# GEOTECHNICAL INVESTIGATION PROPOSED BANNING INDUSTRIAL PARK

NEC Hathaway Street and Nicolet Street
Banning, California
for
First Industrial Realty Trust

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

March 24, 2021
Revised April 15, 2024

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

First Industrial Realty Trust
898 N. Pacific Coast Highway, Suite 175
El Segundo, California 90245

Attention: Mr. Michael Goodwin
Director of Development

Project No.: **21G119-1R3**

Subject: **Geotechnical Investigation**
Proposed Banning Industrial Park
NEC Hathaway Street and Nicolet Street
Banning, California

Dear Mr. Pioli:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,
SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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# 1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

**Geotechnical Design Considerations**

- The subject site is located in a mapped liquefaction hazard zone. However, based on the in-situ soil strength and a groundwater depth that exceeds 50 feet, the liquefaction potential is considered to be very low.
- Engineered fill soils were encountered in all of the borings and trenches performed within the previously overexcavated areas of the site, extending from the ground surface to depths of 6 to 12± feet.
- Artificial (undocumented) fill soils were encountered at some of the boring and trench locations performed for the current investigation, extending from the ground surface to depths of 4½ to 10½± feet. It should be noted that Trench Nos. T-4 and T-5 were terminated within undocumented fill soils. The depth of undocumented fill soils in these areas are unknown.
- Native alluvial soils were encountered at all of the boring and trench locations, extending at least to the maximum depth explored of 15± feet.
- The near-surface native alluvial soils generally consist of non-expansive medium dense to very dense silty sands, gravelly sands and well-graded sands.

**Site Preparation Recommendation**

- Initial site stripping should include removal of the surficial vegetation from the site. These materials should be properly disposed of off-site.
- Demolition of the existing structures and pavements will be required in order to facilitate construction of the new building(s). Demolition should also include all utilities and any other subsurface improvements that will not remain in place for use with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills.
- Remedial grading should be performed within the new building pad area to remove all of the undocumented fill soils and a portion of the upper portion of the native alluvium and engineered fill soils. Based on the conditions encountered at the borings, these fill soils extend to depths of 4½ to 10½± feet below the existing site grades. In addition, the building pad overexcavation should extend to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed pad grade throughout the building area that was not previously overexcavated.
- The proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.
- Depending on the proposed site grades in the new building area, additional grading may be necessary in previously graded areas in order to provide at least 3 feet of compacted fill below foundation bearing grades and to a depth of at least 2 feet below existing grade.
- Following completion of the overexcavation, the exposed soils should be scarified to a depth of at least 12 inches, and thoroughly flooded to raise the moisture content of the underlying soils to at least 0 to 4 percent above optimum moisture content, extending to a depth of at

least 24 inches. The overexcavation subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

- The on-site soils contain significant amounts of oversized materials, including cobbles and boulders. Selective grading techniques will be required to remove the cobbles and/or boulders from these soils prior to reuse as fill.
- It is recommended that all materials greater than 6-inches in size be excluded from the upper 1 foot of the surface of any compacted fills. Materials greater than 6-inches in size but smaller than 12-inches in size can be placed within the upper 8 feet of any compacted fills. Larger boulders (24±-inches in size and larger) should be sorted, hauled off-site or stockpiled. A portion of the 24-inch and greater diameter material can be placed at the bottom of the deeper overexcavations (10 feet or greater below the proposed grades). On-site sandy soils should then be flooded around the oversize material that was placed at the bottom of the overexcavation.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

**Foundation Design Recommendations**

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft$^2$ maximum allowable soil bearing pressure.
- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

**Building Floor Slab Design Recommendations**

- Conventional Slab-on-Grade: minimum 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Reinforcement is not expected to be necessary for geotechnical considerations.
- The actual thickness and reinforcement of the floor slab should be determined by the structural engineer.

**Pavement Design Recommendations**

| **ASPHALT PAVEMENTS (R = 50)** | | | | | |
|---|---|---|---|---|---|
| **Materials** | **Thickness (inches)** | | | | |
| | Parking Stalls (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Truck Traffic | | |
| | | | (TI = 6.0) | (TI = 7.0) | (TI = 8.0) |
| Asphalt Concrete | 3 | 3 | 3½ | 4 | 5 |
| Aggregate Base | 3 | 3 | 4 | 5 | 5 |
| Compacted Subgrade (90% minimum compaction) | 12 | 12 | 12 | 12 | 12 |

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 50) | | | | |
|---|---|---|---|---|
| **Materials** | **Thickness (inches)** | | | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Truck Traffic | | |
| | | (TI =6.0) | (TI =7.0) | (TI =8.0) |
| PCC | 5 | 5 | 5½ | 6½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 | 12 | 12 |

## 2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 21P140, dated January 28, 2021. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the currently proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

# 3.0 SITE AND PROJECT DESCRIPTION

## 3.1 Site Conditions

The subject site is located at the northeast corner of Hathaway Street and Nicolet Street in Banning, California. The site is bounded to the north by the future Wilson Street and the Morongo Indian Reservation, to the west by Hathaway Street, to the south by the future Nicolet Street and the I-10 freeway, and to the east by the future O'Donnell Street and a vacant lot. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The overall site consists of six (6) rectangular to irregular-shaped parcels, totaling 82.81± acres in size. The two (2) northwestern parcels were formerly occupied by ORCO Block & Hardscape, as a concrete block manufacturing facility, which is presently unoccupied. Most of the structures and other improvements associated with this facility have been demolished, with the exception of one building located in the west-central area. The building is a single-story structure of masonry block construction, approximately 4,300 $ft^2$ in size, and is assumed to be supported on conventional shallow foundations with a slab-on-grade floor. Some floor slabs and foundations of former structures are also present in the area surrounding the existing building. A retaining wall ranging from 1 to 6± feet in height and approximately 200 feet in length is present near the southern and eastern areas of the existing building. Ground surface cover in this area consists of asphaltic concrete (AC) and Portland cement concrete (PCC) pavements. The pavements are in poor condition with moderate to severe cracking throughout. Ground surface cover in the remaining areas of these two northwestern parcels consist of exposed soil and sparse to moderate native grass, weed, and small shrub growth.

The remaining four (4) parcels are located in the southern and eastern areas of the overall site. These parcels were graded in 2011 for a previously proposed development which was not completed. SCG provided geotechnical observation and testing services during the rough grading of portions of these parcels. A summary of the grading operations and the results of our observation and testing are discussed in the referenced Interim Rough Grade Compaction Report (Reference No. 2), listed in Section 8 of this report. These parcels are generally vacant and generally undeveloped, with the exception of six (6) existing detention basins. The basins have depths ranging from 7 to 14± feet. Several slopes are present within these parcels, generally located along the boundaries of the four parcels. The inclinations of the slopes range from 2h:1v (horizontal to vertical) to 5h:1v and are 5 to 24± feet in height. Several large stockpiles of boulders and large cobbles are present in the south-central region of the northeastern parcels. The stockpiles are 40 to 90± feet in width and 95 to 180± feet in length and are approximately 4 to 11± feet in height. Ground surface cover throughout the parcels consists of exposed soil with sparse to moderate native grass and weed growth.

Current topographic information for the proposed development was obtained from a preliminary grading plan provided by the client. Based on this plan and with the exception of the aforementioned slopes, stockpiles, previously rough-graded building pads, and basins, the overall site topography generally slopes downward to the southeast at a gradient of 4± percent. The existing site grades range from a maximum elevation of 2,334± feet mean sea level (msl) in the northwestern corner of the site to a minimum elevation of 2,211± feet msl in the southeastern corner.

## 3.2 Proposed Development

Based on the project preliminary grading plan, provided by the client, the site will be developed with one new (1) commercial/industrial building, 1,407,230± ft² in size. The building will be located in the north-central area of the site. Dock-high doors will be constructed in a cross-dock configuration on the north and south sides of the building. The building will be surrounded by AC pavements in the automobile parking and drive areas, PCC pavements in the loading dock areas, and limited areas of concrete flatwork and landscaped planters throughout.

Detailed structural information has not been provided. We assume that the new warehouse will be a single-story structure of tilt-up concrete construction, typically supported on a conventional shallow foundation system with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

No significant amounts of below-grade construction, such as basements or crawl spaces, are expected to be included in the proposed development. Based on the project preliminary grading plan, cuts of up to 37± feet and fills up to 23± feet are expected to be necessary to achieve the proposed building pad grade, approximately 2276.5 feet msl.

## 3.3 Previous Studies

SCG prepared the three referenced geotechnical reports for the previously proposed development at the subject site, listed in Section 8 of this report. Pertinent details of these studies are described below.

SCG previously performed a geotechnical investigation for this site, the results of which were presented in Reference No. 1, dated October 25, 2006. The subject area of this report consisted of the entire subject area with the exception of the two northwestern-most parcels which were previously occupied by the ORCO Block facility. The subsurface exploration conducted for this project consisted twenty-five (25) trenches (identified as Trench Nos. T-1 through T-25). The trenches were excavated to depths of 4 to 14± feet below grade. Immediately beneath any surficial topsoil, all of the trenches encountered native alluvial soils. The alluvium generally consisted of silty fine to coarse sand, with some fine to coarse gravel content, extensive cobbles, and occasional boulders. At depths below 4± feet, the alluvium became coarser, generally consisting of medium dense to dense fine to coarse sands with some fine to coarse gravel content, extensive cobbles and some boulders extending to at least the maximum depth explored of 14± feet. The previous boring and trench locations are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report.

Based on the conditions encountered at the trench locations, it was recommended that remedial grading be performed within the building pad areas. The building pad areas were recommended to be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below the proposed pad grade. Additional overexcavation was recommended within the foundation influence zones extending to a depth of 3 feet below the bearing grade of all foundations.

Reference No. 2 was prepared to document our observation and testing performed at the subject site. At the time of this interim report, remedial grading activities had only been performed in the portions of the future building pad areas which required fill in order to establish the finished rough finished grades. No remedial grading was performed within the "cut" portions of the future building pads. Remedial grading was performed in areas that were to receive fill. The remedial grading consisted of the removal of the upper 4± feet of soils present in the "fill" portion of the proposed building pad areas. Generally, the fill areas were overexcavated to depths ranging from 4 to 24± feet below the proposed pad grades. The on-site soils were then used for structural compacted fill in order to establish the planned pad grades within the fill areas.

Sorting of oversize rock material was performed during the rough grading operations. Cobbles greater than 6± inches in diameter were generally removed from the top 12 inches of the fill in the building pad areas. Rocks greater than 12± inches in diameter were sorted from the top 8 feet of fill in the pad areas. Materials greater than 18± inches in diameter were sorted and hauled off-site or stockpiled. A portion of the 18-inch and greater diameter material was placed at the bottom of the deeper overexcavation (15 to 20 feet below pad grade) at the east end of northeast building pad. On-site sandy soils were then flooded around the oversize material that was placed at the bottom of the overexcavation.

SCG prepared an updated geotechnical report (Reference No. 3) for the subject site, dated March 15, 2018. As part of this update report, subsurface exploration was performed with the area of the former ORCO Block facility. The subsurface exploration consisted of four (4) borings (identified as Boring Nos. B-1 through B-4) advanced to depths of 10 to 20± feet. Asphaltic concrete pavements were present at the ground surface at Boring No. B-2. The asphalt pavements consisted of 1± inch of asphaltic concrete, with no discernable layer of underlying aggregate base. Undocumented fill soils were encountered beneath the asphaltic concrete at Boring B-2 and at the ground surface at Boring Nos. B-3. The fill soils extend to a depth of 2½ to 3± feet below the existing site grades. The fill soils generally consisted of loose to medium dense silty fine to medium sands and fine to coarse sands with varying gravel content. The fill soils possess a disturbed appearance resulting in their classification as undocumented fill. Soils classified as possible fill were encountered beneath undocumented fill soils at Boring B-2 and at the ground surface at Boring Nos. B-1 and B-4, extending to depths of 2½ to 4± feet below the existing site grades. The possible fill soils generally consisted of medium dense fine to coarse sands and fine to coarse sandy gravels to gravelly fine to coarse sands with occasional cobbles. These soils possess a somewhat disturbed appearance, but lack obvious indicators of undocumented fill, resulting in their classification as possible fill. Native alluvial soils were encountered beneath the undocumented fill and/or possible fill soils at all of the boring locations, extending to at least the maximum depth explored of 20± feet below existing site grades. The alluvium generally consisted of medium dense to very dense gravelly fine to coarse sands, fine to coarse sandy gravels, and fine to coarse sands with occasional cobbles. The previous borings are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report.

# 4.0 SUBSURFACE EXPLORATION

## 4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of six (6) borings advanced to depths of 6 to 15± feet below the existing site grades and ten (10) trenches excavated to depths of 6½ to 10½± feet. Three of the borings and seven of the trenches were terminated at depths shallower than proposed after encountering refusal on cobbles and boulders. All of the borings and trenches were logged during the drilling and excavation by members of our staff.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. The trenches were excavated using a backhoe with a 36-inch-wide bucket. Representative bulk and undisturbed soil samples were taken during drilling. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings (identified as Boring Nos. B-1 through B-6) and trenches (identified as Trench Nos. T-1 through T-10) are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report. The Boring and Trench Logs, which illustrate the conditions encountered at the boring and trench locations, as well as the results of some of the laboratory testing, are included in Appendix B.

## 4.2 Geotechnical Conditions

### Engineered Fill

Boring Nos. B-5 and B-6 and Trench Nos. T-9 and T-10 were performed within the previously overexcavated areas of the site. Within this area, Boring Nos. B-5 and B-6 and Trench Nos. T-9 and T-10 encountered engineered fill soils, extending to depth of 12, 10, 6½, and 6± feet below the existing site grades, respectively. It should be noted that Boring Nos. B-5 and B-6 and Trench No. T-9 were terminated in engineered fill due to refusal on dense to very dense cobbles and boulders. The engineered fill soils consist of dense to very dense gravelly sands and silty sands with trace amounts of silt and occasional to extensive amounts of cobbles. These materials were placed and compacted during rough grading procedures, as discussed in the referenced rough grade report.

Artificial Fill (Undocumented Fill)

Artificial fill soils were encountered at the ground surface at Boring No. B-2 and at Trench Nos. T-4 and T-5, extending to depths of 4½ to 10½± feet below ground surface. The fill soils generally consist of medium dense to very dense silty sands, gravelly sands, and well-graded sands, with varying gravel and cobble content. The fill soil possesses a disturbed and mottled appearance, as well as asphaltic concrete, PCC, and CMU fragments and steel pipes, resulting in their classification as artificial fill. It should be noted that Trench Nos. T-4 and T-5 were terminated within artificial fill soils. The depth of artificial fill soils in these areas are unknown. The artificial fill is considered to represent undocumented fill.

Alluvium

Native alluvium was encountered at the ground surface, and beneath the engineered fill soils and the undocumented fill soils, with the exception of Trench Nos. T-4 and T-5, extending to at least the maximum depth explored of 15± feet below ground surface. The alluvial soils generally consist of medium dense to very dense silty sands, gravelly sands, and well- and poorly-graded sands, with varying silt, cobble and boulder content.

Groundwater

Groundwater was not encountered at any of the borings or trenches. Based on the lack of any water within the borings and trenches, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 15± feet below existing site grades, at the time of the subsurface investigation.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well in this database is located approximately 1,600 feet northwest of the site. Water level readings within this monitoring well indicate a high groundwater level of 541± feet below the ground surface in June 2013.

# 5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring and Trench Logs and are periodically referenced throughout this report.

Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring and Trench Logs.

Consolidation

Selected soil samples were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-5 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

Representative samples of the near-surface soil were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

| Sample Identification | Soluble Sulfates (%) | Sulfate Classification |
|---|---|---|
| B-4 @ 0 to 5 feet | 0.001 | Not Applicable (S0) |
| B-6 @ 0 to 5 feet | 0.001 | Not Applicable (S0) |

Corrosivity Testing

Representative samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

| Sample Identification | Saturated Resistivity (ohm-cm) | pH | Chlorides (mg/kg) | Nitrates (mg/kg) |
|---|---|---|---|---|
| B-4 @ 0 to 5 feet | 18,400 | 8.2 | 4.1 | 10 |
| B-6 @ 0 to 5 feet | 7,200 | 7.1 | 4.6 | 49 |

# 6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations.

The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The recommendations are provided with the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to verify compliance with these recommendations. Maintaining Southern California Geotechnical, Inc., (SCG) as the geotechnical consultant from the beginning to the end of the project will provide continuity of services. The geotechnical engineering firm providing testing and observation services shall assume the responsibility of Geotechnical Engineer of Record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

## 6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structures should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low. Based on Map Number 06065C0836G, dated August 28, 2008, prepared by FEMA Flood Maps, the project site is in an area designated as Zone X which is determined to be outside the 0.2% annual chance floodplain.

### Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. Section 1613.1 of the 2022 CBC states that "…structures and their supports and attachments shall be designed and constructed to resist the effects of earthquake motions in accordance with Chapters 11, … of ASCE 7."

Section 11.4.8 of ASCE 7-16 states that "it shall be permitted to perform a site response analysis or in Accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2." Therefore, a site-specific ground motion hazard analysis was performed in accordance with Section 21.2 of ASCE 7-16 to determine the seismic design parameters for the new building at this site.

The site classification was determined using shear wave velocity measurements for the soils present within the upper 100± feet at the subject site. The parameter $V_{100}$ is defined as the shear-wave velocity of the soil or bedrock material present within the upper 100 feet at the site. The shear-wave velocity was determined by a seismic shear wave survey performed by a licensed geophysicist. The results of the shear-wave survey are included in a report prepared by Terra Geosciences, included in Appendix E of this report. Based on the shear-wave survey performed by Terra Geosciences, the $V_{100}$ for the site is 1,891.7 feet per second. Table 20.3-1 of ASCE 7-16 indicates that an average shear velocity ranging between 1,200 and 2,500 feet per second corresponds to Site Class C.

Details regarding the performance of the ground motion hazard analysis are presented in the report prepared by Terra Geosciences, in Appendix E of this report. Seismic design parameters computed during this study are tabulated below.

**SITE-SPECIFIC SEISMIC DESIGN PARAMETERS BASED ON ASCE 7-16 SECTION 21.2**

| Parameter | | Value |
|---|---|---|
| Mapped Spectral Acceleration at 0.2 sec Period | $S_S$ | 2.103 |
| Mapped Spectral Acceleration at 1.0 sec Period | $S_1$ | 0.847 |
| Site Class | --- | C |
| Site Modified Spectral Acceleration at 0.2 sec Period | $S_{MS}$ | 2.290 |
| Site Modified Spectral Acceleration at 1.0 sec Period | $S_{M1}$ | 1.243 |
| Design Spectral Acceleration at 0.2 sec Period | $S_{DS}$ | 1.530 |
| Design Spectral Acceleration at 1.0 sec Period | $S_{D1}$ | 0.830 |

### Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean ($d_{50}$) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Riverside County GIS website indicates that the subject site is located within a zone of moderate liquefaction susceptibility. However, the subsurface conditions encountered at the boring and trench locations are not considered to be conducive to liquefaction. These conditions consist of moderate to high strength engineered fill and native alluvial soils, and the lack of a historic high ground water table within the upper 50± feet of the ground surface within the subject site. Based on these considerations, liquefaction is not considered to be a design concern for this project.

## 6.2 Geotechnical Design Considerations

### General

Boring Nos. B-5 and B-6 and Trench Nos. T-9 and T-10 were performed within the previously rough-graded areas, as indicated on Plate 2 of this report. Based on their strength characteristics, and the previous SCG rough grade compaction report (Reference 2), the existing fill soils encountered within the previously rough-graded areas are considered to represent engineered fill soils. These materials are considered to be suitable for support of the new structure, subject to limited remedial grading discussed below.

Boring No. B-2 and Trench Nos. T-4 and T-5 were performed within the area of the former ORCO Block facility. The near-surface fill soils encountered at these locations are considered to represent undocumented fill and are not suitable for support of new structure. In addition, some of the near-surface alluvial soils possess moisture contents well below the optimum moisture content for compaction. The depth of undocumented fill soils at Trench Nos. T-4 and T-5 are unknown. **Based on the recently provided conceptual grading plan for this project, additional subsurface exploration is recommended within these areas to determine the actual depth of undocumented fill**.

Based on the existing conditions, remedial grading is considered warranted within the proposed building area in order to remove the existing undocumented fill soils, and a portion of the near-surface alluvial soils and engineered fill soils, and replace these materials as compacted structural fill.

### Settlement

The recommended remedial grading will remove all of the undocumented fill soils and a portion of the near-surface native alluvium and engineered fill soils, and replace these soils as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure are expected to be less than 1.0 and 0.5 inches for total and differential settlements of shallow foundations, respectively.

Slope Stability

No evidence of landslides or deep-seated slope instability was noted during our investigation. However, loose granular soils on sloping ground surfaces could be prone to surficial failures.

Based on the project conceptual grading plan, planned grading within the area of the existing slope, which extends up to 24± feet in height and possesses a maximum inclination of 2h:1v, located in the central region of the proposed building area will be necessary to reach the finish pad elevation of the proposed building. A significant portion of the existing slope will be removed in order to facilitate construction of the proposed building. The final configuration of the site will not include any of this existing slope. Based on these conditions, a slope stability analysis for the existing slope is not considered warranted. Slope failure is not expected to occur during grading.

Newly constructed fill slopes, comprised of properly compacted engineered fill, at inclinations of 2h:1v or less will possess adequate gross stability. Cut slopes excavated within the existing granular alluvial soils may be subject to surficial instability due to the lack of cohesion within these materials and low moisture contents. Therefore, stability fills may be required within these areas. This condition may affect the proposed cut slopes at the site. The need for stability fills should be determined by SCG as part of the final detailed grading plan review and/or during grading.

Expansion

The on-site soils generally consist of silty sands, gravelly sands, and well-graded sands with varying amounts of gravel, cobbles, and boulders. These materials have been visually classified as non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site.

Soluble Sulfates

The results of the soluble sulfate testing indicated a sulfate concentration of approximately 0.001 percent for the selected samples of the near-surface soils. This concentration is considered to be "not applicable" (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that the on-site soils possess saturated resistivity values ranging of 7,200 to 18,400 ohm-cm, and pH values ranging of 7.1 to 8.2. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research

Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Sulfides, and redox potential are factors that are also used in the evaluation procedure. We have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH, and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are not considered to be corrosive to ferrous pipes. Therefore, corrosion protection is not expected to be required for cast iron or ductile iron pipes.

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate chloride concentrations of 4.1 to 4.6 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 10 to 49 mg/kg. Based on this test result, the on-site soils are not considered to be corrosive to copper pipe.

Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation of these test results.

Shrinkage/Subsidence

Removal and recompaction of the existing fill soils and near-surface alluvium is estimated to result in an average shrinkage of 3 to 13 percent. It should be noted that the potential shrinkage estimate is based on our experience with similar projects at nearby sites. It was not practical to obtain undisturbed samples based on the gravel, cobble, and boulder content of the onsite soils. Therefore, the actual amount of shrinkage could vary considerably from these estimates. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the trench locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans were not available at the time of this report. It is recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

## 6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the trench locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping

Initial site preparation should include stripping of any surficial vegetation. This includes the removal of the sparse native grass, weeds, and shrubs present at the site. These materials should be disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

The proposed development will require extensive demolition of the existing buildings and pavements. Additionally, any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, floor slabs, utilities, and any other subsurface improvements associated with the existing structures. The existing pavements are not expected to be reused with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the on-site soils, and incorporated into new structural fills. These materials may also be crushed and made into miscellaneous base for use in the proposed pavement areas.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the new building pad area to remove all of the undocumented fill soils and a portion of the near-surface native alluvium and engineered fill soils. Based on the conditions encountered at the borings, the undocumented fill soils extend to depths of at least 4½ to 10½± feet below the existing site grades. In addition, the building pad overexcavation should extend to a depth of at least 4 feet below existing grade and to a depth of at least 4 feet below proposed pad grade throughout the building area that was not previously overexcavated.

Additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of at least 3 feet below proposed foundation bearing grade.

SCG should be provided with the final grading and foundation plans for the proposed building, when they become available, in order to determine the extent of the remedial grading necessary in the previously graded areas. Additional remedial grading will be necessary in previously graded areas, indicated on Plate 2 of this report, in order to provide at least 3 feet of compacted fill below foundation bearing grades and to a depth of at least 2 feet below existing grade.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill below the new foundations. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the overexcavations should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling with a heavy rubber-tire vehicle to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if dry, loose, porous, low density or otherwise unsuitable materials are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and thoroughly flooded to raise the moisture content of the underlying soils to at least 0 to 4 percent above optimum moisture content, extending to a depth of at least 24 inches. The moisture conditioning of the overexcavation subgrade soils should be verified by the geotechnical engineer. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Cut and Fill Slopes

New cut and fill slopes will be constructed within and around the perimeter of the project. Slope heights were not indicated on the provided site plan. Maximum heights of cut and fill slopes were assumed to be within the range of 20± and 30± feet. A keyway should be excavated at the toe of new fill slopes which are not located in fill areas. The keyway should be at least 15 feet in width and 2 feet deep. The recommended width of the keyway is based on 1.5 times the width of typical grading equipment. If smaller equipment is utilized, a smaller keyway may be suitable, at the discretion of the geotechnical engineer. The base of the keyway should slope at least 1 foot downward into the slope. Following completion of the keyway cut, the subgrade soils should be evaluated by the geotechnical engineer to verify that the keyway is founded into competent materials. The resulting subgrade soils should then be scarified to a depth of 10 to 12 inches, moisture conditioned to 0 to 4 percent above optimum moisture content and recompacted. During construction of new fill slopes, the existing slope should be benched in accordance with the detail presented on Plate D-4. Benches less than 4 feet in height may be used at the discretion of the geotechnical engineer.

Should a stability fill for cut slope be necessary, the recommendations for the stability fill will be the same as the recommendations for the fill slopes, mentioned above.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. Erection pads for concrete tilt-up walls are considered part of the foundation system, and the recommended overexcavation should also be performed beneath erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation is not achievable for the proposed walls, the foundations should be redesigned using a lower bearing pressure. The geotechnical engineer of record should be contacted for recommendations pertaining to this type of condition.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing near-surface soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking areas. The grading recommendations presented above do not mitigate the extent of undocumented fill soils in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer. The on-site soils, especially below depths of 1 to 4± feet, possess significant quantities of oversized material, including cobbles and boulders. Some sorting and/or crushing of these materials may be required to generate soils that are suitable for reuse as compacted structural fill.
- All grading and fill placement activities should be completed in accordance with the requirements of the CBC and the grading code of the city of Banning and/or the county of Riverside.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Selective Grading and Oversized Material Placement

The native alluvial soils possess significant cobble and boulder content. It is expected that large scrapers (Caterpillar 657 or equivalent) will be adequate to move the cobble-containing soils as well the soils containing smaller boulders. However, some larger boulders (2± feet in size) are expected to be encountered. It will likely be necessary to move such larger boulders individually, and remove them from the site or place them as oversized materials in accordance with the Grading Guide Specifications, in Appendix D of this report.

It is recommended that all materials greater than 6-inches in size be excluded from the upper 1 foot of the surface of any compacted fills. Materials greater than 6-inches in size but smaller than 12-inches in size can be placed within the upper 8 feet of any compacted fills. Larger boulders (24±-inches in size and larger) should be sorted, hauled off-site or stockpiled. A portion of the 24-inch and greater diameter material can be placed at the bottom of the deeper overexcavations (10 feet or greater below the proposed grades). On-site sandy soils should then be flooded around the oversize material that was placed at the bottom of the overexcavation.

The placement of any oversized materials should be performed in accordance with the Grading Guide Specifications included in Appendix D of this report. If disposal of oversized materials is required, rock blankets or windrows should be used and such areas should be observed during construction and placement by a representative of the geotechnical engineer.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). As discussed previously, imported fill for use below new flatwork should consist of very low expansive (EI < 20) material. Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Banning and/or the County of Riverside. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

## 6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of silty sands, gravelly sands, and well-graded sands with varying gravel, cobble, and boulder content. Based on their composition, moderate to severe caving of shallow excavations may occur in shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavations should be laid back at a slope no steeper than 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

The static groundwater table at this site is considered to exist at a depth in excess of 15± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

## 6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace existing undocumented fill and the upper portion of the native soils. The new structural fill soils are expected to extend to a depth of at least 3 feet below foundation bearing grade underlain by existing native soils that have been densified in place. Based on this subsurface profile, the proposed structure may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft$^2$.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) No. 5 rebars (1 top and 1 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 0 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. **Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.**

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 $lbs/ft^3$
- Friction Coefficient: 0.32

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 3,000 $lbs/ft^2$.

## 6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below finished pad grade. Based on geotechnical considerations, the floor-slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: k = 150 psi/in.

- Minimum slab reinforcement: Not required for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture floor coverings will be used. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 0 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

## 6.7 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades as well as in the dock-high portions of the building. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring and trench locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty sands, gravelly sands, and well-graded sands, with varying amounts of gravel, cobbles and boulders. Based on their classifications, the gravelly sand, sand, and silty sand materials are expected to possess a friction angle of at least 32 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

**RETAINING WALL DESIGN PARAMETERS**

<table>
<tr><th colspan="2" rowspan="2">Design Parameter</th><th>Soil Type</th></tr>
<tr><th>On-site Silty Sands and Sands</th></tr>
<tr><td colspan="2">Internal Friction Angle ($\phi$)</td><td>32°</td></tr>
<tr><td colspan="2">Unit Weight</td><td>136 lbs/ft$^3$</td></tr>
<tr><td rowspan="3">Equivalent Fluid Pressure:</td><td>Active Condition (level backfill)</td><td>42 lbs/ft$^3$</td></tr>
<tr><td>Active Condition (2h:1v backfill)</td><td>64 lbs/ft$^3$</td></tr>
<tr><td>At-Rest Condition (level backfill)</td><td>64 lbs/ft$^3$</td></tr>
</table>

The walls should be designed using a soil-footing coefficient of friction of 0.32 and an equivalent passive pressure of 300 lbs/ft$^3$. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. **However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches.** Some sorting and/or crushing operations may be required. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

## 6.8 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The on-site soils generally consist of silty sands, gravelly sands, and well graded sands with varying amounts of gravel, cobble, and boulders. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 50 to 70. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon a conservatively assumed R-value of 50. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

| Traffic Index | No. of Heavy Trucks per Day |
|---|---|
| 4.0 | 0 |
| 5.0 | 1 |
| 6.0 | 3 |
| 7.0 | 11 |
| 8.0 | 35 |

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

| ASPHALT PAVEMENTS (R = 50) | | | | | |
|---|---|---|---|---|---|
| **Materials** | **Thickness (inches)** | | | | |
| | Parking Stalls (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Truck Traffic (TI = 6.0) | Truck Traffic (TI = 7.0) | Truck Traffic (TI = 8.0) |
| Asphalt Concrete | 3 | 3 | 3½ | 4 | 5 |
| Aggregate Base | 3 | 3 | 4 | 5 | 5 |
| Compacted Subgrade (90% minimum compaction) | 12 | 12 | 12 | 12 | 12 |

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the batch plant-reported maximum density. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within Portland cement concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 50) | | | | |
|---|---|---|---|---|
| **Materials** | **Thickness (inches)** | | | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Truck Traffic | | |
| | | (TI =6.0) | (TI =7.0) | (TI =8.0) |
| PCC | 5 | 5 | 5½ | 6½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 | 12 | 12 |

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

## 7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between trench locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

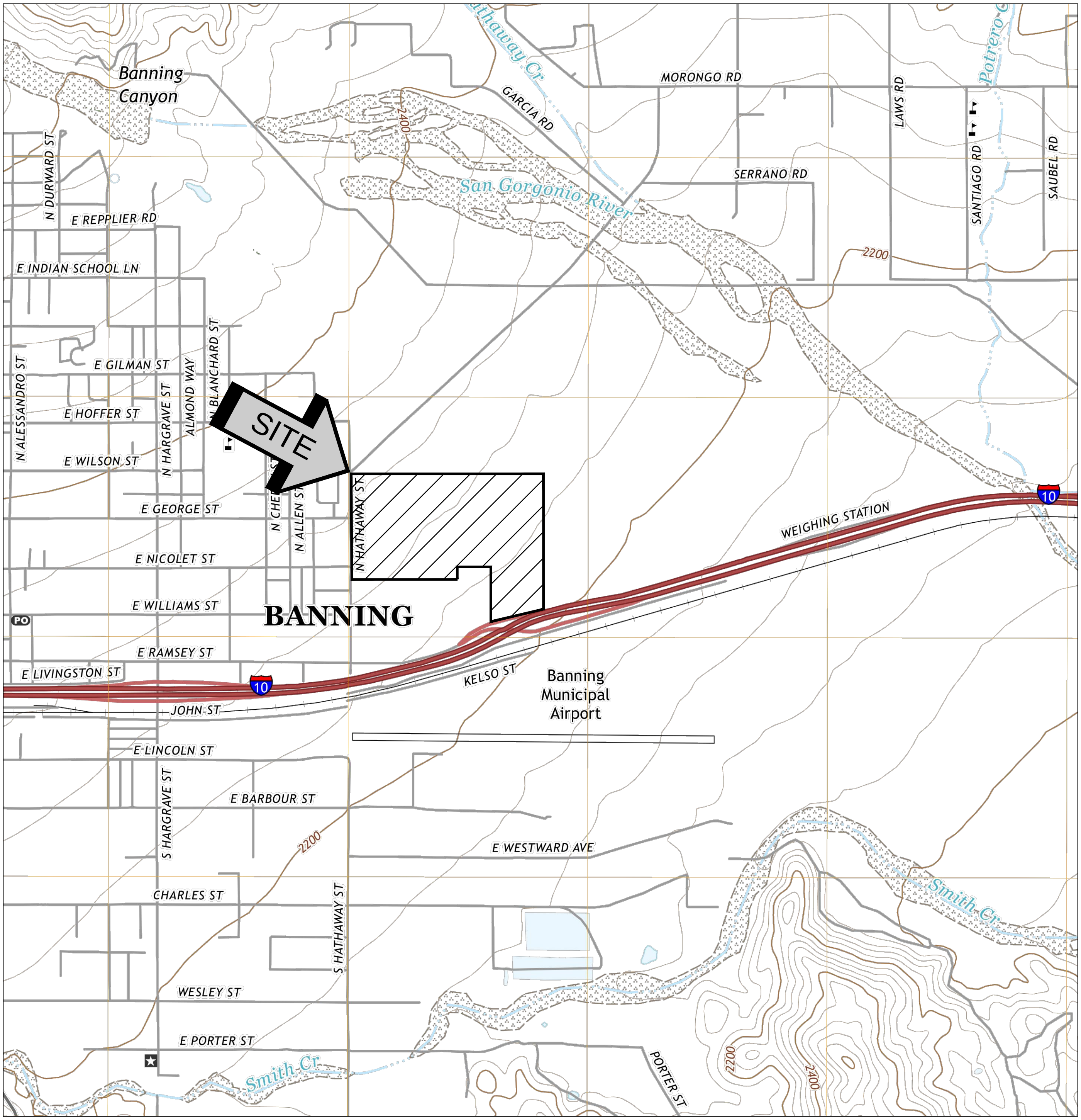
# 8.0 REFERENCES


1) Geotechnical Investigation, Proposed Commercial/Industrial Development, Hathaway Street, North of Ramsey Street, APNs 532-11-003, -008, -009, -010, Banning, California, prepared by SCG for The O'Donnell Group, SCG Project No. 06G227-1, dated October 25, 2006.

2) Interim Rough Grade Compaction Report, Proposed Banning Business Park, Hathaway Street, North of Ramsey Street, Banning, California, prepared by SCG for The O'Donnell Group, SCG Project No. 10M132-4, dated October 13, 2011.

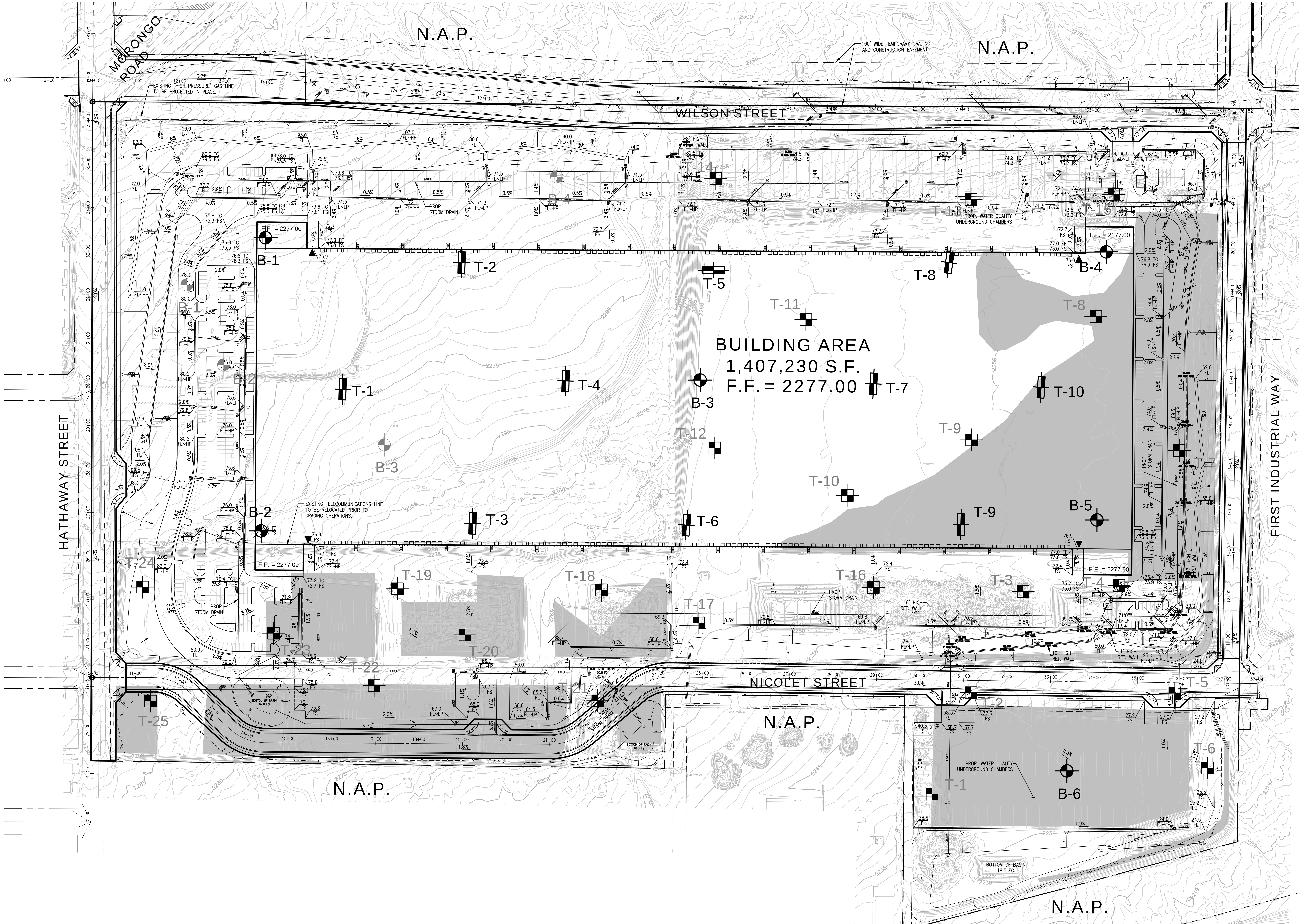
3) Update of Geotechnical Investigation Report, Proposed Stagecoach Business Park, Hathaway Street at Nicolet Street, Banning, California, prepared by SCG for Copart, Inc., SCG Project No. 18G115-1R, dated March 15, 2018.

# APPENDIX A




SOURCE: USGS TOPOGRAPHIC MAP OF THE CABAZON QUADRANGLE, RIVERSIDE COUNTY, CALIFORNIA, 2018

| SITE LOCATION MAP | |
|---|---|
| PROPOSED BANNING INDUSTRIAL PARK | |
| BANNING, CALIFORNIA | |
| SCALE: 1" = 2000' | SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: JAH<br>CHKD: RGT | |
| SCG PROJECT<br>21G119-1 | |
| **PLATE 1** | |




GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE TRENCH LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 18G115-1)
- PREVIOUS TRENCH LOCATION (SCG PROJECT NO. 18G115-1)
- APPROXIMATE LIMITS OF OVER EXCAVATION (SCG PROJECT NO. 10M132-1)

PRELIMINARY GRADING PLAN PREPARED BY STANTEC.

22885 Savi Ranch Parkway
Suite E
Yorba Linda, CA 92887
Phone: (714) 685-1115
Fax: (714) 685-1118
www.socalgeo.com

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

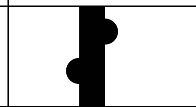
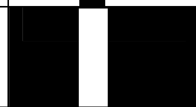
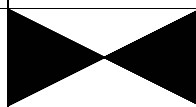
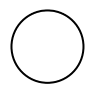
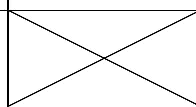
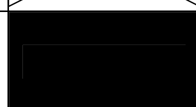
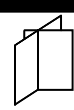
SCALE: 1" = 100'
DRAWN: JLL
CHKD: RGT
SCG PROJECT 21G119-1R

BORING AND TRENCH LOCATION PLAN
PROPOSED BANNING INDUSTRIAL PARK
BANNING, CALIFORNIA

PLATE 2

# APPENDIX B

# BORING LOG LEGEND

| SAMPLE TYPE | GRAPHICAL SYMBOL | SAMPLE DESCRIPTION |
|---|---|---|
| AUGER |  | SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED) |
| CORE |  | ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK. |
| GRAB |  | SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED) |
| CS |  | CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED) |
| NSR |  | NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL. |
| SPT |  | STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED) |
| SH |  | SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED) |
| VANE |  | VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED. |

**COLUMN DESCRIPTIONS**

| | |
|---|---|
| **DEPTH:** | Distance in feet below the ground surface. |
| **SAMPLE**: | Sample Type as depicted above. |
| **BLOW COUNT**: | Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more. |
| **POCKET PEN.**: | Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer. |
| **GRAPHIC LOG**: | Graphic Soil Symbol as depicted on the following page. |
| **DRY DENSITY**: | Dry density of an undisturbed or relatively undisturbed sample in $lbs/ft^3$. |
| **MOISTURE CONTENT**: | Moisture content of a soil sample, expressed as a percentage of the dry weight. |
| **LIQUID LIMIT**: | The moisture content above which a soil behaves as a liquid. |
| **PLASTIC LIMIT**: | The moisture content above which a soil behaves as a plastic. |
| **PASSING #200 SIEVE**: | The percentage of the sample finer than the #200 standard sieve. |
| **UNCONFINED SHEAR**: | The shear strength of a cohesive soil sample, as measured in the unconfined state. |

# SOIL CLASSIFICATION CHART

| MAJOR DIVISIONS | | | SYMBOLS GRAPH | SYMBOLS LETTER | TYPICAL DESCRIPTIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | CLEAN GRAVELS | | GW | WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | | (LITTLE OR NO FINES) | | GP | POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES |
| | MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE | GRAVELS WITH FINES | | GM | SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES |
| | | (APPRECIABLE AMOUNT OF FINES) | | GC | CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES |
| MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | SAND AND SANDY SOILS | CLEAN SANDS | | SW | WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES |
| | | (LITTLE OR NO FINES) | | SP | POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES |
| | MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE | SANDS WITH FINES | | SM | SILTY SANDS, SAND - SILT MIXTURES |
| | | (APPRECIABLE AMOUNT OF FINES) | | SC | CLAYEY SANDS, SAND - CLAY MIXTURES |
| FINE GRAINED SOILS | SILTS AND CLAYS | LIQUID LIMIT LESS THAN 50 | | ML | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY |
| | | | | CL | INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS |
| | | | | OL | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY |
| MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS | LIQUID LIMIT GREATER THAN 50 | | MH | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS |
| | | | | CH | INORGANIC CLAYS OF HIGH PLASTICITY |
| | | | | OH | ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS |
| HIGHLY ORGANIC SOILS | | | | PT | PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS |

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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# BORING NO. B-1

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | DESCRIPTION | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | ALLUVIUM: Light Gray Gravelly fine to coarse Sand, occasional Cobbles, very dense-dry to damp | | | | | | | |
| | X | 50/5" | | | | | 1 | | | | | Disturbed Sample |
| | X | 80 | | | | | 1 | | | | | Disturbed Sample |
| 5 | X | 82 | | | | 117 | 2 | | | | | |
| | | | | | Boring Terminated at 6' due to refusal on very dense Cobbles | | | | | | | |

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-2

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 4 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | DESCRIPTION | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | UNDOCUMENTED FILL: Brown Silty fine to coarse Sand, trace fine to coarse Gravel, occasional Cobbles, medium dense-damp | | | | | | | |
| | | 26 | | | | 98 | 4 | | | | | |
| | | 32 | | | | | | | | | | No Sample Recovery |
| | | | | | ALLUVIUM: Light Gray Brown Gravelly fine to coarse Sand, extensive Cobbles, medium dense to very dense-dry to damp | | | | | | | |
| 5 | | 56 | | | | | 1 | | | | | Disturbed Sample |
| | | 41 | | | | 113 | 2 | | | | | |
| | | 68 | | | | 105 | 2 | | | | | |
| 10 | | | | | | | | | | | | |
| | | 86 | | | | | 2 | | | | | |
| 15 | | | | | Boring Terminated at 15' | | | | | | | |

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-3

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 4 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | DESCRIPTION | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, occasional Cobbles, dense to very dense-dry to damp | | | | | | | |
| | | 72 | | | | 117 | 2 | | | | | |
| | | 72 | | | | | | | | | | No Sample Recovery |
| 5 | | 50/3" | | | | | | | | | | No Sample Recovery |
| | | 50/5" | | | | | 1 | | | | | Disturbed Sample |
| 10 | | 79/9" | | | | | 1 | | | | | Disturbed Sample |
| | | 90/5" | | | | | 1 | | | | | |
| 15 | | | | | Boring Terminated at 15' | | | | | | | |

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-4

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | ALLUVIUM: Gray Brown Gravelly fine to coarse Sand, occasional Cobbles, very dense-dry | | | | | | | |
| | X | 88 | | | | 124 | 1 | | | | | |
| | X | 79/11" | | | | 133 | 1 | | | | | |
| 5 | X | 50/5" | | | | | 1 | | | | | Disturbed Sample |
| | | | | | Boring Terminated at 6.5' due to refusal on very dense Cobbles | | | | | | | |

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-5

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | ENGINEERED FILL: Brown Silty fine to coarse Sand, trace fine to coarse Gravel, occasional Cobbles, very dense-dry to damp | | | | | | | |
| | X | 76 | | | | 127 | 3 | | | | | |
| | X | 50/6" | | | | | 4 | | | | | Disturbed Sample |
| 5 | X | 86/9" | | | | 125 | 2 | | | | | |
| | | | | | ENGINEERED FILL: Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, very dense-dry to damp | | | | | | | |
| | X | 50/4" | | | | 114 | 2 | | | | | |
| 10 | X | 50/4" | | | | 126 | 3 | | | | | |
| | | | | | Boring Terminated at 12' due to refusal on very dense Cobbles | | | | | | | |

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

# BORING NO. B-6

JOB NO.: 21G119-1
PROJECT: Prop. Banning Industrial Park
LOCATION: Banning, California

DRILLING DATE: 2/25/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

| FIELD RESULTS | | | | | | LABORATORY RESULTS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DEPTH (FEET) | SAMPLE | BLOW COUNT | POCKET PEN. (TSF) | GRAPHIC LOG | DESCRIPTION<br>SURFACE ELEVATION: --- MSL | DRY DENSITY (PCF) | MOISTURE CONTENT (%) | LIQUID LIMIT | PLASTIC LIMIT | PASSING #200 SIEVE (%) | ORGANIC CONTENT (%) | COMMENTS |
| | | | | | ENGINEERED FILL: Brown Silty fine to coarse Sand, little to some fine to coarse Gravel, very dense-damp | | | | | | | |
| | X | 94/11" | | | | | 4 | | | | | |
| | X | 78/11" | | | | | 4 | | | | | |
| 5 | | | | | | | | | | | | |
| | X | 80/11" | | | | | 4 | | | | | |
| | | | | | ENGINEERED FILL: Light Gray Brown Gravelly fine to coarse Sand, occasional Cobbles, very dense-dry | | | | | | | |
| | X | 88/9" | | | | | 1 | | | | | |
| 10 | | | | | | | | | | | | |

Boring Terminated at 10'

TBL 21G119-1.GPJ SOCALGEO.GDT 3/24/21

# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO. T-1**

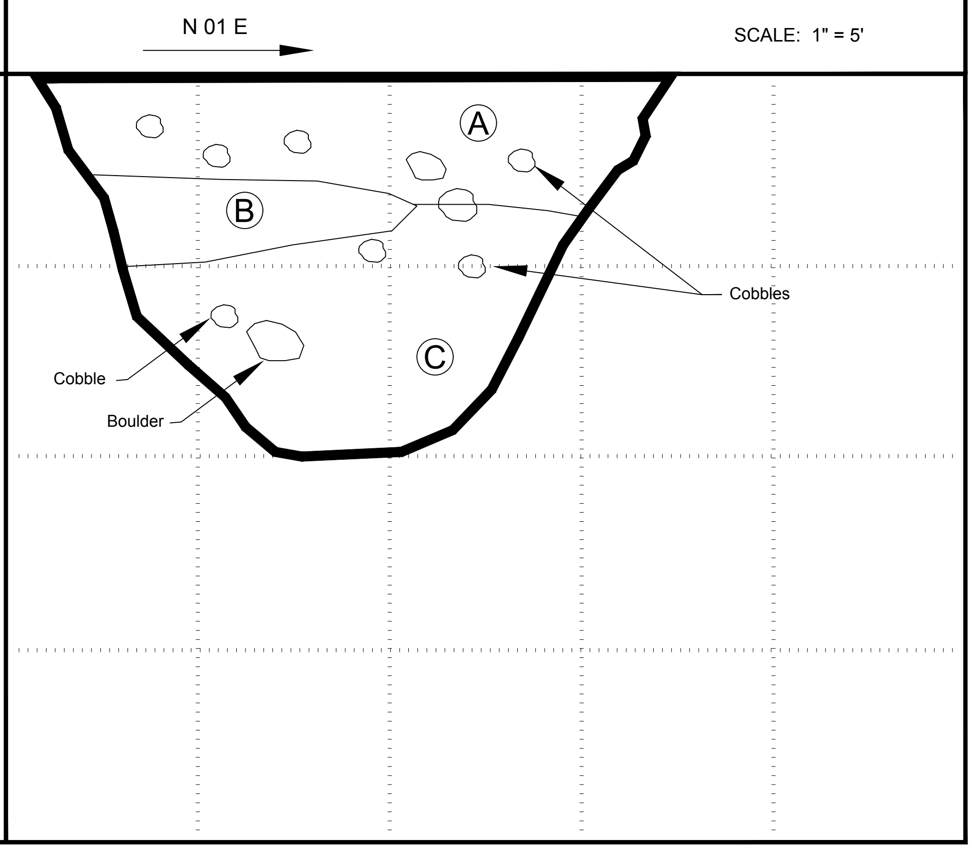
JOB NO.: 21G119-1
PROJECT: Proposed Banning Industrial Park
LOCATION: Banning, California
DATE: 2/25/2021

EQUIPMENT USED: Backhoe
LOGGED BY: Jose Zuniga
ORIENTATION: N 01 E
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | b | | 4 | A: ALLUVIUM: Dark Brown fine Sand, little Silt, little fine to coarse Gravel, occasional Cobbles, medium dense-damp |
| | | | | B: Light Brown Gravelly fine to coarse Sand, medium dense-dry to damp |
| 5 | b | | 2 | C: Brown fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, occasional Boulders, dense to very dense-dry to damp |
| | b | | 3 | |
| | b | | 2 | |
| 10 | | | | Trench Terminated @ 10 feet |
| 15 | | | | |

## GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER (RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-7**

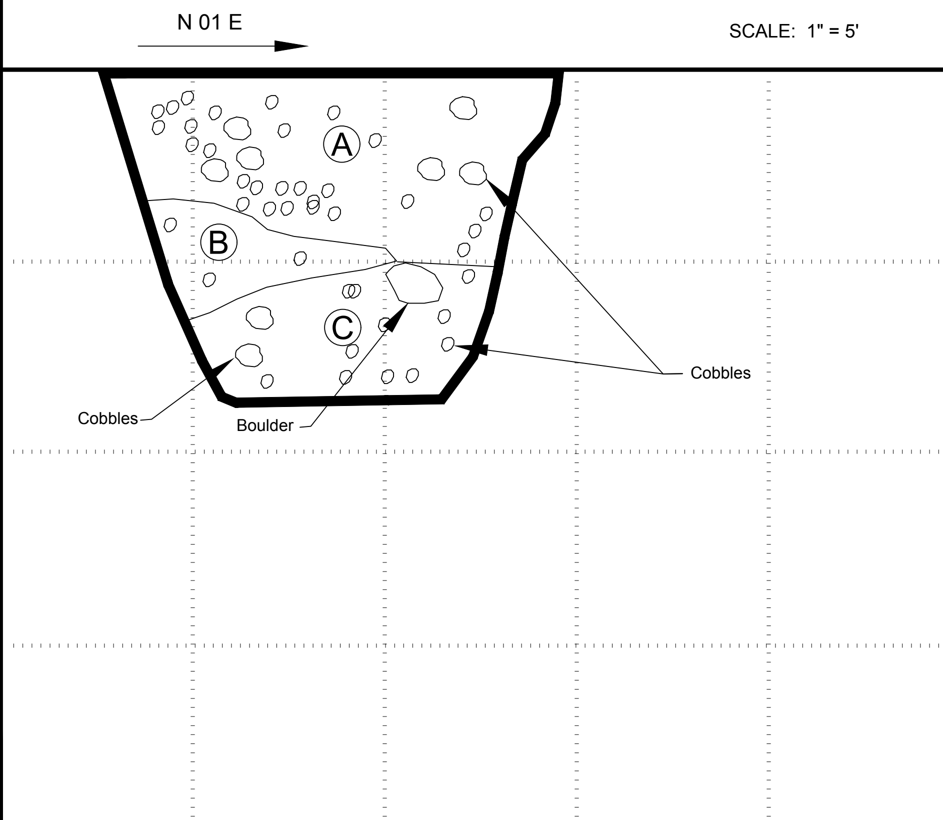
# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.**
**T-2**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Backhoe | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 01 E | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | b | | 2 | A: ALLUVIUM: Dark Brown Gravelly fine to medium Sand, trace coarse Sand, extensive Cobbles, dense-dry |
| 5 | b | | 2 | B: Light Brown Gravelly fine to coarse Sand, occasional Cobbles, dense-dry |
| | b | | 3 | C: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional Boulders, dense-dry to damp |
| | | | | Refusal @ 8.5 feet due to dense Cobbles and Boulders |
| 10 | | | | |
| 15 | | | | |

## GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-8**

# SOUTHERN CALIFORNIA GEOTECHNICAL

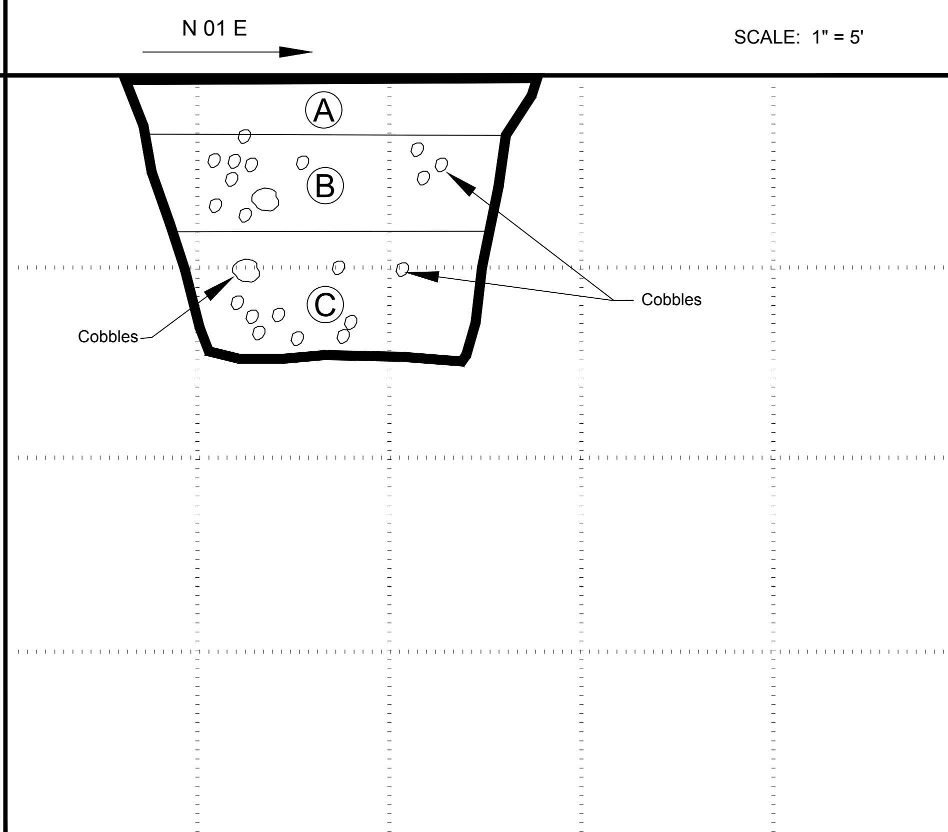
**TRENCH NO. T-3**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Backhoe | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 01 E | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| 0 | b | | 6 | A: ALLUVIUM: Dark Brown Silty fine Sand, little fine to coarse Gravel, medium dense-damp |
| 2 | b | | 6 | B: Brown fine Sand, little fine to coarse Gravel, trace Silt, occasional Cobbles, medium dense-damp |
| 4 | b | | 2 | C: Light Brown fine to coarse Sand, little fine to coarse Gravel, extensive Cobbles, dense-dry to damp |
| 7 | b | | 3 | |
| | | | | Refusal @ 7 feet due to dense Cobbles |

Depth scale: 5, 10, 15

## GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-9**

# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO. T-4**

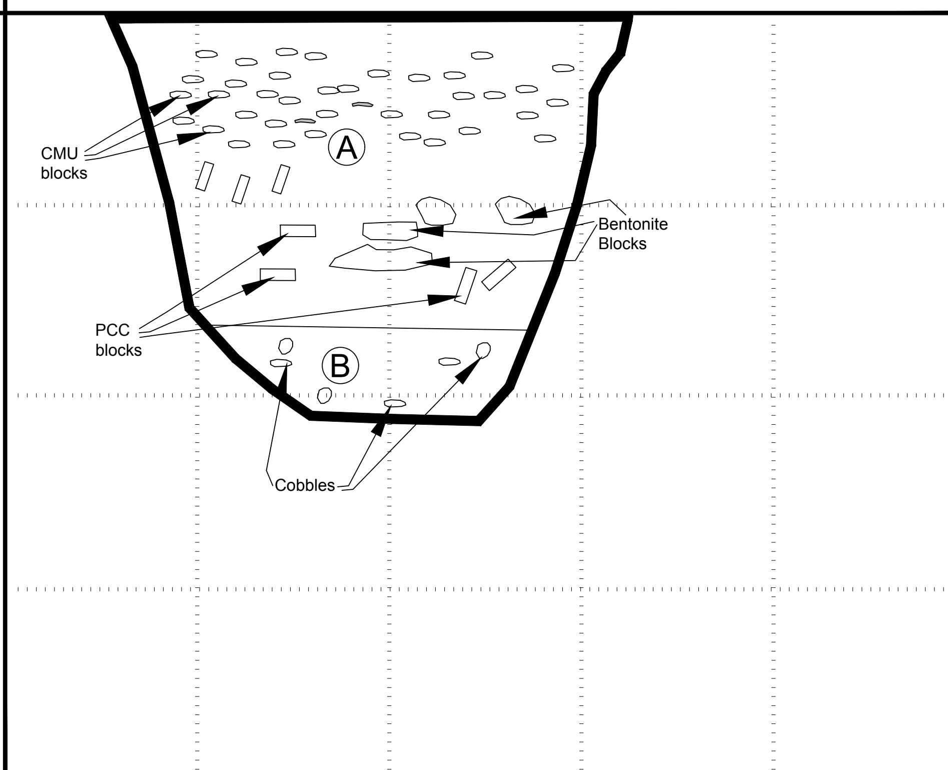
| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Excavator | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 01 E | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | | | | A: UNDOCUMENTED FILL: Brown fine to coarse Sand, little fine to coarse Gravel, trace Brick fragments, medium dense-moist |
| | b | | 6 | |
| 5 | | | | |
| | | | | @ 6 feet, Bentonite Blocks |
| | b | | 5 | B: UNDOCUMENTED FILL: Dark Brown fine to coarse Sand, trace Asphaltic concrete fragments, medium dense-moist<br>@ 9.5 feet, occasional Cobbles |
| 10 | | | | Trench Terminated @ 10.5 feet |
| 15 | | | | |

## GRAPHIC REPRESENTATION

N 01 E

SCALE: 1" = 5'




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER (RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-10**

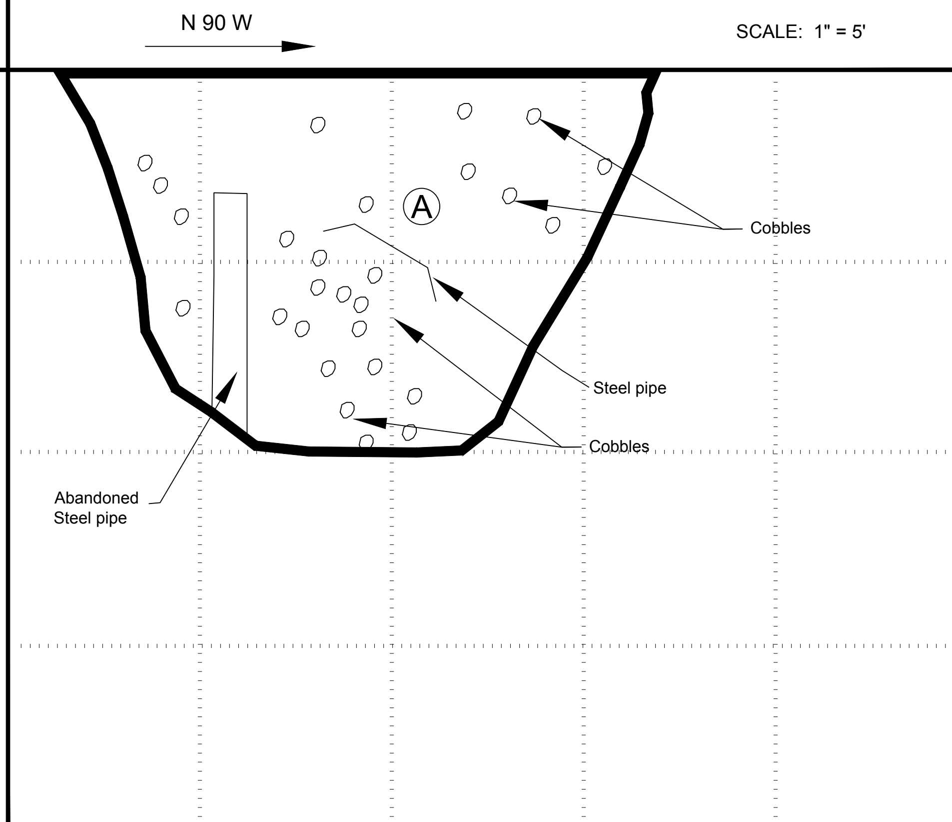
# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.**
**T-5**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Excavator | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jamie Hayward | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 90 W | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: feet msl | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | | | | A: UNDOCUMENTED FILL: Brown Gravelly fine to coarse Sand, extensive Cobbles, occasional steel pipes, dense to very dense-dry to damp |
| 5 | | | | |
| 10 | b | | 2 | Trench Terminated @ 10 feet |
| 15 | | | | |

GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-11**

# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO. T-6**

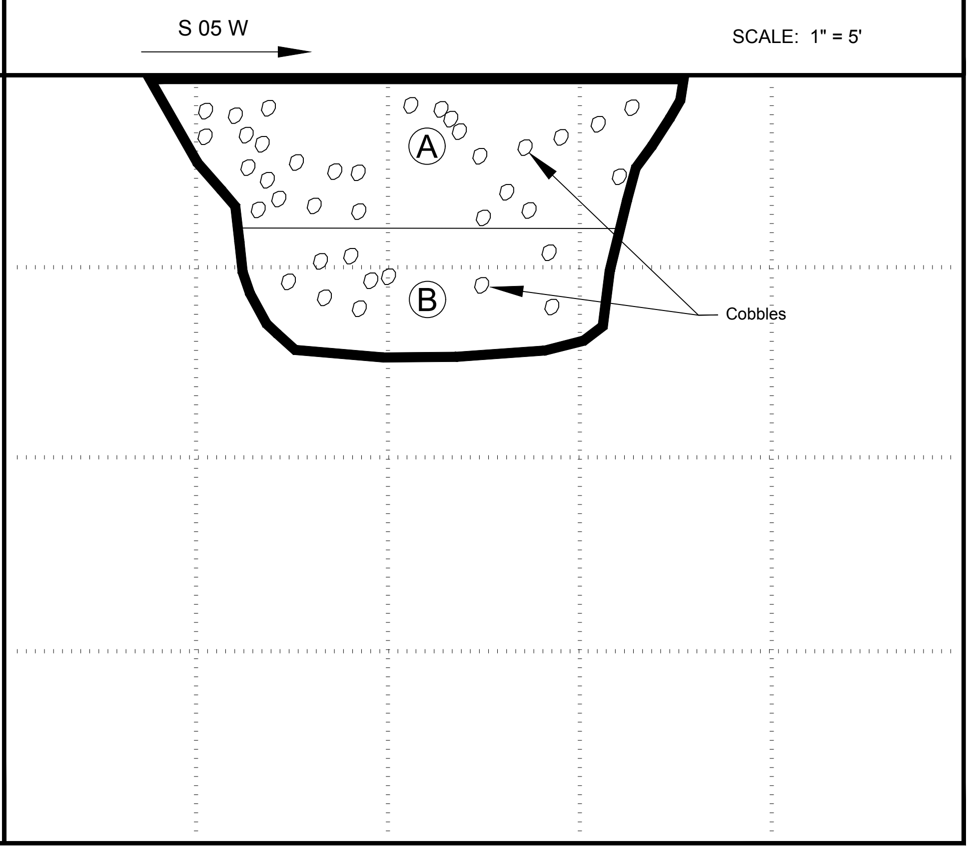
JOB NO.: 21G119-1
PROJECT: Proposed Banning Industrial Park
LOCATION: Banning, California
DATE: 2/25/2021

EQUIPMENT USED: Backhoe
LOGGED BY: Jose Zuniga
ORIENTATION: S 05 W
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | b | | 2 | A: ALLUVIUM: Brown Gravelly fine to coarse Sand, extensive Cobbles, dense-dry to damp |
| 5 | b | | 3 | B: Brown Gravelly fine to medium Sand, extensive Cobbles, very dense-damp |
| | b | | 3 | |
| | | | | Refusal @ 7.5 feet, due to dense Cobbles |
| 10 | | | | |
| 15 | | | | |

GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-12**

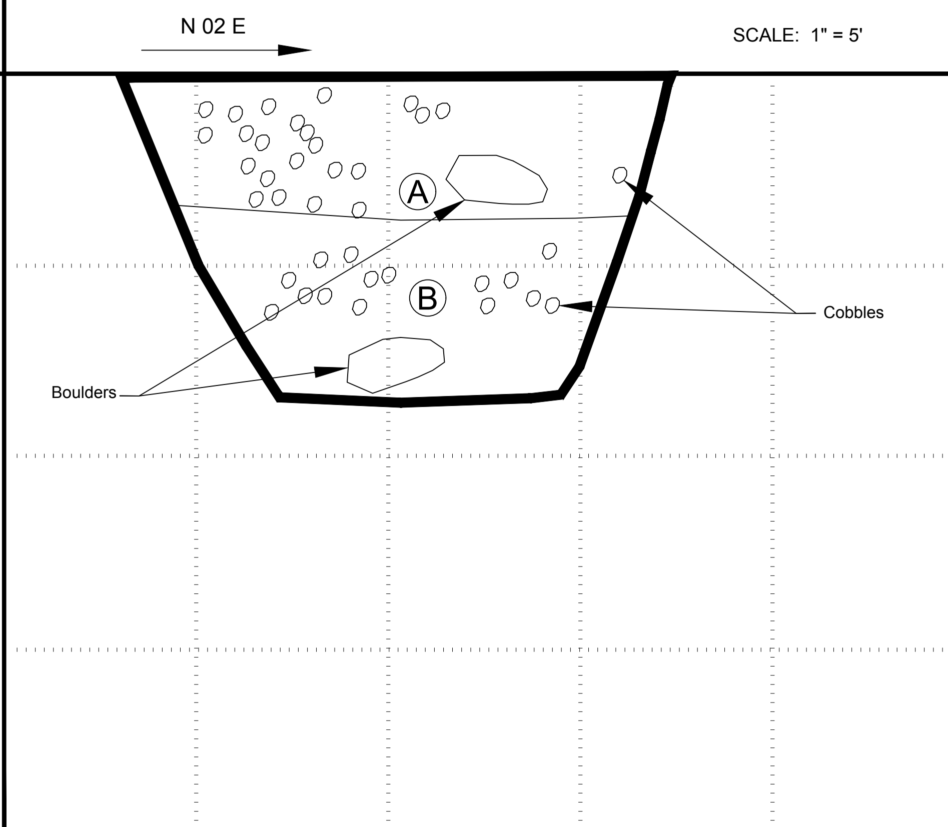
# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.**
**T-7**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Backhoe | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 02 E | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | b | | 2 | A: ALLUVIUM: Brown fine to coarse Sand, trace fine to coarse Gravel, extensive Cobbles, occasional Boulders, dense-dry to damp |
| | | | | B: Brown Gravelly fine to coarse Sand, occasional Cobbles, occasional Boulders, very dense-dry to damp |
| 5 | | | | |
| | b | | 2 | |
| | | | | Refusal @ 8.5 feet, due to dense Cobbles |
| 10 | | | | |
| 15 | | | | |

GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-13**

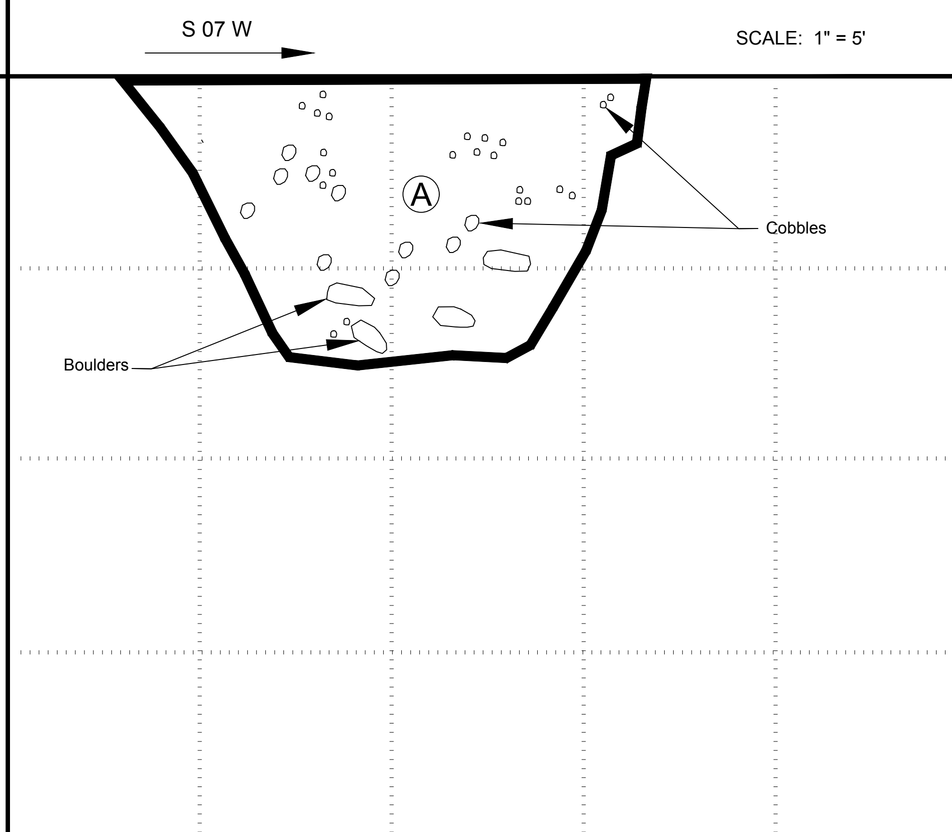
# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO. T-8**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Backhoe | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: S 07 W | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| 0 | | | | A: ALLUVIUM: Brown Gravelly fine to coarse Sand, occasional Cobbles, occasional Boulders, dense-dry to damp |
| 3 | b | | 2 | |
| 7 | b | | 2 | |
| 7.5 | | | | Refusal @ 7.5 feet, due to dense Boulders |

## GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER (RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-14**

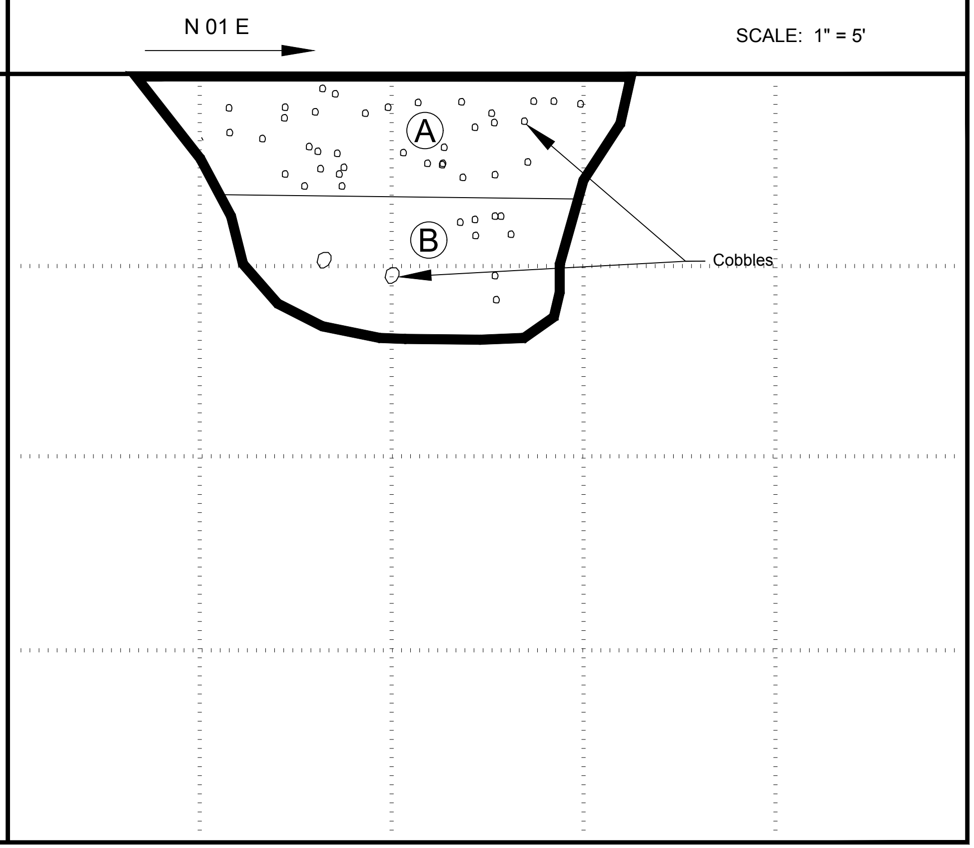
# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.**
**T-9**

| | | |
|---|---|---|
| JOB NO.: 21G119-1 | EQUIPMENT USED: Backhoe | WATER DEPTH: Dry |
| PROJECT: Proposed Banning Industrial Park | LOGGED BY: Jose Zuniga | SEEPAGE DEPTH: Dry |
| LOCATION: Banning, California | ORIENTATION: N 01 E | READINGS TAKEN: At Completion |
| DATE: 2/25/2021 | ELEVATION: --- | |

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | | | | A: ENGINEERED FILL: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, dense-dry to damp |
| | b | | 2 | |
| 5 | | | | |
| | b | | 3 | B: ENGINEERED FILL: Dark Brown Gravelly fine to coarse Sand, occasional Cobbles, dense-damp |
| | | | | Refusal @ 6.5 feet, due to dense Cobbles |
| 10 | | | | |
| 15 | | | | |

GRAPHIC REPRESENTATION




KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-15**

# SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.**
**T-10**

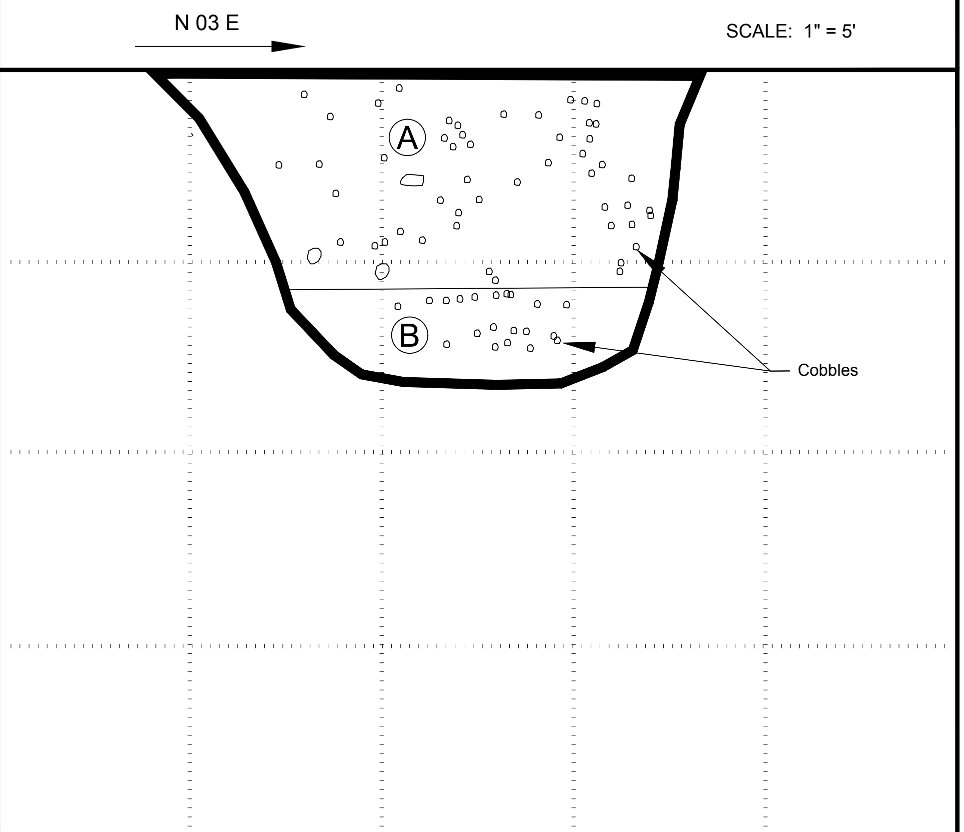
JOB NO.: 21G119-1
PROJECT: Proposed Banning Industrial Park
LOCATION: Banning, California
DATE: 2/25/2021

EQUIPMENT USED: Backhoe
LOGGED BY: Jose Zuniga
ORIENTATION: N 03 E
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

| DEPTH | SAMPLE | DRY DENSITY (PCF) | MOISTURE (%) | EARTH MATERIALS DESCRIPTION |
|---|---|---|---|---|
| | | | | A: ENGINEERED FILL: Brown Gravelly fine to coarse Sand, trace Silt, extensive Cobblers, dense-damp |
| | b | | 3 | |
| 5 | | | | |
| | | | | B: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, extensive Cobbles, very dense-damp |
| | b | | 4 | |
| | | | | Refusal @ 8.5 feet, due to very dense Cobbles |
| 10 | | | | |
| 15 | | | | |

GRAPHIC REPRESENTATION

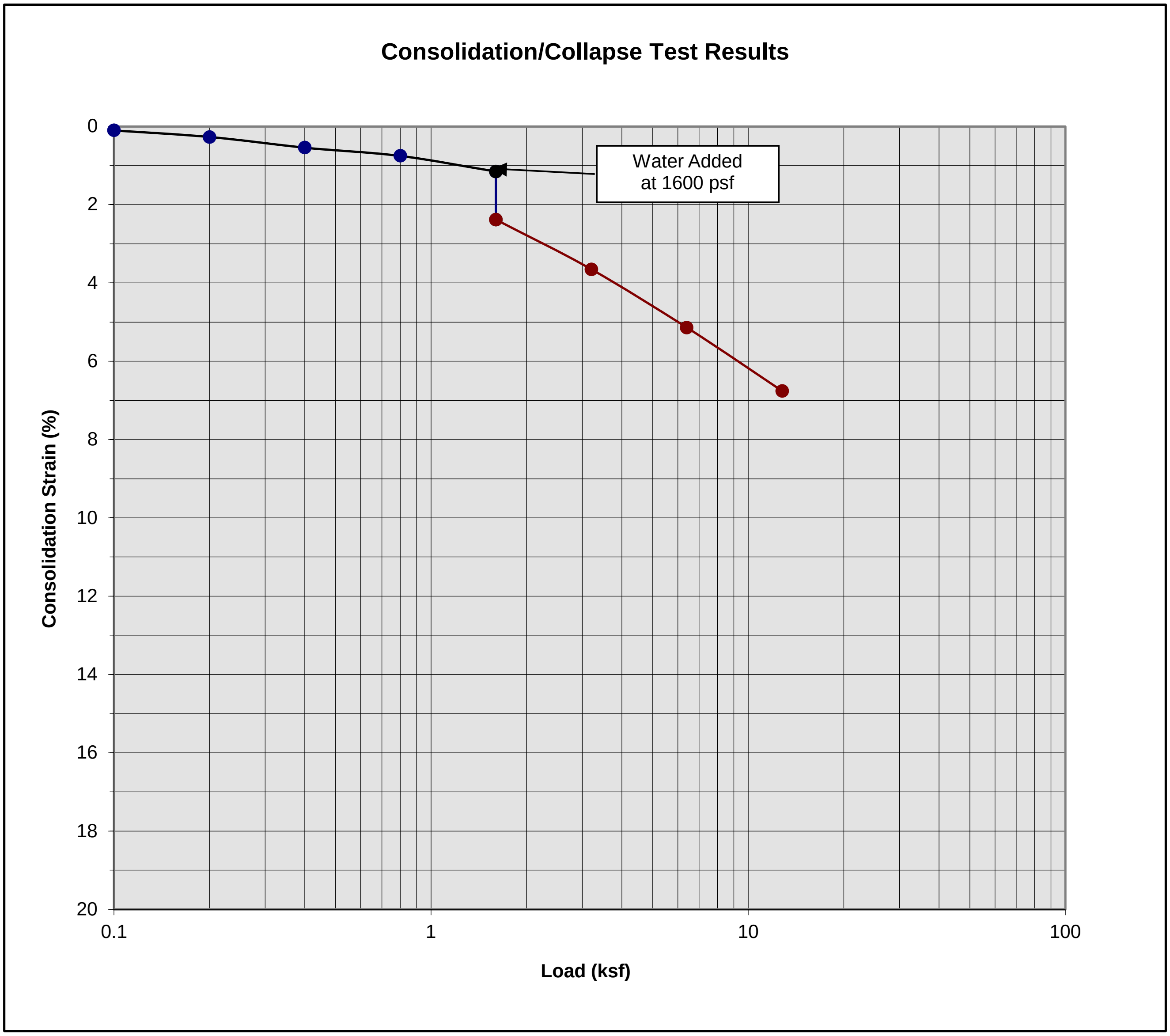



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

**TRENCH LOG**

**PLATE B-16**

# APPENDIX C



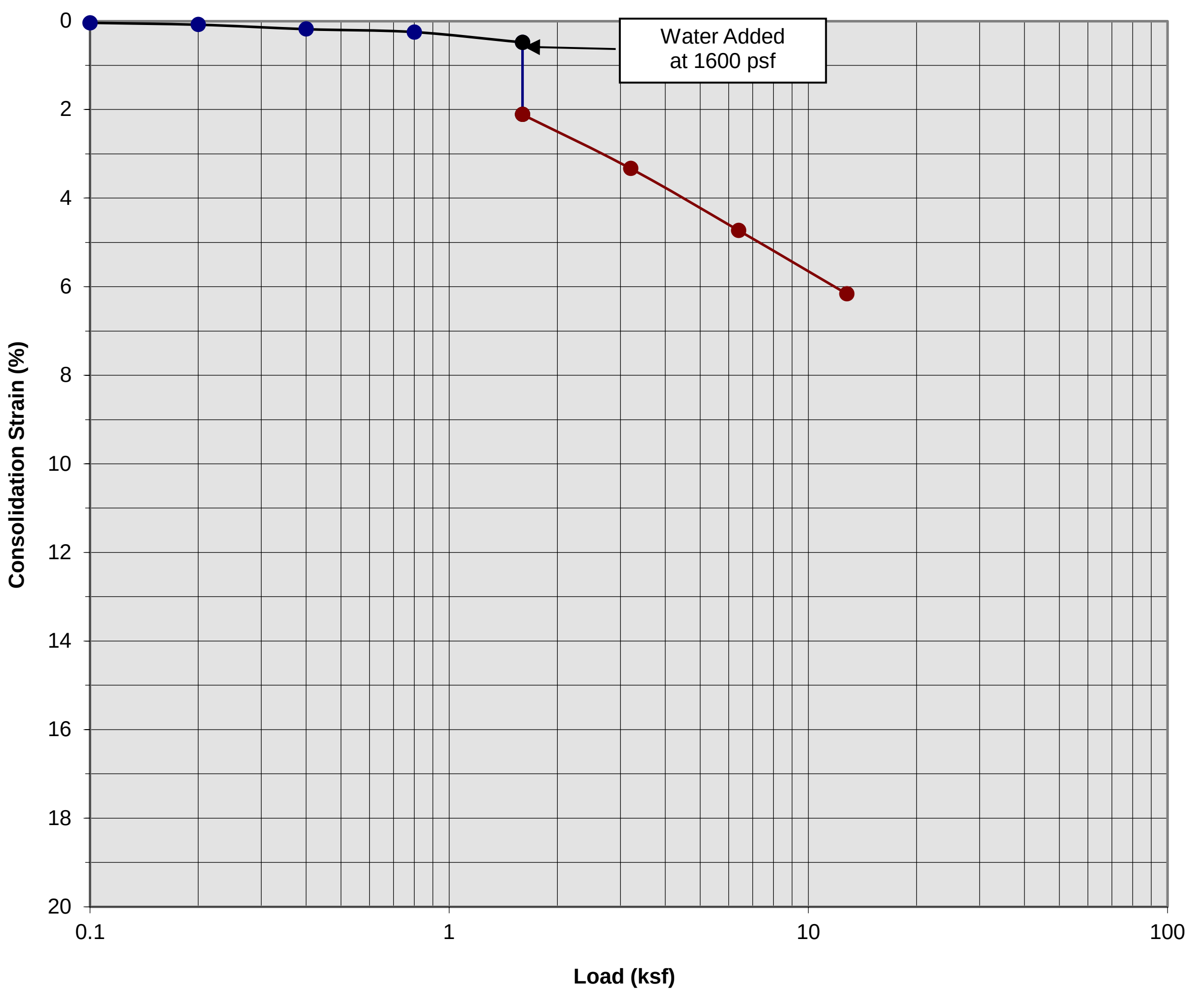

Classification: ENGINEERED FILL: Silty fine to coarse Sand, trace fine to coarse Gravel

| | | | |
|---|---|---|---|
| Boring Number: | B-5 | Initial Moisture Content (%) | 3 |
| Sample Number: | --- | Final Moisture Content (%) | 17 |
| Depth (ft) | 1 to 2 | Initial Dry Density (pcf) | 127.1 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 136.2 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 1.23 |

Proposed Banning Industrial Park
Banning, California
Project No. 21G119-1
**PLATE C-1**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

## Consolidation/Collapse Test Results

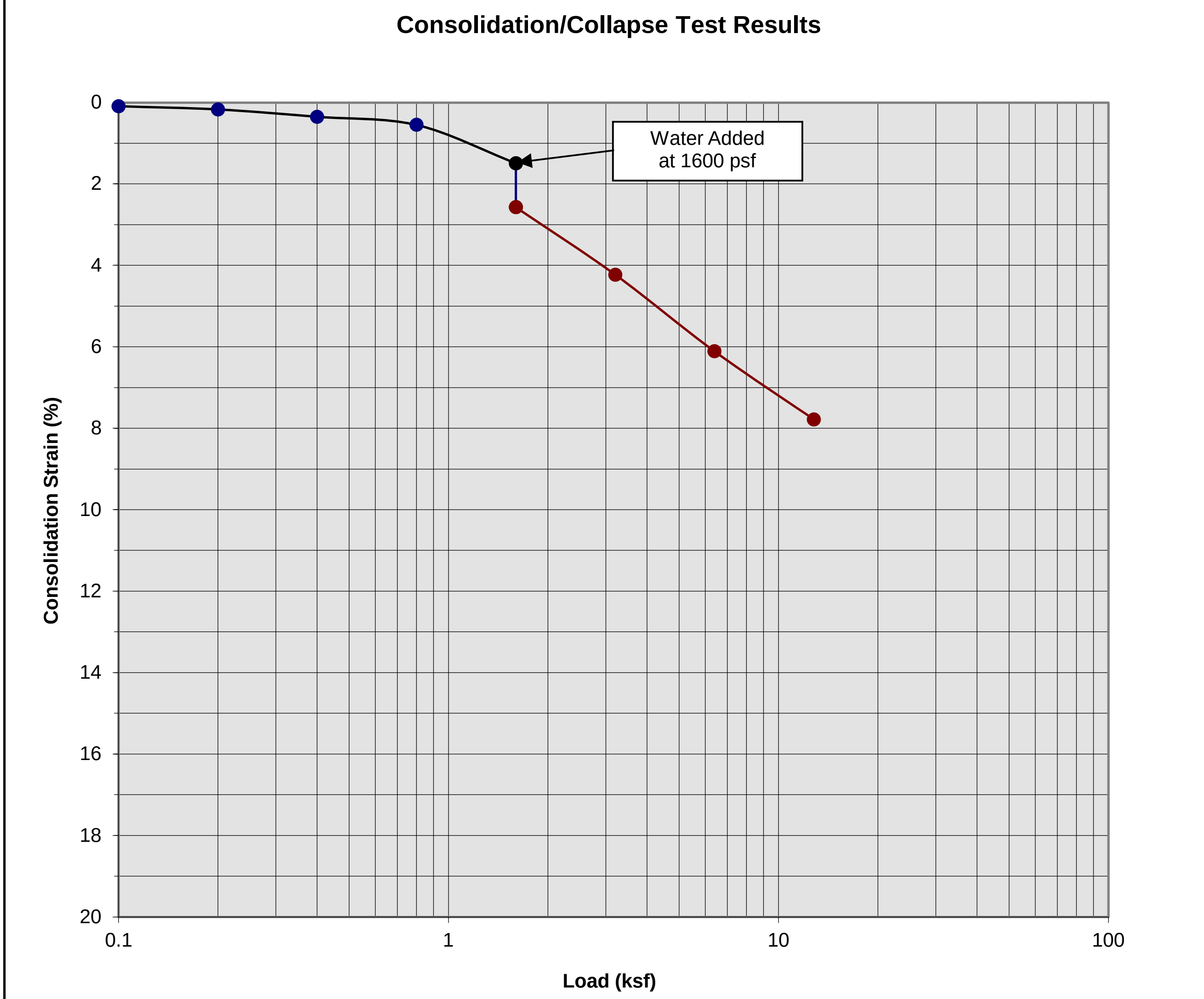



Classification: ENGINEERED FILL: Silty fine to coarse Sand, trace fine to coarse Gravel

| | | | |
|---|---|---|---|
| Boring Number: | B-5 | Initial Moisture Content (%) | 2 |
| Sample Number: | --- | Final Moisture Content (%) | 11 |
| Depth (ft) | 5 to 6 | Initial Dry Density (pcf) | 125.2 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 133.2 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 1.63 |

Proposed Banning Industrial Park
Banning, California
Project No. 21G119-1
**PLATE C-2**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
*A California Corporation*

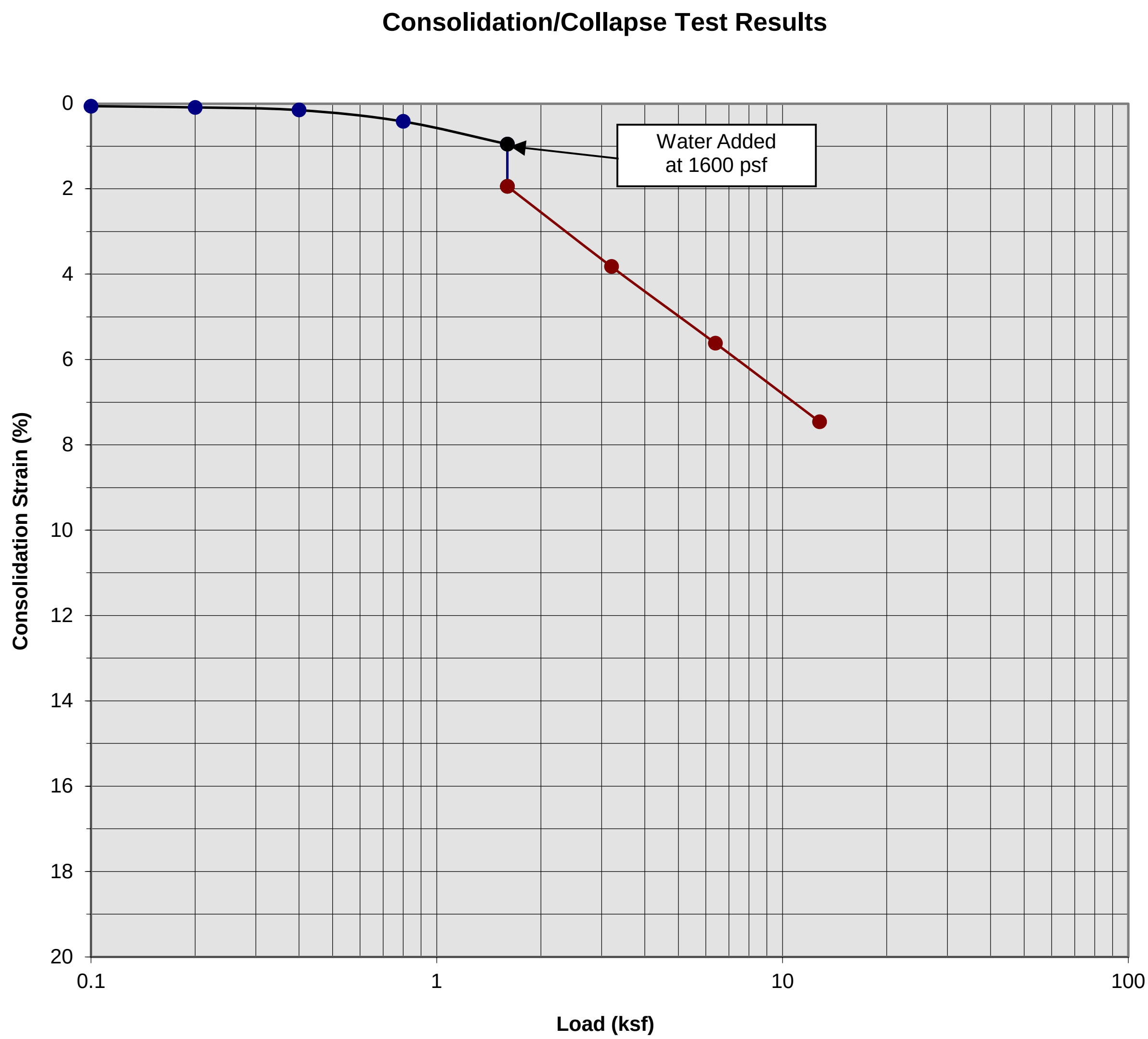



Classification: ENGINEERED FILL: Gravelly fine to coarse Sand, trace Silt

| | | | |
|---|---|---|---|
| Boring Number: | B-5 | Initial Moisture Content (%) | 2 |
| Sample Number: | --- | Final Moisture Content (%) | 12 |
| Depth (ft) | 7 to 8 | Initial Dry Density (pcf) | 114.1 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 124.0 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 1.08 |

Proposed Banning Industrial Park
Banning, California
Project No. 21G119-1
**PLATE C-3**

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

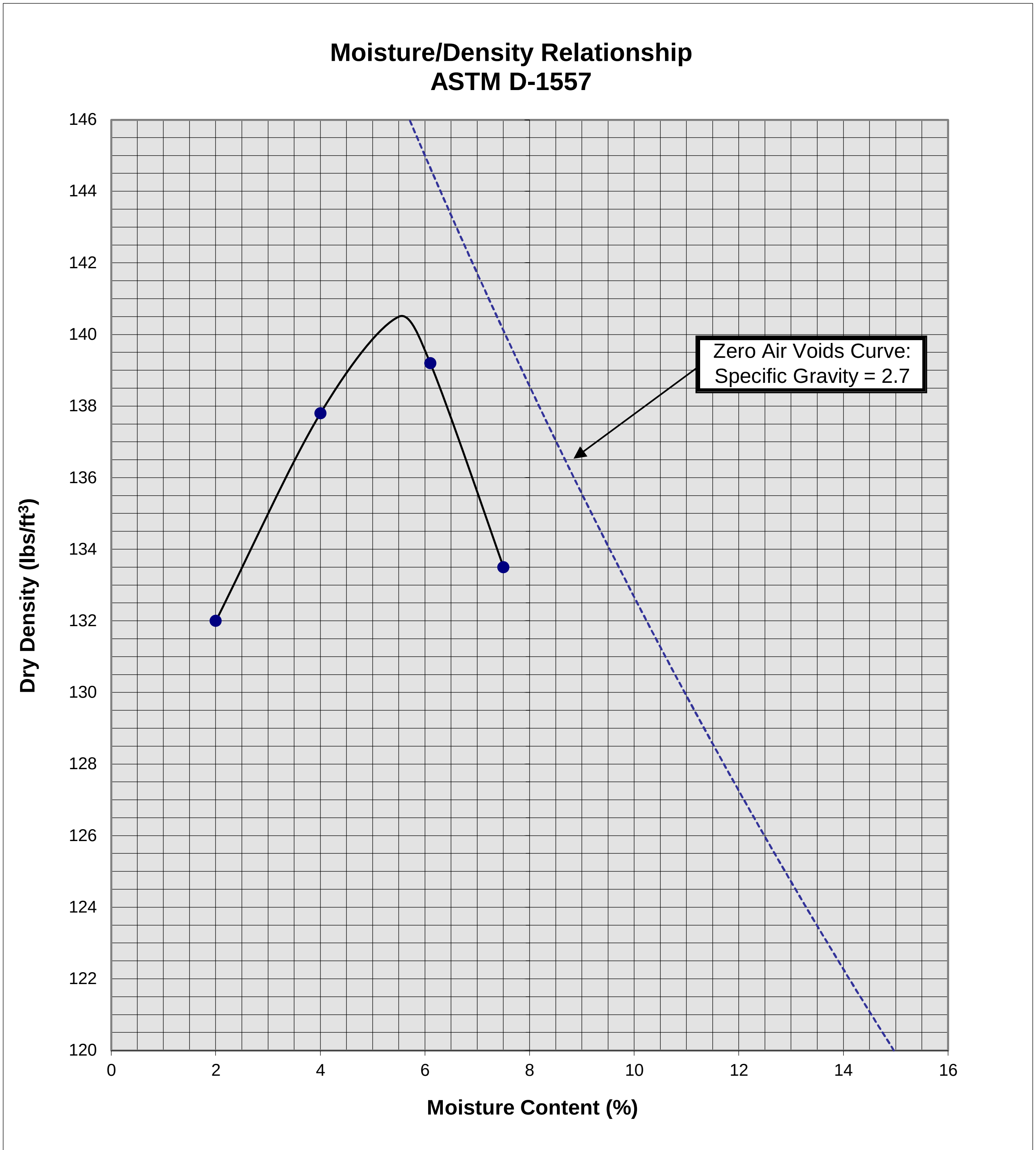



Classification: ENGINEERED FILL: Gravelly fine to coarse Sand, trace Silt

| | | | |
|---|---|---|---|
| Boring Number: | B-5 | Initial Moisture Content (%) | 3 |
| Sample Number: | --- | Final Moisture Content (%) | 11 |
| Depth (ft) | 9 to 10 | Initial Dry Density (pcf) | 126.4 |
| Specimen Diameter (in) | 2.4 | Final Dry Density (pcf) | 136.5 |
| Specimen Thickness (in) | 1.0 | Percent Collapse (%) | 0.99 |

Proposed Banning Industrial Park
Banning, California
Project No. 21G119-1
**PLATE C-4**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation




| | |
|---|---|
| Soil ID Number | B-4 @ 0-5' |
| Optimum Moisture (%) | 5.5 |
| Maximum Dry Density (pcf) | 140.5 |
| Soil Classification | Gray Brown Gravelly fine to coarse Sand |

Proposed Banning Industrial Park
Banning, California
Project No. 21G119-1
**PLATE C-5**

SOCALGEO SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

# APPENDIX D

# GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

## General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

## Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

  - Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

  - To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

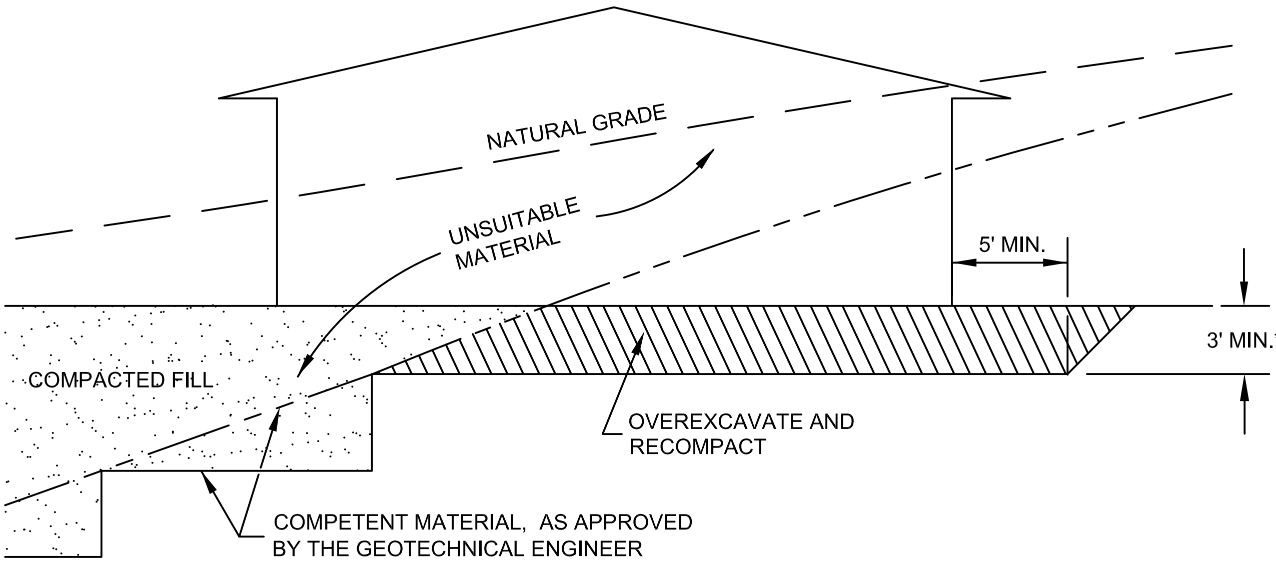
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

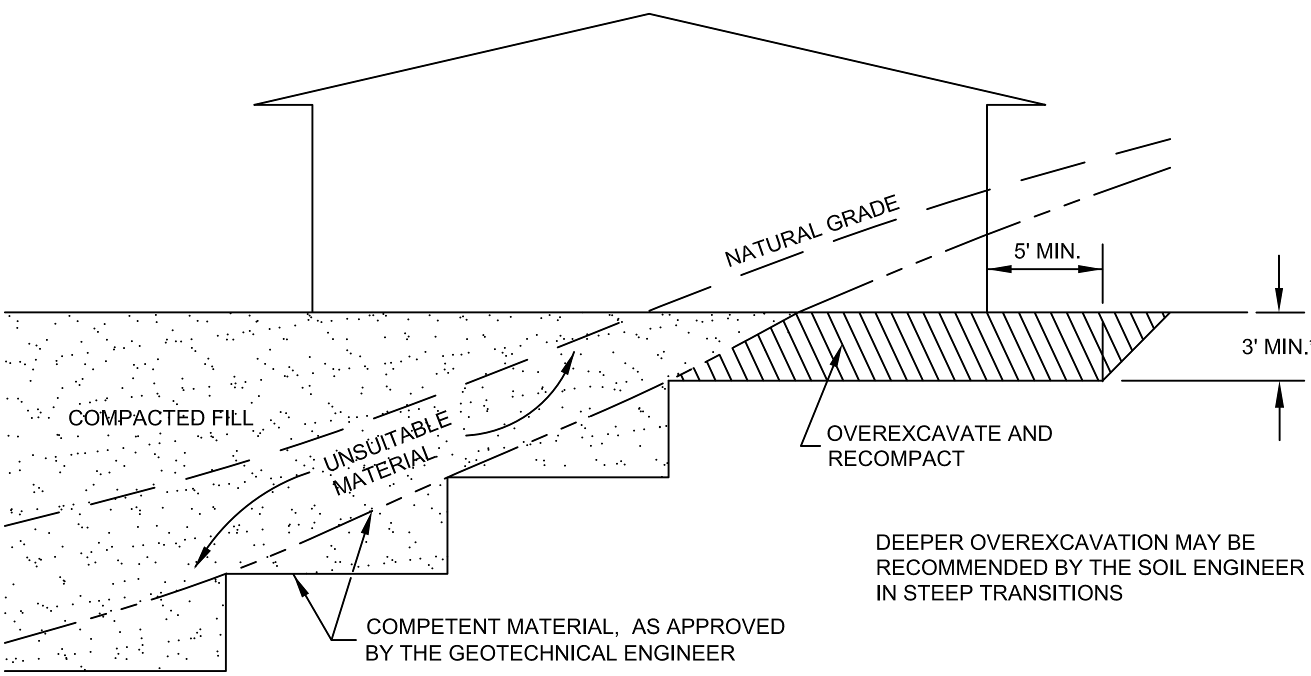
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.

## CUT LOT



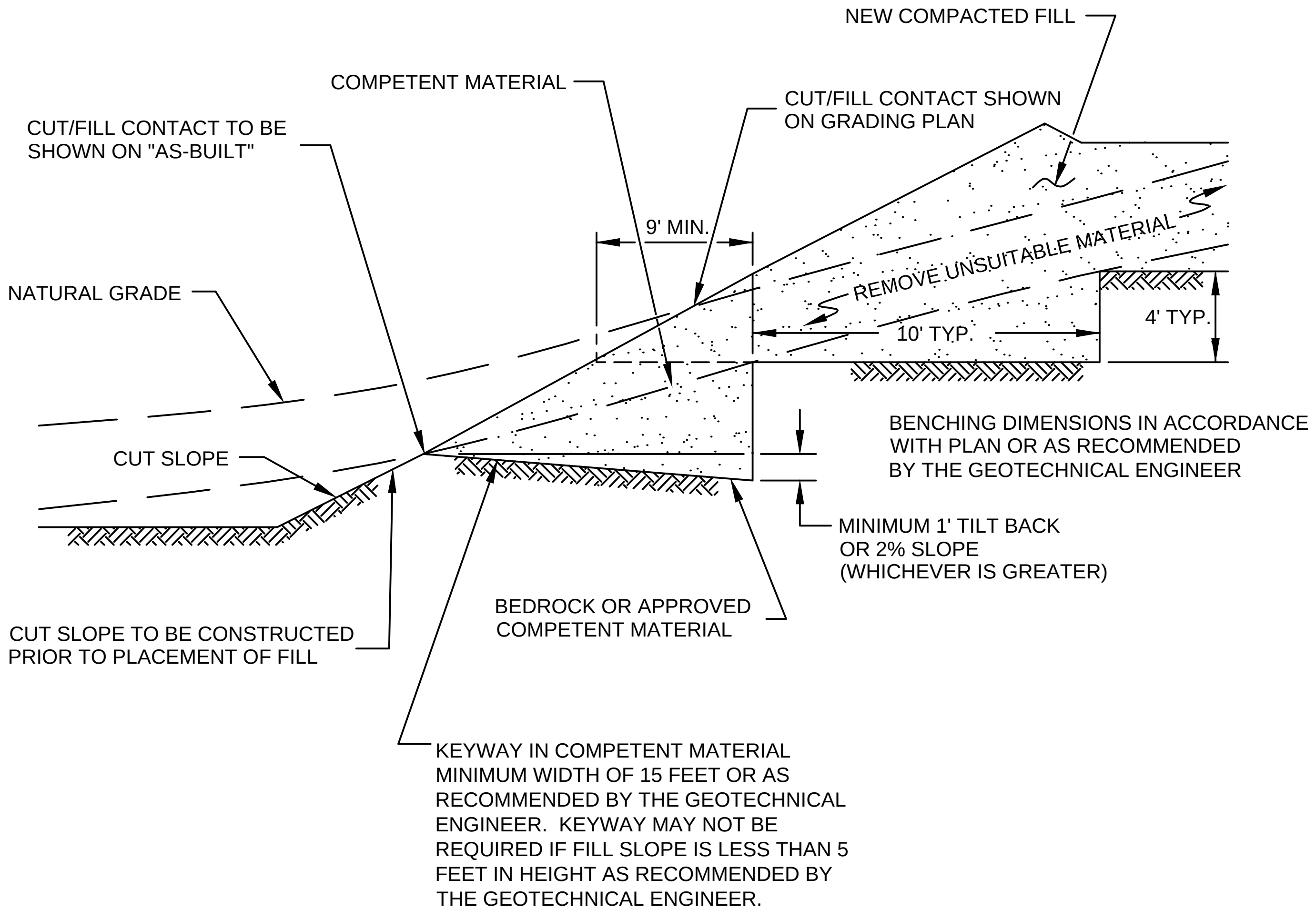

## CUT/FILL LOT (TRANSITION)



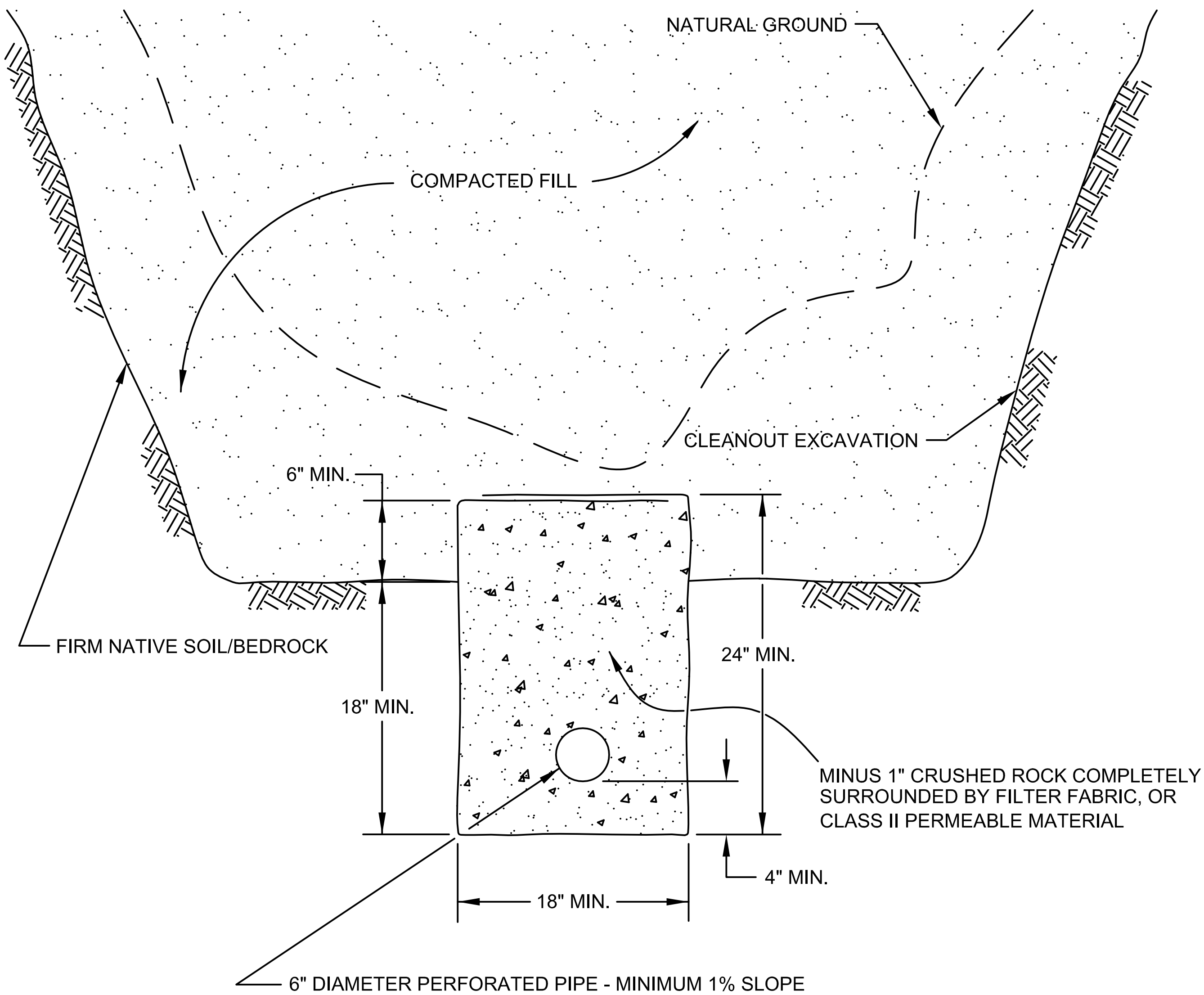

DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

| | |
|---|---|
| **TRANSITION LOT DETAIL** | |
| GRADING GUIDE SPECIFICATIONS | |
| NOT TO SCALE | SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: JAS<br>CHKD: GKM | |
| **PLATE D-1** | |




| | |
|---|---|
| **FILL ABOVE CUT SLOPE DETAIL** | |
| GRADING GUIDE SPECIFICATIONS | |
| NOT TO SCALE | SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL |
| DRAWN: JAS<br>CHKD: GKM | |
| **PLATE D-2** | |




| PIPE MATERIAL | DEPTH OF FILL OVER SUBDRAIN |
|---|---|
| ADS (CORRUGATED POLETHYLENE) | 8 |
| TRANSITE UNDERDRAIN | 20 |
| PVC OR ABS: SDR 35 | 35 |
| SDR 21 | 100 |

**SCHEMATIC ONLY**
**NOT TO SCALE**

**CANYON SUBDRAIN DETAIL**

GRADING GUIDE SPECIFICATIONS

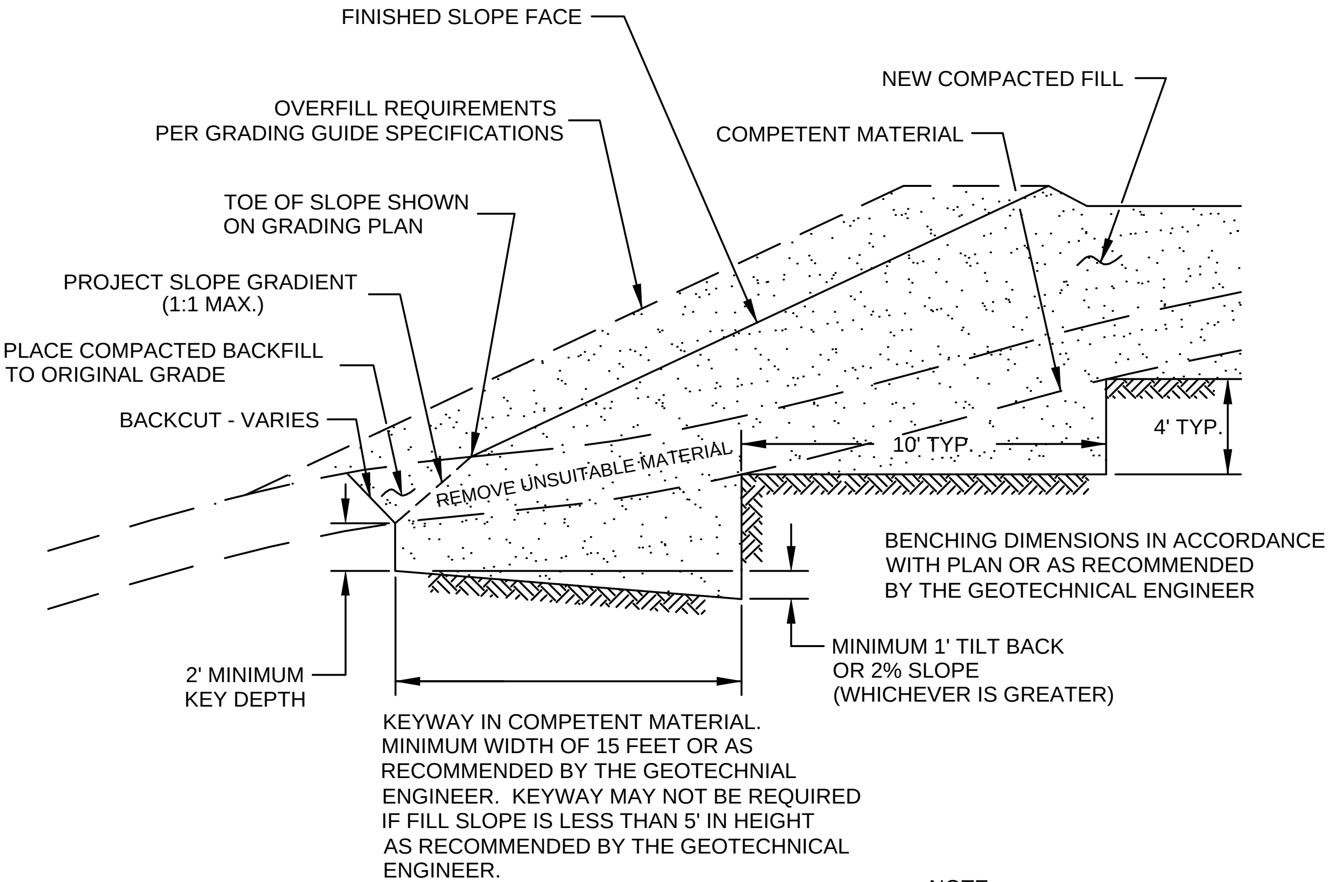
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-3**

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL




NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

**FILL ABOVE NATURAL SLOPE DETAIL**

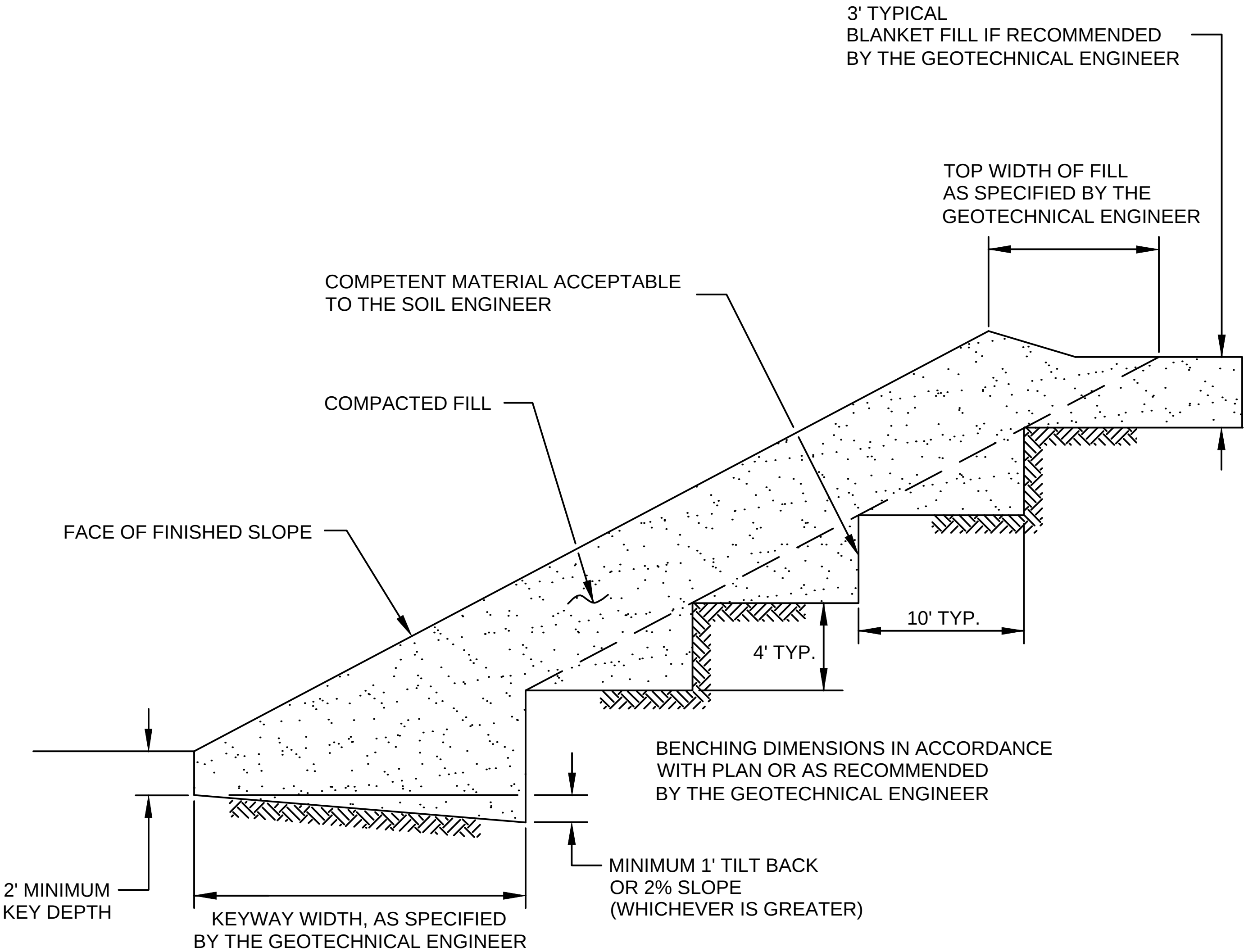
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-4**

SOUTHERN CALIFORNIA GEOTECHNICAL




**STABILIZATION FILL DETAIL**

GRADING GUIDE SPECIFICATIONS

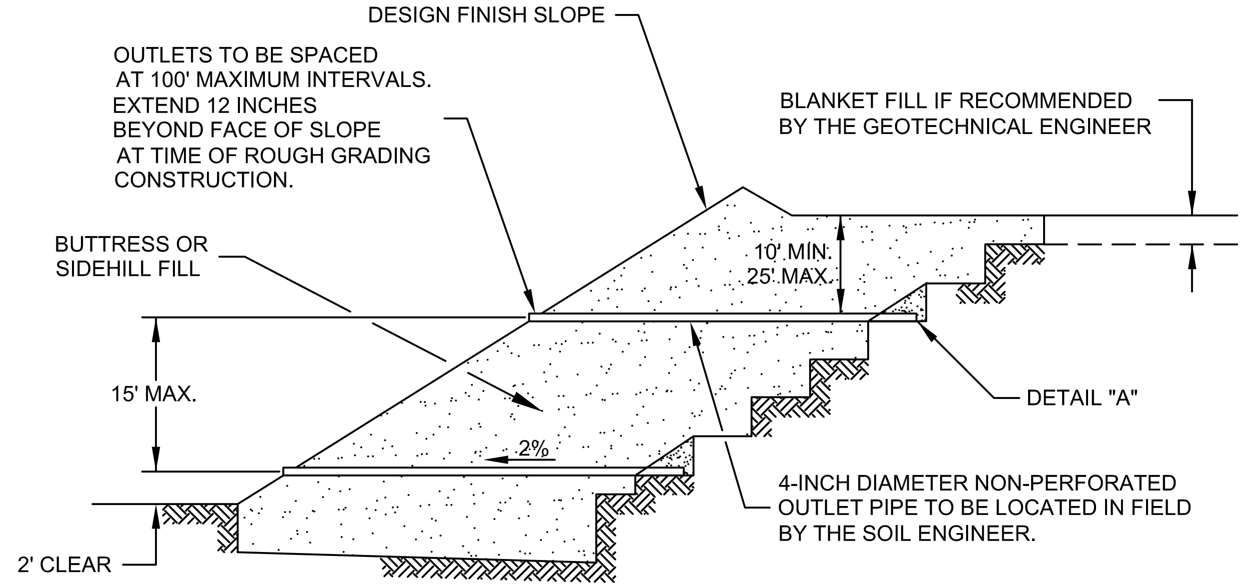
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-5**

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



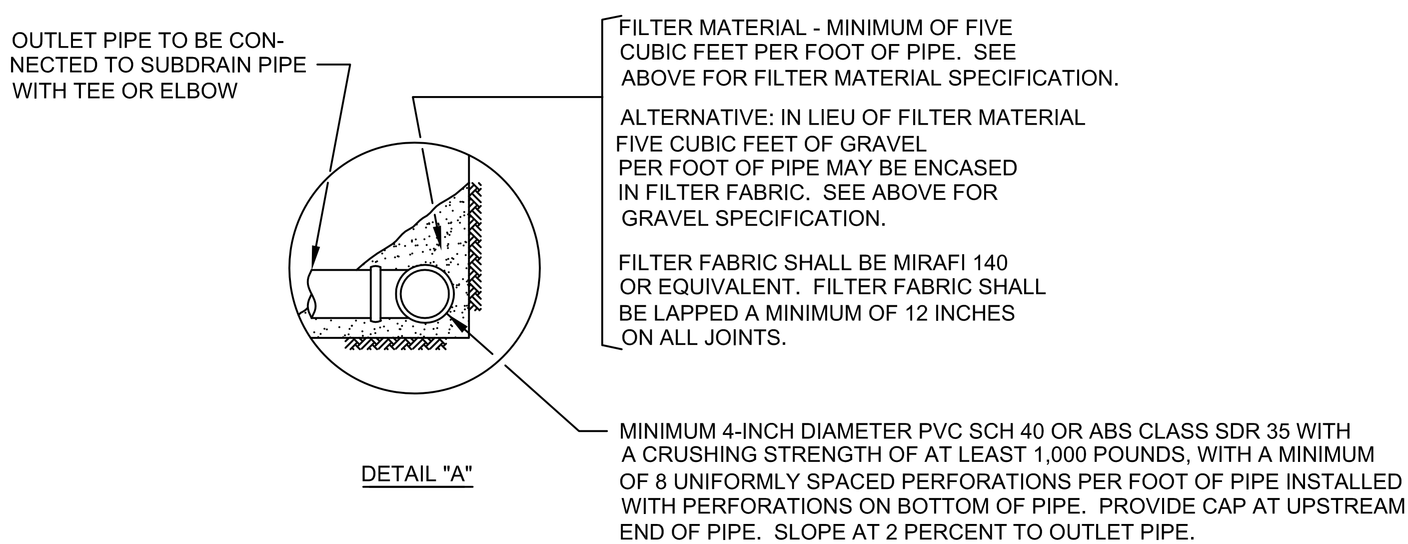

"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

| SIEVE SIZE | PERCENTAGE PASSING |
|---|---|
| 1" | 100 |
| 3/4" | 90-100 |
| 3/8" | 40-100 |
| NO. 4 | 25-40 |
| NO. 8 | 18-33 |
| NO. 30 | 5-15 |
| NO. 50 | 0-7 |
| NO. 200 | 0-3 |

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

| SIEVE SIZE | MAXIMUM PERCENTAGE PASSING |
|---|---|
| 1 1/2" | 100 |
| NO. 4 | 50 |
| NO. 200 | 8 |

SAND EQUIVALENT = MINIMUM OF 50




DETAIL "A"

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

**SLOPE FILL SUBDRAINS**

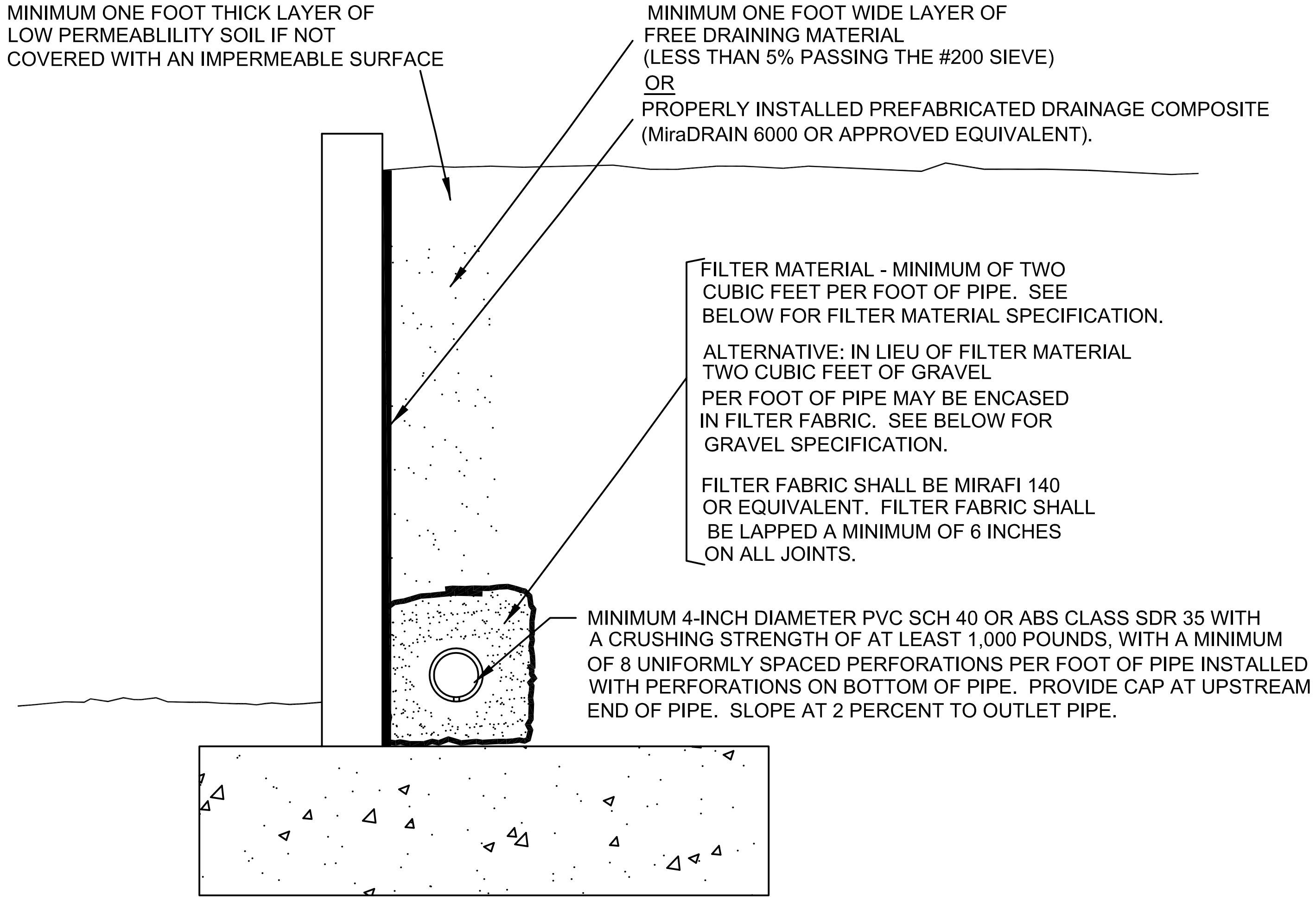
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

**PLATE D-6**

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL




"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

| SIEVE SIZE | PERCENTAGE PASSING |
|---|---|
| 1" | 100 |
| 3/4" | 90-100 |
| 3/8" | 40-100 |
| NO. 4 | 25-40 |
| NO. 8 | 18-33 |
| NO. 30 | 5-15 |
| NO. 50 | 0-7 |
| NO. 200 | 0-3 |

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

| SIEVE SIZE | MAXIMUM PERCENTAGE PASSING |
|---|---|
| 1 1/2" | 100 |
| NO. 4 | 50 |
| NO. 200 | 8 |

SAND EQUIVALENT = MINIMUM OF 50

**RETAINING WALL BACKDRAINS**

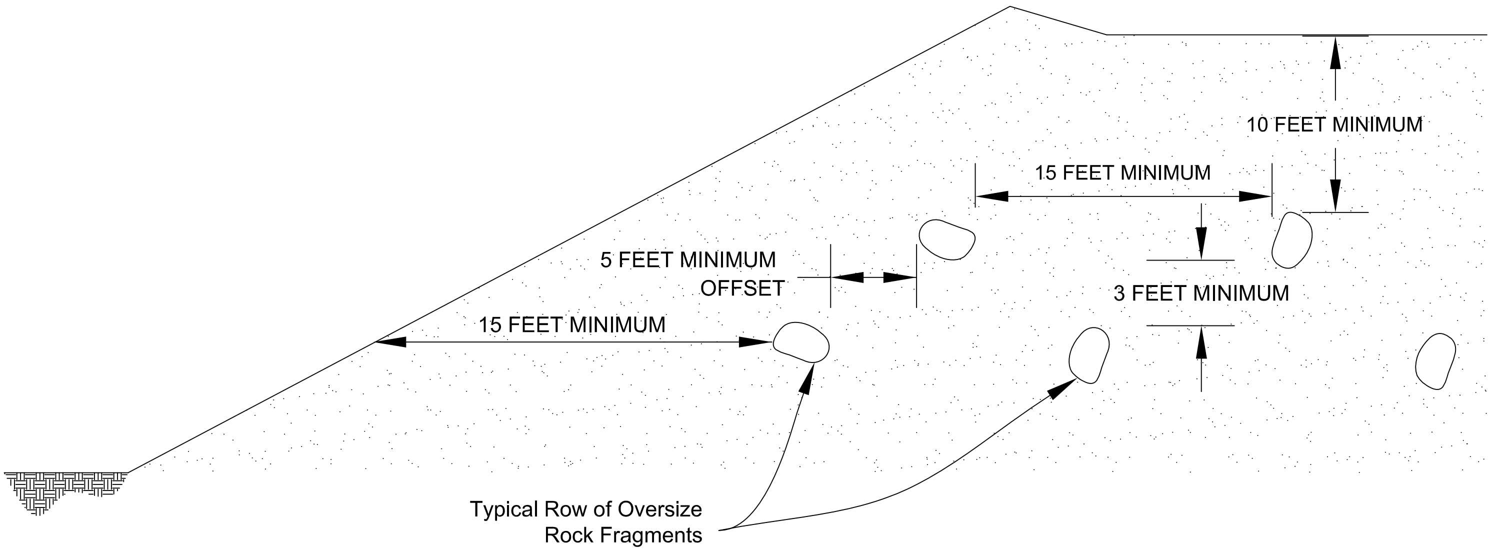
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

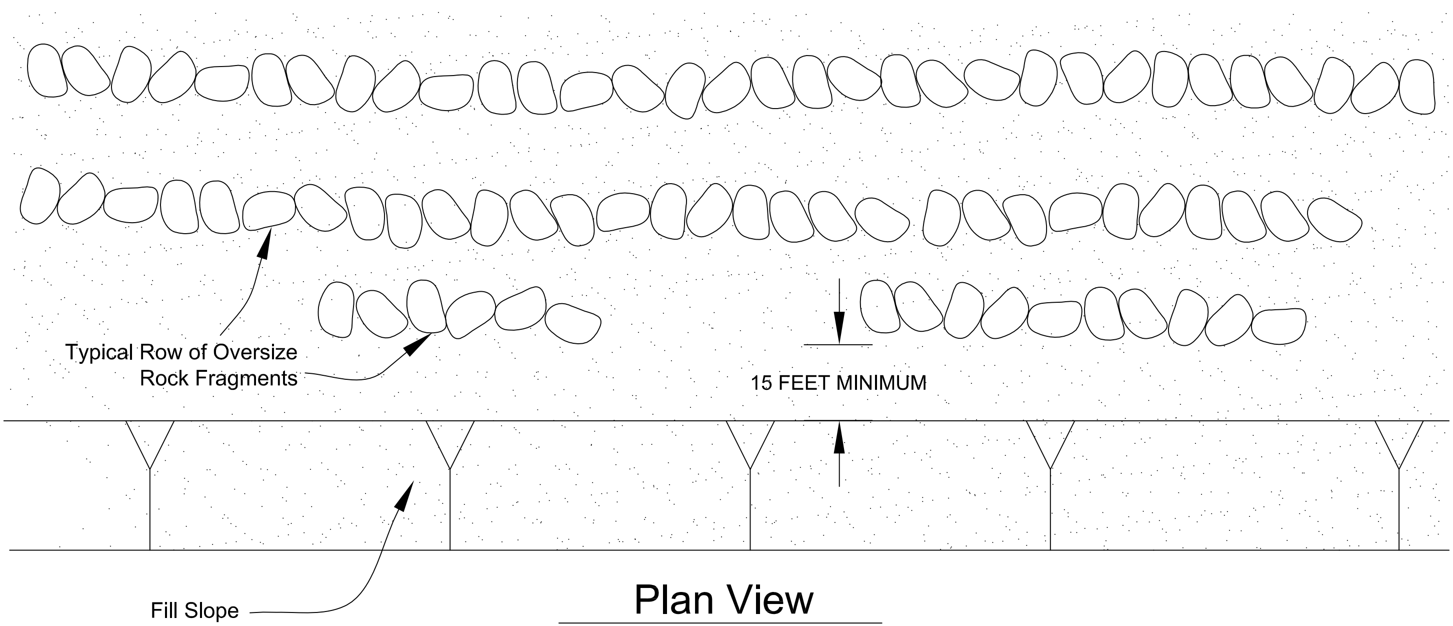
DRAWN: JAS
CHKD: GKM

**PLATE D-7**

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL




Section View




Plan View

| PLACEMENT OF OVERSIZED MATERIAL |
| --- |
| GRADING GUIDE SPECIFICATIONS |

NOT TO SCALE

DRAWN: PM
CHKD: GKM

**PLATE D-8**

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

# APPENDIX E

TERRA GEOSCIENCES

# GROUND-MOTION HAZARD ANALYSIS

# PROPOSED BANNING INDUSTRIAL PARK

# NEC OF HATHAWAY AND NICOLET STREETS

# BANNING, RIVERSIDE COUNTY, CALIFORNIA

Project No. 233943-1

May 1, 2023

**Prepared for:**

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Suite E
Yorba Linda, CA  92887

Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Suite E
Yorba Linda, CA 92887

Attention: Mr. Ricardo Frias, PE, Project Engineer

Regarding: Ground-Motion Hazard Analysis
Proposed Banning Industrial Project
NEC of Hathaway and Nicolet Streets
Banning, Riverside County, California
SCG Project No. 21G119-1

## INTRODUCTION

At your request, this firm has prepared a ground-motion hazard analysis report for the proposed project, as referenced above. 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 current 2022 California Building Code (CBC). Our work included performing a seismic shear-wave study for determining the Site Classification and $V_{S30}$ input values for this analysis. The scope of services provided for this evaluation included the following:

- ***Review of available published and unpublished geologic/seismic data in our files pertinent to the site, including a field reconnaissance.***
- ***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

Plate 1- Seismic Line Location Map
Appendix A - Shear-Wave Survey
Appendix B - Site Specific Ground Motion Analysis
Appendix C - References

## PROJECT SUMMARY

Based on the information that has been provided, we understand that a commercial/industrial building is proposed to be constructed within the subject property, with a footprint area of 1,407,230± $ft^2$ and will be of concrete tilt-up construction. Associated surrounding flatwork and landscaping is also proposed. For this project, we have performed a field reconnaissance, reviewed pertinent available geologic and geotechnical data in our files, along with performing a site-specific seismic shear-wave survey.

To aid in determining the soil Site Classification of the site for ground motion analysis purposes, a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods was performed in order to assess the one-dimensional average shear-wave velocity structure beneath the subject site to a depth of at least 100 feet.

This survey line was performed within the limits of the proposed building, as shown the Seismic Line Location Map, Plate 1, which provided the necessary survey line length that was unobstructed, as well as being representative for the site development. The resultant shear wave velocity ($V_S$) within the upper 100 feet (30 meters) was then used to determine the Site Classification (ASCE, 2017, Table 20.3-1) of the subject project study area for the seismic analysis. The detailed results of this survey, including the supportive data, are presented within Appendix A for reference.

Geologic mapping of the local region by Dibblee (2004), indicates that the subject development area is mantled by Holocene to late Pleistocene age older surficial sediments, comprised of alluvial fan deposits of the San Gorgonio Pass. These deposits are generally described being sand and gravel plutonic and gneissic detritus, that have been derived as outwash from the San Bernardino Mountains to the north. Progressively older and more consolidated alluvial deposits are presumed to underlie the subject property at depth.

The approximate location of the seismic shear-wave traverse (Seismic Line SW-1) is shown on a partial modified copy of the provided 100-scale Boring and Trench Location Plan (SCG, Plate 2), as presented on the Seismic Line Location Map, Plate 1. Photographic views of the seismic line traverse have been included within Appendix A for both visual and reference purposes.

## SITE-SPECIFIC GROUND MOTION ANALYSIS

As requested, we have performed a site-specific seismic ground motion analysis as discussed above. Geographically, the proposed building is centrally located at Latitude 33.9308 and Longitude -116.8548 (World Geodetic System of 1984). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps Tool web application (OSHPD, 2023) and the California Building Code criteria (CBC, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (ASCE, 2017).

The results of this site-specific ground motion analysis have been summarized and are tabulated below, with the detailed analysis being presented within Appendix B:

TABLE 1 – SUMMARY OF SEISMIC DESIGN PARAMETERS

| Factor or Coefficient | Value |
|---|---|
| **$S_S$** | **2.103g** |
| **$S_1$** | **0.847g** |
| **$F_a$** | **1.2** |
| **$F_v$** | **1.4** |
| **$S_{DS}$** | **1.530g** |
| **$S_{D1}$** | **0.830g** |
| **$S_{MS}$** | **2.290g** |
| **$S_{M1}$** | **1.243g** |
| **$T_L$** | **8 Seconds** |
| **$MCE_G$ PGA** | **0.96g** |
| **Shear-Wave Velocity ($V_{30}$)** | **1,891.7 ft/sec** |
| **Site Classification** | **C** |
| **Risk Category** | **II** |

## CLOSURE

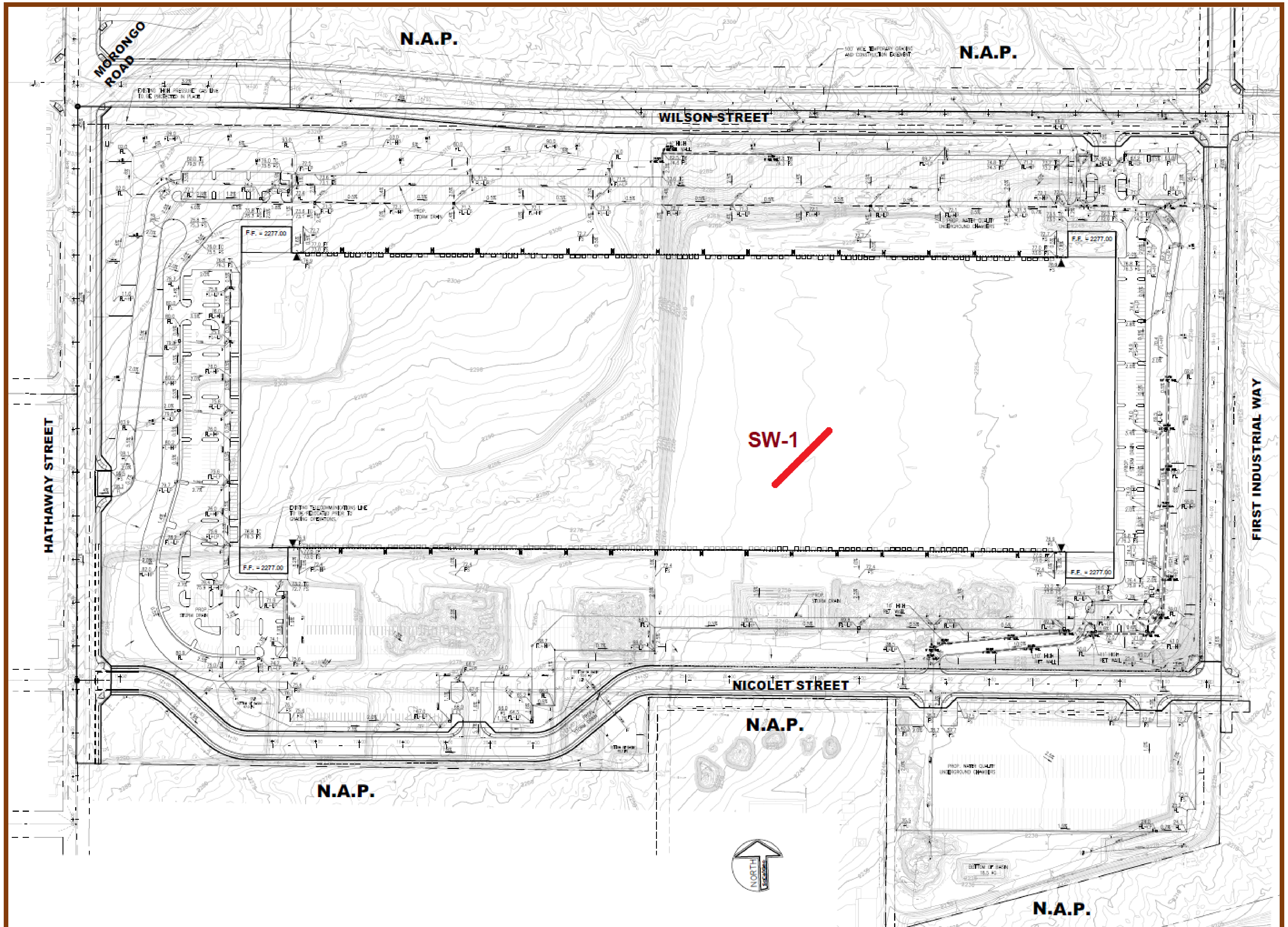
Our conclusions and recommendations are based on an interpretation of available existing geologic, geophysical, geotechnical, and seismic data. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. If this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
**TERRA GEOSCIENCES**

**Donn C. Schwartzkopf**
Certified Engineering Geologist
CEG 1459

Professional Geophysicist
PGP 1002

# SEISMIC LINE LOCATION MAP




Base Map: Partial modified copy of the provided Boring and Trench Location Plan (SCG, Plate 2); Seismic shear-wave survey line (SW-1) shown as red line.

# APPENDIX A

## SHEAR-WAVE SURVEY

# SHEAR-WAVE SURVEY

### *Methodology*

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity ($V_S$) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

## Field Procedures

One seismic shear-wave survey traverse was performed within the proposed commercial/industrial building, as approximated on the Seismic Line Location Map, Plate 1. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals.

For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple shots (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

## Data Processing

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2009). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution $V_s$ curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within this appendix.

## Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,891.7** feet per second, as shown on the Shear-Wave Model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "C" ("Very Dense Soil and Soft Rock" profile), which has a velocity range from 1,200 to 2,500 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface ($V_{100}$).

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

Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 238-foot profile where locally measured. The constrained data is represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

## Limitations

This survey was performed using "state of the art" geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. Compared with traditional borehole shear-wave surveys of which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. It is also important to understand that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

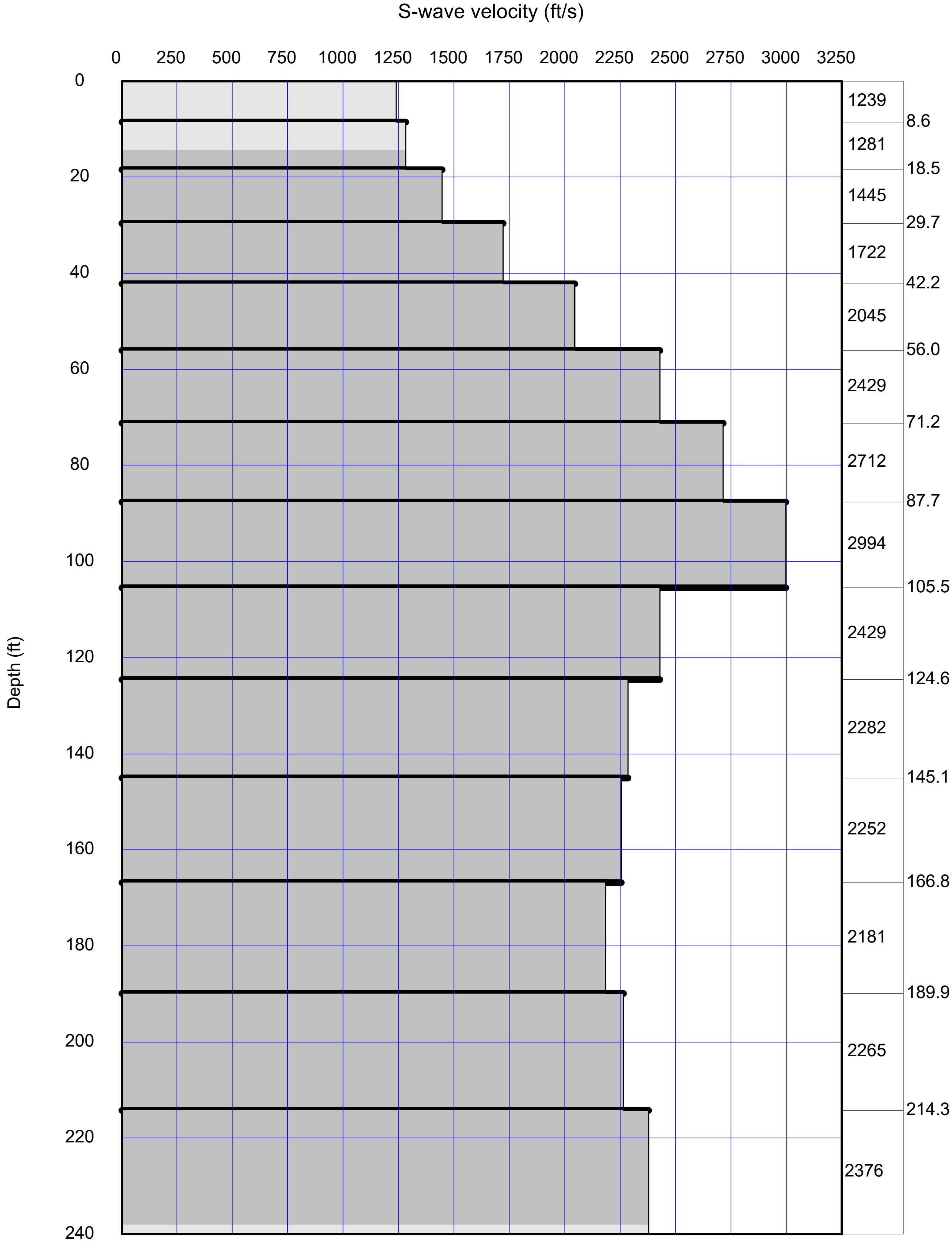
# SHEAR-WAVE SURVEY LINE PHOTOGRAPHS


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


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

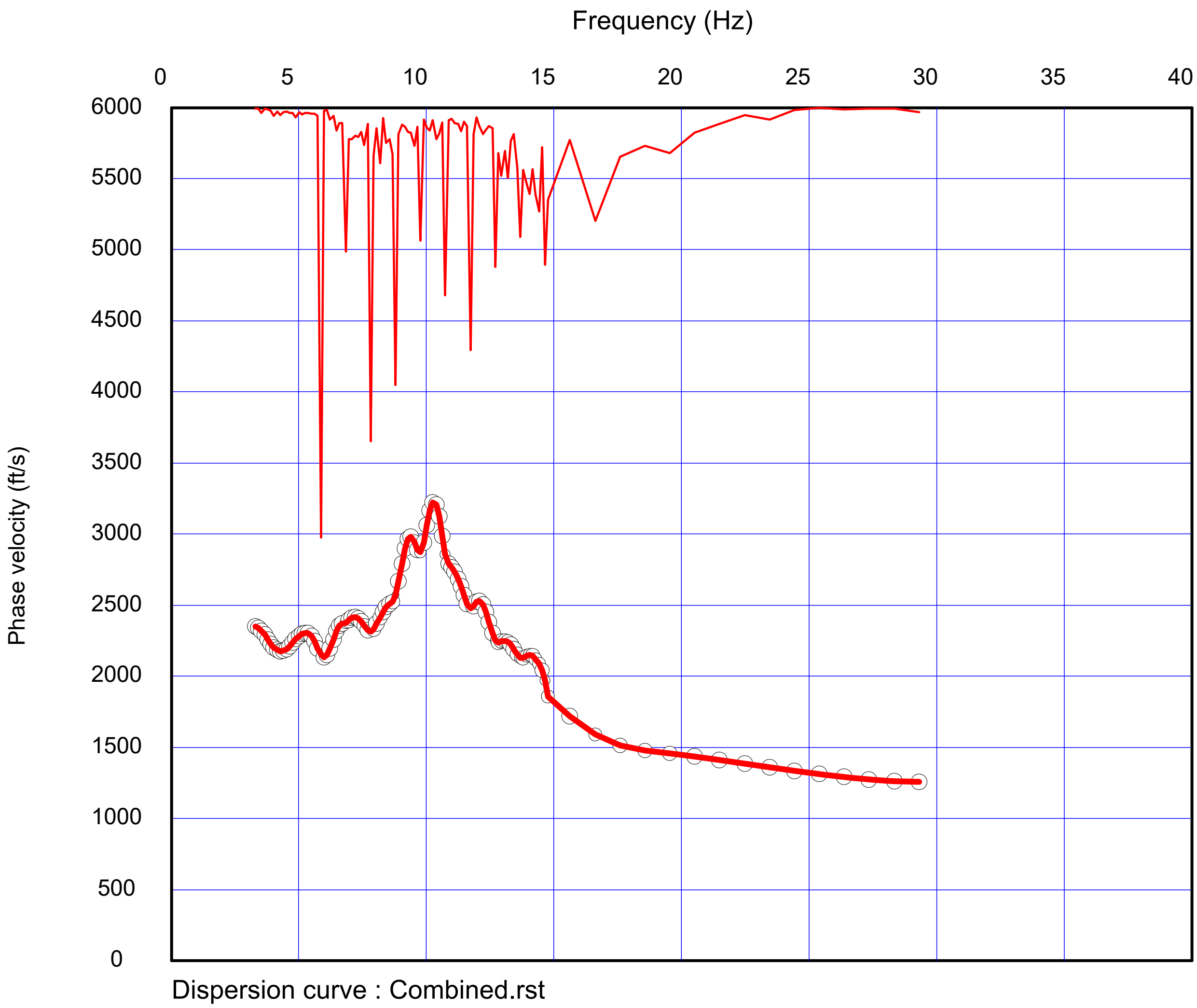
# SEISMIC LINE SW-1

## SHEAR-WAVE MODEL



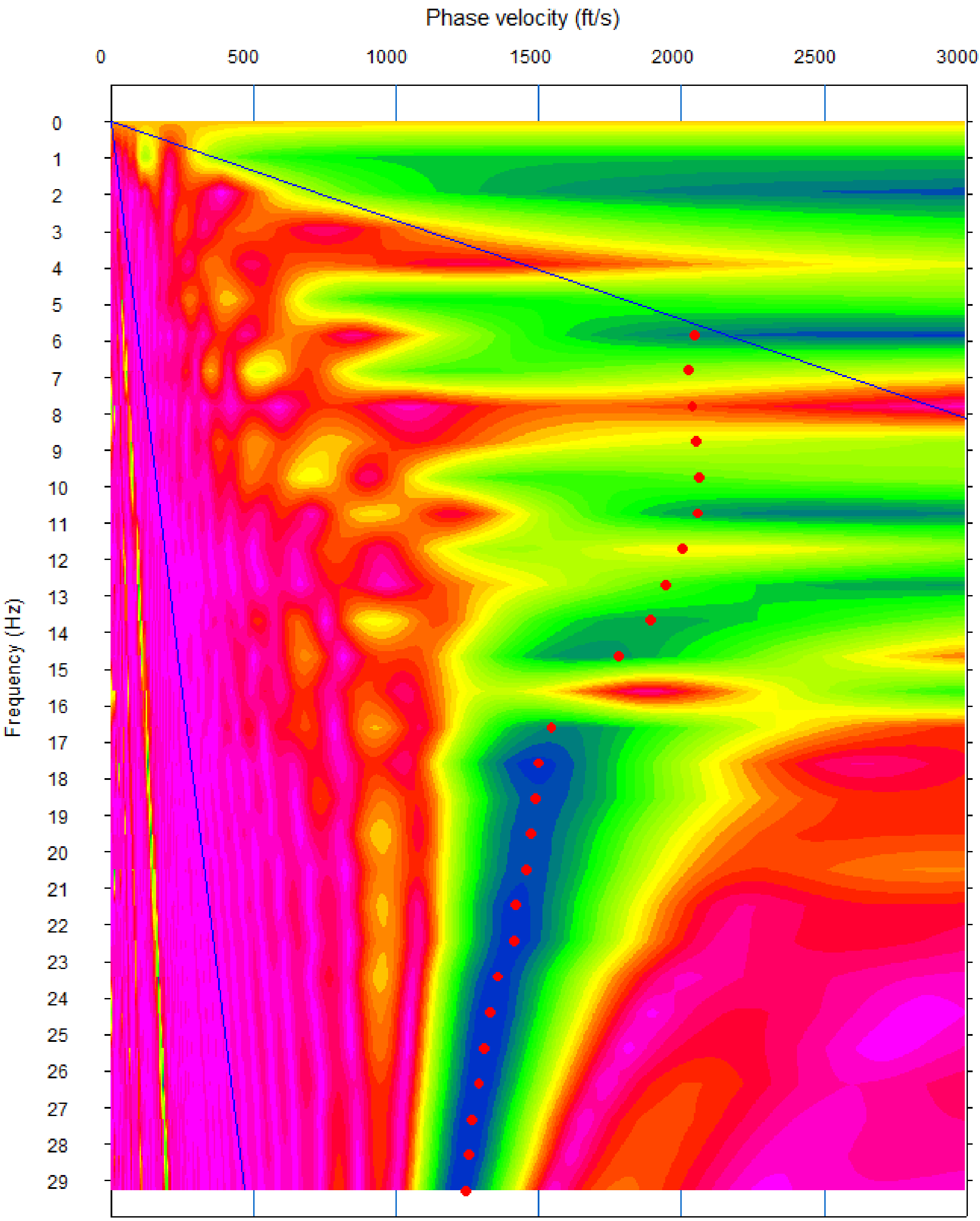

S-wave velocity model (inverted): Final.rst **Average Vs 100ft = 1891.7 ft/sec**

# SEISMIC LINE SW-1




# COMBINED DISPERSION CURVE

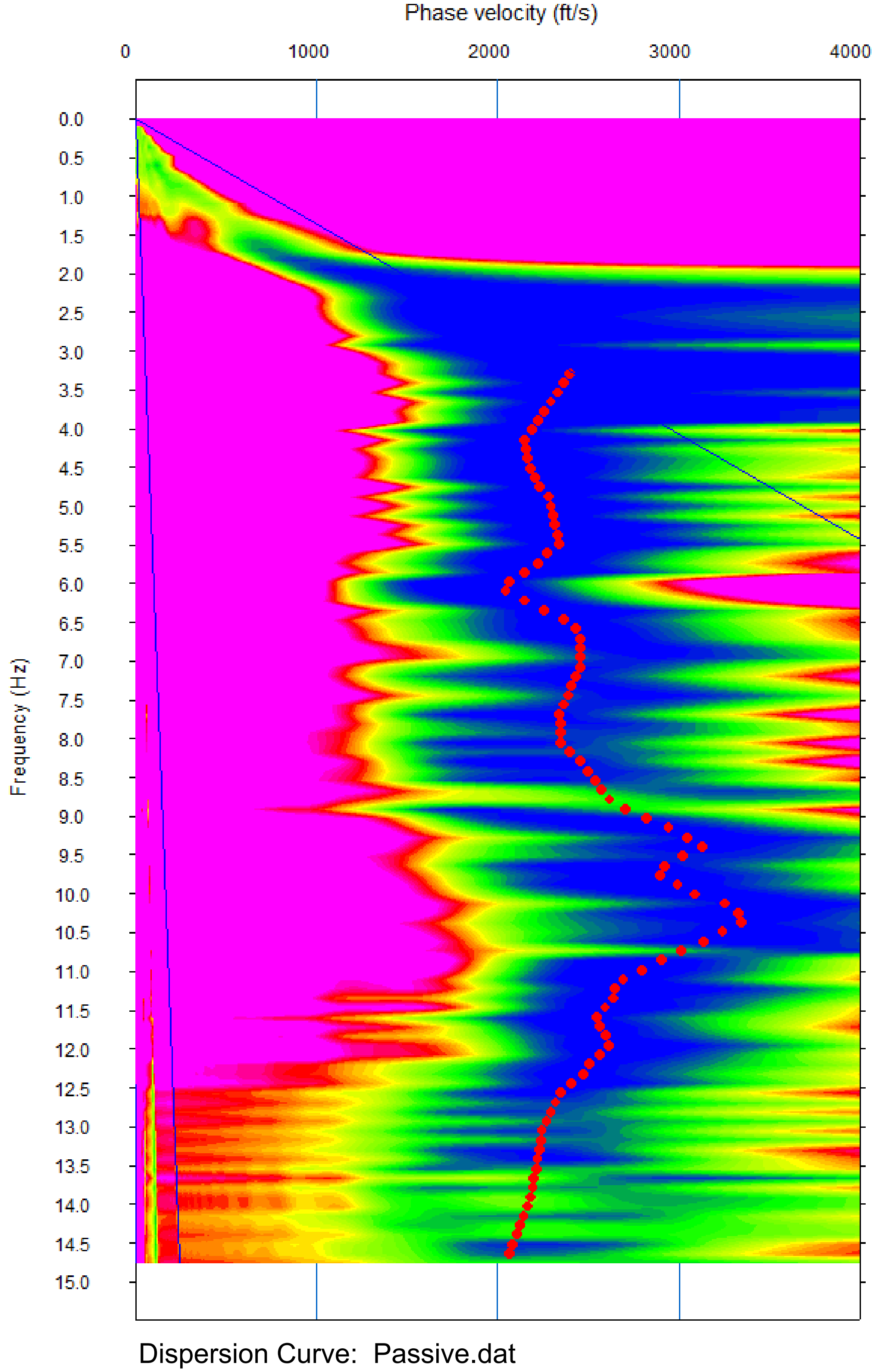
# SEISMIC LINE SW-1




Dispersion Cure:  Active.dat

# ACTIVE DISPERSION CURVE

# SEISMIC LINE SW-1




## PASSIVE DISPERSION CURVE

# APPENDIX B

## SITE-SPECIFIC GROUND MOTION ANALYSIS

# SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2022 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- ***Mapped Spectral Acceleration Parameters (CBC 1613.2.1)*-**

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake ($MCE_R$) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **2.103g** for the 0.2 second period ($S_s$) and **0.847g** for the 1.0 second period ($S_1$) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613.2.1).

- ***Site Classification (CBC 1613.2.2 & ASCE 7-16 Chapter 20)*-**

  Based on the site-specific measured shear-wave value of 1,891.7 feet/second, the soil profile type used should be Site Class **"C."** This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by "Very Dense Soil and Soft Rock" with average shear-wave velocities of 1,200 to 2,500 feet/second, as detailed within this appendix.

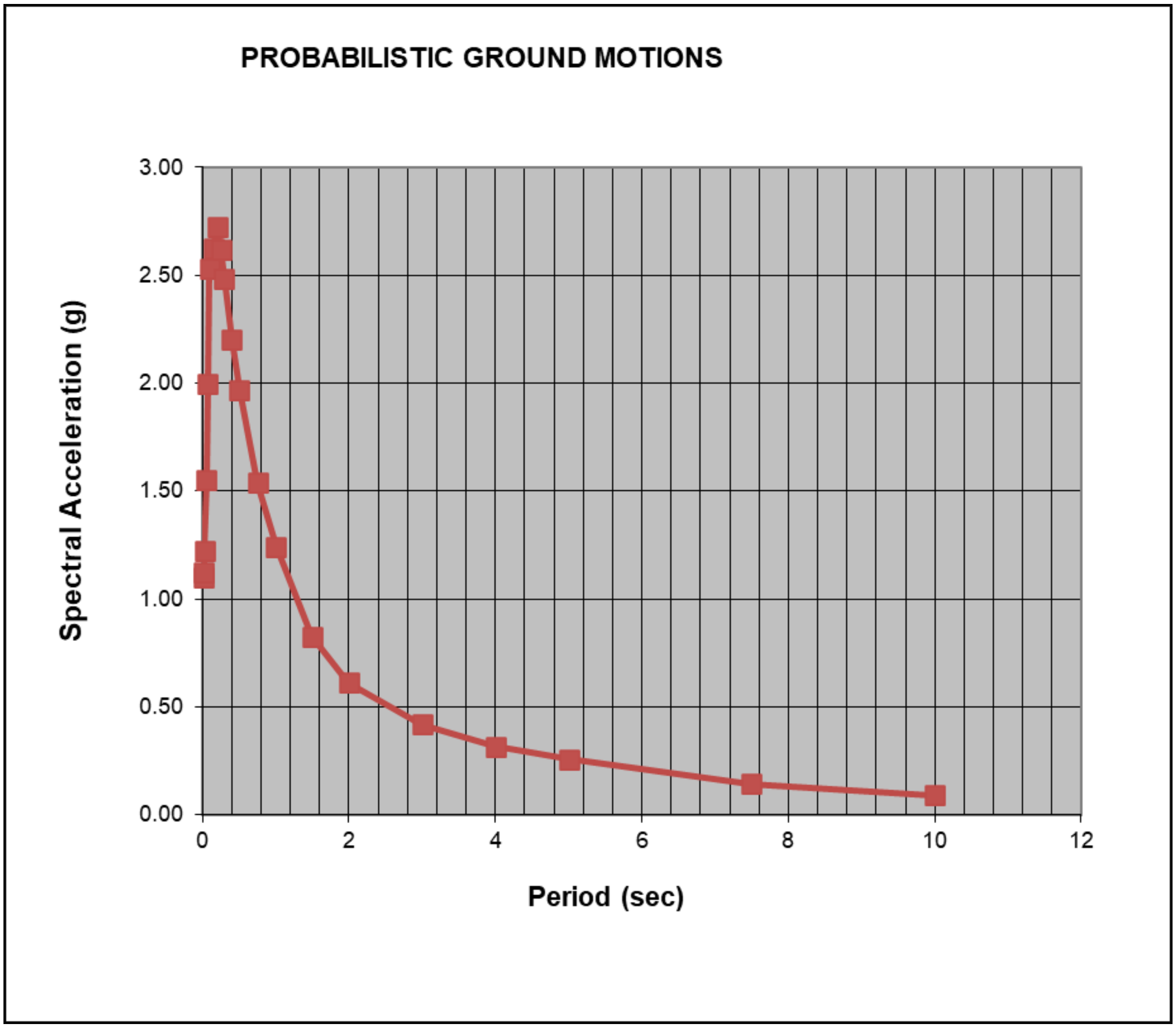
- ***Site Coefficients (CBC 1613.2.3)*-**

  Based on CBC Tables 1613.2.3(1) and 1613.2.3(2), the site coefficient $\mathbf{F_a = 1.2}$ and $\mathbf{F_v = 1.4}$, respectively.

- ***Probabilistic ($MCE_R$) Ground Motions (ASCE 7 Section 21.2.1.1)*-**

  Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of the Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 Years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), and Boore et al. (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient ($C_R$). These values were then modified to produce a spectrum based upon the maximum rotated components of ground motion. The resulting $MCE_R$ Response Spectrum is indicated below:




♦ <u>***Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)***</u>-

The deterministic $MCE_R$ response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted using the average of four Next Generation Attenuation West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abrahamsom et al. (2014), Campbell & Bozorgnia (2014), and Boore et al. (2014).

Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013) and the locations of the nearest and largest regional faults with respect to the subject site, the San Andreas Fault Zone ($M_W$ 8.1), the San Jacinto Fault Zone ($M_W$ 7.8), the Banning Fault ($M_W$ 7.5), and San Gorgonio Pass Fault ($M_W$ 7.4) were used for this analysis.

The analysis determined that the San Gorgonio Pass Fault (due to its proximity) controlled most of the design spectrum up to 5.0 seconds, with the larger and slightly more distant San Andreas Fault controlling the longer periods beyond 5.0 seconds.

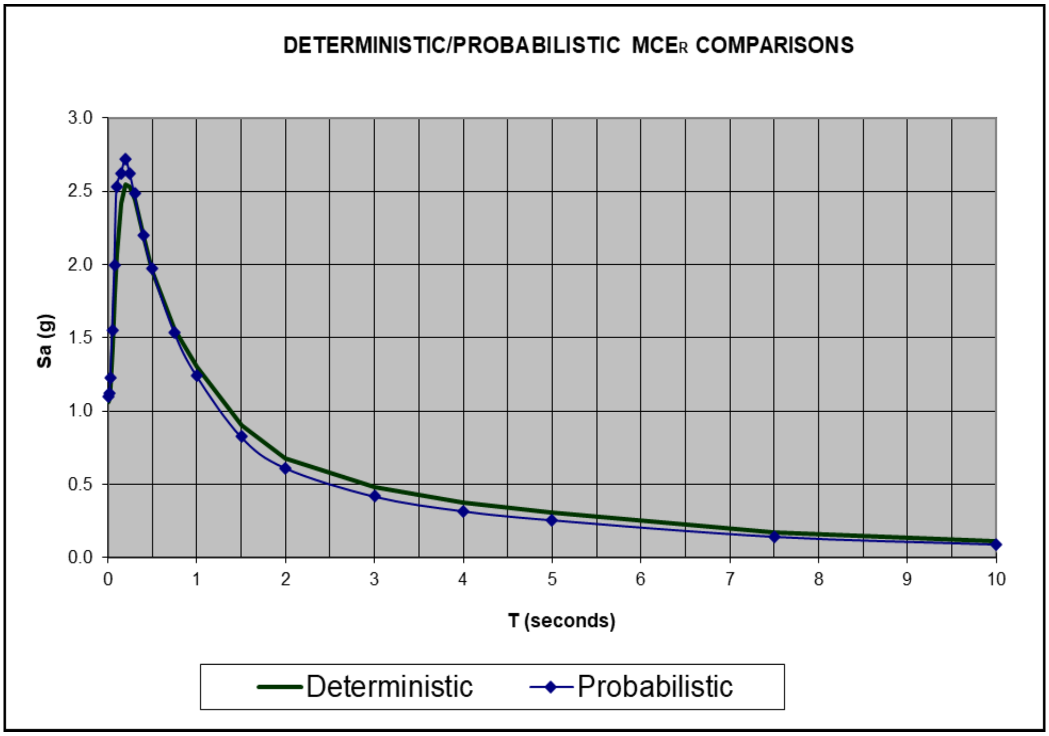
- **Site Specific MCE$_R$ (ASCE 7 Section 21.2.3)**-

The site-specific MCE$_R$ spectral response acceleration at any period, $S_{aM}$, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined in accordance with Section 21.2.1. These results are tabulated below:

***Comparison of Deterministic MCE$_R$ values (Sa) with Probabilistic MCE$_R$ Values (Sa) - Section 21.2.3***

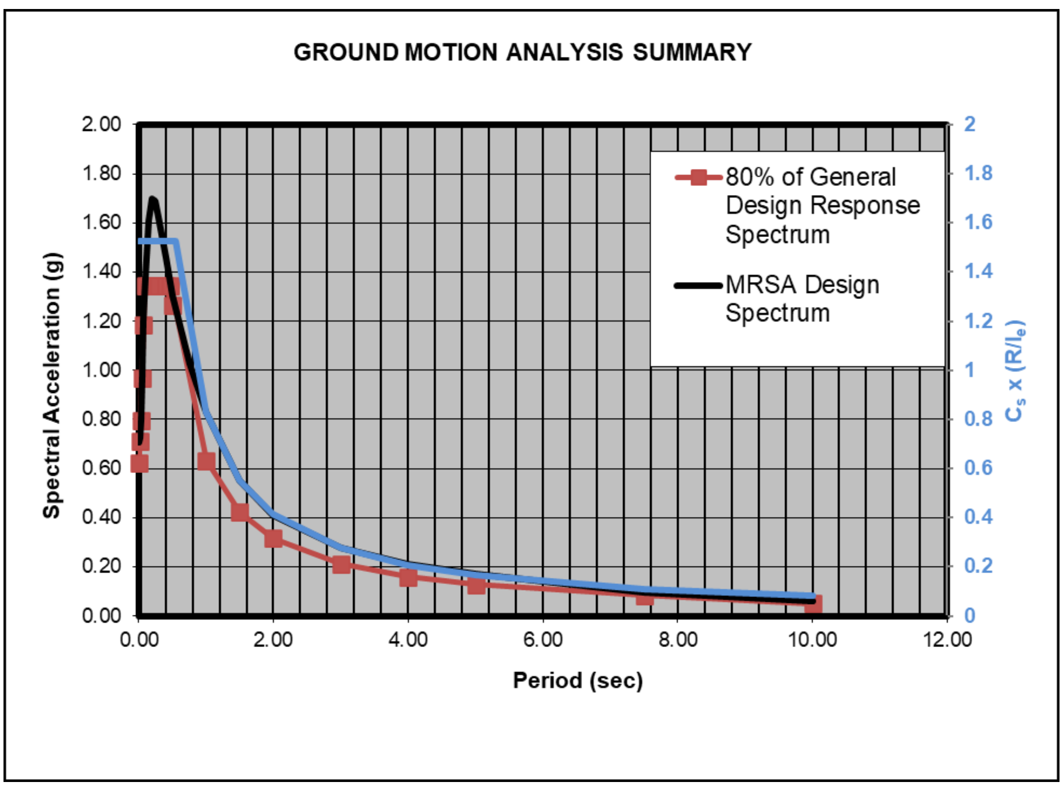
| Period T | Deterministic MCE$_R$ | Probabilistic MCE$_R$ | Lower Value (Site Specific MCE$_R$) | Governing Method |
|---|---|---|---|---|
| 0.010 | 1.06 | 1.10 | 1.06 | Deterministic Governs |
| 0.020 | 1.08 | 1.12 | 1.08 | Deterministic Governs |
| 0.030 | 1.17 | 1.23 | 1.17 | Deterministic Governs |
| 0.050 | 1.44 | 1.55 | 1.44 | Deterministic Governs |
| 0.075 | 1.80 | 1.99 | 1.80 | Deterministic Governs |
| 0.100 | 2.07 | 2.53 | 2.07 | Deterministic Governs |
| 0.150 | 2.42 | 2.62 | 2.42 | Deterministic Governs |
| 0.200 | 2.54 | 2.72 | 2.54 | Deterministic Governs |
| 0.250 | 2.53 | 2.62 | 2.53 | Deterministic Governs |
| 0.300 | 2.44 | 2.48 | 2.44 | Deterministic Governs |
| 0.400 | 2.19 | 2.20 | 2.19 | Deterministic Governs |
| 0.500 | 1.96 | 1.97 | 1.96 | Deterministic Governs |
| 0.750 | 1.56 | 1.54 | 1.54 | Probabilistic Governs |
| 1.000 | 1.30 | 1.24 | 1.24 | Probabilistic Governs |
| 1.500 | 0.90 | 0.83 | 0.83 | Probabilistic Governs |
| 2.000 | 0.68 | 0.61 | 0.61 | Probabilistic Governs |
| 3.000 | 0.48 | 0.42 | 0.42 | Probabilistic Governs |
| 4.000 | 0.38 | 0.32 | 0.32 | Probabilistic Governs |
| 5.000 | 0.31 | 0.26 | 0.26 | Probabilistic Governs |
| 7.500 | 0.17 | 0.14 | 0.14 | Probabilistic Governs |
| 10.000 | 0.11 | 0.09 | 0.09 | Probabilistic Governs |

These comparisons are plotted in the following diagram




- <u>***Design Response Spectrum (ASCE 7 Section 21.3)***</u>-

  In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, where $S_{aM}$ is the MCE$_R$ spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of $S_a$. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



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

  Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter $S_{DS}$ shall obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, $S_a$, at any period larger than 0.2 s. The parameter $S_{D1}$ shall be taken as the greater of the products of Sa * T for periods between 1 and 5 seconds. The parameters $S_{MS}$, and $S_{M1}$ shall be taken as 1.5 times $S_{DS}$ and $S_{D1}$, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.4 for $S_{MS}$, and $S_{M1}$ and Section 11.4.5 for $S_{DS}$ and $S_{D1}$.

- **Site Specific Design Parameters** -

  For the 0.2 second period ($S_{DS}$), a value of 1.530g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.70g occurring at T=0.20 seconds. This was multiplied by 0.9 to produce a value of 1.530g making this the applicable value. A value of 0.830g was calculated for $S_{D1}$ at a period of 1 second (ASCE 7-16, 21.4). For the $MCE_R$ 0.2 second period, a value of 2.290g ($S_{MS}$) was computed, along with a value of 1.243g ($S_{M1}$) for the $MCE_R$ 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

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

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 1.10g. The deterministic geometric mean peak ground acceleration (largest 84$^{th}$ percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.96g. The site-specific $MCE_G$ peak ground acceleration was calculated to be **0.96g**, which was determined by using the lesser of the probabilistic (1.10g) or the deterministic (0.96g) geometric mean peak ground accelerations, but not taken as less than 80 percent of $PGA_M$ (i.e., 1.05g x 0.80 = 0.84g).

## SEISMIC DESIGN PARAMETERS SUMMARY

Project: Banning Industrial Park
Project #: 233943-1
Date: 4/28/2023

Lattitude: 33.9308
Longitude: -116.8548

### CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

**Mapped Acceleration Parameters per ASCE 7-16, Chapter 22**

| | | |
|---|---|---|
| $S_s$= | 2.103 | Figure 22-1 |
| $S_1$= | 0.847 | Figure 22-2 |

**Site Class per Table 20.3-1**

| | |
|---|---|
| Site Class= | C - Very Dense Soil and Soft Rock |

**Site Coefficients per ASCE 7-16 CHAPTER 11**

| | | | | | |
|---|---|---|---|---|---|
| $F_a$= | 1.2 | Table 11.4-1 | = | 1.2 | For Site Specific Analysis per ASCE7-16 21.3 |
| $F_v$= | 1.4 | Table 11.4-2 | = | 1.40 | For Site Specific Analysis per ASCE7-16 21.3 |

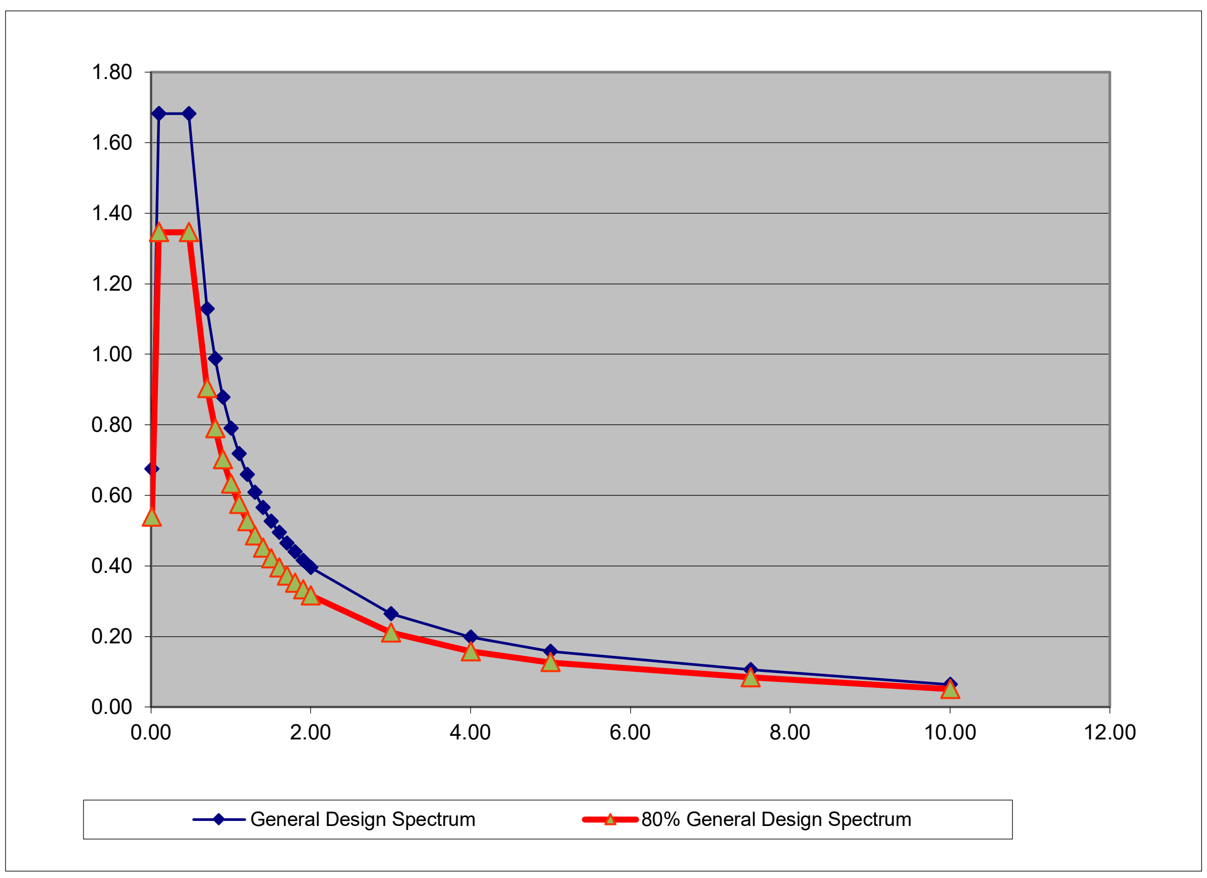
**Mapped Design Spectral Response Acceleration Parameters**

| | | | | |
|---|---|---|---|---|
| $S_{Ms}$= | 2.5236 | Equation 11.4-1 | 2.5236 | For Site Specific Analysis per ASCE7-16 21.3 |
| $S_{M1}$= | 1.186 | Equation 11.4-2 | 1.186 | For Site Specific Analysis per ASCE7-16 21.3 |

| | | | |
|---|---|---|---|
| $T_0$= | 0.094 | sec | |
| $T_S$= | 0.470 | sec | |
| $T_L$= | 8 | sec | From Fig 22-12 |
| PGA | 0.877 | g | |
| $F_{PGA}$= | 1.2 | | From Table 11.8-1 |
| $C_{RS}$= | 0.909 | | Figure 22-17 |
| $C_{R1}$= | 0.888 | | Figure 22-18 |

| | | |
|---|---|---|
| $S_{DS}$= | 1.682 | Equation 11.4-3 |
| $S_{D1}$= | 0.791 | Equation 11.4-4 |

| Period (T) | Sa (ASCE7-16 11.4.6) | 80% General Design Spectrum |
|---|---|---|
| 0.01 | 0.67 | 0.54 |
| 0.09 | 1.68 | 1.35 |
| 0.09 | 1.68 | 1.35 |
| 0.47 | 1.68 | 1.35 |
| 0.70 | 1.13 | 0.90 |
| 0.80 | 0.99 | 0.79 |
| 0.90 | 0.88 | 0.70 |
| 1.00 | 0.79 | 0.63 |
| 1.10 | 0.72 | 0.57 |
| 1.20 | 0.66 | 0.53 |
| 1.30 | 0.61 | 0.49 |
| 1.40 | 0.56 | 0.45 |
| 1.50 | 0.53 | 0.42 |
| 1.60 | 0.49 | 0.40 |
| 1.70 | 0.47 | 0.37 |
| 1.80 | 0.44 | 0.35 |
| 1.90 | 0.42 | 0.33 |
| 2.00 | 0.40 | 0.32 |
| 3.00 | 0.26 | 0.21 |
| 4.00 | 0.20 | 0.16 |
| 5.00 | 0.16 | 0.13 |
| 7.50 | 0.11 | 0.08 |
| 10.00 | 0.06 | 0.05 |



**ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS**

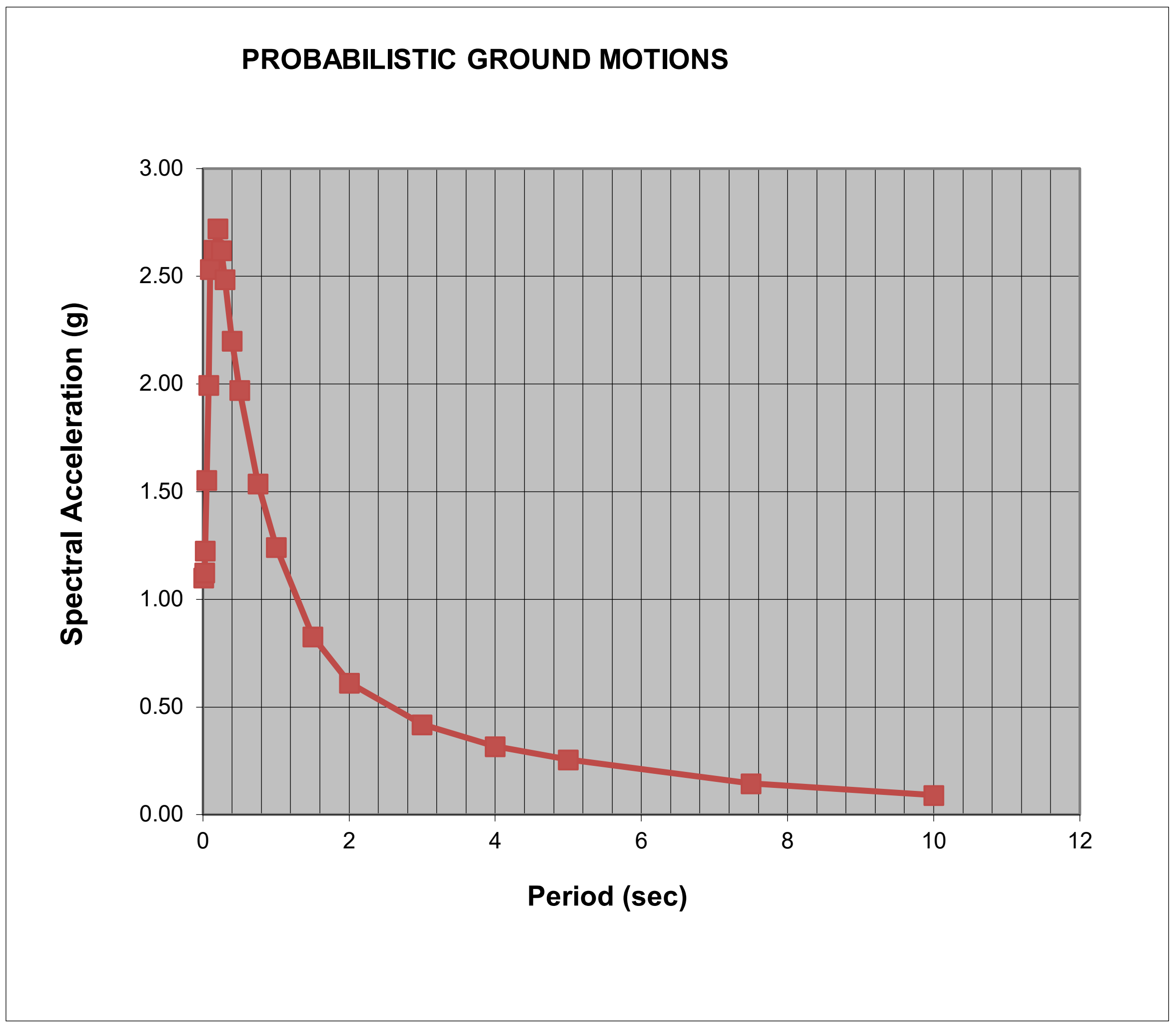
**Use Maximum Rotated Horizontal Component?* (Y/N)** Y

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
Earthquake Rupture Forecast - UCERF3 FM 3.1

**PROBABILISTIC MCER per 21.2.1.1 Method 1**
Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16
OpenSHA data
2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-16

| T | Sa 2% in 50 | MCER |
|---|---|---|
| 0.01 | 1.21 | 1.10 |
| 0.02 | 1.24 | 1.12 |
| 0.03 | 1.35 | 1.23 |
| 0.05 | 1.71 | 1.55 |
| 0.08 | 2.19 | 1.99 |
| 0.10 | 2.53 | 2.53 |
| 0.15 | 2.89 | 2.62 |
| 0.20 | 2.99 | 2.72 |
| 0.25 | 2.89 | 2.62 |
| 0.30 | 2.74 | 2.48 |
| 0.40 | 2.43 | 2.20 |
| 0.50 | 2.19 | 1.97 |
| 0.75 | 1.72 | 1.54 |
| 1.00 | 1.40 | 1.24 |
| 1.50 | 0.93 | 0.83 |
| 2.00 | 0.69 | 0.61 |
| 3.00 | 0.47 | 0.42 |
| 4.00 | 0.36 | 0.32 |
| 5.00 | 0.29 | 0.26 |
| 7.50 | 0.16 | 0.14 |
| 10.00 | 0.10 | 0.09 |

| | | |
|---|---|---|
| $S_s$= | 2.99 | 2.72 |
| $S_1$= | 1.40 | 1.24 |
| PGA | 1.10 g | |



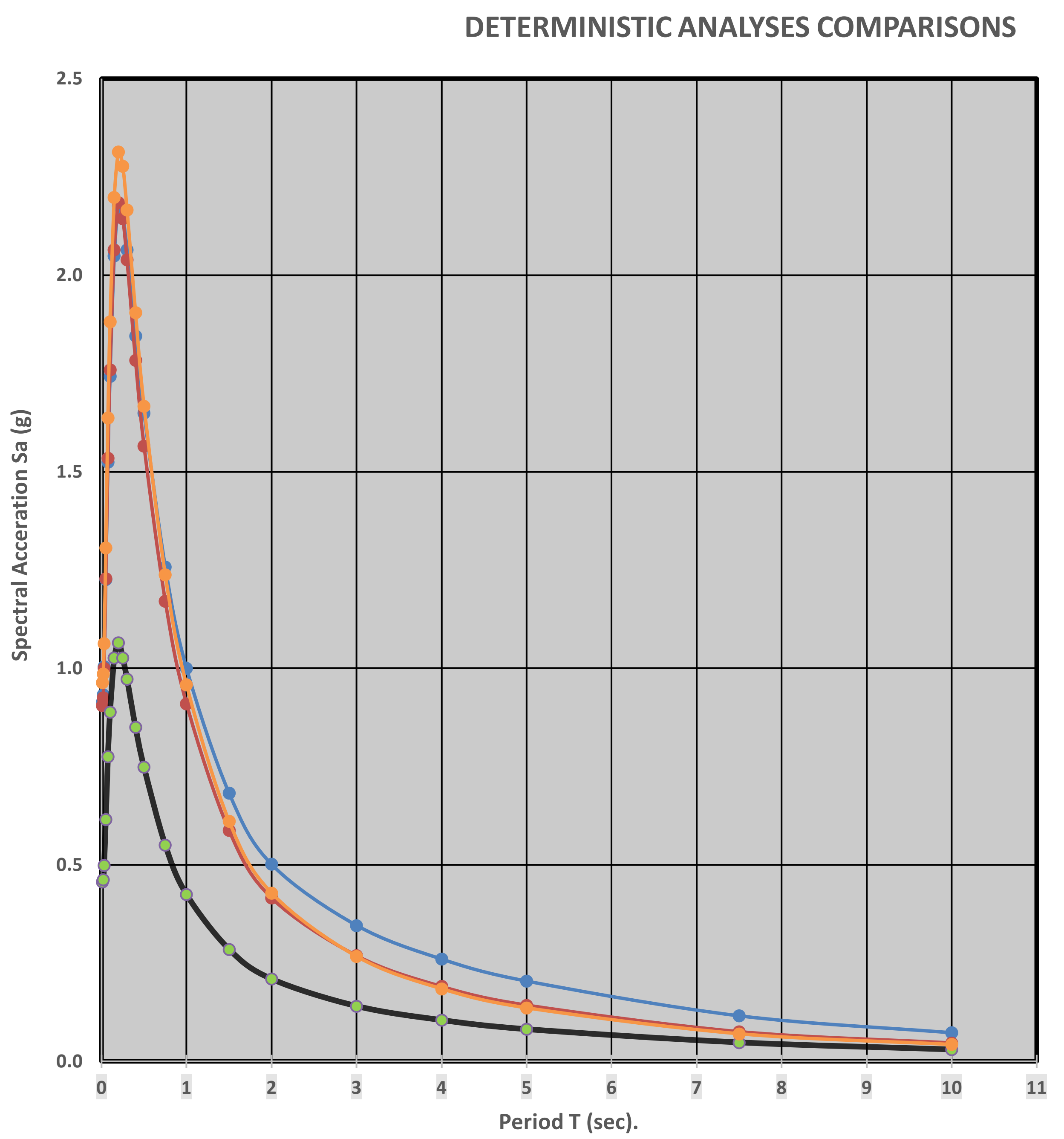

| Risk Coefficients: | | | |
|---|---|---|---|
| $C_{RS}$ | 0.909 | Figure 22-18 | Get from Mapped Values |
| $C_{R1}$ | 0.888 | Figure 22-19 | |
| Fa= | 1.2 | Table 11.4-1 | Per ASCE7-16 - 21.2.3 |
| Is $Sa_{(max)}$<1.2XFa? | | NO | If "YES", Probabilistic Spectrum prevails |

**DETERMINISTIC MCE per 21.2.2**

**Preliminary Assessment:**

| Fault | Distance (km) |
|---|---|
| San Andreas Fault | 4.20 |
| Banning | 3.20 |
| San Gorgonio Pass | 2.00 |
| San Jacinto | 18.30 |

The Probalistic Analyses revealed 3 faults contributing more than 10% to the seismic hazard. These were the San Andreas, San Gorgonio Pass and San Jacinto Faults and were included in the Deterministic Analyses.




**CONTROLING FAULT**:
San Gorgonio Pass
San Andreas

| Input Parameters | | San Andreas Fault | Banning | San Gorgonio Pass | San Jacinto |
|---|---|---|---|---|---|
| Fault | | | | | |
| M | = Moment magnitude | 8.1 | 7.5 | 7.4 | 7.8 |
| $R_{RUP}$ | = Closest distance to coseismic rupture (km) | 4.2 | 3.2 | 2 | 18.3 |
| $R_{JB}$ | = Closest distance to surface projection of coseismic rupture (km) | 4.2 | 3.2 | 2 | 18.3 |
| Rx | = Horizontal distance to top edge of rupture measured perpendicular to strike (km) | 4.2 | 3.2 | 2 | 18.3 |
| U | = Unspecified Faulting Flag (Boore et.al.) | 0 | 0 | 0 | 0 |
| $F_{RV}$ | = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust | 0 | 0 | 1 | 0 |
| $F_{NM}$ | = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique | 0 | 0 | 0 | 0 |
| $F_{HW}$ | = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08 | 0 | 0 | 0 | 0 |
| $Z_{TOR}$ | = Depth to top of coseismic rupture (km) | 0 | 0 | 0 | 0 |
| $\delta$ | = Average dip of rupture plane (degrees) | 90 | 70 | 24 | 90 |
| $V_{S30}$ | = Average shear-wave velocity in top 30m of site profile | 576.6 | 576.6 | 576.6 | 576.6 |
| $F_{Measured}$ | | 1 | 1 | 1 | 1 |
| $Z_{1.0}$ | = Depth to Shear Wave Velocity of 1.0 km/sec (km) | 0.07 | 0.07 | 0.07 | 0.07 |
| $Z_{2.5}$ | = Depth to Shear Wave Velocity of 2.5 km/sec (km) | 1.65 | 1.65 | 1.65 | 1.65 |
| Site Class | | C | C | C | C |
| W (km) | = Fault rupture width (km) | 12.5 | 17 | 22 | 12.4 |
| $F_{AS}$ | = 0 for mainshock; 1 for aftershock | 0 | 0 | 0 | 0 |
| $\sigma$ | =Standard Deviation | 1 | 1 | 1 | 1 |

**Deterministic Summary - Section 21.2.2 (Supplement 1)**

| T | San Andreas Fault | Banning | San Gorgonio Pass | San Jacinto | Maximum $S_{a\ (Average)}$ | Corrected* $S_a$ (per ASCE7-16) | Scaled $S_{a(Average)}$ | Controlling Fault |
|---|---|---|---|---|---|---|---|---|
| 0.010 | 0.91 | 0.91 | 0.96 | 0.46 | 0.96 | 1.06 | 1.06 | San Gorgonio Pass |
| 0.020 | 0.93 | 0.93 | 0.99 | 0.46 | 0.99 | 1.08 | 1.08 | San Gorgonio Pass |
| 0.030 | 1.00 | 1.00 | 1.06 | 0.50 | 1.06 | 1.17 | 1.17 | San Gorgonio Pass |
| 0.050 | 1.23 | 1.23 | 1.31 | 0.62 | 1.31 | 1.44 | 1.44 | San Gorgonio Pass |
| 0.075 | 1.53 | 1.53 | 1.64 | 0.77 | 1.64 | 1.80 | 1.80 | San Gorgonio Pass |
| 0.100 | 1.74 | 1.76 | 1.88 | 0.89 | 1.88 | 2.07 | 2.07 | San Gorgonio Pass |
| 0.150 | 2.05 | 2.06 | 2.20 | 1.03 | 2.20 | 2.42 | 2.42 | San Gorgonio Pass |
| 0.200 | 2.18 | 2.18 | 2.31 | 1.06 | 2.31 | 2.54 | 2.54 | San Gorgonio Pass |
| 0.250 | 2.15 | 2.14 | 2.28 | 1.03 | 2.28 | 2.53 | 2.53 | San Gorgonio Pass |
| 0.300 | 2.07 | 2.04 | 2.17 | 0.97 | 2.17 | 2.44 | 2.44 | San Gorgonio Pass |
| 0.400 | 1.85 | 1.78 | 1.90 | 0.85 | 1.90 | 2.19 | 2.19 | San Gorgonio Pass |
| 0.500 | 1.65 | 1.57 | 1.67 | 0.75 | 1.67 | 1.96 | 1.96 | San Gorgonio Pass |
| 0.750 | 1.26 | 1.17 | 1.24 | 0.55 | 1.26 | 1.56 | 1.56 | San Andreas Fault |
| 1.000 | 1.00 | 0.91 | 0.96 | 0.42 | 1.00 | 1.30 | 1.30 | San Andreas Fault |
| 1.500 | 0.68 | 0.59 | 0.61 | 0.29 | 0.68 | 0.90 | 0.90 | San Andreas Fault |
| 2.000 | 0.50 | 0.42 | 0.43 | 0.21 | 0.50 | 0.68 | 0.68 | San Andreas Fault |
| 3.000 | 0.35 | 0.27 | 0.27 | 0.14 | 0.35 | 0.48 | 0.48 | San Andreas Fault |
| 4.000 | 0.26 | 0.19 | 0.18 | 0.10 | 0.26 | 0.38 | 0.38 | San Andreas Fault |
| 5.000 | 0.20 | 0.14 | 0.14 | 0.08 | 0.20 | 0.31 | 0.31 | San Andreas Fault |
| 7.500 | 0.12 | 0.08 | 0.07 | 0.05 | 0.12 | 0.17 | 0.17 | San Andreas Fault |
| 10.000 | 0.07 | 0.05 | 0.04 | 0.03 | 0.07 | 0.11 | 0.11 | San Andreas Fault |
| PGA | 0.91 | 0.90 | 0.96 | 0.45 | 0.96 | | 0.96 | g |

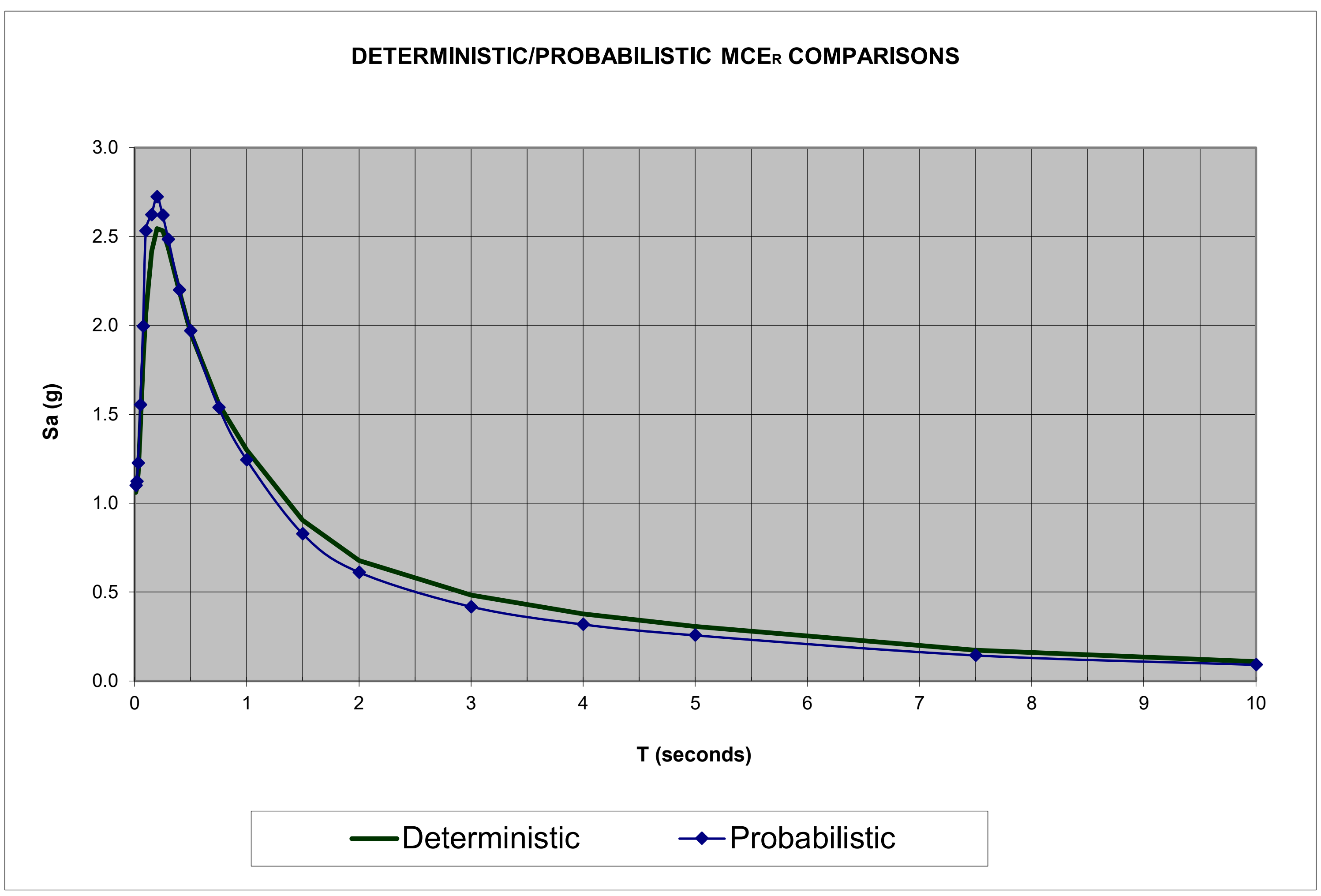
| | | |
|---|---|---|
| Max Sa= | 2.54 | |
| Fa = | 1.20 | Per ASCE7-16 21.2.2 |
| 1.5XFa= | 1.8 | |
| Scaling Factor= | 1.00 | |

* 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 | Deterministic | Probabilistic | Lower Value (Site Specific $MCE_R$) | Governing Method |
|---|---|---|---|---|
| T | $MCE_R$ | $MCE_R$ | | |
| 0.010 | 1.06 | 1.10 | 1.06 | Deterministic Governs |
| 0.020 | 1.08 | 1.12 | 1.08 | Deterministic Governs |
| 0.030 | 1.17 | 1.23 | 1.17 | Deterministic Governs |
| 0.050 | 1.44 | 1.55 | 1.44 | Deterministic Governs |
| 0.075 | 1.80 | 1.99 | 1.80 | Deterministic Governs |
| 0.100 | 2.07 | 2.53 | 2.07 | Deterministic Governs |
| 0.150 | 2.42 | 2.62 | 2.42 | Deterministic Governs |
| 0.200 | 2.54 | 2.72 | 2.54 | Deterministic Governs |
| 0.250 | 2.53 | 2.62 | 2.53 | Deterministic Governs |
| 0.300 | 2.44 | 2.48 | 2.44 | Deterministic Governs |
| 0.400 | 2.19 | 2.20 | 2.19 | Deterministic Governs |
| 0.500 | 1.96 | 1.97 | 1.96 | Deterministic Governs |
| 0.750 | 1.56 | 1.54 | 1.54 | ProbabilisticGoverns |
| 1.000 | 1.30 | 1.24 | 1.24 | ProbabilisticGoverns |
| 1.500 | 0.90 | 0.83 | 0.83 | ProbabilisticGoverns |
| 2.000 | 0.68 | 0.61 | 0.61 | ProbabilisticGoverns |
| 3.000 | 0.48 | 0.42 | 0.42 | ProbabilisticGoverns |
| 4.000 | 0.38 | 0.32 | 0.32 | ProbabilisticGoverns |
| 5.000 | 0.31 | 0.26 | 0.26 | ProbabilisticGoverns |
| 7.500 | 0.17 | 0.14 | 0.14 | ProbabilisticGoverns |
| 10.000 | 0.11 | 0.09 | 0.09 | ProbabilisticGoverns |



**DESIGN RESPONSE SPECTRUM per Section 21.3**

**DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)**

| Period | 2/3*MCE$_R$ | 80% General Design Response Spectrum (per ASCE 7-16 23.3-1) | Design Response Spectrum | TXSa |
|---|---|---|---|---|
| 0.01 | 0.71 | 0.62 | 0.71 | |
| 0.02 | 0.72 | 0.71 | 0.72 | |
| 0.03 | 0.78 | 0.80 | 0.80 | |
| 0.05 | 0.96 | 0.97 | 0.97 | |
| 0.08 | 1.20 | 1.19 | 1.20 | |
| 0.10 | 1.38 | 1.35 | 1.38 | |
| 0.15 | 1.61 | 1.35 | 1.61 | |
| 0.20 | 1.70 | 1.35 | 1.70 | |
| 0.25 | 1.69 | 1.35 | 1.69 | |
| 0.30 | 1.62 | 1.35 | 1.62 | |
| 0.40 | 1.46 | 1.35 | 1.46 | |
| 0.50 | 1.31 | 1.26 | 1.31 | |
| 0.75 | 1.03 | 0.84 | 1.03 | |
| 1.00 | 0.83 | 0.63 | 0.83 | 0.83 |
| 1.50 | 0.55 | 0.42 | 0.55 | 0.83 |
| 2.00 | 0.41 | 0.32 | 0.41 | 0.82 |
| 3.00 | 0.28 | 0.21 | 0.28 | |
| 4.00 | 0.21 | 0.16 | 0.21 | |
| 5.00 | 0.17 | 0.13 | 0.17 | |
| 7.50 | 0.10 | 0.08 | 0.10 | |
| 10.00 | 0.06 | 0.05 | 0.06 | |

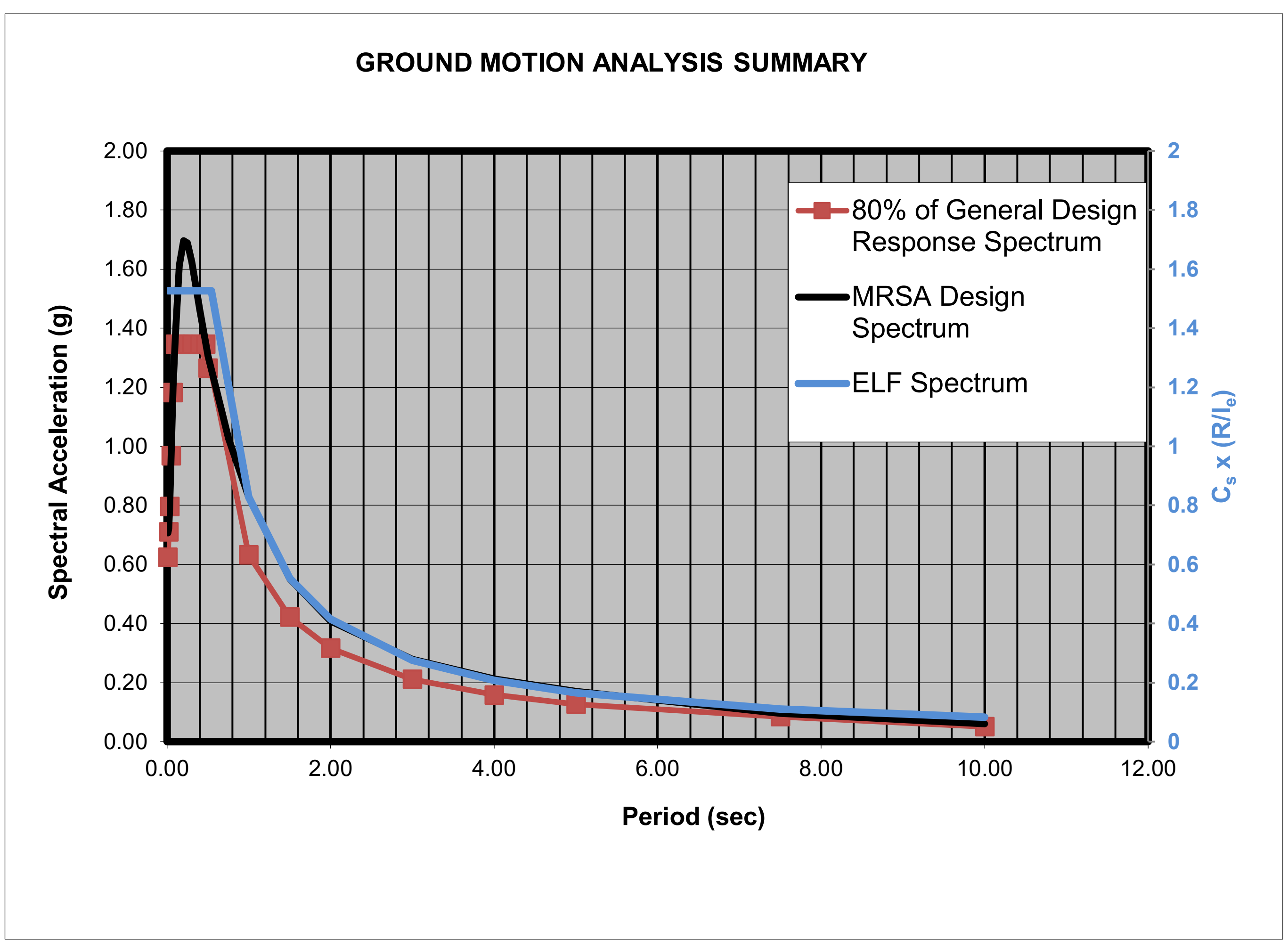
| | |
|---|---|
| Highest value of $S_a$ for any period exceeding 0.2 sec.= | 1.70 |
| 90%of Highest Value = | 1.53 |
| 80% 0f Mapped $S_{DS}$= | 1.35 |
| Max TXsa from T=1s-2s = | 0.83 |
| 80% of Mapped $S_{D1}$= | 0.63 |

| | |
|---|---|
| $S_{DS}$= | 1.53 |
| $S_{D1}$= | 0.83 |
| Ts = | 0.54 |

| | |
|---|---|
| $S_{MS}$= | 2.290 |
| $S_{M1}$= | 1.243 |

**PGA Determination:**

| | | |
|---|---|---|
| Site Coefficient $F_{PGA}$= | 1.2 | |
| Mapped PGA= | 0.88 | Figure 22-7 |
| $PGA_M$ = | 1.05 | g |
| Deterministic PGA = | 0.96 | g |
| Probabilistic PGA = | 1.10 | g |
| Lesser of Deterministic/Probabilistic = | 0.96 | g |
| 80% of $PGA_{M=}$ | 0.84 | g |
| $MCE_G$ PGA= | 0.96 | g |



## SUMMARY OF SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS DATA

| 1 | 2 | 3 | | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Period (sec)** | Mapped $MCE_R$ Spectrum | Mapped Design Spectrum | **Period (sec)** | Risk Coefficient $C_R$ | Scaled $MCE_R$ Deterministic Spectrum | Probabilistic $MCE_R$ Spectrum | Probabilistic w/Risk Coeffcicent $C_R$ | 84th Percentile Deterministic Spectrum | 2/3 Site Specific $MCE_R$ Spectrum | 80% of General Design Spectrum | Site Specific $MCE_R$ Spectrum | Design Response Spectrum |
| **0.01** | 1.01 | 0.67 | **0.01** | 0.909 | 1.06 | 1.10 | 1.10 | 1.06 | 0.71 | 0.62 | 1.06 | 0.71 |
| **0.09** | 2.52 | 1.68 | **0.02** | 0.909 | 1.08 | 1.12 | 1.12 | 1.08 | 0.72 | 0.71 | 1.08 | 0.72 |
| **0.09** | 2.52 | 1.68 | **0.03** | 0.909 | 1.17 | 1.23 | 1.23 | 1.17 | 0.78 | 0.80 | 1.20 | 0.80 |
| **0.47** | 2.52 | 1.68 | **0.05** | 0.909 | 1.44 | 1.55 | 1.55 | 1.44 | 0.96 | 0.97 | 1.46 | 0.97 |
| **0.70** | 1.69 | 1.13 | **0.08** | 0.909 | 1.80 | 1.99 | 1.99 | 1.80 | 1.20 | 1.19 | 1.80 | 1.20 |
| **0.80** | 1.48 | 0.99 | **0.10** | 0.909 | 2.07 | 2.53 | 2.53 | 2.07 | 1.38 | 1.35 | 2.07 | 1.38 |
| **0.90** | 1.32 | 0.88 | **0.15** | 0.909 | 2.42 | 2.62 | 2.62 | 2.42 | 1.61 | 1.35 | 2.42 | 1.61 |
| **1.00** | 1.19 | 0.79 | **0.20** | 0.909 | 2.54 | 2.72 | 2.72 | 2.54 | 1.70 | 1.35 | 2.54 | 1.70 |
| **1.10** | 1.08 | 0.72 | **0.25** | 0.908 | 2.53 | 2.62 | 2.62 | 2.53 | 1.69 | 1.35 | 2.53 | 1.69 |
| **1.20** | 0.99 | 0.66 | **0.30** | 0.906 | 2.44 | 2.48 | 2.48 | 2.44 | 1.62 | 1.35 | 2.44 | 1.62 |
| **1.30** | 0.91 | 0.61 | **0.40** | 0.904 | 2.19 | 2.20 | 2.20 | 2.19 | 1.46 | 1.35 | 2.19 | 1.46 |
| **1.40** | 0.85 | 0.56 | **0.50** | 0.901 | 1.96 | 1.97 | 1.97 | 1.96 | 1.31 | 1.26 | 1.96 | 1.31 |
| **1.50** | 0.79 | 0.53 | **0.75** | 0.895 | 1.56 | 1.54 | 1.54 | 1.56 | 1.03 | 0.84 | 1.54 | 1.03 |
| **1.60** | 0.74 | 0.49 | **1.00** | 0.888 | 1.30 | 1.24 | 1.24 | 1.30 | 0.83 | 0.63 | 1.24 | 0.83 |
| **1.70** | 0.70 | 0.47 | **1.50** | 0.888 | 0.90 | 0.83 | 0.83 | 0.90 | 0.55 | 0.42 | 0.83 | 0.55 |
| **1.80** | 0.66 | 0.44 | **2.00** | 0.888 | 0.68 | 0.61 | 0.61 | 0.68 | 0.41 | 0.32 | 0.61 | 0.41 |
| **1.90** | 0.62 | 0.42 | **3.00** | 0.888 | 0.48 | 0.42 | 0.42 | 0.48 | 0.28 | 0.21 | 0.42 | 0.28 |
| **2.00** | 0.59 | 0.40 | **4.00** | 0.888 | 0.38 | 0.32 | 0.32 | 0.38 | 0.21 | 0.16 | 0.32 | 0.21 |
| **3.00** | 0.40 | 0.26 | **5.00** | 0.888 | 0.31 | 0.26 | 0.26 | 0.31 | 0.17 | 0.13 | 0.26 | 0.17 |
| **4.00** | 0.30 | 0.20 | **7.50** | 0.888 | 0.17 | 0.14 | 0.14 | 0.17 | 0.10 | 0.08 | 0.14 | 0.10 |
| **5.00** | 0.24 | 0.16 | **10.00** | 0.888 | 0.11 | 0.09 | 0.09 | 0.11 | 0.06 | 0.05 | 0.09 | 0.06 |
| **7.50** | 0.16 | 0.11 | | | | | | | | | | |
| **10.00** | 0.09 | 0.06 | | | | | | | | | | |

# APPENDIX C

## REFERENCES

# REFERENCES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