GEOTECHNICAL AND INFILTRATION EVALUATION
PROPOSED LIBRARY LEARNING RESOURCE CENTER AND STUDENT SERVICES BUILDING
NORCO COLLEGE
2001 THIRD STREET
NORCO, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

RIVERSIDE COMMUNITY COLLEGE DISTRICT
3801 MARKET STREET
RIVERSIDE, CALIFORNIA 92501

PREPARED BY

GEOTEK, INC.
1548 NORTH MAPLE STREET
CORONA, CALIFORNIA 92878

PROJECT NO. 4400-CR **NOVEMBER 14, 2025**

GEOTEK

GeoTek, Inc.
1548 North Maple Street, Corona, California 92878
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

November 14, 2025
Project No. 4400-CR

Riverside Community College District
3801 Market Street
Riverside, California 92501

Attention: Mr. Mike Clark

Subject: **Geotechnical and Infiltration Evaluation**
Proposed Library Learning Resource Center and Student Services Building
Norco College
2001 Third Street
City of Norco, Riverside County, California

Dear Mr. Clark:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical and Infiltration Evaluation for the proposed Library Learning Resource Center and Student Services Building to be constructed on the Norco College campus. Norco College is located at 2001 Third Street, in the City of Norco, Riverside County, California. This report presents a discussion of GeoTek's evaluation and provides preliminary geotechnical recommendations for site geologic hazards, site soil infiltration characteristics, site preparation, foundation design, retaining wall design, pavement design, concrete flatwork and construction of the proposed site improvements.

Based upon review and evaluation, site development appears feasible from a geotechnical viewpoint provided that the recommendations included in this report are incorporated into the design and construction phases of site development.

The opportunity to be of service is sincerely appreciated. If you have any questions, please do not hesitate to contact GeoTek.

Respectfully submitted,
GeoTek, Inc.





Bruce A. Hick
GE 2284, Exp. 12/31/26
Geotechnical Engineer

Edward H. LaMont
CEG 1892, Exp. 07/31/26
Principal Geologist



Jianguo Yang
PE C 98289, Exp. 12/31/27
Project Engineer

Distribution: (1) Addressee via email (one PDF file)

TABLE OF CONTENTS

1. PURPOSE AND SCOPE OF SERVICES 1

2. SITE DESCROPTION AND PROPOSED DEVELOPMENT 2
2.1 SITE DESCRIPTION 2
2.2 PROPOSED DEVELOPMENT 2

3. FIELD EXPLORATION AND LABORATORY TESTING 3
3.1 FIELD EXPLORATION 3
3.2 LABORATORY TESTING 5

4. GEOLOGIC AND SOILS CONDITIONS 6
4.1 REGIONAL SETTING 6
4.2 GENERAL SOIL/GEOLOGIC CONDITIONS 6
4.2.1 Artificial Fill (Af) 7
4.2.2 Older Alluvium (Qof) 7
4.2.3 Granitic Bedrock (Kmp) 7
4.3 SURFACE AND GROUNDWATER 8
4.3.1 Surface Water 8
4.3.2 Groundwater 8
4.4 FAULTING AND SEISMICITY 8
4.4.1 Seismic Design Parameters 9
4.5 LIQUEFACTION AND LATERAL SPREAD 10
4.6 SEISMIC SETTLEMENTS 10
4.7 OTHER SEISMIC HAZARDS 10

5. CONCLUSIONS AND RECOMMENDATIONS 11
5.1 GENERAL 11
5.2 EARTHWORK CONSIDERATIONS 11
5.2.1 Site Clearing and Demolition 12
5.2.2 Site Preparation - Building Areas 12
5.2.3 Pavement Areas 13
5.2.4 Hardscape Areas 13
5.2.5 Preparation of Excavation Bottoms 13
5.2.6 Engineered Fills 14
5.2.7 Excavation Characteristics 14
5.2.8 Shrinkage and Subsidence 14
5.2.9 Import Soils 14
5.3 DESIGN RECOMMENDATIONS 15
5.3.1 Foundation and Building Slab Design 15
5.3.2 Moisture Vapor and Retarding System 17
5.3.3 Miscellaneous Foundation Recommendations 18
5.3.4 Foundation Setbacks 18
5.3.5 Soil Corrosivity 18
5.3.6 Soil Sulfate Content 19
5.3.7 Drilled Piers 19
5.3.8 Soil Corrosivity 20

TABLE OF CONTENTS

5.4 RETAINING WALL DESIGN AND CONSTRUCTION 21
5.4.1 General Design Criteria *21*
5.4.2 Cantilevered Walls *22*
5.4.3 Retaining Wall Backfill and Drainage *22*
5.4.4 Restrained Retaining Walls *23*
5.4.5 Other Design Considerations *24*
5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS 24
5.5.1 Asphaltic Concrete Pavement *24*
5.5.2 Interlocking Concrete Pavers *25*
5.5.3 Portland Cement Concrete (PCC) Pavement *25*
5.5.4 Pavement Construction *26*
5.6 CONCRETE FLATWORK 27
5.6.1 Exterior Concrete Slabs, Sidewalks and Driveways *27*
5.6.2 Concrete Performance *27*
5.7 POST CONSTRUCTION CONSIDERATIONS 28
5.7.1 Landscape Maintenance and Planting *28*
5.7.2 Drainage *28*
5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS 29

6. INTENT 29

7. LIMITATIONS 30

8. SELECTED REFERENCES 30

ENCLOSURES

Figure 1 – Site Location Map
Figure 2 – Exploration Location Map
Figure 3 – Geologic Map
Figure 4 – Regional Fault Map
Figure 5 – Site Vicinity Map

Plate 1 – Cross Section Location Map
Plate 2A and 2B – Cross-Section A-A' and Cross-Section B-B'

Appendix A – Logs of Exploratory Borings
Appendix B – Laboratory Test Results
Appendix C – Infiltration Test Data and Porchet Calculations
Appendix D – Ground Motion Hazard Analysis
Appendix E – General Grading Guidelines

1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate geotechnical conditions and possible geologic hazards associated with the construction of the Proposed Library Learning Resource Center and Student Services Building on the Norco College campus, located at 2001 Third Street, in the City of Norco, Riverside County, California, as outlined in GeoTek's proposal P-0804925-CR dated August 15, 2025. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data and general information pertinent to the site,
- A site reconnaissance,
- Site exploration consisting of the excavation, logging, and sampling of eight (8) exploratory test borings extending to depths ranging from approximately 11.5 to 51.5 feet below the ground surface,
- Excavation of two (2) additional borings to a depth of about ten (10) feet below grade and performing an infiltration test in each boring,
- Performance of a seismic surface-wave survey that included one traverse for shear-wave analysis purposes.
- Collection of soil samples in the test borings,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, site geologic hazards and
- Compilation of this geotechnical report which presents preliminary recommendations for site development.

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

2. SITE DESCROPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The existing Norco College campus is presented on Figure 1, Site Location Map. The roughly rectangular-shaped approximate 1-acre project site for the proposed new college building is located in the south-centra portion of the existing Norco College campus, at 2001 Third Street, City of Norco, Riverside County, California (see Figure 5, Site Vicinity Map). Access to this portion of the campus is available from Mustang Circle, a paved and improved access road located adjacent to Third Street, and a paved drive aisle north of existing parking lots.

The project site is presently developed and includes two, two-story institutional structures identified as the Student Services Building and the College Police Office. Both of these buildings are surrounded by concrete walkways, landscape planters, and adjacent paved drive aisles. Numerous above and underground utility lines are present in this area.

The ground surface within and around the project area is generally level, with only minor grade variations across the existing paved and landscaped areas. The project site is situated at an elevation of approximately 638 feet above mean sea level (amsl). Surface drainage is primarily conveyed by sheet flow toward the south and southeast through the existing pavement and landscaped areas. Minor grade differences of less than about 3 to 5 feet are present across the project area.

2.2 PROPOSED DEVELOPMENT

Existing on-site improvements within the project area are to be demolished. Based on review of the schematic design documents prepared by DLR Group for the Riverside Community College District (RCCD), dated October 13, 2025, the proposed development consists of constructing a new three-story institutional building identified as the Library Learning Resource Center and Student Services Building. The proposed building will encompass a total gross floor area of approximately 71,739 square feet, with a maximum height of about 52 feet (to top of parapet). The structure is anticipated to be of Type IIA construction, fully sprinklered, with occupancy classifications of B, A-3, and S-1, in accordance with the 2025 California Building Code (CBC). The proposed site improvements also include new concrete and asphalt pavements, curbs, ramps, and stairs with handrails, as well as retaining and seat walls around the building perimeter. Additional improvements consist of new landscaping areas with trees, crosswalks and accessibility ramps, outdoor seating, and drainage facilities. Sewage disposal is anticipated via an existing public

sewer system. On-site stormwater disposal systems are likely also going to be included for development. The footprint of the proposed development is shown on Plate 1, Cross Section Location Map.

GeoTek anticipates that the proposed structure will be supported by a conventional shallow foundation system with a slab-on-grade floor. Anticipate structural loads are 200 to 300 kips for three-story interior columns and 5 kips for perimeter footings. Drilled pier foundations are anticipated to be utilized for covered walkways, light/flag poles, etc.

Based on the review of the current site topography and proposed schematic design, only minor earthwork is anticipated for site development. No major slopes or significant elevation differentials are expected within the project area.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Field exploration for this project was conducted on October 24 and 31, 2025. For the geotechnical portion of the investigation, eight (8) test borings (Borings B-1 through B-8) were excavated with a hollow-stem auger drill rig to a maximum depth of approximately 51.5 feet below the existing ground surface at the boring locations. A truck mounted hollow-stem auger drill rig with an outside diameter of approximately 8.5 inches was utilized to conduct the borings. The inside diameter of the auger was approximately 4.5 inches.

An engineer/geologist from GeoTek logged the exploratory borings and obtained soil samples from within the borings. The approximate boring locations are presented on Figure 2, Exploration Location Map.

In the geotechnical borings, relatively undisturbed soil samples were recovered at various intervals with a California sampler. The California sampler is 2.9-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler is approximately 18 inches in length. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping approximately 30 inches for each blow. The hammer used for this sampling had an energy transfer ratio of 81 percent. The sampler penetration test data are presented on the Logs of Exploratory Borings in Appendix A. The

relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for evaluation and possible testing.

In Boring B-3 standard penetration tests (SPT) were performed starting at a depth of approximately 25 feet beneath the ground surface with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was approximately 18 inches in length. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping approximately 30 inches for each blow. The hammer used for this testing had an energy transfer ratio of 81 percent. The sampler penetration test data are presented on the Logs of Exploratory Borings in Appendix A.

The Logs of Exploratory Borings are presented in Appendix A of this report. The Logs of Exploratory Borings show subsurface conditions at the dates and locations indicated and may not be representative of other locations and times. The stratification lines presented on the logs represent the approximate boundaries between soil types, and the transitions may be gradual.

Infiltration Testing

Two (2) borings (Borings I-1 and I-2) were excavated on the project site on October 24 and 31, 2025, to evaluate preliminary infiltration rates for the proposed storm water management facilities. Infiltration testing was conducted in these borings in general accordance with County of Riverside guidelines.

The infiltration tests consisted of drilling eight-inch diameter test holes to the desired depth and installing approximately two (2) inches of gravel in the bottom of the holes. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the borings. Water was then placed in the borings to presoak the holes and percolation testing was performed following the pre-soak period. Following presoaking, the percolation tests were performed which consisted of adding water to each test hole and measuring the water drop over a 30-minute period. The water drop was recorded, and 30-minute test intervals were completed. Water was added to the test holes after each test interval. The field percolation rates were then converted to an infiltration rate using the Porchet Method. The infiltration rates calculated using the Porchet Method are presented in the following table:

SUMMARY OF PRELIMINARY INFILTRATION RATES		
Boring	Depth of Test (Feet)	Preliminary Infiltration Rate* (Inches per hour)
I-1	10.0	0.06
I-2	10.0	0.02

*Porchet Method converted infiltration rate from field measured rate.

Copies of the percolation data sheets and the Porchet infiltration rate conversion calculations are presented in Appendix C. No factors of safety were applied to the rates provided. Over the lifetime of the infiltration areas, the infiltration rates may be affected by sediment build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soils. Infiltration rates will vary and are mostly dependent on the underlying consistency of the site soils and relative density. Infiltration rates may be impacted by weight of equipment travelling over the soils, placement of engineered fill and other various factors. GeoTek assumes no responsibility or liability for the ultimate design or performance of the storm water facility.

Representatives of GeoTek should observe the soils exposed at the bottom of the stormwater control facilities during construction/earthwork operations to confirm suitability and that the conditions exposed are as anticipated for the proposed stormwater disposal facilities.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected soil and bedrock samples obtained during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the earth materials encountered and to evaluate the physical properties of the earth materials for use in engineering design and analysis.

Optimum moisture content-maximum dry density relationships were established for typical soil types so that the relative compaction of the soils could be determined. Expansion index testing was performed on selected samples to evaluate the expansion potential of the on-site soils. Direct shear testing was conducted on remolded samples in order to help determine the shear strength of recompacted (fill) soils under saturated conditions. Selected soil samples were tested in order to evaluate their potential for hydro-collapse. Chemical testing, comprised of pH, soluble sulfate, chloride and resistivity testing, was conducted on selected soil samples.

The moisture-density and moisture content determinations are presented on the Logs of Exploratory Borings in Appendix A of this report. The maximum density, expansion index, direct shear, hydro-collapse, Chemical test data are presented in Appendix B of this report.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

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

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by old alluvium fan deposits (Qof) (Morton, D.M., Gray, C.H., Bovard, K.R., and Dawson, Michael, 2002). The nearest active fault is the Elsinore Fault Zone (Glen Ivy segment), located approximately 6 miles southwest of the site; no known active or potentially active faults are mapped crossing or projecting toward the Norco–Corona area. A Geologic Map and Regional Fault Map are presented as Figures 3 and 4, respectively.

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the earth materials encountered on the site is presented in the following sections. Based on the field exploration and observations, the site contains localized deposits of artificial fill associated with previous site improvements, underlain by older alluvium and granitic bedrock. A Geologic Map is presented on Figure 3. Plates 2A and 2B provide cross section depicting the orientation of the geologic units encountered in the borings. The location of the cross sections are provided on Plate 1.

4.2.1 Artificial Fill (Af)

Artificial fill soils were encountered within all exploratory borings below surface pavements (asphalt concrete and PCC) to depths of about 14.5 to 20 feet. The artificial fill was likely placed as compacted engineered fill during construction of the current campus. Considering that the original ground elevations were raised to match current campus grades, the compacted fill layer is relatively thick. The fill encountered generally consisted of silty sand, relatively clean sand, clayey sands and silty sandy clays (SM, SP, SC and CL soil types based upon the Unified Soil Classification System). These fill soils were found to be compact to very compact.

4.2.2 Older Alluvium (Qof)

Older alluvial deposits, consisting of interbedded silty sands, sands with silts and clean sands (SM and SP soil types) were encountered below the artificial fill soils to a maximum depth of approximately 39.5 feet. As encountered in the borings, the older alluvium was generally various shades of red brown to brown in color, and generally moist, and ranged from dense to very dense.

4.2.3 Granitic Bedrock (Kmp)

Granitic bedrock was encountered in Boring B-3 at a depth of approximately 39.5 feet below grade and extended to the maximum explored depth of approximately 51.5 feet. This bedrock is geologically mapped as consist of micropegmatite granite of Gavilan Ring Complex (Morton, et al., 2002). As encountered in boring B-3, the bedrock excavates as fine- to coarse-grained sand (decomposed granite), gray-brown in color with white powdery fines. The bedrock encountered was found to be slightly to moderately weathered.

Based on the laboratory test results, the near surface soils have a "Very Low" (0-20) expansion index when tested in accordance with ASTM D 4829. Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). The site soils are considered to have a very low potential for hydro-consolidation (settlement upon wetting with or without additional loading). The upper site soils, when recompacted, are anticipated to have fair pavement support capacities. The test results are provided in Appendix B.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during the site exploration. If encountered during earthwork construction, surface water on this site would likely be the result of precipitation or possibly some minor surface run-off from surrounding areas. Overall site drainage is generally in a southwesterly direction, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered in any of the borings excavated onsite to a maximum depth of 51.5 feet below the existing ground surface. Review of a previous geotechnical investigation prepared by Leighton Consulting, Inc. (Project No. 602776-001, dated December 23, 2009, titled "Geotechnical Investigation, Proposed Facilities Operation Center, Norco Campus of Riverside Community College") indicated that the underlying granitic bedrock is generally dense and not considered to be water-bearing. The Riverside County Geologic Hazard Map (2004) shows historically shallowest groundwater levels within the valley floor approximately 0.5 mile west of the site at a depth of about 30 feet below ground surface Lake Norconian, located roughly 1,500 feet north of the site, maintains a surface water elevation of approximately 610 feet above mean sea level (about 30 feet below current site elevations).

Based on the results of GeoTek's field exploration, review of the referenced report, and consideration of regional geomorphology and geology, groundwater is not anticipated to adversely affect the proposed improvements.

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 fault system. The site is in a seismically active region. The site is not currently located within a State of California "Alquist-Priolo" Earthquake Fault Zone. The site has not been evaluated by the State of California for liquefaction or seismic induced landslide hazards. The County of Riverside has identified the project site as not in a fault zone or a fault line and in a "high" liquefaction potential and "susceptible" subsidence zone. As previously discussed, historic high groundwater level is estimated to occur at a depth of approximately 30 feet below the existing ground surface.

The nearest active fault to the project site is the Elsinore Fault Zone (Glen Ivy segment), located approximately 6 miles southwest of the site; no known active or potentially active faults are mapped crossing or projecting toward the Norco–Corona area. A Geologic Map and Regional Fault Map are presented as Figures 3 and 4, respectively.

4.4.1 Seismic Design Parameters

The project site is centrally located at approximately 33.9168 degrees Latitude and -117.5691 degrees Longitude (World Geodetic System of 1984). A Ground Motion Hazard Analysis report for the project site was prepared by Terra Geosciences (Terra, 2025) dated November 7, 2025 (Project No. 254169- 1). The purpose of this study was to evaluate the site-specific ground motion parameters to aid in the seismic design for this project, based on the 2025 California Building Code (CBC). This study included performing a seismic shear-wave survey study for determining the Site Classification and V_{100} input values for the seismic design parameter determination. This report is included in Appendix D of this report.

The results of the site-specific seismic ground motion analysis are presented in the following table. More detailed information and analysis are presented in the referenced report dated November 7, 2025.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	2.060g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.650g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.170g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.810g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	1.750g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	1.220g
T_L	8 seconds
MCE_G PGA	0.76g
Shear-Wave Velocity (V_{100})	1,424.2 ft/sec
Risk Category	III
Site Classification	CD

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 LATERAL SPREAD

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

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

Although the historic high groundwater level at the site is estimated to be approximately 30 feet below the ground surface, the subsurface soils are generally dense to very dense, with granitic bedrock present at a depth of about 40 feet below existing site grades. Significant groundwater accumulation is not anticipated within the bedrock. Based on these site conditions, GeoTek considers the potential for liquefaction at the site to be very low.

Due to the subsurface soils encountered, the relatively level site topography, and the very low potential for site liquefaction, lateral spread should not be a consideration in the design of future project site improvements.

4.6 SEISMIC SETTLEMENTS

Due to the relatively dense/hard nature of the subsurface soil and bedrock encountered in the borings, seismic-induced ("dry-sand") settlement is not anticipated to be a design constraint for this project.

4.7 OTHER SEISMIC HAZARDS

Based on the Riverside County Parcel Report, the site is "susceptible" to subsidence. Any subsidence in the area would likely be regional and not adversely affect the subject development specifically.

Evidence of ancient landslides or slope instability at this site was not observed during the field investigation. Thus, the potential for landslides or seismic-induced slope failure for the project site overall is considered negligible.

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 for specific developments at a later date are incorporated into the design and construction phases of development. Site development, grading and foundation plans should be reviewed by GeoTek when they become available so the recommendations contained in this report can be confirmed.

The on-site soils generally exhibit a "Very Low" (0-20) expansion index. Additional expansion index (EI) and soil corrosion potential testing should be conducted at the completion of earthwork operations to verify these soil characteristics.

The upper approximately 14.5 to 20 feet of the artificial fills were found to be compact and have very low potential for hydro-consolidation. These upper fill soils in their current state are suitable for support of structural foundations.

Removal of the existing site improvements and utility lines is anticipated to disturb the upper soils. Following demolition of existing site improvements, remedial grading, consisting of overexcavation and recompaction of the upper site soils, is recommended to provide a uniform bearing for any proposed structure, or pavement/landscaping.

5.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Norco, County of Riverside, the 2025 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix E 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 E.

5.2.1 Site Clearing and Demolition

In areas of planned grading and improvements, the locations of existing utilities should be determined. Existing utilities should be relocated or abandoned. The site should be cleared of existing structures, pavements, slabs, trees, vegetation and other deleterious materials. Demolition of existing buildings/structures should include removal of all shallow foundations, floor slabs and any below-grade construction. Debris should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill.

Portland Cement Concrete (PCC) generated from the demolition of existing site improvements may be incorporated into site fills provided the following guidelines are implemented: 1) concrete should be free of rebar or other deleterious materials and should be broken down to a maximum dimension of six (6) inches; 2) concrete should not be placed within three (3) feet of finish grade in the building pad areas or within one (1) foot of subgrade elevations in the street/drive areas; 3) concrete should be distributed in the fill and should not be "nested" or placed in concentrated pockets.

Asphalt concrete (AC) should not be used in fill for building pads or landscape areas. This material may be used in pavement areas of the project or should be removed from the site.

Alternatively, existing PCC concrete, AC and aggregate base may be pulverized and reused as Crushed Miscellaneous Base (CMB) in new pavement sections. These pulverized materials should not contain any detrimental or deleterious materials (reinforcing steel, nails, metal, glass, plastic rubber, wood, organic matter, friable materials, elongated or laminated pieces, etc.). Any materials to be used as CMB should be tested to confirm conformance with appropriate Standard Specifications for Public Works Construction ("Greenbook") specifications (Section 200-2.4).

5.2.2 Site Preparation - Building Areas

Due to the anticipated disturbance of the upper site soils resulting from demolition of the existing structures and improvements, it is recommended that the site soils be removed beneath the planned building footprints to a depth of at least 3 feet below existing grades, or 2 feet below the base of the proposed foundations, whichever is greater. All disturbed soils and any undocumented fill soils shall be removed as part of remedial grading. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building limits, where obtainable. Removal bottoms should be relatively uniform in soil type and not adversely porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures. Deeper removals may be required and should be determined during site grading by a GeoTek representative.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. All soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative.

Following proof rolling and removal of any unsuitable bearing soil, the exposed subgrade should be scarified to a depth of about 12 inches, be moisture conditioned to at least the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM D-1557 test procedures.

5.2.3 Pavement Areas

Undocumented fill should be removed below proposed pavement areas. If no undocumented fill is encountered or is relatively shallow, the natural soils should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. Finished grade is defined as the top of the subgrade. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to slightly above the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.4 Hardscape Areas

Undocumented fill should be removed below hardscape areas. If no undocumented fill is encountered or is relatively shallow, the natural soils should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to slightly above the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.5 Preparation of Excavation Bottoms

A representative of the geotechnical consultant should observe the bottom of all excavations. Upon approval, the exposed soils should be scarified to a depth of approximately eight inches, moistened to slightly above the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.6 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris, asphaltic concrete, oversize material (greater than 6 inches in maximum dimension) and other deleterious material. Engineered fill should be placed in loose lifts with a thickness of 8 inches or less and moisture conditioned to slightly above the optimum moisture content.

Below and within the proposed structure and pavement/hardscape areas, engineered fill should be moisture conditioned to slightly above the optimum moisture content and be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.7 Excavation Characteristics

Excavation of the on-site soils is expected to be feasible utilizing heavy-duty grading equipment in good operating condition. 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 feet in height.

5.2.8 Shrinkage and Subsidence

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

Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 10 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.10 foot may be anticipated for the underlying soils.

5.2.9 Import Soils

Import soils should have a "Very Low" (0-20) Expansion Index. GeoTek also recommends that the proposed import soils be tested for expansion and corrosivity potential. GeoTek should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation and Building Slab Design

Foundation design criteria for a conventional foundation system, in general conformance with the 2025 CBC, are presented below. Based on laboratory test results, subsequent to earthwork operations it is anticipated that the near-surface soils will have a "Very Low" (0-20) expansion index.

The conventional foundation elements for the proposed building should bear entirely in compacted engineered fill soils. Foundations should be designed in accordance with the 2025 CBC.

Additional 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 the foundation design recommendations is presented in the following table:

DESIGN PARAMETER	"VERY LOW" EXPANSION INDEX
Building Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12 – One- and two-stories 18- Three stories
Minimum Foundation Width (Inches)(1)	12 – One- and two-stories 15- Three stories
Minimum Slab Thickness (actual)	4 inches – actual
Minimum Slab Reinforcing	6" x 6" – W1.4 xW1.4 welded wire fabric or No. 3 bars at 24-inch centers placed in middle of slab
Under Slab Sand and Moisture Retardant Membrane	2 inches of sand(3) overlying moisture vapor retardant membrane (10-mil minimum thickness) overlying 2 inches of sand(3)
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom of the footing
Minimum Tie Beam Dimensions(4)	12 inches in width and 18 inches in depth
Effective Plasticity Index(2)	N/A
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 100 percent of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

(1) Code minimums per Table 1809.7 of the 2025 CBC should be complied with.
(2) Effective Plasticity index should be confirmed via laboratory testing post grading.
(3) Sand should have a minimum Sand Equivalent (SE) of 30.
(4) Final design dimensions to be determined by the structural engineer.

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results additional laboratory testing of samples obtained at/near finish pad grade. It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and foundation reinforcement based on actual loading conditions.

An allowable bearing capacity of 3,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings 18 inches deep and 12 inches wide, and pad footings 24 inches square and 18 inches deep. This bearing capacity includes a minimum factor of safety of 3. This allowable soil bearing capacity may be increased by 400 psf for each additional foot of footing depth and 200 psf for each additional foot of footing width to a maximum value of 4,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).

The recommended allowable bearing capacity is based on an estimated maximum post-construction settlement of 1-inch. Differential settlement of about one-half of the total settlement over a horizontal distance of 40 feet could result. Seismic-induced settlements are estimated to be minimal. The project structural engineer, foundation engineer and earth retention structure designer should incorporate these settlement estimates into the design, as appropriate.

The passive earth pressure may be computed as an equivalent fluid having an ultimate density of 300 psf per foot of depth, to a maximum earth pressure of 3,600 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements or concrete.

Based upon review, a modulus of subgrade reaction (E_1) of 150 pci may be used in the design of the structure foundation system. It should be noted that this value is based upon standard one foot plate load tests. Depending upon the design methodology and foundation geometry this value may need to be modified.

A grade beam, a minimum of 12 inches wide and 18 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

5.3.2 Moisture Vapor and Retarding System

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 2025 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2025 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

The effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures. 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 6-mil vapor retarder membrane, 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 such as thickness, composition, strength, and permeability to achieve the desired performance level. Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as a flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within buildings be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person should provide recommendations related to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate.

In addition, the recommendations in this report and GeoTek's services in general are not intended to address mold prevention, since GeoTek, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

5.3.3 Miscellaneous Foundation Recommendations

To minimize 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.

5.3.4 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2025 CBC, City of Norco or County of Riverside requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movement and/or differential settlement. If large enough, these movements can compromise the integrity of the improvements.

- The outside top edge of all footings should be set back a minimum of H/3, where H is the slope height, from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.
- The bottom of any proposed foundations should be deepened so as to extend below a 1:1 upward projection from the bottom edge of the nearest excavation and the bottom edge of the closest footing.

5.3.5 Soil Corrosivity

Based on the chemical test results presented in Appendix B, the corrosivity test results indicate that the on-site soils are "mildly corrosive" (12,060 to 13,400 ohm-cm) to buried ferrous metal. This corrosion classification is obtained from "Corrosion Basics: An Introduction," by Pierre R. Roberge, 2nd Edition, 2005. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer.

5.3.6 Soil Sulfate Content

Based on the chemical test results presented in Appendix B, the sulfate test results on two (2) samples obtained from the project site indicate a soluble sulfate content of less than 0.1% by weight. Soluble sulfate contents of this level would be in the range of "negligible" (i.e., "S0" exposure classification) in accordance with ACI 318-19. Based on the test results and Table 19.3.1.1 of ACI 318-19, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.3.7 Drilled Piers

Drilled piers are a design option for project foundations for this project. Drilled piers utilized as structural foundations for solar canopies, shade structures, light poles, etc. should extend to a minimum depth of at least eight (8) feet below finish grade and be a minimum of 18 inches in diameter. The drilled piers can be designed using an average allowable skin friction value of 400 pounds per square foot. The top foot of embedment depths for the drilled pier foundations should be neglected unless the ground surface is covered by concrete or pavement. This value incorporates a safety factor of 3.0 and may be increased by one-third for temporary wind and/or seismic loading. Alternatively, the drilled pier can be designed using an allowable end-bearing capacity of 8,000 psf. End-bearing and skin friction values cannot be combined.

A representative of the geotechnical engineer should be present during installation to verify the embedment depth. The capacities are based on soil strength considerations, and stresses within the piers may impose more severe limitations. The recommendations are for single-acting piers. If necessary, the capacities for closely spaced piers should be reduced in accordance with the efficiency formula provided below:

$$E = 1 - \frac{\varnothing[(n-1)m+(m-1)n]}{90mn}$$

where:

E = Pier efficiency
$\varnothing = \tan^{-1}(d/s)$
d = pier diameter
s = spacing of piers center to center
m = number of rows
n = number of piers in a row

For insertion into the efficiency formula, groups containing three piers should be considered to have two rows and two piers per row.

Groundwater is not anticipated to create issues during site excavation. Predominantly granular soils were encountered on the project site. Appropriate equipment and/or excavation techniques should be utilized on this project. If caving soils are encountered, drilled piers should be provided with temporary steel casing. The temporary casing may be withdrawn as concrete is placed within the drilled pier, keeping a concrete head of at least two feet above the bottom of the casing as it is being removed.

After excavating the drilled piers, the excavations should be observed by the geotechnical consultant of record and concrete placed as quickly as possible to avoid exposure of the foundation sidewalls to wetting and drying. Surface run-off water should be drained away and not be allowed to pond adjacent to the excavations. It is recommended that foundation concrete be placed within 24 hours of the drilled pier excavation being completed.

If it is required that foundation excavations be left open for more than one day, they should be properly covered to prevent personal injury and fall hazards and be protected to reduce evaporation or entry of moisture.

Where drilled piers are to be placed closer than 2.5 diameters center to center, the closely spaced piers should be constructed on different days.

The vertical uplift capacity of the piers due to friction is considered two-thirds (2/3) of the download friction capacity (260 psf), plus the weight of the pier.

Lateral bearing capacity of drilled piers may be computed using any accepted code-allowed "pole" formula. The allowable lateral passive earth pressure for piers may be considered to develop at a rate of 300 pounds per square foot per foot of depth to a maximum value of 3,600 psf. This value may be increased by one-third for wind or seismic loading. However, the allowable passive pressure can be doubled where the piers will not be adversely affected by ½ inch lateral movements at the ground surface. The effective width of the drilled piers can be assumed to be twice the pier diameter for passive resistance calculations.

For piers thus designed and constructed, it is anticipated a maximum settlement of less than one inch will occur.

5.3.8 Soil Corrosivity

Based on the chemical test results presented in Appendix B, the corrosivity test results indicate that the on-site soils are "mildly corrosive" (14,740 ohm-cm) to buried ferrous metal. It is

recommended that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete vertical retaining 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 18 inches into engineered fill. Retaining wall foundations should be designed in accordance with Section 5.3.1 of this report. 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 5.4.3 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 (horizontal: vertical) 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.

5.4.2 Cantilevered Walls

The recommendations presented below are for typical masonry or concrete cantilevered retaining walls. 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. Native soils are not approved to be used as select backfill due to their expansion index.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal: vertical)	Equivalent Fluid Pressure (pcf) Select Backfill*
Level	40
2:1	70

*The design pressures assume the backfill material has an expansion index less than or equal to 20. Backfill zone includes area between back of the wall to a plane (1:1 horizontal: vertical) up from bottom of the wall foundation (on the backside of the wall) to the ground surface.

For walls with a retained height greater than six (6) feet, an incremental seismic pressure should be included in the wall design. Where needed, it is recommended that an incremental seismic load for unrestrained walls with level backfill of $12H^2$ [Units: pounds per lineal foot of wall] should be included into the wall design to account for seismic loading conditions, where H is the retained height of the wall. For unrestrained walls with a retained height greater than six (6) feet with backfill of a 2:1 [horizontal: vertical] gradient, a dynamic load increment of $20H^2$ should be included in the wall design. These incremental seismic loads may be assumed to be applied at a point 1/3H above the base of the wall.

5.4.3 Retaining Wall Backfill and Drainage

Wall backfill should include a minimum one-foot-wide section of ¾- to one-inch clean crushed rock or approved equivalent. The rock should be placed immediately adjacent to the back of the wall and extend up from a backdrain to within approximately 12 inches of finish grade. The portion of the rock opposite the back of the wall adjacent to the soil backfill should be covered with a layer of filter fabric comprised of Mirafi 140N or the equivalent. The upper 12 inches of backfill should consist of compacted on-site soil. Backfill placed within the active zone as defined

by a 1:1 (h:v) projection from the back of the retaining wall footing up to the retained surface behind the wall should consist of "Very Low" expansive soil. The presence of other soils placed within the 1:1 projection will necessitate revision to the parameters provided.

The backfill soil should be placed in lifts no greater than eight inches in thickness, moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC or approved equivalent) in a minimum of one cubic foot per linear foot of ¾-inch or one-inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and the water should be directed to an appropriate disposal area.

As an alternative to the drain, rock and fabric, a pre-manufactured wall drainage product (example: Mira Drain 6000 or approved equivalent) may be used behind the retaining wall. The wall drainage product should extend from the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the wall drainage product.

Walls less than two (2) feet in height do not require backdrains. Walls from two to four feet in height may be drained using localized gravel packs (e.g., approximately 1.5 cubic feet of gravel in a woven plastic bag) behind weep holes at 10 feet maximum spacing. Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

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

5.4.4 Restrained Retaining Walls

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, 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.

5.4.5 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 evident by compression tests 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.

5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

5.5.1 Asphaltic Concrete Pavement

Preliminary pavement thickness design is based on the CalTrans Highway Design Manual (2020). A Traffic Index (TI) of 5.0 was assumed for parking lots and 5.5 for drive lanes. An R-value of 40 was used. The table below provides the street area/usage, associated with TI and the recommended minimum pavement sections for the subject project.

Street Type	Traffic Index	Asphaltic Concrete (inches)	Aggregate Base (inches)
Parking Lots	5.0	3.5	4.0
Parking Lot Drive Lanes	5.5	4.0	5.0

Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

Traffic Indices (TIs) used in the pavement design for the indicated pavement types 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 may 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.

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of Riverside County or City of Norco.

5.5.2 Interlocking Concrete Pavers

Interlocking concrete pavers can be used for this project. The pavers are assumed to be approximately 3.2 inches (80 millimeters) in thickness. Concrete pavers should be underlain by 1.0 inch to 1.5 inches of bedding sand overlying four inches of aggregate base founded on compacted subgrade soils. The aggregate base should be compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

Where possible, the aggregate base should extend beyond the perimeter of the pavers a minimum distance of four inches. The bedding sand should be placed and lightly moistened and compacted. Since this compaction cannot be tested it should be observed by a representative of this firm.

Historically, paver systems have experienced failures in areas where water has degraded the support characteristics of the underlying base and/or subgrade soils. Since paver systems are permeable and allow transmission of water into the underlying materials, it may be prudent to discuss with the paver designer/manufacturer what methods may be employed to address the issue of potential water introduction into the underlying materials. Underdrain systems, local subgrade reinforcement, or additional structural elements such as geotextiles can be considered, particularly in high traffic areas and/or low areas where water will tend to collect. The recommendations of the designer/manufacturer should then be implemented into the design and construction of the paver system.

5.5.3 Portland Cement Concrete (PCC) Pavement

For the proposed pedestrian concrete pavement sections, it is recommended that a minimum of 5.0 inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures and moisture conditioned to at least two percent above the optimum moisture content be utilized. For truck delivery lanes, it is recommended that a minimum of six inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density and moisture conditioned to at least two percent above the optimum moisture content be utilized. This section should also be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches.

Requirements of Section 90 of Caltrans Standard Specifications, and various ACI and ASTM standards regarding mixing and placing concrete should be followed. The PCC pavement should have a minimum modulus of rupture of 500 pounds per square inch, and a minimum 28-day compressive strength of 4,000 pounds per square inch. Concrete should incorporate 1-inch maximum size aggregate and should be proportioned to achieve a maximum slump of four inches.

Instead of increasing the water content, a plasticizing admixture may be utilized to increase the workability of the concrete. The concrete should be properly cured after placement. Concrete should not be placed during hot and windy weather.

Crack control joints should be provided in the transverse direction spaced at horizontal intervals ranging from 24 to 36 times the thickness of the concrete.

5.5.4 Pavement Construction

All pavement installation, including preparation and compaction of subgrade and base material, placement and rolling of asphaltic concrete and placement of concrete pavement, should be done in accordance with the City of Norco or County of Riverside guidelines, and under the observation and testing of GeoTek and a City/County inspector, where required.

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 to two percent above the optimum moisture content, and recompacted to at least 95 percent of the maximum dry density as determined by ASTM D1557 test procedures. 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.

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of Riverside County.

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. The 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 Riverside County specifications, and under the observation and testing of GeoTek and a County/City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

5.6 CONCRETE FLATWORK

5.6.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices utilized in construction.

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

Subgrade soils should be pre-moistened prior to placing concrete. For very low expansive soils, the subgrade soils below exterior slabs and sidewalks should be pre-saturated to a minimum of 100 percent of the optimum moisture content to a depth of 12 inches.

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

5.6.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are hairline to more than 1/8 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 that occur, some cracking will occur despite the best efforts to minimize it. Concrete undergoes chemical processes that are dependent on a wide range of variables which are difficult, at best, to control. Concrete, while seemingly 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 directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

5.7 POST CONSTRUCTION CONSIDERATIONS

5.7.1 Landscape Maintenance and Planting

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

Overwatering should be avoided. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundations. This type of landscaping should be avoided.

5.7.2 Drainage

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 and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

5.8 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

It is recommended that specifications and foundation and grading plans be reviewed by GeoTek prior to construction to check for conformance with the recommendations of this report. It is also recommended 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. 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. It is recommended 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 this evaluation is limited to the boundaries of the subject property. 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 of this evaluation is based on GeoTek's understanding

of the project and geotechnical engineering standards normally used on similar projects in this locality.

7. LIMITATIONS

GeoTek's findings are based on site conditions observed and the stated sources. Thus, GeoTek's comments are professional opinions that are limited to the extent of the available data.

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.

Since GeoTek's recommendations are based on the site conditions observed and encountered, and laboratory testing, GeoTek's 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

American Concrete Institute (ACI), 2006, Publication 302.2R-06, Guide for Concrete Slabs That Receive Moisture Sensitive Flooring Materials.

_____, 2010, Publications 360R-10, Guide to Design of Slabs-On-Ground.

_____, 2018, Publications 318-19, Building Code Requirements of Structural Concrete.

American Society of Civil Engineers (ASCE), 2022, "Minimum Design Loads for Buildings and Other Structures," ASCE/SEI 7-22, First Edition.

Bryant, W.A., and Hart, E.W., 2007, "Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps," California Geological Survey: Special Publication 42.

California Code of Regulations, Title 24, 2025, "California Building Code," 3 volumes.

California Department of Transportation, 2020, "Highway Design Manual," 7th Edition.

California Geological Survey (CGS, formerly referred to as the Division of Mines and Geology), 1977, "Geologic Map of California."

_____, 2008, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117A.

_____, 2018, "Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42 (Revised), California Geological Survey (CGS), 1980, "State of California, Special Studies Zone, Elsinore Quadrangle," dated January 1.

County of Riverside, 2011, "Low Impact Development BMP Design Handbook, Appendix A – Infiltration Testing."

C. W. Jennings and W.A. Bryant, 2010, "Fault Activity Map of California," California Geologic Survey, Geologic Data Map No. 6.

DLR Group, 2025, "Schematic Design Progress, Norco College Library Learning Resource Center and Student Services, 2001 Third St., Norco, California," dated October 13

GeoTek, Inc., In-house proprietary information.

Leighton Consulting, Inc., 2009, "Geotechnical Investigation, Proposed Facilities Operation Center, Norco Campus of Riverside Community College, 2001 Third Street, City of Norco, California," Project No. 602776-001, dated December 23.

OSHPD Seismic Design Maps (https://seismic maps.org/).

Roberge, P. R., 2005, "Handbook of Corrosion Engineering," 2nd Edition.

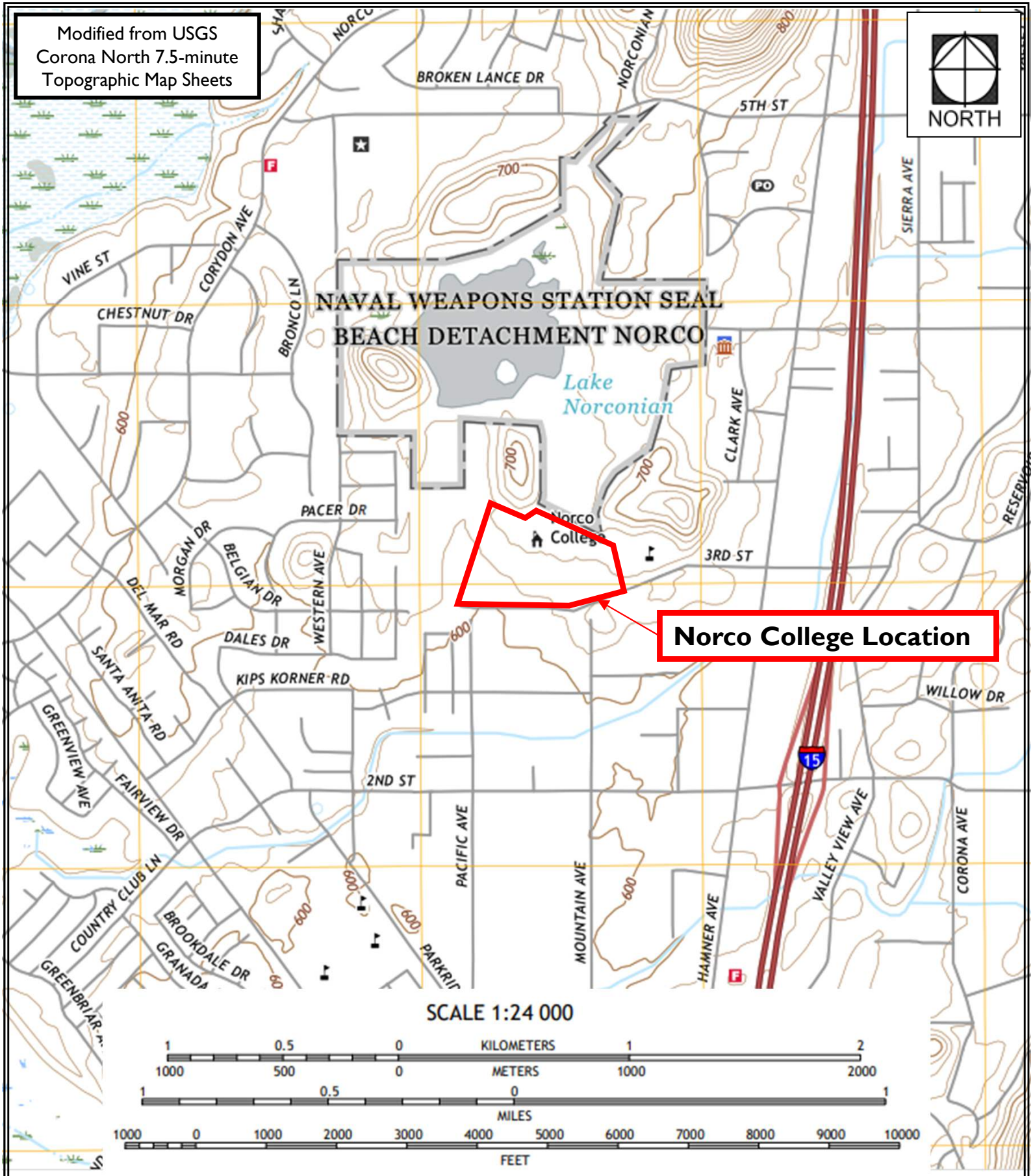
Riverside County GIS website, "Map My County".

_____, 2004, Riverside County Geologic Hazard Map (2004), June.

Seismic Design Values for Buildings (http://geohazards.usgs.gov/designmaps/us/application.php).

Southern California Earthquake Center (SCEC), 1999, Martin, G. R., and Lew, M., ed., "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California,", dated March 1999.

U.S. Geological Survey, 2002, "Geologic Map of the Corona North 7.5' Quadrangle, Riverside and San Bernardino Counties, California," Open-File Report OF-2002-22, scale 1:24,000.



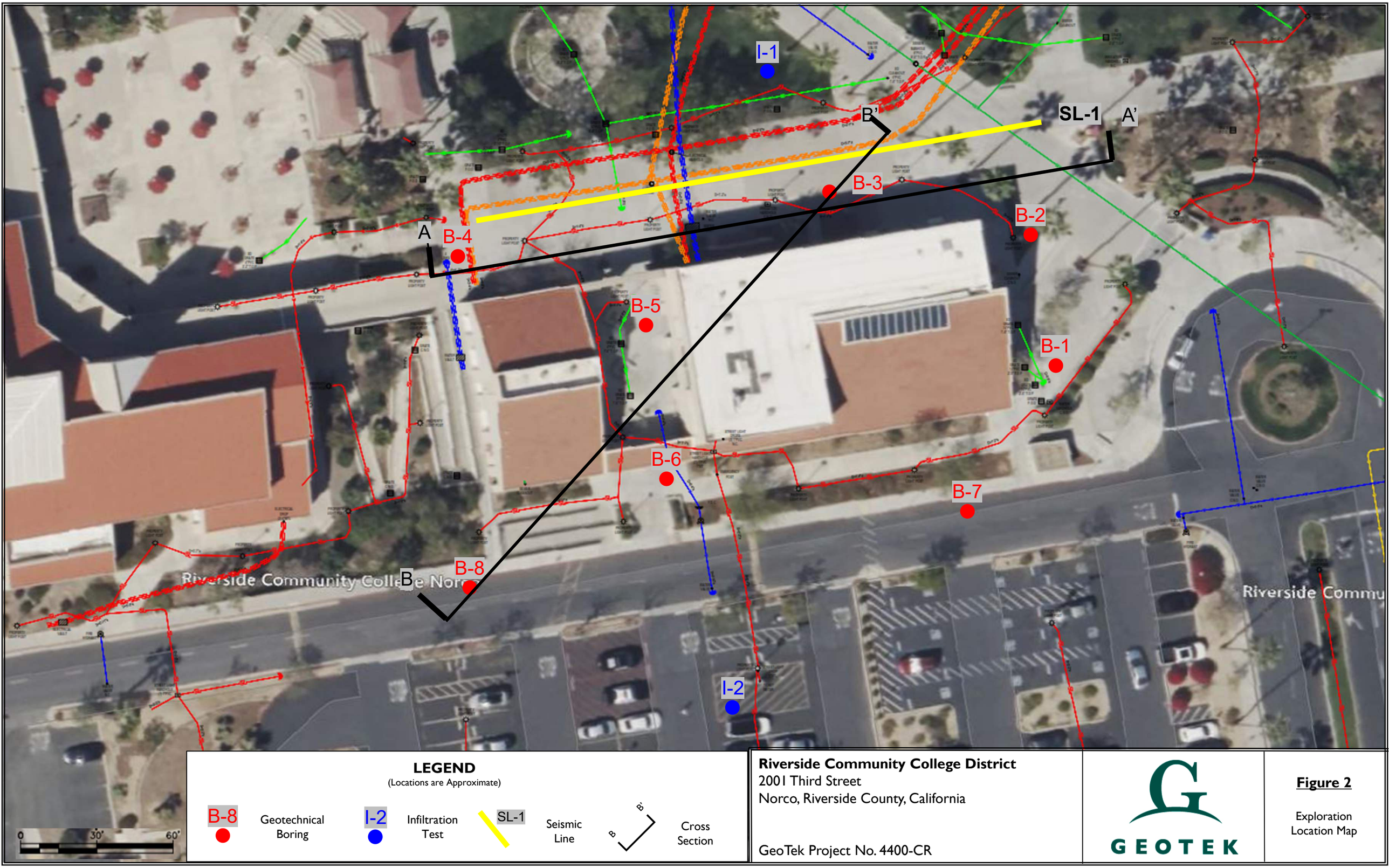
Riverside Community College District
2001 Third Street
Norco, Riverside County, California

GeoTek Project No. 4400-CR

Figure 1

Site Location Map

GEOTEK



LEGEND
(Locations are Approximate)

B-8 Geotechnical Boring

I-2 Infiltration Test

SL-1 Seismic Line

B, B' Cross Section

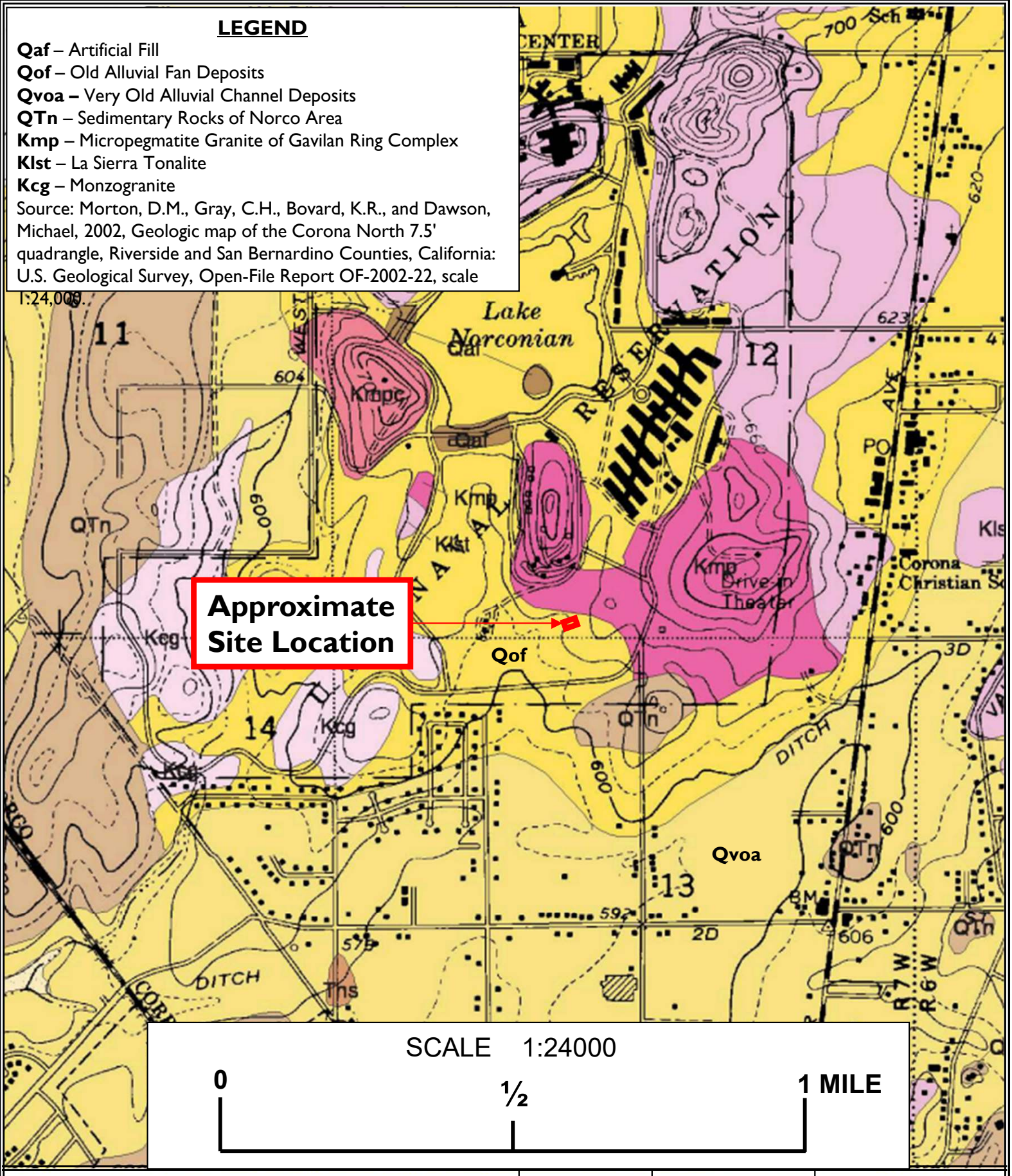
Riverside Community College District
2001 Third Street
Norco, Riverside County, California

GeoTek Project No. 4400-CR

GEOTEK

Figure 2

Exploration Location Map



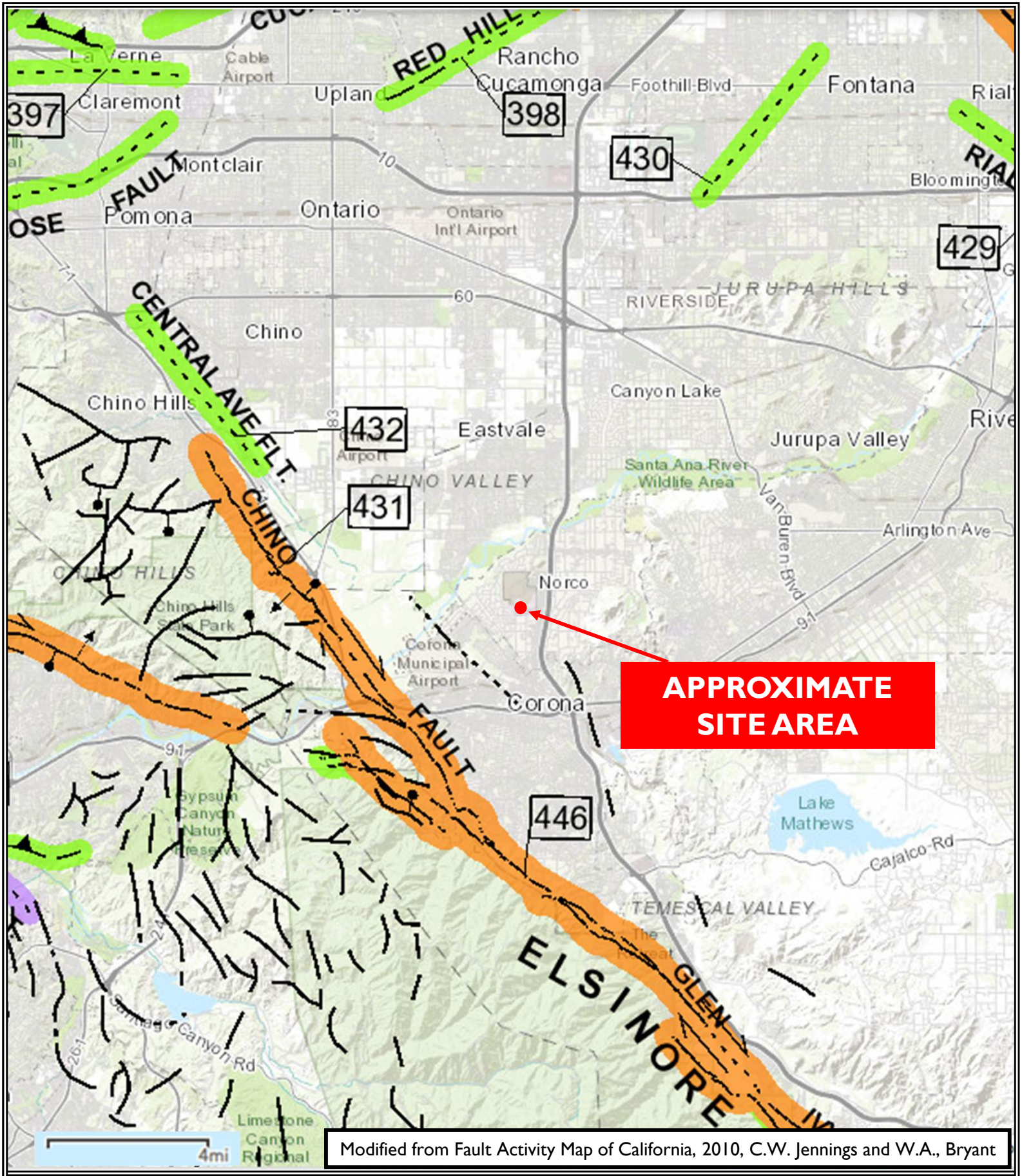
Riverside Community College District
2001 Third Street
Norco, Riverside County, California

Project No. 4400-CR

Figure 3

Geologic Map

GEOTEK



Modified from Fault Activity Map of California, 2010, C.W. Jennings and W.A., Bryant

Riverside Community College District
2001 Third Street
Norco, Riverside County, California

Project No. 4400-CR

NORTH

Figure 4

Regional Fault Map

GEOTEK



Riverside Community College District
2001 Third Street
Norco, Riverside County, California

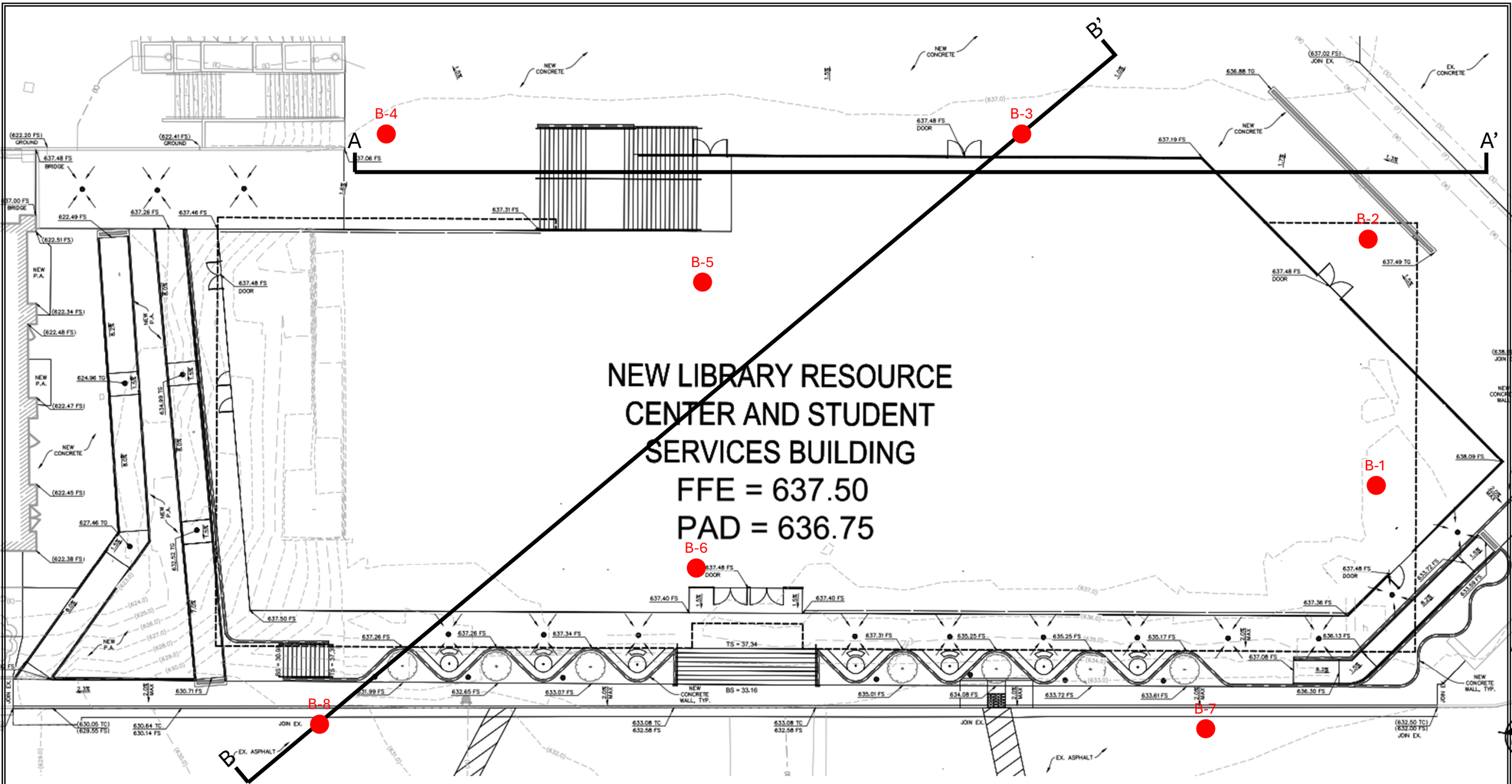
Project No. 4400-CR

NORTH

Figure 5

Site Vicinity Map

GEOTEK



LEGEND

(Locations are Approximate)

B-8 Hollow-stem Boring Location

B — B' Cross Section

Riverside Community College District
2001 Third Street
Norco, Riverside County, California

GeoTek Project No. 4400-CR

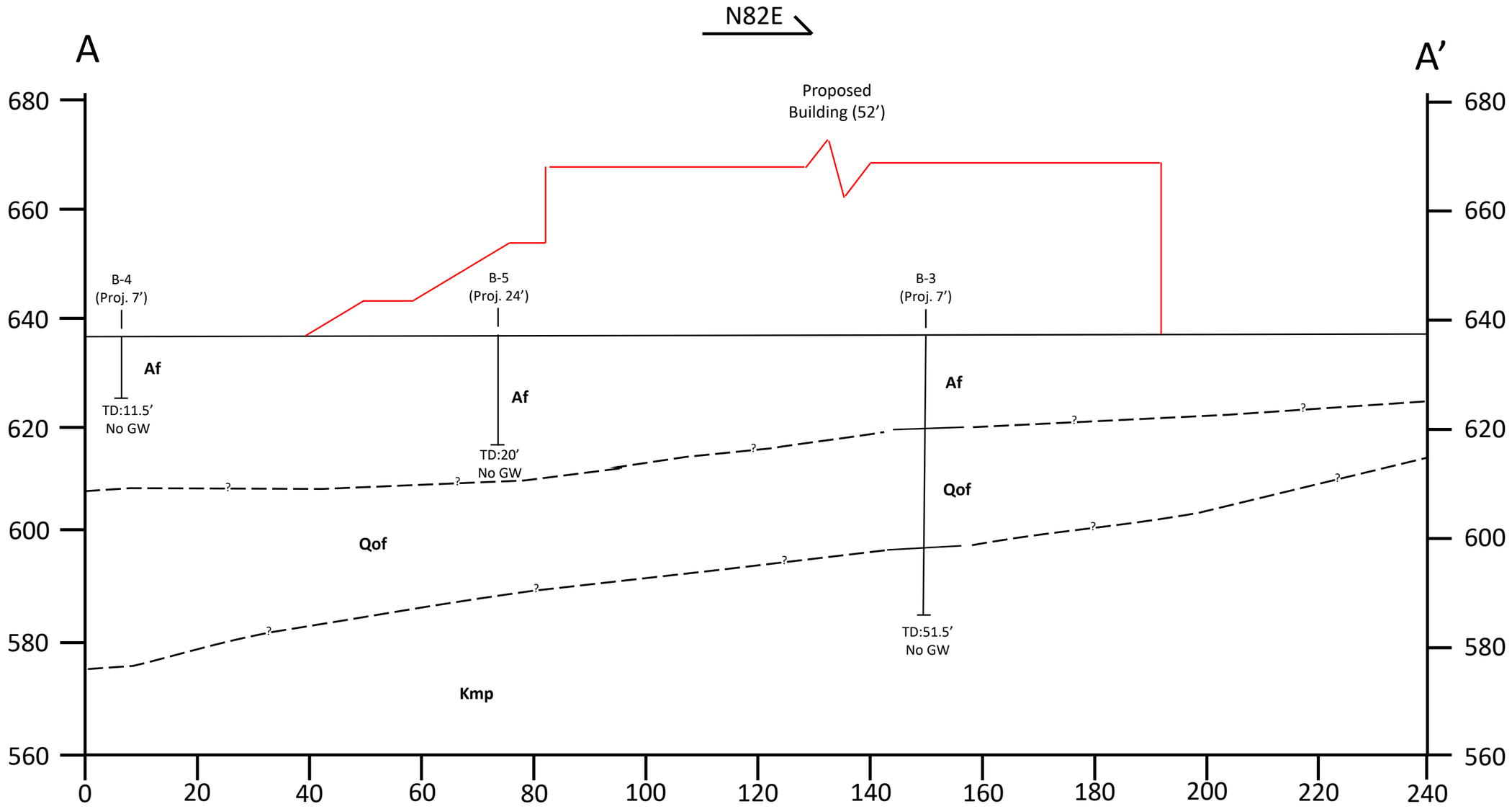
GEOTEK

Plate 1

Cross Section
Location Map

LEGEND

Af Artificial Fill | **Qof** Older Alluvium | **Kmp** Granitic Bedrock | Existing Ground | Proposed Building | Geologic Contact (Dashed where inferred)



Riverside Community College District
2001 Third Street
Norco, Riverside County, California

GeoTek Project No. 4400-CR

SCALE: 1:20

0 feet 20 feet

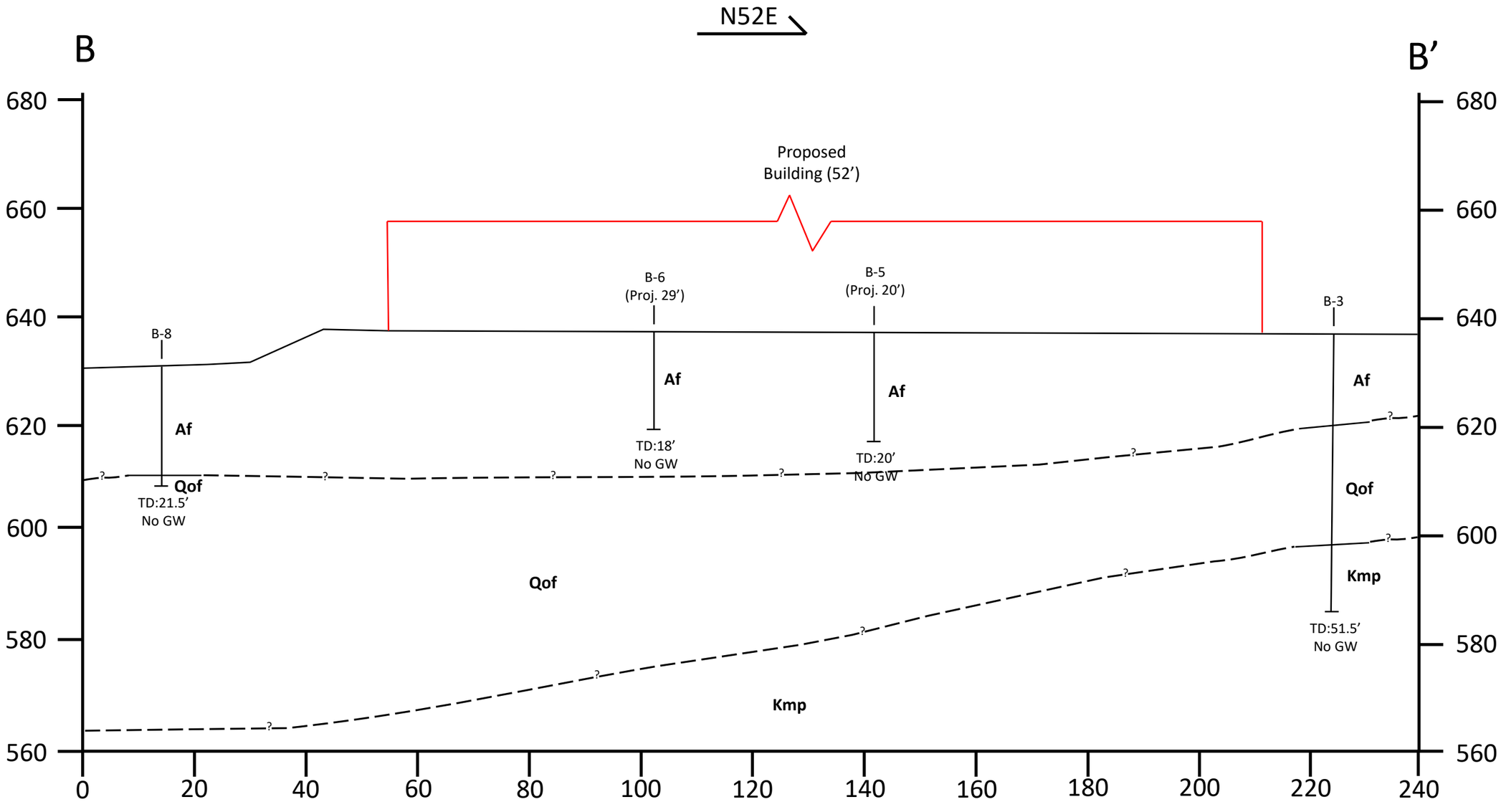
Plate 2A

Cross Section A-A'

GEOTEK

LEGEND

Af Artificial Fill | **Qof** Older Alluvium | **Kmp** Granitic Bedrock | Existing Ground | Proposed Building | Geologic Contact (Dashed where inferred)



Riverside Community College District
2001 Third Street
Norco, Riverside County, California

GeoTek Project No. 4400-CR

SCALE: 1:20

0 feet 20 feet

Plate 2B

Cross Section B-B’

GEOTEK

APPENDIX A

LOGS OF EXPLORATORY BORINGS

Proposed Library Learning Resource Center and Student Services Building
Norco College
Norco, Riverside County, California
Project No. 4400-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)

The Ring sampler is driven into the ground at various depths in accordance with ASTM D 3550 test procedures. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, and split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. Disturbed samples are removed from the sample barrel, sealed in a plastic bag, and transported to the laboratory for testing.

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/TRENCH LOG LEGEND

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

SOILS

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

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C: Contact line	
……..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
———	Thick solid line denotes end of boring/trench

(Additional denotations and symbols are provided on the log of borings/trenches)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
LOCATION:	See Boring Location Map			**DATE:**	10/31/2025

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
0					**Artificial Fill (Af):**			
	Ring	10 24 36		SM	Silty F-m SAND, light brown, moist, dense	14.6	114.7	
5	Ring	17 31 42		SP	F-SAND, light brown, moist, dense, trace silts	8.5	123.5	
	Ring	16 26 32		SC	Clayey F-c SAND, olive-gray brown, moist, dense, trace gravel			
10					**BORING TERMINATED AT 10.0 FEET** No groundwater encountered Set pipe, sock, and gravel			
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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Victor Z.
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Track Rig (GT-16)
LOCATION:	See Boring Location Map			**DATE:**	10/24/2025

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
0					**4" Asphaltic Concrete Over 6" Crushed Miscellaneous Base**			
					Artificial Fill (Af):			
	■	10 8 12		SM	Silty F-c SAND, light brown, moist, medium dense, few gravels	10.5	120.4	
5	■	20 34 50/5		SP	F-c SAND, gray-brown, moist, very dense, few gravels and silts	3.5	132.5	
	■	20		SC	Clayey F-c SAND, gray-brown, very moist, very dense			
	■	50/6		SP	F-c SAND, gray-brown, slightly moist, very dense			
10					**BORING TERMINATED AT 10.0 FEET**			
					No groundwater encountered Set pipe, sock, and gravel			
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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
LOCATION:	See Boring Location Map			**DATE:**	10/31/2025

Boring No.: B-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
0					**6" Concrete**			
					Artificial Fill (Af):			
		17		SM	Silty F-m SAND, brown, moist, very dense	3.6	119.9	MD, SH
		39						Expansion Index=0
		44				7.1	120.2	
		19		SP	F-m SAND, gray-brown, moist, very dense, few silts			
		50/5.5				10.1	118.3	
5		24		SM	Silty F-SAND, brown with olive tint, moist, very dense			
		40						
		50/5				11.3	126.1	
		20		SC	Clayey F-c SAND, gray-brown, very moist, dense			
		22						
		25				12.7	123.4	
		12		SC	Clayey F-c SAND, light brown, moist, dense			
10		25		CL	F-c Sandy CLAY, gray brown, very moist, hard			
		21						
		7			Becomes olive-gray brown, moist, stiff			
		7						
15		10						
					Older Alluvium (Qof):	6.9	122.7	
		14		SM	Silty F-m SAND, red brown, moist, medium dense			
		17						
20		19						
					BORING TERMINATED AT 20.0 FEET			
					No groundwater encountered			
					Boring backfilled with 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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
LOCATION:	See Boring Location Map			**DATE:**	10/31/2025

Boring No.: B-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
0					**Artificial Fill (Af):**			
	■	13 17 31		SM	Silty F-m SAND, brown, slightly moist, dense	4.7	114.1	
	■	19 34 50/5.5			Becomes moist and very dense	7.1	120.9	
5	■	19 34 35			Becomes brown with olive tint and dense	9.6	119.2	
	■	12 19 23		SC	Clayey F-c SAND, olive-gray brown, moist, medium dense	11.1	125.3	
10	■	12 14 12		CL	F-c Sandy CLAY. red-brown to olive-brown, very moist, very stiff	8.9	121.8	
15	■	9 11 12			Becomes red-brown	14.5	114.7	
20	■	17 21 32			Becomes hard			
25					**BORING TERMINATED AT 21.5 FEET** No groundwater encountered Boring backfilled with grout			
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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
LOCATION:	See Boring Location Map			**DATE:**	10/31/2025

Boring No.: B-3

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
0					**5" Concrete with Rebar**			
					Artificial Fill (Af):	7.6	118.9	
		6 28 32		SM	Silty F-m SAND, brown, moist, dense			SR
		16 32 41			Becomes olive-gray brown	10.2 10.3	123.3 114.7	
5		19 25 31		SC	Clayey F-c SAND, gray-brown, moist, dense			
		18 41 32			Becomes very moist and trace gravel	6.7 12.1	129.4 124.2	SH
10		7 9 13		CL	F-c Sandy CLAY, olive-gray brown, moist, stiff to very stiff			
15		8 18 22		SC	Clayey F-c SAND, brown with red tint, moist, medium dense			
					Older Alluvium (Qof):	7.4	124.6	
20		20 24 21		SM	Silty F-SAND, brown, moist, dense			
25		5 20 20				11.2		
30		10 19 25		SM	Silty F-c SAND, brown, moist, dense			

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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
COORDINATES:	See Boring Location Map			**DATE:**	10/31/2025

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	Boring No.: B-3 (continued) / MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium (Qof):			
						23.3		
35		12 16 44		SP	F-c SAND, gray-brown, slightly moist, very dense			
40		50/4			**Granitic Bedrock (Kmp):** Excavates as F-c SAND, gray-brown with white powder, slightly moist, very dense slightly to moderate weathered			
45		50/4.5				3.9		
50		50/6				4.3		
					BORING TERMINATED AT 51.5 FEET Boring backfilled with grout No groundwater encountered			
55								
60								

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; C = Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Victor Z.
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Track Rig (GT-16)
LOCATION:	See Boring Location Map			**DATE:**	10/24/2025

Boring No.: B-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					**6" Concrete**			
					Artificial Fill (Af):			
	■	22 36 38		SP	F-SAND, light brown, slightly moist, dense	9.2	114.6	
	■	31 38 50/6			Becomes gray-brown, very dense, trace gravel	10.7	121.6	
5						12.7	128.5	
	■	16 16 30		SC	Clayey F-c SAND, gray to light brown, moist, dense, trace gravels	13.2	119.9	
	■	20 20 15			Becomes olive to gray-brown, medium dense, no trace gravels	9.3	123.3	
10	■	12 17 18						
					BORING TERMINATED AT 11.5 FEET			
					No groundwater encountered Boring backfilled with 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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Victor Z.
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Track Rig (GT-16)
LOCATION:	See Boring Location Map			**DATE:**	10/24/2025

Boring No.: B-5

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0					**6" Concrete**			
					Artificial Fill (Af):	5.4	117.6	
		8						
		15		SP	F-m SAND, light brown, slightly moist, medium dense			MD, SH
		13						Expansion Index=1
						7.5	115.1	
		7						
		5						
		8						
5						14.5	120.7	
		5		SP	F-SAND, gray-brown, very moist, medium dense			
		8						
		10						
						7.2	122.2	
		14		SC	Clayey F-c SAND, gray to light brown, moist, medium dense			
		16						
		12						
						11.7	117.4	Collapse
		50/6			Becomes very dense			
10								
						10.3	125.6	
		6			Becomes reddish to gray-brown, medium dense			
		6						
15		21						
						12.1	121.4	
		9			Becomes olive to gray-brown			
		17						
20		22						
					BORING TERMINATED AT 20.0 FEET			
					No groundwater encountered			
					Boring backfilled with 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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Victor Z.
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Track Rig (GT-16)
LOCATION:	See Boring Location Map			**DATE:**	10/24/2025

Boring No.: B-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					**6" Concrete**			
					Artificial Fill (Af):			
						3.3	106.4	
		8		SP	F-c SAND, light brown, slightly moist, loose, trace gravel			
		5						
		4						
						5.4	106.5	
		8						
5		4						
		8						
						5.7	122.5	
		12			Becomes reddish olive-brown, moist, medium dense, trace clay			
		20						
		22						
						9.9	128.1	
		20		SM	Silty F-c SAND, dark brown, very moist, dense, trace clay and gravel			
		35						
		35						
						10.3	129.6	
10		14		SC	Clayey F-c SAND, olive-brown, moist, medium dense, trace gravel			
		20						
		24						
15		5						
		9						
		18				14.9		
		5		SC	Clayey F-c SAND, olive-brown, moist, medium dense			
		7						
		10						
					BORING TERMINATED AT 18.0 FEET			
20					No groundwater encountered Boring backfilled with 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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Carlos
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Truck Rig (CME-75)
LOCATION:	See Boring Location Map			**DATE:**	10/31/2025

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
0					**4" Asphaltic Concrete Over 5" Crushed Miscellaneous Base**			
					Artificial Fill (Af):			
		14 21 22		SM	Silty F-m SAND, dark brown, moist, medium dense	8.6	125.5	
		9 12 17		SM	Silty F-SAND, gray-brown, very moist, medium dense	15.4	115.1	
5						15.7	118.7	Collapse
		8 9 18		SC	Clayey F-c SAND, olive gray-brown, moist, medium dense			
		9 11 21			Becomes gray-brown to red-brown	12.1	124.0	
10						14.2	119	
		13 17 19		CL	F-c sandy CLAY, red brown with olive tint, moist, very stiff			
15		17 35 45		SP	**Older Alluvium (Qof):** F-m SAND, olive-gray, moist very dense, few silts	8.5	124.1	
					BORING TERMINATED AT 16.5 FEET No groundwater encountered Boring backfilled with 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; C= Collapse; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	RCCD	**DRILLER:**	2R Drilling	**LOGGED BY:**	LN
PROJECT NAME:	Norco College	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Victor Z.
PROJECT NO.:	4400-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	Track Rig (GT-16)
LOCATION:	See Boring Location Map			**DATE:**	10/24/2025

Boring No.: B-8

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
0					**5" Asphaltic Concrete Over 4" Crushed Miscellaneous Base**			
					Artificial Fill (Af):			
		20 40 21		SM	Silty F-SAND, brown with grayish tint, very moist, dense, trace gravel	7.3	117.8	SR
		30 34 38		SM	Silty F-m SAND, dark brown with grayish tint, moist, dense, few gravel	9.6	121.2	
5						12.9	114.6	
		13 14 18		SC	Clayey F-m SAND, gray-brown, very moist, medium dense			
		8 14 30			Becomes reddish gray-brown, trace gravel	9.4	125.1	
10		14 21 30			Becomes dense, trace organics, no gravel			
15		20 20 31			trace gravel	10.4	126	
20		18 32 50/6		SP	**Older Alluvium (Qof):** F-m SAND, gray-brown, moist, very dense	6.6	124.2	
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with 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; C= Collapse; MD = Maximum Density

APPENDIX B

LABORATORY TEST RESULTS

Proposed Library Learning Resource Center and Student Services Building
Norco College
Norco, Riverside County, California
Project No. 4400-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the boring logs in Appendix A.

Direct Shear
Shear testing was performed on an in-situ sample and on remolded samples in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rates of deformation were approximately 0.01 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The results of the testing are presented graphically in Appendix B.

Expansion Index
The expansion index of the soils was determined by performing expansion index testing on two (2) samples obtained from the field exploration in general accordance with ASTM D 4829. The results of the testing are provided below and in Appendix B:

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	0-5	Silty Sand (SM)	0	Very Low
B-5	0-5	Sand (SP)	1	Very Low

Hydro-Collapse
Selected soil samples was tested in order to evaluate their potential for hydro-collapse. Testing was performed in general accordance with ASTM Test Method D 4546. The results of the testing are presented graphically in Appendix B.

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 Logs of Exploratory Borings at the appropriate sample depths in Appendix A.

Moisture-Density Relationship
Laboratory testing consisting of a moisture-density relationship was performed on two (2) samples obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content were determined in general accordance with ASTM D 1557 test procedures. The results of the testing are provided below and graphically in Appendix B:

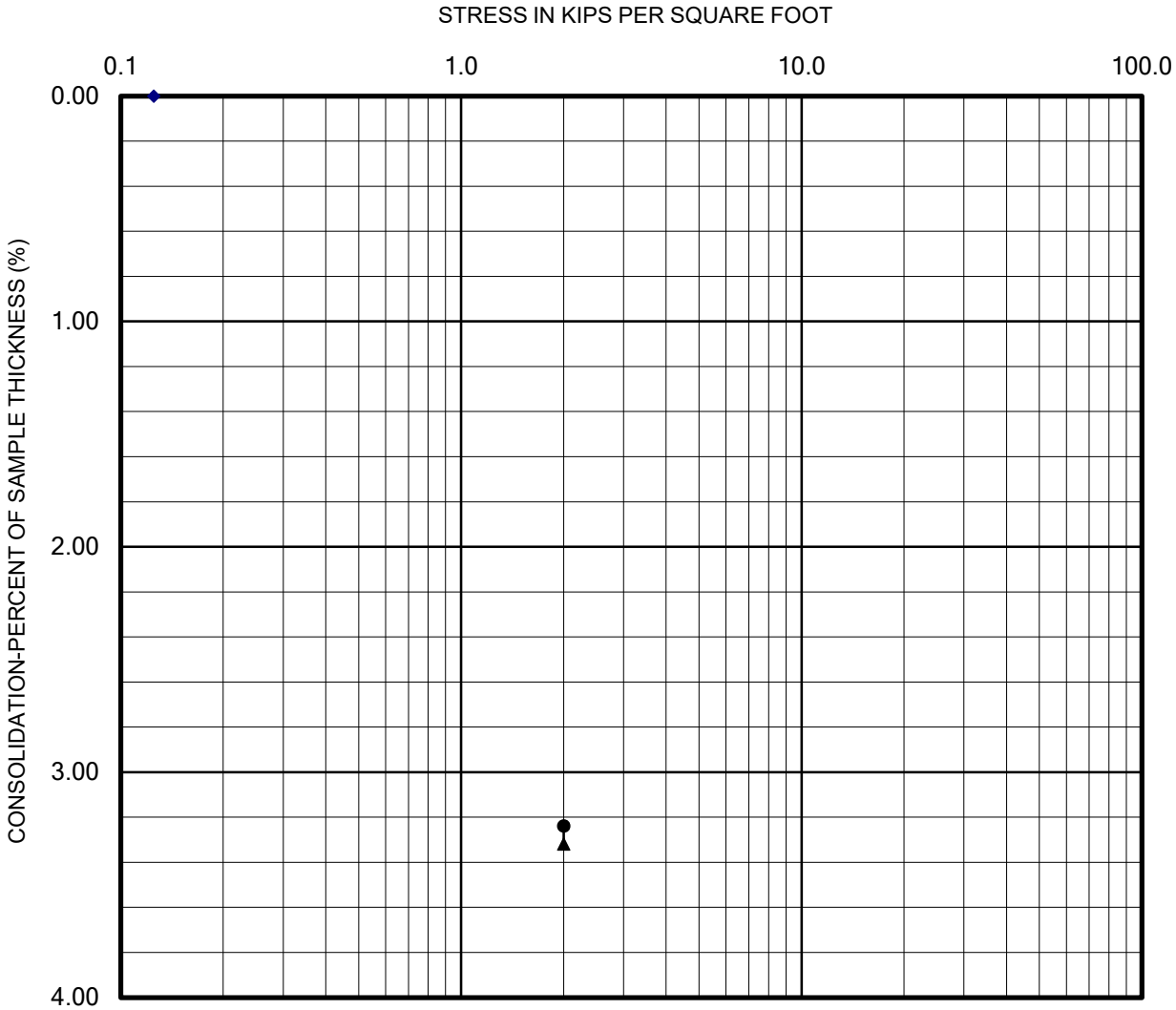
Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	0-5	Fine Sandy Silt (ML)	118.7	10.4
B-5	0-5	Silty Fine Sand (SM)	121.6	7.6

SUMMARY OF LABORATORY TESTING (continued)

Sulfate Content, Resistivity and Chloride Content

Laboratory testing was performed by Project X Corrosion Engineering on two (2) samples obtained from the field exploration in general accordance with ASTM procedures. The testing included pH and water-soluble sulfate content determinations, resistivity and chloride content testing. The results of the testing are provided below, and in Appendix B:

Boring No.	Depth (ft.)	pH ASTM G 51	Chloride ASTM D 4327 (% by weight)	Sulfate ASTM D 516 (% by weight)	Resistivity ASTM G 187 (ohm-cm)
B-3	0-5	8.4	0.0004	0.0014	13,400
B-8	0-5	8.1	0.0035	0.0046	12,060



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

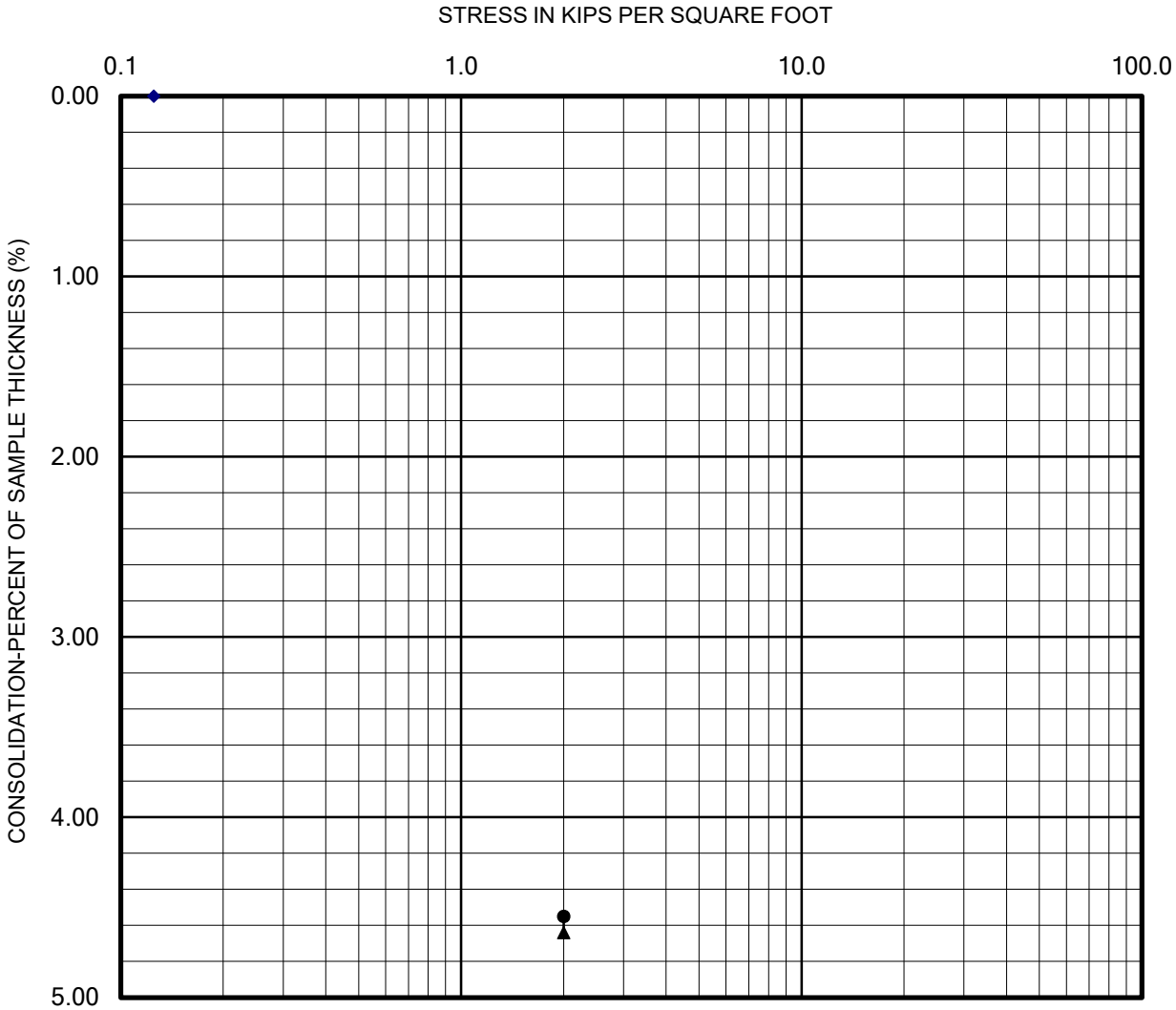
CHECKED BY: EC	Lab: Corona
PROJECT NO.: 4400-CR	Date: 11/7/2025

COLLAPSE REPORT

Sample: B-5 @ 9 Feet

New Library Learning Resource Center & Student Services Building

Plate B-1



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

CHECKED BY: EC	Lab: Corona
PROJECT NO.: 4400-CR	Date: 11/10/2025

COLLAPSE REPORT

Sample: B-7 @ 6 Feet

New Library Learning Resource Center & Student Services Building

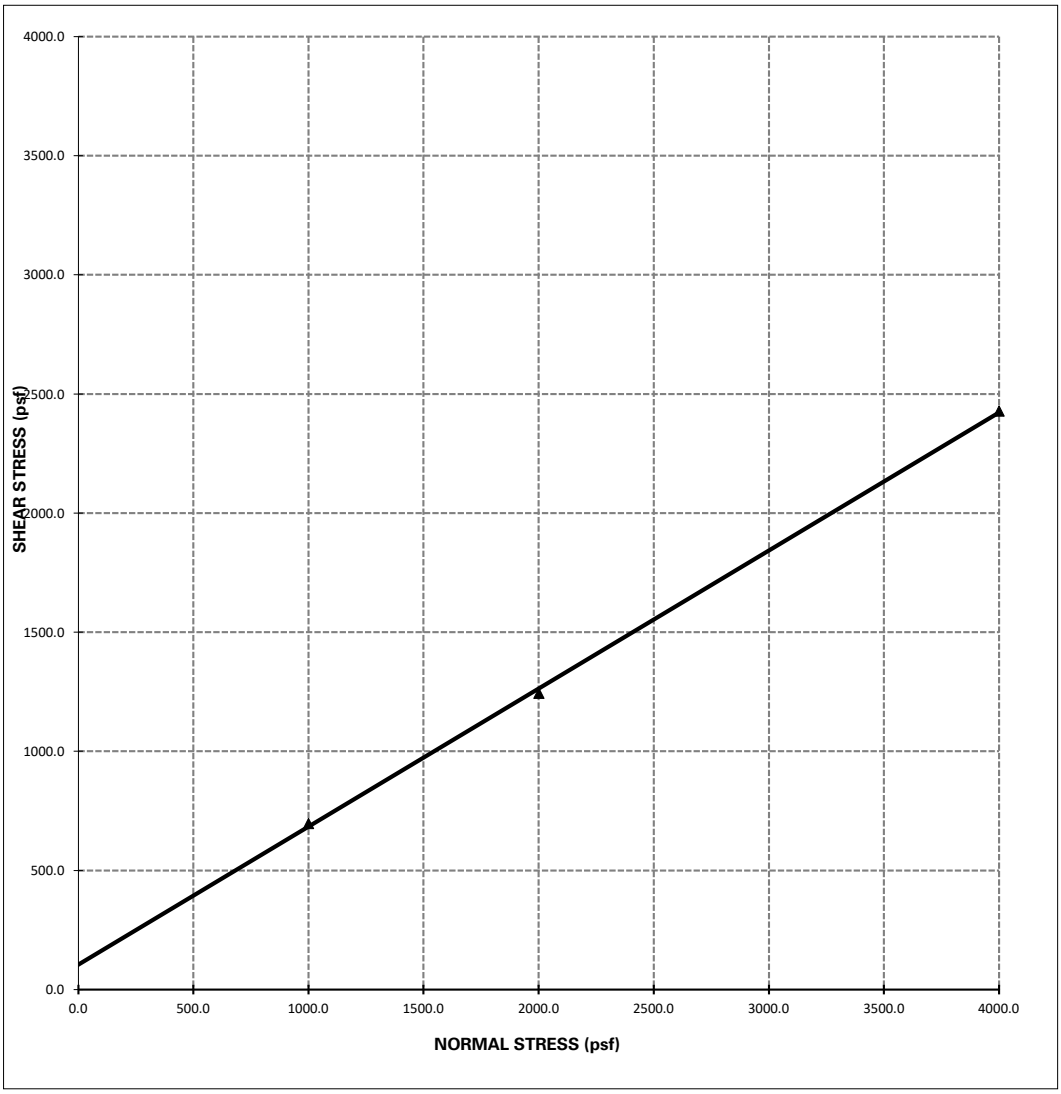
Plate B-2

GEOTEK

DIRECT SHEAR TEST

(ASTM D3080)

Project Name:	New Library Learning Resource	***Sample Location:***	B-1 @ 0-5 Feet
Project Number:	4400-CR	***Date Tested:***	11/10/2025



Shear Strength: **Φ = 30 °, C = 105 psf**

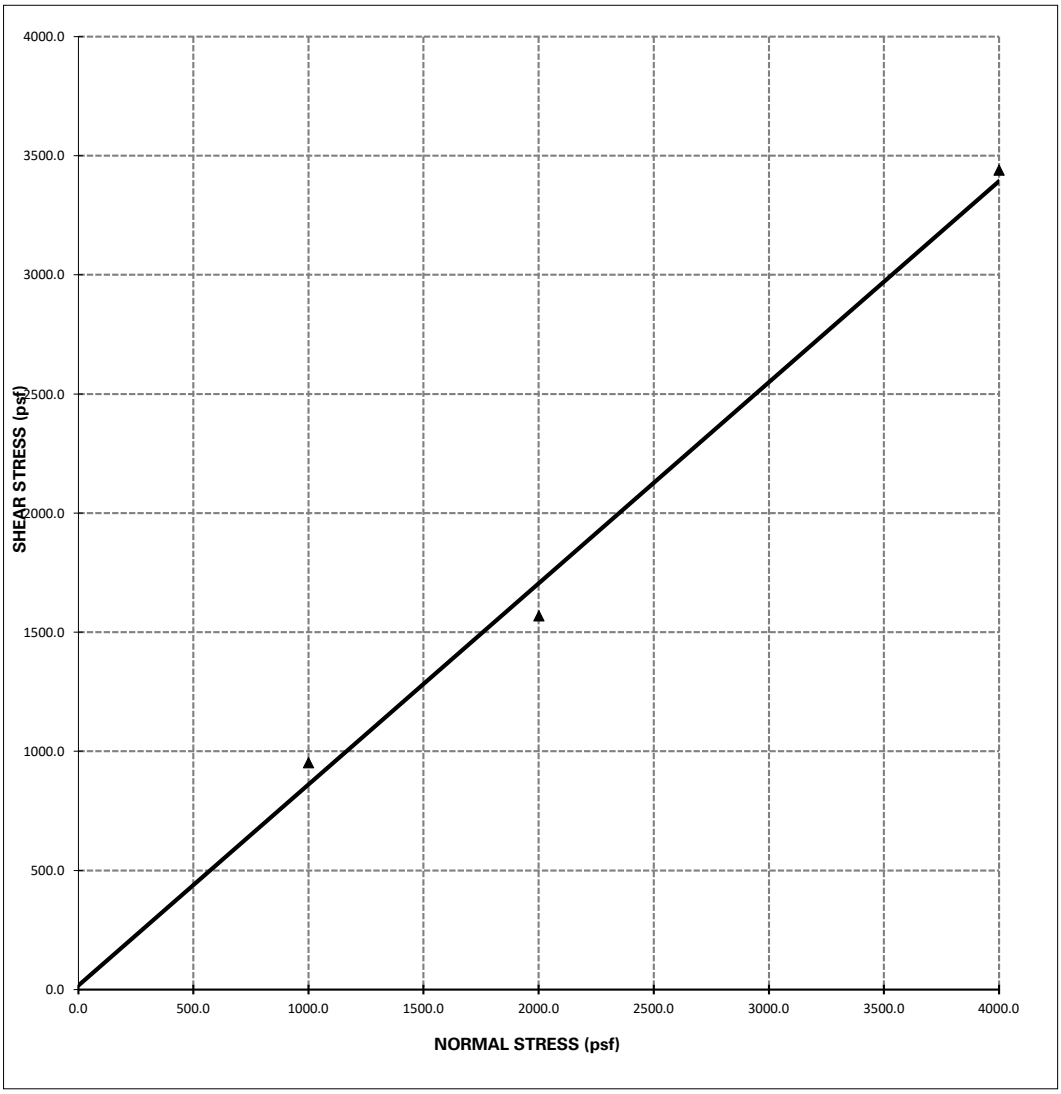
Notes:
1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
2 - The above reflect direct shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.01 in/min.

GEOTEK

DIRECT SHEAR TEST

(ASTM D3080)

Project Name:	New Library Learning Resource	***Sample Location:***	B-3 @ 9 Feet
Project Number:	4400-CR	***Date Tested:***	11/5/2025



Shear Strength: Φ = 40 ° , C = 17 psf

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

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

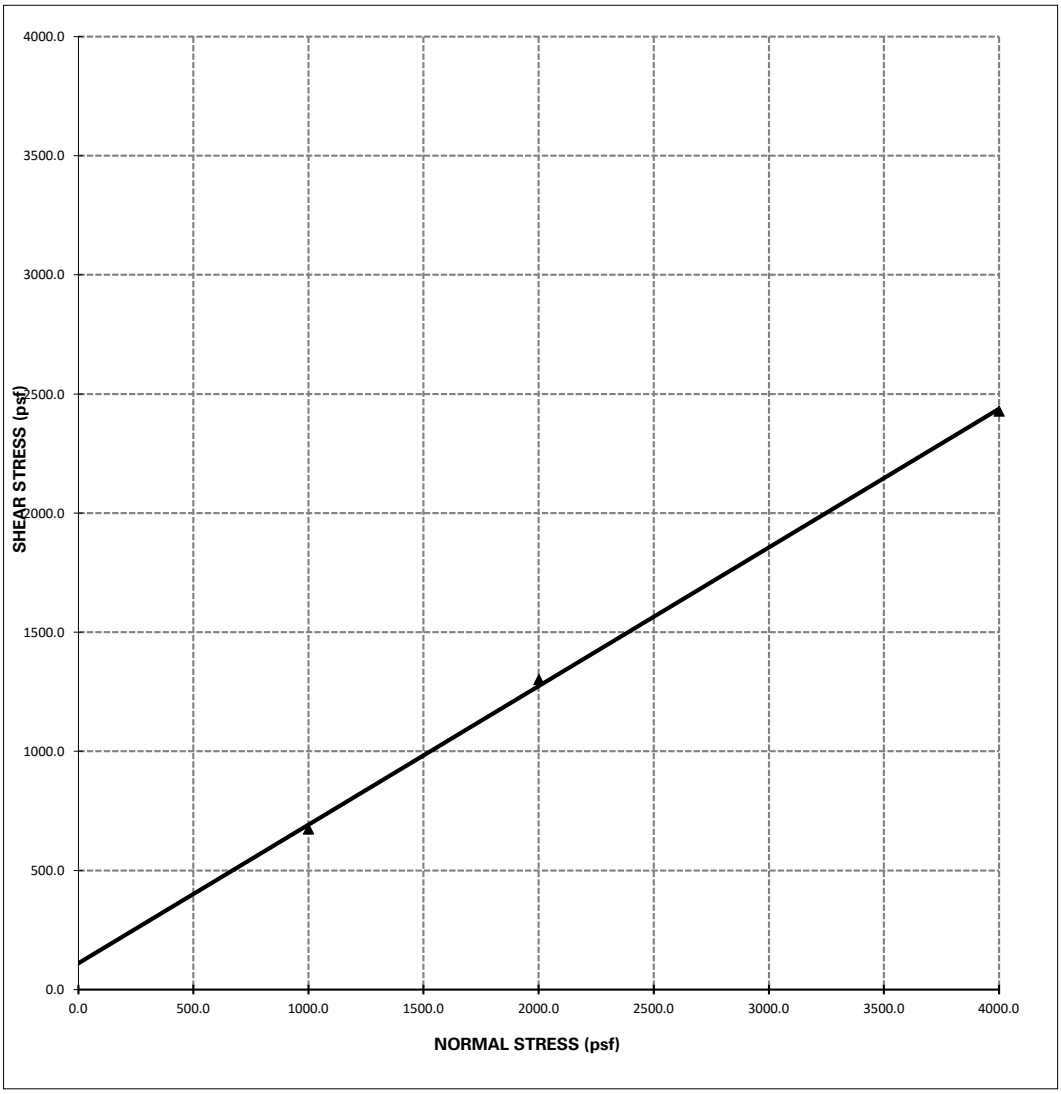
3 - The tests were run at a shear rate of 0.01 in/min.

GEOTEK

DIRECT SHEAR TEST

(ASTM D3080)

Project Name:	New Library Learning Resource	***Sample Location:***	B-5 @ 0-5 Feet
Project Number:	4400-CR	***Date Tested:***	11/10/2025



Shear Strength: **Φ = 30 °, C = 111 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 0.01 in/min.

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Riverside Community College District	***Tested/ Checked By:***	UP Lab No Corona
Project Number:	4400-CR	***Date Tested:***	11/4/2025
Project Location:	New Library Learning Resource Center and Student Se	***Sample Source:***	B-1 @ 0-5 Feet
		Sample Description:	

Ring #:_____ Ring Dia. : 4.01" Ring Ht.1"

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	769.0
Weight of ring (gm)	365.9
Net weight of sample (gm)	**403.1**
Wet Density, lb / ft3 (C*0.3016)	**121.6**
Dry Density, lb / ft3 (D/1.F)	**111.2**

SATURATION DETERMINATION

Moisture Content, %	9.3
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
% Saturation	**48.8**

READINGS

DATE	TIME	READING	
11/4/2025		0.8070	Initial
11/4/2025		0.8070	10 min/Dry
11/5/2025		0.8070	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
786.5	**13.6**

EXPANSION INDEX = 0

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Riverside Community College District	***Tested/ Checked By:***	UP Lab No Corona
Project Number:	4400-CR	***Date Tested:***	11/4/2025
Project Location:	New Library Learning Resource Center and Student Se	***Sample Source:***	B-5 @ 0-5 Feet
		Sample Description:	

Ring #: Ring Dia. : 4.01" Ring Ht.1"

DENSITY DETERMINATION

Weight of compacted sample & ring (gm)	766.0
Weight of ring (gm)	363.3
Net weight of sample (gm)	**402.7**
Wet Density, lb / ft3 (C*0.3016)	**121.5**
Dry Density, lb / ft3 (D/1.F)	**111.1**

SATURATION DETERMINATION

Moisture Content, %	9.3
Specific Gravity, assumed	**2.70**
Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
% Saturation	**48.6**

READINGS

DATE	TIME	READING	
11/4/2025		0.5620	Initial
11/4/2025		0.5620	10 min/Dry
11/5/2025		0.5630	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
784.5	**13.9**

EXPANSION INDEX = 1

GeoTek - Corona
1548 N. Maple St.
Corona, CA 92880
Phone: (951) 710-1160
Fax: (951) 710-1167

Proctor Report

Report No: PTR:25-00193-S01

Client: Riverside Community College District **CC:**
Attn: Bart Doering
Riverside CA 92501

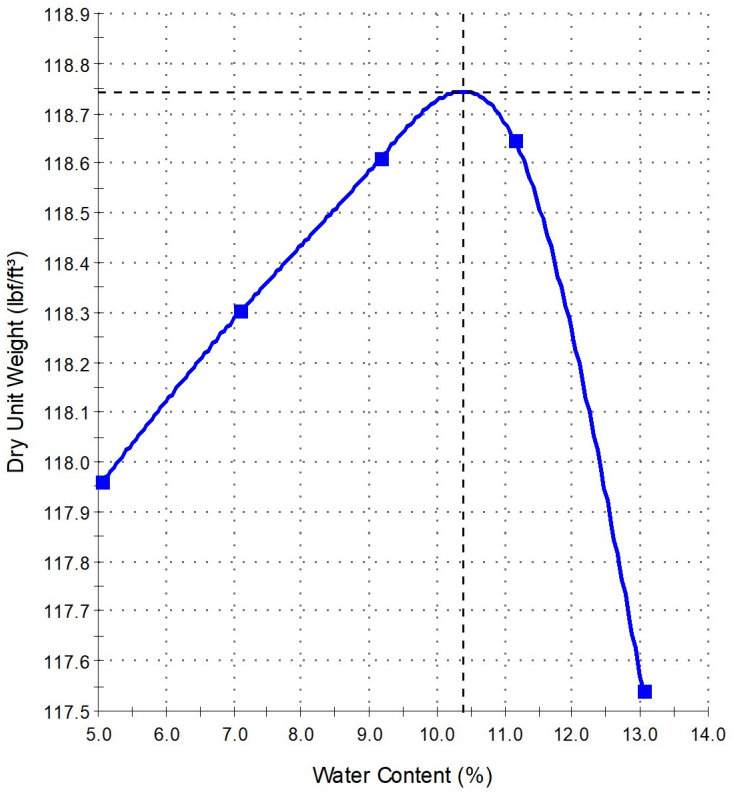
Project: 4400-CR
New Library Learning Resource Center and
Student Services Building

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID: 25-00193-S01 **Date Sampled:** 10/31/2025
Sampled By:
Material: Fine Sandy SILT
Location: B-1 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**118.7**
Optimum Water Content (%):	**10.4**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	0
Passing Sieve No 4 (4.75mm) (%):	100
Tested By:	Eduardo Cuevas
Date Tested:	11/4/2025

Comments

© 2000-2025 QESTLab by SpectraQEST.com

GeoTek - Corona
1548 N. Maple St.
Corona, CA 92880
Phone: (951) 710-1160
Fax: (951) 710-1167

Report No: PTR:25-00193-S02

Proctor Report

Client: Riverside Community College District **CC:**
Attn: Bart Doering
Riverside CA 92501

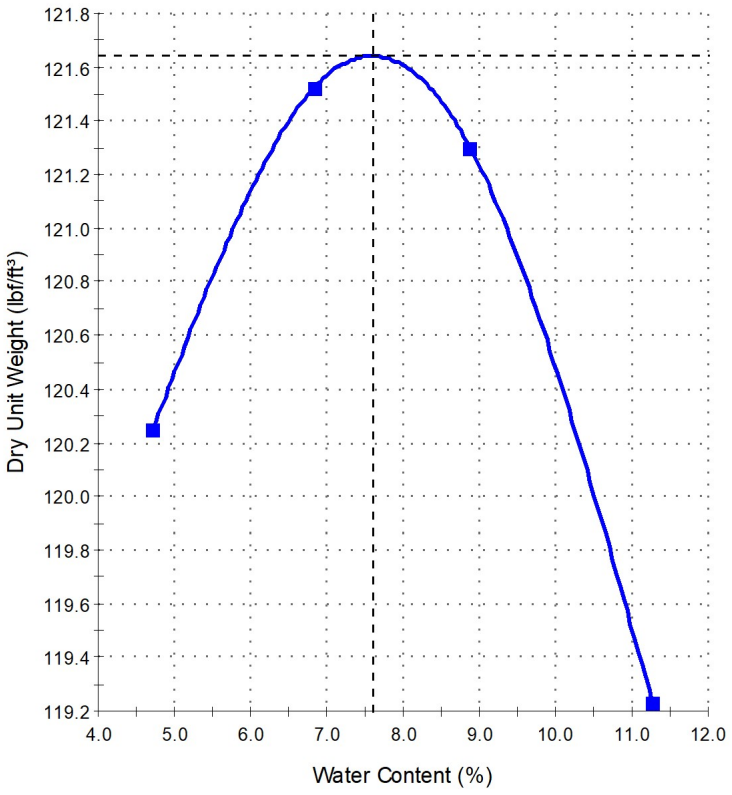
Project: 4400-CR
New Library Learning Resource Center and
Student Services Building

THIS DOCUMENT SHALL NOT BE REPRODUCED EXCEPT IN FULL

Sample Details

Sample ID: 25-00193-S02 **Date Sampled:** 10/31/2025
Sampled By:
Material: Silty Fine SAND
Location: B-5 @ 0-5 Feet

Dry Unit Weight - Water Content Relationship



Test Results

ASTM D 1557

Maximum Dry Unit Weight (lbf/ft³):	**121.6**
Optimum Water Content (%):	**7.6**
Method:	A
Preparation Method:	Moist
Retained Sieve No 4 (4.75mm) (%):	1
Passing Sieve No 4 (4.75mm) (%):	99
Tested By:	Eduardo Cuevas
Date Tested:	11/4/2025

Comments

© 2000-2025 QESTLab by SpectraQEST.com

Results Only Soil Testing for New Library Learning Resource Center & Student Services Building

November 5, 2025

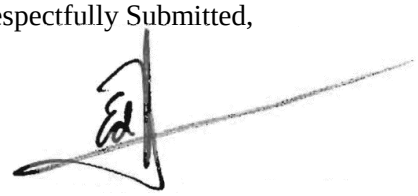
Prepared for:

Eddy Cuevas
GeoTek USA
1548 N. Maple Avenue
Corona, CA 92878
Ecuevas@geotekusa.com, jbrucelas@geotekusa.com

Project X Job#: S251104A
Client Job or PO#: 4400-CR

Prepared by:
D. Bobrova

Respectfully Submitted,



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



Soil Analysis Lab Results

Client: GeoTek USA
Job Name: New Library Learning Resource Center & Student Services Building
Client Job Number: 4400-CR
Project X Job Number: S251104A
November 5, 2025

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{2-}	**Phosphate** PO_4^{3-}
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ω-cm)**	**(Ω-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B-3	0-5	14.1	0.0014	4.2	0.0004	26,130	13,400	8.4	135	1.65	0.9	3.7	ND	25.1	14.0	26.7	71.8	6.6	1.8
B-8	0-5	45.9	0.0046	35.4	0.0035	14,740	12,060	8.1	122	3.09	1.3	2.3	ND	54.3	10.8	38.3	125.9	6.9	4.9

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

For AWWA 105C: 0-3mg/kg sulfide = Negative; 3-6mg/kg = trace; >6mg/kg = Positive

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

APPENDIX C

INFILTRATION TEST DATA
AND PORCHET CALCULATIONS

Proposed Library Learning Resource Center and Student Services Building
Norco College
Norco, Riverside County, California
Project No. 4400-CR

GEOTEK

PERCOLATION DATA SHEET

Project: New Library Learning Resource Center and Student Services **Job No.:** 4400-CR

Test Hole No.: I-1 **Tested By:** JC **Date:** 10/31/2025

Depth of Hole As Drilled: 120" **Before Test:** 120" **After Test:** 120"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Rate (Minutes per Inch)	Comments
1	7:00 AM		120	24				Pre-soaked with 5+ gallons
	7:25 AM	25			23.28	0.72	34.7	of clear water prior to Trials
2	7:26 AM		120	24				
	7:51 AM	25			23.40	0.60	41.7	
1	7:52 AM		120	24				
	8:22 AM	30			23.28	0.72	41.7	
2	8:23 AM		120	24				
	8:53 AM	30			23.40	0.60	50.0	
3	8:54 AM		120	24				
	9:24 AM	30			23.40	0.60	50.0	
4	9:25 AM		120	24				
	9:55 AM	30			23.52	0.48	62.5	
5	9:56 AM		120	24				
	10:26 AM	30			23.52	0.48	62.5	
6	10:27 AM		120	24				
	10:57 AM	30			23.52	0.48	62.5	
7	10:58 AM		120	24				
	11:28 AM	30			23.64	0.36	83.3	
8	11:29 AM		120	24				
	11:59 AM	30			23.64	0.36	83.3	
9	12:00 PM		120	24				
	12:30 PM	30			23.64	0.36	83.3	
10	12:31 PM		120	24				
	1:01 PM	30			23.64	0.36	83.3	
11	1:02 PM		120	24				
	1:32 PM	30			23.64	0.36	83.3	
12	1:33 PM		120	24				
	2:03 PM	30			23.64	0.36	83.3	

GEOTEK

PERCOLATION DATA SHEET

Project: New Library Learning Resource Center and Student Services **Job No.:** 4400-CR

Test Hole No.: I-2 **Tested By:** JC **Date:** 10/24/2025

Depth of Hole As Drilled: 120" **Before Test:** 120" **After Test:** 120"

Reading No.	Time	Time Interval (Min)	Total Depth of Hole (Inches)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Rate (Minutes per Inch)	Comments
1	8:00 AM		120	24				Pre-soaked with 5+ gallons
	8:25 AM	25			23.64	0.36	69.4	of clear water prior to Trials
2	8:26 AM		120	24				
	8:51 AM	25			23.76	0.24	104.2	
1	8:52 AM		120	24				
	9:22 AM	30			23.76	0.24	125.0	
2	9:23 AM		120	24				
	9:53 AM	30			23.88	0.12	250.0	
3	9:54 AM		120	24				
	10:24 AM	30			23.88	0.12	250.0	
4	10:25 AM		120	24				
	10:55 AM	30			23.88	0.12	250.0	
5	10:56 AM		120	24				
	11:26 AM	30			23.88	0.12	250.0	
6	11:27 AM		120	24				
	11:57 AM	30			23.88	0.12	250.0	
7	11:58 AM		120	24				
	12:28 PM	30			23.88	0.12	250.0	
8	12:29 PM		120	24				
	12:59 PM	30			23.88	0.12	250.0	
9	1:00 PM		120	24				
	1:30 PM	30			23.88	0.12	250.0	
10	1:31 PM		120	24				
	2:01 PM	30			23.88	0.12	250.0	
11	2:02 PM		120	24				
	2:32 PM	30			23.88	0.12	250.0	
12	2:33 PM		120	24				
	3:03 PM	30			23.88	0.12	250.0	

GEOTEK

Client:	**RCCD**
Project:	**New Library Learning Resource Center**
Project No:	**4400-CR**
Date:	**10/31/2025**

Boring No. **I-1**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	96.36
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	96
Total Test Hole Depth, D_T =	120

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

$H_O = D_T - D_O$ =	24
$H_F = D_T - D_F$ =	23.64
$\Delta H = \Delta D = H_O - H_F$ =	0.36
$H_{avg} = (H_O+H_F)/2$ =	23.82

I_t = 0.06 **Inches per Hour**

GEOTEK

Client:	**RCCD**
Project:	**New Library Learning Resource Center**
Project No:	**4400-CR**
Date:	**10/24/2025**

Boring No. **I-2**

Percolation to Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	96.12
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	96
Total Test Hole Depth, D_T =	120

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

$H_O = D_T - D_O$ =	24
$H_F = D_T - D_F$ =	23.88
$\Delta H = \Delta D = H_O - H_F$ =	0.12
$H_{avg} = (H_O+H_F)/2$ =	23.94

I_t = 0.02 **Inches per Hour**

GEOTEK

APPENDIX D

GROUND MOTION HAZARD ANALYSIS

Proposed Library Learning Resource Center and Student Services Building
Norco College
Norco, Riverside County, California
Project No. 4400-CR

GEOTEK

TERRA GEOSCIENCES

GROUND-MOTION HAZARD ANALYSIS

LIBRARY LEARNING RESOURCE CENTER

NORCO COMMUNITY COLLEGE

2001 THIRD STREET

NORCO, CALIFORNIA

□r□□□□□N□□□□□□□□□□□□

N□□□□ □□r □□□□□□

Prepared for:

□ □□T□□□In□□
□□□□ N□r□□ M□□□□ □r□□□
□□r□n□□□A □□□□□

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

□ □□T□□□ln□□
□□□□□ N□r□□ M□□□□ □r□□□□
□ □r□n□□□A □□□□□□

A□□□n□□□n□ Mr□D□ni□□□ □i□□ □r□□□□□□□ □□□□□i□□

R□□□rdin□□ □r□□nd□M□□i□n □□□□rd An□□□□i□
□i□r□r□ □□□□rnin□ R□□□□r□□ □ □n□□r
N□r□□ □ □□ □ □ni□□ □ □□□□□□
□□□□ T□ird □r□□□□
N□r□□□□ □i□□rni□
□ □□T□□ □r□□□□N□□□□□□□□R

INTRODUCTION

A□□□□r r□□□□□□□□□i□ ir□ □□□ □r□□□r□d □ □r□□nd□□ □i□n □□i□□ i□ □n□□□□i□r□□r□□r □□□□ □r□□□□□d □i□r□r□ □□□□rnin□ R□□□□r□□ □ □n□□r □□i□din□ □□ □□ □□n□r□□□□d □i□□in □□□□ □□i□in□ □□□□□□□ □□□□ □□□ □□ r□□□r□n□□d □□□□□□□ T□□ □□r□□□□ □□ □□i□ □□□d□ □ □□□ □□ □□□□□□□□□□ □□□□ □i□□□□□□□□i□ □r□□nd □ □i□n □□r□□ □□□r□ □□ □id in □□□□ □□i□□ i□d□□i□n □□r □□i□ □r□□□□□□ □□□□d □n □□□□ □□□□□ □ □i□□rni□ □□i□din□ □□d□ □□□ □□□□ □□r □□r□ in□□□d□d □□r□□r□ in□ □ □□i□□ i□ □□□□□r□□ □□□□ □□□d□ □□r d□□□r□inin□ □□□□ □i□□ □ □□□□i□□□i□n □nd □□□□□ in□□□ □□□□□□□ □□r □□i□ □n□□□□i□□ T□□ □□□□□□ □□ □□r□i□□□ □r□□id□d □□r □□i□ □□□□□□□□i□n in□□d□d □□□□ □□□□□□□in□□

- ***Review of available published and unpublished geologic/seismic data in our files pertinent to the site.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***
- ***Evaluation of the local and regional tectonic setting including performing a site-specific CBC ground motion analysis.***
- ***Preparation of this report presenting our findings, with respect to the seismic design parameters.***

Accompanying Map and Appendices

□□□□□ □□ □□i□□ i□ □in□ □□□□□i□n M□□
A□□□ndi□ A □ □□□□r□□ □□□□ □□r□□□
A□□□ndi□ □ □ □i□□ □□□□i□i□ □r□□nd M□□i□n An□□□□i□
A□□□ndi□ □ □ R□□□r□n□□□

PROJECT SUMMARY

□□□□d □n □□□□ in□r□ □i□n □□□□□ □□□□ □□□□n □r□□id□d□ □□ □nd□r□□□nd □□□□□ □n □□i□in□ □□id□din□ i□ □r□□□□□d □□ d□□ □i□□□d □nd □ n□□ □□□□□□□ □r□□□ □□□□r□ □□□□□ □□□□□□□□ □□□i□n□□□□ □□□□r□ □□□□□□□□r□□□□□r□ □□□nd□□□n□ □□id□din□ □i□□□□ □□n□r□□□□d in i□□ □□□□□□□ F□r □□i□ □r□□□□□□□□□ □□ □□□□□ □□r□□r□ □d □ i□d r□□□□nn□i□□□n □□□□ □□□□□r□□d □□□□ □□□□□□r□□□r□ □□rin□ □□□□□□□□□i□n□ d□rin□ □□□□ i□ □ □□drin□□□r□i□□□d □□r□in□n□□□□i□□□□□ □□□□□□□i□ □nd □□□□□□□□□ni□□□d□□□ in □□r i□□□□□ □□□n□ □i□□ □□r□□r□in□ □ □i□□□□□□□□□i□ □r□□nd □ □□i□n □n□□□□i□ □nd □ □□i□□ i□ □□□□□r□□ □□□□ □□r□□□□

T□ □id in d□□□r□inin□ □□□□ □□i□□i□□ □ □□□□□i□i□□□i□n □□□□□□ □i□□ □□r □r□□nd □ □□i□n □n□□□□i□ □□r□□□□□□□ □ □□i□□ i□ □□□□□r□□ □□□□ □□r□□□ □□in□ □□□□ □ □□i□i□□□□□□□nn□□ □n□□□□i□ □□□ □□r□□□□□ □ □□□□□ □MA□□ □□nd □i□r□□r□□ □r □rr□□ □ □□□□□r□□ □n□□ □MAM□□ □□□□d□ □ □□□ □□r□□r□ □d in □rd□r □□ □□□□□□□□ □□□□ □n□□di□n□i□n□□□□□r□□□□ □□□□□r□□ □□□□ □□□□□□i□□ □r□□□□□r□ □□n□□□□ □□□□ □□□□□□□□□□□d□ □r□□□□□□ □ d□□□□□ □□□□□□□□□□□□□□□ □□□□□□ T□i□ □□r□□□ in□ □ □□□ □□r□□r□ □d □□□n□ □□□□ n□r□□□rn □□ri□ □□□r □□□□□□ □r□□□□□□d n□□ □□id□din□ □□□□ □□□□□n □n □□□□□ □□□□□ i□□ □r□□id□d □□□□ n□□□□□□□□r□ □□r□□□ in□ □□n□□□ □□□n□ □n □□□□□□□i□□□ □r□□□□□□ □ □□□□□□ □□in□ r□□r□□□n□□i□□ □□r □□□□ i□□ d□□□□□□□□□ □n□□□□□□□ □r□□□□□d □□id□din□□

T□□ r□□□□□□□□n□ □□□□□r □ □□□□ □□□□□□i□□ □□□□r□□□ □□i□ □□r□□□ in□ □i□□in □□□□ □□□□□r □□□□ □□□□□ □□□□ □ □□□r□□□ □□ □□□□n □□□□d □□□ □□□□□ d□□□r□in□ □□□□ □i□□ □ □□□□□□i□□□□□□□□n □A□□E□□□□□□□□T□□□□□ □□□□□□□□□□□□□□ □□□□□□□□□□r□□□□□□□□□d□ □r□□□□□□ □ □□□□□□ □□in□ □□□□d □□□r □□□□ □□ in□□□□□□□□□□□□ □□□ □□□□ □i□□□□□□□□□□□□□□□□ □□□ □□i□□ i□ □n□□□□i□□ T□□ d□□□□i□□d r□□□□□□□ □□□□□i□ □□r□□□ □r□ □r□□□□n□□d □i□□in A□□□□ndi□A □□r r□□□r□n□□□□

□ □□□□□□□i□ □ □□□□in□ □□□□□□□ □r□□ □□□ □□□□ M□r□□n □nd □r□□ □□□□□□□□□indi□□□□□□ □□□□□□□□□ □r□□□□□□ □□□d□ □r□□ i□ □ □n□□□d □□□ □□□□□□ □□□ □ id□d□□ □□□□i□□□□□□□n□ □□□□ □□r□i□□□□□di□ □n□□□□□□n□r□□□□ d□□□□ri□□d □□ □□in□ □□□□ □ri□□d □□□□r□n□□□□□□□□ □□□□r□ □□□□r□□ □□nd □r□□□□□ □□r□ in□ □□nd□ □□nd□ □□d □□□□□□i□□□□□□n d□□□□□i□□ □□□i□i□ind□r□□□□d□ □nd□r□in□ □□□□□□□ d□□□□□i□□ □□d□□□□□ □nd □□ □ □□□□□d □□□□□□□□□□□□□□□□□□ □r□□□□□□d □□n□□r□□□i□n □r□□ □r□ □r□□□□□ □d □□□ □□□ □r□□□□□□□□□□□ □□□□ □r□ni□i□ r□□□□□ r□□□□r□d □□□ □□□ Mi□r□□□□□□ □□i□i□ □r□ni□□ □□□□ □□i□□n Rin□ □ □□□ □□□□□□□□□□□□□ i□ □□n□r□□□□ □ in□□□□r□in□d □□□□□□□□r□□i□ □r□ni□□ □□□□in□ □ di□□in□□□□□ □i□r□□□□□□ □□i□i□ □□□□□□r□□ Addi□i□n□□□□□ □□□□□□□ d□□□□□i□□ □□□ □r□i□i□□□ i□□□ □□□ □n□n□□□n □□□□ □□□□i□i□n □nd □□i□n□□□□ □ □□□ □□□□□ □□□ □r□□□□n□□□

T□□□ □□□□r□□i□ □□□□ □□□□□□□i□n □□□□□□ □□i□□i□ □□□□□□r□□□□ □□□□□r□□ □□□□i□□□i□ □in□ □□ □□□□i□ □□□□□n □n □ □□□□□□□□r□d □ □□□□□□□□□ E□r□□ □□□□□□□□□i□ □□□□□□□□ □r□□□n□d □n □□□□ □□i□□□i□ □in□ □□□□□□□i□n M□□□□ □□□□□□□ □□□ Addi□i□n□□□□□ □□□□□□□□□r□□□□i□ □i□□□ □□□ □□□□ □□i□□□i□ □in□ □r□□□□r□□□ □□□□□ □□□□n in□□□d□d □i□□in A□□□□ndi□A □□r □□□□□ □i□□□□□□□nd r□□□□r□n□□ □□□r□□□□□□□

□r□□□□□N□□□□□

□r□□□□□N□□□□□

SEISMIC LINE LOCATION MAP



Base Map: Google™ Earth (2025); Seismic shear-wave survey line (SW-1) shown as blue line.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

T□□ □□nd□□ □n□□□□r□□ i□□ □□□□i□ □□r□□□ □□□□□ □□□□ □□□□□□□□□□□□ E□r□□ i□ □□□□ □□□□ in □ □□i□n □□ □□ri□□□ □□i□□ i□ r□□□□□n i□□□□ T□□□□ r□□□i□□□□ □□n□□□n □ i□r□i□n□ □□ □□□□ E□r□□□□ □□r□□□□ □r□ □□□□d □ i□r□r□□ □r□□□ i□□ □r□ □□r□ □□ □□□□i□□ r□□□□□□□□ □□ □i□d□ □nd □r□ □□n□r□□□ r□□□rr□d □□ □□ □□□□□r□□nd □n i□□□□ □□□□□ □□n□in □□□nd□n□ □□r□□□□ □ □□□□□□ T□□□□ □i□r□r□□ □r□ □r□ □□□□□d □□ □□□□ □□□□ □n □□i□i□□ i□□□□□ □□□□r□□n i□□□□ r□□i□□ □□□□□ri□□ □□□□□ □nd n□□r□□ □□□n□□ □n□n i□□□□ □ind□ □ □□□ □ □i□n□r□in□ □□□ □□□□□ri□ □r□□□□r□□ □□□□□ □ i□□ □□□□ n□□ □□□□□□ □ r□□□rd□d □□ □□□□□□ i□n□□ in□r□ i□n□ A□□□□□□□ □□□□□ i□n□□ □r□ □□n□r□□□ □□r□ □ □□□□ □□□ r□□□rdin□□ □□ □ii□i□n□ □nd □r□□□□in□ □□ □□□□□□□ □□r□□□□ □ □□□□ □□□ r□□□□□ i□ r□□□d □□ □□□ □□□ □□□□□n □□□□□□□ i□ □r□□□d □□i□□ i□ r□□□rdin□ in□r□□ □n□□i□n □nd r□□□n□□d □□□□□□□□d □□□ □□□□r □□□□□ r□□ F□r □□i□ □□□i□□i□n□ □ □ □r□ □ □in□ □□n□□rn□d □i□□ □□□ R□□□i□□ □ □□□ □□r□in □□ □□□□ □□i□□ i□ □i□n □□□□□□ □i□□ i□ □□□□ r□□□rr□d □□ □□ □□r□□nd r□□□□in□□ □□□ R□□□i□□ □ □□□□ i□ □□□□ d□□ in□n □□□□ □□n□n □□□□r□□nd r□□□

F□r □□□ □□r□□□□□ □□ □□i□ □□□d□□ □□□r□ □r□ □□□ □ □□□□ □□□□ □□□ □□r□□□□ □ □□□□ □□r□ r□□□rd□d□□n□ □□in□ □□□□i□□□ □nd □□□ □□□□r □□in□ □□□□□i□□□ A□i□□ □ □□n□ □□□□□□i□□ i□ □n□r□□ i□ in□□n□i□n□□□ □□n□r□□d □□□ □□□□i□i□ □□□□□□□□ r□□□i□□ □□ □□□ □□r□□ □□r□□d □nd r□□□rdin□ □□□in□ □ □□n □□□ □□□r□□ □n□r□□ i□ i□ □□r□□d in□ □□□ □r□□nd i□□□□ MA□□ □□r□□□ □□□□□ni□□□□□ □ □□□i□□ □□r□□□in□□□□ □□□d □□ i□r□r□□ □r □□r□□□in□□□i□ □ □□r□ □□□□ □□i□□ □□r□□□ r□□□rd□ □□ □i□n □□□□□□r□□nd □i□r□i□n□ i□□□□MAM □□r□□□ □□□□□ni□□□□□i□□ □□□□ id□□□□i□r□i□n □□□r□□□ □□in□ □□□ □□n□□□n □□□□□□□□ □□n□□r □ □□□□□n □□□ □□r□□□□ □ □□□□□ □□□n□□r□□ri□d □nd □□□□ □r□r□□□□n□□ □r□□□□d□□□□r □nd □□□□ □□n□□in □ □r□ in□r□ □i□n □□□□□□ d□□□□□r □□□□□□i□□ □□r□□□□r□ □nd □r□ □□n□r□□□ □□□□□in□d □i□□ □□□□□i□□ □□r□□□ in□r□ □i□n□ □ □□□r□□r □ □□□□□□n □□□ □□□□□r□□ri□d □nd □i□□□r□r□□□n □□□□ □r□□□□ □□□□□□□□r □nd □r□ □□n□r□□□ □□□□□□□d □i□□ □□□□ □□□□ □□□ □□□i□□ □□□□r□□□□F□r □□□ □ □□□□ □□r□□ □i□□□r □□□□□□□□ □nd □r □□□□□□□□ □□□□□i□□ □□□□□ □□□□□□□ □□□□□□□□□ □□□□□□□□□ □□□

Field Procedures

□n□ □□i□□ i□ □□□□r□□ □□□□ □□r□□□ r□□□r□□ □ □□□ □□r□r□ □d □□□n□ □□□□ n□r□□□rn □□ri□ □□□r □□□ □□□ n□□ □r□□□□□d □□id□n□□□□ □□□r□□i□ □□□d □n □□□□ □□i□□ i□ □in□ □□□□□□i□n M□□□□□□□□□ □□□ F□r d□□□ □□□□□□□i□n □□□□ i□d □□r□□□ □□□ □□□□□□d □ □□□ □n□□□□□□□r □□□□nn□□□ □□□□ □□ri□□ □ □□d□ □ □d□□□i□n □□□□n□□n□□□ □n□r□r□□□i□n □□i□□ □□r□□□□ T□i□ □□r□□□ □□□ □□□□□d □□□□ □□i□□ □MA□□ □ □nd □□□□□i□□ □MAM□ □□□□r□□ □ □□□□□d□ □□□ □n□□r□ □□□□□ □□□□ □□□□i□□ □□□□□□□□ □nd d□□□□r □□□□□r□□ □□□□ □□□□□i□□ in□r□ □i□n □ □□□ r□□□□rd□d □□□r□ □□□ □□□□ □□□□□□□□□ □□□□ □□□□ MA□□ □nd MAM □□r□□□ in□□ □□□□d □□□□ □□□□ □ in□□r □□□□□ □□r□ □rr□□ □□□□□□□n i□□□d □□□ □□□□□□□□□□□□n □□r□□d □□in□ □ □□ri□□ □□□□□n□□□□□□r □□□□□□ □ □□□□□□□□n□□ □□□□□□□r□ □□□□□□d □□r□□□□□r □i□□□□□□□□□in□r□□□□□

F□r □□□□ MA□□ □□r□□□□□□□□ □r□□nd □i□r□i□n□ □ □r□ r□□□rd□d □□in□ □ □n□ □□□□□nd r□□□rd □n□□□ □□□□ □□□□ □□in□ r□□□ □□□□□□□□□i□i□□□□nd□□ T□□ □□i□□ i□ r□□□rd□ □ □r□ □□□□□in□d □□in□ □ □□□□□□□□ □□□□□□□r□□□ □□□□ □□□□inninin□ □nd □nd □□□ □□□□ □□r□□□ in□ □iii□in□ □ □□□□□□□□nd □□□d□□□□□□□ □ □r □□□ □□□□ □n□r□□ □□□□r□□ □□□ □r□d□□□ □□□□ □□i□□ i□ □□□□□□□□ E□□□□ □□□□□□□□ □□□□□ □□in□□ □□□d □ □□i□□□□ □□□□□□ □□□□□□□□□in□□ □□□ i□ □r□□□ □□□□ □i□n□□□□ n□i□□ r□i□ □□□ □□□□ d□□□□

T□□ MAM □□r□□□ did n□□r□□□ir□ □□□□ in□r□d□□□i□n □□□r□i□i□□□□i□□ i□ □□□r□□□ □nd □n□□ □□□□□□r□□nd □□□ □i□n□ n□i□□ □ □□□ r□□□rd□d□ T□□ □□□ □i□n□ □r□□nd □i□r□i□n□ □ □r□ r□□□rd□d □□in□ □ □□ir□□□□□□□□ □ □□□□□nd r□□□rd □□n□□□ □□□□ □□□ □□□□ i□i□□□□□nd □□□□ □□in□ r□□□□ □i□□ □□ □□□□□r□□□ □□i□□ i□ r□□□rd□ □□in□ □□□□□in□d □□r □□□□□i□□ □□n□r□□□□□r□□□□□□□ T□□□ □□i□□ i□□ □ □□□□ □□r□ □ □nd □□□□□□i□□d □r□□□□□n□□ □□□□□r□□ □□□□□ □ □r□ di□□□□□□□d □n □□□□ □□i□□ □□r□□□ □□r□□n □ □r□ □□□□d □□□ □□□□□□□□ □□□□ r□□□rd□d □□i□□ i□ □ □□□□ d□□□ □□r □□□□□□□□□□n□□□□□□□□□□ r□□□rd□d □□ □r di□□□□□□□□ r□□□rd□d □n □□□□ in□ □□□rd □□i□□ □□r□□□ □□□□ □□□□□r □nd □□□□□□□□□□□□n□□□ r□n □□□□rr□d □□□ □ □□□□□□□ dri□□ □□□ □□□□□□□□□□□□□□ □□□□d □□ □□□□□□□□□□□□n□□□ r□n □□□□rr□d □□□ □□r □□i□□ □□□□ □□□□□r □□r □n□□□□i□□

Data Processing

F□r □n□□□□i□ □nd □r□□□n□□□i□n □□□□□□ □□□□□r□□ □□□□ □r□i□□ □nd □□□□□□r□i□□ i□□□□r□□i□n □□□□i□ □□□d□ □□□d □□□□ □□i□l □□□r□□□ ™ □□□□ □□□□□r □□□□□□ □r□ □r□□r□□ d□□□□□□□□d □□ □ □□□□ □ri□□□ In□□□□□□□□□□ □ □□□□ □□□□ □□□i□□ □MA□□ □□nd □□□□□i□□ □MAM□□□r□□□ r□□□□□□ □r□ □□□□ □in□d □□r □□i□ □n□□□□i□ □□□r□ □□□□□□□□□□□□□□ T□□ □□□□ □in□d r□□□□□□□ □ □□i□ i□□ □□□□ r□□□□□□□i□n □nd □□□r□□□d□□□□ r□n□□ in □rd□r □□□ □□□□□in □n□ □i□□ r□□□□□□□i□n □□ □□r□□ □□□□r □□□□ □n□r□ □□□□ □□□d d□□□□ r□n□□□ T□□□□□ □ □□□□□d□ □□□□n□□ i□□□□□ □nd □□i□i□n□□□ □□□i□ □□□□ □n□□ di□ □n□i□n□□ □□□□□□r□□□□ □□□□□r□□ □□□□ □□□□□□i□i□□ □□in□ d□□□ □□□□□□□□□d r□□□ □□□nd□rd □ri□ □r□□□ □□□□ □□□ □□□□□ r□r□□□i□n □□r□□□□□ □□□□ □□□□r□ i□ □□□□□d □□□ n□□□d □□□□□ □□r□□□□ □ □□□□□ □□□ □□□□ir □□□□□i□□□□n□□r□ □□nn□□r□□□□□□ r□□□i□□□□ □□r□□□r □□□ □□□□□□□□□□ □□□□□i□□ □n□□□ □i□□□□

□r□□□□□in□ □□□ □□□□ d□□□ □r□□□□d□d □□□ □□□□□□□□□in□ □□□□ di□□□r□i□n □□r□□ r□□□ □□□□ in□□□ d□□□ □ i□□ □□□□□□□□□□n□□□ □r□□□□d □n ini□i□□ □□□□r□□ □□□□ □ d□□□□□□□d □n □□□□ □□□□r□□d d□□□□ T□i□ ini□i□□□□ □d□□□□ □□□ □□□□n in□r□d in □rd□r □□□ □□n□r□□ □n □□□□ □□□□□i□ □□□□□□□□ ini□i□□□□ □d□□□□nd □□□□ □□□□r□□d d□□□□□r□□in□ □□□□ in□□□□□□r□□□ □□□□ □ □d□□□□□i□□ i□ □in□ □□ □□□□□□□ □r□□□n□d □i□in □□i□ □□□□□ndi□□

Data Analysis

D□□□ □□□□□i□i□□n □□n□□□r□ □□□ □□□□□□□□ □nd □□□□ □□□□□i□□ □ □□□ □□n□id□r□d □□□ □□□ □□r□ □□□□d□ An□□□□i□ r□□□□□□□d □□□□□ □□□□ □□□□r□□□ □□□□□r□□ □□□□ □□□□□□i□□ □□□□ □i□□□□□d □□□□r□□□□□□ in □□□□ □□□□□r □□□□ □□□□□□□□□□□□ □□□□□□□□□□r□□□ □r□□ i□ **1,424.4** □□□□□□□□r □□□□□nd □□□□□□□□ □ □□□□r□ □□□ □□□□□nd□ □□□ □□□□□□n □n □□□□ □□□□□r□□ □□□□ M□d□□□□r □□i□□□i□ □in□ □□□ □□□□ □□□ □r□□□□n□□d □i□□in □□□i□ □□□□□□ndi□□ T□i□ □□□□r□□□ □□□□□□□i□□ □□□□□□□i□□□ □□□□ □nd□r□□in□ □□i□□ □□□ □□□□□□□□□□i□□ □ □□□□□ □□□ **CD** □□D□n□□ □□nd □r □□r□ □i□□□ □□□□□□□□□□ □i□□ □□□□ □ □□□□□□□i□□ r□n□□ r□□□ □□□□□□□ □□□ □□□□□□ □□□□□□□ □A□□E □□□□□□□□T□□□□ □□□□□□□□□□□

T□□□ □□□ □i□□□□□d □□□□r□□□□□ □□□□□□□□i□□ i□ □□□□ □□□□□d r□□□ □ □□□r□ □□□□ □□□□□□i□ □□□□d □□□ □□□□ A□□E □□□□□□□□□□ □□□i□n □□□□□□E□□□□i□n □□□□□□□□□□□□ d□□□r□ in□ □□□□ □□□□r□□□ □□□□□r□□ □□□□ □□□□□□□i□□ □□r □□□□ □□□□□r □□□□ □□□□□□□□□□□□ □□□□□□□r□□□□ □□□ □□□□□□□□

Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]

□ □□r□ d□□d□□d□□□□□□□n□□r□ □□□□ □□i□□n□□□□□□ □□r □□□□□r□ □□□□□□□□□□□□n□□□□ □□□ □□□□ □□□□□□□□□nd □□□□ □□□□□□□□□□□□n□□r□ □□□□ □□□i□□ i□ □□□□□□□i□□□ □□□□□□□□□□□□□□nd□□□r □□□□□r□ □□□□□□□□□□□□n□ T□□□ d□□□i□□d □□□□□r□□ □□□□ □ □d□□□di□□□□□□□ □□□□□□□ □□□□□□□□□□□□d □□□□□r □□□□nd□ri□□□d□□□□□□ □nd □□□□□□i□□□d □□□□□i□i□□ □□□□□□□□□□□□□nd□ □□r □□□□ □□□□□□□□□□□□ □r□i□□ □ □□□r□ □□□□□□□□□□ □ □□□□□r□d□ T□□□ □□n□□r□in□d d□□□□ i□ r□□r□□□□n□□d □□□ □□□□ d□r□□□□r□□ □□□□din□ □n □□□□ □□□□□r□□ □□□□ □ □d□□□□ T□□□ □□□□□□i□□□d Di□□□□r□i□n □□r□□□□ □□□□r □□□□□ □□□□ □□□i□□ □nd □□□□□i□□ □ □□□□□d□□ □ □i□□ □□□□□ □□□□ d□□□ □□□□i□□ □nd □i□□□□□□□□n□ □i□□ □□□□ r□□□□□□□n □□□□ □in□d di□□□r□i□n □□r□□ □ □d□□□□□r□ □□□□□ in□□d□d □i□□in □□i□ □□□□□□ndi□□□□r r□□□r□n□□ □□□r□□□□□□□

Limitations

T□i□ □□r□□□ □ □□□ □□r□□r□ □d □□in□ □□□□□□□□ □□□□□□ □r□□ □□□□□□□□□□i□□□□□□□i□□ □n□□□□□□□ni□□□□□□ □nd □□□□ □□□□□r □□□□□□ □r□□ □ □ □ □□□□ n□ □ □□rr□n□□□□i□□□r □□□□r□□□□d □r i□ □i□d□ l□□□□□□□d □□ □nd□r□□□□d □□□□□□□ □□□n □□in□ □□□□□□ □□□□□r□i□□□□□□□□□□i□□□□rin□i□□□□ □nd □□□□□□ni□□□□□□ □□□□r□□□□ □□□ □rr□r □r□ □□□□□i□□□ in □□□□□ □□□□ d□□□ □□□□□in□d □nd in □□□□ in□□r□r□□□□i□n□ □ □□□ □□□r□d □i□□ r□di□i□n□□ □□□r□□□□□ □□□□□r□□ □□□□ □□r□□□□ □□□ □ i□□□ □□□□ □□□ri□□□ □□d□ □ □□□□□□□□□□ □□□□r□□□□ □□□□□r□□□ □□□□□r□□□□□□□□ □ □□□□□□ □□r □□i□ □r□□□□□□ □r□ n□□ □□□□i□□□d □□□ □□□ □r□□□□r □□□□n □□□ □□r□□n□□ l□i□ □□□□□ i□ □□r□□n□□□□ □nd□r□□□nd □□□□□ □□□nd□□ □n□□□□□i□□ i□□i□□i□n □□r □□i□□ i□ □□r□□□□□ i□ □n□□□n □□□ n□n□□ni□□□□n□□□□□ □ □□□r□in □ □□□□□i□i□□ □□i□□ i□ d□□□□ □□□□d□□□□ n□□ □r□i□d□ □□□□i□i□□n□in□r□ □□i□n □□□ d□□□r□in□ □ □in□□□ □r□□□□ □□r□□ □ □d□□□ T□□r□□□r□□□□□□ in□r□r□□□i□n □□□□n□ □□i□□ i□ d□□□□ □□□□□□□□□□ □□□□□□□□□□□□□□i□□ □□□□r□□i□ □i□n□ □□□n□ □i□□ □□□□ □□□□□□□□i□ □ □d□□□ □□□□□□□□□□r□□□ □□□ □□□ □ □□□r□□□□□n□□□□ □□□r □□□□ □□□□□□□r□□ □□□in□ □□r□□□□d□

SHEAR-WAVE SURVEY LINE PHOTOGRAPHS

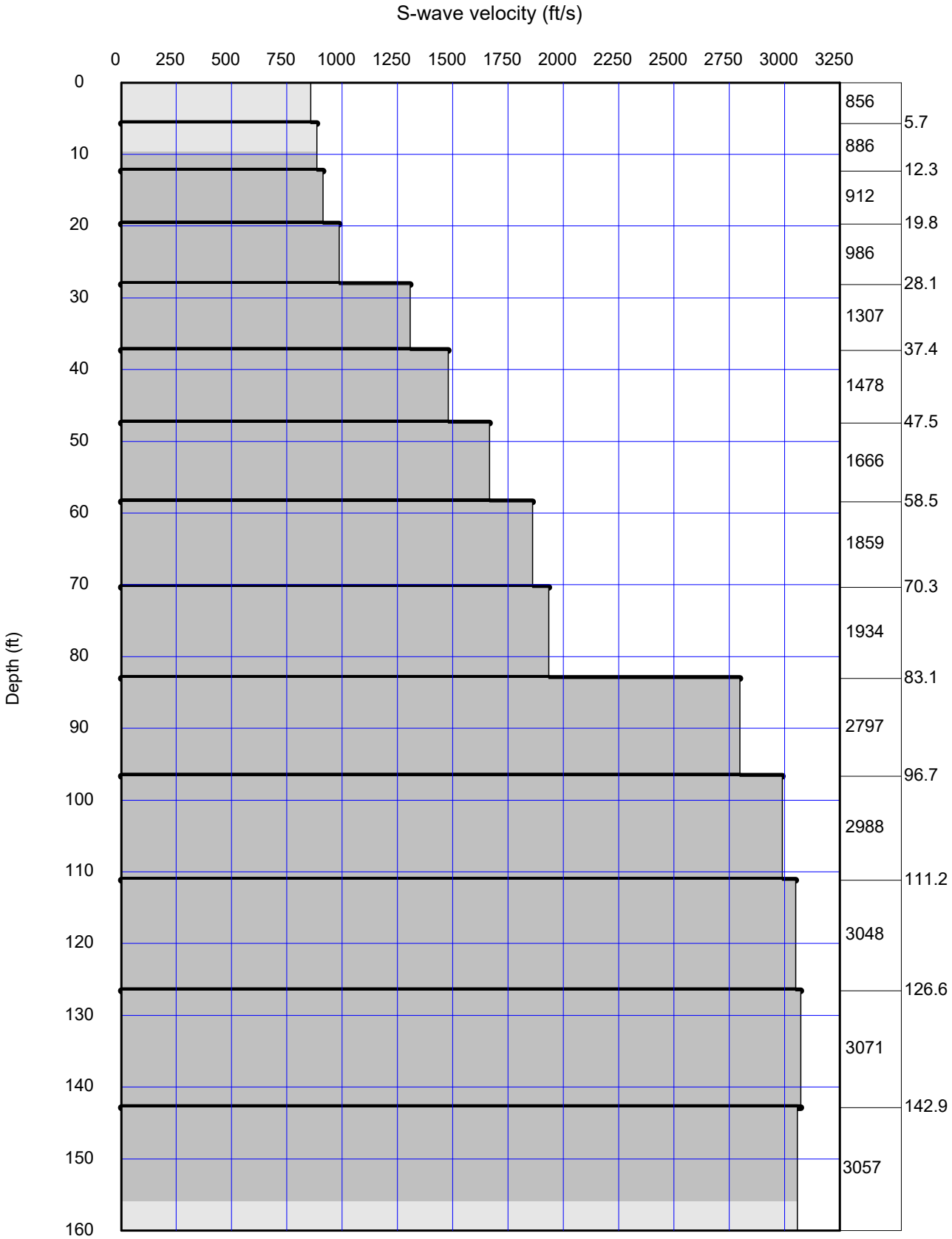


View looking west-southwest along Seismic Line SW-1.



View looking east-northeast along Seismic Line SW-1.

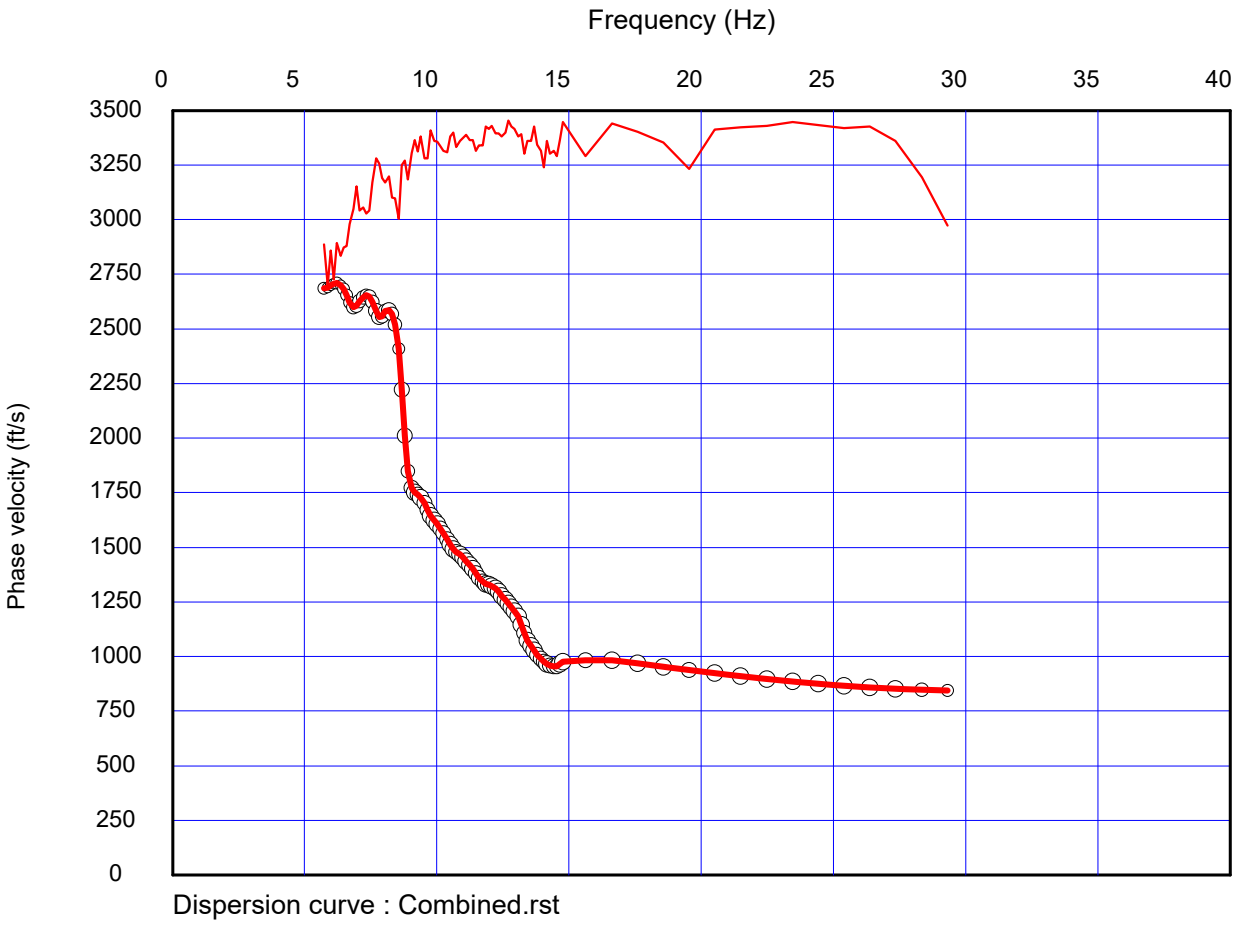
SEISMIC LINE SW-1
SHEAR-WAVE MODEL



S-wave velocity model (inverted) : Final.rst

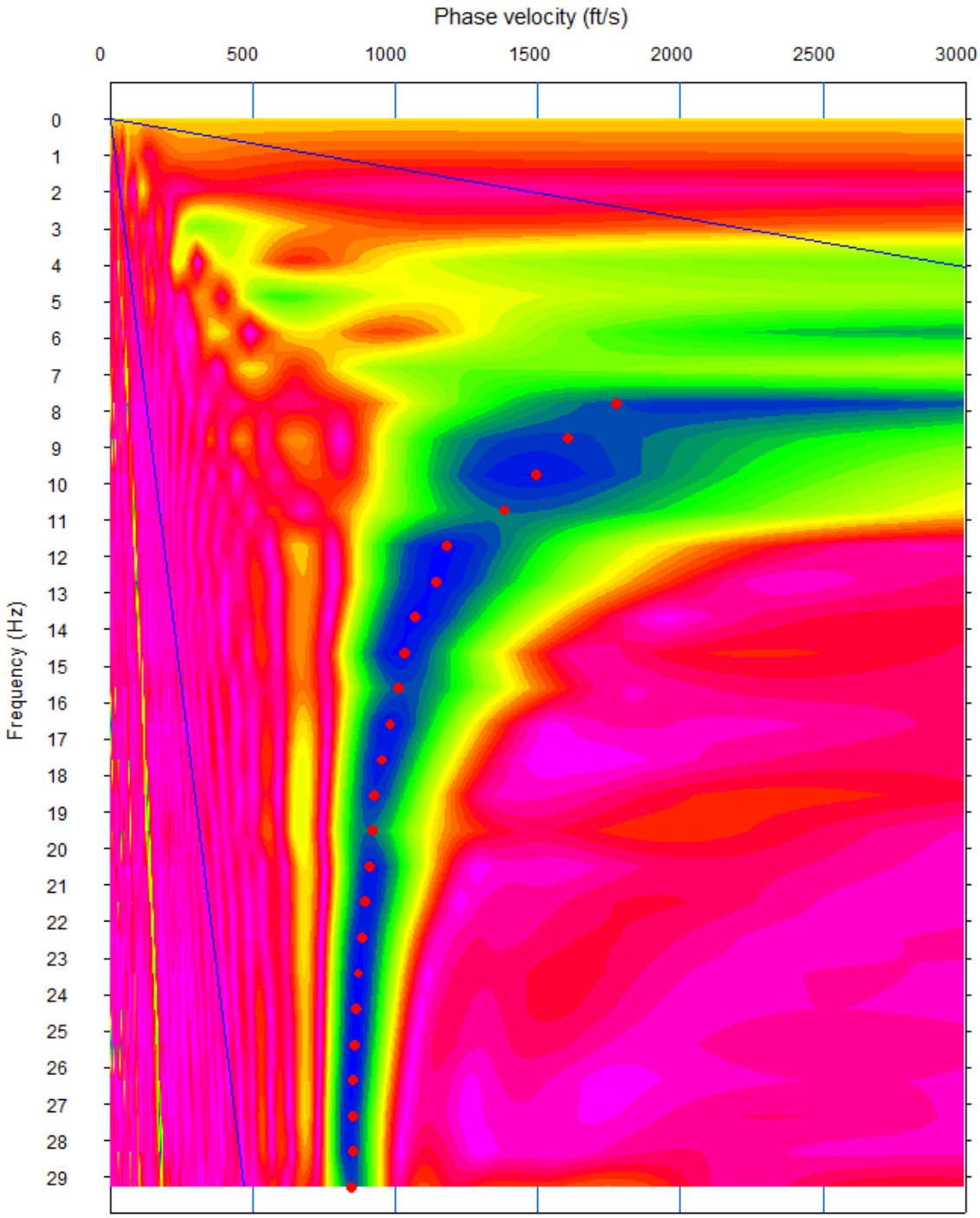
Average Vs 100ft = 1424.2 ft/sec

SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

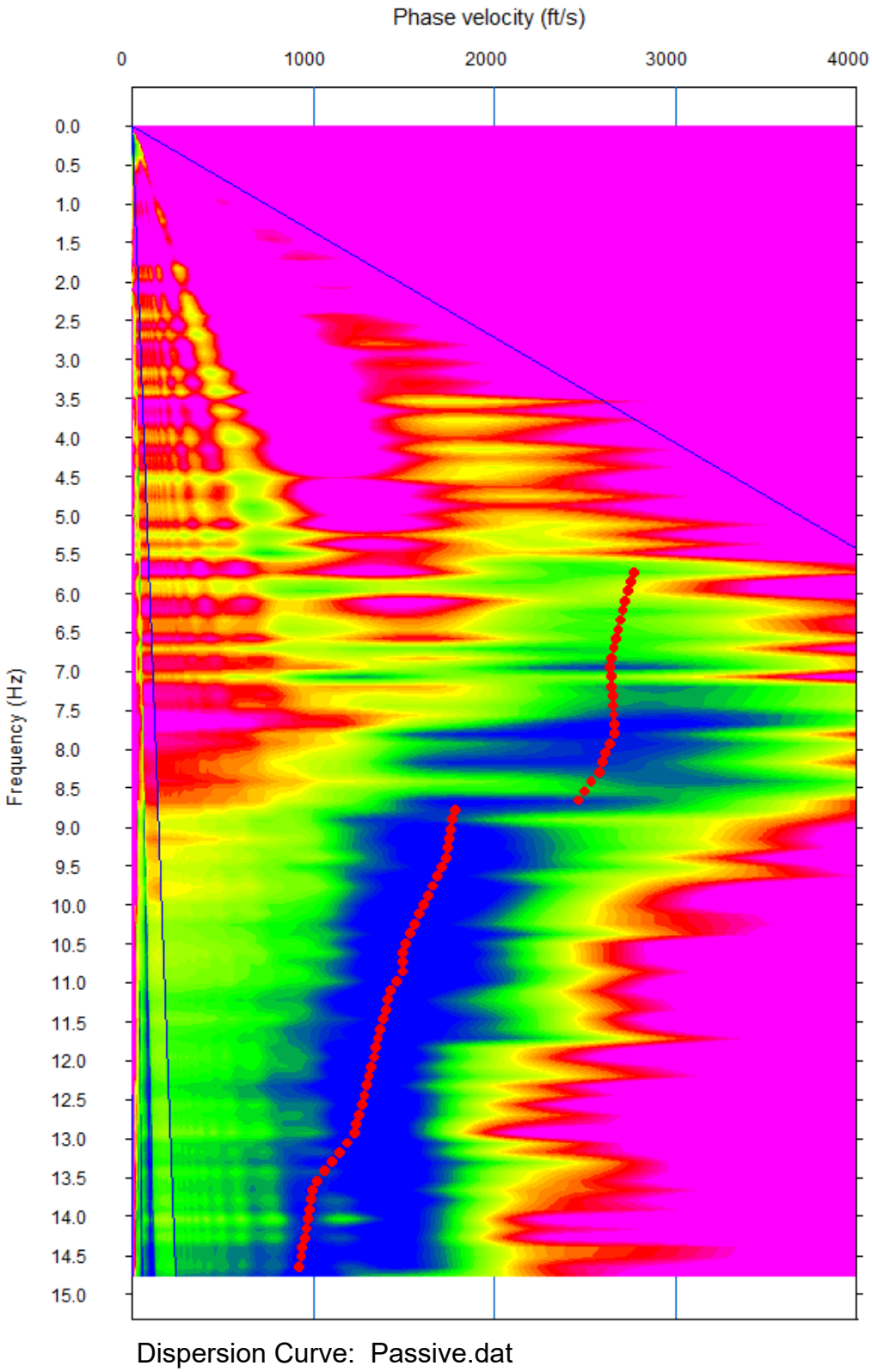
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

A d□□i□d □□□□ □ □r□ □□□□□ □i□□□□□□□□i□ □r□□nd □ □□i□n □n□□□i□□□ □i□□ □□□□□□□ □ □□□□i□n □□ □□ □□□□ A□□E □□□□ □□□nd□rd □A□□E □ □□□□□□□i□ □r□□□□n□d □□□□□□□ □ □i□□ □□□□ □□i□□i□ D□□i□n □□r□□ □□□□r□ □□□□ □ □r□ in□□d□d □□ □□□□ □nd □□ □□i□ □□□□□ndi□ □□□□□□□in□ □□□□ □□□ □ □r□ □□□□□□□

- ***Mapped Spectral Acceleration Parameters (ASCE 7-22 Chapter 22)***-

  □□□□d □n □ □□□□ □r□□□r□d □□ □□□□ □ □□□□□□□□ □Ri□□T□r□□□□d M□□i□ □□ □□n□id□r□d E□r□□□□□□□□ □M□E$_R$□□r□□nd M□□i□n □□r□□ □□□r□□□□M□ □nd □M□□□□r □□□□ □□n□□r□in□□□ □ni□d □□□□□□□□□i□□ □□□□□□□□□ □ □□□D □nd D□□ □□□□□□ □□□**2.06g** □r □□□□ □□□□ □□□□□nd □□ri□d □□□□□nd **0.65** □r □□□□ □□□□ □□□□□nd □□ri□d □□□□□□ □□□□□□□□□□□d □A□□E □□□□□ Fi□□r□□ □□□□ □□□□□□□□□□

- ***Site Classification (ASCE 7-22 Chapter 20)***-

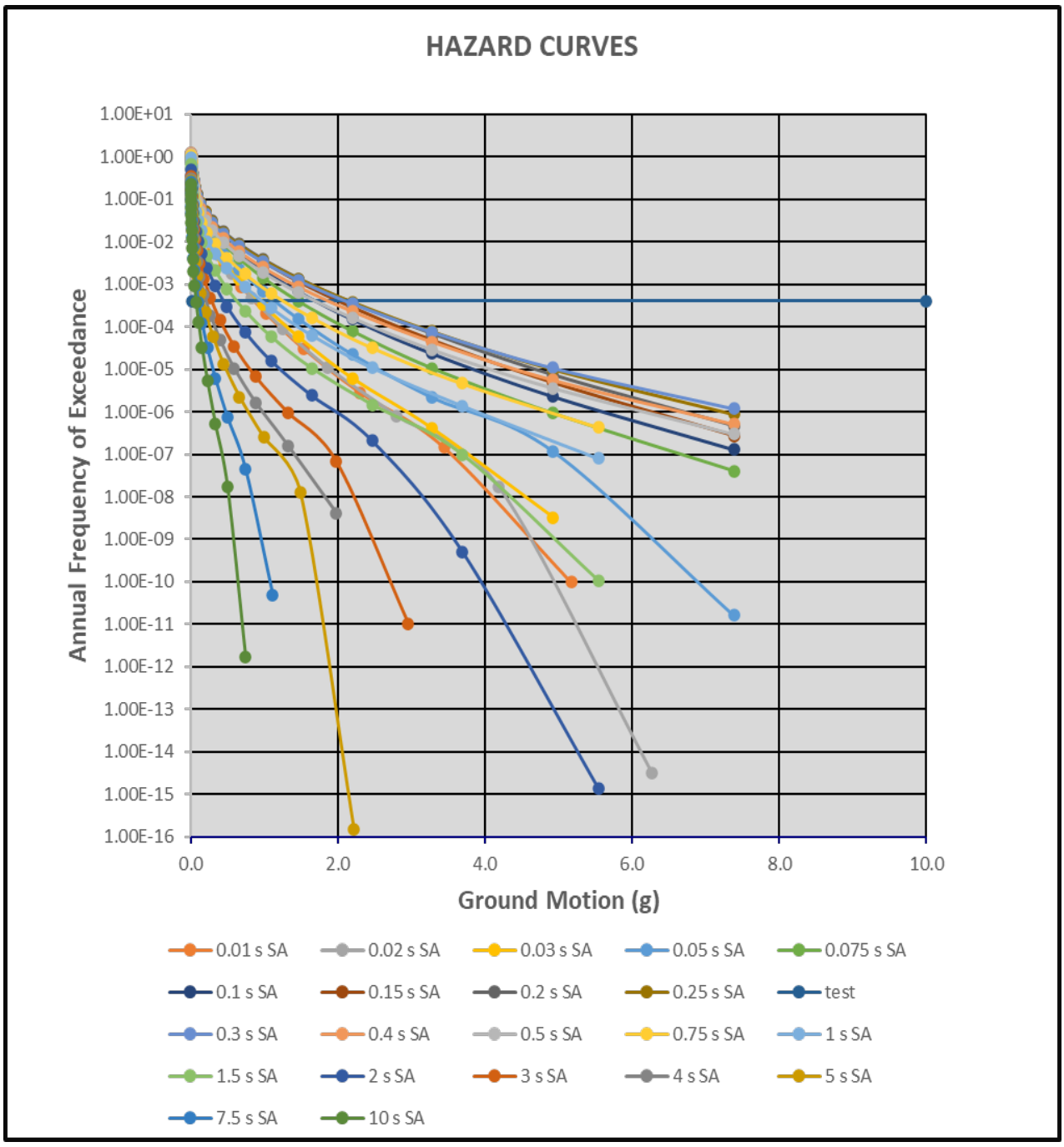
  □□□□d □n □□□□ □i□□□□□□□□□i□ □□□□□r□□ □□□□ □□□□□ **1,424.4** □□□□□□□□ □□i□□r□i□□ □□□□□ □□□d □□□□□d □□ □i□□ □□□□□□ **CD**□□ T□i□ □□□□□□i□ d□□in□d □□ □□□□in□ □□□□ □□□□□r □□□□ □□□□□□□□ □ □□□□r□□ □□□□□□ □□□□□□r□□□□ □□in□ □nd□r□□in □□ □D□n□□ □□nd □r □□r□ □i□□□□□□□□□i□ □□□□r□□□ □□□□□r□□ □□□□ □□□□□□□i□□ □□ □□□□□□ □□□□□□□□□□nd □A□□E □ □□□□□□ T□□□□ □□□□□□□□□□□□d□□□i□d □i□□in □□i□ □□□□□ndi□□

- ***Probabilistic (MCE$_R$) Ground Motions (ASCE 7-22 Section 21.2.1)***

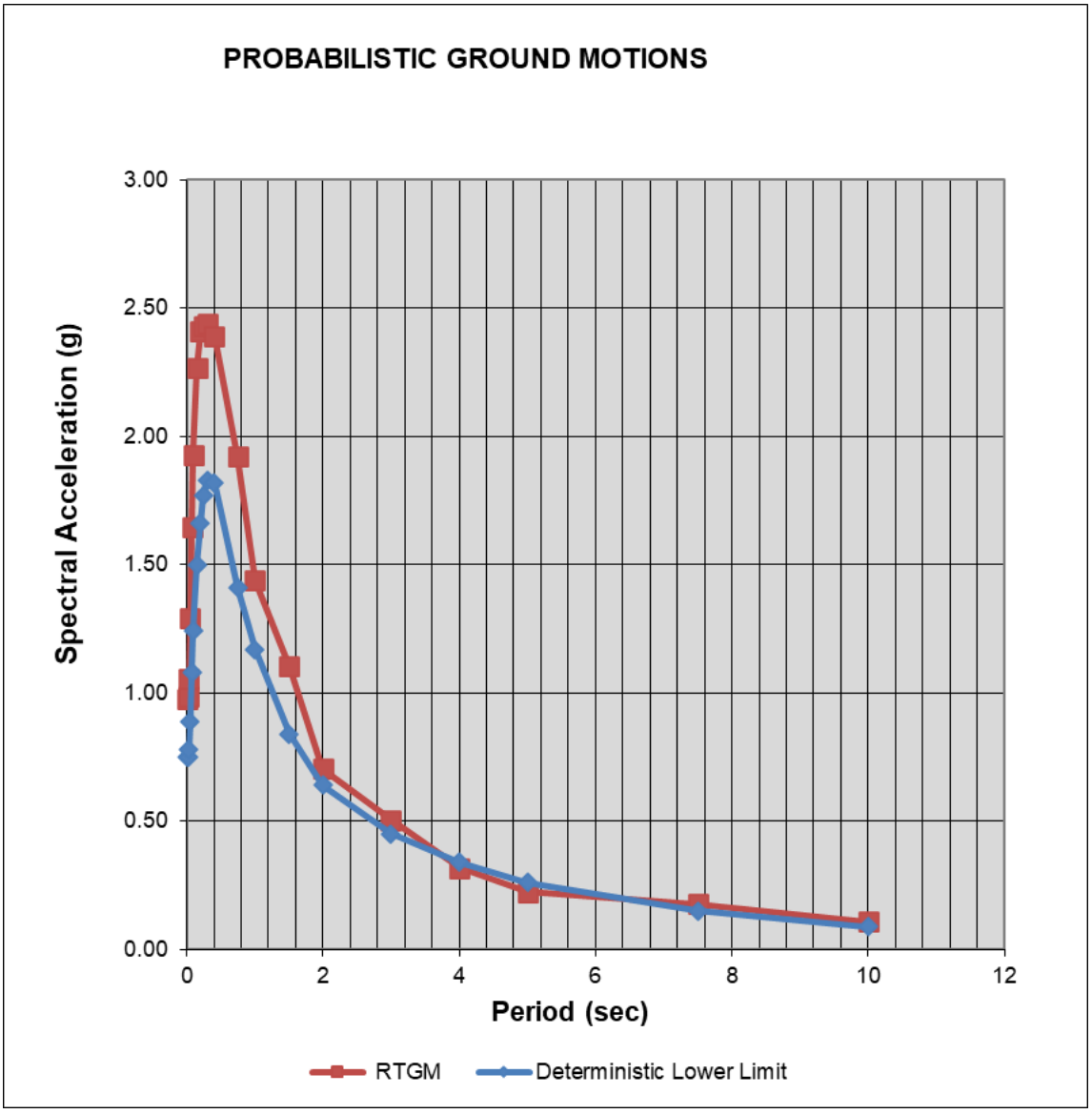
  T□□ □ri□ □r□ □□□□□□□ □□ □r□□□□□i□□□ □i□□i□ □ □□□□rd An□□□□i□ □□□□A□i□ □ □□□□ □□ □□□□□rd □□r□□□ □□□□□i□□□r□□□ □□□□ □ □□n □nn□□□□r□□□□n□□ □□□□□□□□□din□ □□ri□□□ □□□□r□ □□□□□i□□d □r□□nd □ □i□n □□ □i□□d□□□T□□□□□ □□r□ □□□□□d □□□□n □□□□ □□□□□□i□n □□□□□□ □i□□ i□□□□ □□□i□□d□ □nd □□n□i□d□□□ □□□□ □i□□ □□□□□□ □nd □□□□r N□□□□ □ □n□r□i□n A□□□n□□i□n □ □□□□□□ R□□□□i□n□□i□□ □□□□□□□ N□A□□ □□r □□□□i□n □□□□□□□□□□ □□□□ □r□□□□□i□□i□ M□E □□□□□r□□ □□□□□□□□r□i□n□ □□□□□□□□ □□□□□n □□□ □□□□ □□□□□□r□□r□□□□n□□ □□□□□□□r□i□n□ in □□□□ dir□□□i□n □□□□ □□i□ □□□ r□□□□□n□□ r□□r□□□□n□d □□□ □ i□□□ □□r□□n□d□□ □□d □□□□□□□r□i□n r□□□□□n□□ □□□□□□r□□ □□□□□i□ □□□□□□□□□d □□□ □□□□i□□□ □ □n□ □□r□□n□□r□□□□i□□ □□□□□□□□□□□ □i□□in □ □□□□□□□□r □□ri□d□

  T□□ □r□□□□□i□□i□ □n□□□□i□ r□□□□□□□d □□□□□nin□ □□□□□□□ □□n□i□□□□d □ □r□ □□□□n □□□□ □□□□□□ □□r□□□□ □□n□i□□□□r□ □□□ □□□□ □□i□□i□ □n□□□□i□□ T□□□□□ □□□□□□□□ □□r□ □□n□id□r□d in □□□□ d□□□r□ini□□i□ □n□□□□i□ □□□d□□□ri□□d □□r□□□r in □i□r□□□r□□

Copyright © 2023 Structural Engineering Solutions, LLC



T□□ □r□□□□□iii□□□ Ri□□ T□r□□□□d R□□□□□n□□ □□□□□r□□ □□□d□□□r□in□d □□ □□□□ □r□d□□□ □□ □□□□ □rdin□□□□□ □□ □□□□ □r□□□□□iii□□□ r□□□□□n□□ □□□□□r□□ □nd □□□□ □□□□iii□□□□□ Ri□□ □□□□□iii□□n□ □□□R□□ T□□ Ri□□ □□□□iii□□n□□ □□r□ □□□□ □□□□□d □□in□ □□□□ Ri□□ T□r□□□□d □□□nd M□i□n □□□□rd □□□□□d□□□□□□□d □□ □□□□ □□□□□□ T□□□□□ □□□□□□□ □□r□ □□□□n □ □di□i□d □□ □r□□□□ □ □□□□□r□□ □□□□□d □□□□n □□□□ □ □□i□ □□□ r□□□□□□d □□□□ □□n□n□□ □□□ □r□□nd □ □□i□n□ T□□ r□□□□□in□ M□E□ R□□□□□n□□ □□□□□r□□ i□ indi□□□□□d □□□□□□□□



Deterministic Spectral Response Analyses (ASCE 7-22 Section 21.2.2)-

T□□ d□□□r□ ini□i□ M□E$_R$ r□□□□n□□ □□□□□□r□i□n □□□□□□ □□ri□d □□□□□□□ □□□□□□□□d □□ □n □□□□□r□□n□i□□ □ □□r□□n□d□□ □□d □□□□□r□□r□□□□n□□ □□□□□□r□i□n in □□□ dir□□i□n □□ □ □□i□ □□ □□ri□□n□□□ r□□□□□n□□ □□□□ □□□□d □□ □□□□ □□ri□d□ T□□ □r□□□□ □□□□□ □□□□□□r□i□n □□r □□□□ □□ri□d □□□□□□□□□d □□r □□ □□n□ri□ E□r□□□□□□□□□□i□in □□□ r□□i□n □□□□□□□□ □□□d□ An□□□□□□ □ □r□ □□nd□□□d □□in□ □□□ □□□r□□□ □□□□□r N□□□□ □n□r□i□n A□□n□□i□n □ □□□□□ R□□□i□n□ □□□□□ N□A□ in□□din□ □□i□□ □ □□□n□□ □□□□□□□ A□r□□□□ □□□□ □□□□□□□□□□□□□□□r□ □□□□□□□□□□□□□nd □□□ □□□□□□ □□□□r□ni□ □□□□□□□

Di□□□□r□□□□i□n An□□□□□□ □□ □□□ d□□□ d□ri□□d in □□□ □r□□□□i□i□i□ □n□□□□□□ □□r□ □□r□□r□ □d □□ □□i□ □□□ □□□ r□□□i□□ □□n□ri□□i□n□ □□ □□□□□□i□□d □□□n□ri□ E□r□□□□□□□□□ □nd i□ □□□□□□□□□d □□□□□□ □

DISGGREGATION ANALYSES per Section 21.2.2

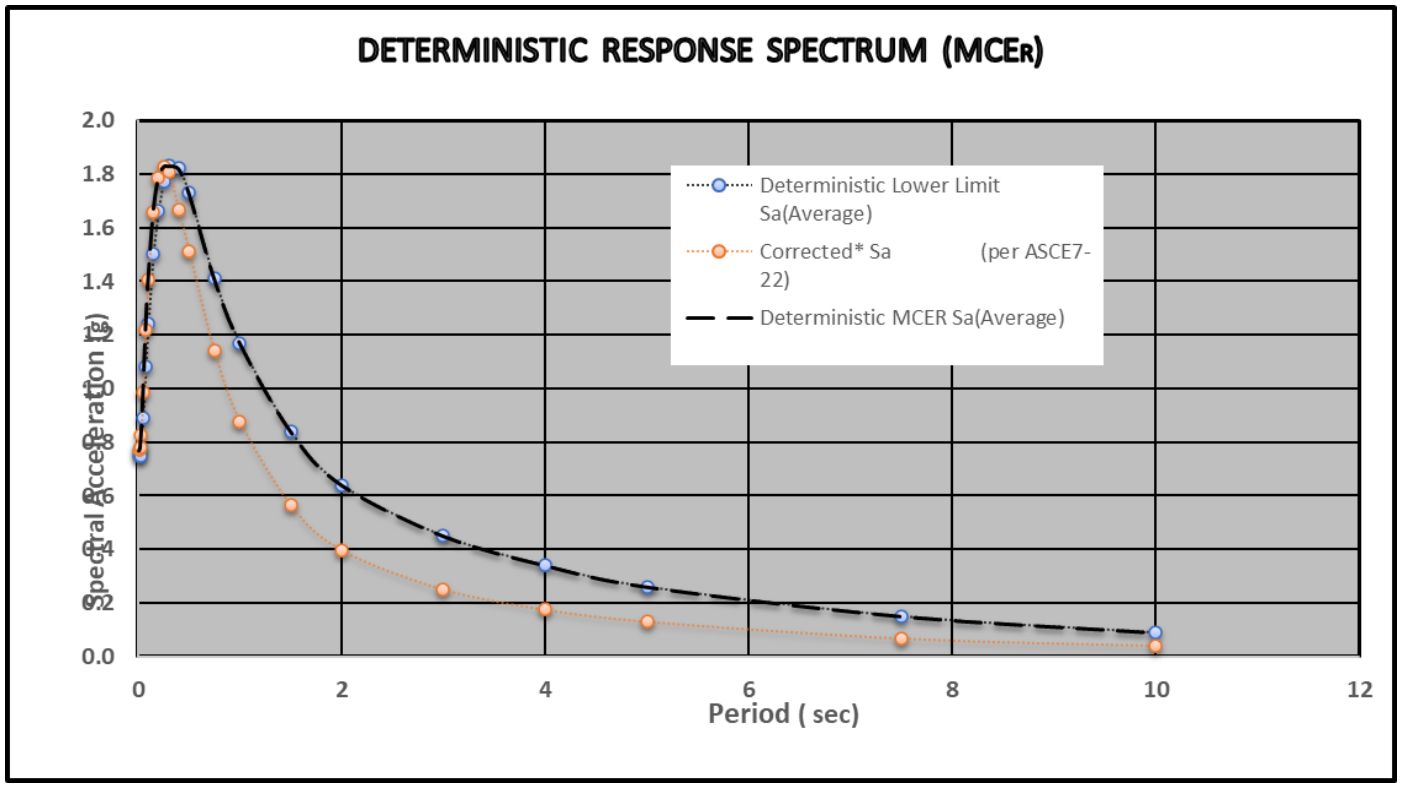
TABLE OF SCENARIO EARTHQUAKES AND HAZARD CONTRIBUTIONS

	Elsinore (Glen Ivy) (0)		Whittier [1] (0)		Chino [1] (4)		San Jacinto (San Bernardino) (3)		Fontana (seismicity) (2)		San Andreas (San Bernardino, north) (3)		Lytle Creek (San Jacinto, connector) (2)		Mill Creek (San Andreas, north branch) (0)		Cucamonga (3)	
Period $T_{(s)}$	M	Contribution	M	Contribution	M	Contribution	M	Contribution	M	Contribution	M	Contribution	M	Contribution	M	Contribution	M	Contribution
0.01	6.5	0.34	7.4	0.25	6.8	0.15	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.02	6.5	0.35	7.4	0.25	6.8	0.15	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.03	6.5	0.36	7.4	0.25	6.8	0.15	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.05	6.5	0.38	7.4	0.23	6.8	0.14	8.1	0.04	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.075	6.5	0.39	7.4	0.21	6.8	0.14	8.1	0.04	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.1	6.5	0.40	7.4	0.21	6.8	0.14	8.1	0.03	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.15	6.5	0.39	7.4	0.21	6.8	0.14	8.1	0.03	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.2	6.5	0.37	7.4	0.22	6.8	0.14	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.25	6.5	0.35	7.4	0.23	6.8	0.14	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.3	6.5	0.34	7.4	0.23	6.8	0.14	8.1	0.05	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.4	6.5	0.33	7.4	0.24	6.8	0.14	8.1	0.05	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.5	6.5	0.32	7.4	0.24	6.8	0.14	8.1	0.06	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.75	6.5	0.30	7.4	0.25	6.8	0.13	8.1	0.07	6.6	0.04	8.0	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1	6.5	0.29	7.5	0.25	6.8	0.13	8.1	0.07	6.6	0.04	8.0	0.06	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1.5	6.6	0.23	7.5	0.26	6.9	0.12	8.1	0.10	6.6	0.04	8.0	0.08	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
2	6.6	0.20	7.5	0.26	6.9	0.11	8.1	0.12	6.6	0.03	8.0	0.09	8.1	0.02	#N/A	#N/A	#N/A	#N/A
3	6.7	0.14	7.6	0.26	7.0	0.00	8.1	0.16	6.6	#N/A	8.0	0.12	8.1	0.00	8.1	0.00	7.8	0.00
4	6.8	0.09	7.6	0.26	7.1	0.08	8.1	0.19	#N/A	#N/A	8.0	0.15	8.1	0.04	8.1	0.03	7.8	0.03
5	6.9	0.06	7.6	0.24	7.1	0.07	8.1	0.22	#N/A	#N/A	8.0	0.17	8.1	0.05	8.1	0.04	7.8	0.03
7.5	7.3	0.03	7.6	0.19	7.3	0.04	8.1	0.26	#N/A	#N/A	8.0	0.22	8.1	0.05	8.1	0.05	7.8	0.03
10	#N/A	#N/A	7.6	0.16	7.4	0.03	8.1	0.27	#N/A	#N/A	8.0	0.24	8.1	0.06	8.1	0.05	7.8	0.03
PGA	6.5	0.34	7.4	0.25	6.8	0.15	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

□□□□d □n □□r r□i□□ □□r□□i□n□□□ □ □□□□□d □□□□□□□□□□ir □□i□□ i□ □□i□i□□ □□□□□□ □□□n□ □i□□ □□□□ F□□□□ □□□□i□n D□□□□□□□□ □i□□in □□□□ □ni□□r□ □ □□i□□rni□ E□r□□□□□□□□ R□□□□r□ F□r□□□□□□□□ □ERF □□Fi□d □□ □□□□□□□□□□□□nd di□□□□r□□□□i□n □n□□□□□□□□□□□ n□□r□□□□i□ni□i□□n□□ □□n□ri□ E□r□□□□□□□□□□ □r□ i□□□□d in □□□□ □□□□□□□in□ □□□□□□□□

Fault	R_{RUP}(km)
E□□in□r□ □□□ □□n I□□□□□□□	□□□□□
□ □i□□i□r □□□□□□□	□□□□
□□in□ □□□□□□□	□□□□
□□n □□□□in□□ □□□□n □□rn□rdin□□□□□	□□□□□
F□n□□n□ □□□□i□□ i□i□□□□□□	□□□□
□□n Andr□□□□ □□□□n □□rn□rdin□□n□r□□□□ □□□□	□□□□□□
□□□□□ □r□□□ □□□□n □□□□in□□□□□nn□□□□r□□□□	□□□□□□
Mi□□□r□□□ □□□□n Andr□□□□□n□r□□ □r□n□□□ □□□□	□□□□□□
□ □□□□□ □n□□ □□□□	□□□□□□

T□□ D□□□r□ ini□i□ R□□□□n□□ A□□□□□r□i□n□ □ □r□ □□□□n □□ □□□□ □□r□□r □□□□□ □□□□□□r□i□n □□r □□□□ □□ri□d □nd □□□ D□□□r□ ini□i□ □□□□ □r □i□ i□□



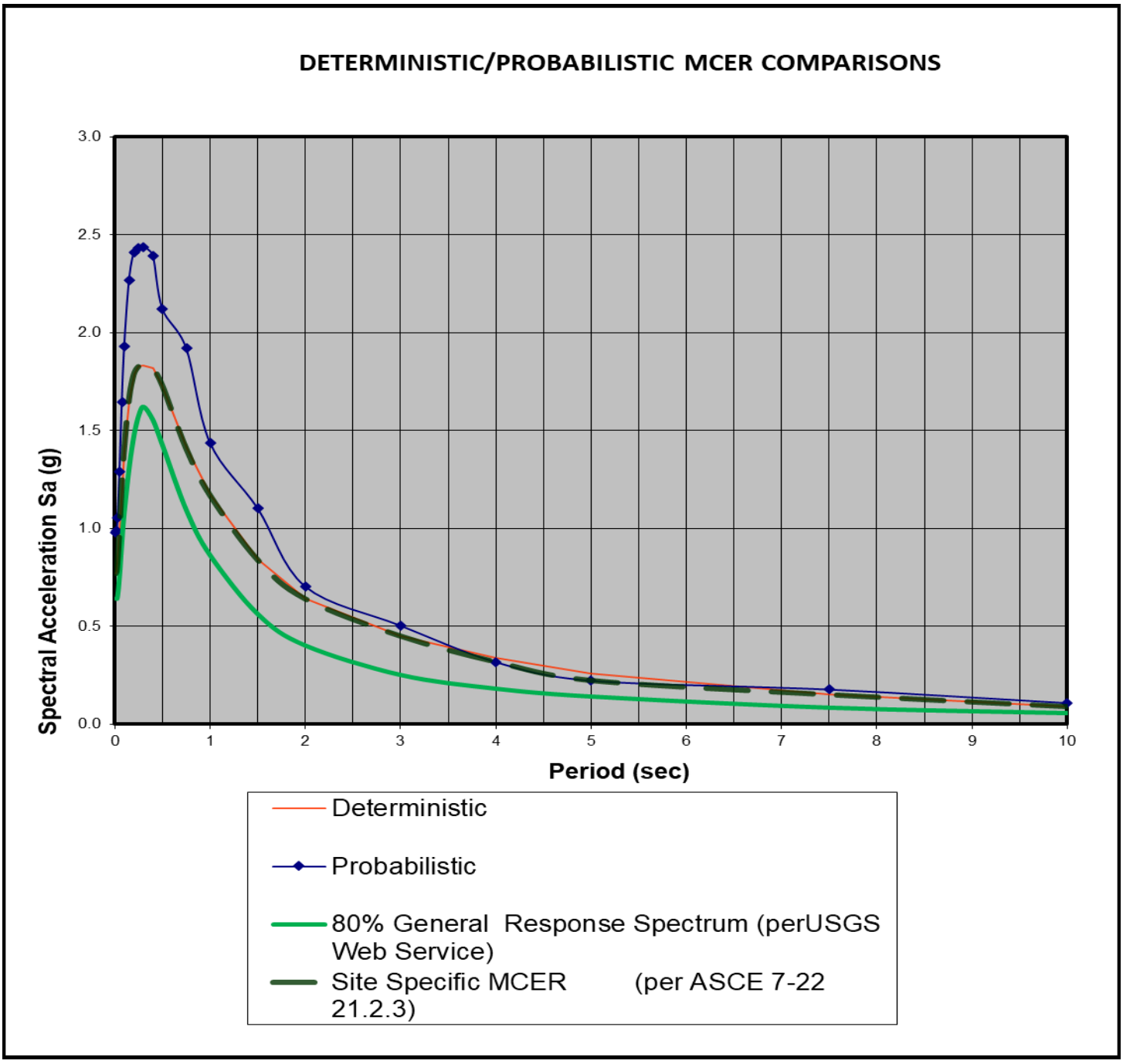
- **Site Specific MCE_R (ASCE 7-22 Section 21.2.3)-**

T[illegible] [illegible]i[illegible]i[illegible] M[illegible]E$_R$ [illegible]r[illegible]r[illegible]n[illegible] [illegible]r[illegible]i[illegible]n [illegible]n[illegible] [illegible]ri[illegible]d[illegible]M[illegible] [illegible]n [illegible] [illegible]r [illegible] [illegible] [illegible]r[illegible]r[illegible]n[illegible] [illegible]r[illegible]i[illegible]n[illegible] [illegible]r[illegible] [illegible] [illegible]r[illegible]i[illegible]i[illegible] [illegible]r[illegible]nd [illegible] [illegible]i[illegible]n[illegible] [illegible]i[illegible]n [illegible]nd [illegible] d[illegible]r[illegible]ini[illegible]i[illegible] [illegible]r[illegible]nd [illegible] [illegible]i[illegible]n[illegible] [illegible]i[illegible]n [illegible] T[illegible] r[illegible]in[illegible] [illegible] [illegible]n[illegible] [illegible]n [illegible] [illegible] [illegible]n[illegible]r[illegible]R[illegible]n[illegible] [illegible]r[illegible] [illegible]in[illegible]d [illegible] [illegible] [illegible] [illegible] [illegible]r[illegible]i[illegible]

Compare Deterministic MCER Values (Sa) with Probabilistic MCER Values (Sa) per 21.2.3)

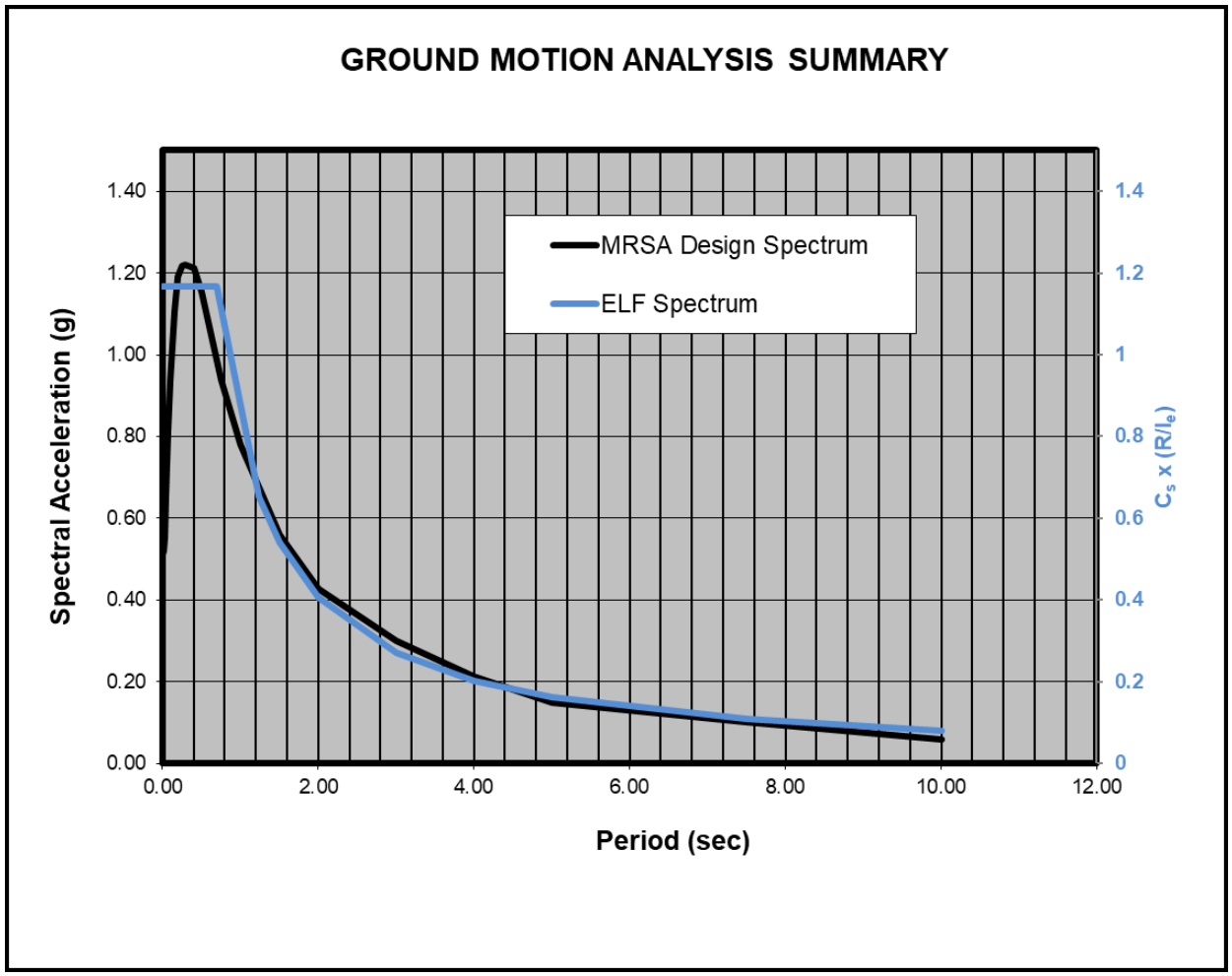
[illegible]ri[illegible]d	D[illegible]r[illegible]ini[illegible]i[illegible]	[illegible]r[illegible]i[illegible]i[illegible]i[illegible]	[illegible]r [illegible]i[illegible] [illegible]i[illegible] M[illegible]E$_R$[illegible]	[illegible]rnin[illegible] M[illegible]d		[illegible] [illegible]n[illegible]r[illegible] R[illegible]n[illegible] [illegible]r[illegible] [illegible]r [illegible] [illegible] [illegible]r[illegible]i[illegible]	[illegible]i[illegible] [illegible]i[illegible]i[illegible] M[illegible]E$_R$ [illegible]r A[illegible]E [illegible] [illegible]
T	M[illegible]E$_R$	M[illegible]E$_R$					
[illegible]	0.77	0.98	0.77	Deterministic Governs		0.64	0.77
[illegible]	0.78	0.99	0.78	Deterministic Governs		0.64	0.78
[illegible]	0.83	1.05	0.83	Deterministic Governs		0.67	0.83
[illegible]	0.98	1.29	0.98	Deterministic Governs		0.78	0.98
[illegible]	1.22	1.65	1.22	Deterministic Governs		0.96	1.22
[illegible]	1.41	1.93	1.41	Deterministic Governs		1.10	1.41
[illegible]	1.66	2.27	1.66	Deterministic Governs		1.32	1.66
[illegible]	1.79	2.41	1.79	Deterministic Governs		1.48	1.79
[illegible]	1.83	2.43	1.83	Deterministic Governs		1.58	1.83
[illegible]	1.83	2.44	1.83	Deterministic Governs		1.62	1.83
[illegible]	1.82	2.39	1.82	Deterministic Governs		1.55	1.82
[illegible]	1.73	2.12	1.73	Deterministic Governs		1.42	1.73
[illegible]	1.41	1.92	1.41	Deterministic Governs		1.09	1.41
[illegible]	1.17	1.44	1.17	Deterministic Governs		0.86	1.17
[illegible]	0.84	1.10	0.84	Deterministic Governs		0.56	0.84
[illegible]	0.64	0.70	0.64	Deterministic Governs		0.40	0.64
[illegible]	0.45	0.50	0.45	Deterministic Governs		0.25	0.45
[illegible]	0.34	0.32	0.32	Probabilistic Governs		0.18	0.32
[illegible]	0.26	0.22	0.22	Probabilistic Governs		0.14	0.22
[illegible]	0.15	0.18	0.15	Deterministic Governs		0.08	0.15
[illegible]	0.09	0.11	0.09	Deterministic Governs		0.06	0.09

T□□□□ □□□ □□ri□n□ □r□ □□□□□d in □□□ □□□□□in□ di□□r□□ □



- ***Design Response Spectrum (ASCE 7-22 Section 21.3)-***

  In □□□□rd□n□□ □i□□ □□□□i□n □□□□□□□□□ D□□i□n R□□□□□n□□ □□□□□r□□ □ □□□ d□□□□□□□□d □□ □□□□ □□□□□□□in□ □□□□□i□n □ □□ □ □□□□□□M□ □□□□r□ □□M i□ □□□□ M□E$_R$ □□□□□r□□r□□□□n□□ □□□□□□□r□i□n □□□□in□d □r□□ □□□□i□n □□□□□ □r □□□□□□ T□□□□ □r□ □□□□□□□d □nd □□□□ □□r□d □i□□ □□□□ □□□□□□ □ □□□ □□□□□r□□ □□□□□□□ in □□□□ □□□□□□□in□ di□□r□□ □



- ***Design Acceleration Parameters (ASCE 7-22 Section 21.4)-***

  □ □□r□ □□□□ □i□□□□□□□□i□□ □r□□□□d□r□ i□ □□□□d □□ d□□□r□in□ □□□□ d□□i□n □r□□nd □ □□i□n in □□□□□rd□n□□ □i□□ □□□□i□n □□□□□□□□□ □□r□□ □□□r □D□ □□□□□□□□□□□in□d □r□□ □□□□ □i□□□□□□□□i□□ □□□□□□r□ □□□□ □□ri□d □□□ □□□□ □□□□□□□□□□□□□□□□i□□□□□□n□□□□ □□□□□n □□□□ □□□n □□ □□r□□n□□□□□□□ □□□□□ □□□□□□r□□□□□□□□□□r□i□n□□□□□ □n□ □i□□in □□□□ r□n□□ □r□□ □□□□ □□□ □ □□□□□□nd□□ T□□ □□r□□ □□□□r □D□ □□□□□□□□□□ □□□□□n □□ □□□□ □r□□□□r □□ □□□□ □r□d□□□□ □□□□□ □T □□r □□ri□d□ □□□□□□□n □ □nd □ □□□□□□nd□ □□r □i□□ □ □□□□□ **CD**□□□□□n□□□□□□□ □□□□n □□□□□ □□□□□□ □□□□□□ □□

- **Site-Specific MCE_G Peak Ground Accelerations (ASCE 7-22 Section 21.5)**-

T[illegible] r[illegible]i[illegible]i[illegible] [illegible]ri[illegible] [illegible]n [illegible] r[illegible]nd [illegible]r[illegible]i[illegible]n [illegible] [illegible]r[illegible]n[illegible]r[illegible]i[illegible] [illegible]d[illegible]n[illegible] [illegible]i[illegible]in [illegible] [illegible]r [illegible]ri[illegible]d[illegible] [illegible] [illegible]d [illegible] T[illegible] d[illegible]r[illegible]ini[illegible]i[illegible] [illegible]ri[illegible] [illegible]n [illegible] r[illegible]nd [illegible]r[illegible]i[illegible]n [illegible]r [illegible]n[illegible]ri[illegible] [illegible]r[illegible]i[illegible]n [illegible]r[illegible]i[illegible]n[illegible] [illegible]d [illegible] T[illegible] [illegible]i[illegible]i[illegible] M[illegible]E[illegible] [illegible] r[illegible]nd [illegible]r[illegible]i[illegible]n [illegible] [illegible]d [illegible] **0.76g**[illegible] [illegible]i[illegible] [illegible] d[illegible]r[illegible]in[illegible]d [illegible] [illegible]in[illegible] [illegible] [illegible]r [illegible] [illegible]r[illegible]i[illegible]i[illegible] [illegible]r [illegible] d[illegible]r[illegible]ini[illegible]i[illegible] [illegible]ri[illegible] [illegible]n [illegible] r[illegible]nd [illegible]r[illegible]i[illegible]n[illegible] [illegible]n [illegible]n [illegible] [illegible]n [illegible] [illegible]r[illegible]n[illegible] A_M [illegible]

SEISMIC DESIGN PARAMETERS SUMMARY

Project: Norco Community College
Project #: 254169-1
Date: 10/29/2025

Lattitude: 33.9168
Longitude: -117.5690

MAPPED ACCELERATION PARAMETERS per ASCE 7-22, Chapter 22

S_s=	2.06	Figure 22-1
S_1=	0.65	Figure 22-2

Site Class per Table 20.3-1

Site Class=	CD

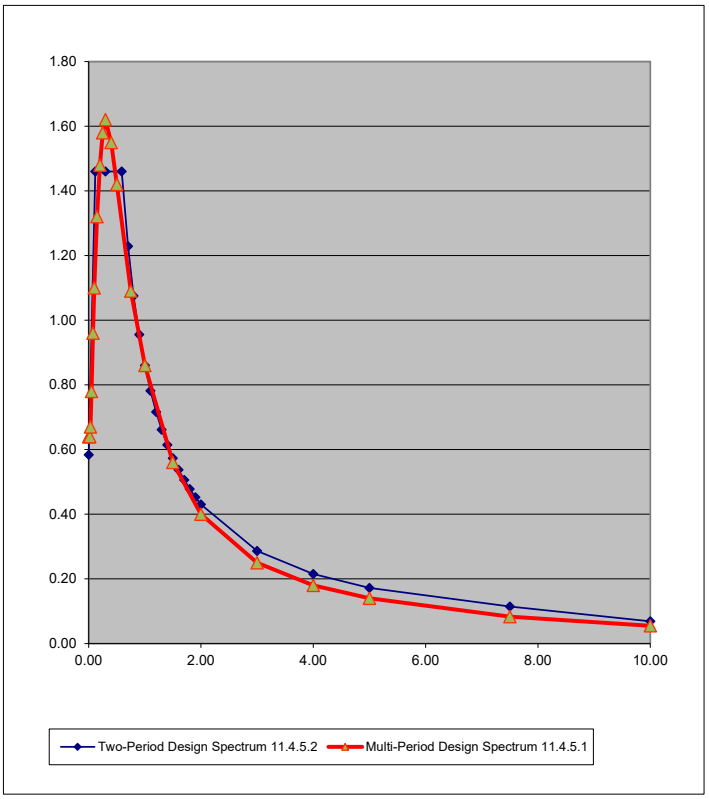
Mapped Design Spectral Response Acceleration Parameters

S_{Ms}=	2.19	Section 11.4-3
S_{M1}=	1.29	Section 11.4-3

S_{DS}=	1.460	Equation 11.4-1
S_{D1}=	0.860	Equation 11.4-2

Period (T)	Two-Period Design Spectrum 11.4.5.2
0.00	0.58
0.12	1.46
0.30	1.46
0.59	1.46
0.70	1.23
0.80	1.08
0.90	0.96
1.00	0.86
1.10	0.78
1.20	0.72
1.30	0.66
1.40	0.61
1.50	0.57
1.60	0.54
1.70	0.51
1.80	0.48
1.90	0.45
2.00	0.43
3.00	0.29
4.00	0.22
5.00	0.17
7.50	0.11
10.00	0.07

Period (T)	Multi-Period Design Spectrum 11.4.5.1
0.00	0.64
0.01	0.64
0.02	0.64
0.03	0.67
0.05	0.78
0.08	0.96
0.10	1.10
0.15	1.32
0.20	1.48
0.25	1.58
0.30	1.62
0.40	1.55
0.50	1.42
0.75	1.09
1.00	0.86
1.50	0.56
2.00	0.40
3.00	0.25
4.00	0.18
5.00	0.14
7.50	0.08
10.00	0.06



T_0=	0.118	sec	ASCE7-22 -Section 11.4.5.2
T_S=	0.589	sec	ASCE7-22 -Section 11.4.5.2
T_L=	8.000	sec	From Fig 22-14 through 22-17
PGA_M	0.800	g	

ASCE 7-22 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

Use Maximum Rotated Horizontal Component?* (Y/N) **Y**

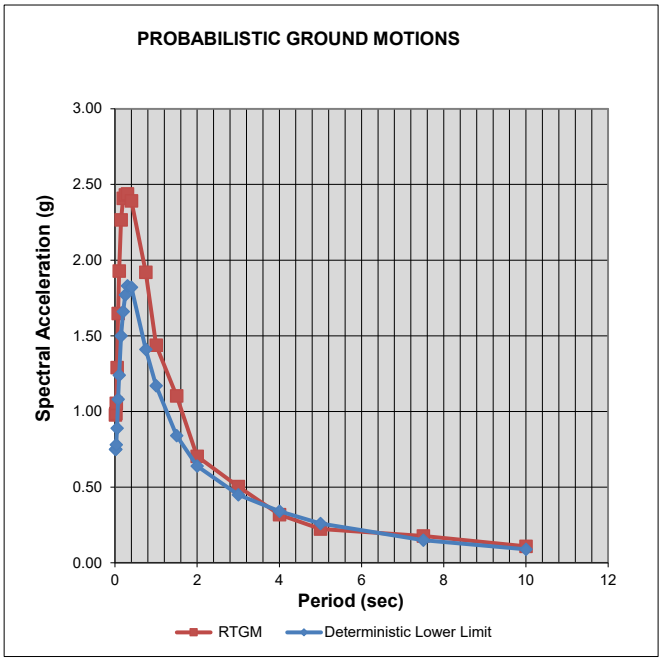
Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships
UCERF3 Fault Model 3.1

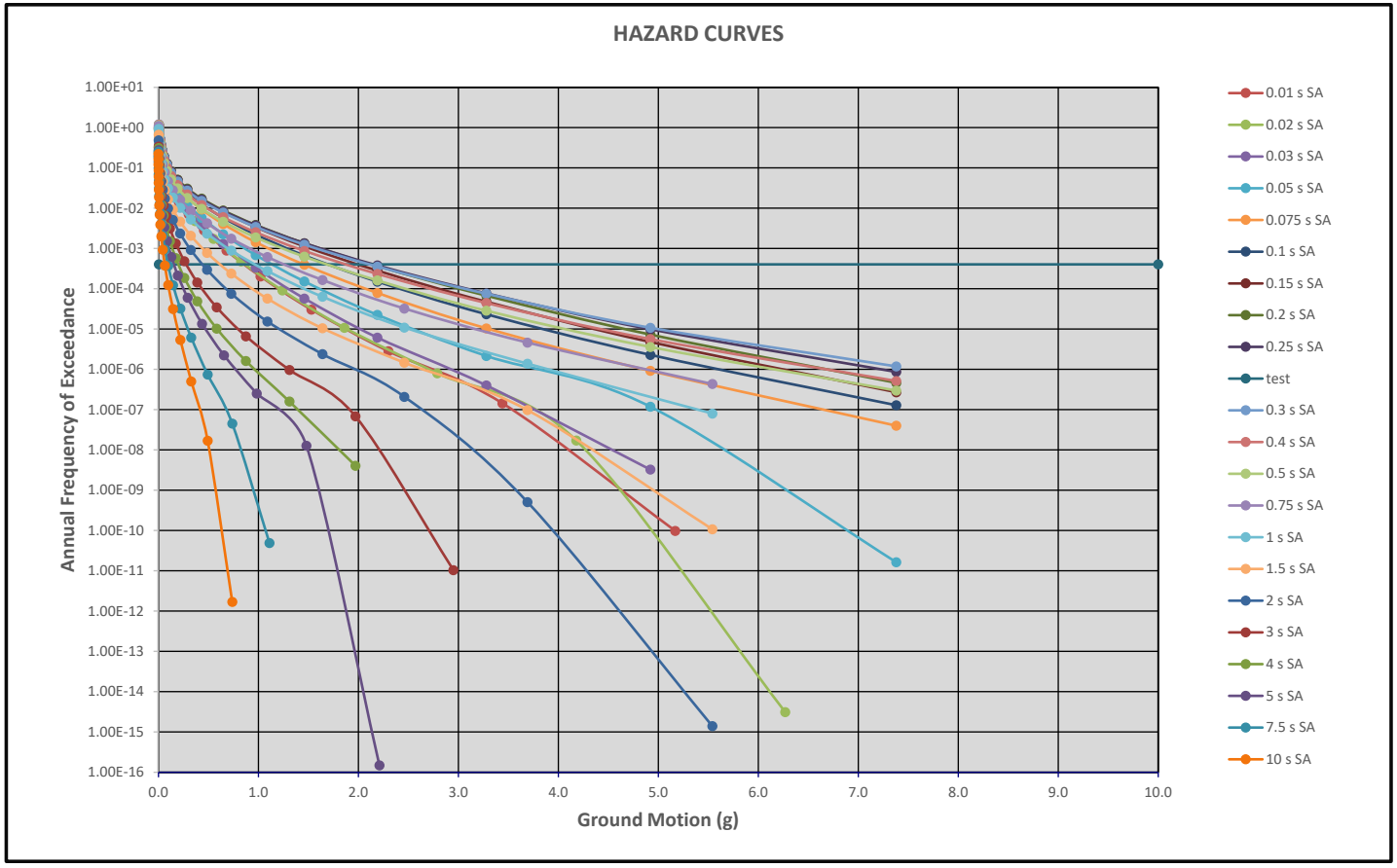
PROBABILISTIC MCER per 21.2.1.1
Risk Coefficients Computed by USGS RTGM Calculator

2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-22

T	UHGM	Risk Coefficient	RTGM	Deterministic Lower Limit	Is RTGM<LL?
0.01	1.01	0.96	0.98	0.75	No
0.02	1.03	0.96	0.99	0.75	No
0.03	1.10	0.96	1.05	0.78	No
0.05	1.35	0.96	1.29	0.89	No
0.08	1.74	0.95	1.65	1.08	No
0.10	2.01	0.96	1.93	1.24	No
0.15	2.37	0.96	2.27	1.50	No
0.20	2.54	0.95	2.41	1.66	No
0.25	2.58	0.99	2.43	1.77	No
0.30	2.59	0.94	2.44	1.83	No
0.40	2.55	0.94	2.39	1.82	No
0.75	2.06	0.93	1.92	1.41	No
1.00	1.55	0.93	1.44	1.17	No
1.50	1.20	0.92	1.10	0.84	No
2.00	0.76	0.92	0.70	0.64	No
3.00	0.55	0.92	0.50	0.45	No
4.00	0.35	0.91	0.32	0.34	Yes
5.00	0.25	0.91	0.22	0.26	Yes
7.50	0.20	0.90	0.18	0.15	No
10.00	0.12	0.89	0.11	0.09	No

S_s=	2.54	2.41
S_1=	1.55	1.44
PGA	0.81 g	




HAZARD CURVES
Annual Frequency of Exceedance
1.00E+01
1.00E+00
1.00E-01
1.00E-02
1.00E-03
1.00E-04
1.00E-05
1.00E-06
1.00E-07
1.00E-08
1.00E-09
1.00E-10
1.00E-11
1.00E-12
1.00E-13
1.00E-14
1.00E-15
1.00E-16
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
Ground Motion (g)
0.01 s SA
0.02 s SA
0.03 s SA
0.05 s SA
0.075 s SA
0.1 s SA
0.15 s SA
0.2 s SA
0.25 s SA
test
0.3 s SA
0.4 s SA
0.5 s SA
0.75 s SA
1 s SA
1.5 s SA
2 s SA
3 s SA
4 s SA
5 s SA
7.5 s SA
10 s SA

DISGGREGATION ANALYSES per Section 21.2.2

TABLE OF SCENARIO EARTHQUAKES AND HAZARD CONTRIBUTIONS

	Elsinore (Glen Ivy) (0)		Whittier [1] (0)		Chino [1] (4)		San Jacinto (San Bernardino) (3)		Fontana (seismicity) (2)		San Andreas (San Bernardino, north) (3)		Lytle Creek (San Jacinto, connector) (2)		Mill Creek (San Andreas, north branch) (0)		Cucamonga (3)	
Period $T_{(s)}$	#REF!	#REF!	M	#REF!	M	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!
0.01	6.5	0.34	7.4	0.25	6.8	0.15	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.02	6.5	0.35	7.4	0.25	6.8	0.15	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.03	6.5	0.36	7.4	0.25	6.8	0.15	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.05	6.5	0.38	7.4	0.23	6.8	0.14	8.1	0.04	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.075	6.5	0.39	7.4	0.21	6.8	0.14	8.1	0.04	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.1	6.5	0.40	7.4	0.21	6.8	0.14	8.1	0.03	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.15	6.5	0.39	7.4	0.21	6.8	0.14	8.1	0.03	6.6	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.2	6.5	0.37	7.4	0.22	6.8	0.14	8.1	0.04	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.25	6.5	0.35	7.4	0.23	6.8	0.14	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.3	6.5	0.34	7.4	0.23	6.8	0.14	8.1	0.05	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.4	6.5	0.33	7.4	0.24	6.8	0.14	8.1	0.05	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.5	6.5	0.32	7.4	0.24	6.8	0.14	8.1	0.06	6.6	0.04	8.0	0.04	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
0.75	6.5	0.30	7.4	0.25	6.8	0.13	8.1	0.07	6.6	0.04	8.0	0.05	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1	6.5	0.29	7.5	0.25	6.8	0.13	8.1	0.07	6.6	0.04	8.0	0.06	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1.5	6.6	0.23	7.5	0.26	6.9	0.12	8.1	0.10	6.6	0.04	8.0	0.08	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
2	6.6	0.20	7.5	0.26	6.9	0.11	8.1	0.12	6.6	0.03	8.0	0.09	8.1	0.02	#N/A	#N/A	#N/A	#N/A
3	6.7	0.14	7.6	0.26	7.0	0.00	8.1	0.16	6.6	#N/A	8.0	0.12	8.1	0.00	8.1	0.00	7.8	0.00
4	6.8	0.09	7.6	0.26	7.1	0.08	8.1	0.19	#N/A	#N/A	8.0	0.15	8.1	0.04	8.1	0.03	7.8	0.03
5	6.9	0.06	7.6	0.24	7.1	0.07	8.1	0.22	#N/A	#N/A	8.0	0.17	8.1	0.05	8.1	0.04	7.8	0.03
7.5	7.3	0.03	7.6	0.19	7.3	0.04	8.1	0.26	#N/A	#N/A	8.0	0.22	8.1	0.05	8.1	0.05	7.8	0.03
10	#N/A	#N/A	7.6	0.16	7.4	0.03	8.1	0.27	#N/A	#N/A	8.0	0.24	8.1	0.06	8.1	0.05	7.8	0.03
PGA	6.5	0.34	7.4	0.25	6.8	0.15	8.1	0.05	6.6	0.05	8.0	0.03	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

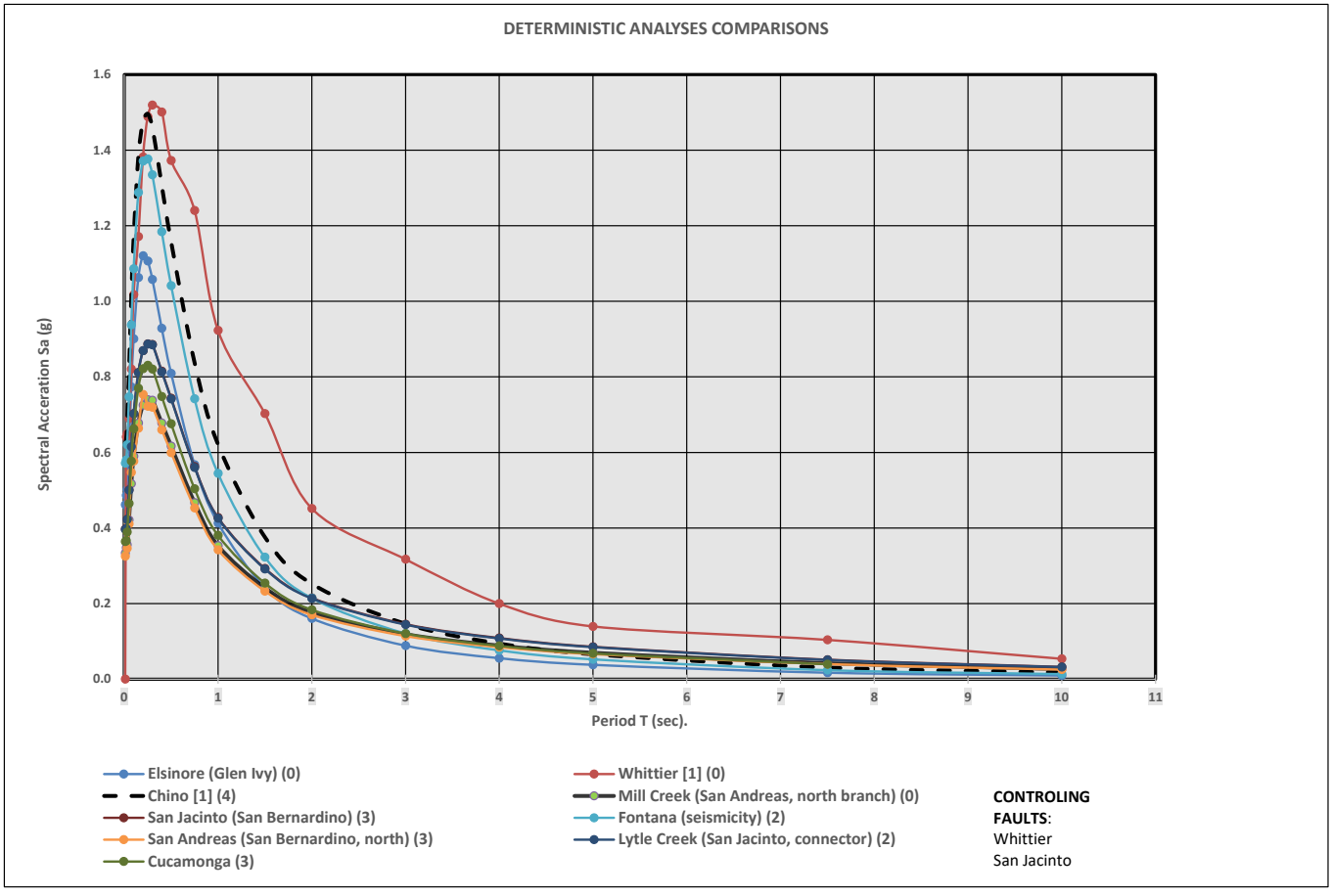
Fault	R_{RUP}(km)
Elsinore (Glen Ivy) (0)	10.1
Whittier [1] (0)	8.9
Chino [1] (4)	7.3
San Jacinto (San Bernar	30.5
Fontana (seismicity) (2)	7.8
San Andreas (San Berna	38.59
Lytle Creek (San Jacinto,	30.16
Mill Creek (San Andreas,	38.48
Cucamonga (3)	28.45

The Probabilistic Analyses revealed 9 faults contributing more than 10% of the largest contributor to the seismic hazard. These were considered in the Deterministic Analyses.

DETERMINISTIC MCE_R per 21.2.2

Input Parameters:

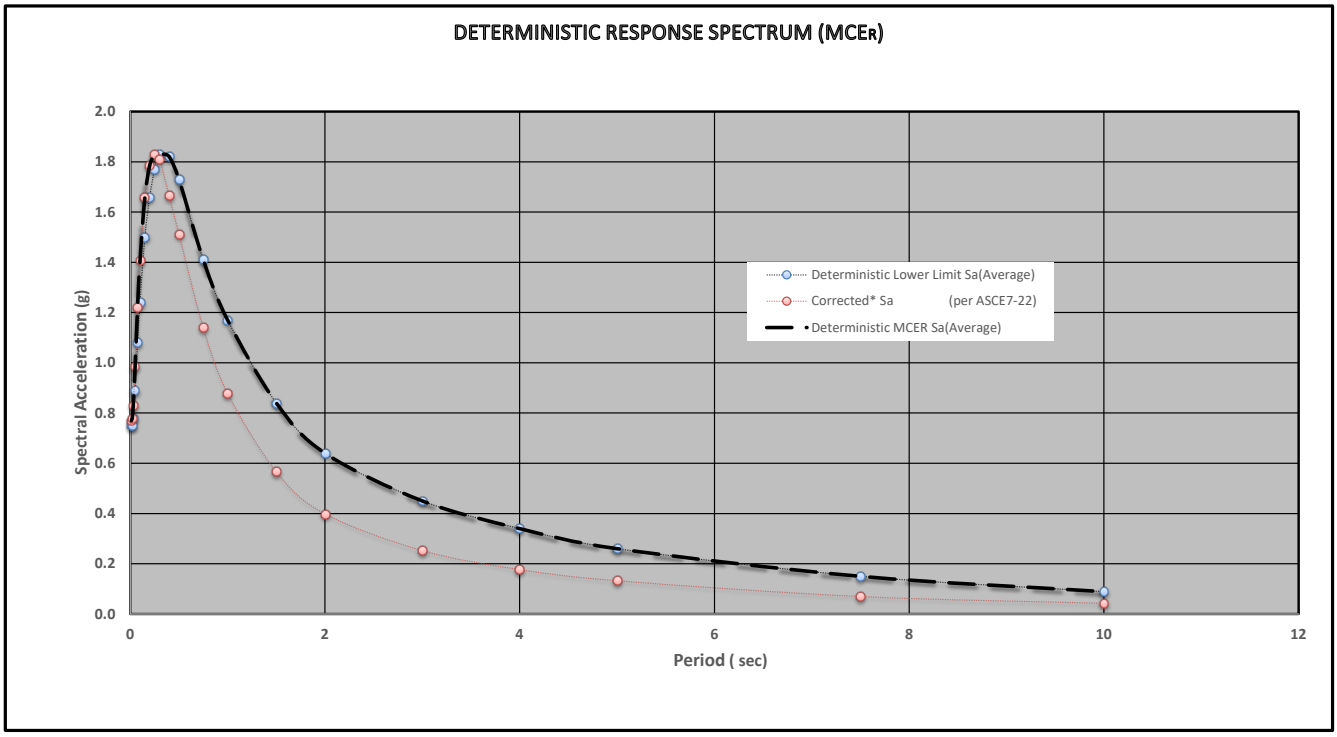
Input Parameters / Fault	Elsinore (Glen Ivy) (0)	Whittier [1] (0)	Chino [1] (4)	San Jacinto (San Bernardino) (3)	Fontana (seismicity) (2)	San Andreas (San Bernardino, north) (3)	Lytle Creek (San Jacinto, connector) (2)	Mill Creek (San Andreas, north branch) (0)	Cucamonga (3)
M	6.49	7.39	6.79	8.1	6.61	8.0	8.1	8.1	7.8
R_{RUP}	10.05	8.87	7.25	30.45	7.8	38.6	30.2	38.5	28.5
R_{JB}	10.05	8.87	7.25	30.45	7.8	38.6	30.2	38.5	28.5
Rx	10.05	8.87	7.25	30.45	7.8	38.6	30.2	38.5	28.5
R_{y0}	0	0	0	0	6.4	0.0	0.0	0.0	0.0
U	0	0	0	0	0	0	0	0	0
F_{RV}	0	0	0	0	0	0	0	0	0
F_{NM}	0	0	0	0	0	0	0	0	0
F_{HW}	0	0	0	0	0	0	0	0	0
Z_{TOR}	0	0	0	0	0	0.0	0.0	0.0	0.0
δ	90	70	55	90	80	90	90	76	45
V_{S30}	434.1	434.1	434.1	434.1	434.1	434.1	434.1	434.1	434.1
$F_{Measured}$	1	1	1	0	0	0.0	0.0	0.0	0.0
$Z_{1.0}$	0.15	0.15	0.15	0.15	0.15	0.07	0.07	0.07	0.07
$Z_{2.5}$	0.35	0.35	0.35	0.35	0.35	0.4	0.4	0.4	0.4
Site Class	CD	CD	CD	CD	CD	CD	CD	CD	CD
W (km)	15.0	13.2	14.8	16.1	16.6	12.8	16.1	18.8	18.4
F_{AS}	0	0	0	0	0	0	0	0	0
σ	1	1	1	1	1	1	1	1	1



Deterministic Summary - Section 21.2.2

T	Elsinore (Glen Ivy) (0)	Whittier [1] (0)	Chino [1] (4)	San Jacinto (San Bernardino) (3)	Fontana (seismicity) (2)	San Andreas (San Bernardino, north) (3)	Lytle Creek (San Jacinto, connector) (2)	Mill Creek (San Andreas, north branch) (0)	Cucamonga (3)	Maximum S_a (Average)	*Corrected* S_a (per ASCE7-22)*	*Deterministic Lower Limit $S_{a(Average)}$*	*Deterministic MCE_R $S_{a(Average)}$*	Controlling Fault
0.010	0.46	0.64	0.62	0.40	0.57	0.33	0.40	0.33	0.36	0.64	0.77	0.75	0.77	Whittier [1] (0)
0.020	0.49	0.65	0.63	0.40	0.58	0.39	0.40	0.35	0.37	0.65	0.78	0.75	0.78	Whittier [1] (0)
0.030	0.50	0.69	0.68	0.42	0.62	0.35	0.42	0.36	0.39	0.69	0.83	0.78	0.83	Whittier [1] (0)
0.050	0.61	0.82	0.81	0.50	0.75	0.41	0.50	0.42	0.47	0.82	0.98	0.89	0.98	Whittier [1] (0)
0.075	0.77	1.02	1.01	0.62	0.94	0.55	0.62	0.52	0.58	1.02	1.22	1.08	1.22	Whittier [1] (0)
0.100	0.90	1.17	1.17	0.70	1.09	0.58	0.70	0.59	0.66	1.17	1.41	1.24	1.41	Whittier [1] (0)
0.150	1.06	1.38	1.38	0.81	1.29	0.66	0.81	0.68	0.77	1.38	1.66	1.50	1.66	Whittier [1] (0)
0.200	1.12	1.49	1.48	0.87	1.37	0.75	0.87	0.72	0.82	1.49	1.79	1.66	1.79	Whittier [1] (0)
0.250	1.11	1.52	1.50	0.89	1.38	0.72	0.89	0.74	0.83	1.52	1.83	1.77	1.83	Whittier [1] (0)
0.300	1.06	1.50	1.46	0.89	1.33	0.72	0.88	0.74	0.82	1.50	1.81	1.83	1.83	Whittier [1] (0)
0.400	0.93	1.37	1.31	0.82	1.18	0.66	0.81	0.68	0.75	1.37	1.66	1.82	1.82	Whittier [1] (0)
0.500	0.81	1.24	1.16	0.74	1.04	0.60	0.74	0.62	0.68	1.24	1.51	1.73	1.73	Whittier [1] (0)
0.750	0.57	0.92	0.84	0.56	0.74	0.45	0.56	0.47	0.50	0.92	1.14	1.41	1.41	Whittier [1] (0)
1.000	0.41	0.70	0.62	0.43	0.54	0.34	0.43	0.35	0.38	0.70	0.88	1.17	1.17	Whittier [1] (0)
1.500	0.24	0.45	0.38	0.29	0.32	0.23	0.29	0.24	0.25	0.45	0.57	0.84	0.84	Whittier [1] (0)
2.000	0.16	0.32	0.25	0.22	0.21	0.17	0.21	0.18	0.18	0.32	0.40	0.64	0.64	Whittier [1] (0)
3.000	0.09	0.20	0.15	0.15	0.12	0.11	0.14	0.12	0.12	0.20	0.25	0.45	0.45	Whittier [1] (0)
4.000	0.06	0.14	0.09	0.11	0.08	0.08	0.11	0.09	0.09	0.14	0.18	0.34	0.34	Whittier [1] (0)
5.000	0.04	0.10	0.07	0.09	0.05	0.07	0.08	0.07	0.07	0.10	0.13	0.26	0.26	Whittier [1] (0)
7.500	0.02	0.05	0.03	0.05	0.02	0.04	0.05	0.04	0.04	0.05	0.07	0.15	0.15	Whittier [1] (0)
10.000	0.01	0.03	0.02	0.03	0.01	0.03	0.03	0.03	0.02	0.03	0.04	0.09	0.09	San Jacinto
PGA	0.46	0.64	0.62	0.40	0.57	0.32	0.39	0.33	0.36	0.64	0.76	0.56	0.76	Whittier [1] (0)

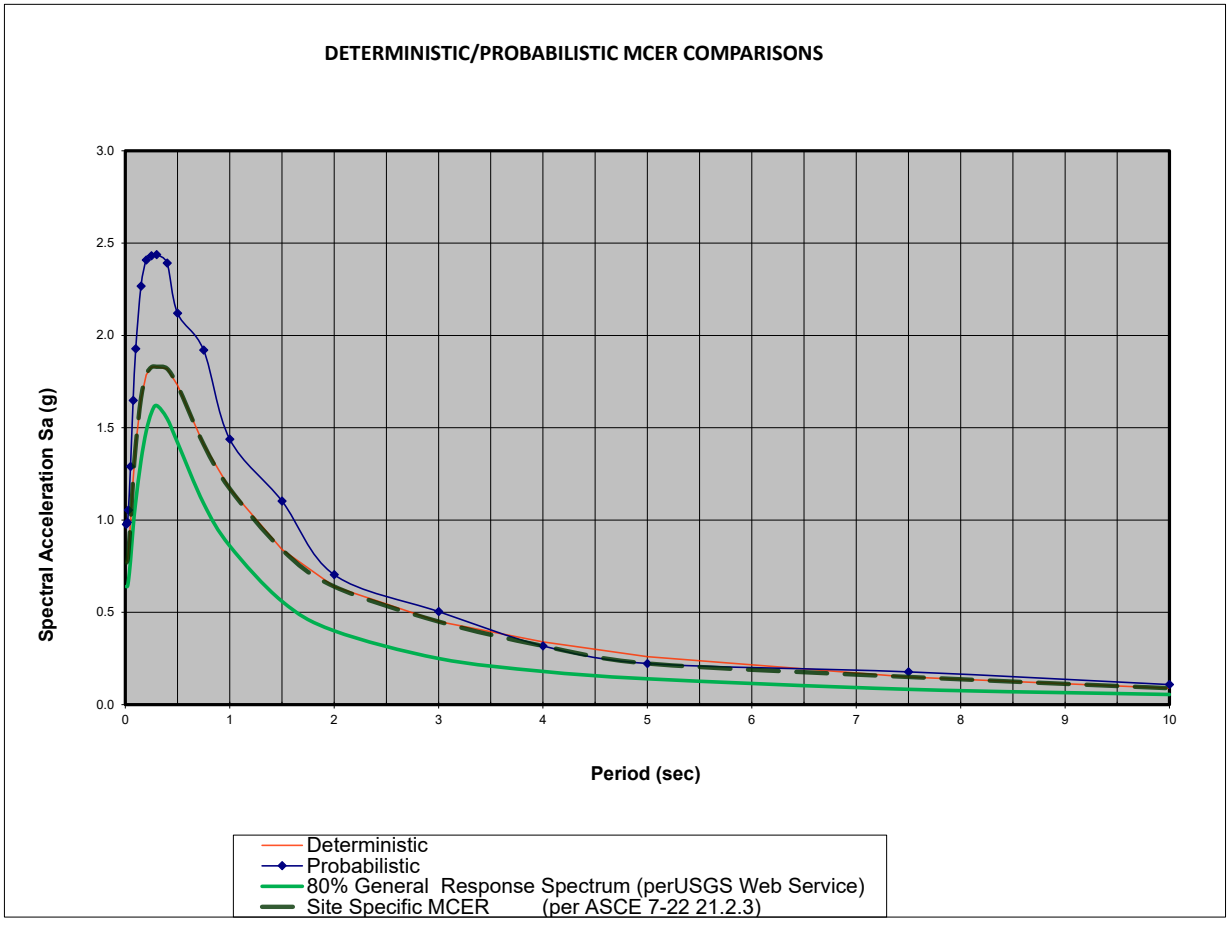
* Correction is the adjustment for Maximum Rotated Value if Applicable



SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific $MCE_{R)}$	Governing Method	80% General Response Spectrum (perUSGS Web Service)	Site Specific MCE_R (per ASCE 7-22 21.2.3)
0.010	0.77	0.98	0.77	Deterministic Governs	0.64	0.77
0.020	0.78	0.99	0.78	Deterministic Governs	0.64	0.78
0.030	0.83	1.05	0.83	Deterministic Governs	0.67	0.83
0.050	0.98	1.29	0.98	Deterministic Governs	0.78	0.98
0.075	1.22	1.65	1.22	Deterministic Governs	0.96	1.22
0.100	1.41	1.93	1.41	Deterministic Governs	1.10	1.41
0.150	1.66	2.27	1.66	Deterministic Governs	1.32	1.66
0.200	1.79	2.41	1.79	Deterministic Governs	1.48	1.79
0.250	1.83	2.43	1.83	Deterministic Governs	1.58	1.83
0.300	1.83	2.44	1.83	Deterministic Governs	1.62	1.83
0.400	1.82	2.39	1.82	Deterministic Governs	1.55	1.82
0.500	1.73	2.12	1.73	Deterministic Governs	1.42	1.73
0.750	1.41	1.92	1.41	Deterministic Governs	1.09	1.41
1.000	1.17	1.44	1.17	Deterministic Governs	0.86	1.17
1.500	0.84	1.10	0.84	Deterministic Governs	0.56	0.84
2.000	0.64	0.70	0.64	Deterministic Governs	0.40	0.64
3.000	0.45	0.50	0.45	Deterministic Governs	0.25	0.45
4.000	0.34	0.32	0.32	ProbabilisticGoverns	0.18	0.32
5.000	0.26	0.22	0.22	ProbabilisticGoverns	0.14	0.22
7.500	0.15	0.18	0.15	Deterministic Governs	0.08	0.15
10.000	0.09	0.11	0.09	Deterministic Governs	0.06	0.09



DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	Design Response Spectrum (2/3*MCE_R per ASCE7-	TXSa
0.01	0.51	
0.02	0.52	
0.03	0.55	
0.05	0.66	
0.08	0.81	
0.10	0.94	
0.15	1.11	
0.20	1.19	
0.25	1.22	
0.30	1.22	
0.40	1.21	
0.50	1.15	
0.75	0.94	
1.00	0.78	0.78
1.50	0.56	0.84
2.00	0.43	0.85
3.00	0.30	0.90
4.00	0.21	0.85
5.00	0.15	0.74
7.50	0.10	
10.00	0.06	

Design Acceleration Parameters per Section 21.4:

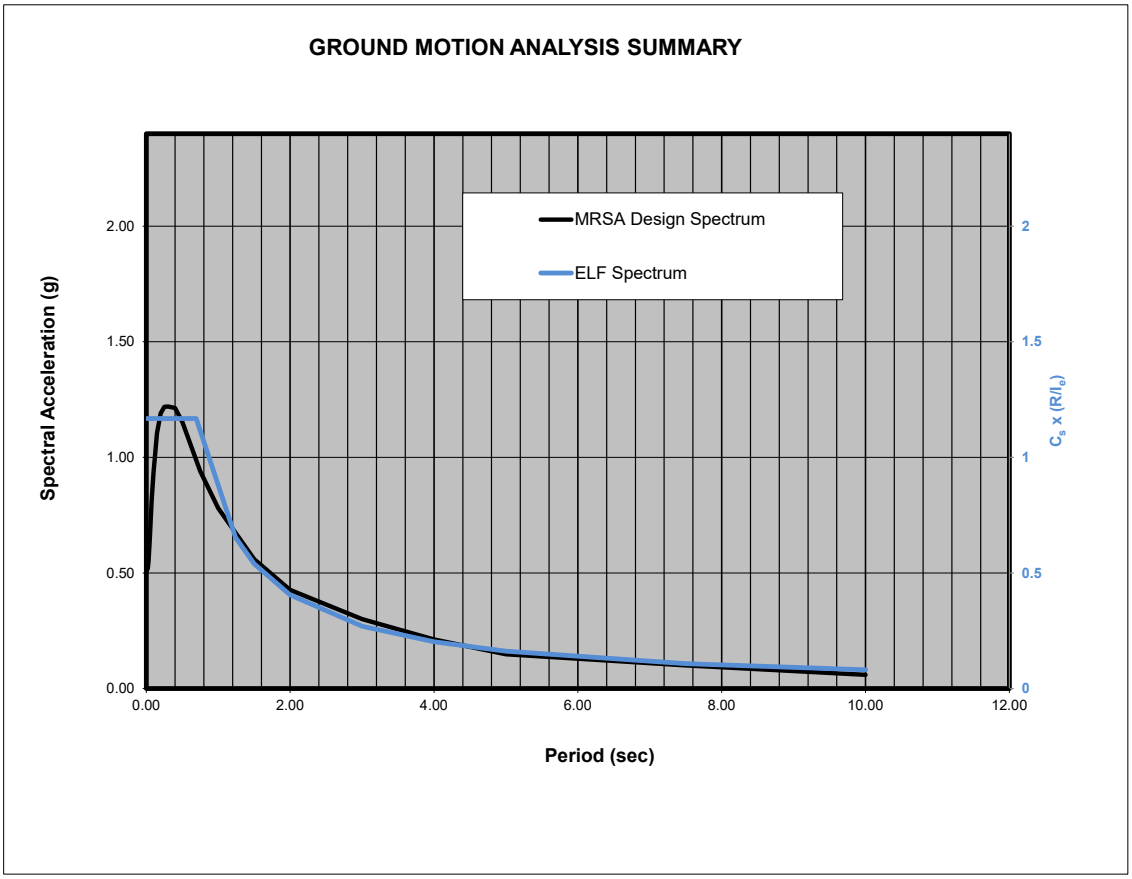
Highest value of S_a for any period exceeding 0.2 sec.=	1.22
90%of Highest Value =	1.10
80% 0f Mapped S_{DS}=	1.17
Max TXsa from T=1s-5s =	0.90
90% 0f Max TxSa =	0.81

S_{DS}=	1.17
S_{D1}=	0.81
Ts =	0.69

S_{MS}=	1.75
S_{M1}=	1.22

PGA Determination:

PGA_M =	0.80	g
PGA_G =	0.56	g (Table 21-2-1)
Deterministic PGA =	0.76	g
Probabilistic PGA =	0.81	g
Lesser of Deterministic/Probabilistic =	0.76	g
80% of PGA_M=	0.64	g
MCE_G PGA=	0.76	g



Deterministic Controls

APPENDIX C

REFERENCES



REFERENCES

American Society of Civil Engineers (ASCE) Mini D i n d nd A i d ri ri r i din nd r r r A E nd rd

American Society for Testing and Materials, Intl. (ASTM) nd rd id r in i i R r i n M d r r In i i n A TM D i n i n D

California Building Standards Commission (CBSC) i rni i din d i rni d R i n Ti r

California's Office of Statewide Health Planning and Development (OSHPD) D i i D i n M T i i n i i r

California State Board for Geologists and Geophysicists D r n n r A ir id in r i R r r En ir n n nd En in rin

Crice, Douglas B. nd d r T ni nd R rin d ri nn i rni

Field, E.H., et al. ni r i rni E r R r F r r i n ERF T Ti Ind nd n M d n Fi R r i R r nd rn i rni E r n r i i n

Geometrics, Inc. r i r™ N r i n nd R r n M n R i i n M r r i n n i rni

Geometrics, Inc. i l r An i r i in r i n nd E r i n

Google™ Earth r r i n i

Iwata, T., Kawase, H., Satoh, T., Kakehi, Y., Irikura, K., Louie, J. N., Abbott, R. E., and Anderson, J. G. Arr Mi r r r M r n R n N d A r E Tr n A r i ni n n F

Louie, J.N. F r r r r i M r D r R r i n Mi r r r Arr *in,* in i i i i A ri

Morton, D.M. and Gray, C.H., Jr. □ □□□□□□ □ □□□□□□□i□ M□□ □□ □□□□ □ □r□n□ N□r□□ □□□□□ □ □□dr□n□□□□ Ri□□r□id□ □nd □□n □□rn□rdin□ □ □□□n□i□□□□ □ □i□□□rni□□ □□□□□□ □□□ □□□□□□□□□ □□□□□□ □□□ □□□ □□n□Fi□□ R□□□r□□□□□□□□

Okada, H. □ □□□□□□ T□□ Mi□r□□r□□□ □r □□r□□□□ M□□□□□d□ □□□i□□□ □□ E□□□□r□i□n □ □□□□□□□□i□i□□□□□□ □□□□□□□□i□□□M□n□□r□□□ □□ri□□N□□ □□r □□□□□□□□ □□□□

Park, C.B, Milner, R.D., Rynden, N., Xia, J., and Ivanov, J. □□□□□□□□□□ □□□ □in□d □□□□ □□ A□i□□ □nd □□□□□j□□ □□r□□□□□ □ □□□□□□ *in* □□□□rn□□ □□ En□ir□n□ □n□□□□ □nd En□in□□rin□ □ □□□□□□□□i□□□□□ □□□□□ □ □□□□l□□□□□ □□□□□□□□□□□□□□□□□□

Saito, M. □□□□□□□□ □ □□□ □□□□□□i□n□ □□R□□□□□□i□i□□ □nd □□r□□□□□ □ □□□□ Di□□□□r□i□n □□r□□□□ □□r □□□□□r□d M□di□□□□□□nd □ □□□□ □nd □□ □ □□□□□□□□□□□□□□□□□□

APPENDIX E

GENERAL EARTHWORK GRADING GUIDELINES

Proposed Library Learning Resource Center and Student Services Building
Norco College
Norco, Riverside County, California
Project No. 4400-CR

GEOTEK

GENERAL GRADING GUIDELINES

The 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, CBC 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, and 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 aspects 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 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 E-1, E-2 and E-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 backhoe 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 Plates E-1 and E-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 E-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 four (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 E-2, E-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 E-1, E-2, and E-3).
2. Fill over cut slopes should be constructed in the following manner:
   a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
   b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction) and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
   c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate E-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 E-2.
4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate E-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 E-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 E-5 shows a schematic of buttress construction.

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

Lot Capping

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

ROCK PLACEMENT AND ROCK FILL GUIDELINES

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 inches) should be placed in windrows.
    a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.

b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).

c) The maximum rock size allowed in windrows is four feet

2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate E-4).

3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.

4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.

a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.

b) The over size rock trenches should be no closer together than 15 feet from any slope face.

c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.

d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

Structural Rock Fills

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

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may affect foundation design.

2. Rock fills are required to be placed in horizontal layers that should **not exceed two feet in thickness, or the maximum rock size present, which ever is less**. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:

a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill,

b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,

c) the portion of the rock above grade is covered with a second lift.

3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

Compaction Procedures

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

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

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

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

4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:

a) the general segregation of rock size,
b) for any unfilled spaces between the large blocks, and
c) the matrix compaction and moisture content.

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

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

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

Piping Potential and Filter Blankets

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

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

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

Subdrainage

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

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

Monitoring

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

UTILITY TRENCH CONSTRUCTION AND BACKFILL

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

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

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.

4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractor's procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractor's attention.

JOB SAFETY

General

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

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

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

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

Test Pits Location, Orientation and Clearance

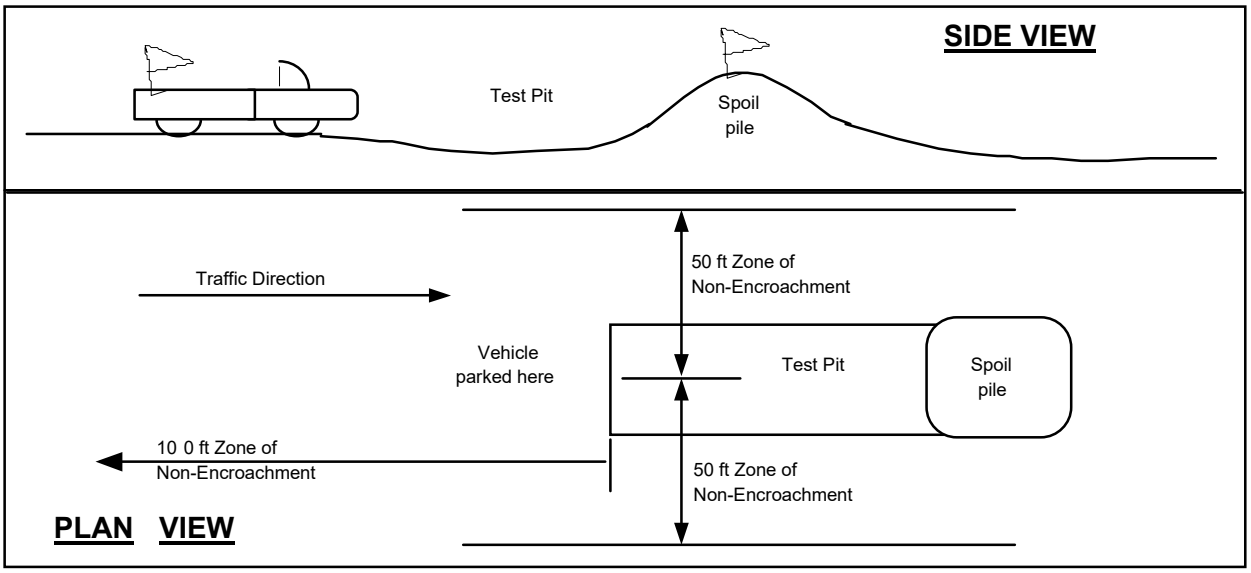
The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading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

contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

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

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

TEST PIT SAFETY PLAN



Slope Tests

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

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

Trench Safety

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

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

Our personnel are directed not to enter any excavation which;

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

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

Procedures

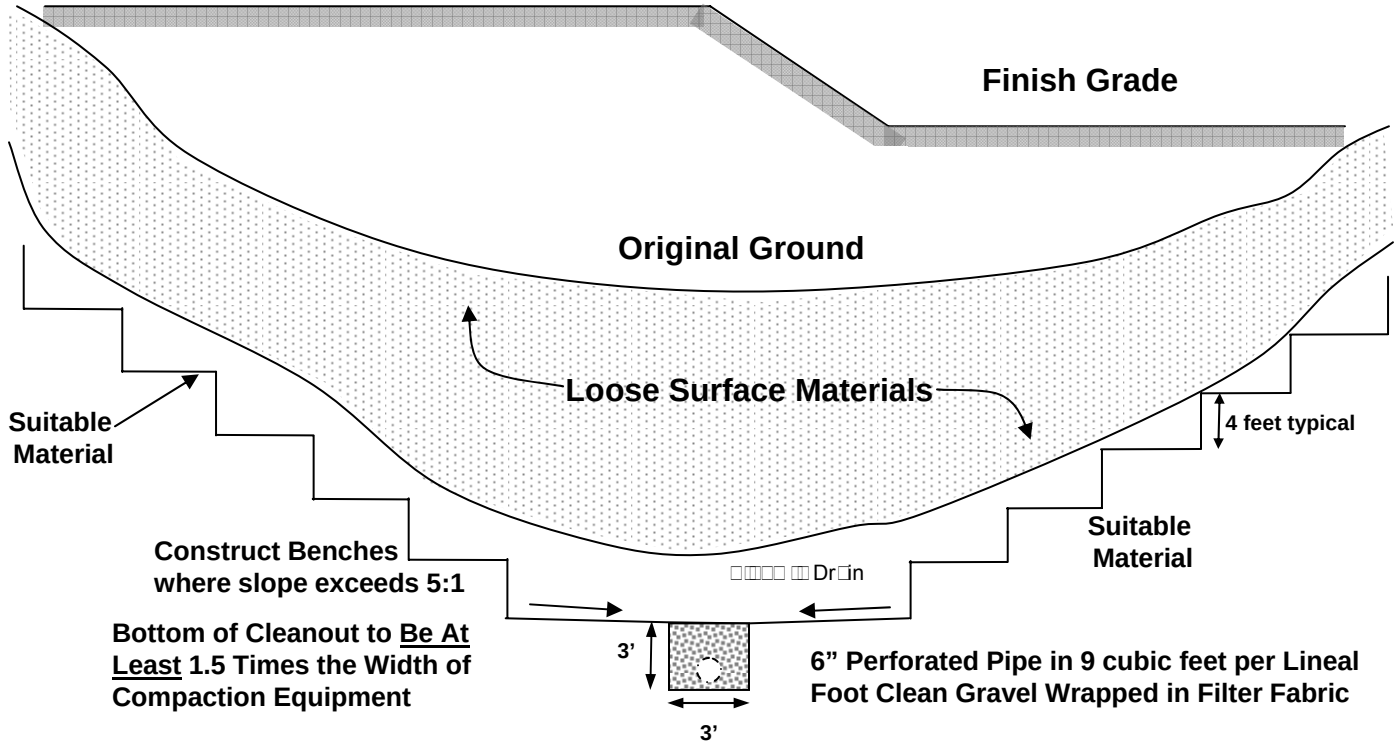
In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by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

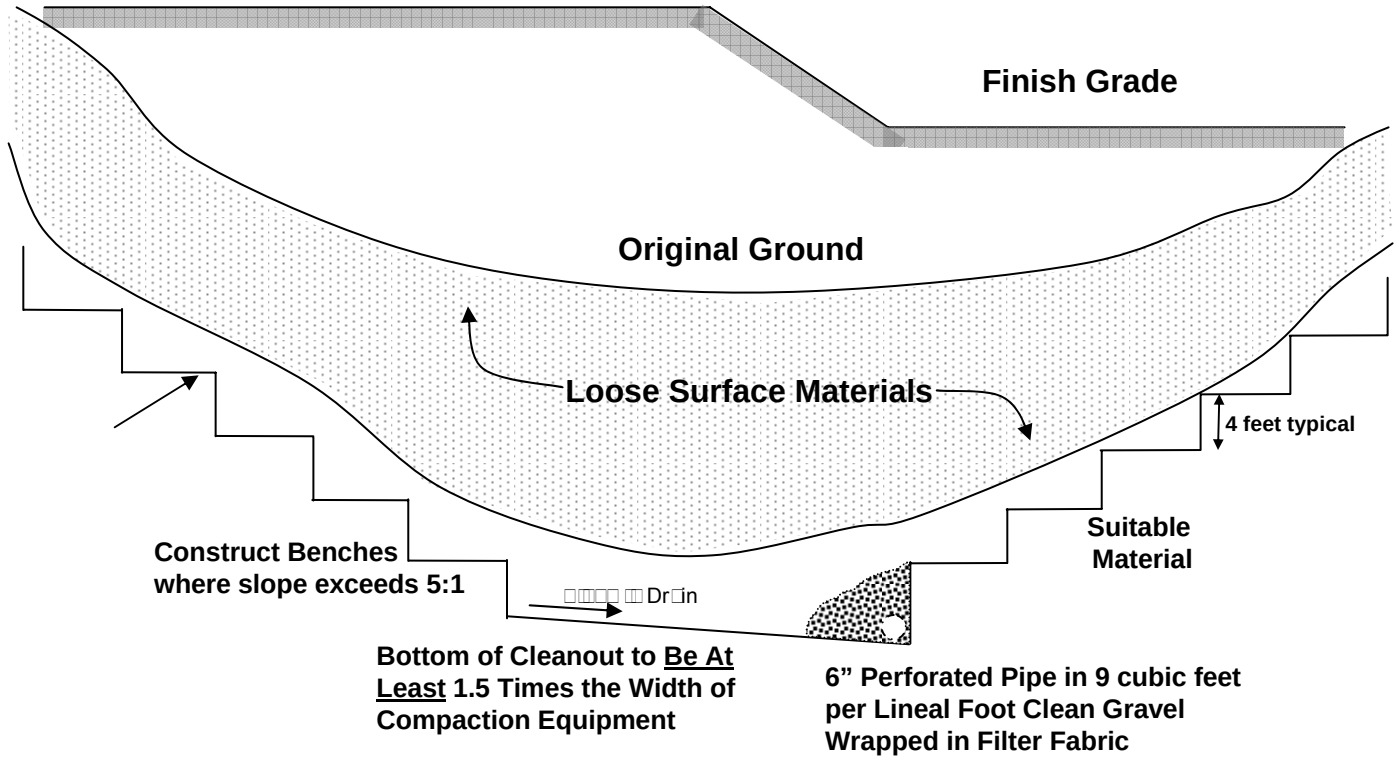
In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technician's attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and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

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

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

ALTERNATES





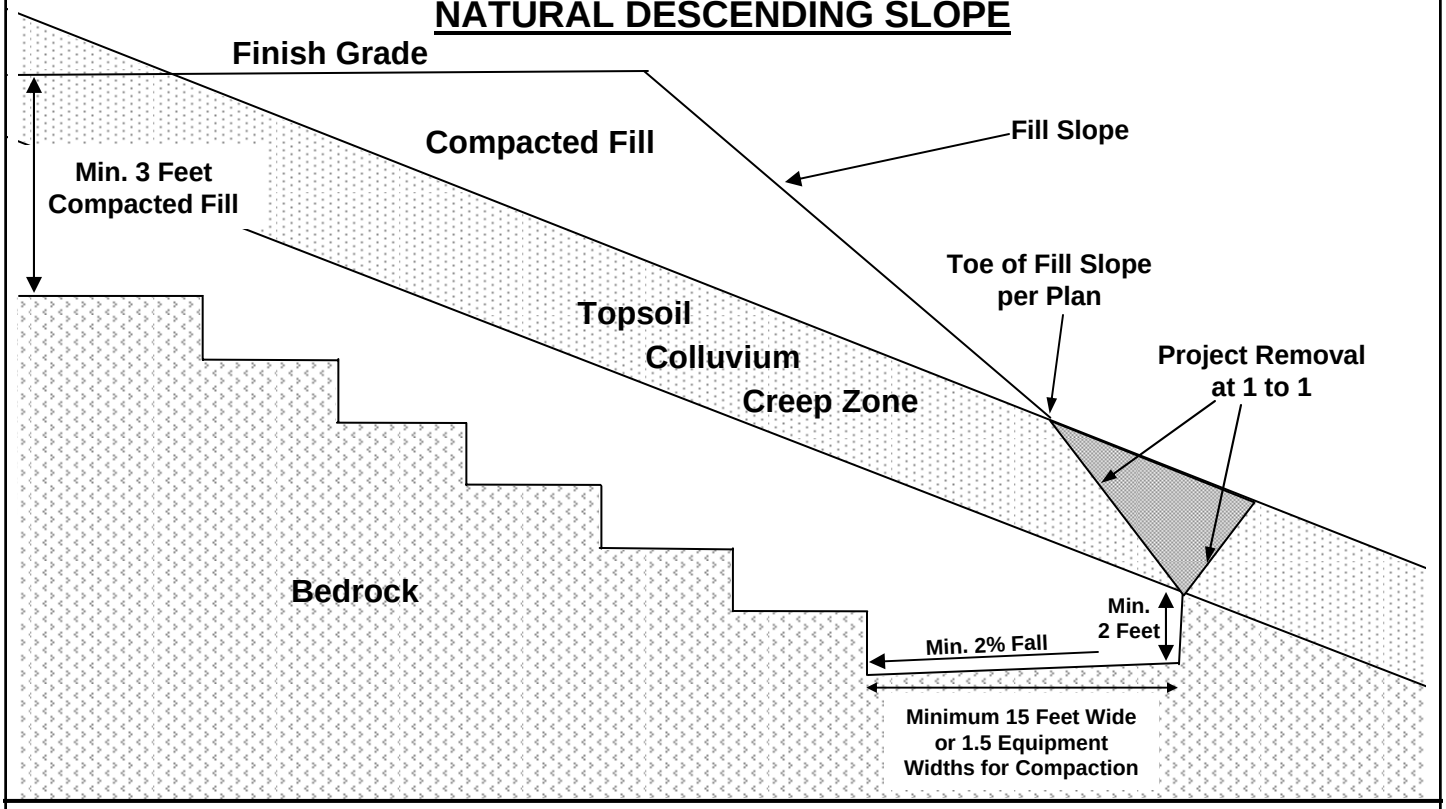
GEOTEK
1548 North Maple Street
Corona, California 92880

TYPICAL CANYON CLEANOUT

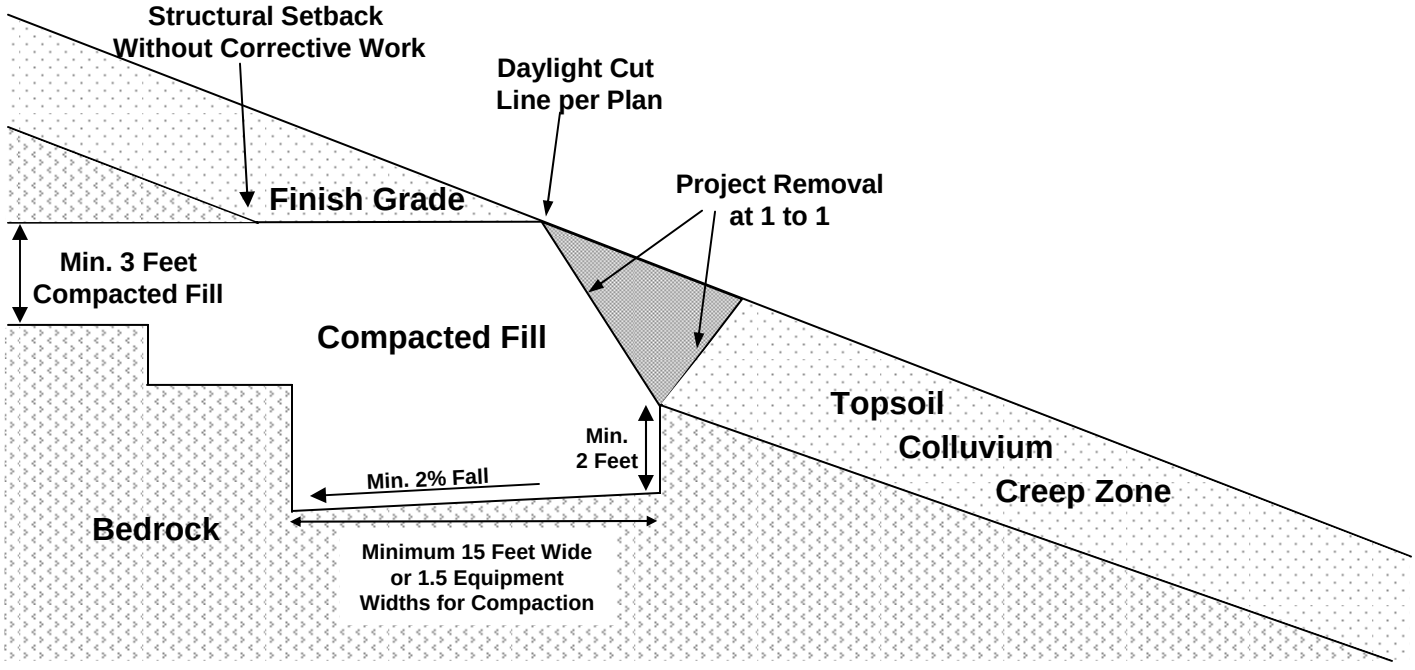
STANDARD GRADING GUIDELINES

PLATE E-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



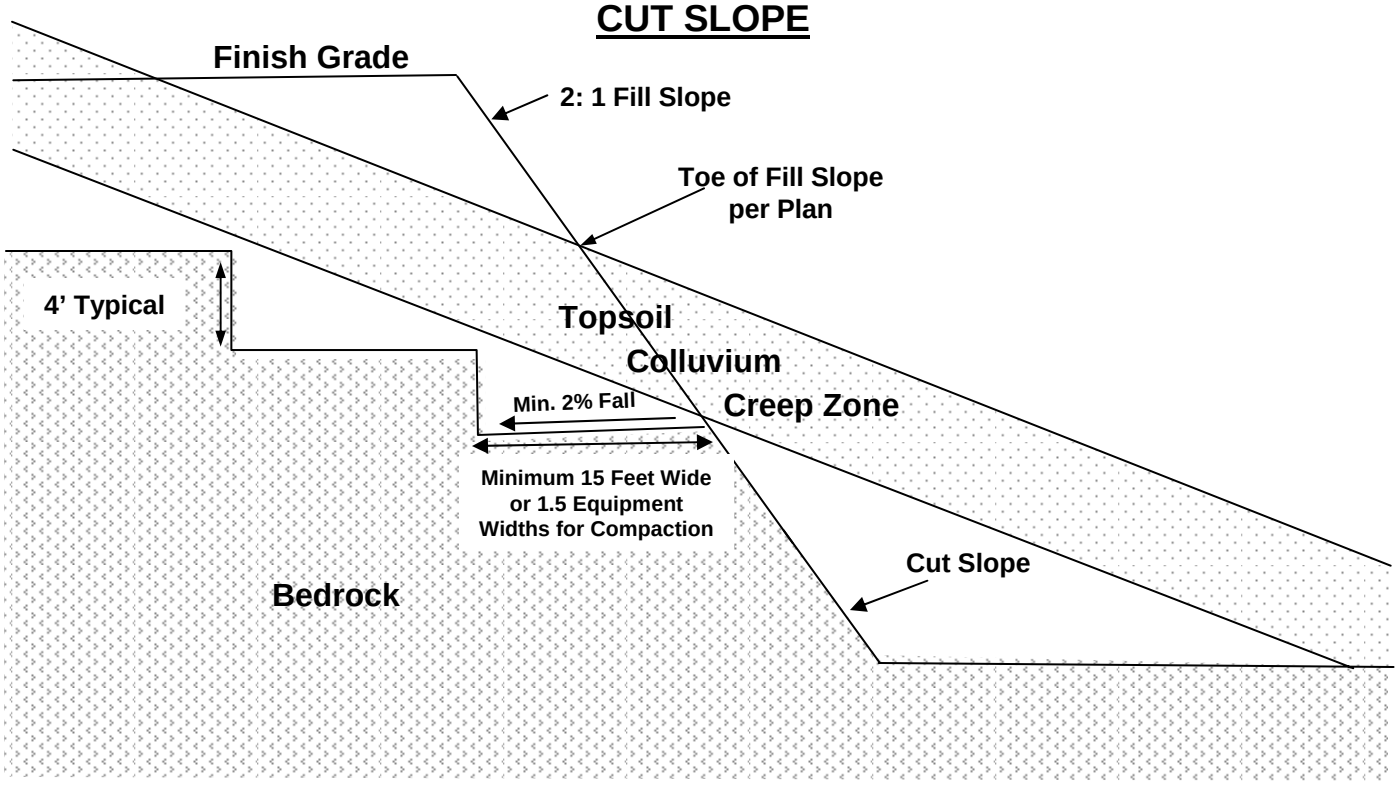
GEOTEK
1548 North Maple Street
Corona, California 92880

TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE E-2

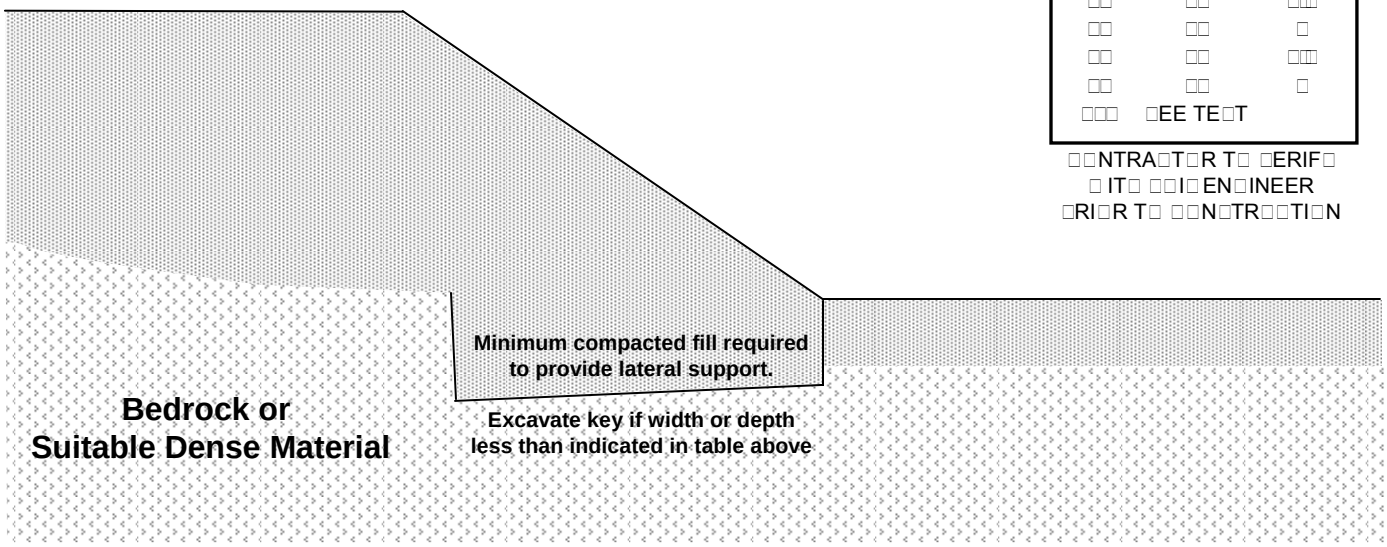
TYPICAL FILL SLOPE OVER CUT SLOPE



TYPICAL FILL SLOPE

□□□□E □EI□□T	MIN□□E□ □IDT□	MIN□□E□ DE□T□
□	□	□
□□	□□	□□□
□□	□□	□
□□	□□	□□□
□□	□□	□
□□□	□EE TE□T	

□□NTRA□T□R T□ □ERIF□
□IT□ □□I□EN□INEER
□RI□R T□ □□N□TR□□TI□N



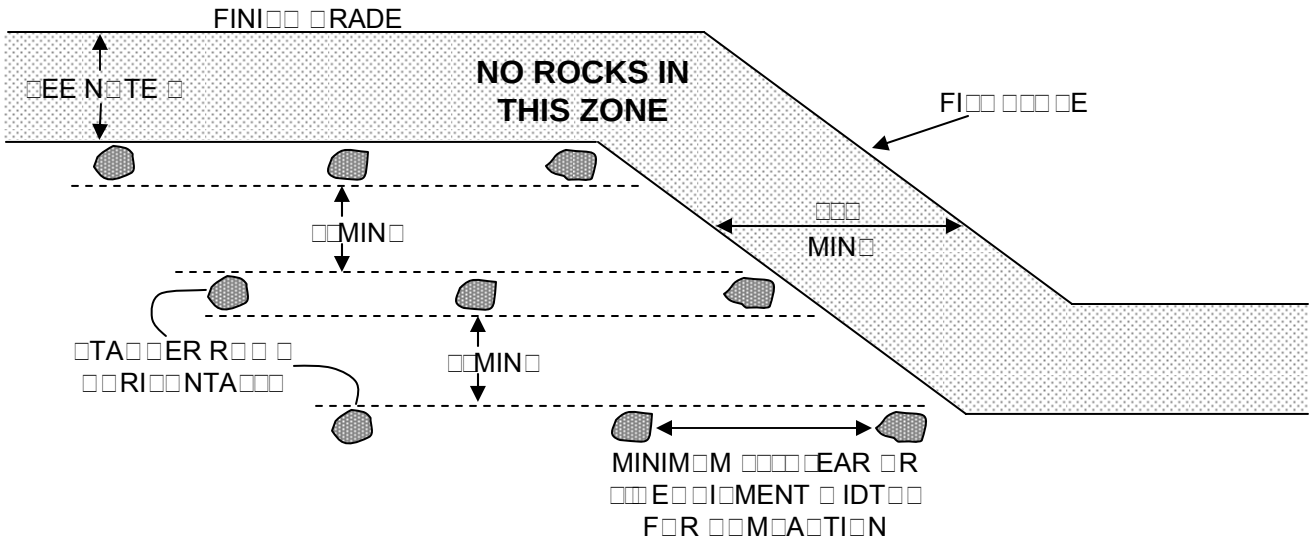
GEOTEK
1548 North Maple Street
Corona, California 92880

COMMON FILL SLOPE KEYS

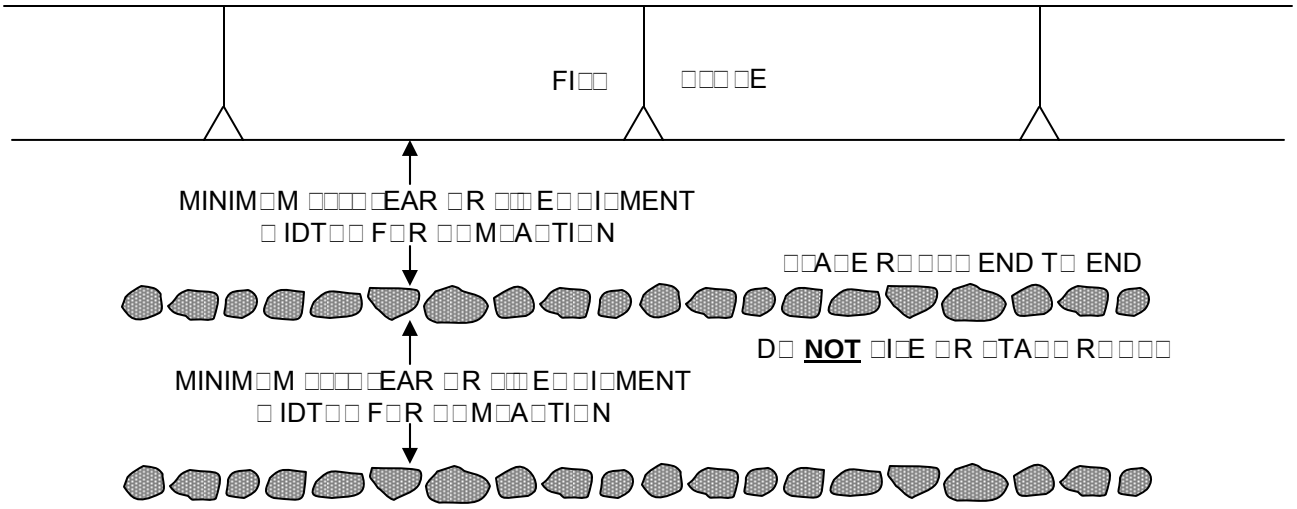
STANDARD GRADING GUIDELINES

PLATE E-3

CROSS SECTIONAL VIEW



PLAN VIEW



N□TE□□

□□ □□I□FI□□ □□ER □ INDR□□ □□□□□E □E □FEET □R □ER □□RI□D□I□TI□NA□ □TANDARD□ AND □□FFI□IENT F□R F□T□RE E□□A□ATI□N□ T□ A□□ID R□□□□

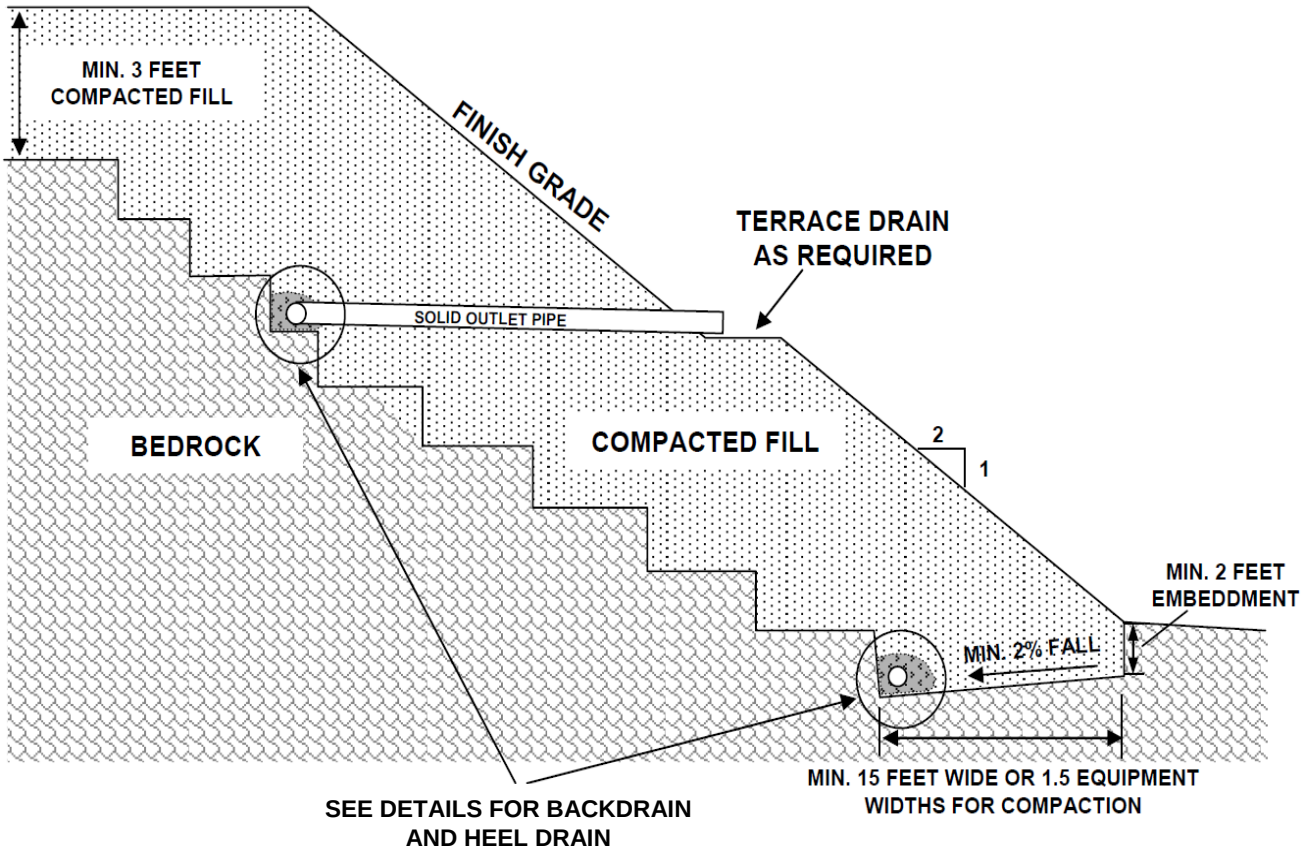
□□ MA□IM□M R□□□ □I□E IN □ INDR□□ □ I□ □FEET IN DIAMETER

□□ □□I□AR□ □ND □ INDR□□ □ T□ □E □AND□ MATERIA□ □□□□E□T T□ □□I□EN□INEER A□□E□TAN□E

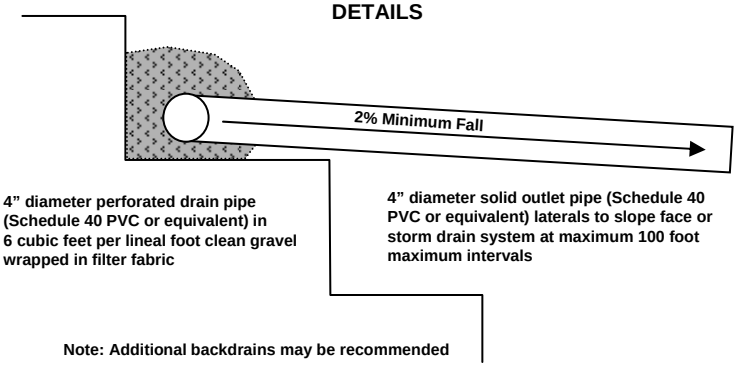
□□ □□A□IN□ AND □□EARAN□E□ M□□T □E □□FFI□IENT T□ A□□□□ F□R □R□□ER □□M□A□TI□N

□□ INDI□D□A□ □AR□E R□□□□ MA□ □E □□RIED IN □IT□□

GEOTEK 1548 North Maple Street Corona, California 92880	ROCK BURIAL DETAILS	STANDARD GRADING GUIDELINES PLATE E-4



BACKDRAIN DETAILS



4" diameter perforated drain pipe (Schedule 40 PVC or equivalent) in 6 cubic feet per lineal foot clean gravel wrapped in filter fabric

4" diameter solid outlet pipe (Schedule 40 PVC or equivalent) laterals to slope face or storm drain system at maximum 100 foot maximum intervals

Note: Additional backdrains may be recommended

HEEL DRAIN DETAILS



6" diameter perforated drain pipe in 6 cubic feet per lineal foot clean gravel wrapped in filter fabric, outlet pipe to gravity flow with 2% minimum fall

GEOTEK	1548 North Maple Street Corona, California 92880	TYPICAL BUTTRESS AND STABILIZATION FILL	STANDARD GRADING GUIDELINES PLATE E-5