APPENDIX I

Feasibility Study_Report of Soils and Foundation Evaluations
Feasibility Study
Report of Soils and Foundation Evaluations
Proposed Four (4) detached Mixed-Use/ Retail Structures
Planned Addition to Moreno Valley Festival Business Park
Moreno Valley Festival Business Park
NEC Heacock Street & Hemlock Avenue
City of Moreno Valley, California

Project No. 15025-F
May 24, 2018

Prepared for:
Black Ridge Real Estate Group, Inc.
c/o Mr. Ryan Martin
16901 Millikan Avenue
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Established 1984
May 24, 2018

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

Attention: Mr. Ryan Martin

Subject: Feasibility Study-Report of Soils and Foundation Evaluations
Proposed Four (4) Mixed-Use/Retail Structures
Planned Addition to Moreno Valley Festival Business Park
NEC Heacock Street & Hemlock Avenue
City of Moreno Valley, California

Reference: Conceptual Site Plan Prepared by Herdman Architecture Design

Gentlemen:

Presented herewith are the Report of Soils and Foundation Evaluations conducted for the site of the planned four (4) mixed-use/retail structures to the Moreno Valley Business Park, located near the northeast intersection of Heacock Street and Hemlock Avenue, City of Moreno Valley, California. In absence of development details, the recommendations included should be considered as “preliminary”, subject to revision following detailed grading and development plans review.

Based on test explorations completed at this time it is our opinion that, in general, the soils existing primarily consist of upper compressible, dry to damp, loose to medium dense, fine to medium coarse silty gravelly sand with pebbles and minor rocks, overlying deposits of medium dense to dense, silty gravelly sand to the maximum 41 feet depth explored. No shallow-depth groundwater or bedrock was encountered.

Based on review of the available public documents, it is our opinion that the site is not situated within an A-P Special Studies Zone, and the site is considered non-susceptible to soils liquefaction in event of a strong motion earthquake.

Compressible in nature, near grade soils existing as described are considered unsuitable for directly supporting structural loadings without excessive differential settlements to load bearing footings and concrete slabs-on-grade. When, however, graded as recommended herein, the structural pads thus constructed should be adequate for the development proposed.

If you have any questions regarding this report, please contact the undersigned.

Respectfully submitted,
Soils Southwest, Inc.

Moloy Gupta, RCE 31709

John Flippin
Project Coordinator

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Established 1984
Presented herewith are the Report of Preliminary Soils and Foundation Evaluations conducted for the site of the planned 4-detached mixed-use/retail structure to the existing Moreno Valley Business Park, located near the northeast intersection of Heacock Street and Hemlock Avenue, City of Moreno Valley, 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, slab-on-grade, paving, parking, site grading, utility trench 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 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, four detached multi-tenant/retail structures of conventional wood-frame and stucco, or concrete tilt-up construction with continuous wall and isolated spread footings with concrete slab-on-grade. While three of the planned structures, namely building 1 (174,720 sft), building 2 (49,419 sft), and building 3 (106,250 sft) are proposed along the west side of the partially paved interior Davis Drive, the remaining proposed building 4 (33,674 sft) is proposed adjacent to Ironwood Drive near the north.

Supplemental construction is anticipated to include paving/parking, driveways, relocation of the existing surface drainage, among others. Considering minor sloping with uneven natural grades, moderate site preparations and grading should be anticipated for the development plan.

1.2 Site Description

Being a part of an existing shopping plaza, areas of the planned structures are currently vacant and unimproved. In general, the site in overall is bounded by Ironwood Avenue on the north, by Hemlock Avenue on the south, by commercial retail and vacant undeveloped property on the east, and by Heacock Street on the west. The overall vertical relief within the property is currently unknown; however surface water 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 Work

Being beyond scope of work, no Geologic or Phase I site Assessments are included. Reports on such will be supplied, if requested. Geotechnical evaluations included subsurface explorations, soil sampling, necessary laboratory testing, engineering analyses and the preparation of this report. In general, scope of work included the following tasks:

- **Field Testing**

  Twelve (12) exploratory test borings explored using a Hollow-Stem Auger (HSA) drill-rig equipped for undisturbed soils sampling and Standard Penetration Testing (SPT). The exploratory depth was advanced to maximum refusal depth of 41 feet below the current grade surface.

  During excavations, the 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 testing. Description of the soils encountered is shown on the attached Log of Boring in Appendix A.

- **Laboratory Testing**

  Representative samples on selected bulk and undisturbed site soils were tested in the laboratory to aid in the soils classification and to evaluate relevant engineering properties of the existing site soils pertaining to the project requirements. These tests may include some or all of the following tests depending upon site requirements:

  - In-situ moisture contents and dry density (ASTM Standard D2216)
  - Consolidation (D2835)
  - Gradation analysis (ASTM Standard D422)
  - Maximum dry density and optimum moisture content (ASTM Standard D1557)
  - Sand equivalent (ASTM Standard D2419)
  - Direct Shear (ASTM Standard D3080)

  Description of the test results and test procedures used are provided in Appendix B.

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

Based on the test explorations completed at the locations and to the depths as described in this report, it is our opinion that the site soils encountered primarily consist of upper compressible, dry to damp, loose to medium dense, fine to medium coarse silty sand with pebbles and rock fragments up to about 3 to 5 feet, overlying deposits of medium dense to dense, silty gravelly sand with minor rocks and traces of clay to the maximum 41 feet depth explored. No free groundwater or shallow depth bedrock was encountered.

It is our opinion that the upper low-density, compressible and variable consistency soils varying in depth from 3 to 5 feet existing grade surface existing as described should be considered inadequate for structural support without excessive differential settlements to load bearing footings and concrete slab-on-grade. When however, graded in form of subexcavations of the near grade soils and their replacement as engineered fills compacted to minimum 90%, the structural pads thus constructed should be adequate for the proposed construction described.

Laboratory shear tests conducted on the area surface bulk soil samples remolded to 90 percent indicate moderate shear strengths 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 soils remolded to 90% 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 upon exploratory test borings and laboratory testing completed at this time, it is our opinion that the upper 3 to 5 feet of the loose and dry soils existing as encountered should be considered compressible/collapsible in nature, and may be susceptible to excessive settlements under conventional structural loadings. Accordingly, for adequate load bearing support, it is opinion that the near surface soils should be subexcavated, followed by their replacement as engineered fills compacted to minimum 90 percent.

The detailed subexcavation requirements are described in Section 4.1.1 of this report. It is recommended that subexcavation depths for each structural pad should be verified and approved by soils engineer prior to new structural fill soils placement. Local soils free of organic should be considered suitable for re-use during grading.

3.1.2 Expansive Soils

Based on laboratory testing on representative site soils procured, the sandy silty soils as encountered are considered "very low" expansion potential. It is recommended that, during and post-grading, soils expansion potential should be further verified, based on which revised structural recommendations may be warranted.

3.2 Excavatability

It is our opinion that the grading required for the project may be accomplished using conventional heavy-duty construction equipment. Presence of no shallow-depth bedrock or impenetrable subgrades is expected.
3.3 Groundwater

No Groundwater was encountered within the maximum 41 feet depth explored. However, fluctuations in groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, altered natural drainage paths, and other factors that are obvious at the time the test explorations completed. Although no shallow depth groundwater should be expected during site preparation and grading, it is our opinion that the project civil engineer and grading contractor should establish surface water runoff pattern that is directed away from the structural pad once constructed.

The following table describes the nearest well and historical groundwater information of the referenced site and its vicinity as listed by the local reporting agency:

<table>
<thead>
<tr>
<th>Reporting Agency</th>
<th>Water Master Support Services-San Bernardino Valley Conservation District/Western Municipal Water District Cooperative Well Measuring Program, Fall 2016</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well Number</td>
<td>03S/3W-6N003S</td>
</tr>
<tr>
<td>Well Monitoring Agency</td>
<td>EMWD 46 Edgemont Gardens 02</td>
</tr>
<tr>
<td>Well Location: Township/Range/Section</td>
<td>T3S-R3W-Section 6</td>
</tr>
<tr>
<td>Well Elevation:</td>
<td>1622</td>
</tr>
<tr>
<td>Current Depth to Water (Measured in feet)</td>
<td>59.9</td>
</tr>
<tr>
<td>Current Date Water was Measured</td>
<td>September 29, 2016</td>
</tr>
<tr>
<td>Depth to Water (Measured in feet) (Shallowest)</td>
<td>58.3</td>
</tr>
<tr>
<td>Date Water was Measured (Shallowest)</td>
<td>April 11, 2016</td>
</tr>
</tbody>
</table>
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, the site being within Southern California, with the current industry knowhow, the planned development should be considered feasible when graded and designed using the recommendations included, including those applicable seismic design parameters as the current 2016 CBC as described.

4.2 Direct or Primary Seismic Hazards

Surface ground rupture associated with ground shaking represents primary or direct seismic hazards to structures. There are no known active or potentially active faults that pass through or its adjacent and the site is not situated within an AP Special Studies Zone. According to the current CBC, the site is considered situated within Seismic Zone 4. As a result, it is likely that during life expectancy of the proposed construction, moderate to severe ground shaking may have potential for adverse effects on the structure built requiring minor to moderate repair.

4.3 Induced or Secondary Seismic Hazards

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

4.4 Flooding

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

4.5 Land Sliding

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

4.6 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. Based on the general topography of the site and its adjacent, it is our opinion that the potential for seismically induced lateral ground spreading should be considered “remote”.

4.7 Site Specific Seismic Effects

The site is situated at about 3.8 miles from the San Jacinto-San Jacinto Valley Fault capable of generating an earthquake magnitude of M=6.9 and PGA of 0.561g.
4.8 Seismic Design Coordinates

The design spectrum for the site is developed based on the 2016 CBC. Site Coordinates of 33.944274°N, -117.241836°W are used to establish the seismic parameters as presented below.

4.9 Seismic Design Coefficients

Recommended values are based upon the USGS ASCE 7-10 (March 2013 errata) Parameters and the California Geologic Survey: PSHA Ground Motion Interpolator Supplemental seismic parameters as provided in Appendix C of this report. The following presents the seismic design parameters as based on the currently published California Geological Survey and 2016 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.

In design, vertical accelerations may be assumed to about 1/3 to 2/3 of the estimated horizontal ground acceleration (PGA) as described in the following sections.

**TABLE 4.9 A.1 Seismic Design Parameters**

<table>
<thead>
<tr>
<th>CBC Chapter 16</th>
<th>2016 ASCE 7-10 Standard Seismic Design Parameters</th>
<th>Recommended Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>1613A.5.2</td>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>1613.5.1</td>
<td>The mapped spectral accelerations at short period</td>
<td>S₁</td>
</tr>
<tr>
<td>1613.5.1</td>
<td>The mapped spectral accelerations at 1.0-second period</td>
<td>S₁</td>
</tr>
<tr>
<td>1613A5.3(1)</td>
<td>Seismic Coefficient, S₃</td>
<td>1.70 lg</td>
</tr>
<tr>
<td>1613A5.3(2)</td>
<td>Seismic Coefficient, S₁</td>
<td>0.743 g</td>
</tr>
<tr>
<td>1613A5.3(1)</td>
<td>Site Class D / Seismic Coefficient, F₄</td>
<td>1.00 g</td>
</tr>
<tr>
<td>1613A5.3(2)</td>
<td>Site Class D / Seismic Coefficient, F₅</td>
<td>1.50 g</td>
</tr>
<tr>
<td>16A-37 Equation</td>
<td>Spectral Response Accelerations, S₅₆ = F₅ S₅</td>
<td>1.70 lg</td>
</tr>
<tr>
<td>16A-38 Equation</td>
<td>Spectral Response Accelerations, S₅₇ = F₁ S₁</td>
<td>1.11 lg</td>
</tr>
<tr>
<td>16A-39 Equation</td>
<td>Design Spectral Response Accelerations, S₆₆ = 2/3 x S₅₆</td>
<td>1.13 lg</td>
</tr>
<tr>
<td>16A-40 Equation</td>
<td>Design Spectral Response Accelerations, S₀₁ = 2/3 x S₅₁</td>
<td>0.743 g</td>
</tr>
</tbody>
</table>

**TABLE 4.9 A.2 Seismic Source Type**

Based on California Geological Survey (CGS)-Probabilistic Seismic Hazard Assessment Peak Horizontal Ground Acceleration (PGA) having a 10 percent probability of exceedance in a 50- year period is as described below:
<table>
<thead>
<tr>
<th>Seismic Source Type / Appendix C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nearest Maximum Fault Magnitude</td>
</tr>
<tr>
<td>Peak Horizontal Ground Acceleration (PGA)</td>
</tr>
</tbody>
</table>

Structural design should be intended so as to resist total structural collapse due to the PGA as described. Being located within Southern California, it is our opinion that, during life time use of the structure built, some structural damage may occur requiring minor to major repair.
5.0 Evaluations and Recommendation

5.1 General Evaluations

The conclusions contained herein are based 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 cavity 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 compressible during grading and utility installations.

(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 at 90% or better. In event new fill soils are required over the grades existing, such should be placed following subgrade preparations as described. No footings and/or new fills should be placed directly bearing on the compressible surface soils existing.

(iv) The sub-excitation depths described in this report should be considered as ‘minimum to moderate’. During grading localized deeper sub-excavations may be required following vegetation removal and within areas of buried debris, irrigation pipes etc. It will be the responsibility of the grading contractor to inform soils engineer the presence of such 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 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/UBC and as described earlier in Section 4.9 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 Pads

Considering near grade variable consistency of 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.

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%.

In general, for each of the structural pads proposed, the grading should include sub-excavations of the near surface soils to (i) 5 feet below the current grade surface, or (ii) to the depth as required to expose 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. Supplemental subexcavations and grading may be warranted for the planned structures #2 and #3 when load bearing footings are established with the current low-lying drainage as existing as described. Supplemental detailed recommendations will be supplied following grading and development plan review.

The site preparations, subexcavations and grading described should encompass, in minimum, the individual planned structural foot-print areas, and minimum 5 feet beyond. The engineered fills for structural support should be compacted to minimum 90% 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 may be considered suitable for re-use as structural backfill.

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.

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 gravely 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:

<p>| | |</p>
<table>
<thead>
<tr>
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<th></th>
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</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td>&lt;35</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>&lt;15</td>
</tr>
<tr>
<td>Expansion Index</td>
<td>&lt;20</td>
</tr>
</tbody>
</table>

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 90 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 90%. From geotechnical view point, 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 and soil lateral passive resistance, as well as 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 90%. 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 increment in foundation bearing capacity may be considered for each 1 foot increment in footing embedment up to a total not exceeding 4000 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.
5.3.2 Concrete Slab-on-Grade

The prepared subgrades compacted to minimum 90% prepared to receive footings should be adequate for concrete slab-on-grade placement. For interior commercial use, 5" 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.

5.3.3 Concrete Driveways

For normal vehicular support and driveways, concrete slabs should be minimum 6-inch thick, placed over local sandy soils compacted to at least 95%. Driveway slab reinforcing and construction and expansion joints etc. should be incorporated as required by the project structural engineer. 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.

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 STRUCTURAL 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°F OR 80°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 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.
5.4 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 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:

<table>
<thead>
<tr>
<th></th>
<th>Active:</th>
<th>35 pcf</th>
</tr>
</thead>
<tbody>
<tr>
<td>At Rest:</td>
<td>80 pcf</td>
<td></td>
</tr>
</tbody>
</table>

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 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 12 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 purposes, 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.

5.6 Construction Consideration

5.6.1 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.

5.6.2 Supported Excavations

If vertical excavations exceeding 5 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-exca va tion of the upper loose, disturbed compressible soils, stock-piling, moisturization to near Optimum Moisture contents of the soils used. Site preparations 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 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.

5.9 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.

<table>
<thead>
<tr>
<th>Service Area</th>
<th>Traffic Index, TI</th>
<th>Pavement Type</th>
<th>Paving Thickness (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>On-site paving/parking for commercial vehicle/conventional passenger cars</td>
<td>6.5</td>
<td>a.c. over CL. II base</td>
<td>4.0 over 6.0</td>
</tr>
</tbody>
</table>

Within paving areas, subgrade soils should be scarified to 12-inch, 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 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.

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.
5.10 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

5.10.1 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.

5.11 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.

5.12 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.

5.13 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.

5.14 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.
5.15 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.

5.16 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 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 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 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
Planned Addition to Existing Moreno Valley Business Park
Planned 4-detached Warehouse/Retail Structures
NEC Heacock & Hemlock Avenues
City of Moreno Valley, California

(Not to Scale)

Legend: ◆ B-1 Approximate Location of Test Borings

Soils Southwest, Inc.  May 24, 2018  Page 21
8.0 APPENDIX A

Field Explorations

Field evaluations included site reconnaissance and twelve (12) exploratory 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 TEST EXPLORATIONS
**LOG OF BORING B-1**

**Project:** Moreno Valley Business Park Expansion  
**Job No.:** 15025-F

**Logged By:** John F.  
**Boring Diam.:** 8"HSA  
**Date:** May

<table>
<thead>
<tr>
<th>Standard Penetration Test</th>
<th>Sample Type</th>
<th>Water Content in %</th>
<th>Dry Density in PCF</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
<th>Description and Remarks</th>
</tr>
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<td>SP-SM</td>
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<td>weeds, scattered concrete debris</td>
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<td></td>
<td></td>
<td>- color change to gray, rock fragments</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>- silty, fine to medium, scattered rock fragments, dry to damp, loose</td>
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<td>- dense</td>
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<td></td>
<td></td>
<td></td>
<td>SP</td>
<td></td>
<td>20</td>
<td>- color change to reddish brown, traces of silt and clay, fine to medium coarse, pebble, rock fragments, damp dense</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>- color change to light brown, fine to medium coarse</td>
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<td>- silty, fine</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SM-ML</td>
<td></td>
<td>25</td>
<td>- silty, fine to medium, pebble, occasional rock fragments, dry to damp</td>
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<tr>
<td></td>
<td></td>
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<td>- silty, fine, damp</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>SM-ML</td>
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<td>30</td>
<td>- fine to medium coarse, traces of silt, gravely, pebble, rock fragments</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- End of test boring</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- no bedrock</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- no groundwater</td>
</tr>
</tbody>
</table>

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

**Site Location**  
Planned addition to Existing  
Moreno Valley Business Park NEC  
Hemlock and Heacock Avenue  
Moreno Valley, California

**Plate #**

- Bulk/Grab sample
- Standard penetration test
- California sampler
### LOG OF BORING B-2

**Project:** Moreno Valley Business Park Expansion  
**Job No.:** 15025-F  
**Logged By:** John F.  
**Boring Diam.:** 8" HSA  
**Date:** May

<table>
<thead>
<tr>
<th>Depth in Ft</th>
<th>Description and Remarks</th>
</tr>
</thead>
</table>
| 5           | tilled weeds and scattered debris  
|             | SAND - light brown, silty, fine to medium pebbles, occasional rock fragments, dry |
| 10          | - tiny pinholes, silty, fine, scattered pebbles and rock fragments |
| 15          | - silty, fine, occasional pebbles, dry  
|             | - End of test boring @ 10.0 ft.  
|             | - no bedrock  
|             | - no groundwater |

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

**Site Location**  
Planned addition to Existing Moreno Valley Business Park NEC  
Hemlock and Heacock Avenue  
Moreno Valley, California

**Plate #**

- Bulk/Grab sample
- Standard penetration test
- California sampler
### LOG OF BORING B-3

**Project:** Moreno Valley Business Park Expansion  
**Job No.:** 15025-F  
**Logged By:** John F.  
**Boring Diam.:** 8" HSA  
**Date:** May

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<th>Standard Penetration (Blows per Ft)</th>
<th>Water Content in %</th>
<th>Dry Density inpcf</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Depth in Feet</th>
<th>Description and Remarks</th>
</tr>
</thead>
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<td>SM-ML</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>19</td>
<td>tilled weeds, scattered debris (concrete)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5</td>
<td>SAND - light brown, silty, fine, scattered pebbles and rock fragments, dry</td>
</tr>
<tr>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16</td>
<td>- silty, fine to medium, occasional pebbles and rock fragments, medium dense</td>
</tr>
<tr>
<td>SP-SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td>- slightly silty, fine to medium coarse pebbles, rock fragments, dry</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td>- slightly silty, fine to medium, pebble, scattered, rock fragments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td>- End of test boring @ 11.0 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>25</td>
<td>- no bedrock</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>30</td>
<td>- no groundwater</td>
</tr>
</tbody>
</table>

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

**Site Location**  
Planned addition to Existing Moreno Valley Business Park NEC  
Hemlock and Heacock Avenue  
Moreno Valley, California

**Plate #**  
Bulk/Grab sample  
Standard penetration test  
California sampler
LOG OF BORING B-4

Project: Moreno Valley Business Park Expansion  Job No.: 15025-F
Logged By: John F.  Boring Diam.: 8" HSA  Date: May

Description and Remarks

5.4 101.2 77.2  SP

- tilled weeds, scattered debris
- SAND - light brown, traces of silt, fine scattered pebbles, and rock fragments dry
- silt-sand mix, fine, scattered pebble and rock fragments, dry, and loose
- color change to gray-brown, fine to medium coarse, pebbles, rock fragment loose

7

- traces of clay, fine to medium, pebbles, occasional rock fragments
- traces of clay, gravely, fine to coarse, pebbles, rock fragments, damp

17

- End of test boring @ 16.0 ft.
  - no bedrock
  - no groundwater

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

Site Location
Planned addition to Existing Moreno Valley Business Park NEC
Hemlock and Heacock Avenue
Moreno Valley, California

Plate #

Bulk/Grab sample  Standard penetration test  California sampler
<table>
<thead>
<tr>
<th>Depth in Feet</th>
<th>Description and Remarks</th>
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<td>Surface weeds</td>
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<td>SAND - light brown, silty, fine to medium scattered pebbles and rock fragments dry</td>
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<td>5</td>
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<tr>
<td></td>
<td>- color change to gray-brown, silty, fine, scattered pebbles, damp</td>
</tr>
<tr>
<td>10</td>
<td>SP</td>
</tr>
<tr>
<td></td>
<td>- color change to dark brown, dg origin material, slightly silty, fine to medium, pebbles, damp</td>
</tr>
<tr>
<td></td>
<td>- End of test boring @ 10.0 ft.</td>
</tr>
<tr>
<td></td>
<td>- no bedrock</td>
</tr>
<tr>
<td>15</td>
<td></td>
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<tr>
<td>20</td>
<td></td>
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<tr>
<td>25</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

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

Site Location: Planned addition to Existing  
Moreno Valley Business Park NEC  
Hemlock and Heacock Avenue  
Moreno Valley, California  

Plate #  

Bulk/Grab sample  
Standard penetration test  
California sampler
**LOG OF BORING B-6**

**Project:** Moreno Valley Business Park Expansion  
**Job No.:** 15025-F  
**Logged By:** John F.  
**Boring Diam.:** 8" HSA  
**Date:** May

<table>
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<tr>
<th>Standard Penetration (Sample Pct)</th>
<th>Water Content in %</th>
<th>Dry Density in PCF</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
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<tbody>
<tr>
<td>SM</td>
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<td></td>
<td></td>
<td>Silt</td>
<td></td>
<td>5</td>
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<td>SM-ML</td>
<td></td>
<td></td>
<td></td>
<td>Silt</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>SM</td>
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<td>Silt</td>
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<td>15</td>
</tr>
<tr>
<td>SM-ML</td>
<td></td>
<td></td>
<td></td>
<td>Silt</td>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Silt</td>
<td></td>
<td>25</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td>Silt</td>
<td></td>
<td>30</td>
</tr>
</tbody>
</table>

**Description and Remarks**

- *surface weeds*
  - SAND - light brown, silty, fine to medium pebbles, occasional rock fragments, dry
  - color change to yellowish light brown silty, fine
  - color change to orangish reddish brown to yellowish brown, silty, fine to medium, pebble and rock fragments very dense, dry to damp
  - color change to light brown, silty, fine to medium, pebbles, scattered rock fragments, damp, very dense
  - color change to light brown, silty, fine, scattered pebble, damp
  - color change to light brown, silty, fine to medium, pebble, scattered rock fragments & rock 1/2"-1", damp
  - silty, fine, medium dense, damp

- *End of test boring @ 20.0 ft.*
  - no bedrock
  - no groundwater

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

**Site Location**

- Planned addition to Existing Moreno Valley Business Park NRC  
- Hemlock and Heacock Avenue  
- Moreno Valley, California

**Plate #**
Description and Remarks:

Surface weeds

SAND - light brown, slightly silty, fine to medium coarse, damp to moist

- Color change to dark brown, traces of clay, moist, lumpy

- Clayey, lumpy, moist

- End of test boring @ 11.0 ft.
  - No bedrock
  - No groundwater

Groundwater: n/a
Approx. Depth of Bedrock: n/a
Datum: n/a
Elevation: n/a
Soils Southwest, Inc.
897 Via Lata, Suite N
Colton, CA 92324
(909) 370-0474  Fax (909) 370-3156

LOG OF BORING B-8

Project: Moreno Valley Business Park Expansion  Job No.: 15025-F
Logged By: John F.  Boring Diam.: 3"HSA  Date: May

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<thead>
<tr>
<th>Standard Penetration (Blows per ft.)</th>
<th>Water Content in %</th>
<th>Dry Density in PCF</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
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</thead>
<tbody>
<tr>
<td>6.5</td>
<td>106.9</td>
<td>82</td>
<td></td>
<td>SM</td>
<td></td>
<td>5</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td></td>
<td>SM-SC</td>
<td></td>
<td>15</td>
</tr>
<tr>
<td>11</td>
<td>123.9</td>
<td>95</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Description and Remarks

- weeds, gravelly sands, scattered debris
- SAND - light brown, silty, fine to medium
- gravelly, fine to medium coarse, pebbles and rock fragments
- color change to reddish dark reddish brown, slightly clayey, scattered pebbles, rock fragments, medium dense damp
- traces of clay
- End of test boring @ 16.0 ft.
  - no bedrock
  - no groundwater

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

Site Location
Planned addition to Existing Moreno Valley Business Park NEC Hemlock and Heacock Avenue Moreno Valley, California

Plate #

Bulk/Grab sample  Standard penetration test  California sampler
### LOG OF BORING B-9

<table>
<thead>
<tr>
<th>Standard Penetration (BF)</th>
<th>Sample Code</th>
<th>Water Content in %</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>SM-ML</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.6</td>
<td>124.1</td>
<td>95</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5.6</td>
<td>114.9</td>
<td>88.0</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td></td>
<td></td>
<td>20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Description and Remarks**

- Surface weeds, gravels, scattered debris
- SAND - light yellowish brown, silty, fine
  - traces of clay, fine to medium, pebbles, dry
  - dense
  - silt, fine, pebble, occasional rock fragments, dry
  - fine to medium coarse, pebbles, rock fragments
  - fine to medium, pebble, scattered rock fragments, dry
  - color change to gray-brown, silty, traces of clay, fine to medium, pebbles
    - End of test boring @ 20.0 ft.
      - no bedrock
      - no groundwater

---

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

**Site Location**

Planned addition to Existing Moreno Valley Business Park NEC Hemlock and Heacock Avenue Moreno Valley, California

**Plate #**

- Bulu/Grab sample
- Standard penetration test
- California sampler
<table>
<thead>
<tr>
<th>Depth (Ft)</th>
<th>SM-ML</th>
<th>SM-M</th>
<th>SP-SM</th>
<th>SM</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>SM</td>
<td>SM-M</td>
<td>SP-SM</td>
<td>SM</td>
</tr>
<tr>
<td>5-10</td>
<td>SM-M</td>
<td>SM-M</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td>10-15</td>
<td>SM-M</td>
<td>SM-M</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td>15-20</td>
<td>SM-M</td>
<td>SM-M</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td>20-25</td>
<td>SM-M</td>
<td>SM-M</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td>25-30</td>
<td>SM-M</td>
<td>SM-M</td>
<td>SP</td>
<td>SM</td>
</tr>
</tbody>
</table>

**Description and Remarks**

Gravels, scattered weeds

SAND - light brown, silty, pebbles, fine to medium, dry
- traces of clay

- color change to yellowish light brown silty, fine, pebbles, occasional rock fragments, dry
- color change to light brown, silty, traces of clay, fine, dry

- silty, fine to medium coarse, pebble rock fragments, scattered rock 1/4" dry to damp, dense

- color change to gray-brown, silty, fine to medium, pebble, rock fragment damp
- color change to gray-brown, silty, fine to medium coarse, pebble

- silty, fine, pebbles, damp

- traces of silt, fine to medium coarse pebbles, rock fragments

- scattered pebble and rock fragments

- color change to gray-brown, slightly silty, gravelly, fine to medium coarse pebble, rock fragments, damp

- silty, fine to medium, pebble, dry

**Groundwater:** n/a

**Approx. Depth of Bedrock:** n/a

**Datum:** n/a

**Elevation:** n/a
<table>
<thead>
<tr>
<th>Standard Penetration (Bored per ft.)</th>
<th>Water Content (in %)</th>
<th>Dry Density (pcf)</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>36</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>55</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>70</td>
</tr>
</tbody>
</table>

Description and Remarks:
- Silty, fine to medium, scattered rock fragments, dense, damp
- End of test boring @ 36.0 ft.
- No bedrock
- No groundwater
## LOG OF BORING B-11

**Project:** Moreno Valley Business Park Expansion  
**Job No.:** 15025-F  
**Logged By:** John F.  
**Boring Diam.:** 8"HSA  
**Date:** May

<table>
<thead>
<tr>
<th>Standard Penetration (Bamper per ft)</th>
<th>Unit Weight inpcf</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Depth in Feet</th>
<th>Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>115.4</td>
<td>88.1</td>
<td>SM</td>
<td></td>
<td></td>
<td>surface weeds</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP-SM</td>
<td>5</td>
<td></td>
<td>SAND - light brown, silty, fine to medium, pebble</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SM-SC</td>
<td>10</td>
<td></td>
<td>color change to light reddish brown, slightly clayey, silty, fine, pebbles</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td>- NO SAMPLE RECOVERY</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td>- End of test boring @ 11.0 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>25</td>
<td>- no bedrock</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>30</td>
<td>- no groundwater</td>
</tr>
</tbody>
</table>

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

**Site Location**  
Planned addition to Existing Moreno Valley Business Park NBC Hemlock and Heacock Avenue Moreno Valley, California

**Plate #**
# LOG OF BORING B-12

**Project:** Moreno Valley Business Park Expansion  
**Logged By:** John F.  
**Boring Diam.:** 8"HSA  
**Date:** May  
**Job No.:** 15025-F

<table>
<thead>
<tr>
<th>Sample Point</th>
<th>Depth (ft)</th>
<th>Percent Compaction</th>
<th>Unified Classification System</th>
<th>Graphic</th>
<th>Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>7.7 121.3 93</td>
<td>SM</td>
<td></td>
<td>gravel, scattered weeds and brush</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td>5</td>
<td>trace of silt and clay, fine to medium coarse, pebble, rock fragments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SM</td>
<td>10</td>
<td>color change to light yellowish brown silty, fine to medium</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td>15</td>
<td>trace of silt, fine to medium, pebbles, occasional rock fragments, scattered rock 1/2&quot;-1&quot;, dry to damp</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP-SM</td>
<td>20</td>
<td>color change to light yellowish brown to orangish light brown, slightly silty fine to medium coarse, pebbles, rock fragments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td>25</td>
<td>color change to light brown, traces of silt, pebbles, rock fragments, damp</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ML</td>
<td>30</td>
<td>color change to light brown, traces of silts, fine to medium coarse, pebbles, rock fragments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SP</td>
<td></td>
<td>color change to gray-brown, fine to medium coarse, pebbles, rock fragment scattered rock 1/2&quot;, damp</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>color change to light brown, traces of silts and clays, fine to medium pebbles, rock fragments</td>
</tr>
</tbody>
</table>

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

**Site Location**  
Planned addition to Existing Moreno Valley Business Park NBC  
Hemlock and Heacock Avenue  
Moreno Valley, California

**Plate #**

- Bulb/Grab sample  
- Standard penetration test  
- California sampler
**LOG OF BORING B-12**

**Project:** Moreno Valley Business Park Expansion  
**Logged By:** John F.  
**Boring Diam.:** 8" HSA  
**Date:** May

<table>
<thead>
<tr>
<th>Description and Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>- very dense</td>
</tr>
<tr>
<td>- End of test boring @ 41.0 ft.</td>
</tr>
<tr>
<td>- no bedrock</td>
</tr>
<tr>
<td>- no groundwater</td>
</tr>
</tbody>
</table>
1. Exploratory borings were drilled on May 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 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 tests, 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)

<table>
<thead>
<tr>
<th>Test Boring No.</th>
<th>Sample Depth, ft.</th>
<th>Dry Density, pcf.</th>
<th>Moisture Content, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>124.2</td>
<td>6.1</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>108.5</td>
<td>5.4</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>101.2</td>
<td>5.4</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
<td>119.1</td>
<td>14.0</td>
</tr>
<tr>
<td>6</td>
<td>15</td>
<td>120.4</td>
<td>13.2</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>106.9</td>
<td>6.5</td>
</tr>
<tr>
<td>8</td>
<td>15</td>
<td>123.9</td>
<td>11.4</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>124.1</td>
<td>5.6</td>
</tr>
<tr>
<td>9</td>
<td>10</td>
<td>114.9</td>
<td>5.6</td>
</tr>
<tr>
<td>10</td>
<td>7</td>
<td>116.4</td>
<td>5.2</td>
</tr>
<tr>
<td>10</td>
<td>15</td>
<td>125.6</td>
<td>5.4</td>
</tr>
<tr>
<td>11</td>
<td>5</td>
<td>115.4</td>
<td>6.3</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>121.3</td>
<td>7.7</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Sample Location, @ Depth, ft.</th>
<th>Max. Dry Density, pcf</th>
<th>Opt. Moisture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 1-5</td>
<td>131.0</td>
<td>9.0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Sample Location &amp; Soils Type</th>
<th>Soil Expansion Index, EI</th>
<th>Expansion Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 1-5' Sand-gravelly, fine to medium coarse with occasional rock 1&quot;</td>
<td>13</td>
<td>&quot;very low&quot;</td>
</tr>
<tr>
<td>B-8 @ 3'-5' Sand-silty fine to medium</td>
<td>19</td>
<td>&quot;very low&quot;</td>
</tr>
</tbody>
</table>
Direct Shear Tests

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>LOCATION</th>
<th>DEPTH (FT)</th>
<th>TEST CONDITION</th>
<th>COHESION (psf)</th>
<th>FRICTION (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>1 to 5</td>
<td>Remolded to 90%</td>
<td>240.00</td>
<td>41.93</td>
<td></td>
</tr>
</tbody>
</table>

Proposed Warehouse and Commercial Development
NE Heacock Street & Hemlock Avenue
Moreno Valley, California

SOILS SOUTHWEST, INC.
Consulting Foundation Engineers
CONsolidation Tests

Loads in kips per square foot

Sample A B-1 @ 1-5 ft.
Bulk Remolded to 90%
Initial Moisture = 9.0%
Final Moisture = 19.0%

- WATER PERMITTED TO CONTACT SAMPLE

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>Proposed Warehouse &amp; Commercial Retail Development</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NE Heacock St. &amp; Hemlock Ave, Moreno Valley</td>
</tr>
<tr>
<td>PROJECT NO.</td>
<td>15025-F</td>
</tr>
</tbody>
</table>

SOILS SOUTHWEST INC.
Consulting Foundation Engineers
APPENDIX C

Supplemental Seismic Design Parameters
State of California
Department of Conservation

Ground Motion Interpolator (2008)

Longitude: -117.241864
Latitude: 33.944274
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

Inputs:                           Result:
-117.241864, 33.944274
vs30: 270 m/sec
10% in 50 years
PGA

0.561 g

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http://www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html
Design Maps Summary Report

Jser-Specified Input

- **Report Title**: Moreno Valley Business Park, Hemlock Ave., Moreno Valley, CA
  - **Date**: Wed May 23, 2018 21:12:48 UTC

- **Building Code Reference Document**: ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)

- **Site Coordinates**: 33.94427°N, 117.24186°W

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

- **Risk Category**: I/II/III

---

**JSGS-Provided Output**

\[
\begin{align*}
S_a &= 1.701 \text{ g} \\
S_{ns} &= 1.701 \text{ g} \\
S_{ot} &= 1.134 \text{ g} \\
S_i &= 0.743 \text{ g} \\
S_{ni} &= 1.144 \text{ g} \\
S_{oi} &= 0.743 \text{ g}
\end{align*}
\]

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please refer to the application and elect the "2009 NEHRP" building code reference document.

---

Design Maps Detailed Report

ASCE 7-10 Standard (33.94427°N, 117.24186°W)

Site Class D - "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_2$) and 1.3 (to obtain $S_1$). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 \[^{[1]}\]

$S_2 = 1.701$ g

From Figure 22-2 \[^{[2]}\]

$S_1 = 0.743$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$\bar{v}_s$</th>
<th>$\bar{N}$ or $\bar{N}_{in}$</th>
<th>$\bar{s}_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Hard Rock</td>
<td>$&gt;5,000$ ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B. Rock</td>
<td>2,500 to 5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C. Very dense soil and soft rock</td>
<td>1,200 to 2,500 ft/s</td>
<td>$&gt;50$</td>
<td>$&gt;2,000$ psf</td>
</tr>
<tr>
<td>D. Stiff Soil</td>
<td>600 to 1,200 ft/s</td>
<td>15 to 50</td>
<td>1,000 to 2,000 psf</td>
</tr>
<tr>
<td>E. Soft clay soil</td>
<td>$&lt;600$ ft/s</td>
<td>$&lt;15$</td>
<td>$&lt;1,000$ psf</td>
</tr>
</tbody>
</table>

Any profile with more than 10 ft of soil having the characteristics:
- Plasticity index $P_I > 20$,
- Moisture content $w \geq 40\%$, and
- Undrained shear strength $\bar{s}_u < 500$ psf

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For SI: $1$ ft/s = 0.3048 m/s 1 lb/ft$^2$ = 0.0479 kN/m$^2$
### Table 11.4-1: Site Coefficient F,

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE, Spectral Response Acceleration Parameter at Short Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S_s \leq 0.25 )</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
</tbody>
</table>

F

See Section 11.4.7 of ASCE 7

Note: Use straight-line interpolation for intermediate values of \( S_s \)

For Site Class = D and \( S_s = 1.70 \) g, \( F_s = 1.00 \)

### Table 11.4-2: Site Coefficient F,

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE, Spectral Response Acceleration Parameter at 1-s Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S_s \leq 0.10 )</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
</tbody>
</table>

F

See Section 11.4.7 of ASCE 7

Note: Use straight-line interpolation for intermediate values of \( S_s \)

For Site Class = D and \( S_s = 0.743 \) g, \( F_s = 1.500 \)
Equation (11.4-1): \[ S_{NS} = F_s S_s = 1.000 \times 1.701 = 1.701 \text{ g} \]

Equation (11.4-2): \[ S_{NH} = F_s S_l = 1.500 \times 0.743 = 1.114 \text{ g} \]

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3): \[ S_{0S} = \frac{3}{4} S_{NS} = \frac{3}{4} \times 1.701 = 1.134 \text{ g} \]

Equation (11.4-4): \[ S_{0I} = \frac{3}{4} S_{NH} = \frac{3}{4} \times 1.114 = 0.743 \text{ g} \]

Section 11.4.5 — Design Response Spectrum

From Figure 22-12\( ^{(3)} \) \[ T_L = 8 \text{ seconds} \]

From Figure 11.4-1: Design Response Spectrum

- \( T < T_s : S_s = S_{0s} (0.4 + 0.6 T / T_s) \)
- \( T_s \leq T \leq T_L : S_s = S_{0s} \)
- \( T_s < T \leq T_L : S_s = S_{PS} / T \)
- \( T > T_L : S_s = S_{PS} T_L / T^2 \)

Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE) Response Spectrum

The MCE Response Spectrum is determined by multiplying the design response spectrum above by 1.5.
Design Maps Detailed Report

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 (4)

Equation (11.8-1):

\[ \text{PGA}_\text{h} = F_{\text{PGA}} \times \text{PGA} = 1.000 \times 0.667 = 0.667 \text{ g} \]

Table 11.8-1: Site Coefficient \(F_{\text{PGA}}\)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE Geometric Mean Peak Ground Acceleration, PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA ≤ 0.10</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.667 g, \(F_{\text{PGA}} = 1.000\)

Section 21.2.1.1 — Method 1 (from Chapter 21 - Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 (5)

\[ C_{\text{AS}} = 1.020 \]

From Figure 22-18 (6)

\[ C_{\text{RI}} = 0.992 \]
Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF $S_{0s}$</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>$S_{0s} &lt; 0.167g$</td>
<td>A</td>
</tr>
<tr>
<td>$0.167g \leq S_{0s} &lt; 0.33g$</td>
<td>B</td>
</tr>
<tr>
<td>$0.33g \leq S_{0s} &lt; 0.50g$</td>
<td>C</td>
</tr>
<tr>
<td>$0.50g \leq S_{0s}$</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = I and $S_{0s} = 1.134$ g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF $S_{01}$</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>$S_{01} &lt; 0.067g$</td>
<td>A</td>
</tr>
<tr>
<td>$0.067g \leq S_{01} &lt; 0.133g$</td>
<td>B</td>
</tr>
<tr>
<td>$0.133g \leq S_{01} &lt; 0.20g$</td>
<td>C</td>
</tr>
<tr>
<td>$0.20g \leq S_{01}$</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = I and $S_{01} = 0.743$ g, Seismic Design Category = D

Note: When $S_{1}$ is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV, irrespective of the above.

Seismic Design Category = "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf
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 SSW'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 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. 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.