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

July 26, 2023

Project No. 20022-APPL

Black Ridge Real Estate Group, Inc.
c/o Mr. Ryan Martin
16901 Millikan Avenue
Irvine, California 92606

Subject: Opinion Letter-Applicability of the Existing Soils Report
Proposed Office/Warehouse Building
75542 Heacock Street
Moreno Valley, California

Reference: Report of Geotechnical Investigations
Prepared by Soils Southwest, Inc., dated August 5, 2020

Gentlemen:

Based on recent site visit, since no evidence is noted of new fill soils placement, or new cut to the grades are apparent based on which the referenced soils report was prepared, it is our opinion that the referenced Report of Geotechnical Investigations, dated August 5, 2020 should be considered valid and applicable for future proposed development.

Respectfully submitted,
Soils Southwest, Inc.

Malay Gupta, RCE 31708

REGISTERED PROFESSIONAL ENGINEER
MALAY K. GUPTA
No 31708
Exp. 12-31-24
CIVIL
STATE OF CALIFORNIA

Dist./1-addressee Ryan Martin (rmartin@ledocg.com)

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Report of Geotechnical Investigations & Soil Infiltration Testing for WQMP-BMP Design

Proposed Office and Warehouse Structures
Proposed Heacock Industrial Development
75542 Heacock Street @ Ironwood Avenue
City of Moreno Valley, California

Project No. 20022-F/BMP

August 5, 2020

Prepared for:

Black Ridge Real Estate Group, Inc.
c/o Mr. Ryan Martin
16901 Millikan Avenue
Irvine, CA 92606

soilssouthwest@aol.com
Established 1984

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August 5, 2020

Project No. 20022-F/BMP

Black Ridge Real Estate Group, Inc.
16901 Millikan Avenue
Irvine, CA 92606

Attention: Mr. Ryan Martin

Subject: Report of Geotechnical Investigations &
Soil Infiltration Testing for WQMP-BMP Design
Planned Office and Warehouse Structure
Proposed Heacock Industrial Development
75542 Heacock Street @ Ironwood Avenue, City of Moreno Valley, California

Reference: Conceptual Site Plan Prepared by SB&O, Inc.

Gentlemen:

Presented herewith are the Reports of Soils and Foundation Evaluations and Soil Infiltration Testing for WQMP-BMP design conducted for the site of the planned office-warehouse structure to be located at 75542 Heacock Street, City of Moreno Valley, California. In absence of development details, the recommendations included should be considered "preliminary", subject to revision following development plan review.

Based on test explorations completed at this time it is our opinion that, in general, the soils encountered primarily consist of deposits of highly compressible, dry to damp, loose, silty gravelly sandy fills up to about 10 feet below grade, overlying deposits of variegating layers of moderately dense gravely medium to coarse sands to the maximum 31 feet depth explored. No shallow-depth groundwater or bedrock was encountered.

Based on review of the available published public documents, it is understood that the site is not situated within an A-P Special Studies Zone, and with groundwater table at a depth in excess of 50 feet, as per the California DMG Special Publication SP-117, the site is considered non-susceptible to soils liquefaction in event of a strong motion earthquake.

Based on the field and laboratory testing completed it is our opinion that with the presence of the highly compressible low SPT soils encountered up to about 8 to 10 feet should be considered unsuitable for directly supporting structural loadings without excessive differential settlements. It is our opinion that when graded as recommended herein, the structural pad thus constructed, should be adequate for the development proposed.

Respectfully submitted,
Soils Southwest, Inc.

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

Moloy Gupta, RCE 31708

John Flippin
Project Coordinator

Dist/ 1-addressee, cc: SB&O, Civil Engineers

soilssouthwest@aol.com
Established 1984

1.0 Introduction

Presented herewith are the Reports of Preliminary Soils and Foundation Evaluations and Soil Infiltration Testing for WQMP-BMP design conducted for the site of the office-warehouse structure to be located at 75542 Heacock Street at southeast intersection of Heacock Street and Ironwood Avenue, City of Moreno Valley, California.

The purpose of this evaluation is to determine the nature and engineering properties of the site soils and to provide geotechnical recommendations for foundation design, concrete slab on-grade, paving, parking, site grading, utility trench backfills and inspection during construction. The report also include necessary soil infiltration testing and recommendations for WQMP-BMP design as requested.

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 exploratory depths described.

The recommendations supplied should be considered valid when the following conditions are fulfilled:

i. *Pre-grade meeting with contractor, public agency, project civil and soils engineers,*
ii. *Continuous grading observations and excavated bottom verifications by soils engineer prior to engineered 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

No detailed development plans are prepared and none such is available for review. However, based on the preliminary information supplied, it is understood that the subject development will primarily include one industrial warehouse structure of conventional wood-frame and stucco, or concrete tilt-up construction with continuous wall and isolated spread footings with concrete slab-on-grade. Based on available preliminary drawings it is understood that an approximately 223,436 square feet warehouse structure is proposed. Supplemental construction is anticipated to include paving/parking, driveways, among others. Considering minor sloping with uneven grades, moderate site preparations and grading should be anticipated.

1.2 Site Description

The existing rectangular-shaped property is currently vacant and unimproved. In general, the site overall is bounded by Ironwood Avenue on the north, by new industrial office/warehouses on the south and east, and by Heacock Street on the west. The overall vertical relief within the property is currently unknown; however based on site reconnaissance, incidental surface runoff appears to flow towards the west and to the southeast. With the exception of scattered debris, soil stockpiles, along with an existing natural drainage ravine, no other significant features are noted.

2.0 Scope of Services

Geotechnical evaluations included review of the available publications for the site and its adjacent, along with necessary sub-surface explorations, soil sampling, necessary laboratory testing, engineering analyses and the preparation of this report. In general, our Scope of Services included the following:

o **Field Explorations**

For geotechnical evaluations, five (5) exploratory test borings (B-1 to B-5) were made using a hollow-stem auger drilling rig advanced to maximum 31 feet below existing grade. Supplemental three(3) explorations (P-1 to P-3) are made for WQMP-BMP testing advanced to depth in the range below existing grade at the approximate locations as suggested by the project design engineer. Prior to test excavations, an underground utility clearance was established with Underground Service Alert (USA) of Southern California to avoid possible subsurface life-line obstruction and rupture. Following necessary soil sampling and in-situ testing, the soil sample test excavations were backfilled with local soils using minimum compaction effort. Collected samples were subsequently transferred to our laboratory for necessary geotechnical testing. Approximate test excavation locations are shown on the attached Plate 1.

During excavations, the soils encountered were continuously logged and bulk and undisturbed samples were procured. Collected samples were subsequently transferred to our laboratory for necessary geotechnical testing. Description of the soils encountered is shown on the Test Exploration Logs in Appendix A.

o **Laboratory Testing**

Representative bulk and undisturbed site soils were tested in laboratory to aid in the soils classification and to evaluate relevant engineering properties pertaining to the project requirements. The laboratory tests completed include the following:

- In-situ moisture contents and dry density (ASTM Standard D2216),
- Maximum Dry Density and Optimum Moisture Content (ASTM Standard D1557),
- Direct Shear (ASTM Standard D3080),
- Soil consolidation (ASTM Standard D2435),
- Soils Gradation evaluations (ASTM D422*)*,
- Soils Sand Equivalent, SE (ASTM D 2419). and
- Expansion Potential Index (ASTMD4829)

No soils chemical analysis is currently included. Post-grading soil chemical analysis analyses, including pH, sulfate, chloride, and resistivity should be performed prior to actual construction and concrete pour. Reports on such will be supplied when requested. Description of the test results and test procedures used are provided in Appendix B.

o Based on the field investigation and laboratory testing completed, engineering analyses and evaluations are made on which to base our preliminary recommendations for foundations design, slab-on-grade, paving/ parking, site preparations and grading, monitoring during construction, and preparation of this report for initial use by the project design professionals.

3.0 Geotechnical Descriptions

3.1 Soils Conditions

Based on test explorations completed at this time it is our opinion that, in general, the soils encountered primarily consist of deposits of highly compressible, dry to damp, loose, silty gravelly sandy fills up to about 10 feet below grade, overlying deposits of variegating layers of moderately dense gravely medium to coarse sands to the maximum 31 feet depth explored. No shallow-depth groundwater or bedrock was encountered. Descriptions of the soils encountered are provided in the Log of Borings B-1 to B-5 and infiltration test borings P-1 to P-3, attached.

Laboratory shear tests conducted on the upper bulk samples remolded to higher density indicate moderate shear strengths under increased soil moisture conditions. Results of the laboratory shear tests are provided in Appendix B of this report.

Sandy silty in nature, the site soils are considered "very low" in expansion characteristics with Expansion Index, EI, less than 20, thereby requiring no special construction requirements other than those as described herein.

3.2 Subsurface Variations

During site preparations and grading, presence of scattered buried debris, organic and others non-structural materials may be anticipated. In addition, variations in soil strata and their continuity and orientations may be expected. Due to the nature and depositional characteristics of the fill and natural soils existing as described, care should be exercised in interpolating or extrapolating the subsurface soils conditions existing in between and beyond the test explorations conducted.

3.3 Excavatibility

It is our opinion that the grading required for the project may be accomplished using conventional heavy-duty construction equipment. Based on upper very low-density fill soils with low SPT blow counts as encountered, it is our opinion no special construction equipment should be warranted in site preparations and grading. Use of no blasting or jack-hammering should be anticipated.

3.4 Soil Corrosivity

Since change in soils chemical compositions are expected following site preparations and grading, no laboratory soil corrosivity potential evaluations are currently initiated. Following mass grading completion, results of the soil corrosivity testing, including in minimum the pH, sulfate, chloride and resistivity concentrations will be supplied on request.

3.5 Groundwater

Groundwater was not encountered within the maximum depth of 31 feet explored and none such is anticipated during grading and construction. The following table lists the historical groundwater table based on the information as supplied by the local reporting agency.

GROUNDWATER TABLE	
Reporting Agency	Water Master Support Services-San Bernardino Valley Conservation District/Western Municipal Water District Cooperative Well Measuring Program, Fall 2018
Well Number	03S/03W-06D Hemlock /Davis
Well Monitoring Agency	Eastern Water Municipal Water District
Well Location: Township/Range/Section	T3S-R3W-Section 06
Well Elevation:	1654.90
Current Depth to Water (Measured in feet)	73.20
Current Date Water was Measured	November 8, 2018
Depth to Water (Measured in feet) (Shallowest)	72.10
Date Water was Measured (Shallowest)	March 8, 2018

4.0 Faulting and Seismicity

4.1 Faulting and Seismicity

Based on the information published by the Department of Conservation, State of California, it is understood that the subject site is not situated within an A-P Special Study Zone, where a fault(s) runs through or its immediate adjacent. However, considering Southern California being in a seismically risky area, it is our opinion that while it is not economically feasible to develop a site that are totally resistant to earthquake-related hazards, implementation of the design and construction knowhow using the current CBC design procedures may benefit the development planned.

4.2 Direct or Primary Seismic Hazards

Surface ground rupture along with active fault zones and ground shaking represent primary or direct seismic hazards to structures. There are no known active or potentially active faults that pass through or towards the subject site, and the site is not situated within an AP Special Studies Zone. According to the current CBC, the site is considered within Seismic Zone 4. As a result, it is likely that moderate to severe ground shaking may be experiences for the development proposed.

4.3 Induced or Secondary Seismic Hazards

In addition to ground shaking, effects of seismic activity may include flooding, land-sliding, lateral spreading, ground settlements, and subsidence. Potential effects of such are discussed as below.

4.3.1 Flooding

Flooding hazards include tsunamis (seismic sea waves), Seiches, and failure of manmade reservoirs, tanks and aqueducts. In absence of such nearby potential for flooding is considered remote.

4.3.2 Land Sliding

Considering the subject site being near level with developed surrounding, potential for seismically induced land sliding is considered "remote".

4.3.3 Lateral Spreading

Structures or facilities proposed are expected to withstand predicted ground softening and/or predicted vertical and lateral ground spreading/displacements, to *an acceptable level of risk*. Seismically induced lateral spreading involves lateral movement of soils due to ground shaking.

The topography of the site being near level, it is our opinion that the potential for seismically induced lateral ground spreading should be considered "remote".

4.4 Site Specific Seismic Effects

The site is situated at about 3.86 miles from the San Jacinto Fault capable of generating an earthquake magnitude of M=7.5 to 7.7and PGA of 0.562g. Considering the recorded historic groundwater levels being over 50 below existing surface grade, along with the cohesive silty soils no site soils liquefaction evaluation is included and none such should be considered necessary for the project described.

4.5 Seismic Design Coefficients

Using s Site Coordinates of 33.945626°N, -117.242573W and considering the site being situated at about 3.86 miles from the San Jacinto Fault. For foundation and structural design, the following seismic parameters are suggested based on the current 2019 CBC:

Recommended values are based upon the USGS ASCE 7-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

The following presents the seismic design parameters as based on available publications as currently published by the California Geological Survey and 2019 CBC.

TABLE 4.5.1 Seismic Design Parameters

CBC Chapter 16	2019 ASCE 7-16 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 D / Seismic Coefficient, S_s	1.751 g
1613A5.3(2)	Site Class D / Seismic Coefficient, S_1	0.685 g
1613A5.3(1)	Site Class D / Seismic Coefficient, F_a	1.000 g
1613A5.3(2)	Site Class D / Seismic Coefficient, F_v	NA
16A-37 Equation	Spectral Response Accelerations, $S_{Ms} = F_a S_s$	1.751 g
16A-38 Equation	Spectral Response Accelerations, $S_{M1} = F_v S_1$	NA
16A-39 Equation	Design Spectral Response Accelerations, $S_{Ds} = 2/3 \times S_{Ms}$	1.168g
16A-40 Equation	Design Spectral Response Accelerations, $S_{D1} = 2/3 \times S_{Ms}$	NA

TABLE 4.5A.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>\=7.5-7.7
Peak Horizontal Ground Acceleration	0.562g

In design, vertical acceleration may be assumed to about 1/3 to 2/3 of the estimated horizontal ground accelerations described.

It should be noted that lateral force requirement in design by structural engineer should be intended to resist total structural collapse during an earthquake. During lifetime use of the structure built, it is our opinion that some structural damage may be anticipated requiring some structural repairs. Adequate structural design and implementation of such in construction should be strictly observed.

5.0 Evaluations and Recommendation

5.1 General Evaluations

The conclusions contained herein are base upon subsurface explorations, laboratory testing, and necessary engineering evaluations completed as described. Although no significant variations in soil conditions are anticipated, actual soils conditions may, however, vary during construction 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, 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 compressible during grading and utility installations.

(iii) Because of the near surface slightly compressible fill soils existing as described, conventional grading should be in form of subexcavations, scarification and moisturization, followed by their replacement as engineered fills compacted to 95% or better. In event new fill soils are required over the grades existing, such should be placed following the subgrade preparations as described. No footings and/or new fills should be placed directly bearing on the compressible surface soils existing.

(iv) Considering areas of fill soils encountered, it is our opinion that during grading localized deeper sub-excavations may be warranted within areas of buried debris, irrigation pipes etc. It will be the responsibility of the grading contractor to inform soils engineer of the presence of buried utilities or supplemental fills soils 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 95% 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 also 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 implementation of the seismic design requirements and the procedures as outlined in the current CBC as described earlier in Section of this report.

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

5.1.1 Preparations for Structural Pad

Considering near grade variable consistency loose and highly compressible soils existing as encountered, it is our opinion that no structural footing or concrete slab-on-grade should be established directly bearing on the near surface soils currently existing.

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 95%.

In general, for the structural pad proposed, the grading should include sub-excavations of the subgrade soils up to about (i) 8 to 10 feet below the current grade surface. Actual subexcavation depth should be determined by soils engineer during mass grading by. Supplemental detailed sub-excavation recommendations will be supplied following detailed development plan review.

The site preparations, subexcavations and grading described should encompass, in minimum, the individual planned structural foot-print areas, and minimum 8-10 feet beyond. The engineered fills for structural support should be compacted to minimum 95% of the soil Maximum Dry Density as determined by the ASTM D1557 test method.

The sub-excavation depths described should be considered as "preliminary". Localized additional sub-excavations 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.

5.2 Structural Fill

5.2.1 Structural Fill Material

Local soils free of debris, organic, roots, debris and rocks larger than 6-inch in diameter should be considered suitable for re-use as structural backfill.

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.

Backfills placed should be compacted to minimum 95% of the soil Maximum Dry Density as determined by the ASTM D1557 test method. Import soils, if required, should be gravelly sandy soils similar to the local soils or its better as approved by soils engineer.

In general, fill soils for structural support should meet the following criteria:

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

5.2.2 Structural Fill Soils Placement

During grading, structural fills should be placed in 6 to 8-inch loose lifts, at near Optimum Moisture conditions and compacted to minimum 95 percent. No fill shall be placed, spread, or compacted during unfavorable weather conditions.

5.3 Structural Foundations

5.3.1 Spread Foundations

The proposed structures may be supported by continuous wall and/or isolated spread footings founded exclusively into engineered fills of local soils compacted to minimum 95%. From geotechnical viewpoint, conventional footings may be sized to a minimum 15" wide, embedded to minimum 24" below lowest adjacent final grade. Actual foundation dimensions, however, should be determined by structural engineer based on anticipated structural loading, soil vertical bearing capacity, soil lateral pressures, and the described PGA, among others. Structural design should conform to the current CBC Seismic Design requirements as described in earlier section of this report.

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

For design, an allowable soil vertical bearing capacity of 2500 psf may be considered for the local soils when used as structural fills compacted to minimum 95%. 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 250 psf increment in foundation bearing capacity may be considered for each 1- foot increment in footing embedment up to a total not exceeding 3500 psf.

From geotechnical viewpoint, footing reinforcements consisting of 2-#4 rebar placed near the top and 2-#4 near bottom of continuous footings are suggested. Actual reinforcements as 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 assumed structural loadings of 40 kips and 4 klf for isolated and wall footings, respectively, are expected to be within tolerable limits. For static loading condition, over a span of 40 ft, estimated total and differential settlements are estimated to about 1 and 1/2-inch, respectively.

5.3.2 Concrete Slab-on-Grade for Industrial Use

The prepared subgrades compacted to minimum 95% prepared to receive footings should be adequate for concrete slab-on-grade placement. For industrial use, 6 to 8-inch thick (net) concrete slab-on-grade may be considered, reinforced with #4 rebar at 24" on-center, 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. Alternative reinforcement using "fiber mesh" may be considered entirely at the discretion of the addressee.

5.3.3 Concrete Driveways

For loaded track traffic, with estimated Traffic Index, TI, of 7.0, concrete driveways should consist of minimum 6-inch thick concrete, placed over 4-inch Class II or CMB base compacted to 95%, overlying graded local soils similarly compacted to at least 95%. Actual concrete slab-on-grade thickness should be determined by the project structural engineer considering a soil Modular of Subgrade Reaction, k_s, of 450 kcf. Driveway slab reinforcing and construction/expansion joints etc. should be incorporated as recommended by the project structural engineer. Use of thicker driveway edges are strongly suggested. Suggested minimum reinforcing requirement is #4 rebar at 24" on-center. Alternative reinforcing use may be considered entirely at the discretion of the addressee.

Subgrades to receive concrete should be "pre-moistened" as would be expected in any such concrete placement. 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.

5.3.4 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 THAT CONCRETE SLABS ARE PROPERLY "CURED".
3. DO NOT POUR CONCRETE WHEN OUTSIDE TEMPERATURE EXCEEDS 90° F OR 80° F WHEN THE WIND EXCEEDS 12 MPH. 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 the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur. For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Shorter distance between joint spacing would provide greater crack control. Joints at curves and angle points are suggested, as recommended by structural engineer.

5.3.5 Structural Asphalt Concrete Pavement Thickness

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	7.0	a.c. over CL. II base	6.0 over 4.0

Within paving areas, subgrade soils should be scarified moisture conditioned from 3% to 5% percent 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 base material recommended should also be compacted to minimum 95%.

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

5.4 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 0.30 may be assumed with normal dead load forces for footing established on compacted fill.

An allowable passive lateral earth resistance of 250 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 2500 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:	60 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.

5.5 Shrinkage and Subsidence

Based on the results of field observations and laboratory testing, it is our opinion that the upper soils when used graded may be subjected to a volume change. Assuming a 95% 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 15 percent or more. Further volume change may be expected following removal of buried utilities and debris, if any. 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.

5.6 Construction Consideration

5.6.1 Unsupported Excavation

Temporary construction excavation up to a depth of 4 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.

5.6.2 Supported Excavations

If vertical excavations exceeding 4 feet in depths become warranted such should be achieved using shoring to support side walls.

5.7 Site Preparations

The site preparation should include sub-excavation of the upper loose and compressible soils varying in depth of 8 to 10 feet, stockpiling, moisturization to near Optimum Moisture content. Site preparations should also include replacement of the excavated soils and other approved imported fills, if any, in 6 to 8-inch thick, compacted to the minimum 95% relative compaction. Such earth work should be in accordance with the applicable grading recommendations provided in the current CBC and as recommended in Section 5.0 of this report.

5.8 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.

Concrete Paving, if considered, should be at least 6-inch thick reinforced with #5 rebar at 18" 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.

5.9 Utility Trench Backfill

In absence of precise grading and development plan review, it is our opinion that utility trench backfills within proposed structural pad 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

5.9.1 Utilities

Considering seismically susceptible ground shaking, use of commercially available flexible connections for utilities and life-line services are 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.

5.10 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.

5.11 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.

5.12 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.

5.13 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.

5.14 Observations and Testing During Construction

Recommendations provided are based on the assumption that structural footings and slab-on-grade be established exclusively into compacted fills. Excavated footings should be inspected, verified and certified by soils engineer prior to steel and concrete placement to ensure their sufficient embedment and proper bearing as recommended. 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.

5.15 Plan Review

Based on the site plan supplied, the recommendations supplied should be considered 'preliminary'. It is recommended that grading and development plans should be reviewed when prepared in order verify adequacy of the geotechnical recommendations supplied. Supplemental recommendations may be warranted following grading plan review.

6.0 Earth Work/General Grading Recommendations

Site preparations and grading should involve over-excavation and replacement of local soils as structural fill compacted to 95% 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 95% 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 95% 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 compaction methods. All in-situ field density of compacted fill shall be determined by the ASTM D1556 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.

7.0 WQMP-BMP Stormwater Disposal Design
Water Infiltration Rate Using Porchet Method

Presented herewith are the preliminary results of soils infiltration testing performed for the planned storm water disposal design system proposed for the project site described. Considering the relatively homogenous silty sand during preliminary site explorations, no known changes are anticipated during site grading, however test results should be considered tentative given the potential for changes to site finish grade(s) or changes in soil conditions during grading.

Three (3) infiltration tests were performed at about 10 to 12 feet below the current grades as suggested by the project civil engineer within the approximate location of the proposed underground stormwater chamber as supplied by the project engineering proposed site plan. Although tested, the original P-1 test is omitted from the analyses and reporting since soil conditions varied greatly compared to the other two infiltration test borings and adjacent soil sample borings where very coarse gravely sandy conditions resulted in excessive infiltration test results more than doubled that of P-2 and P-3.

Tests were performed using the standardized "falling-head" test converted using the Porchet Method to infiltration rate as per the guidelines in accordance with the Table 1, Infiltration Basin Option 2 of Appendix A of the Riverside County-Low Impact Development (LID) BMP design Handbook/ Approximate test locations are shown on Plate 1, attached.

The soils encountered consist in general of upper fine to medium course silty sands to the maximum 12 feet depth explored and proposed chamber bottom (P-2 and P-3). For the purposes of determining the presence/or lack of presence of groundwater or any impermeable soils, soils encountered below twelve (12) feet to maximum depth of thirty one (31) feet consists, in general of, silty fine fill and local sands overlying variegating layers of silty and slightly clayey sands and fine to medium coarse gravely sands with pebbles and rock fragments, test boring (B-1),

No free groundwater was encountered. Descriptions of the soils encountered are provided in the Log of Borings, P-1 to P-3 attached.

Based on the field infiltration testing completed, it is our opinion that for the infiltration system design proposed at between 10 to 12 feet below grade, the observed soils infiltration rates are 5.2 and 5.4 in/hr.

For design, it is suggested that, use of an appropriate factor of safety as determined by the design engineer should considered to the observed rate to account for long-term saturation, inconsistencies in subsoil conditions, potential for silting and lack of maintenance. The observed soils percolation rates are provided in Table7.4.1 in Section 7.4 of this report.

7.1 EXCAVATED TEST BORINGS

For BMP soil infiltration testing at the location as shown on the accompanying Plate 1, three (3) tests borings (P-1 to P-3) were made using a 8-inch diameter hollow-stem auger drilling rig, advanced to approximately 10 and 12 feet below the current grade as suggested the project engineer. Water used during infiltration percolation testing was supplied by using water jugs and a water tank.

7.2 METHODOLOGY AND TEST PROCEDURES:

EQUIPMENT SET-UP (POST EXCAVATION) PROCEDURES

Following test boring completion, each of the test holes were fitted with perforated pvc pipes backfilled with 2-inch thick crushed rock at the bottom to minimize potentials for scouring and caving. For testing, each test hole was initially filled using water supplied by water jugs.

Prior to actual testing, in order to determine test intervals, as per the Section 2.3 for deep percolation testing of the referenced handbook guideline, one to two consecutive readings were performed to determine if six (6) or more inches of water seeped in 25 minutes. Since 6 inches or more of water seeped away in less than 25 minutes for both P-2 and P-3, subsequent percolation testing were performed at 10-minute time intervals for at least the minimum six hours or until the rates were consistent.

Testing included water placement at about 7-10 feet below existing grade surface (inlet depth or 24 inches above infiltration system bottom).

The final 10-minute recorded percolation test rates were converted into an Infiltration Rate (I_t) for inches per hour using the "Porchet Method" equation as described in the Reference 2, Riverside County Low Impact Development BMP Design Handbook.

7.3 INFILTRATION TEST RESULT

Based on the soils infiltration testing completed at the test locations and at the test depth as described, the observed soil percolation rates are 5.2"/hr and 5.4"/hr for the test locations P-2 and P-3 respectively.

Calculations to convert the percolation test rate to infiltration test rates in accordance with Section 2.3 of the County Handbook are presented in Table I and II below. For design, it is suggested that, use of a factor of safety of 2.0 to 3.0, or an appropriate Factor of Safety as selected by the design engineer should be considered to the observed field percolation rate described.

7.3.1. Conversion Calculations & Summary:

TABLE I

Conversion Table (Porchet Method)

Test No.	*Depth Test Hole (inches)*	*Time Interval*	*Initial Depth (inch)*	*Final Depth (inch)*	*Initial Water Height (inch)*	*Final Water Height (inch)*	*Change Height/ Time*	*Average Head Height/Time*
	D_T	$\Delta_{T\,(Min)}$	$D_{O\,(in)}$	$D_{f\,(in)}$	$H_o=D_t-D_o$	$H_f=D_t-D_f$	$\Delta H= H_f-H_O$	$H_{avg} = (H_o+H_f)/2$
P-1	~~120~~	~~10~~	~~86~~	~~110.0~~	~~34.0~~	~~10.0~~	~~24.0~~	~~22.00~~
P-2	144	10	120	125.5	24.0	18.5	5.5	21.25
P-3	110	10	86	95.5	24.0	14.5	9.5	19.25

	Infiltration Rate (It)=ΔH60r/Δt(r+2Havg)		
	A	B	C
Test No.	ΔH60r	Δt(r+2Havg)	A/B=in/hr
P-1	~~5760~~	~~480~~	~~12.0~~
P-2	1320	252.5	5.23
P-3	2280	425.0	5.36

TABLE II

For WQMP-BMP design, based on the soils infiltration testing completed and, on the calculations as described, the following infiltration rates may be considered. Actual field test data are attached.

Observed Infiltration Rate for Design

Test Date Test No. (7-9-2020)	Relative Site Location	Test Depth (ft.) Below Grade	Observed Rate (inch/hour) Porchet Method
~~P-1~~	~~South~~	~~10.0~~	~~12.0~~
P-2	North	12.0	5.23
P-3	West	10.0	5.36

Use of safety factor should be considered to account for long-term saturation, inconsistencies in subsoil conditions, along with the potential for silting of percolating soils.

The infiltration rate described is based on the in-situ testing completed at the locations as suggested by the project civil engineer. In event the final chamber location and depth vary considerably from those as described herein, supplemental soils infiltration testing may be warranted.

It should be noted that over prolong use and lack of maintenance the detention/infiltration basins or deep chambers constructed based on the suggested design rate may experience much lower infiltration rate due to the accumulation of silts, fines, oils and others. Regular maintenance of the chambers in form of removal of debris, oil and fines are strongly recommended. A maintenance record of such is suggested for future use, if any.

<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 its adjacent.

Stormwater infiltration shall not place an increased surcharge on structures or foundations on or its adjacent. 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.

Once installed, regular maintenance of the detention basin is recommended.

8.0 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.

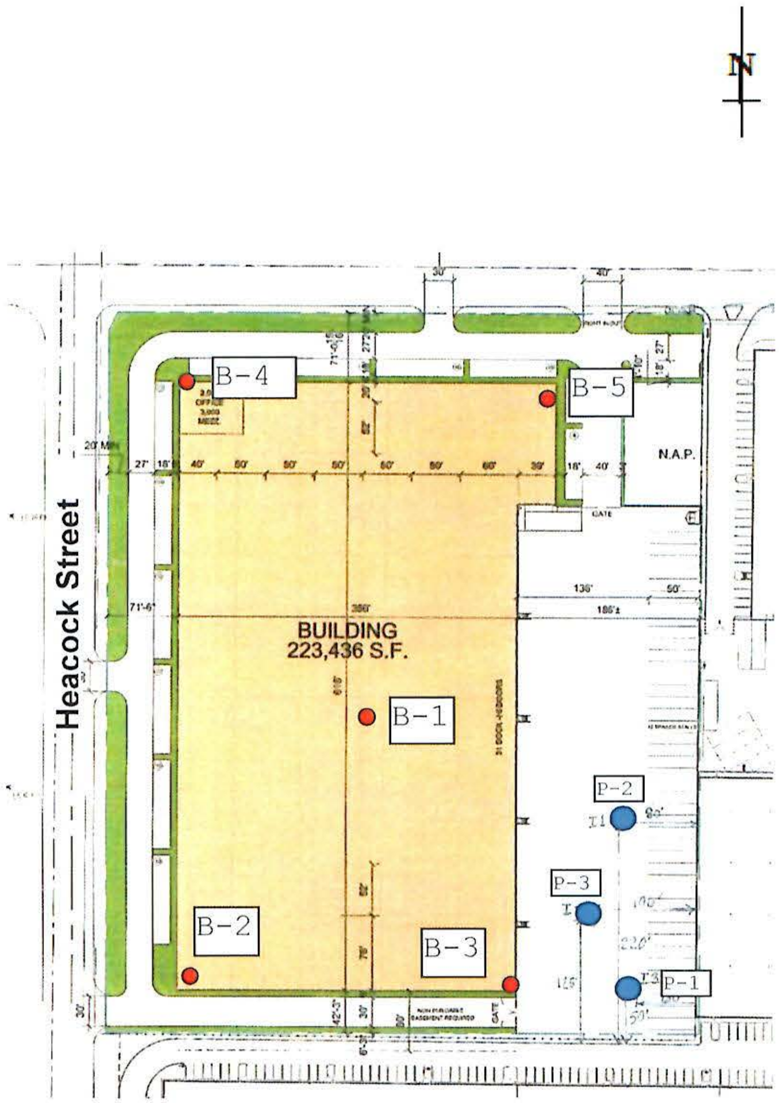
Should the project be delayed beyond one year after the date of this report; the recommendations presented shall be reviewed to consider any possible change in site conditions.

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
Proposed Office/Warehouse Structure
Heacock Industrial
75542 Heacock Street @ Ironwood Avenue
City of Moreno Valley, California

(Not to Scale)



Legend: B-1 Approximate Location of Test Borings
P-1 Approximate Location of Infiltration Test Boring

Plate 1

8.0 APPENDIX A

Field Explorations

Field evaluations included site reconnaissance and five (5) exploratory soil sample test borings and three (3) infiltration test borings using a truck mounted hollow-stem auger drill-rig. 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 BORINGS
&
INFILTRATION FIELD DATA

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

LOG OF BORING P-1

Project: Heacock Industrial **Job No.:** 20022-F/BMP

Logged By: John F. **Boring Diam.:** 8"HSA **Date:** June 26,2020

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-SC			disturbed rough graded soils SAND-dark gray brown, slightly clayey, fine to medium, pebble, damp to moist
					SP		5	- color change to dark brown, fine to medium coarse, pebbles and rock fragments damp
					GP-SP			- color change to light brown, gravely, rock framents, occasional rock, medium to coarse, dry
							10	- End of infiltration test boring @ 10.0 ft. - no bedrock - no groundwwater - 3" PVC pipe installed with gravel at bottom
							15	
							20	
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

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

LOG OF BORING P-2

Project: Heacock Industrial		**Job No.:**	20022-F/BMP
Logged By: John F.	**Boring Diam.:** 8"HSA	**Date:**	June 26,2020

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			weeds
								SAND - brown, silty, fine, scattered pebbles and rock fragments
							5	
					SM			- color change to light brown, silty, fine to medium, pebbles, dry to damp
							10	
								- End of infiltration test boring @ 12 ft. - no bedrock - no groundwater - 3" PVC pipe installed with gravel at bottom
							15	
							20	
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

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

LOG OF BORING P-3

Project: Heacock Industrial		**Job No.:**	20022-F/BMP
Logged By: John F.	**Boring Diam.:** 8"HSA	**Date:**	June 26,2020

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			weeds
								SAND - dark gray-brown, silty, fine to medium coarse, pebble, rock fragments damp
							5	
							10	- color change to light brown, silty, fine to medium, pebble, rock fragments, dry
								- End of infiltration test boring @ 10.0 ft. - no bedrock - no groundwater - 3" PVC pipe installed with gravel at bottom
							15	
							20	
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

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

LOG OF BORING B-1

Project: Heacock Industrial		**Job No.:** 20022-F/BMP	
Logged By: John F.	**Boring Diam.:** 8"HSA	**Date:** June 26,2020	

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
					FILL			weeds
		5.3	98.2	76.6			5	SAND - light brown to brown, silty, fine, occasional pebbles, scattered rock fragments, damp to moist, very loose/soft, scattered debris (glass shards) - fine to medium, dry, loose - (Max Dry Density = 128 pcf @ 8.5 %)
					SM-ML		10	- color change to light brown, silty, fine, pebbles, loose/medium stiff
6					SP		15	- color change to light gray-brown, gravely, medium to coarse, pebbles, rock fragments very loose to loose - color change to dark gray-brown, traces of silt, gravely, fine to coarse, pebbles rock fragments, damp
35					SM		20	- color change to light brown, silty, fine to medium coarse, pebbles, rock fragments scattered rock 1/2", damp, dense
					SM-SC		25	- color change to gray-brown, silty, clayey fine to medium, occasional pebble, damp, very stiff
32					SM		30	- color change to light brown, silty, fine to medium coarse, pebbles, rock fragments - dense
								- End of test boring @ 31.0 ft. - no bedrock - no groundwater

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

California sampler   Bulk/Grab sample   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-2

Project: Heacock Industrial			**Job No.:** 20022-F/BMP
Logged By: John F.	**Boring Diam.:** 8"HSA		**Date:** June 26,2020

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
					FILL			weeds
								SAND (fill)- light yellow brown, silty, fine, pebble, dry
13					SM			- color change to grayish light brown, silty, fine to medium, traces of clay, pebbles, rock fragments, dry
		7.6	117.0	91.4			5	- color change to light yellow orangish brown, silty, fine to medium, pebble, rock fragments, dry, loose
4					GP-SP			- gravely, medium coarse to coarse, pebbles rock fragments, very dry
							10	
							15	
		4.2	110.4	86.3	SM			- color change from light brown to orangish brown, silty, fine to medium, pebbles, some rock fragments
								- End of test boring @ 16.0 ft. - no bedrock - no groundwater
							20	
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

California sampler Bulk/Grab sample 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-3

Project: Heacock Industrial **Job No.:** 20022-F/BMP
Logged By: John F. **Boring Diam.:** 8"HSA **Date:** June 26,2020

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
					FILL			weeds
								SAND (fill) - light brown to brown, silty, fine, pebble, dry to damp
4	Standard penetration test				SM-ML		5	- color change to grayish light brown, silty fine, pebbles, rock fragments, scattered rock 1/2"
					SM-SC			- color change to gray-brown, silty, slightly clayey, fine to medium, pebbles rock fragments, damp to moist, very loose medium stiff
	California sampler	5.3	98.0	76	SM			
3	Standard penetration test						10	- color change to light brown, silty, fine to medium, pebbles, rock fragments, dry to damp - very loose/medium stiff
					SM-SC		15	- color change to dark gray, brown, silty, slightly clayey, pebbles, rock fragments
							20	- End of test boring @ 20.0 ft. - no bedrock - no groundwater
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

Standard penetration test California sampler

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

LOG OF BORING B-4

Project: Heacock Industrial		**Job No.:**	20022-F/BMP
Logged By: John F.	**Boring Diam.:** 8"HSA	**Date:**	June 26,2020

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			weeds SAND - brown, silty, fine to medium, pebbles rock fragments, dry to damp, very loose
5					SM		5	- color change to gray-brown, silty, fine, damp, very loose/medium stiff
4								- very loose/medium stiff
6					SM-ML			- color change to light brown, silty, fine pebbles, scattered rock fragments, dry to damp
		6.5	118.8	92.8	SM		10	- silty, fine to medium, pebbles, occasional rock fragments. dense
23					SM-SC		15	- color change to reddish brown, clayey, silty, fine to medium coarse, pebble, rock fragments, damp to moist, medium dense
							20	- End of test boring @ 16.0 Ft. - no bedrock - no groundwater
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

California sampler
Bulk/Grab sample
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-5

Project: Heacock Industrial		Job No.: 20022-F/BMP
Logged By: John F.	Boring Diam.: 8"HSA	Date: June 26,2020

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
					FILL			weeds
								SAND - brown, silty, fine, pebbles
	California sampler	7.1	118.5	92.6				- scattered debris (concrete)
							5	- color change to light brown, silty, fine to medium, pebbles, rock fragments
					SM-ML			- color change to gray-brown, silty, fine pebbles, damp
							10	
6	Standard penetration test				SM			- color change to light brown, silty, traces of clay, gravely, fine to medium coarse, pebbles, rock fragments, damp to moist, medium dense to dense
					SM-SC			- color change to reddish gray-brown, lumpy clayey, silty, fine to medium, pebbles rock fragments, damp to moist, loose to medium stiff
							15	
							20	
25	Standard penetration test							- medium dense/stiff
								- End of test boring @ 21.0 ft. - no bedrock - no groundwater
							25	
							30	

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

Site Location

Proposed Industrial Building
75542 Heacock Street
Moreno Valley, California

Plate #

California sampler | Bulk/Grab sample | Standard penetration test

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Poorly graded sand with clay
	Poorly graded sand
	Poorly graded gravel and sand
	Poorly graded silty fine sand
	Silty sand
	Poorly graded clayey silty sand
	Fill
Soil Samplers	
	California sampler
	Bulk/Grab sample
	Standard penetration test

Notes:

1. Exploratory borings were drilled on June 26,2020 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.

Percolation Test Data Sheet

7-9-20

Project:	HEACOCK/INDUSTRIAL	Project No:	20022-BMP	Date:	6-26-20
Test Hole No:	P-1	Tested By:	A.O.		
Depth of Test Hole, D_T:	120	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	11:45	12:10	25	86	118	32	Y
2	12:22	12:47	25	86	116	30	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	1:38	1:48	10	86	112.5	26.5	
2	1:51	2:01	10	86	111.75	25.75	
3	2:04	2:14	10	86	111	25	
4	2:16	2:26	10	86	111	25½	
5	2:29	2:39	10	86	111	25	
6	2:40	2:50	10	86	110	24	
7	2:51	3:01	10	86	110	24	
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

7-9-20

Project:	HEACOCK / INDUSTRIAL		Project No:	20022-BMP		Date:	~~6-26-20~~
Test Hole No:		P-2	Tested By:				
Depth of Test Hole, D_T:		144	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	8:58	9:23	25	120	139.5	19.5	Y
2	9:26	9:51	25	120	136.25	16.25	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	10:02	10:12	10	120	128	8	
2	10:15	10:25	10	120	126 3/4	6 3/4	
3	10:27	10:37	10	120	126 1/2	6 1/2	
4	10:39	10:49	10	120	126 1/2	6 1/2	
5	10:51	11:01	10	120	126	6	
6	11:03	11:13	10	120	125 1/2	5 1/2	
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	HEACOCK / INDUSTRIAL	Project No:	20022-BMP		Date:	6-26-20	
Test Hole No:		P-3	Tested By:				
Depth of Test Hole, D_T:		110	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	9:18	9:43	25	86	105.5	19.5	Y
2	9:46	10:11	25	86	104.75	18.75	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	10:20	10:30	10	86	95 1/2	9 1/2	
2	10:33	10:43	10	86	95 1/2	9 1/2	
3	10:46	10:56	10	86	95 1/2	9 1/2	
4	10:58	11:08	10	86	95 1/2	9 1/2	
5	11:10	11:20	10	86	95 1/2	9 1/2	
6	11:22	11:32	10	86	95 1/2	9 1/2	
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

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 (D2937):

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.

Laboratory Test Results

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

Test Boring No.	Sample Depth, ft.	Dry Density, pcf.	Moisture Content, %
1	3	98.2	5.3
2	5	117.0	7.6
2	15	110.4	4.2
3	7	98.0	5.3
4	10	118.8	6.5
5	3	118.5	7.1

Table II: Max. Density/Optimum Moisture Content (ASTM D1557-91)

Sample Location, @ Depth, ft.	Max. Dry Density, pcf	Opt. Moisture (%)
B-1 @ 3-5 Sand-lt.brn fills, silty, traces of clay, fine, pebbles, occasional rock fragments and rock 1", occasional bits of asphalt and concrete	128.0	8.5

Table III: Direct Shear (ASTM D3080)

Test Boring & Sample Depth (ft)	Test Condition	Cohesion (PSF)	Friction (Degree)
B-1 @ 3-5	Remolded to 95%	415	37

Table IV: Consolidation (D2435)

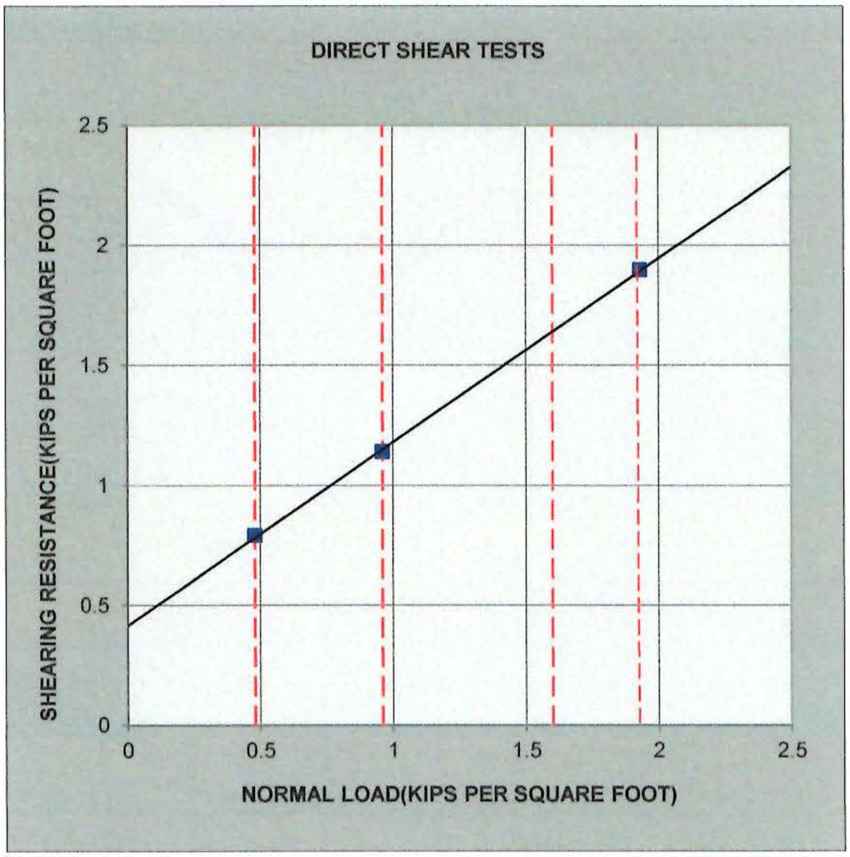
Boring B #	Depth (ft.)	Consolidation prior to saturation (%) @ 2 kips	Hydro collapse (%) @ 2 kips	Total Consolidation (%@ 8 kips) (saturated)
1 (remolded)	3-5	0.4	0.2	2.7
2 (undisturbed)	5	0.7	7.6	14.1
3 (undisturbed)	7	0.5	5.5	9.2

E. Table V: Sand Equivalent, SE (ASTM D2419)

Sample Location @ depth, ft.	Sand Equivalent Average
B-1 @ 3-5	12.57

F. Table VI: Soils Expansion Index, EI. (ASTM D4829)

Sample Location & Soils Type	Soil Expansion Index, EI	Expansion Potential
B-1 @ 3-5' Sand-lt.brn fills, silty, traces of clay, fine, pebbles, occasional rock fragments and rock 1", occasional bits of asphalt and concrete	19.75	"very low to low"

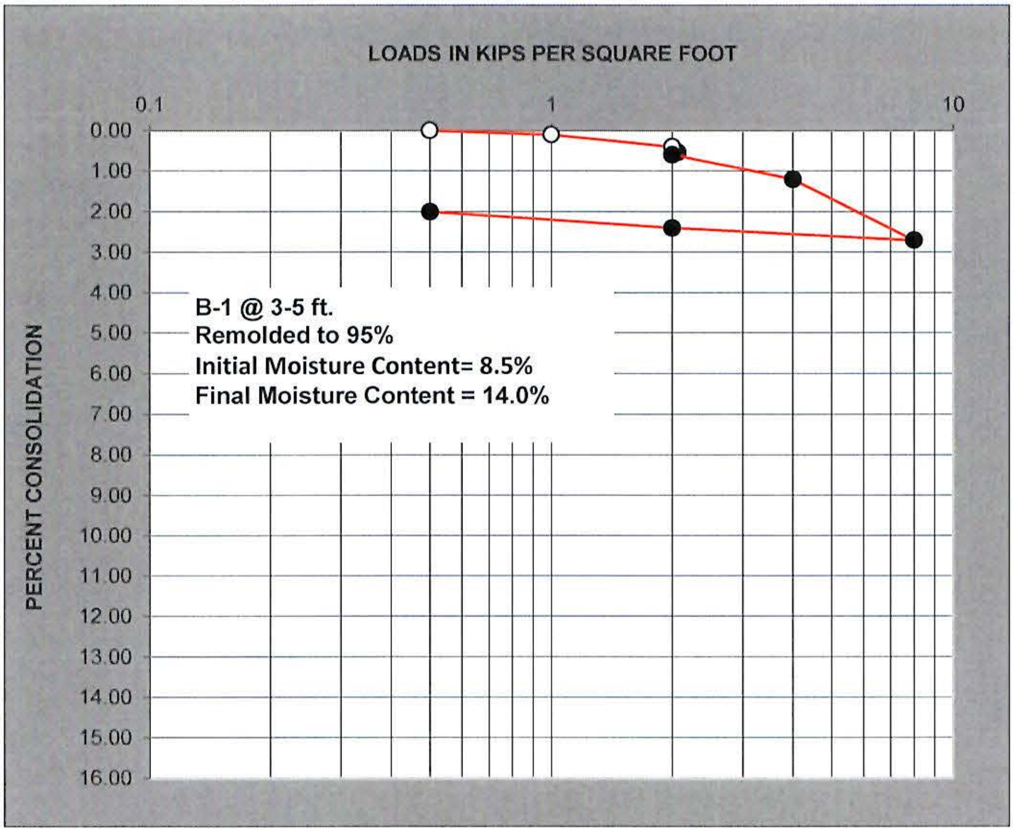


SYMBOL	LOCATION	DEPTH (FT)	TEST CONDITION	COHESION (psf)	FRICTION (degree)
■	B-1	3 to 5	Remolded to 95%	415.54	37.44

Proposed Industrial Warehouse 75542 Heacock St. @ Ironwood Avenue Moreno Valley, California	PROJECT NO.	20022-F
	PLATE	B-1

SOILS SOUTHWEST, INC.
Consulting Foundation Engineers

CONSOLIDATION TESTS

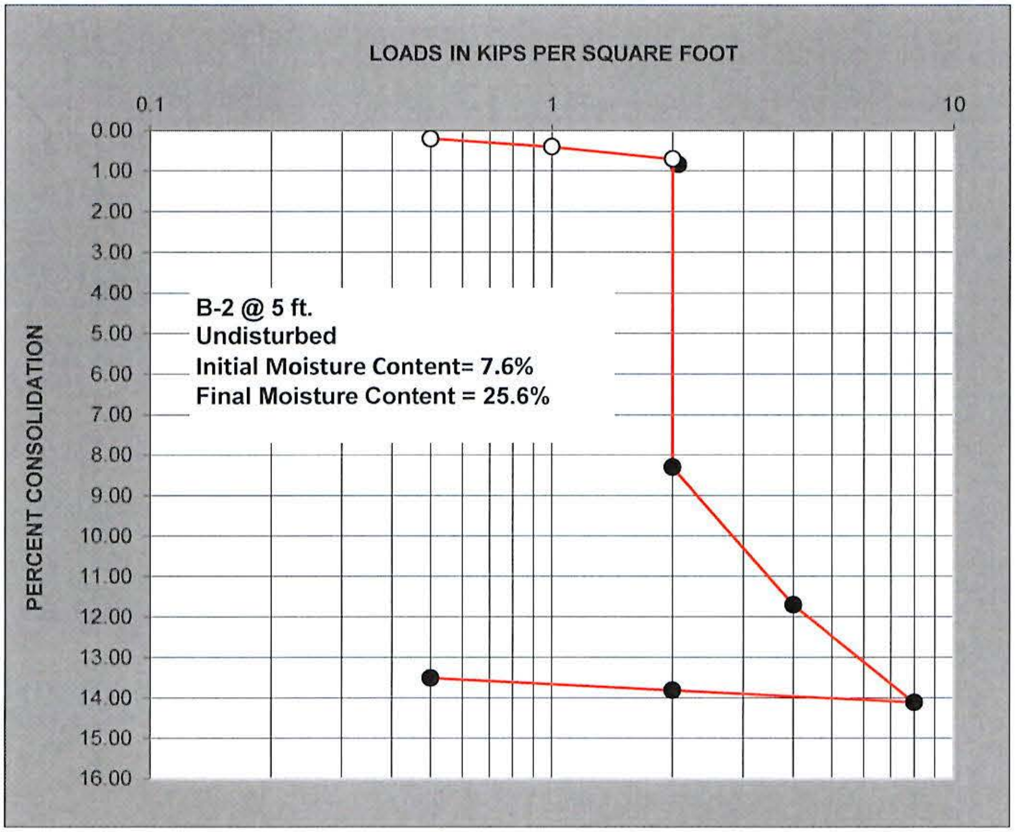


● **WATER PERMITTED TO CONTACT SAMPLE**

	PROJECT	Proposed Industrial Warehouse 75542 Heacock Street @ Ironwood Ave., Moreno Valley		
	PROJECT NO.	20022-F	PLATE	B-2

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

CONSOLIDATION TESTS

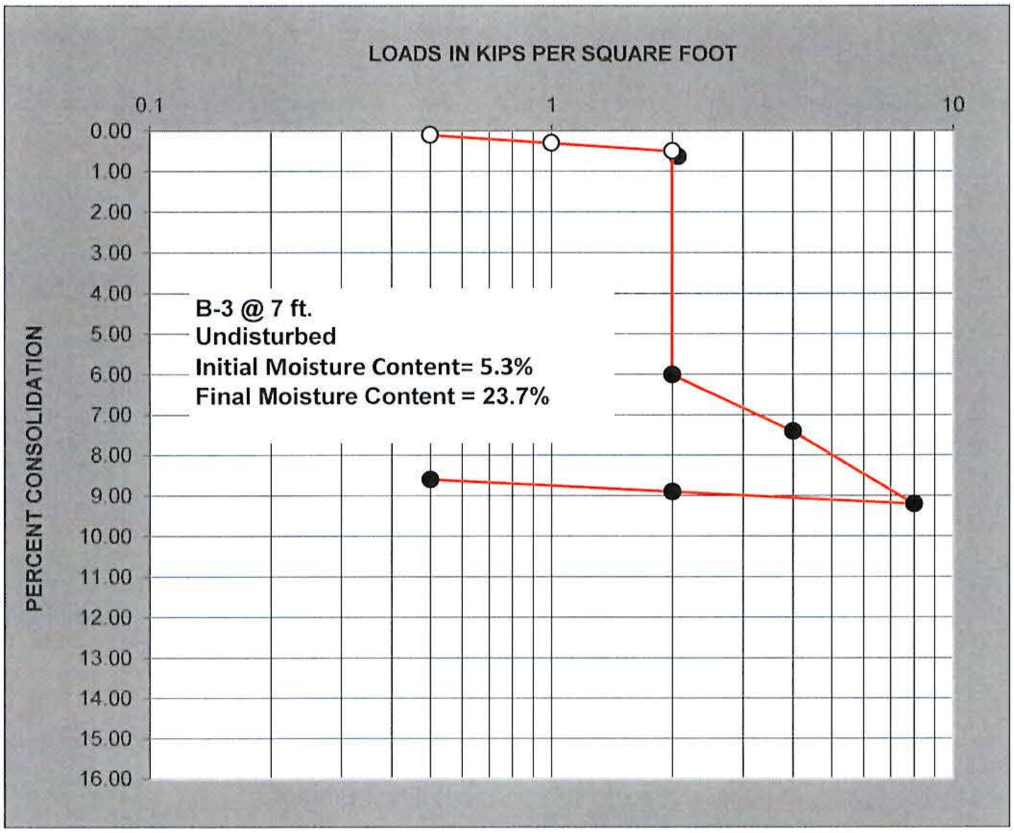


● **WATER PERMITTED TO CONTACT SAMPLE**

PROJECT	Proposed Industrial Warehouse 75542 Heacock Street @ Ironwood Ave., Moreno Valley			
PROJECT NO.	20022-F	PLATE	B-2-1	

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

CONSOLIDATION TESTS



● **WATER PERMITTED TO CONTACT SAMPLE**

PROJECT	Proposed Industrial Warehouse 75542 Heacock Street @ Ironwood Ave., Moreno Valley			
PROJECT NO.	20022-F	PLATE	B-2-2	

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

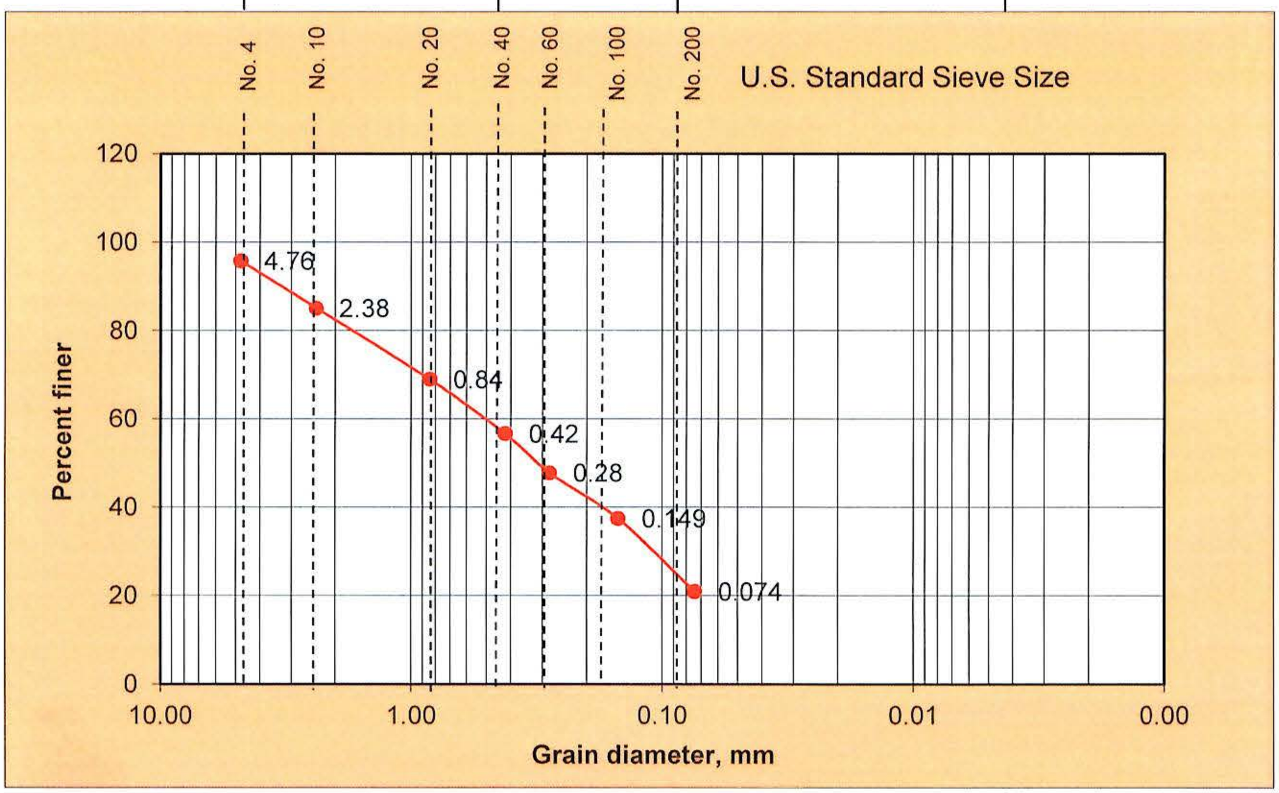
GRAIN SIZE DISTRIBUTION

Project: Heacock Industrial **Job #** 20022-F
Location: Heacock Ave, Moreno Valley **Boring No:** B-1 @ 3-5' **Sample No:** 1
Description of Soil: Sandy, brown, silty, rock frags up to 1/2" with some pebbles
Date of Sample: 7/9/2020
Tested By: Alex **Date of Testing:** 7/27/2020

Sieve No.	Sieve Openings in mm	Percent Finer
4	4.76	95.88
10	2.38	85.12
20	0.84	69.06
40	0.42	56.72
60	0.28	47.82
100	0.149	37.52
200	0.074	21.00

Grain Size	% Retained
Gravel	4
Med. to Crs	39
Fines	32
Silts	25
Clays	0

Gravel	Sand		Silt	Clay
	Coarse to Medium	Fine		



Visual Soil Description : Sand - fine to medium coarse

Soil Classification: SM

System: USC

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

Expansion Index

ASTM D 4829

Machine No:	1	**Project Name:**	Heacock Industrial
Project No:	20022-F	**Lot/Boring/Trench:**	B-1 @ 3-5'
Depth (ft):	3 to 5	**Tract No:**	
Location: 75542 Heacock St. Moreno Valley		**Technician:**	JF
Date:	7/14/2020		

TEST DATA		Load: 144 lb	Ring = 1" x 4"
	Dial Reading	Time (h:m)	Date
Dry / 10 min	0	11:05	7/14/2020
Inundate	0	11:15	7/14/2020
Reading	4	11:14	7/14/2020
Reading	8	11:40	7/14/2020
Reading	13	1:30	7/14/2020
EI (measured)	16	9:45	7/16/2020

DEGREE OF SATURATION DATA	Test A	Test B
A. Initial Moisture Content (%)	9.61%	0.00%
B. Weight of wet soil + Ring (g)	610.80	0.00
C. Weight of Ring (g)	188.70	0.00
D. Weight of Wet Soil (g) (B-C)	422.10	0.00
E. Weight of Dry Soil (g) (D/(1 + A))	385.09	0.00
F. Wet Density (pcf) D g/cubic cm/207 cubic cm convert to pcf (x 62.4) (1gram/cubic cm = 62.4 lbs cubic foot)	127.24	0.00
G. Dry Density (pcf) E g/cubic cm/207 cubic cm convert to pcf (x 62.4)	116.09	0.00
H. Weight of Water (pcf) (A x G)	11.16	0.00
I. Volume of Solids (cubic ft) (G/(2.7 sp. Gravity x 62.4))	0.69	0.00
J. Volume of Voids (cubic ft) (1-I)	0.31	1.00
Degree of saturation (%) Volume of water/volume of void x 100 H/62.4/J (%)	57.49	0.00

Expansion Potential			
	Test A	Test B	
0 - 20	↔	N/A	VERY LOW
21 - 50	N/A	N/A	LOW
51 - 90	N/A	N/A	MEDIUM
91 - 130	N/A	N/A	HIGH
>130	N/A	N/A	VERY HIGH

FINAL RESULTS		
Expansion Index (EI60) (A)	19.75	Final Moisture Content (%) 15.29
Expansion Index (EI60) (B)		← Note: Disregard Test (B) if Degree of Saturation is 0.0

CORRECTION FOR DEGREE OF SATURATION

EI60 = EI measured - (50-S measured) x ((65 + EI measured) / (220 - S measured))

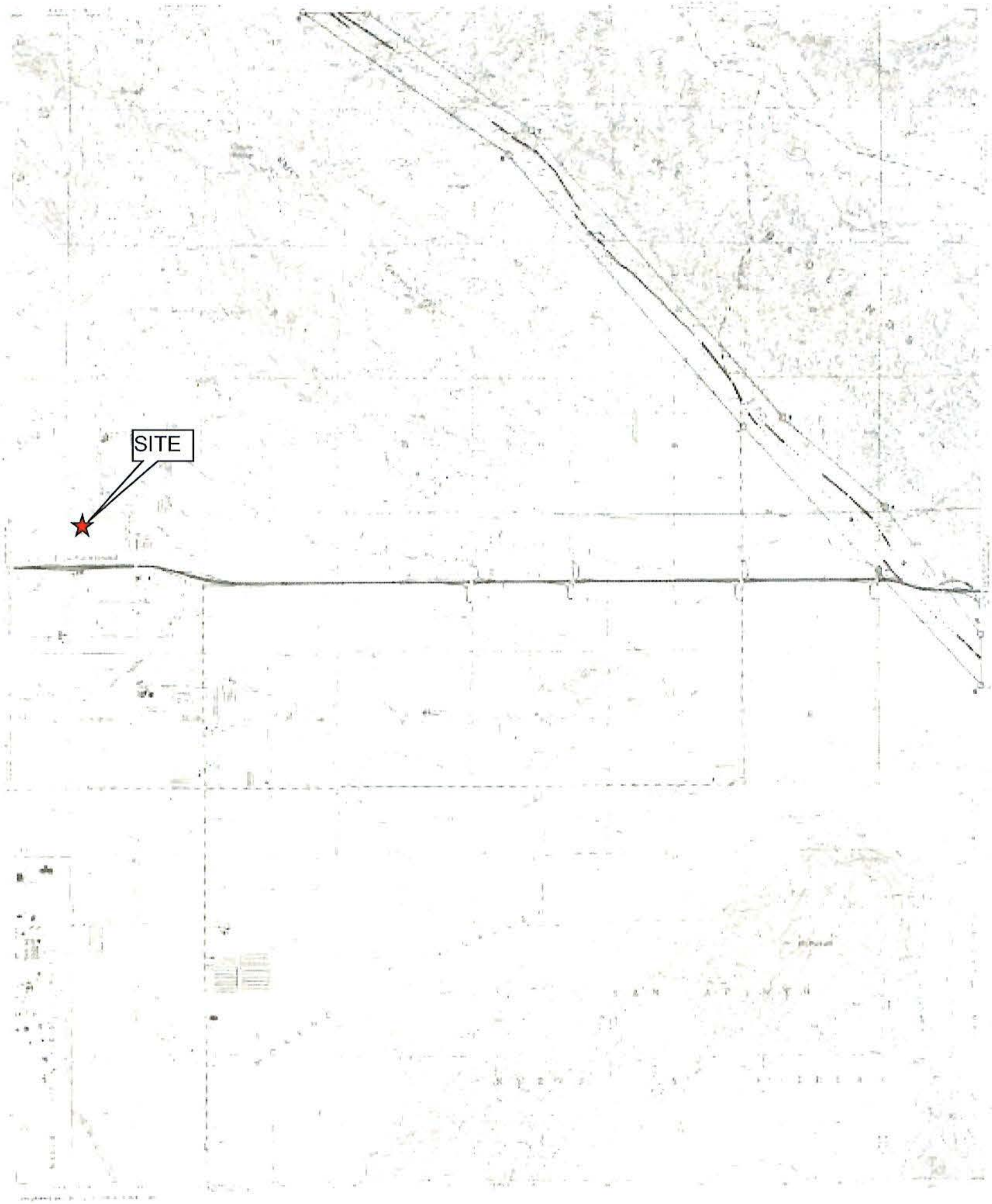
Soils Southwest, Inc

APPENDIX C

Supplemental Seismic Design Parameters

CALIFORNIA DIVISION OF MINES AND GEOLOGY
JAMES E. SLOSSON, STATE GEOLOGIST

THE RESOURCES AGENCY
DEPARTMENT OF CONSERVATION



MAP EXPLANATION

Potentially Active Faults

[illegible]

[illegible]

Special Studies Zone Boundaries

[illegible]

[illegible]

SCALE 1:24 000

STATE OF CALIFORNIA

SPECIAL STUDIES ZONES

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

SUNNYMEAD QUADRANGLE

OFFICIAL MAP

Effective: July 1, 1974

State Geologist

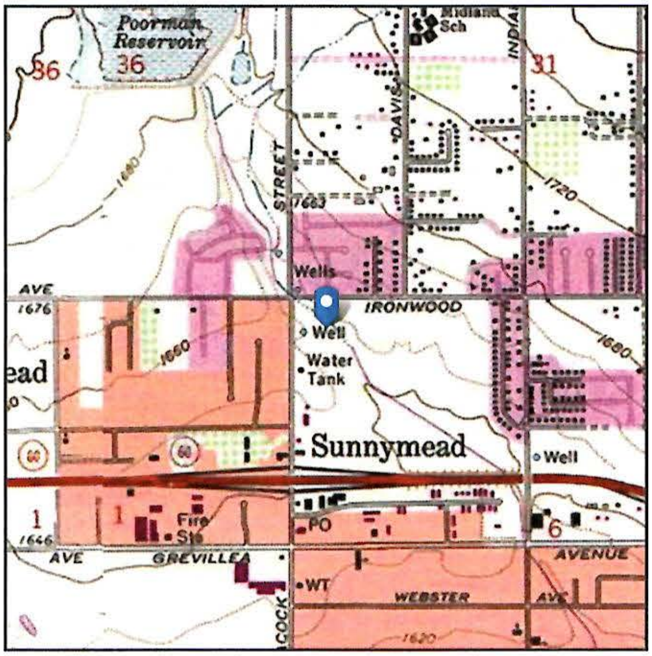
IMPORTANT - PLEASE NOTE

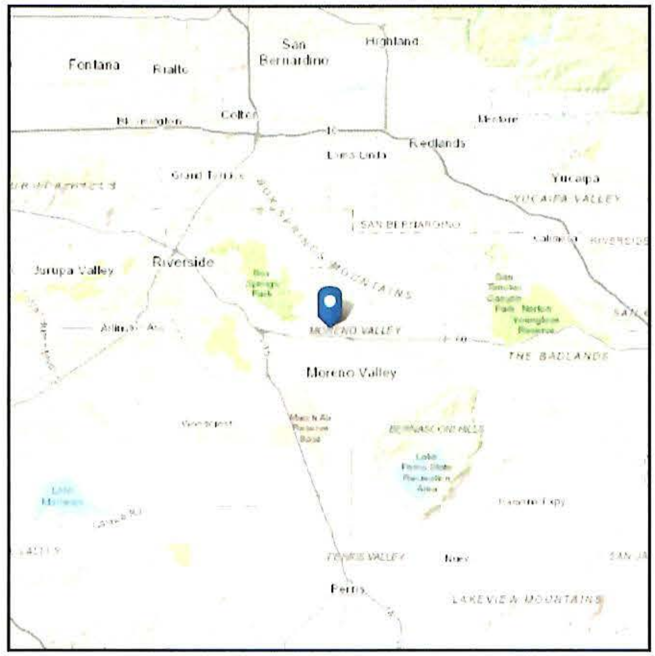
1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]

[illegible]

ASCE 7 Hazards Report

Address:	**Standard:**	ASCE/SEI 7-16	**Elevation:**	1651.21 ft (NAVD 88)
No Address at This Location	**Risk Category:**	III	**Latitude:**	33.945626
	Soil Class:	D - Stiff Soil	**Longitude:**	-117.242573





ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.751	S_{D1} :	N/A
S_1 :	0.685	T_L :	8
F_a :	1	PGA :	0.741
F_v :	N/A	PGA_M :	0.815
S_{MS} :	1.751	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.168	C_v :	1.45

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

Data Accessed: Thu Aug 06 2020

Date Source: USGS Seismic Design Maps

CA.GOV Select Language

California
Department of Conservation

Home | CGS | Ground Motion Interpolator

Ground Motion Interpolator

Ground Motion Interpolator (2008)

Longitude: -117.242573

Latitude: 33.945626

Site Condition (VS30): 270 (180-1050 m/sec)

Return Period:

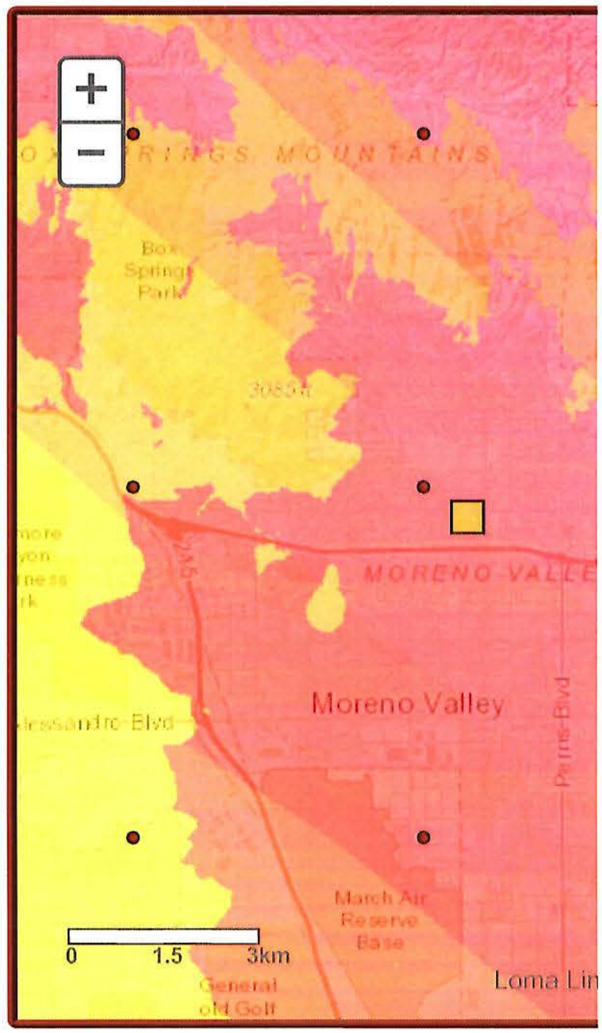
2% in 50 years   10% in 50 years

Spectral Acceleration:

PGA   0.2 second SA   1.0 second SA

Submit

Inputs:	**Result:**
-117.242573, 33.945626 vs30: 270 m/sec 10% in 50 years PGA	**0.562 g**



Information and Disclaimer

CGS MENU

U.S. Geological Survey - Earthquake Hazards Program

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)
3.86	San Jacinto;SJV+A	CA	n/a	90	V	strike slip	0	17	89
3.86	San Jacinto;SJV+A+CC	CA	n/a	90	V	strike slip	0	16	136
3.86	San Jacinto;SJV+A+CC+B	CA	n/a	90	V	strike slip	0.1	15	170
3.86	San Jacinto;SJV+A+CC+B+SM	CA	n/a	90	V	strike slip	0.1	15	196
3.86	San Jacinto;SJV+A+C	CA	n/a	90	V	strike slip	0	17	136
3.86	San Jacinto;SJV	CA	18	90	V	strike slip	0	16	43
3.86	San Jacinto;SBV+SJV	CA	n/a	90	V	strike slip	0	16	88
3.86	San Jacinto;SBV+SJV+A	CA	n/a	90	V	strike slip	0	16	134
3.86	San Jacinto;SBV+SJV+A+C	CA	n/a	90	V	strike slip	0	17	181
3.86	San Jacinto;SBV+SJV+A+CC	CA	n/a	90	V	strike slip	0	16	181
3.86	San Jacinto;SBV+SJV+A+CC+B	CA	n/a	90	V	strike slip	0.1	15	215
3.86	San Jacinto;SBV+SJV+A+CC+B+SM	CA	n/a	90	V	strike slip	0.1	15	241
4.93	San Jacinto;SBV	CA	6	90	V	strike slip	0	16	45
8.55	San Jacinto;A+C	CA	n/a	90	V	strike slip	0	17	118
8.55	San Jacinto;A+CC+B	CA	n/a	90	V	strike slip	0.1	15	152
8.55	San Jacinto;A+CC+B+SM	CA	n/a	90	V	strike slip	0.1	15	178
8.55	San Jacinto;A	CA	9	90	V	strike slip	0	17	71

U.S. Geological Survey - Earthquake Hazards Program

2008 National Seismic Hazard Maps - Source Parameters

New Search

Fault Name	State
San Jacinto;SJV+A	**California**

GEOMETRY	
Dip (degrees)	90
Dip direction	V
Sense of slip	strike slip
Rupture top (km)	0
Rupture bottom (km)	17
Rake (degrees)	180
Length (km)	89

MODEL VALUES		
Slip Rate	n/a	
Probability of activity	1	
	ELLSWORTH	HANKS
Minimum magnitude	6.5	6.5
Maximum magnitude	7.47	7.44
b-value	0.8	0.8

Fault Model	Deformation	Char Rate[1]	GR-a-	Weight

	Model		value[1]	
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

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.