**SOILS SOUTHWEST, INC.**

SOILS, MATERIALS AND ENVIRONMENTAL ENGINEERING CONSULTANTS

897 VIA LATA, SUITE N • COLTON, CA 92324 • (909) 370-0474 • (909) 370-0481 • FAX (909) 370-3156


# Report of
# Soils and Foundation Evaluations & Soil Infiltration Testing for WQMP-BMP Design

Planned Office-Warehouse Complex
1050 S. 6th Street at Fogg Street
City of Colton
San Bernardino County, California
APN: 0163-281-31

Project No. 19059-F/BMP
January 10, 2020

Prepared for:

Mr. Karapetian Vahe Trust
% U.S. Auction
130 E. 9th Street
Upland, California 91784


soilssouthwest@aol.com
Established 1984

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January 10, 2020 — Project No. 19059-F/BMP

Mr. Karapetian Vahe Trust
% U.S. Auction
130 E. 9th Street
Upland, California 91784

Attention: Mr. Armando Camarena

Subject: Report of Soils and Foundation Evaluations and
Soil Infiltration Testing for WQMP-BMP Design
Planned Office-Warehouse Complex
1050 S. 6th Street at Fogg Street, City of Colton, California

Reference: Site Plan prepared by Jonathan Zane Architecture

Gentlemen:

Presented herewith is the Feasibility Studies Report of Soils and Foundation Evaluations and Soil Infiltration Testing for WQMP-BMP design for the planned office- warehouse complex to be constructed at 1050 S. 6th Street, near the northeast corner of 6th and Fogg Street, City of Colton, California. In absence of grading and project plans the recommendations supplied should be considered "preliminary", subject to revisions following development plan review.

Based on the test explorations completed, it is our opinion that the soils encountered primarily consist of upper dry, loose, compressible silty fine to medium coarse sands, overlying variegating layers of silt and sand along with layers of gravely medium coarse to coarse sands to the maximum depth of 51 feet explored. No shallow depth groundwater was encountered. Based on review of the available documents published by the local governing agency, CGS and others, it is our opinion the site in not located within and A-P Special Study Zone, where active earthquake fault is known to pass through or towards the site.

Compressible e in nature, the near grade soils encountered are considered unsuitable for directly supporting structural loadings without excessive settlements to load bearing footings and concrete slab-on-grade. When, however, graded in form of sub-excavations of the near grade soils and their replacement as engineered fills as described herein, the local soils thus used, should be suitable for structural support for the development planned.

This report has been substantiated by subsurface explorations and mathematical analysis made in accordance with the generally accepted engineering principles, including those field and laboratory testing considered necessary in the circumstances. We offer no other warranty, either express or implied.

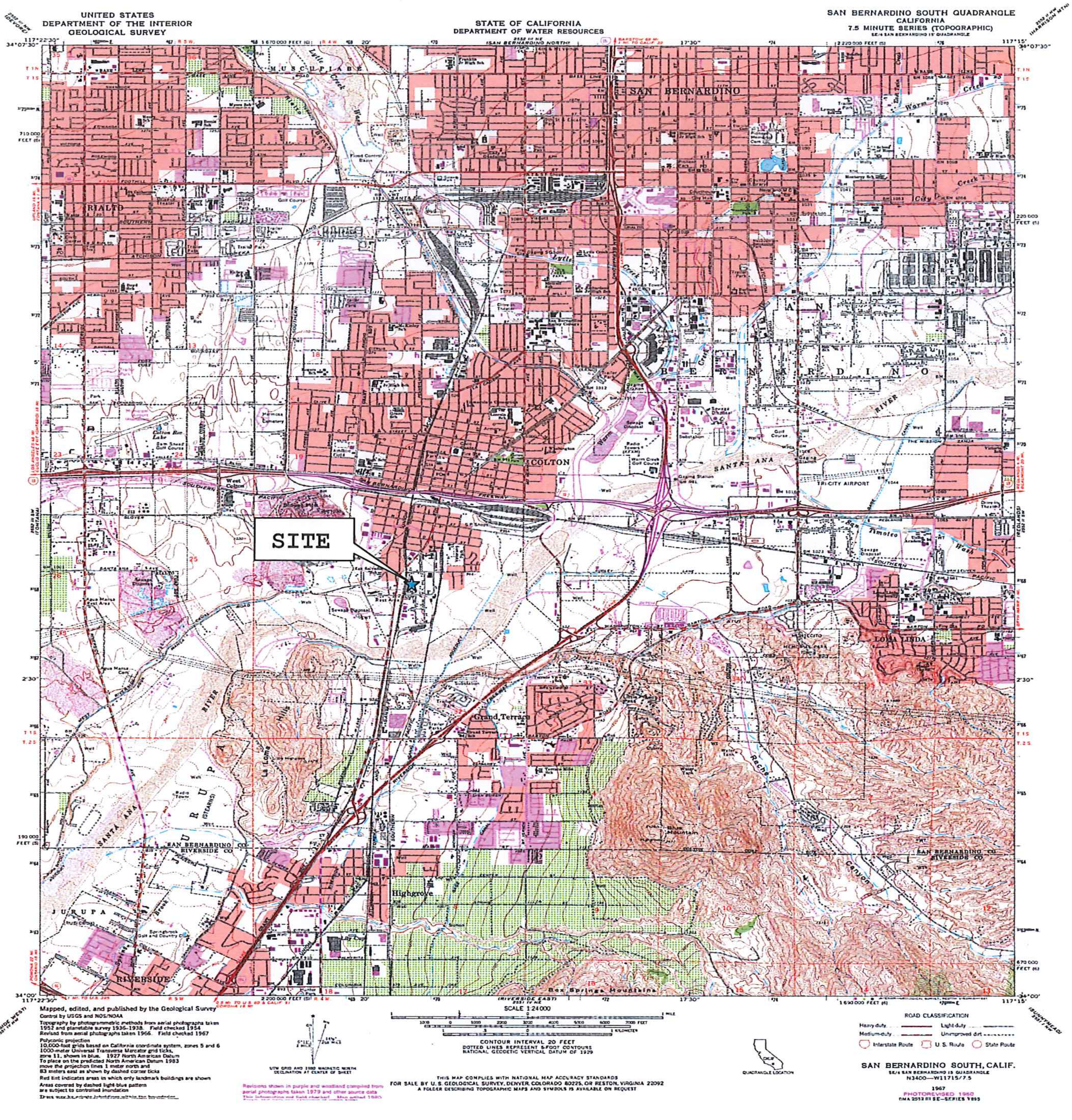
Respectfully submitted,
Soils Southwest, Inc.

Moloy Gupta, RCE 31708

REGISTERED PROFESSIONAL ENGINEER
MOLOY K. GUPTA
No. 31708
Exp. 12-31-20
CIVIL
STATE OF CALIFORNIA

John Flippin

## Site Vicinity Map



## 1.0 Introduction

Presented herewith is the Report of Soils and Foundation Evaluations and the Soils Infiltration Testing for WQMP-BMP Design for the planned office warehouse development to be located at 1050 S. 6th Street, near the northeast intersection of 6th and Fogg Street, City of Colton, , California

The purpose of this evaluation is to determine the nature and engineering properties of the near grade and subsurface soils, and to provide geotechnical recommendations for foundation design, concrete slab-on-grade, retaining walls (if proposed), paving, parking, site grading, utility trench excavations and backfills, and inspection during construction.

The recommendations contained reflect our best estimate of the soils conditions as encountered during field investigations conducted. It is not to be considered as a warranty of the soils for other areas, or for the depths beyond the explorations advanced at this time.

*The recommendations supplied should be considered valid and applicable when the following conditions, are fulfilled:*

*i. Pre-grade meeting with contractor, public agency and soils engineer,*
*ii. Excavated bottom inspections and verifications by soils engineer prior to backfill placement,*
*iii. Continuous observations and testing during site preparation and structural fill soils placement,*
*iv. Observation and inspection of footing trenching prior to steel and concrete placement,*
*v. Plumbing trench backfill placement prior to concrete slab-on-grade placement,*
*vi. On and off-site utility trench backfill testing and verifications, and*
*vii Consultations as required during construction, or upon your request.*

### 1.1 Proposed Development

Based on the preliminary project information supplied it is understood that the subject development, among others, will include a commercial/industrial facility to accommodate one two-story office and a detached warehouse of conventional metal and/or concrete tilt-up construction with load bearing wall and isolated spread footings and concrete slab-on-grade. Installation of on-site WQMP-BMP infiltration basin, paving, parking, driveways and offsite improvements are anticipated to complete the project. Moderate site preparations and grading should be expected with the development planned. For preliminary analyses, structural loadings of 30 kip and 3 klf are assumed for isolated spread and load bearing wall foundations, respectively.

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### 1.2 Site Description

The irregularly-shaped parcel is currently vacant and undeveloped. In general, the site is bounded by a mixture of industrial and residential property on the north, east, and southeast, Fogg Street on the south, South Sixth Street on the west. Based on the provided site topographic map for review, overall vertical relief within the property is about 15 feet with onsite surface water tending to flow to the south/southeast. Apart from seasonal weeds and grasses, scattered shrub trees along with round concrete slabs at north and northwest, no other significant features were noted.

## 2.0 Scope of Work

Geotechnical evaluations included subsurface explorations, soil sampling, necessary laboratory testing, engineering analyses and the preparation of this report. The scope of work included the following:

- **Field Explorations**

  Field investigations included four (4) exploratory soil test borings using a Hollow-Stem Auger (HSA) drill-rig equipped for undisturbed soils sampling and Standard Penetration Testing (SPT), along with two (2) borings for WQMP-BMP infiltration testing. Approximate test excavation locations are shown on Plate 1.

  During excavations, the sub-soils encountered were continuously logged, bulk and undisturbed samples were procured and Standard Penetration Test (SPT) blow-counts were recorded at frequent intervals. Collected samples were subsequently transferred to our laboratory for necessary geotechnical evaluations. Description of the soils encountered is shown on the Log of Boring in Appendix A.

- **Laboratory Testing**

  Representative samples on selected bulk and undisturbed site soils were tested in our laboratory to aid in the soils classification and to evaluate relevant engineering properties pertaining to the project requirements. Laboratory testing included the following:

- In-situ moisture contents and dry density (ASTM Standard D2216),
- Maximum Dry Density-Optimum Moisture content (ASTM Standard D1557),
- Direct Shear (ASTM Standard D3080),
- Soil Consolidation (ASTM Standard D2435),
- Soil Sand Equivalent, SE (ASTM D2419),
- Soil Grain size analysis (ASTM D422), and
- Expansion Index Potential (ASTM D4829)

General descriptions of the test procedures and test results are provided in Appendix B.

- Based on the field investigation and laboratory testing, engineering analyses and evaluations were made on which to base our preliminary recommendations for design of foundations, slab-on-grade, paving and parking, site grading, utility trench backfill, site preparations and grading, and monitoring during construction.

- Preparation of this report for initial use by the project design professionals.

The recommendations supplied should be considered as 'tentative' and may require revision and/or upgrading following review of the final grading and development plans, when supplied.

# 3.0 Site Conditions

## 3.1 Subsurface Conditions

Based on the test borings completed for the locations as described, it is our opinion that the site soils, in general, primarily consist of upper dry, loose, compressible silty fine to medium sands, overlying variegating layers of silt and sand along with silty fine sands with isolated layers of gravely medium coarse sands or sands containing some clays to the maximum depth of 51 feet explored. No shallow depth groundwater was encountered.

The upper compressible soils existing as described should be considered inadequate for directly supporting structural loading without excessive total and differential settlements to footings and concrete slab-on-grade. When, however, graded in form of subexcavations of the upper soils and their replacement as engineered fills compacted to 90%, the structural pad thus constructed should be adequate for the proposed development with tolerable settlements.

Based on review of the available documents published by the local governing agency, CGS and others, it is our opinion the site in not located within and A-P Special Study Zone, where active earthquake fault is known to pass through, or towards the project site area.

Laboratory shear tests conducted on the upper bulk soil sample remolded to 90 percent indicate moderate shear strength under increased moisture conditions. Results of the laboratory shear tests are provided in Plate B-1 of this report.

Consolidation tests conducted on the upper undisturbed soils indicate moderate potential for compressibility under anticipated structural loading. Results of the laboratory determined soils consolidation potential is shown on Plate B-2 in Appendix B.

### 3.1.1 Compressible and Collapsible Soils

Based on laboratory testing completed on the samples procured, it is our opinion that the upper 4 to 5 feet of the near surface soils existing should be considered moderately compressible, and may be susceptible to excessive settlements under conventional structural loadings. Accordingly, for adequate load bearing, it is opinion that the upper 5 feet of soils should be subexcavated, followed by their replacement as engineered fills compacted to minimum 90 percent.

In general, the subexcavations described should encompass, in minimum, the planned building areas and five (5) feet beyond. Where restricted due to exiting development, the lateral extent beyond footing described may be compensated by using deepened foundations.

The detailed subexcavation requirements are described in section 4.1.1 of this report. It is recommended that subexcavation depths should be verified and approved by soils engineer prior to new structural fill soils placement. Local soils free of organic and rocks larger than 6" in overall diameter should be considered suitable for re-use as structural fill.

### 3.1.2 Expansive Soils

Considering the sandy silty and slightly gravely soils as encountered, the site soils are considered to have a "very low" expansion potential with an Expansion Index, EI, less than 20. However, additional testing should be conducted during grading.

## 3.2 Excavatability

It is our opinion that the grading required for the project may be accomplished using conventional heavy-duty construction equipment. No bedrock or impenetrable subsurface layers should be encountered requiring blasting or jack-hammering.

## 3.3 Groundwater

No groundwater was encountered within the maximum 51 feet depth explored and none such is anticipated within the maximum excavations that will be required during grading and construction. No special construction requirements including de-watering, etc., are expected, however, it is our opinion that provisions should be made so as to dispose of surface runoff away from structural pad once constructed.

Fluctuations in groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, altered natural drainage paths, and other factors not evident at the time the borings were advanced. Consequently, the designer and contractor should be aware of this possibility while designing and constructing the building(s).

The following table lists the nearest well to the site as listed by the reporting agency.

| GROUNDWATER TABLE | |
|---|---|
| Reporting Agency | Water Master Support Services-San Bernardino Valley Conservation District/Western Municipal Water District Cooperative Well Measuring Program, Fall 2016 |
| Well Number | 01S/04W-29K001S /Fogg2 |
| Well Monitoring Agency | U.S. Geological Survey/SBV MWD |
| Well Location: Township/Range/Section | T1S-R4W-Section 29 |
| Well Elevation: | 925 |
| Current Depth to Water (Measured in feet) | 101.09 |
| Current Date Water was Measured | October 10, 2018 |
| Depth to Water (Measured in feet) (Shallowest) | 73.48 |
| Date Water was Measured (Shallowest) | April 26, 2007 |

## 3.4 Soil Corrosivity Analyses

Considering the expected change in soils chemical characteristics during mass grading, no soil chemical analysis is currently initiated. It is our opinion that following mass grading completion the representative soils should be laboratory tested to determine pH, sulfate, chloride and resistivity. When equated, results of such will be supplied immediately prior to concrete placement.

## 3.5 Faulting and Seismicity

### 3.5.1 Direct or Primary Seismic Hazards

Surface ground rupture along with active fault zones and ground shaking represent primary or direct seismic hazards to structures. In absence of known earthquake faults passing through or towards the site, the subject site should be considered not situated within an AP Special Studies Zone.

According to the current CBC, the site is considered to be within Seismic Zone 4, as a result, it is likely that during the life expectancy of the proposed construction, "moderate" ground shaking may have adverse effects on the proposed development.

### 3.5.2 Induced or Secondary Seismic Hazards

In addition to ground shaking, effects of seismic activity may include surface fault rupture, soil liquefaction, and differential settlement, ground lurching, landslides, lateral spreading, and earthquake induced flooding. Opinions regarding site specific secondary effects are explained as follows.

#### 3.5.2.1 Surface Fault Rupture

Based on review of the CGS available publications, it is understood that no major fault crosses through or extends towards the site. The potential for surface rupture resulting from nearby fault movement is not known for certainty, but is considered "low" considering to the distance of the site to the known nearest San Jacinto:SBV+SJV+A fault as situated at about 2.29 miles away as described.

#### 3.5.2.2 Flooding

Flooding hazards include tsunamis (seismic sea waves), seiches, and failure of manmade reservoirs, tanks and aqueducts. The potential for these hazards is considered remote due to the inland site location, and distance to any known nearby bodies of water.

#### 3.5.2.3 Land-Sliding

Considering that the site soils primarily consisting of silty sands with high SPT blow-counts, the potential for seismically induced landslide should be considered remote .

#### 3.5.2.4 Lateral Spreading

Seismically induced lateral spreading involves lateral movement of existing soils due to ground shaking. Lateral spreading is demonstrated by near vertical cracks with predominantly horizontal movement of the soil mass involved. In absence of obvious presence of such, it is our opinion that the potential for lateral spreading should be considered remote.

#### 3.5.2.5 Settlement and Subsidence

Considering the presence of upper loose and compressible soils as encountered and described, it is our opinion that potential/possibility for ground settlement due to strong motion seismically induced ground shaking should be considered.

### 3.6 Seismically Induced Settlement and Subsidence

The site is situated at about 2.29 miles from the San Jacinto: SBV+SJV+A fault capable of generating an earthquake magnitude M=7.63 and Peak Ground Acceleration, PGA of 0.603g. Considering the proximity of the earthquake fault as described, along with the presence of very loose sandy subgrades conditions, no seismically induced excessive ground total and differential settlements should be expected in excess of 1" and ½", respectively.

### 3.7 Seismic Design Coefficients

Using site coordinates of 34.053354.°N, and -117.328566°W, proximity of the closest San Jacinto:SBV+SJV+A fault is estimated to about 2.29 miles away. For foundation and structural design, the following seismic parameters are suggested based on the current CBC.

Recommended values are based upon the USGS ASCE 7-16 Hazard Reports Parameters and the California Geologic Survey: PSHA Ground Motion Interpolator Supplemental seismic parameters are provided in Appendix C of this report. The following presents the seismic design parameters as based on the available publications as currently published by the California Geological Survey and 2019 CBC. In design, vertical acceleration may be assumed to about 1/3 to 2/3 of the estimated horizontal ground acceleration (PGA) as described in the following sections

**TABLE 3.7.1 Seismic Design Parameters**

| CBC Chapter 16 | 2016 ASCE 7-1****0 Standard Seismic Design Parameters | Recommended Values |
|---|---|---|
| 1613A.5.2 | Site Class | D |
| 1613.5.1 | The mapped spectral accelerations at short period | $S_s$ |
| 1613.5.1 | The mapped spectral accelerations at 1.0-second period | $S_1$ |
| 1613A5.3(1) | Site Class B / Seismic Coefficient, $S_s$ | 2.059 g |
| 1613A5.3(2) | Site Class B / Seismic Coefficient, $S_1$ | 0.817 g |
| 1613A5.3(1) | Site Class C / Seismic Coefficient, $F_a$ | 1.000 g |
| 1613A5.3(2) | Site Class C / Seismic Coefficient, $F_v$ | n/a |
| 16A-37 Equation | Spectral Response Accelerations, $S_{Ms} = F_a S_s$ | 2.059 g |
| 16A-38 Equation | Spectral Response Accelerations, $S_{M1} = F_v S_1$ | n/a |
| 16A-39 Equation | Design Spectral Response Accelerations, $S_{Ds} = 2/3 \times S_{Ms}$ | 1.373 g |
| | | |

| | | |
|---|---|---|
| 16A-40 Equation | Design Spectral Response Accelerations, $S_{D1} = 2/3 \text{ x } S_{M1}$ | n/a |

## TABLE 3.7.2 Seismic Source Type

Based on California Geological Survey-Probabilistic Seismic Hazard Assessment Peak Horizontal Ground Acceleration (PHGA) having a 10 percent probability of exceedance in a 50 year period is described as below:

| Seismic Source Type / Appendix C | |
|---|---|
| Nearest Maximum Fault Magnitude | M>\=7.63 |
| Peak Horizontal Ground Acceleration | 0.603g |

# 4.0 Evaluations and Recommendations

## 4.1 General Evaluations

The conclusions contained herein are based upon surface and subsurface explorations at the test locations described. Although no significant variations in soil conditions are anticipated, actual soils conditions may vary in the event subgrades exposed during construction are found different from those as described in this report. It will be the subcontractor's responsibility to notify Soils Southwest about subsoil variations, if any, for revised/updated recommendations.

While caving was not encountered, it is possible that a trench, exploratory boring, or excavation would react in an entirely different manner. All shoring and bracing, if required, shall be in accordance with the current requirements of the State of California Division of Industrial Safety and other public agencies having jurisdiction.

Based on field explorations, laboratory testing and subsequent engineering analysis, the following conclusions and recommendations are presented for the site under study:

(i) Moderate site clearance should be expected, including, but not be limited to loose fills, roots, stumps, buried irrigation systems, and others.

(ii) From geotechnical viewpoint, the site is considered grossly stable for the proposed development. Minor rocks may be encountered during grading and utility installation.

(iii) Because of the near surface compressible soils existing as described, conventional grading should be in form of subexcavations, scarification and moisturization, followed by their replacement as engineered fills compacted to 90% or better. In event new fill soils are required over the grades currently existing, such should be placed following subgrade preparations as described in this report. No footings and/or new fills should be placed directly bearing on the compressible surface soils existing.

(iv) The sub-excavation depths described should be considered as 'minimum'. During grading localized deeper sub-excavations may be warranted within areas of buried debris, dumped loose fills, irrigation pipes etc. It will be the responsibility of the grading contractor to inform soils engineer the presence of such, if and when exposed.

(v) In order to minimize potential excessive differential settlements, it is recommended that structural footings should be established exclusively into engineered fills of local sandy soils or its equivalent or better, compacted to minimum 90% of the soils Maximum Dry Density at near Optimum Moisture conditions. Construction of footings and slabs straddling over cut/fill transition should be avoided.

(vi) Structural design considerations should include probability for "moderate to high" peak ground acceleration from relatively active nearby earthquake faults. The effects of ground shaking, however, can be minimized by implementating the seismic design requirements and the procedures as outlined in the current CBC/UBC and as described in earlier Section of this report.

(vii) Provisions should be maintained during construction to divert incidental rainfall away from the structural pad constructed.

(viii) It is our opinion that, if site preparations and grading are performed as per the generally accepted construction practices, the proposed development will not adversely affect the stability of the site, or the properties adjacent.

### 4.1.1 Preparations for Structural Pad

In absence of precise grading plan, the planned pad grade is expected to be established at or near existing grade surface. Considering the presence of upper dry, low density compressible soils existing as described, it is our opinion that for adequate structural support, moderate site preparations and grading should be anticipated in form of subexcavations of the near grade soils and their replacement as engineered fills compacted to minimum 90%.

For the structural pad proposed, the grading should include subexcavations to (i) 5 feet below the current grade surface, however deeper excavations up to 10 feet will be required due to the presence of debris fills up to 5-10 feet encountered during excavation of TP-2 and TP-3 or (ii) to the depth as required to expose the underlying moist and dense natural soils, or (iii) to the depth as required to maintain a 24" thick compacted fill mat blanket below foundation bottoms, *whichever is greater.* Actual subexcavation depth should be determined by soils engineer during grading.

Unless otherwise required by the local agency, the site preparations and grading described should encompass, in minimum, the planned structural foot-print area and minimum 5 feet beyond. Where restricted due to existing structure or development, adjacent to such, deeper subexcavations may be warranted using alternate slot-cut method. The load bearing engineered fills described should be compacted to minimum 90% of the soil Maximum Dry Density as determined by the ASTM D1557 test method. During grading, use of heavy-duty construction equipment is suggested.

The subexcavation depths described should be considered as "preliminary". Localized additional subexcavations may be required within areas underlain by undocumented old fills, buried utilities and abandoned sewer and/or buried septic systems. It is recommended that the excavated subgrades should be verified and approved by soils engineer prior to structural fill soil placement.

General Earthwork recommendations are enclosed in Section 5 of this report.

## 4.2 Structural Fills

### 4.2.1 Structural Fill Material

The local soils free of organic, roots, debris and rocks larger than 8-inch in diameter may be considered suitable for re-use as structural backfill.

Backfills placed should be compacted to minimum 90% of the soil Maximum Dry Density as determined by the ASTM D1557 test method. Import soils, if required, should be equivalent to local soils, or better as approved by soils engineer. In general, fill soils for structural support shall be "zero-to-low" in expansion potential, meeting the following criteria:

| | |
|---|---|
| Liquid Limit | <35 |
| Plasticity Index | <15 |
| Expansion Index | <20 |

### 4.2.2 Structural Fill Placement

All structural fill shall be placed in 6 to 8-inch loose lifts and uniformly moisture conditioned. Each lift should be compacted to minimum 90 percent. No fill shall be placed, spread, or compacted -during unfavorable weather conditions.

## 4.3 Structural Foundations

Considering the close proximity of the nearby earthquake fault, and the PGA as described, for structural support, the following foundation recommendations are suggested.

### 4.3.1 Spread Foundations

Structural design should conform to the current CBC/UBC Seismic Design requirements as described in earlier sections of this report.

In general, the proposed structure may be supported by continuous wall and/or isolated spread footings founded exclusively into engineered fill compacted to minimum 90%. Considering the commercial/industrial structure nature of the subject development, it is recommended that from geotechnical view point, conventional footings may be sized to a minimum 15" wide, embedded to minimum 18" below the lowest outside adjacent final grade (and not measured from interior floor grades). Actual foundation dimensions, however, should be determined by the project structural engineer based on anticipated structural dead and live loadings, soil vertical bearing capacity, soil lateral active pressures and passive resistance, as well as the described PGA, among others as determined by the project design engineer.

Use of footings straddling over cut/fill transition, shall be avoided. Excavated footings trenches should be sufficiently "moistened", re-compacted if necessary and verified and approved in writing by soils engineer immediately prior to concrete placement.

For structural design, with a Factor of Safety of 3.0, the recommended allowable vertical soil bearing capacity is 2500 psf. If normal code requirements are applied, the above capacities may further be increased by an additional 1/3 for short duration of loading which includes the effect of wind and seismic forces. Supplemental 500 psf increments in foundation bearing capacity may be considered for each 1 foot increments in footing embedment, up to a total not exceeding 3500 psf.

From geotechnical view point, footing reinforcements consisting of 2-#4 rebar placed near the top and 2-#4 near bottom of continuous footings are suggested. Additional reinforcements if specified by project structural engineer should be incorporated during construction.

The settlements of properly designed and constructed foundations supported exclusively into engineered fills of site soils or its equivalent or better, and carrying the maximum anticipated structural loadings, are expected to be within tolerable limits. For static loading condition, over a span of 40 ft, estimated total and differential settlements are about 1 and 1/2-inch, respectively.

Should the project structural engineer determine that more stringent design criteria are required, those criteria should supersede the design parameters supplied herein.

Excavated footing trenches prepared to receive concrete should be verified and approved by soils engineer in writing prior to forming and reinforcement placement.

## 4.4 Interior Concrete Slab-on-Grade Flooring

The prepared subgrades compacted to minimum 90% prepared to receive footings should be adequate for concrete slab-on-grade placement. For interior commercial use, 4" thick (net) concrete slab-on-grade may be considered underlain by 2-inch of compacted clean sand, followed by 10-mil thick commercially available vapor barrier, such as Stego-Wrap or its equivalent, or better. The installations of such should be as per manufacturer's specifications. The gravelly sands used underneath vapor barrier should have a Sand Equivalent, SE, of 30 or greater.

### Exterior Rigid Driveways

Concrete Paving, if considered, should be at least 6-inch thick reinforced with #5 rebar at 24" o/c, placed directly over the local sandy gravelly soils compacted to minimum 95%. Actual paving thickness, however, should be supplied by the project structural engineer based on soil Subgrade Reaction, $k_s$, of 450 kcf as described. Use of adequate construction/expansion joints are suggested at intervals 24 to 36 times the concrete thickness, or as determined by the project design engineer. Use of low-slump concrete is recommended.

In addition, it is recommended that utility trenches underlying concrete slabs and driveways should be thoroughly backfilled with gravelly sandy soils mechanically compacted to the recommended minimum prior to concrete pour. No water jetting should be allowed in lieu of the recommended mechanical compaction.

### 4.4.1 Concrete Curing and Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to concrete curing or settlement. However, even when the following recommendations have been implemented; foundations, stucco walls and concrete slabs-on-grade may display some minor cracking due to minor soil movement and/or concrete shrinkage.

To reduce and/or control concrete shrinkage, curling or cracking, concrete slabs shall be "cured" by using water prior to structural load placement. The following general procedures are recommended:

1. CONCRETE STRENGTH @ 28 DAYS SHOULD BE AS DETERMINED BY STRUCTUAL ENGINEER.
2. BEFORE OPERATING VEHICLES AND EQUIPMENT ON SLABS, INSURE CONCRETE SLABS HAVE PROPERLY CURED.
3. DO NOT POUR CONCRETE WHEN THE TEMPERATURE EXCEEDS 90$^{O}$ F OR 80$^{O}$ F WHEN THE WIND EXCEEDS 12MPH. CONCRETE POURING IN EXTREME WEATHER CONDITIONS IS NOT RECOMMENDED.
4. START CURING AS SOON AS HARD TROWELING IS DONE. ALL CURING SHALL BE WET CURING BY USING BURLAP FOR A MINIMUM OF 7 DAYS. BURLAP MUST BE PLACED WITHIN 2 HOURS OF POURING (NO SPRAY CURING).
5. WHEN WIND, TEMPERATURE AND HUMIDITY CONDITIONS CAUSE EARLY DISAPPEARANCE OF BLEED WATER, STEPS SHALL BE TAKEN TO USE A FOG SPRAY. CURING SHALL COMMENCE IMMEDIATELY AFTER FINISHING TROWELING.

The occurrence of concrete cracking may also be reduced and/or controlled by limiting concrete slump, concrete curing, and by placement of crack control joints at intervals not exceeding 24 to 30 times the slab thickness, or as determined by the project structural engineer.

## 4.5 Resistance to Lateral Loads

Resistance to lateral loads can be restrained by friction acting at the base of foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with normal dead load forces for footing established on compacted fill.

An allowable passive lateral earth resistance of 230 pounds per square foot per foot of depth may be assumed for the sides of foundations poured against compacted fill local soils or its similar. The maximum lateral passive earth pressure is recommended not to exceed 2300 pounds per square foot.

For design, lateral pressures from local soils when used as level backfill may be estimated from the following equivalent fluid density:

| | |
|---|---|
| Active: | 35 pcf |
| At Rest: | 75 pcf |

The above values may be increased by 1/3 when designing for short duration wind or seismic forces. The above values are based on footings placed on compacted engineered fills. In the case where footing sides are formed, all backfill placed against the footings should be compacted to at least 90 percent of maximum dry density.

### 4.6 Shrinkage and Subsidence

Based on the results of field observations and laboratory testing, it is our opinion that the upper soils when graded may be subjected to a volume change. Assuming a 90% relative compaction for structural fills and assuming an over-excavation and recompaction depth of about 5 feet, such volume change due to shrinkage may be on the order of 10 to 15 percent. Further volume change may be expected following removal of buried utilities, roots and surface vegetation. Supplemental shrinkage is expected during preparation of the underlying natural soils prior to compacted fill placement.

For estimation purposed, site subsoils subsidence may be approximated to about 2.5-inch when conventional construction equipments are used. Lesser shrinkage and subsidence is expected for the soil existing at 5 feet and below.

### 4.7 Construction Consideration

#### 4.7.1 Unsupported Excavation

Temporary construction excavation up to a depth of 5 feet may be made without any lateral support. It is recommended that no surcharge loads such as construction equipments, be allowed within a line drawn upward at 45 degree from the toe of temporary excavations. Use of sloping for deep excavation may be considered where plan excavation dimensions are not constrained by any existing structure.

#### 4.7.2 Supported Excavations

If vertical excavations exceeding 5 feet in depths become warranted, such should be achieved using shoring to support side walls.

### 4.8 Site Preparations

Site preparations for structural pad should include subexcavation of the upper loose, disturbed compressible soils, stock-piling, moisturization and/or aeration to 5% to 8% over the laboratory determined Optimum Moisture contents of the soils used. Site preparation should also include replacement of the excavated soils and other approved imported fills compacted to the minimum percentage described. Such earth work should be in accordance with the applicable grading recommendations provided in the current CBC and as recommended in earlier Section of this report.

### 4.9 Soil Caving

Considering the sandy site soils, minor caving may be expected during deep excavations. Temporary excavations in excess of 5 feet should be made at a slope ratio of 2 to 1 (h:v) or flatter, or as per the construction guidelines as provided by Cal-Osha.

### 4.10 Structural Pavement Thickness

**Flexible Asphalt Paving:** Based on laboratory determined soil Sand Equivalent, SE, and on an estimated soil R-value of about 50, the following flexible pavement sections are provided for preliminary estimation purposes.

| Service Area | Traffic Index, TI | Pavement Type | Paving Thickness (inch) |
|---|---|---|---|
| On-site paving/parking for commercial vehicle/conventional passenger cars | 6.5 | a.c. over CL. II base | 4.5 over 5.5 |

Within paving areas, subgrade soils should be scarified to 12-inch, moisture conditioned from 5% to 8% over Optimum, and recompacted to at least 95 percent relative to soil's maximum Dry Density as determined by the method ASTM D1557 test procedures. The asphalt used and the Class II base recommended, should also be required to be compacted to minimum 95%, unless otherwise specified by the local governing agency having jurisdiction.

The pavement evaluations are based on estimated Traffic Index (TI) as shown and on the soil R-value as described. It is recommended that following mass grading completion, representative site soils should be laboratory tested to determined actual soil R-value, based on which and on the TI as provided by the local public agency designed paving thickness should be determined for actual implementation on site.

### 4.11 Utility Trench Backfill

Utility trench backfill within the structural pad and beyond should be placed in accordance with the following recommendations:

- Trench backfill should be placed in thin lifts compacted to 90 percent or better of the laboratory maximum dry density for the soils used. As an alternative; clean granular sand may be used having a SE value greater than 30. Jetting is not recommended within utility trench backfill.

- Exterior trenches along a foundation or a toe of a slope and extending below a 1:1 imaginary line projected from the outside bottom edge of the footing or toe of the slope should be compacted to 90 percent of the Maximum Dry Density for the soils used during backfill. All trench excavations should conform to the requirements and safety as specified by the Cal-Osha

### 4.11.1 Utilities

With the potential surface manifestation of liquefaction, including earthquake induced ground settlement, etc., the use of commercially available flexible connections for utilities and life-line services provided from outside the structure is suggested.

Utility knockouts in foundation walls should be oversized to accommodate differential movements. Utility trenches are a common source of water infiltration and migration. If granular fill materials are placed beneath the building, utility trenches that penetrate beneath the building should be effectively sealed to restrict water intrusion and flow through the trenches that could migrate below the building.

## 4.12 Pre-construction Meeting

It is recommended that no clearing of the site or any grading operation be performed without the presence of a representative of this office. An on-site pre-grading meeting should be arranged between the soils engineer and the grading contractor prior to any construction.

## 4.13 Seasonal Limitations

No fill shall be placed, spread or rolled during unfavorable weather conditions. Where the work is interrupted by heavy rains, fill operations shall not be resumed until moisture conditions are considered favorable by the soils engineer.

## 4.14 Planters

To minimize potential differential settlement to foundations, planters requiring heavy irrigation should be restricted from using adjacent to footings. In event such becomes unavoidable, planter boxes with sealed bottoms, should be considered.

## 4.15 Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Pad drainage should be directed towards streets and to other approved areas away from foundations. Slope areas should be planted with draught resistant vegetation. Over watering landscape areas could adversely affect the proposed site development during its life-time use.

## 4.16 Observations and Testing During Construction

Prior to grading and construction, it is recommended that final grading, foundation plan and foundation details should be available to Soils Southwest for review and approval.

Recommendations provided are based on the assumption that mass grading for structural pad and beyond, as well as post grading including structural footings and slab-on-grade be established exclusively into compacted fills verified and approved by Soils Southwest, the project Geotechnical Engineer. Excavated footings should be inspected, verified and certified by prior to steel and concrete placement to ensure their sufficient embedment and proper bearing as recommended in this report. Structural backfills discussed should be placed under direct observations and testing by this facility. Excess soils generated from footing excavations should be removed from pad areas and such should not be allowed on subgrades underlying concrete slab. In event other geotechnical consultants are retained during grading, Soils Southwest, Inc. will not be held responsible for any distress that may occur during life-time use of the structures constructed.

## 4.17 Plan Review

In absence of grading and development plan, the recommendations supplied should be considered "preliminary". It is recommended that grading and development plans, when prepared, should be available for review to verify adequacy of the geotechnical recommendations supplied for the project. Supplemental recommendations may be warranted prior to actual grading and construction.

## 5.0 Earth Work/General Grading Recommendations

Site preparations and grading should involve over-excavation and replacement of local soils as structural fill compacted to 90% or better. Although no significant variations in soil conditions are anticipated, actual soils conditions may vary in the event subgrades exposed during construction are found different from those as described in this report. It will be the subcontractor's responsibility to notify Soils Southwest about sub soil variation, if any, for revised/updated recommendations.

### Structural Backfill:

Local soils free of debris, large rocks and organic should be considered suitable for reuse as backfill. Loose soils, formwork and debris should be removed prior to backfilling retaining walls. On-site sand backfill should be placed and compacted in accordance with the recommended specifications provided below. Where space limitations do not allow conventional backfilling operations, special backfill materials and procedures may be required. Pea gravel or other select backfill can be used in limited space areas. Additional recommendations on such will be supplied when requested.

### Site Drainage:

Adequate positive drainage should be maintained away from the structural pads constructed. A 2% desirable slope for surface drainage is recommended. Planters and landscaped areas adjacent to building should be designed as such so as to minimize water infiltration into sub-soils. Adjacent to footings, use of planter areas with closed bottoms and controlled drainage, should be considered.

### Utility Trenches:

Buried utility conduits should be bedded and backfilled around the conduit in accordance with the project specifications. Where conduit underlies concrete slab-on-grade and pavement, the remaining trench backfill above the pipe should be mechanically compacted.

### General Grading Recommendations:

Recommended general specifications for surface preparation to receive fill and compaction for structural and utility trench backfill and others are presented below.

1. Areas to be graded, backfilled or paved, shall be grubbed, stripped and cleaned of all buried and undetected debris, structures, concrete, vegetation and other deleterious materials prior to grading.

2. Where compacted fill is to provide vertical support for foundations, all loose, soft and other incompetent soils should be removed to full depth as approved by soils engineer, or at least up to the depth as previously described in this report. The areas of such removal should extend at least 5 feet beyond the perimeter of exterior foundation limit or to the extent as approved by soils engineer during grading.

3. The fills to support foundations and slab-on-grade should be compacted to minimum 90% of the soil's Maximum Dry Density at 3 to 5% over Optimum. In order to minimize potential differential settlements to foundations and slabs straddling over cut and fill transition, cut portions following cut, should be further over excavated and such be replaced as engineered fill compacted to at least 90% of the soil's Maximum Dry Density as described in this report.

4. Utility trenches within building pad areas and beyond should be backfilled with granular material and such

should be mechanically compacted to at least 90% of the maximum density for the material used.

5. Compaction for structural fills shall be determined relative to the maximum dry density as determined by ASTM D1557-91 compaction methods. All in-situ field density of compacted fill shall be determined by the ASTM D1556-82 standard methods or by other approved procedures.

6. All new imported soils if required shall be clean granular non-expansive material or as approved by the soils engineer.

7. During grading, fill soils shall be placed as thin layers, thickness of which following compaction shall not exceed six to eight inches.

8. No rocks over six to eight inches in diameter shall be permitted to use as a grading material without prior approval of soils engineer.

9. No jetting and/or water tampering be considered for backfill compaction for utility trenches without prior approval of the soils engineer. For such backfill, hand tampering with fill layers of 8 to 12 inches in thickness, or as approved by the soils engineer is recommended.

10. Utility trenches at depth and cesspool and abandoned septic tank existing within building pad areas and beyond, should be excavated and removed, or such should be backfilled with gravel, slurry or by other material as approved by soils engineer.

11. Imported fill soils if required, should be equivalent to site soils or better. Such should be approved by the soils engineer prior to their use.

12. Grading required for pavement, side-walk or other facilities to be used by general public, should be constructed under direct observation of soils engineer or as required by the local public agencies.

13. A site meeting should be held between grading contractor and soils engineer prior to actual construction. Two days of prior notice will be required for such meeting.

## 6.0 On-Site WQMP-BMP Storm Water Infiltration Testing

Two (2) infiltration testing are performed at 8 feet below grade using the standardized "falling-head" test converted to infiltration rate as per the guidelines of the Table 1, Infiltration Basin Option 2 of test procedures using Porchet methods as described in the Riverside County-Low Impact Development (LID) BMP design handbook. Approximate test locations are as selected by the project civil engineer are as shown on the attached Plate 1.

The soils encountered consist, in general, of upper fine to medium coarse sands with traces of silt, pebbles, rock fragments and occasional rock overlying silty fine to medium coarse with pebbles and rock fragments to the maximum depth described. No shallow depth groundwater was encountered. Descriptions of the soils encountered are provided in the Log of Borings, attached.

Based on the field infiltration testing completed, it is our opinion that the observed average soils infiltration rate is 11.21 in/hr. For design, it is suggested that, use of an appropriate factor of safety should considered to the observed rate as selected by design engineer to account for long-term saturation, inconsistencies in subsoil conditions, potential for silting and lack of maintenance.

### 6.1 Excavated Test Borings

For soil infiltration testing at the locations as shown on the accompanying sketch, two (2) test borings (P-1 & P-2) were made using an 8-inch diameter hollow-stem auger drilling rig, each advanced to 8 feet below the current grade. Water used during percolation testing was supplied by using a 5-gallon water jugs

### 6.2 Methodology and Test Procedures

Equipment Set-Up:

Following two (2) test boring completion, each of the test holes were fitted with perforated pvc pipes backfilled with 6-inch thick crushed rock at the bottom to minimize potentials for scouring and caving.

Prior to actual testing, each excavated test holes were backfilled with water to determine test intervals that will be used during testing. In two consecutive readings, since 6 inches or more of water seeped away within 25 minutes, subsequent percolation testing were performed at 10 minute time intervals for minimum one hour or until the rate became consistent. Testing included water placement up to about 72 inches below the existing grade surface.

The final recorded percolation test rates were converted into an Infiltration Rate ($I_t$) for inch/hour using the "Porchet Method" as described in the Riverside County Low Impact Development (LID) BMP Design Handbook.

## 6.3 Soil Infiltration Test Results

Based on the soils infiltration testing completed at the test locations and to the test depth described, the observed soil percolation rates are described below.

The results of soils percolation rate to soil infiltration rates are in accordance to the Section 2.3 of the Riverside County Handbook.

**TABLE I**

WQMP/BMP Infiltration Percolation Field Data

| Test Date Test No. (1-6-20) | Test Depth (ft.) Below Grade | Observed Percolation Rate (min/inch) | Change in Water Level (inches) | Infiltratoin Rate using ***(Porchet Method Conversion)*** (inch/hour.) |
|---|---|---|---|---|
| P-1 | 105.0 | 0.67 | 15.0 | 9.73 |
| P-2 | 89.0 | 0.56 | 18.0 | 12.70 |

Based on the observed test results described, it is our opinion that for design, an average infiltration rate is 11.21 inches/hour. For design, use of an appropriate safety factor is suggested to the observed rate described.

***NOTE: Test data are attached (Appendix D).***

<u>Suggested Site Requirements for Stormwater BMP installation</u>

The invert of stormwater infiltration shall be at least 10 feet above the groundwater elevation. Stormwater infiltration BMPs shall not be placed on steep slopes and shall not create the condition or potential for slopes instability.

Stormwater infiltration shall not increase the potential for static or seismic settlement of structures on or adjacent to the site. Potential geotechnical hazards that shall be addressed including potentials for collapsible and liquefaction, if any.

Stormwater infiltration shall not place an increased surcharge on structures or foundations on or its adjacents. The pore-water pressure shall not be increased on soil retaining structures on or adjacent to the site.

The invert of stormwater infiltration shall be set back at least 15 feet, and outside a 1:1 plan drawn up from the bottom of adjacent foundations.

Stormwater infiltration shall not be located near utility lines where the introduction of stormwater could cause damage to utilities or settlement of trench backfill. Stormwater infiltration is not allowed within 100 feet of any potable groundwater production well.

## 7.0 Closure

The conclusions and recommendations presented are based on the findings and observations made at the time of subsurface test explorations. The recommendations should be considered 'preliminary' since they are based on soil samples only. Supplemental investigation and engineering evaluations may be required following grading plan review.

If during construction, the subsoils exposed appear to be different from those as described in this report, this office should be notified to consider any possible need for revised/updated geotechnical recommendations.

Recommendations provided are based on the assumptions that structural footings will be established exclusively into compacted fill. No footings and/or slabs are allowed straddling over cut/fill transition interface.

Final grading and foundation plans should be reviewed by this office when they become available. Site grading must be performed under inspection by geotechnical representative of this office. Excavated footings should be inspected and approved by soils engineer prior to steel and concrete placement to ensure that foundations are founded into satisfactory soils and excavations are free of loose and disturbed materials.

A pre-grading meeting between grading contractor and soils engineer is recommended prior to construction preferably at the site, to discuss the grading procedures to be implemented and other requirements described in this report to be fulfilled.

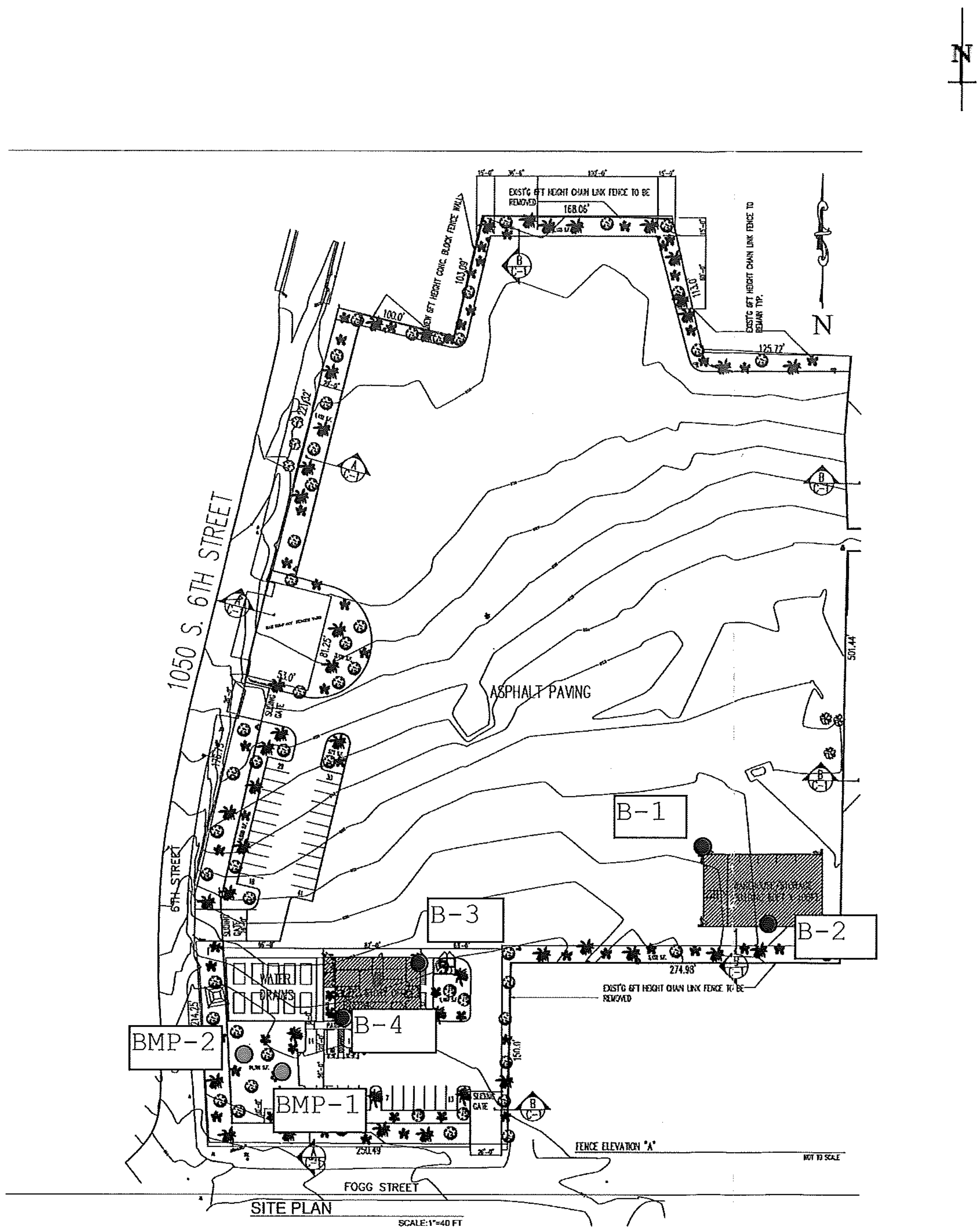
This report has been prepared exclusively for the use of the addressee for the project referenced in the context. It shall not be transferred or be used by other parties without a written consent by Soils Southwest, Inc. We cannot be responsible for use of this report by others without inspection and testing of grading operations by our personnel.

Should the project be delayed beyond one year after the date of this report; the recommendations presented shall be reviewed to consider any possible change in site conditions.

The recommendations presented are based on the assumption that the necessary geotechnical observations and testing during construction will be performed by a representative of this office. The field observations are considered a continuation of the geotechnical investigation performed.

IF ANOTHER FIRM IS RETAINED FOR GEOTECHNICAL OBSERVATIONS AND TESTING, OUR PROFESSIONAL LIABILITY AND RESPONSIBILITY SHALL BE LIMITED TO THE EXTENT THAT SOILS SOUTHWEST, INC. WOULD NOT BE THE GEOTECHNICAL ENGINEER OF RECORD. FURTHER, USE OF THE GEOTECHNICAL RECOMMENDATIONS BY OTHERS WILL RELIEVE SOILS SOUTHWEST, INC. OF ANY LIABILITY THAT MAY ARISE DURING LIFETIME USE OF THE STRUCTURES CONSTRUCTED.

PLOT PLAN AND TEST LOCATIONS
(Not to Scale)




Legend:

- ● B-1 Approximate Location of Test Borings
- ● BMP-1 Approximate Location of BMP Test Boring

Plate 1

# 8.0 APPENDIX A

## Field Explorations

Field investigations included four (4) exploratory soil test borings using a Hollow-Stem Auger (HSA) drill-rig equipped for undisturbed soils sampling and Standard Penetration Testing (SPT) along with two (2) borings for infiltration percolation testing. Approximate test excavation locations are shown on Plate 1. During site reconnaissance, the surface conditions were noted and test excavation locations were determined.

Soils encountered during explorations were logged and such were classified by visual observations in accordance with the generally accepted classification system. The field descriptions were modified, where appropriate, to reflect laboratory test results. Approximate test locations are shown on Plate 1.

Where feasible, relatively undisturbed soils were sampled using a drive sampler lined with soil sampling rings. The split barrel steel sampler was driven into the bottom of test excavations at various depths. Soil samples were retained in brass rings of 2.5 inches in diameter and 1.00 inch in height. The central portion of each sample was enclosed in a close-fitting waterproof container for shipment to our laboratory. In addition to undisturbed sample, bulk soil samples were procured as described in the logs.

Logs of test explorations are presented in the following summary sheets that include the description of the soils and/or fill materials encountered.

## LOG OF TEST EXPLORATIONS

# 9.0 APPENDIX B

## Laboratory Test Programs

Laboratory tests were conducted on representative soils for the purpose of classification and for the determination of the physical properties and engineering characteristics. The number and selection of the types of testing for a given study are based on the geotechnical conditions of the site. A summary of the various laboratory tests performed for the project is presented below.

Moisture Content and Dry Density (D2216):

Data obtained from these test, performed on undisturbed samples are used to aid in the classification and correlation of the soils and to provide qualitative information regarding soil strength and compressibility.

Direct Shear (D3080):

Data obtained from this test performed at increased and field moisture conditions on relatively remolded soil sample is used to evaluate soil shear strengths. Samples contained in brass sampler rings, placed directly on test apparatus are sheared at a constant strain rate of 0.002 inch per minute under saturated conditions and under varying loads appropriate to represent anticipated structural loadings. Shearing deformations are recorded to failure. Peak and/or residual shear strengths are obtained from the measured shearing load versus deflection curve. Test results, plotted on graphical form, are presented on Plate B-1 of this section.

Consolidation (D2835):

Drive-tube samples are tested at their field moisture contents and at increased moisture conditions since the soils may become saturated during life-time use of the planned structure.

Data obtained from this test performed on relatively undisturbed and/or remolded samples, were used to evaluate the consolidation characteristics of foundation soils under anticipated foundation loadings. Preparation for this test involved trimming the sample, placing it in one inch high brass ring, and loading it into the test apparatus which contained porous stones to accommodate drainage during testing. Normal axial loads are applied at a load increment ratio, successive loads being generally twice the preceding.

Soil samples are usually under light normal load conditions to accommodate seating of the apparatus. Samples were tested at the field moisture conditions at a predetermined normal load. Potentially moisture sensitive soil typically demonstrated significant volume change with the introduction of free water. The results of the consolidation tests are presented in graphical forms on Plate B-2.

Potential Expansion (D4829)

Considering silty sand and gravely riverbed type sandy nature, the site soils are considered non-expansive in contact with water, and consequently, no expansion tests are performed and none such are considered necessary at this time.

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF TEST PIT B-1

**Project:** U.S. Auction/ Mr. Karapetian Vahe Trust **Job No.:** 19059-F/BMP

**Logged By:** John F. **Boring Diam.:** 8" HSA **Date:** December 20,2019

| Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| Bulk/Grab | | | | SM-ML | | | seasonal weeds and grass. SAND - light brown, silty, fine, damp, scattered pebbles |
| California sampler | 4.0 | 113.3 | 96.8 | SP | | 5 | - color change to gray-brown, fine to medium pebbles, rock fragments, scattered 1" rock damp - traces silts and clay, fine to medium coarse, pebble, occasional rock fragments scattered rock 1", dry |
| | | | | SM-ML | | 10 | - silty, fine, damp, scattered rock fragment |
| Standard penetration test | | | | VS | | 15 | SILT/SAND MIXTURE- color change to light gray-brown, fine, pebbles, scattered rock fragments, dry to damp medium dense |
| | | | | SM-ML | | 20 | SAND - silty, fine , damp - color change to gray-brown, |
| Standard penetration test | | | | SP | | 25, 30 | - color change to brown, gravely, fine to medium coarse, pebbles, rock fragments, scattered 1/2" rock |
| Standard penetration test | | | | | | | - color change to grayish-light brown, gravely, traces of silt, fine to coarse, dry, very dense |
| | | | | GM-SM | | | GRAVELS with fine silty sand |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**
proposed 2-story office and warehouse
1050 6th St. c/s Fogg
Colton, California

**Plate #**

Bulk/Grab sample | California sampler | Standard penetration test

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897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING B-1

**Project:** U.S. Auction/ Mr. Karapetian Vahe Trust **Job No.:** 19059-F/BMP
**Logged By:** John F. **Boring Diam.:** 8" HSA **Date:** December 20,2019

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | | GP | | 40 | GRAVELS-rocks and cobbles |
| 71 | | | | | GP-SP | | | SAND - color change to light brown, gravely, traces of silt, fine to coarse, very dense |
| | | | | | GP | | | GRAVEL |
| | | | | | | | 45 | |
| | | | | | | | 50 | |
| 44 | | | | | SP | | | SAND- color change to light gray brown, fine to medium, dense |
| | | | | | | | | - End of test boring @ 51.0 ft.<br>- no bedrock<br>- no groundwater |
| | | | | | | | 55 | |
| | | | | | | | 60 | |
| | | | | | | | 65 | |
| | | | | | | | 70 | |

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897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING B-2

**Project:** U.S. Auction/ Mr. Karapetian Vahe Trust **Job No.:** 19059-F/BMP
**Logged By:** John F. **Boring Diam.:** 8" HSA **Date:** December 20,2019

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | | SM-ML | | | seasonal grass and weeds, scattered organic debris |
| | | | | | | | | SAND - gray brown, silty, fine, loose |
| | California sampler | 4.9 | 94.3 | 80.6 | SW | | 5 | - fine, scattered pebbles, loose |
| 5 | Standard penetration test | | | | SM | | | - color change to grayish light brown, fine to medium, pebble, scattered rock, dry |
| | California sampler | 8.6 | 100.6 | 86.0 | VS | | 10 | SILT/SAND MIXTURE - fine, dry to damp, loose |
| | | | | | | | 15 | |
| 6 | Standard penetration test | | | | ML | | | SILT- color change to gray-brown, moist, loose |
| | | | | | | | 20 | - End of test boring @ 16.0 ft.<br>- no bedrock<br>- no groundwater |
| | | | | | | | 25 | |
| | | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**
proposed 2-story office and warehouse
1050 6th St. c/s Fogg
Colton, California

**Plate #**

Bulk/Grab sample    California sampler    Standard penetration test

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897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING B-3

| **Project:** U.S. Auction/ Mr. Karapetian Vahe Trust | | **Job No.:** 19059-F/BMP |
|---|---|---|
| **Logged By:** John F. | **Boring Diam.:** 8" HSA | **Date:** December 20,2019 |

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | | SM-ML | | | seasonal grass, weeds, scattered organics, and pebbles |
| | | | | | | | | SAND - brown to gray, silty, fine, occasional pebble and rock, damp |
| 28 | Bulk/Grab sample | | | | SM | | 5 | - color return to brown, silty, fine to medium, pebble, scattered rock fragments and 1" rock, medium dense |
| | | | | | SM-ML | | | - (Max Dry Density = 117 pcf @ 11.0 %) |
| | | | | | | | | - color change to grayish light brown with occasional rock fragments |
| 6 | Standard penetration test | | | | SP | | | - color change to light brown, gravelly, medium coarse |
| | | | | | ML | | 10 | SILT- medium stiff |
| | California sampler | 15 | 90.9 | 77.7 | VS | | | SILT/SAND MIXTURE- color change to gray brown with trace orange brown, soft |
| | | | | | | | 15 | |
| | | | | | SP-SM | | | SAND - color change to gray-brown, slightly silty fine to medium, pebbles, damp |
| 24 | Standard penetration test | | | | | | 20 | - color change to light brown, silty, fine to medium, occasional pebbles, dry |
| | | | | | | | | - End of test boring @ 21. 0 ft.<br>- no bedrock<br>- no groundwater |
| | | | | | | | 25 | |
| | | | | | | | 30 | |

| | | |
|---|---|---|
| **Groundwater:** n/a<br>**Approx. Depth of Bedrock:** n/a<br>**Datum:** n/a<br>**Elevation:** n/a | **Site Location**<br>proposed 2-story office and warehouse<br>1050 6th St. c/s Fogg<br>Colton, California | **Plate #** |

Bulk/Grab sample    California sampler    Standard penetration test

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING B-4

**Project:** U.S. Auction/ Mr. Karapetian Vahe Trust | **Job No.:** 19059-F/BMP
**Logged By:** John F. | **Boring Diam.:** 8" HSA | **Date:** December 20,2019

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|
| | | | | | SM-ML | | low weeds |
| | | | | | | | SAND - brown, silty, fine, pebbles, damp |
| | | | | | | | - color change to gray-brown, sitly, fine rock fragments, scattered rock |
| | | | | | SP | 5 | - color change return to brown, fine to medium, pebbles, rock fragments, occasional rock |
| 23 | Standard penetration test | | | | SM-SC | | |
| | California sampler | 10 | 107.6 | 92 | | | - color change to light brown to gray-brown slightly clayey, fine, occasional pebble scattered rock fragments, medium dense |
| | | | | | | 10 | - silty, fine, scattered pebble and rock fragments, dense |
| 6 | Standard penetration test | | | | | | - color change to yellowish brown, fine |
| | | | | | | | - very loose |
| | | | | | ML | 15 | SILT- color change to light yellow |
| 12 | Standard penetration test | | | | SM-SC | | SAND- color change to gray-brown, silty, clayey, scatter pebble moist |
| | | | | | GP-SP | | - color change to light gray, gravely, medium coarse, dry, low to medium dense |
| | | | | | | | - End of test boring @ 16.0 ft. |
| | | | | | | | - no bedrock |
| | | | | | | 20 | - no groundwater |
| | | | | | | 25 | |
| | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**
proposed 2-story office and warehouse
1050 6th St. c/s Fogg
Colton, California

**Plate #**

Bulk/Grab sample | California sampler | Standard penetration test

Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING WQMP/ BMP-1

| **Project:** U.S. Auction/ Mr. Karapetian Vahe Trust | | **Job No.:** 19059-F/BMP |
|---|---|---|
| **Logged By:** John F. | **Boring Diam.:** 8" HSA | **Date:** December 20,2019 |

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | | SM | | | scattered low weeds and surface seasonal weeds |
| | | | | | | | | SAND - gray-brown, silty, fine to medium, pebbles, rock fragments, occasional 1/4" to 1" rock, damp |
| | | | | | | | 5 | |
| | | | | | SM-ML | | | - with traces of clay, pebbles, scattered rock fragments |
| | | | | | | | | - End of infiltration test boring @ 8.0 ft.<br>- no bedrock<br>- no groundwater<br>- perforated pvc pipe installed |
| | | | | | | | 10 | |
| | | | | | | | 15 | |
| | | | | | | | 20 | |
| | | | | | | | 25 | |
| | | | | | | | 30 | |

| **Groundwater:** n/a<br>**Approx. Depth of Bedrock:** n/a<br>**Datum:** n/a<br>**Elevation:** n/a | **Site Location**<br>proposed 2-story office and warehouse<br>1050 6th St. c/s Fogg<br>Colton, California | **Plate #** |
|---|---|---|

Bulk/Grab sample — California sampler — Standard penetration test

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897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

# LOG OF BORING WQMP/ BMP-2

| | | | |
|---|---|---|---|
| **Project:** U.S. Auction/ Mr. Karapetian Vahe Trust | | **Job No.:** 19059-F/BMP | |
| **Logged By:** John F. | **Boring Diam.:** 8" HSA | **Date:** December 20,2019 | |

| Standard Penetration (Blows per Ft.) | Sample Type | Water Content in % | Dry Density in PCF | Percent Compaction | Unified Classification System | Graphic | Depth in Feet | Description and Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | | SP | | | scattered low weeds and surface seasonal weeds |
| | | | | | | | | SAND - tan-brown, gravely, traces of silt, fine to medium coarse, pebbles,rock fragments, damp |
| | | | | | SM-ML | | 5 | - color change to gray-brown, silty, traces of clay, fine, with occasional pebbles, scattered rock, damp |
| | | | | | SM | | | - silty, fine to medium coarse, pebbles rock fragments |
| | | | | | | | 10 | - End of infiltration test boring @ 8.0 ft.<br>- no bedrock<br>- no groundwater<br>- perforated pvc pipe installed |
| | | | | | | | 15 | |
| | | | | | | | 20 | |
| | | | | | | | 25 | |
| | | | | | | | 30 | |

**Groundwater:** n/a
**Approx. Depth of Bedrock:** n/a
**Datum:** n/a
**Elevation:** n/a

**Site Location**
proposed 2-story office and warehouse
1050 6th St. c/s Fogg
Colton, California

**Plate #**

Bulk/Grab sample    California sampler    Standard penetration test

# KEY TO SYMBOLS

| Symbol | Description |
|---|---|
| **Strata symbols** | |
| | Poorly graded silty fine sand |
| | Poorly graded sand |
| | Variable sand and silt mix |
| | Silty sand and gravel |
| | Poorly graded gravel |
| | Poorly graded gravel and sand |
| | Well graded sand |
| | Silty sand |
| | Silt |
| | Poorly graded sand with silt |
| | Poorly graded clayey silty sand |

| Symbol | Description |
|---|---|
| **Soil Samplers** | |
| | Bulk/Grab sample |
| | California sampler |
| | Standard penetration test |

Notes:

1. Exploratory borings were drilled on December 20,2019 using a 4-inch diameter continuous flight power auger.

2. No free water was encountered at the time of drilling or when re-checked the following day.

3. Boring locations were taped from existing features and elevations extrapolated from the final design schematic plan.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.

## Laboratory Test Results

Table I: In-Situ Moisture-Density (ASTM D2216)

| Test Boring No. | Sample Depth, ft. | Dry Density, pcf. | Moisture Content, % |
|---|---|---|---|
| 1 | 5 | 113.3 | 4.0 |
| 2 | 3 | 98.9 | 4.9 |
| 2 | 8 | 100.6 | 8.6 |
| 3 | 10 | 90.9 | 14.7 |
| 4 | 7 | 107.6 | 10.4 |

Table II: Max. Density/Optimum Moisture Content (ASTM D1557)

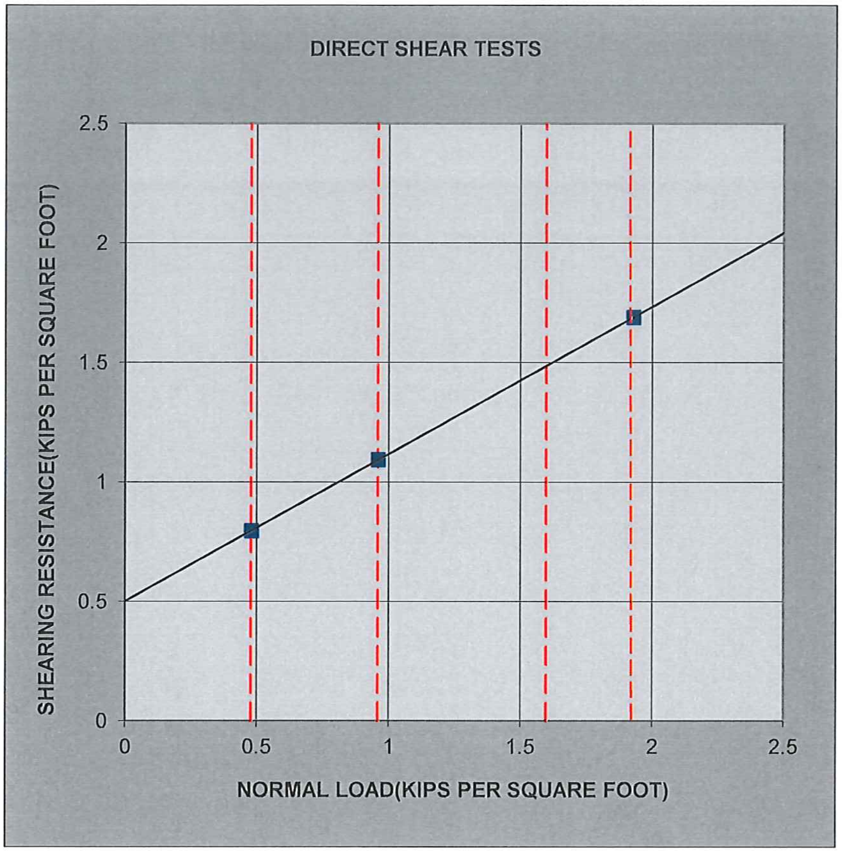
| Sample Location, @ Depth, ft. | Max. Dry Density, pcf | Opt. Moisture (%) |
|---|---|---|
| B-3 @ 3-5 | 117.0 | 11.0 |

Table III: Direct Shear (ASTM D3080)

| Test Boring & Sample Depth | Test Condition | Cohesion (PSF) | Friction (Degree) |
|---|---|---|---|
| B-4 @ 3-5 ft | Remolded to 90% | 500.0 | 43.60 |
| B-1 @ 5.0 | Undisturbed | 200 | 50.00 |

Table IV: Consolidation (D2435)

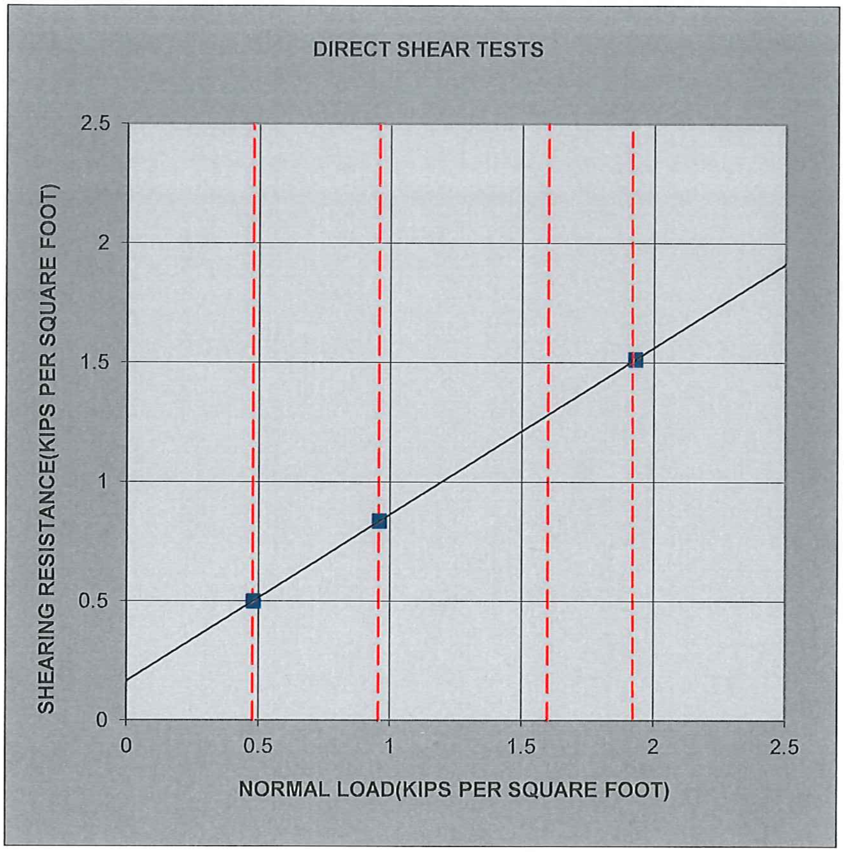
| Boring B # | Depth (ft.) | Consolidation prior to saturation (%) @ 2 kips | Hydro collapse (%) @ 2 kips | Total Consolidation (%@ 8 kips) (saturated) |
|---|---|---|---|---|
| 1 (remolded) | 3 - 7 | 0.8 | 1.9 | 8.8 |
| 1 (undisturbed) | 5.0 | 0.8 | -0.8 | 1.4 |




| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | B-4 | 3 to 5 | Remolded to 90% | 500.84 | 31.65 |

| | | |
|---|---|---|
| Proposed Office and Warehouse<br>1050 S. 6th Street and Fogg St.<br>Colton, California | PROJECT NO. | 19059-F |
| | PLATE | B-1 |

**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**

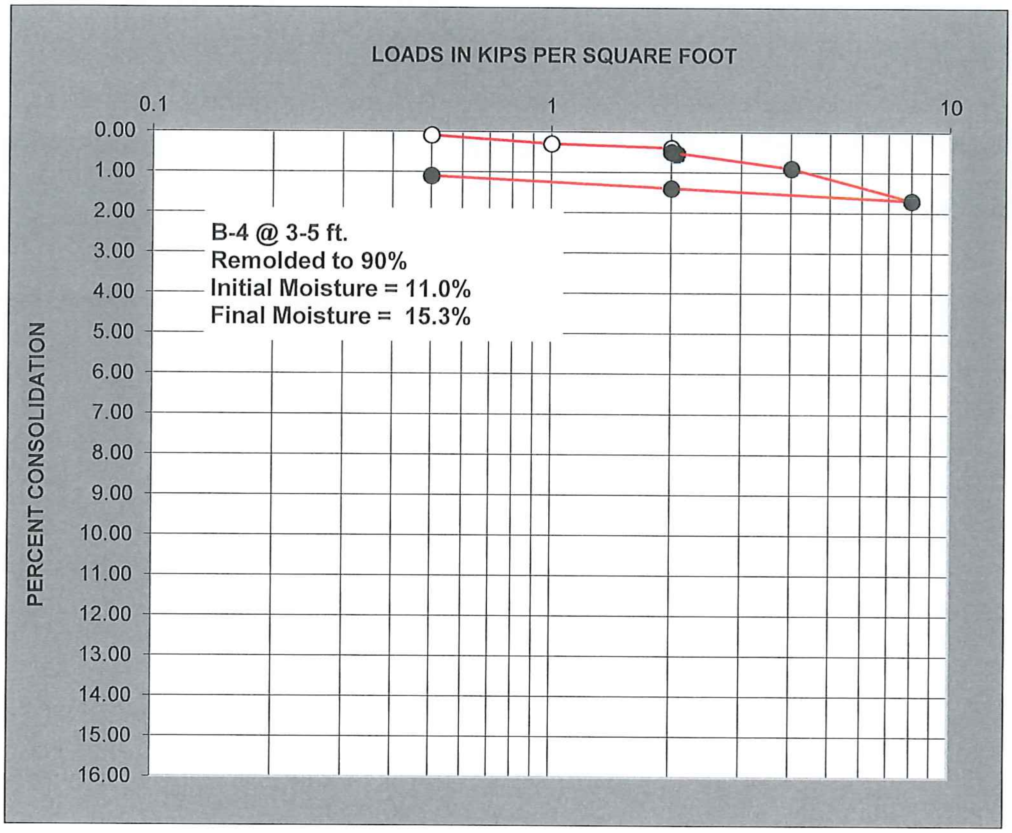



| SYMBOL | LOCATION | DEPTH (FT) | TEST CONDITION | COHESION (psf) | FRICTION (degree) |
|---|---|---|---|---|---|
| ■ | B-1 | 5.0 | Undisturbed | 165.00 | 34.96 |

| | | |
|---|---|---|
| Proposed Office and Warehouse<br>1050 S. Sixth St and Fogg Street<br>Colton, California | PROJECT NO. | 19059-F |
| | PLATE | B-1-1 |

**SOILS SOUTHWEST, INC.**
**Consulting Foundation Engineers**

# CONSOLIDATION TESTS

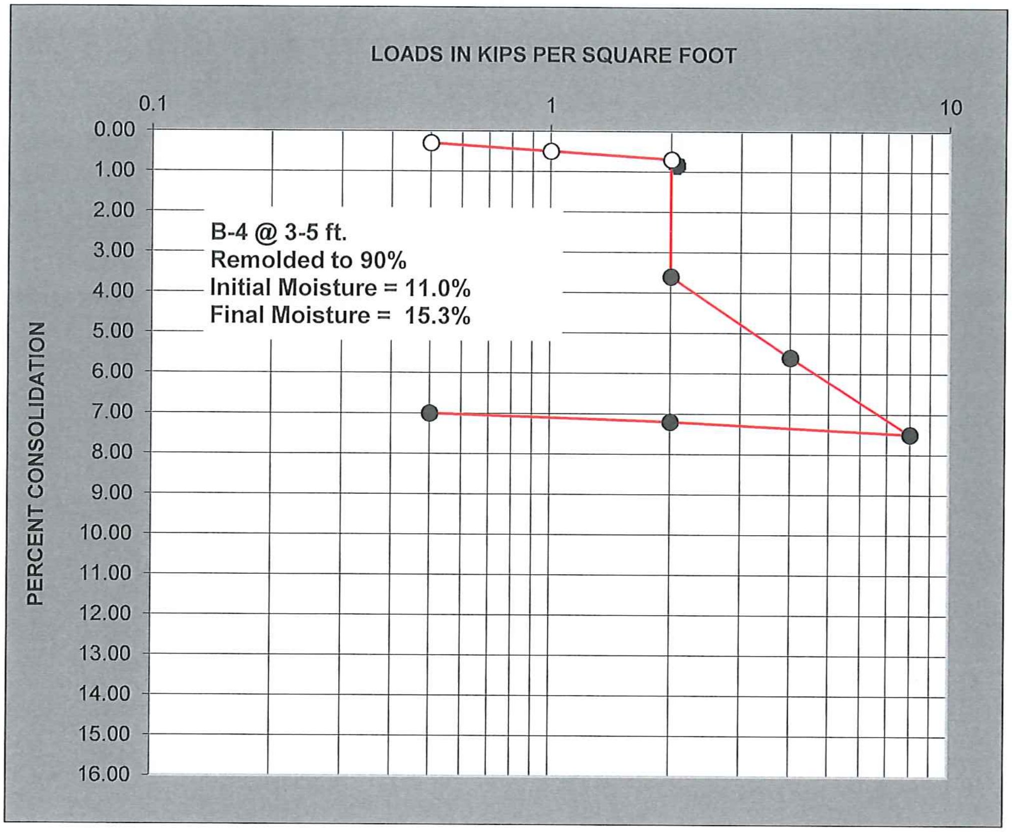



● **WATER PERMITTED TO CONTACT SAMPLE**

| | | | | |
|---|---|---|---|---|
| PROJECT | Proposed Office and Warehouse<br>1050 S. Sixth Street and Fogg Street, Colton, CA | | | |
| PROJECT NO. | 19059-F | PLATE | B-2 | |

**SOILS SOUTHWEST INC.**
**Consulting Foundation Engineers**

# CONSOLIDATION TESTS




● **WATER PERMITTED TO CONTACT SAMPLE**

| | | | | |
|---|---|---|---|---|
| PROJECT | Proposed Office and Warehouse<br>1050 S. Sixth Street and Fogg Street, Colton, CA | | | |
| PROJECT NO. | 19059-F | PLATE | B-2-1 | |

## SOILS SOUTHWEST INC.
**Consulting Foundation Engineers**

# APPENDIX C

Supplemental Seismic Design Parameters

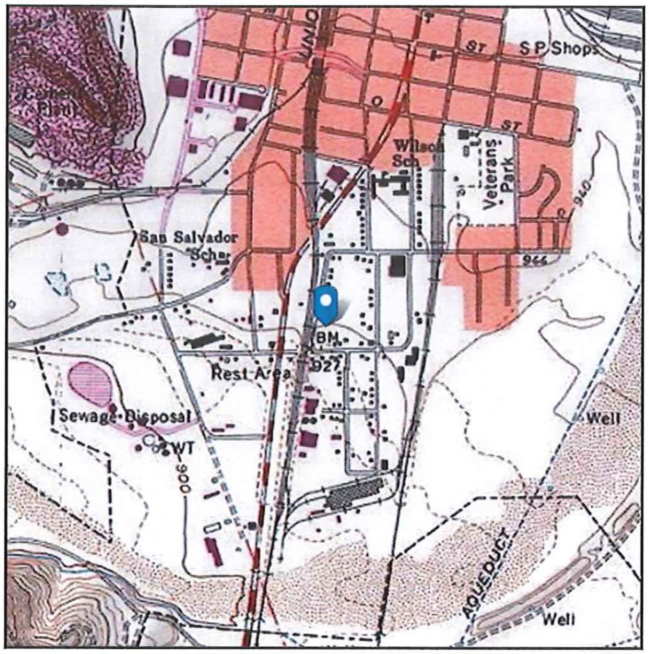
ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

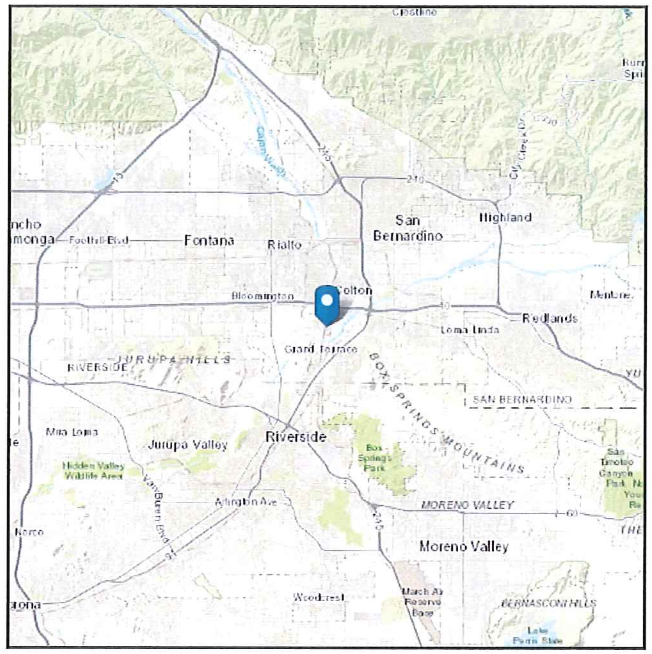
# ASCE 7 Hazards Report

**Address:**
No Address at This Location

**Standard:** ASCE/SEI 7-16
**Risk Category:** III
**Soil Class:** D - Stiff Soil

**Elevation:** 921.76 ft (NAVD 88)
**Latitude:** 34.053354
**Longitude:** -117.328566



ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

## Seismic

**Site Soil Class:** D - Stiff Soil

**Results:**

| | | | |
|---|---|---|---|
| $S_S$ : | 2.059 | $S_{D1}$ : | N/A |
| $S_1$ : | 0.817 | $T_L$ : | 8 |
| $F_a$ : | 1 | PGA : | 0.869 |
| $F_v$ : | N/A | $PGA_M$ : | 0.956 |
| $S_{MS}$ : | 2.059 | $F_{PGA}$ : | 1.1 |
| $S_{M1}$ : | N/A | $I_e$ : | 1.25 |
| $S_{DS}$ : | 1.373 | $C_v$ : | 1.5 |

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

**Data Accessed:** Thu Feb 20 2020

**Date Source:** USGS Seismic Design Maps

Ground Motion Interpolator (2008)

**Longitude:** -117.328566

**Latitude:** 34.053354

**VS30:** 270 (180-1050 m/sec)

**Return Period:**

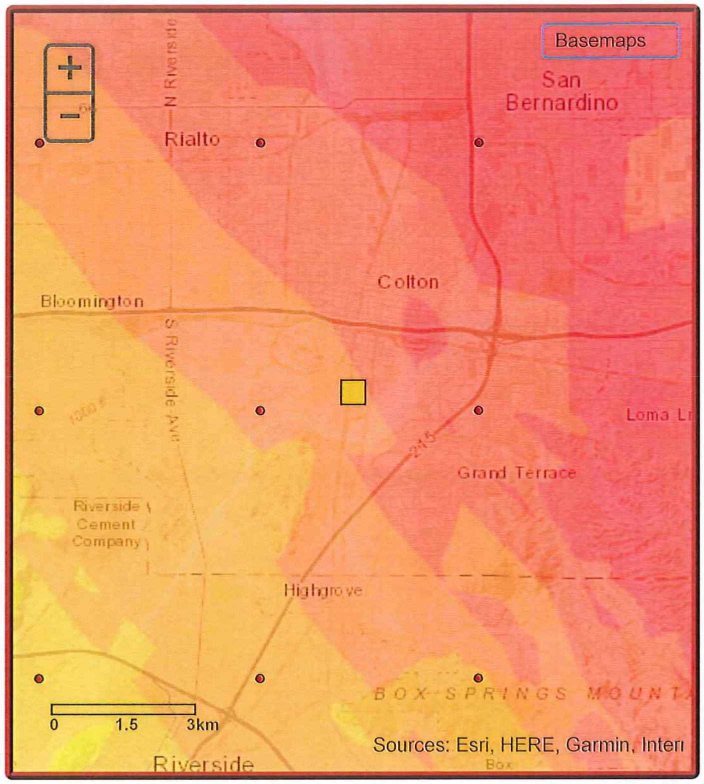
2% in 50 years 10% in 50 years

**Spectral Acceleration:**

PGA 0.2 second SA 1.0 second SA

Submit

| Inputs: | Result: |
|---|---|
| -117.328566, 34.053354<br>vs30: 270 m/sec<br>10% in 50 years<br>PGA | 0.603 g |

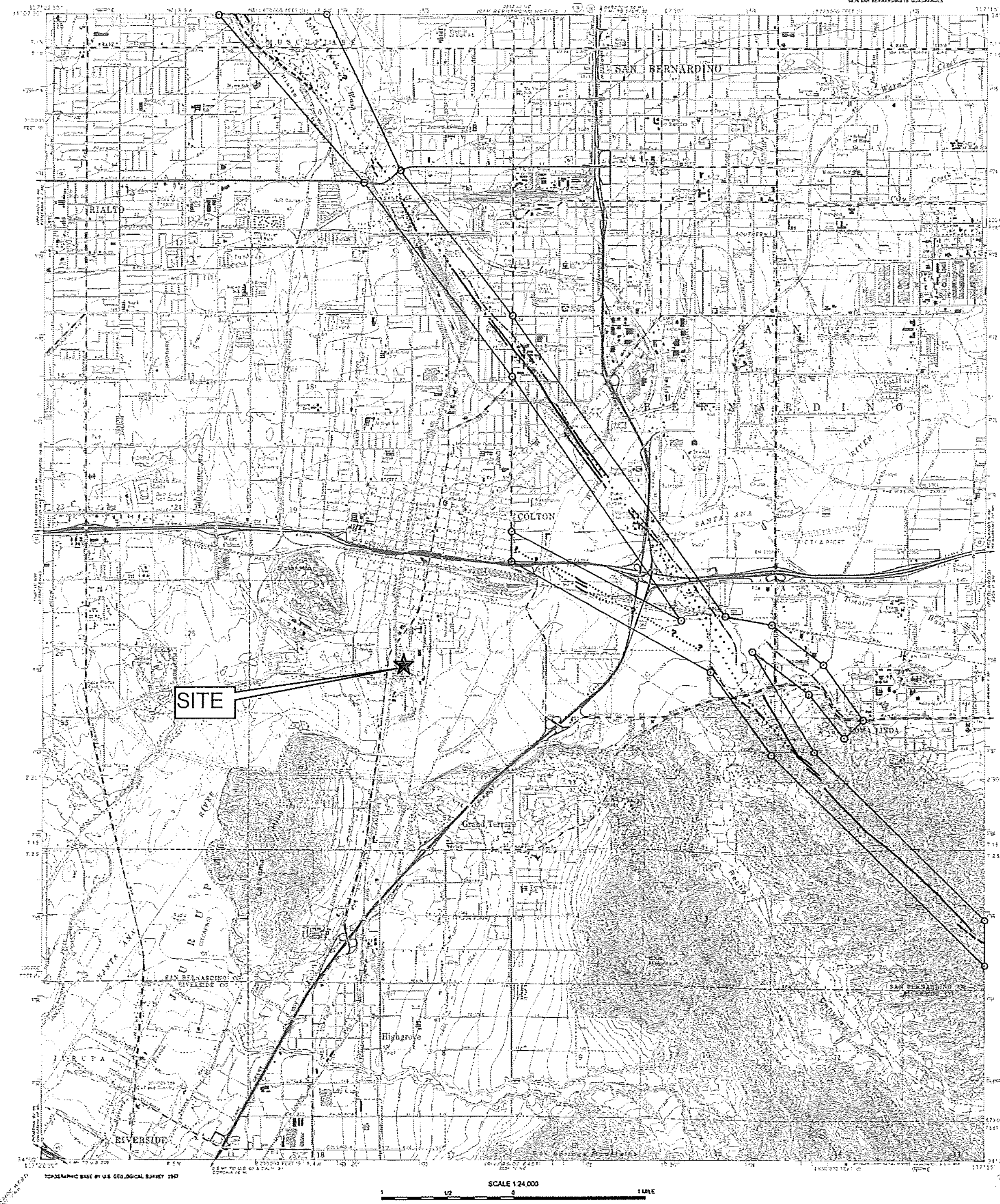



Information and Disclaimer

CALIFORNIA DIVISION OF MINES AND GEOLOGY
THOMAS E. GAY JR., ACTING STATE GEOLOGIST

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF CONSERVATION

SAN BERNARDINO SOUTH QUADRANGLE
CALIFORNIA
7.5 MINUTE SERIES (TOPOGRAPHIC)




## MAP EXPLANATION

**Potentially Active Faults**

Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Aerial photo lineaments (not field checked); based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

**Special Studies Zone Boundaries**

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

# STATE OF CALIFORNIA
# SPECIAL STUDIES ZONES

Delineated in compliance with
Chapter 7.5, Division 2 of the California Public Resources Code

## SAN BERNARDINO SOUTH QUADRANGLE

**REVISED OFFICIAL MAP**

Effective: January 1, 1977

T. E. Gay Jr. Acting State Geologist

### REFERENCES USED TO COMPILE FAULT DATA

San Bernardino South Quadrangle


Dutcher, L.C., and Garrett, A.A., 1963, Geologic and hydrologic features of the San Bernardino area, California, with special reference to underflow across the San Jacinto fault: U.S. Geological Survey Water-Supply Paper 1419, 113 p.
Huber, O.L., 1974, Personal communication based on unpublished data on file with the California Department of Water Resources (Final geologic report, Santa Ana Valley pipeline — Devil Canyon Power Plant to Perris Reservoir).
Sharp, R.V., 1972, Map showing recently active breaks along the San Jacinto fault zone between the San Bernardino area and Borrego Valley, California: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-675.
Sieh, K., 1974, Lineament based on aerial photographic interpretation.
Sieh, K., and others, 1973, Geologic investigations of portions of the San Jacinto fault zone, San Bernardino Valley, California, in W.A. Elders (Editor), Geologic investigations of the San Jacinto fault zone, and aspects of the socio-economic impact of earthquakes in the Riverside-San Bernardino area, California: University of California, Riverside, Campus Museum Contributions No. 3, p. 1-49.


### IMPORTANT - PLEASE NOTE

1) *This map may not show all potentially active faults, either within the special studies zones or outside their boundaries.*
2) Faults shown are the basis for establishing the boundaries of the special studies zones.
3) The identification of these potentially active faults and the location of such fault traces are based on the best available data. Traces have been drawn as accurately as possible at this map scale, however, the quality of data used is highly varied. The faults shown have not been field checked during this map compilation.
4) Fault information on this map is not sufficient to serve as a substitute for information developed by the special studies that may be required under Chapter 7.5, Division 2, Section 2623 of the California Public Resources Code.

# 2008 National Seismic Hazard Maps - Source Parameters

New Search

| Distance in Miles | Name | State | Pref Slip Rate (mm/yr) | Dip (degrees) | Dip Dir | Slip Sense | Rupture Top (km) | Rupture Bottom (km) | Length (km) |
|---|---|---|---|---|---|---|---|---|---|
| 2.29 | San Jacinto;SBV+SJV+A | CA | n/a | 90 | V | strike slip | 0 | 16 | 134 |
| 2.29 | San Jacinto;SBV+SJV+A+C | CA | n/a | 90 | V | strike slip | 0 | 17 | 181 |
| 2.29 | San Jacinto;SBV+SJV+A+CC | CA | n/a | 90 | V | strike slip | 0 | 16 | 181 |
| 2.29 | San Jacinto;SBV+SJV+A+CC+B | CA | n/a | 90 | V | strike slip | 0.1 | 15 | 215 |
| 2.29 | San Jacinto;SBV+SJV+A+CC+B+SM | CA | n/a | 90 | V | strike slip | 0.1 | 15 | 241 |
| 2.29 | San Jacinto;SBV | CA | 6 | 90 | V | strike slip | 0 | 16 | 45 |
| 2.29 | San Jacinto;SBV+SJV | CA | n/a | 90 | V | strike slip | 0 | 16 | 88 |
| 5.77 | San Jacinto;SJV+A | CA | n/a | 90 | V | strike slip | 0 | 17 | 89 |
| 5.77 | San Jacinto;SJV+A+CC | CA | n/a | 90 | V | strike slip | 0 | 16 | 136 |
| 5.77 | San Jacinto;SJV+A+CC+B | CA | n/a | 90 | V | strike slip | 0.1 | 15 | 170 |
| 5.77 | San Jacinto;SJV+A+CC+B+SM | CA | n/a | 90 | V | strike slip | 0.1 | 15 | 196 |
| 5.77 | San Jacinto;SJV | CA | 18 | 90 | V | strike slip | 0 | 16 | 43 |
| 5.77 | San Jacinto;SJV+A+C | CA | n/a | 90 | V | strike slip | 0 | 17 | 136 |
| 8.74 | S. San Andreas;BB+NM+SM+NSB+SSB+BG+CO | CA | n/a | 85 | | strike slip | 0.1 | 13 | 390 |
| 8.74 | S. San Andreas;CH+CC+BB+NM+SM+NSB+SSB+BG+CO | CA | n/a | 86 | | strike slip | 0.1 | 13 | 512 |
| 8.74 | S. San Andreas;NSB+SSB+BG+CO | CA | n/a | 79 | | strike slip | 0.2 | 12 | 206 |
| 8.74 | S. San Andreas;SM+NSB+SSB+BG | CA | n/a | 81 | | strike slip | 0 | 13 | 234 |

# 2008 National Seismic Hazard Maps - Source Parameters

New Search

| Fault Name | State |
|---|---|
| San Jacinto;SBV+SJV+A | California |

| GEOMETRY | |
|---|---|
| Dip (degrees) | 90 |
| Dip direction | V |
| Sense of slip | strike slip |
| Rupture top (km) | 0 |
| Rupture bottom (km) | 16 |
| Rake (degrees) | 180 |
| Length (km) | 134 |

| MODEL VALUES | | |
|---|---|---|
| Slip Rate | n/a | |
| Probability of activity | 1 | |
| | ELLSWORTH | HANKS |
| Minimum magnitude | 6.5 | 6.5 |
| Maximum magnitude | 7.62 | 7.63 |

| | | |
|---|---|---|
| b-value | 0.8 | 0.8 |

| Fault Model | Deformation Model | Char Rate[1] | GR-a-value[1] | Weight |
|---|---|---|---|---|
| Moment Balanced | 2.1 | 4.81e-04 / 4.81e-04 | NA / NA | 0.25 |
| Moment Balanced | 2.2 | 4.81e-04 / 4.81e-04 | NA / NA | 0.10 |
| Moment Balanced | 2.3 | 4.81e-04 / 4.81e-04 | NA / NA | 0.15 |

[1] 1st Value is based on Ellsworth relation and 2nd value is based on Hanks and Bakun relation

# APPENDIX D

Field Infiltration Test Data

# Percolation Test Data Sheet

| | | | | | |
|---|---|---|---|---|---|
| Project: | U,S Auction | Project No: | 19059-BMP | Date: | 1-6-20 |
| Test Hole No: | P-1 | Tested By: | JF | | |
| Depth of Test Hole, $D_T$: | 105 inch | USCS Soil Classification: | | | |

| Test Hole Dimensions (inches) | | | Length | Width | |
|---|---|---|---|---|---|
| Diameter (if round)= | 8 inch | Sides (if rectangular)= | | | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 9:08 | 9:33 | 25 | 81 | 103 | 22.0 | Y |
| 2 | 9:36 | 10:01 | 25 | 81 | 98 | 17.0 | Y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | Δt Time Interval (min.) | $D_o$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | ΔD Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 10:11 | 10:21 | 10 | 81 | 99 | 18.0 | 0.56 |
| 2 | 10:24 | 10:34 | 10 | 81 | 98 | 17.0 | 0.59 |
| 3 | 10:36 | 10:46 | 10 | 81 | 98 | 17.0 | 0.59 |
| 4 | 10:47 | 10:57 | 10 | 81 | 97 | 16.0 | 0.63 |
| 5 | 10:58 | 11:08 | 10 | 81 | 97 | 16.0 | 0.63 |
| 6 | 11:13 | 11:23 | 10 | 81 | 97 | 16.0 | 0.63 |
| 7 | 11:27 | 11:37 | 10 | 81 | 97 | 16.0 | 0.63 |
| 8 | 11:39 | 11:49 | 10 | 81 | 97 | 16.0 | 0.63 |
| 9 | 11:54 | 12:04 | 10 | 81 | 96 | 15.0 | 0.67 |
| 10 | 3:18 | 3:28 | 10 | 81 | 96 | 15.0 | 0.67 |
| 11 | | | | | | | |
| 12 | | | | | | | |
| 13 | | | | | | | |
| 14 | | | | | | | |
| 15 | | | | | | | |

COMMENTS:

# Percolation Test Data Sheet

| Project: | U.S Auction | Project No: | 19059-BMP | Date: | 1-6-20 |
|---|---|---|---|---|---|
| Test Hole No: | P-2 | Tested By: | JF | | |
| Depth of Test Hole, $D_T$: | 95 in | USCS Soil Classification: | | | |
| Test Hole Dimensions (inches) | | | Length | Width | |
| Diameter (if round)= | 8 inches | Sides (if rectangular)= | | | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 9:06 | 9:31 | 25 | 71 | 95 | 24 | Y |
| 2 | 9:36 | 10:03 | 25 | 71 | 93 | 22 | Y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | Δt Time Interval (min.) | $D_o$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | ΔD Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 12:47 | 12:57 | 10 | 65 | 89 | 24.0 | 0.42 |
| 2 | 1:01 | 1:11 | 10 | 65 | 88 | 23.0 | 0.43 |
| 3 | 1:14 | 1:24 | 10 | 65 | 86 | 21.0 | 0.48 |
| 4 | 1:31 | 1:42 | 10 | 65 | 85 | 20.0 | 0.50 |
| 5 | 1:49 | 1:59 | 10 | 65 | 84 | 19.0 | 0.53 |
| 6 | 2:03 | 2:13 | 10 | 65 | 84 | 19.0 | 0.53 |
| 7 | 2:16 | 2:26 | 10 | 65 | 83.5 | 18.5 | 0.54 |
| 8 | 2:30 | 2:40 | 10 | 65 | 83 | 18.0 | 0.56 |
| 9 | 2:47 | 2:57 | 10 | 65 | 83 | 18.0 | 0.56 |
| 10 | 3:02 | 3:12 | 10 | 65 | 83 | 18.0 | 0.56 |
| 11 | 3:16 | 3:26 | 10 | 65 | 83 | 18.0 | 0.56 |
| 12 | | | | | | | |
| 13 | | | | | | | |
| 14 | | | | | | | |
| 15 | | | | | | | |

COMMENTS:

FOR 10 MINUTE TESTING, DEPTH OF TEST HOLE, DT ADJUSTED TO 89 IN

## PROFESSIONAL LIMITATIONS

Our investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances by other reputable Soils Engineers practicing in these general or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

The investigations are based on soil samples only, consequently the recommendations provided shall be considered 'preliminary'. The samples taken and used for testing and the observations made are believed representative of site conditions; however, soil and geologic conditions can vary significantly between test excavations. If this occurs, the changed conditions must be evaluated by the Project Soils Engineer and designs adjusted as required or alternate design recommended.

The report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineers. Appropriate recommendations should be incorporated into structural plans. The necessary steps should be taken to see that out such recommendations in field.

The findings of this report are valid as of this present date. However, changes in the conditions of a property can occur with the passage of time, whether they due to natural process or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur from legislation or broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by change outside of our control. Therefore, this report is subject to review and should be updated after a period of one year.

## RECOMMENDED SERVICES

The review of grading plans and specifications, field observations and testing by a geotechnical representative of this office is integral part of the conclusions and recommendations made in this report. If Soils Southwest, Inc. (SSW) is not retained for these services, the Client agrees to assume SSI's responsibility for any potential claims that may arise during and after construction, or during the life-time use of the structure and its appurtenant.

The recommendations supplied should be considered valid and applicable, provided the following conditions, in minimum, are met:

- i. Pre-grade meeting with contractor, public agency and soils engineer,
- ii. Excavated bottom inspections and verification s by soils engineer prior to backfill placement,
- iii. Continuous observations and testing during site preparation and structural fill soils placement,
- iv. Observation and inspection of footing trenching prior to steel and concrete placement,
- v. Subgrade verifications including plumbing trench backfills prior to concrete slab-on-grade placement,
- vi On and off-site utility trench backfill testing and verifications,
- vii Precise-grading plan review, and
- viii. Consultations as required during construction, or upon your request.

Soils Southwest, Inc. will assume no responsibility for any structural distresses during its life-time use; in event the above conditions are not strictly fulfilled.