Inches x 10exp-4	3	2	7
Stability 2,000 lbs (160psi)	12 / 22	16 / 35	11 / 17
Turns Displacement	4.38	5.10	4.33
R-Value Uncorrected	78	64	83
R-Value Corrected	78	64	82
Dry Density, pcf	131.8	128.1	132.9

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.23	0.37	0.18
G. E. by Expansion	0.10	0.07	0.23

Equilibrium R-Value	**75** by EXUDATION	Examined & Checked: 6 /1/ 18
REMARKS:	Gf = 1.25 3.0% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 43678

DATE: 6 /1/ 18

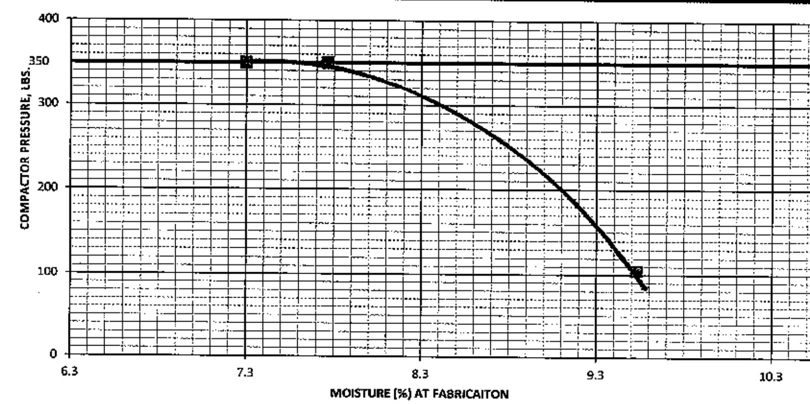
REMARKS:

BORING NO. B-6 @ 1-5'

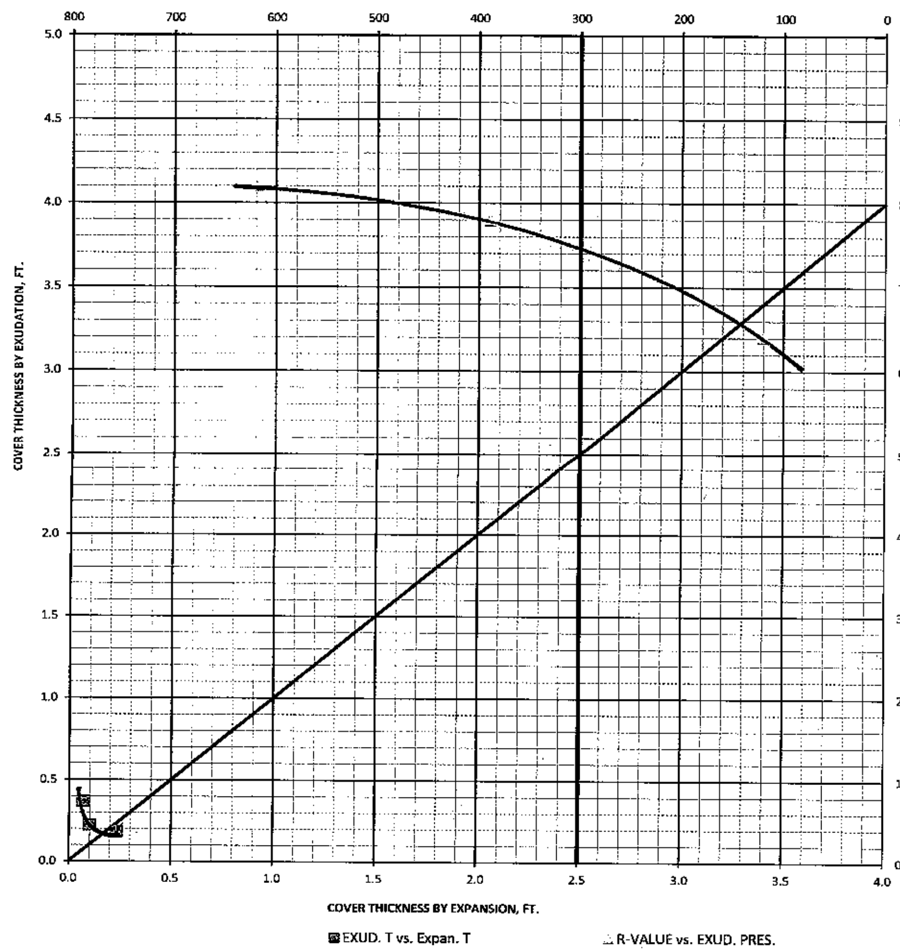
Nutmeg South LLC

W.O.# 3539-SD

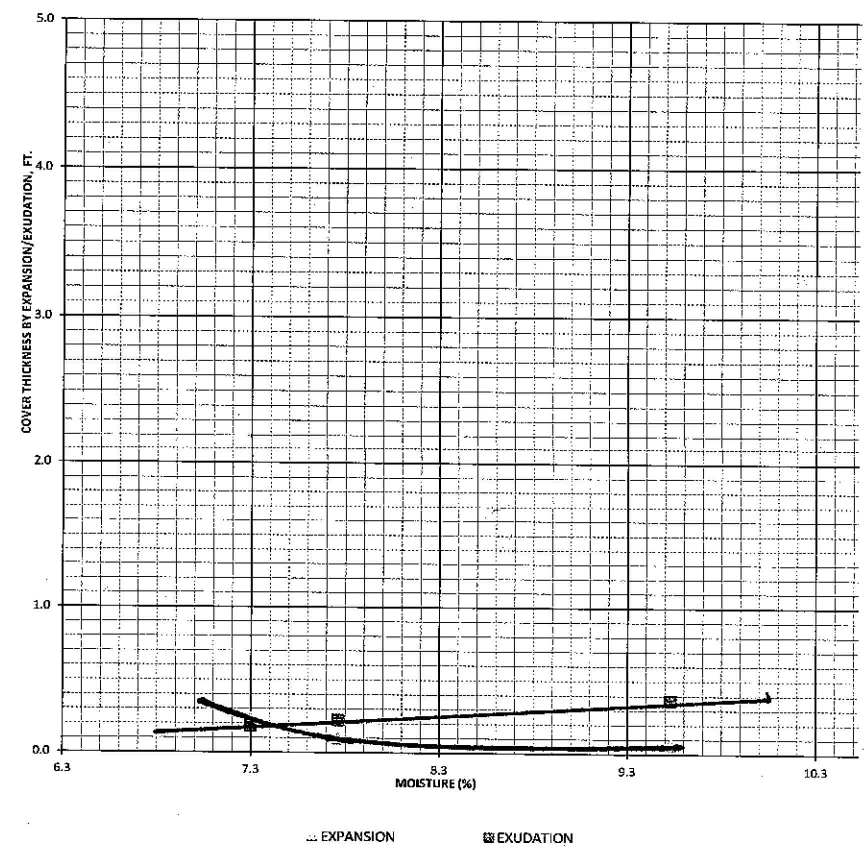
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 43678

DATE: 6/1/2018

BORING NO. B-12 @ 1-5'

Nutmeg South LLC

W.O.# 3539-SD

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	10	11	12
Water added, grams	24	39	14
Initial Test Water, %	8.7	10.1	7.8
Compact Gage Pressure,psi	90	60	250
Exudation Pressure, psi	427	186	739
Height Sample, Inches	2.43	2.49	2.41
Gross Weight Mold, grams	3097	3116	3092
Tare Weight Mold, grams	1947	1952	1948
Sample Wet Weight, grams	1150	1164	1144
Expansion, Inches x 10exp-4	5	1	14
Stability 2,000 lbs (160psi)	14 / 28	22 / 46	10 / 17
Turns Displacement	4.75	5.08	4.70
R-Value Uncorrected	71	55	82
R-Value Corrected	70	55	81
Dry Density, pcf	131.9	128.6	133.4

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.31	0.46	0.19
G. E. by Expansion	0.17	0.03	0.47

Equilibrium R-Value	**62** by EXUDATION	Examined & Checked: 6 /1/ 18
REMARKS:	Gf = 1.25 2.0% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 43678

DATE: 6 /1/ 18

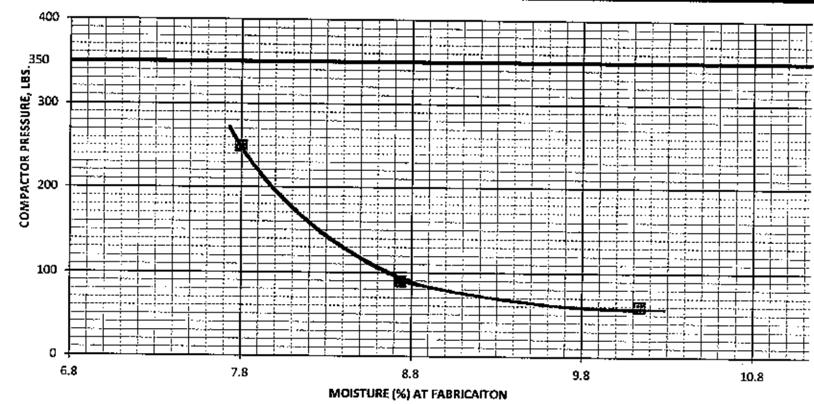
REMARKS:

BORING NO. B-12 @ 1-5'

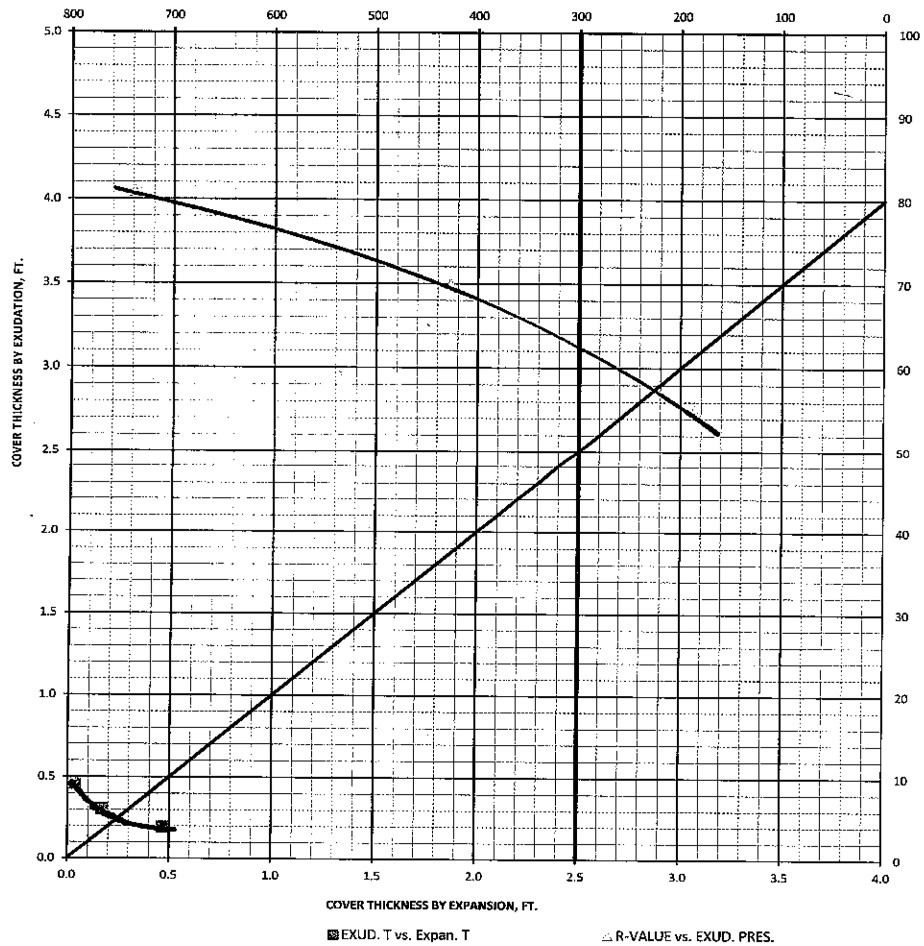
Nutmeg South LLC

W.O.# 3539-SD

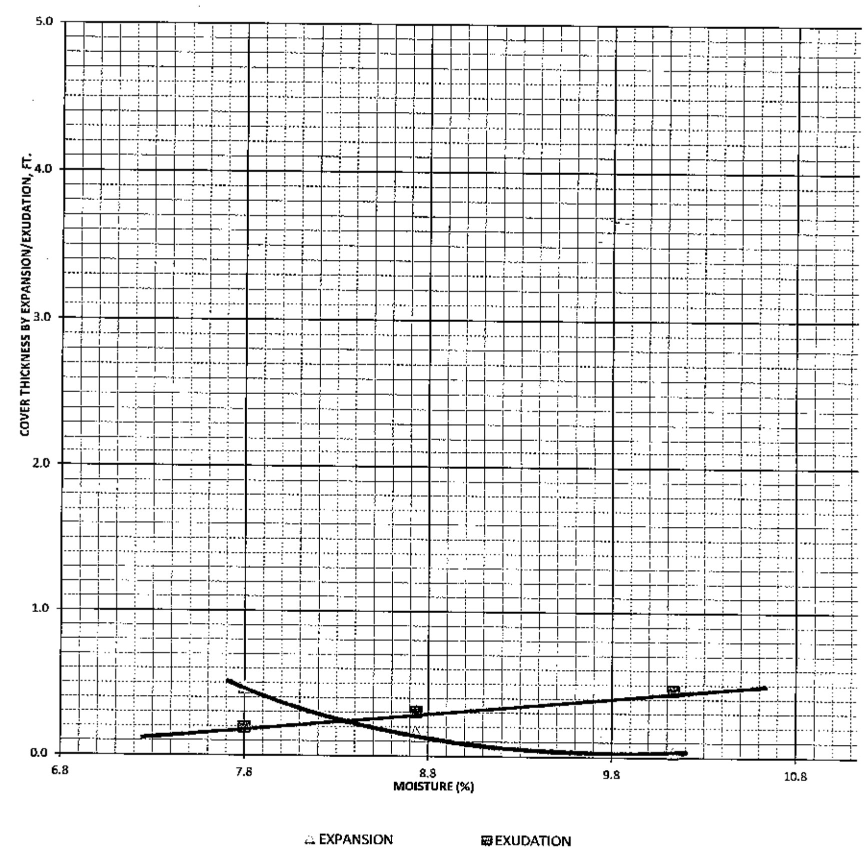
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 43678

DATE: 6/1/2018

BORING NO. B-8 @ 1-5'

Nutmeg South LLC

W.O.# 3539-SD

SAMPLE DESCRIPTION: Brown Silty Sand

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	4	5	6
Water added, grams	56	71	51
Initial Test Water, %	7.8	9.2	7.3
Compact Gage Pressure,psi	150	50	210
Exudation Pressure, psi	450	198	765
Height Sample, Inches	2.40	2.48	2.38
Gross Weight Mold, grams	3092	3121	3089
Tare Weight Mold, grams	1959	1960	1958
Sample Wet Weight, grams	1133	1161	1131
Expansion, Inches x 10exp-4	11	10	18
Stability 2,000 lbs (160psi)	14 / 26	20 / 43	13 / 22
Turns Displacement	4.62	5.22	4.25
R-Value Uncorrected	74	57	79
R-Value Corrected	72	57	78
Dry Density, pcf	132.7	129.9	134.2

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.29	0.44	0.23
G. E. by Expansion	0.37	0.33	0.60

Equilibrium R-Value	**64** by EXUDATION	Examined & Checked: 6 /1/ 18
REMARKS:	Gf = 1.25 1.8% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 43678

DATE: 6 /1/ 18

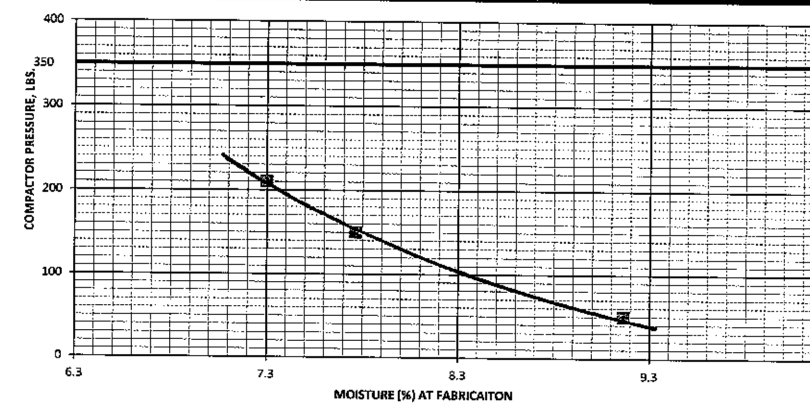
REMARKS:

BORING NO. B-8 @ 1-5'

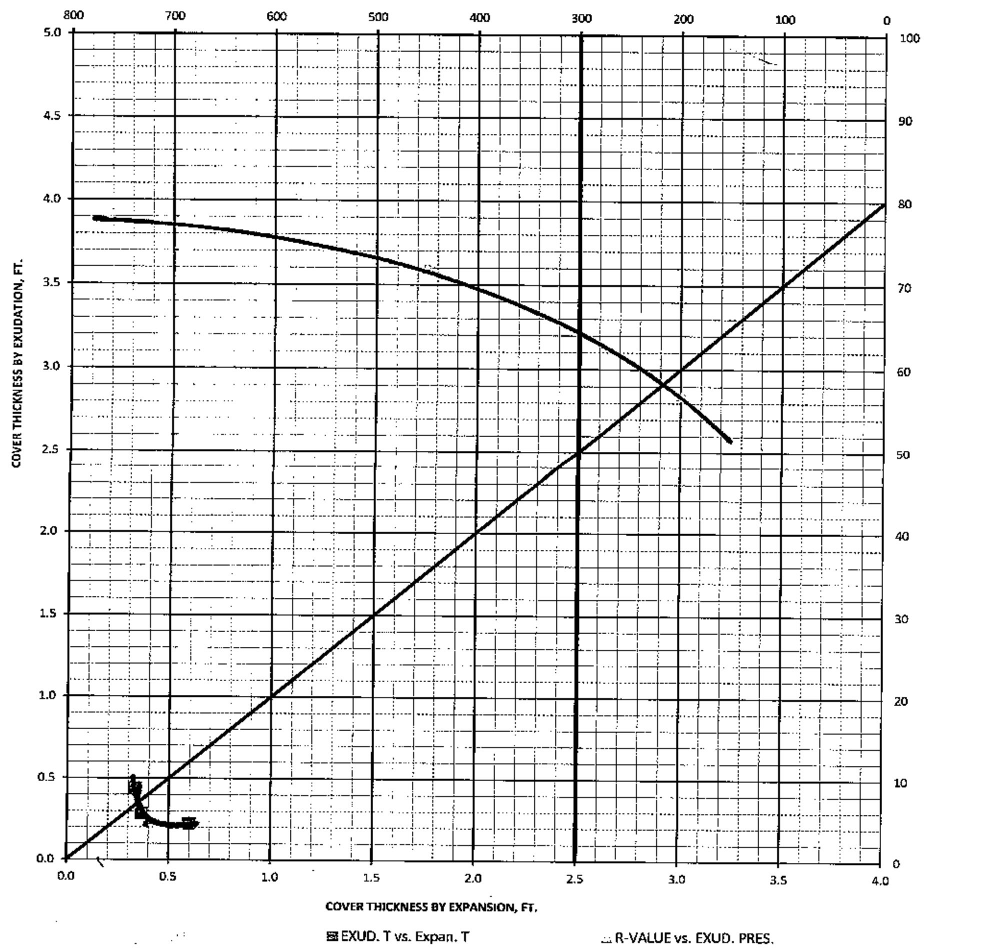
Nutmeg South LLC

W.O.# 3539-SD

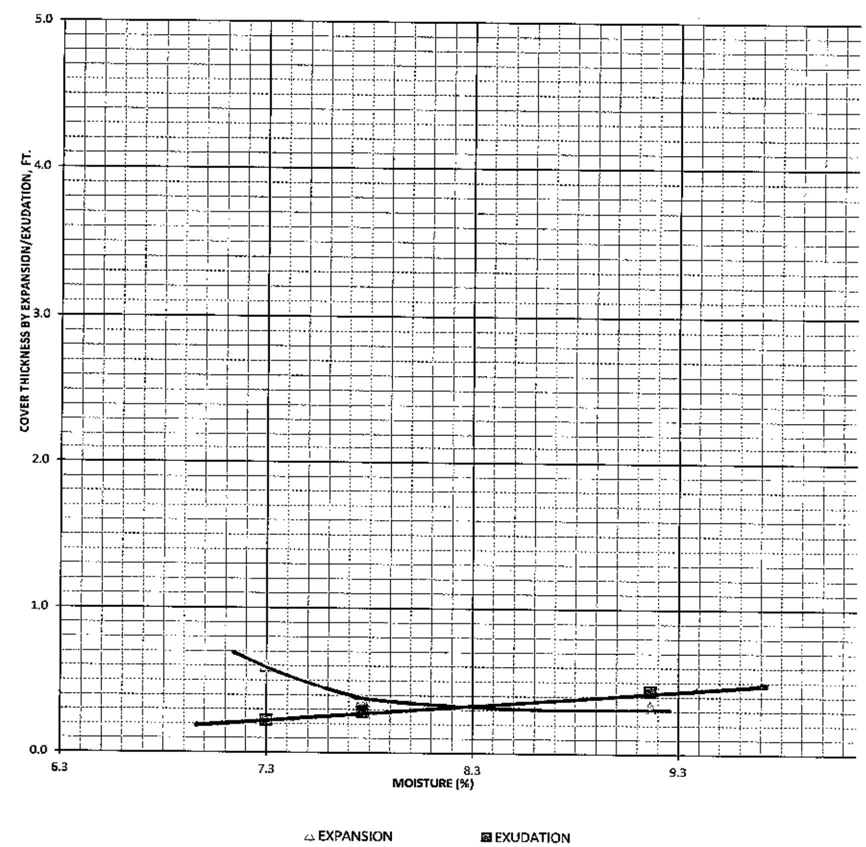
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



Results Only Soil Testing for Nutmeg

June 8, 2018

Prepared for:
Anna Scott
Geotek Inc
1548 North Maple Street
Corona, CA 92880
ascott@geotekusa.com

Project X Job#: S180604E
Client Job or PO#: 3539-SD

Soil Analysis Lab Results

Client: Geotek Inc
Job Name: Nutmeg
Client Job Number: 3539-SD
Project X Job Number: S180604E
June 8, 2018

	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)	
B-12	10.0	46,230	22,110	150	0.0150	90	0.0090	90	16.5	1.65	145	7.30
B-8	1-5	536,000	5,427	60	0.0060	75	0.0075	ND	15.0	1.20	78	7.93

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.

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Results Only Soil Testing
for
North Nutmeg St

May 4, 2018

Prepared for:
Anna Scott
Geotek Inc
1548 North Maple Street
Corona, CA 92880
ascott@geotekusa.com

Project X Job#: S180430M
Client Job or PO#: 3539-SD

Soil Analysis Lab Results

Client: Geotek Inc
Job Name: North Nutmeg St
Client Job Number: 3539-SD
Project X Job Number: S180430M
May 2, 2018

	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)	
B-1	0.0-5.0	38,860	3,819	120	0.0120	120	0.0120	36	21.5	1.80	172	9.00
B-2	0.0-5.0	14,740	804	180	0.0180	144	0.0144	36	2.0	0.06	187	8.46

Unk = Unknown
NT = Not Tested
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
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance to the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the log of exploratory borings in Appendix B.

Consolidation
One-dimensional collapse testing was performed on selected samples of the site soils according to ASTM Test Method D 2435. The results of this testing are presented as Enclosures C-1 and C-2.

Expansion Index
Expansion Index testing was performed on three soil samples. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below.

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	0-5	Clayey Sand	18	Very Low
B-2	0-5	Sandy Silt	61	Medium
B-5	0-5	Silty Sand	27	Low

In-Situ Moisture and Density
The natural water content was determined (ASTM D 2216) on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density determinations (ASTM D 2937) were performed on relatively undisturbed samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths in Appendix B.

Moisture-Density Relationship
Laboratory testing was performed on four samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D 1557. The results of the testing are provided below.

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	0-5	Brown Clayey Sand	128.5	9.5
B-2	0-5	Olive Brown Silty Clay	113.0	15.0
B-6	1-5	Gray Brown Sand w/ Silt	132.5	7.5
B-8	1-5	Brown Silty Sand	137.0	7.0

Direct Shear
Direct shear testing was performed on samples collected from the site and remolded to approximately 90 percent of the soil's maximum dry density as determined per ASTM D 1557. In addition, "undisturbed" bedrock samples were tested in shear. The samples were tested saturated. The results are presented as Enclosures C-3 through C-8.

R-Value
The R-value testing was conducted by others on three near surface samples collected along the subject segment of North Centre City Parkway. The tests were performed in general accordance with Caltrans test method 301. Test results are presented as Enclosures C-9 through C-14.

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed by others in general accordance with California Test No. 417. Resistivity testing was completed by others in general accordance with California Test No. 643. Testing to determine the chloride content was performed by others in general accordance with California Test No. 422. The results of the testing are provided below.

Boring No.	Depth (ft)	Resistivity (ohms-cm)	Sulfates (wt%)	Chlorides (ppm)	pH
B-1	0-5	3,819	0.0120	120	9.00
B-2	0-5	804	0.0180	144	8.46
B-8	1-5	5,427	0.0060	75	7.93
B-12	10	22,110	0.0150	90	7.30

APPENDIX D

GENERAL GRADING GUIDELINES

Nutmeg South, LLC
Escondido, California
Project No. 3539-SD

GEOTEK

GENERAL GRADING GUIDELINES

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

General

Grading should be performed to at least the minimum requirements of governing agencies, 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.

Piping Potential and Filter Blankets:

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

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

Subdrainage

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

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

Monitoring

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the 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.

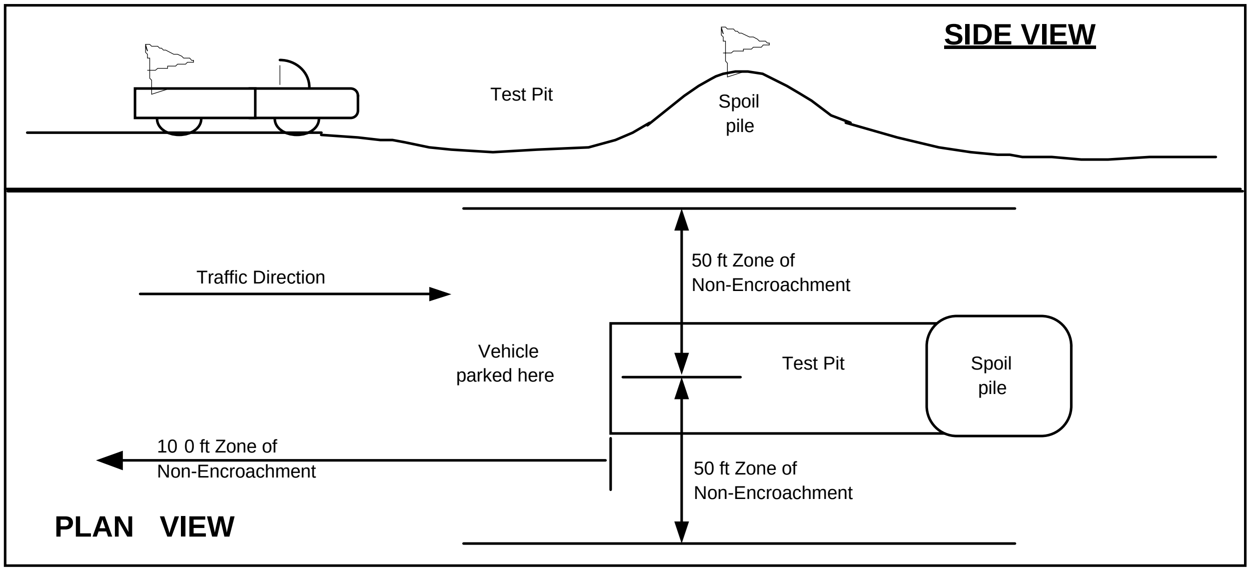
Test Pits Location, Orientation and Clearance

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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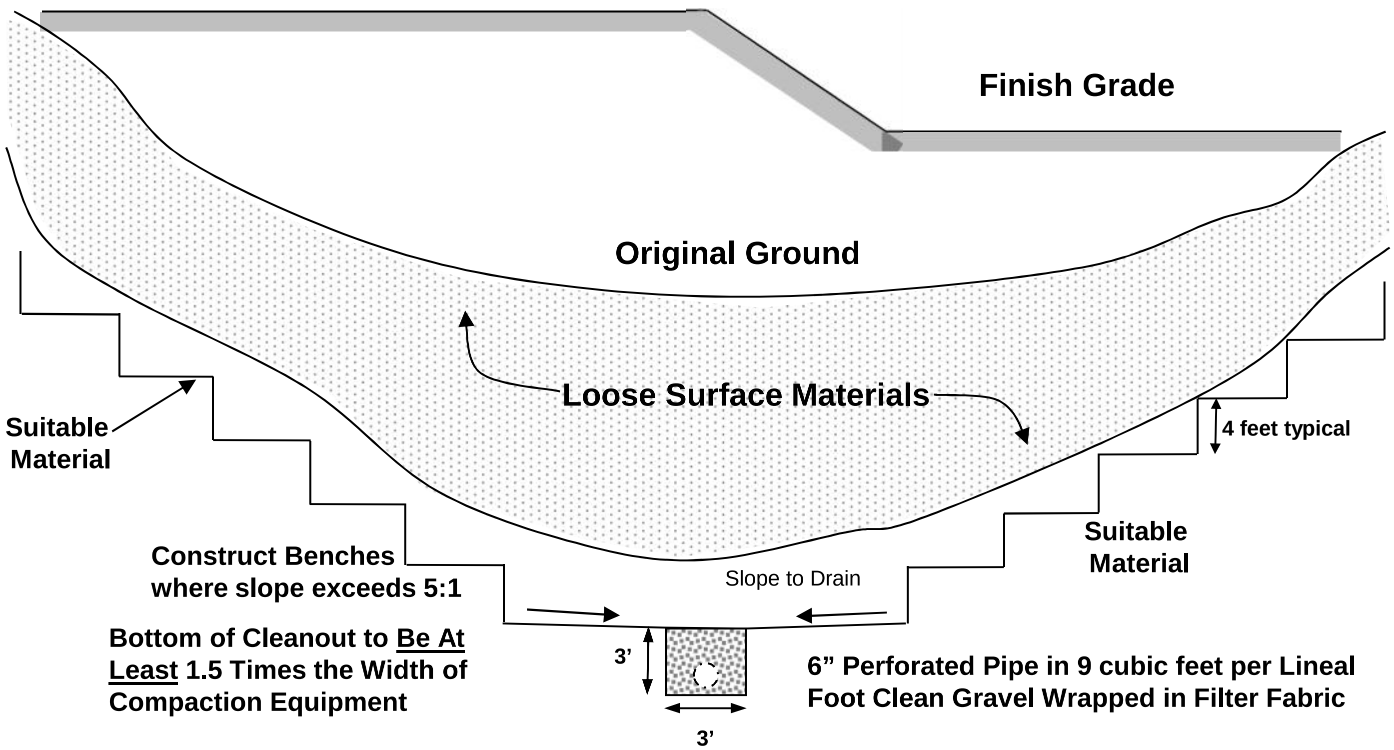
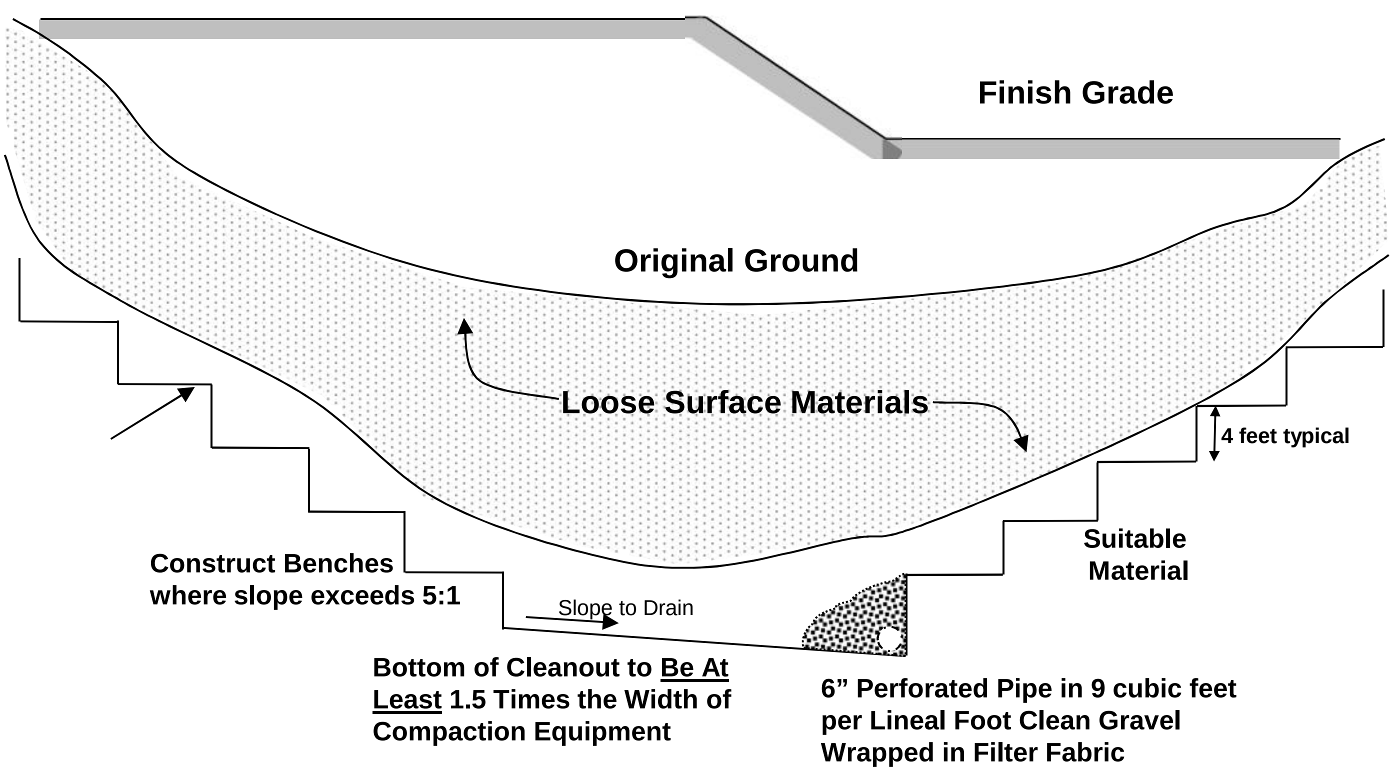
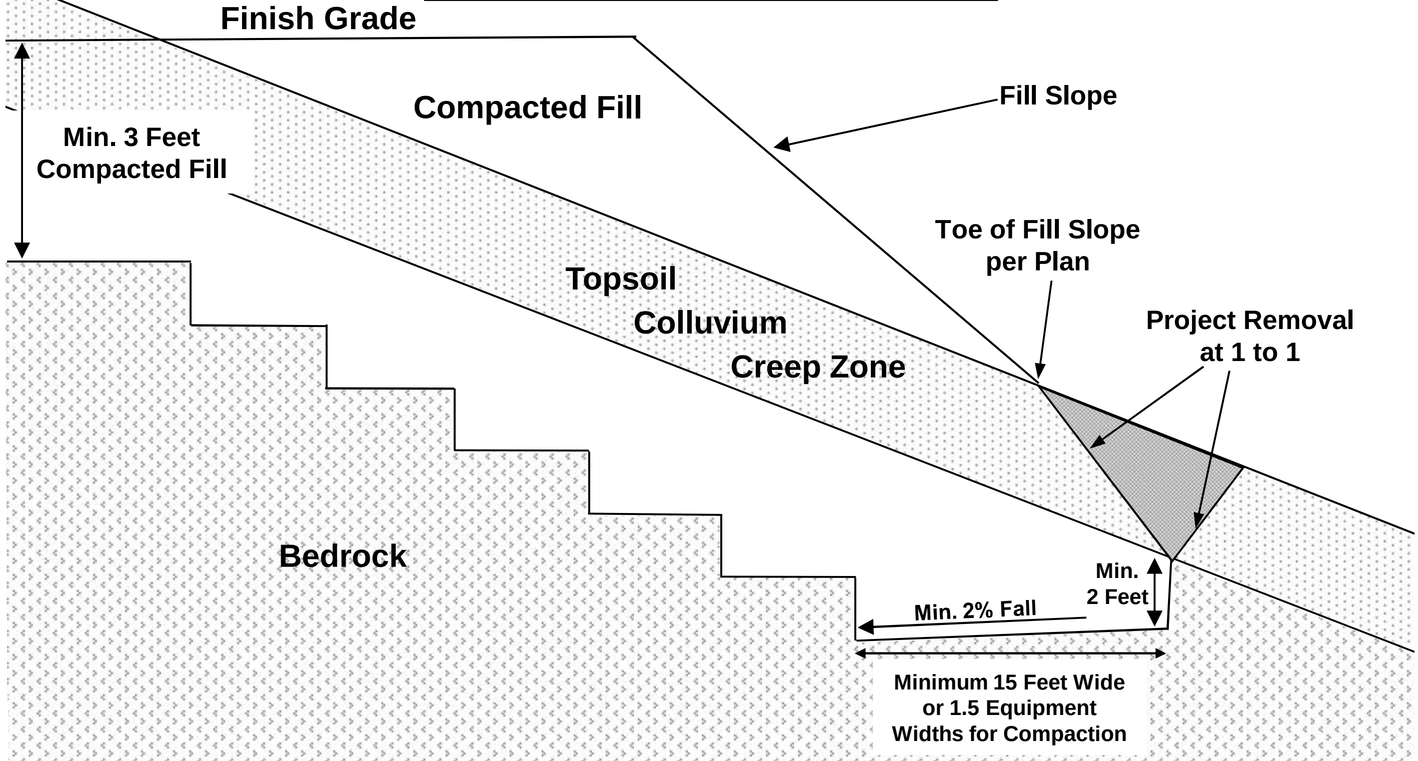
ALTERNATES



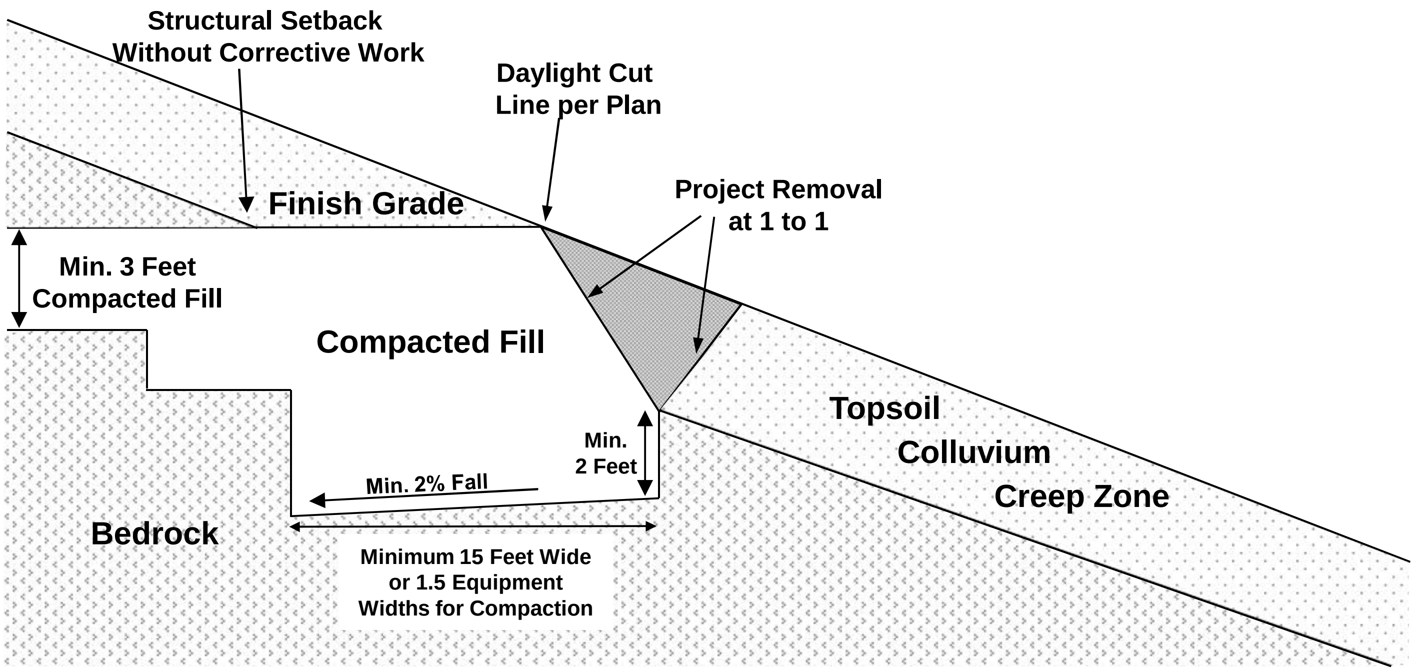


GEOTEK	1384 Poinsettia Avenue, Suite A Vista, California 92083	TYPICAL CANYON CLEANOUT	STANDARD GRADING GUIDELINES PLATE G-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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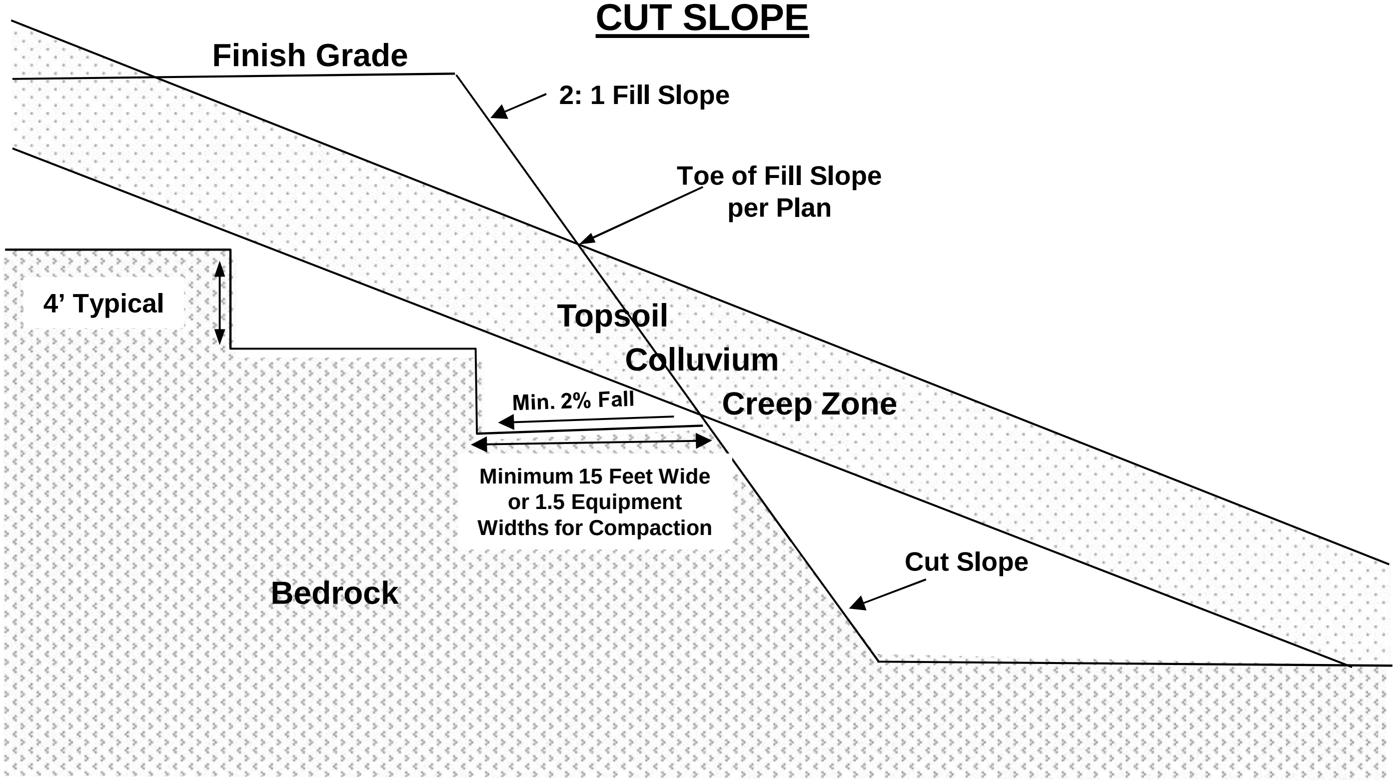
TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE G-2

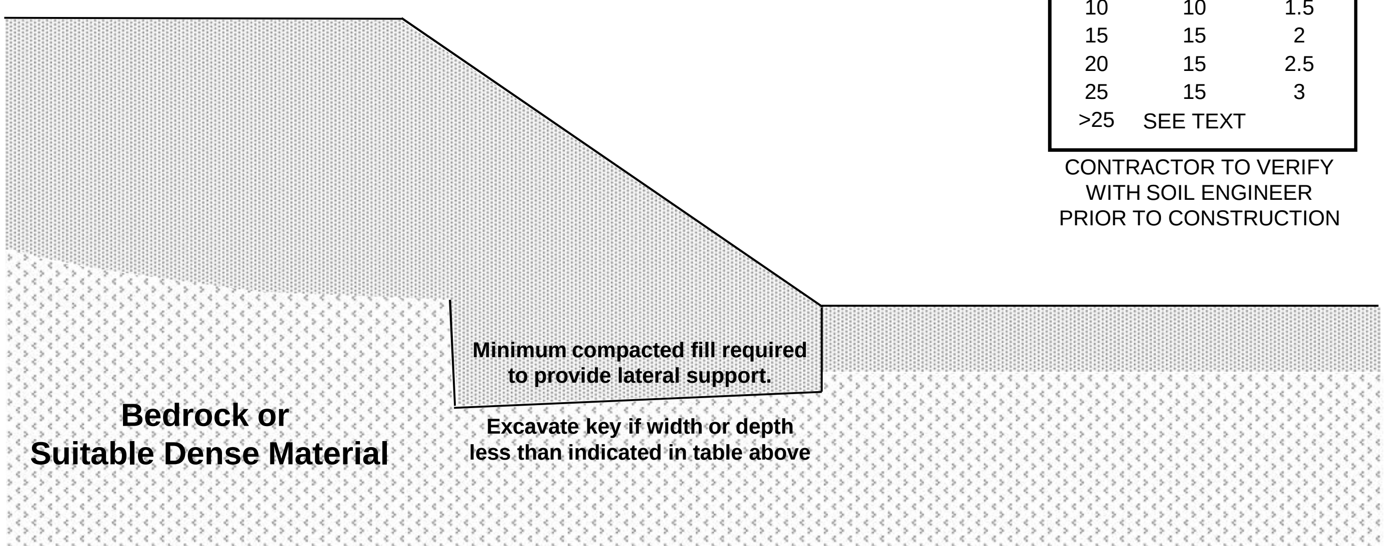
TYPICAL FILL SLOPE OVER CUT SLOPE



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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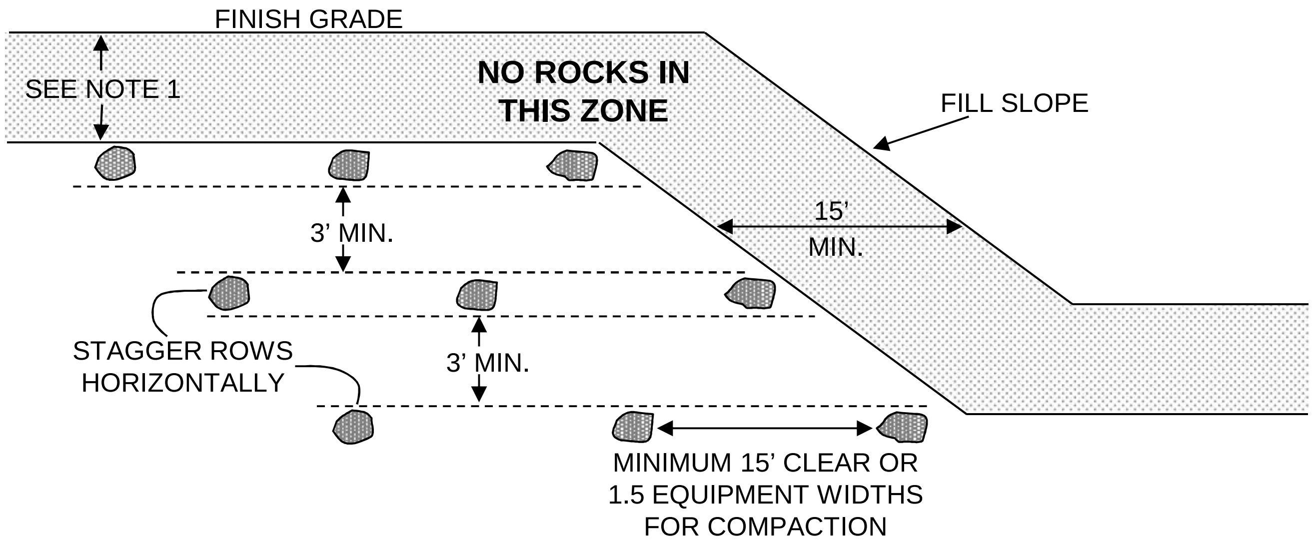
COMMON FILL SLOPE KEYS

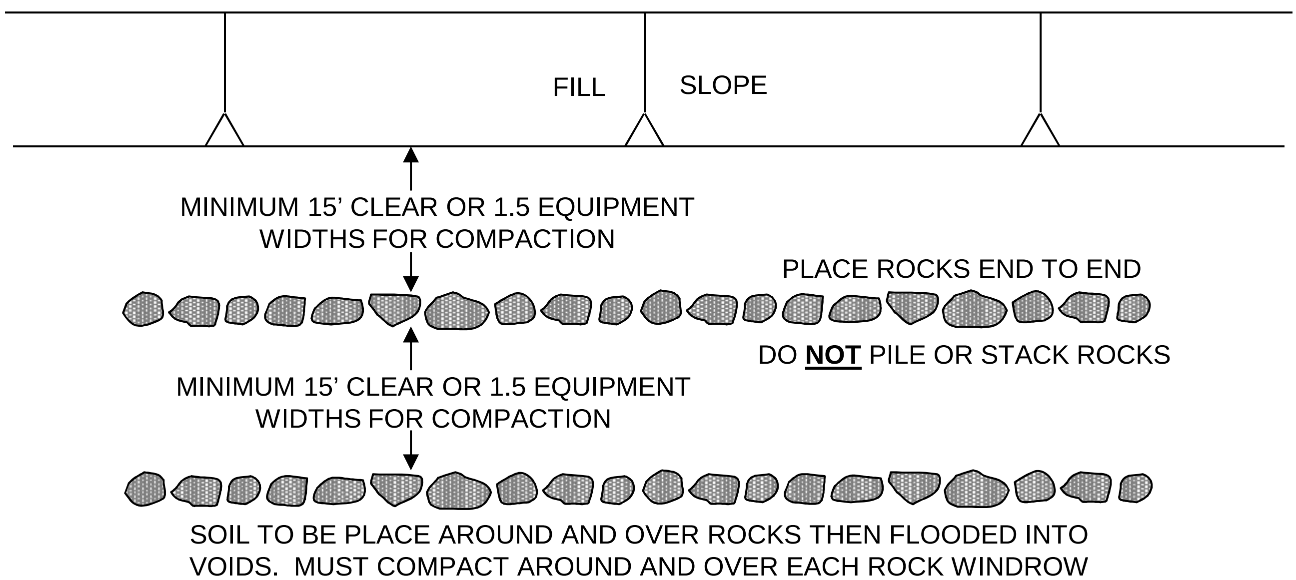
STANDARD GRADING GUIDELINES

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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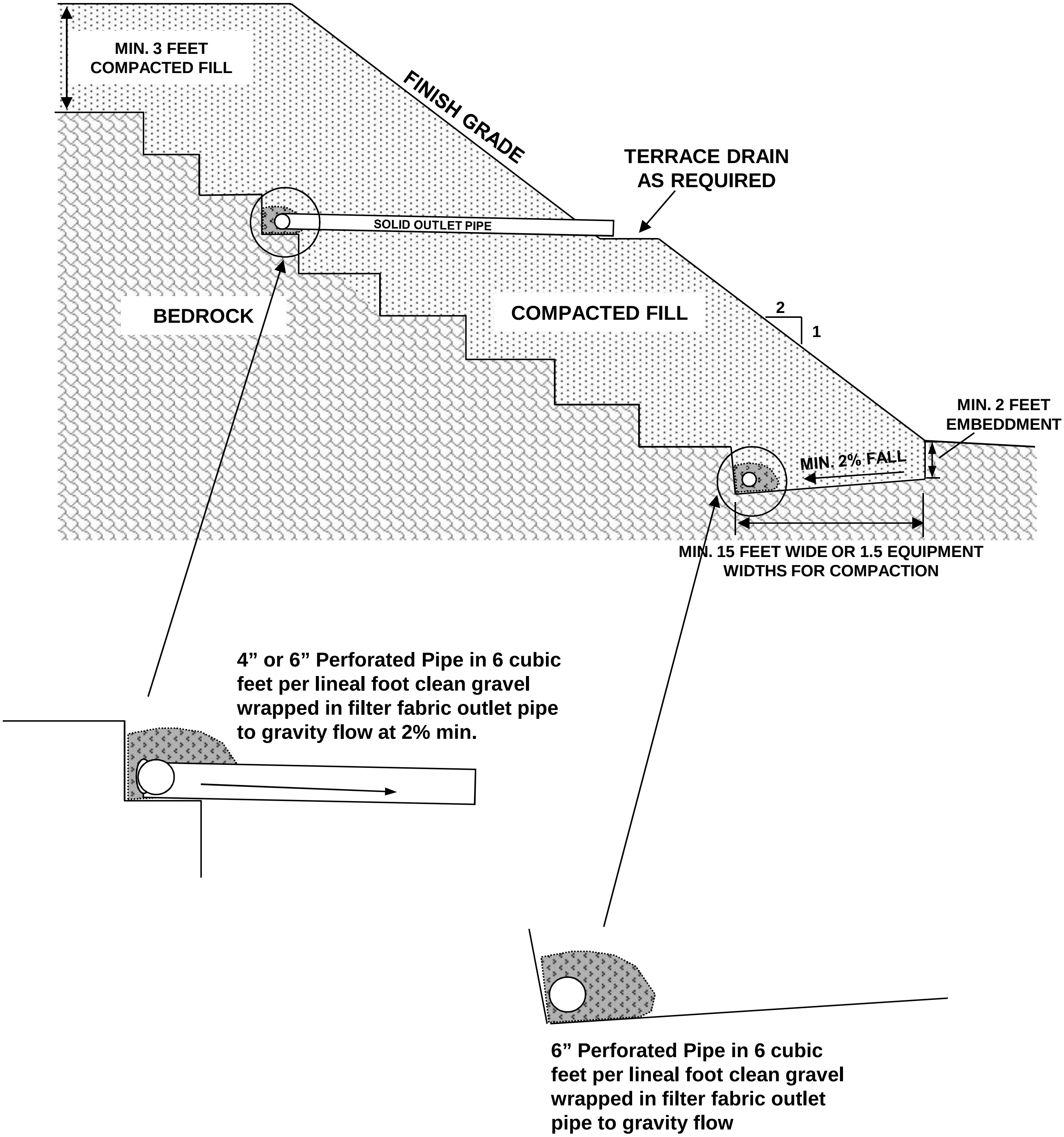
ROCK BURIAL DETAILS

STANDARD GRADING GUIDELINES

PLATE G-4



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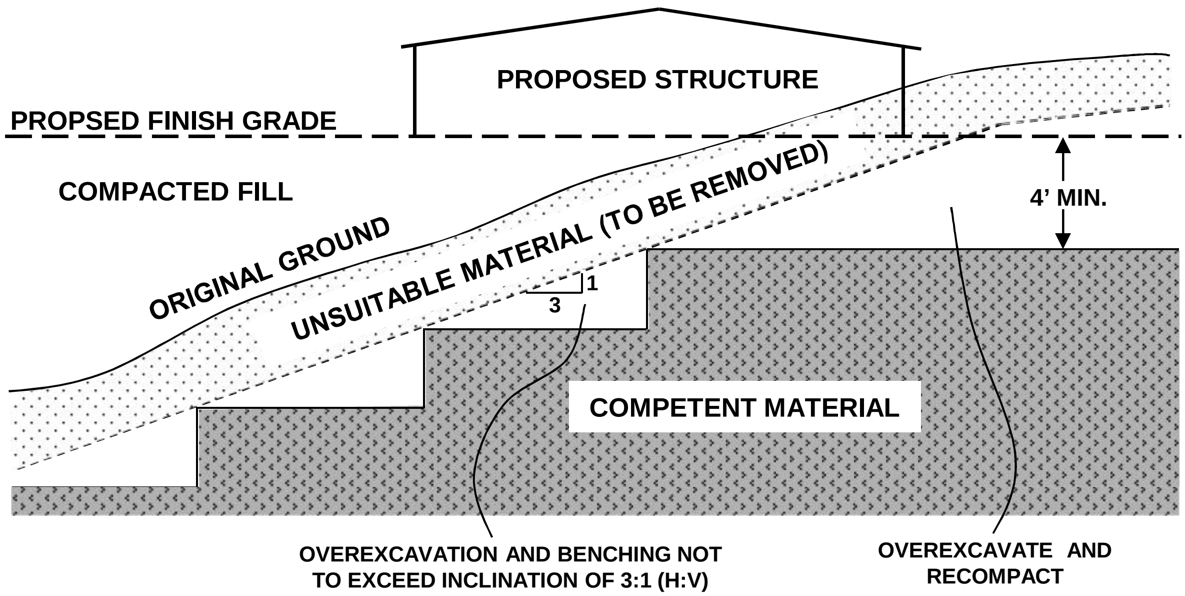
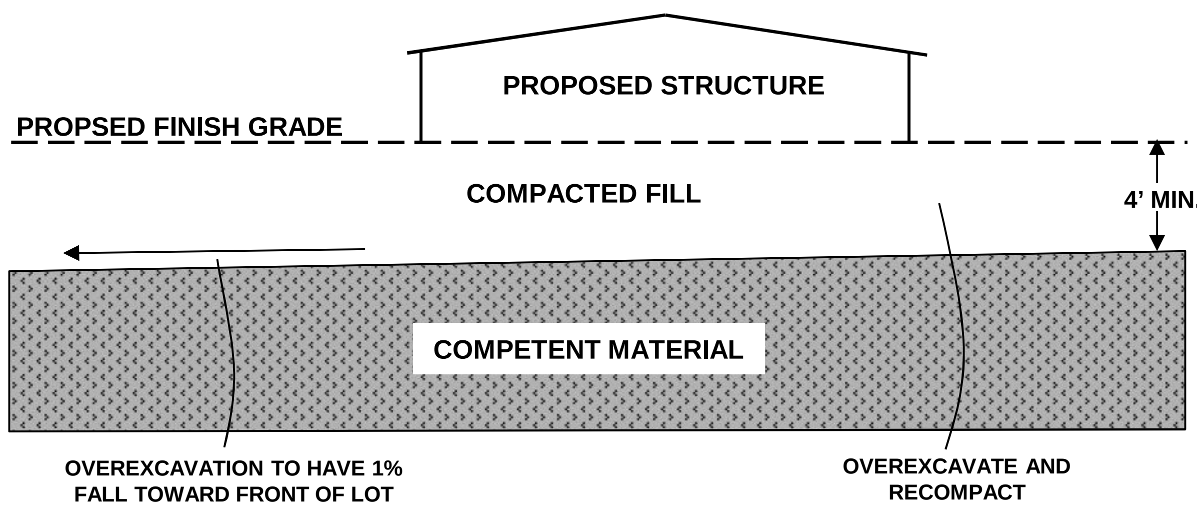
Typical Buttress and Stabilization Fill

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