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

Preliminary Report of
Geotechnical Investigations &
Soil Infiltration Testing for WQMP-BMP Design
Proposed Beaumont Station Commercial Development
Planned Gas Station, Retails, Convenience Store & Restaurant
NWC Oak Valley Parkway & Golf Club Drive
Beaumont, California

APN: 400-530-006 & 07

Project No. 18059-F/BMP
December 19, 2018

Prepared for:

Gil Zulueta Mendoza & Associates
6185 Magnolia Avenue, #129
Riverside, California 92506

soilssouthwest@aol.com
Established 1984

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December 19, 2018. Project No. 18059-F/BMP

Gil Zulueta Mendoza & Associates
6185 Magnolia Avenue, #129
Riverside, California 92506

Attention: Mr. Gil Mendoza, PE

Subject: Preliminary Report of Geotechnical Investigation &
Soil Infiltration Testing for WQMP-BMP Design
Proposed Beaumont Gas Station, Retails, Convenience Store & Restaurant
NWC Oak Valley Parkway & Golf Club Drive, Beaumont, California

Reference: Preliminary Grading Plan by GZM Civil Engineers, dated September 7,2 018

Gentlemen:

Presented herewith are the Reports of Geotechnical Investigation and Soil Infiltration testing for WQMP BMP design for the site of the proposed commercial development to be located on the northwest corner of Oak Valley Parkway and Golf Club Drive, City of Beaumont, California. Based on the preliminary project information supplied, it is understood that the subject development, among others, will include a gasoline dispensing station with convenient store, along with multi-tenant retails and a restaurant.

Based on the test explorations completed it is our opinion that the site is underlain by upper dry, compressible and variable consistency fills of fine to medium coarse sands, overlying silty fine to medium coarse sand of moderate consistency. No shallow depth groundwater was encountered.

Based on review of the available published documents, it is our understanding that the site is not situated within an AP Special Studies Zone, and with groundwater at a depth in excess of 50 feet, the site is considered non-susceptible to soils liquefaction in event of a strong motion earthquake.

Compressible in nature, the near grade soils encountered are considered unsuitable for directly supporting structural loadings without excessive differential settlements to load bearing foundations and concrete slab-on-grade. When, however graded in form of subexcavations of the near grade soils and their replacement as engineered fills to a higher density, the structural fills thus placed should be considered adequate for the development proposed.

This report has been substantiated by subsurface explorations and mathematical analyses made in accordance with the generally accepted engineering principles, including those field and laboratory testing completed as described. We offer no other warranty, express or implied.

Respectfully submitted,
Soils Southwest, Inc.

Moloy Gupta, RCE 31708

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

John Flippin
Project Coordinator

soilssouthwest@aol.com
Established 1984

1.0 Introduction

This report presents the results of Geotechnical Evaluations and soils infiltration testing for WQMP-BMP detention basin design conducted for the site of the proposed commercial development to be located on the northwest corner of Oak Valley Parkway and Golf Club Drive, City of Beaumont, California.

The purpose of this evaluation is to determine the nature and engineering properties of the near grade and sub-grade soils, and to provide geotechnical recommendations for foundation design, slab-on-grade, retaining wall, paving, parking, site grading, utility trench excavations and backfill, and inspections during construction.

The recommendations contained reflect our best estimate of the soils conditions as encountered during test explorations completed at the locations as described. It is not to be considered as a warranty of the soils for other areas, or for the dissimilar imported fills when used during grading.

The preliminary recommendations supplied should be considered valid and applicable for the soils as encountered, as well as when the following conditions are fulfilled:

i. Pre-grade meeting with contractor, public agency and soils engineer,
ii. Excavated bottom inspections and verifications by soils engineer prior to backfill placement,
iii. Continuous observations and testing during site preparation and structural fill soils placement,
iv. Observation and inspection of footing trenching prior to steel and concrete placement,
v. Plumbing trench backfill placement prior to concrete slab-on-grade placement,
vi. On and off-site utility trench backfill testing and verifications, and
vii. Review of the précised grading and construction details prepared by others.

1.1 Proposed Development

Based on review of the preliminary project descriptions supplied, it is understood that, among others, the subject development will include (i) gas station with a convenient store and (ii) detached restaurant and multi-tenant retails. Conventional wood-frame and stucco construction with wall foundations and spread footings with concrete slab-on-grades are in preparing this report. Installation of on-site WQMP-BMP detention basin, paving, parking, driveways and off-site street widening anticipated to complete the project. Moderate site preparations and grading should be expected with the development planned. For preliminary analyses, structural loadings of 30 kips and 3 klf are assumed for isolated spread and load bearing wall foundations, respectively.

1.2 Site Description

The irregular shaped parcel of unknown acreage is bounded by Oak Valley Village Circle to the north, by Oak Valley Parkway to the south, by Golf Club Drive on the east, and by other unimproved vacant graded pads to the west. Overall vertical relief within the parcel is currently unknown, but sheet flow from incidental rainfall appears to flow towards the southwest. With the exception of a billboard near the south, no other significant features pertinent to the planned development were noted.

2.0 Scope of Work

Being beyond scope of work, no Geologic and/or Environmental Site Assessment is included. Reports on such will be provided on request.

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

- **Field Explorations**

  Field investigations included ten (10) geotechnical exploratory test borings and two (2) shallow depth 8" diameter test excavations using a Hollow-Stem Auger (HSA) drill-rig equipped for undisturbed soils sampling and Standard Penetration Testing (SPT). Approximate test excavation locations are shown on attached Plate A.

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

- **Laboratory Testing**

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

- In-situ moisture contents and dry density (ASTM Standard D2937),
- Maximum Dry Density-Optimum Moisture content (ASTM Standard D1557),
- Direct Shear (ASTM Standard D3080),
- Soil Consolidation (ASTM Standard D2435),
- Soil Sand Equivalent, SE (ASTM D2419), and
- Soil Grain size analysis (ASTM D422).

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

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

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

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

3.0 Existing Site Conditions

3.1 Subsurface Conditions

In general, the site is underlain by upper dry and compressible fills of silty fine to medium coarse sands with some gravels rock fragments, overlying dry to damp silty fine to medium coarse sands with pebbles and occasional rock fragments and small rocks to the maximum 31 feet explored. No shallow depth groundwater was encountered.

The near surface compressible soils existing as described should be considered unsuitable for directly supporting structural loadings or the new (imported) structural fill soils placement without excessive differential settlements to load bearing footings and concrete slab-on-grade. When, however, graded in form of subexcavations of the upper soils and their replacement as engineered fills as described herein, the structural pads thus constructed should be considered adequate for the development planned.

Laboratory shear tests conducted on the local undisturbed and on upper bulk soils remolded to 90% relative compaction indicate moderate shear strengths under increased moisture conditions. Results of the laboratory shear tests are provided in Plate B-1 of this report.

While soil consolidation tests conducted on the upper undisturbed soils indicate moderate potential for compressibility, results of the similar tests conducted on samples remolded to 90% indicate acceptable potential for hydro-consolidation under anticipated structural loadings. Results of the laboratory determined soils consolidation potential is shown on Plate B-2 in Appendix B.

Silty sandy in nature, the site soils are considered "very low" in expansion characteristic with an Expansion Index, EI, less than 20.

It is recommended that during and following mass grading completion, additional laboratory testing should be performed to verify the expansion potential for the soils in contact with concrete slab-on-grade and load bearing foundations.

3.2 Excavatibility

It is our opinion that grading and excavations required for the project may be accomplished using conventional heavy-duty construction equipment.

3.3 Groundwater

No groundwater was encountered within the maximum exploratory depth of 31 feet below grade.

It is our opinion that during construction, no special construction requirements including de-watering, etc should be expected. However, provisions should be considered for disposing of surface runoff away from structural pads once constructed.

Fluctuations in groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, altered natural drainage paths, and other factors not evident at the time of this investigation. The designer and contractor, however, should be aware of possibility of groundwater fluctuations while designing and during construction.

The following table describes the historical and the current groundwater levels as recorded in the nearest well as listed by the local reporting agency.

GROUNDWATER TABLE	
Reporting Agency	California Department of Water Resources Marcelo Montagna 2008 Maps http://wdl.water.ca.gov/waterdatalibrary/
Well Number	02S/01W-33M001S (northeast of site)
Well Monitoring Agency	5167
Well Location: Township/Range/Section	T2S-R01W-Section 33
Well Elevation:	2569
Current Depth to Water (Measured in feet)	338
Current Date Water was Measured	October 25, 1999
Depth to Water (Measured in feet) (Shallowest)	305
Date Water was Measured (Shallowest)	February 15,1989

3.4 Subsurface Variations

It is our opinion that variations in subsoils continuity and depths of subsoil deposits may be expected. Due to the nature and depositional characteristics of the soils underlying, care should be exercised in interpolating or extrapolating of the subsurface conditions existing in between and beyond the test explorations completed as described. Although not encountered, based on prior historical use of the property, presence of underlying buried utilities may be expected.

3.5 Soil Corrosivity Analyses

Since during mass grading, local surface soil matrix are expected to change considerably, no soil chemical; analysis is included at this time. It is recommended that following mass grading completion the representative site soils should be laboratory tested to determine pH, sulfate, chloride and resistivity. Results of such will be provided on request.

3.6 Faulting And Seismicity

3.6.1 Direct or Primary Seismic Hazards

Surface ground rupture along with active fault zones and ground shaking represent primary or direct seismic hazards to structures. Based on review of the CGS: Division of Mines and Geology El Casco Quadrangle Map dated June1,1995, it is our understanding that the site is not situated within an AP Special Studies Zone.

According to the current (2016) CBC, the site is considered to be within Seismic Zone 4. As a result, it is likely that during the life expectancy of the structures planned moderate to severe ground shaking may be anticipated.

3.6.2 Induced or Secondary Seismic Hazards

In addition to ground shaking, effects of seismic activity may include surface fault rupture, differential settlements, ground lurching and lateral spreading. Effects of such are discussed below.

3.6.2.1 Surface Fault Rupture

Based on review of the CGS: Division of Mines and Geology El Casco Quadrangle Map dated June1, 1995, it is our understanding that the site is not situated within an AP Special Studies Zone, where an earthquake fault passes through the site or its adjacent. Potential for surface rupture resulting from nearby fault movement is not known for certainty, it is our opinion that such potential, if any, should be relatively "low" considering the proximity of the nearest San Andreas Fault at about 5.6 miles away.

3.6.2.2 Flooding

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

3.6.2.3 Land-Sliding

Seismically induced landslides and other slope failures are common occurrences during or soon after and earthquake. With the near level existing and future structural pad(s) as planned, it is our opinion that the potential for seismically induced landslides may be considered as remote.

3.6.2.4 Lateral Spreading

Seismically induced lateral spreading involves lateral movement of soils due to ground shaking. Lateral spreading is demonstrated by near vertical cracks with predominantly horizontal movement of the soil mass involved. The topography of the subject site and the adjacent properties has a near-zero slope ratio. Accordingly, it is our opinion the potential for lateral spreading of the subject site is considered remote.

3.6.2.5 Liquefaction

Liquefaction is caused by build-up of excess hydrostatic pressure in saturated cohesion-less soils due to cyclic stress generated by ground shaking during an earthquake. The significant factors on which soil liquefaction potential depends include, among others, the soil type, soil relative density, intensity of earthquake, duration of ground-shaking and depth of groundwater.

With the historical groundwater table at a depth in excess of 50 feet as per the Department of Conservation Special Publication 117, along with the presence of underlying medium dense to dense sandy soils with high SPT blow counts, it is our opinion that site soil liquefaction susceptibility potential during an earthquake, should be considered "remote".

3.6.2.6 Settlement and Subsidence

With an earthquake magnitude of M=7.4 and ground acceleration of 0.565g, along with high SPT blow counts as recorded as described, it is our opinion that seismically induced ground settlements may be estimated to about ½- inch or less.

3.7 Seismic Design Parameters

The design spectrum was developed based on the 2016 CBC with site coordinates of 33.946959°N and -116.999985°W. Results of the seismic parameters are presented below.

3.8 Seismic Design Coefficients

Based on EQFAULT computer program, it is understood that the subject site is situated at about 5.6 miles from the San Jacinto;San Jacinto Valley Fault. For foundation and structural design, the following seismic parameters are suggested based on the current 2016 CBC.

Recommended values are based upon the USGS ASCE 7-10 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 available publications as currently published by the California Geological Survey and 2016 CBC.

TABLE 3.8A.1 Seismic Design Parameters

CBC Chapter 16	2016 ASCE 7-10 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	1.500 g
1613A5.3(2)	Site Class D / Seismic Coefficient, S_1	0.630 g
1613A5.3(1)	Site Class D / Seismic Coefficient, F_a	1.000 g
1613A5.3(2)	Site Class D / Seismic Coefficient, F_v	1.500 g
16A-37 Equation	Spectral Response Accelerations, $S_{Ms} = F_a S_s$	1.500 g
16A-38 Equation	Spectral Response Accelerations, $S_{M1} = F_v S_1$	0.945 g
16A-39 Equation	Design Spectral Response Accelerations, $S_{Ds} = 2/3 \times S_{Ms}$	1.000 g
16A-40 Equation	Design Spectral Response Accelerations, $S_{D1} = 2/3 \times S_{Ms}$	0.630 g

TABLE 3.8A.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>\=7.4
Peak Horizontal Ground Acceleration (PHGA)	0.565g

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 should be intended to resist total structural collapse due to the described PHGA of 0.565g or greater. However, during life time use of the structure built, it is our opinion that some structural damage may be anticipated requiring structural repairs. Use of flexible lifelines connections is suggested.

4.0 Evaluations and Recommendations

4.1 General Evaluations

Based on the field investigations, laboratory testing and subsequent engineering analysis completed at this time, it is our opinion that from geotechnical viewpoint, the site should be suitable for the development proposed, provided the recommendations presented are incorporated in final design and construction.

With the presence of the upper compressible dry, loose and old fill soils existing as described, it is our opinion that *no new structural fills or load bearing footings should be established bearing directly on the surface soils existing.* For adequate support, subgrade preparations should be considered, including subexcavations of the upper compressible old fills, followed by their replacement with engineered fills compacted to 90% or better.

Site preparations and grading should be performed in accordance with the current CBC and as per the general applicable grading recommendations as provided Section 5 of this report.

4.1.1 Preparations for Structural Pads

Based on field explorations and laboratory testing, it is our opinion that, in general, the project site consists of near surface minimum 3 to 5 feet of low-density compressible soils considered inadequate for directly supporting structural loadings without excessive differential settlements to footings and concrete slab-on-grade. For adequate structural support, it is suggested that structural pad preparations should include subexcavations of the near surface loose and compressible soils should be subexcavated to either (i) 5 feet below the present grade surface, or (ii) to the depth of the underlying moist and dense natural soils as approved by soils engineer during grading, whichever is greater. Site preparations should also include 6-inch scarification, pre-saturation, and recompaction prior to the excavated soils replacement in 6 to 8-inch thick vertical lifts compacted to minimum 90 percent. In general, a minimum 18-inch thick compacted fill mat blanket should be maintained below load bearing footing bottoms.

The subexcavation depths described should be considered as "preliminary". Localized additional subexcavations may be required within areas underlain by undocumented old fills, buried utilities and abandoned sewer and/or buried septic systems and others. Actual subexcavation depths, however, should be determined by soils engineer during grading.

It is unknown if imported fills soils will be required for the finished pad grades proposed. Imported fill soils, if required, should be free of debris, roots, organic and clay similar to or better than the local soils exposed as described. Recommendations for General Earthwork are enclosed in Section 5 of with this report.

4.1.2 Structural Fill Soils Requirements

The on-site soils free of organic, debris and rocks larger than 8-inch in diameter, should be considered suitable for re-use as structural backfills. Imported fills, if required, should have the following geotechnical characteristics:

Expansion Index, EI	Less than 20
Plasticity Index, PI	Less than 15
Percent Passing 200 sieve	Less than 20
Maximum Rock Size	8-inch

Prior to importation, representative imported soils should be verified and approved by soils engineer.

4.2 Spread Foundations

The structures planned may be supported by continuous wall and/or isolated spread footings founded exclusively into engineered fills of local sandy soils or similar imported fills compacted to minimum 90%. Use of footings straddling over cut/fill transition, shall be avoided.

Since finish pad grade elevations are currently unknown, for adequate structural bearing, the following general recommendations are supplied:

(a) For the pads proposed following cuts to current grades, it is recommend that following such cuts, the cut surface should be further subexcavated to a minimum vertical depth equal to the planned footing embedment plus 24-inch. The site grading should also include local soils replacement as engineered fills compacted to 90% or better.

(b) Within low-lying areas requiring new fill soils placement for over the current grade surface, following removal of near surface loose and compressible soils to full depth as required to expose the underlying moist dense subgrades, site grading should include further scarification, moisturization and recompaction, followed by thye local excavated soils placement for structural support compacted to 90%. For adequate support, compacted fill mat thickness below foundation bottoms should be at least 24-inch.

(c) Within areas of cut/fill transition pads, if any, it is recommended that within areas requiring cuts, the cut portions of the pad should be further subexcavated and replaced with engineered fills, the overall depth of below foundation bottoms should be at least 24-nch. Within areas requiring fill soils to proposed finished grade, depth of engineered fills below foundation bottoms should similarly be 24-inch thick as described earlier.

Foot-print areas described should be defined as the area extending from the outer edge of the planned structure, plus, either to:

(i) a distance of 5 feet, or
(ii) to the nearest property line, or
(iii) to the nearest constraint, such as existing foundations, or
(iv) as determined by soils engineer during grading.

Supplemental grading recommendations should be warranted following site topographic and grading plan review.

With the silty gravely sandy nature of the local soils, it is recommended that excavated footing trenches should be sufficiently "moistened" immediately prior to steel and concrete placement.

For design, allowable vertical soil bearing capacity may be estimated from the following equations:

Continuous Footing: $q_{allowable} = 2250 + 73d + 300b$
Isolated Square: $q_{allowable} = 3000 + 73d + 120b$, where

$q_{allowable}$ = allowable soil vertical bearing capacity, in psf.
d= footing depth, min. 24-inch, b = footing width, min. 15-inch

For the structures planned, footings should be sized to minimum 15-inch wide, embedded to minimum 18-inch below the lowest adjacent final grade surface or as designed by the structural engineer based upon seismic design parameters and horizontal peak ground acceleration (PGA) as provided in this report. The above soil bearing capacities may be increased by about 300 psf for each additional footing depth in excess of the minimum as recommended.

Total maximum vertical bearing capacity is recommended not to exceed 3000 psf and 4000 psf for continuous wall and isolated footings, respectively. 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.

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

Based on the laboratory determined soils consolidation characteristics, settlements to 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. Over a 40-ft. span, estimated total and differential settlements are about 1 and 1/2-inch, respectively. When gravelly sandy soils are used, most of the elastic deformations, however, should be expected to occur during construction.

It is recommended that excavated footing trenches should be verified, tested and certified by soils engineer prior to actual concrete placement. Soils Southwest, Inc. will assume no responsibility for any structural distress in event excavated footings is poured without verifications as suggested.

4.3 Concrete Slab-on-Grade

No concrete slabs, sidewalks and flatworks should be placed bearing directly on the surface soils existing. The prepared subgrades to receive footings should be adequate for concrete slab-on-grade placement. For normal use, considering the PGA described, for normal use 4-inch thick concrete slabs reinforced with #3 rebar at 18-inch o/c is recommended, Actual slab thickness, however, should be designed by project structural engineer based upon structural loadings, along with the seismic design parameters and horizontal peak ground acceleration (PGA) as provided in this report. Additionally, concrete slabs must maintain positive contact with footings as designed by the project structural engineer. For driveways, concrete slabs should be 5-inch thick, placed over local or similar imported gravelly sandy soils compacted to at least 95%. Driveway slab reinforcing and construction and expansion joints etc. should be incorporated if required by the project structural engineer.

Within moisture sensitive areas, concrete slabs should be underlain by 2-inch of compacted clean sand, followed by 6-mil thick vapor barrier such as commercially available StegoWrap or its similar. The gravelly sands used should have a Sand Equivalent, SE, of 30 or greater.

Subgrades to receive concrete should be adequately "'dampened" (not flooded) 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 minimum percent compaction as recommended.

4.3.1 Concrete Curing and Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade.

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, it is suggested that construction/expansion joints should be considered at spacing not exceeding 24 to 36 time the slab thickness. Shorter distance between joint spacing would provide greater crack control. Joints at curves and angle points are suggested, as recommended by structural engineer.

4.4 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 200 pounds per square foot per foot of depth may be assumed for the sides of foundations poured against compacted fills. The maximum lateral passive earth pressure is recommended not to exceed 2000 pounds per square foot.

For design, lateral pressures from local soils or its similar or better imported fills used as level backfill may be estimated from the following equivalent fluid density:

Active:	40 pcf
At Rest:	60 pcf

4.5 Shrinkage and Subsidence

It is our opinion that the local or similar imported fills when used in grading may be subjected to a volume change. Assuming a 95% relative compaction, and assuming an overexcavation and re-compaction depth of about 5 to 8 feet, such volume change for current grades due to shrinkage may be on the order of 8 to 10 percent. fill placement. For estimation purpose, 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 encountered at about 8 feet and below.

4.6 Construction Consideration

4.6.1 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.

4.6.2 Supported Excavations

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

4.7 Site Preparations

The site preparation should include subexcavation of the upper loose and disturbed soils, stock-piling, moisturization and/or aeration to 3% to 5% over optimum moisture content. Site preparation should also include re-placement of the excavated soils and other approved imported fills compacted to 90 percent or better. Such earth work should be in accordance with the applicable grading recommendations provided in the current CBC/UBC and as recommended in Section 5.0 of this report.

4.8 Soil Caving

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

4.9 Structural Pavement Thickness

Flexible Asphalt Paving: Based on estimated Traffic Index (TI), laboratory determined soil Sand Equivalent and on soil the R-value of 45 for the local soils existing as encountered, for preliminary estimation, the following flexible pavement sections may be considered.

Service Area	Traffic Index, TI	Pavement Type	Paving Thickness (inch)
On-Site paving/parking for commercial/industrial vehicular traffic, including fire engine and garbage truck etc.	6.5	ac over Cl. II base	4.0 ac over 6.0 base

Within paving areas subgrade soils should be scarified to 18-inch, moisture conditioned from 3% to 5% percent over optimum, and recompacted to at least 95% to soil's Maximum Dry Density as determined by the method ASTM D1557-91, or other approved test procedures. The asphalt and base materials used should be similarly compacted to minimum 95%.

Since the site preparations and grading are expected to generate large quantities of old asphalt and concrete derived from removals of existing covering, it is our opinion that the excavated asphalt and concrete may be re-used as Processed Miscellaneous Base (PMB), meeting the minimum gradation requirements as described in Section Standard 200-2.5 of Green Book.

The pavement evaluations are based on estimated Traffic Index (TI) as shown, and on soil R-value of 60. 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, paving thickness should be determined for actual implementation on site

.

Concrete Paving, if considered, should be at least 6-inch thick, reinforced with #4 rebar at 18" o/c, placed directly over the local sandy gravelly soils compacted to minimum 95%. Actual paving thickness should be supplied by the project structural engineer based on soil Subgrade Reaction, ks, of 250 pcf.

4.10 Retaining Wall (if needed)

Based on the project information supplied, it is understood that major retaining structures may be planned along and adjacent to the Marshall Creek at the west. Although type of wall proposed is currently unknown, for conventional concrete retaining structure, if planned, the following equivalent fluid density may be considered for preliminary design purpose, provided the imported fills are considered similar to the local soils or its equivalent or better.

Slope Surface of Retained Material (horz. to vert.)	Equivalent Fluid Density (pcf) Imported Clean Sand	Local Site Soil
Level	30	40
2:1	35	60

For design, retaining wall foundation bearing capacity may be estimated from the bearing capacity equations described earlier.

The recommended lateral pressures do not include any surface load surcharge. Use of heavy equipment near retaining wall may develop lateral pressure in excess of the parameters described above. Walls adjacent to traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, which is a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back ten feet from the wall, the traffic surcharge may be neglected.

Installation of 'french-drain' behind retaining walls is recommended to minimize water pressure build-up. Use of impervious material is preferred within upper 18 inches of the backfill placed.

Backfill behind retaining wall should be compacted to a minimum 90 percent relative laboratory Maximum Dry Density as determined by the ASTM D1557 test method. Flooding and/or jetting behind wall should not be permitted. Local sandy soils or its equivalent or better imported fills may be considered as backfill. Adjacent to existing creek, retaining wall foundations should be deepened so as to maintain a minimum 10 feet lateral setback measuring horizontally in between the outer face of the footings to the creek embankment slope surface.

As an alternative to conventional concrete or block retaining structure, from economical and aesthetic viewpoint, it is our opinion that “keystone” or similar segmented retaining structures may be considered supporting reinforced earth backfills. Detailed recommendations of such will be provided on request. For adequate horizontal setback, crib wall footings may be deepened by susing “gabions’. Detailed recommendations on such will be supplied on request.

4.11 Utility Trench Backfill

Utility trench backfill within the structural pads, gas station and beyond should be placed in accordance with the following recommendations:

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

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

4.12 Pre-Construction Meeting

It is recommended that no grading operation should be commenced without the presence of a representative of this office. An on-site pre-grading meeting should be arranged in between soils engineer, grading contractor, project civil engineer, local governing agencies and others prior to any construction.

4.13 Seasonal Limitations

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

4.14 Planters

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

4.15 Landscape Maintenance

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

4.16 Observations and Testing During Construction

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.

4.17 Plan Review

In absence of site-specific detailed development plan and the soils types that will be imported for backfills, the recommendations supplied should be considered as “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.

4.18 Recommendations for On-Site WQMP-BMP Storm Water Infiltration System Design

Two (2) infiltration testing are performed at 5 feet below the current grades using the standardized "falling-head" test converted to infiltration rate as per the guidelines of the Table 1, Infiltration Basin Option 2 of test procedures using Porchet methods as described in the Riverside County-Low Impact Development (LID) BMP design handbook. Approximate test locations are as selected by the project civil engineer are as shown on the attached Plate 1.

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

Based on the field infiltration testing completed the observed acreage infiltration rate is ***5.15 in/hr.*** For design, it is suggested that, use of an appropriate factor of safety should considered to the observed rate for design to account for long-term saturation, inconsistencies in subsoil conditions, potential for silting and lack of maintenance.

4.18.1 Excavated Test Borings

For soil infiltration testing at the locations as shown on the accompanying sketch, two (2) test borings (P-1 & P-2) were made, each advanced to 5 feet below the current grade. Water used during percolation testing was supplied by using a portable water tank.

4.18.2 Methodology and Test Procedures

Equipment Set-Up (Post-Excavation)

Following test boring completion, each of the test holes were fitted with perforated pvc pipes backfilled with 6-inch thick crushed rock at the bottom to minimize potentials for scouring and caving. Prior to actual testing, each excavated test holes were backfilled with water to determine test intervals that will be used during testing.

To determine test intervals, in two consecutive readings, since 6 inches or more of water seeped away in less than 25 minutes, subsequent six percolation testing were performed at 10 minute time intervals for one hour. Testing included water placement up to about 36 inches below the existing grade surface.

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

4.18.3 Infiltration Test Results

Based on the soils infiltration testing completed at the test locations and at the test depth as described, the observed soil percolation rates are 3.91"/hr. and 6.38 "/hr. for the test locations P-1 & P-2 described.

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 Table II below.

For design, it is suggested that, use of an appropriate factor of safety as selected by the design engineer should be considered to the observed rate described.

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	Δ_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	60	10	12	60	48.0	34.0	14.0	41.0
P-2	60	10	12	60	48.0	27.0	21.0	37.5

	Infiltration Rate (It)=ΔH60r/Δt(r+2Havg)		
	A	B	C
Test No.	ΔH60r	Δt(r+2Havg)	A/B=in/hr
P-1	3360	860	3.91
P-2	5040	790	6.38

Observed average infiltration rate: **5.15 in/hr**.

5.0 Earth Work/General Grading Recommendations

Site preparations and grading should involve overexcavation and replacement of local soils as structural fill compacted to 90% or better.

Structural Backfill:

During grading, excavated site soils or its equivalent or better imported fills, should be considered suitable for reuse as backfill material. Loose soils, formwork and debris should be removed prior to backfilling the 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. Recommendations for placement and densification of pea gravel or other special backfill can be provided during construction.

Site Drainage:

Adequate positive drainage should be provided away from the structure to prevent water from ponding and to reduce percolation of water into backfill. A desirable slope for surface drainage is 2 percent in landscape areas and 1 percent in paved areas. Planters and landscaped areas adjacent to building perimeter should be designed to minimize water filtration into subsoils. Considerations should be given to the use of closed planter bottoms, concrete slabs and perimeter subdrains where applicable.

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 placed and compacted in accordance with the following grading specifications.

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 recommended compaction for fill to support foundations and slab-on-grade is 90% of soil's Maximum Dry Density at or near Optimum Moisture Content. 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 compacted to at least 90% of the maximum density for the material used.

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

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

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

8. No rocks over six inches in diameter shall be permitted to use as a grading material without prior approval of the 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. Any and all utility trenches at depth as well as cesspool and abandoned septic tank within building pad area and beyond, should either be completely excavated and removed from the site, or should be backfilled with gravel, slurry or by other material, as approved by soils engineer.

11. Any and all import soils if required during grading should be equivalent to the site soils or better. Such should be approved by the soils engineer prior to their use.

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

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

6.0 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.

If during construction, the subsoils exposed appear to be different from those as used during this evaluations, this office should be notified to consider any possible need for modifications to the design parameters described.

Recommendations provided are based on the assumptions that structural footings will be established exclusively into compacted fills of the local soils or its equivalent or better. No footings and/or slabs should be 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. Footing excavations should be inspected prior to steel and concrete placement to ensure that foundations are founded into satisfactory soils and excavations are free of loose and disturbed materials.

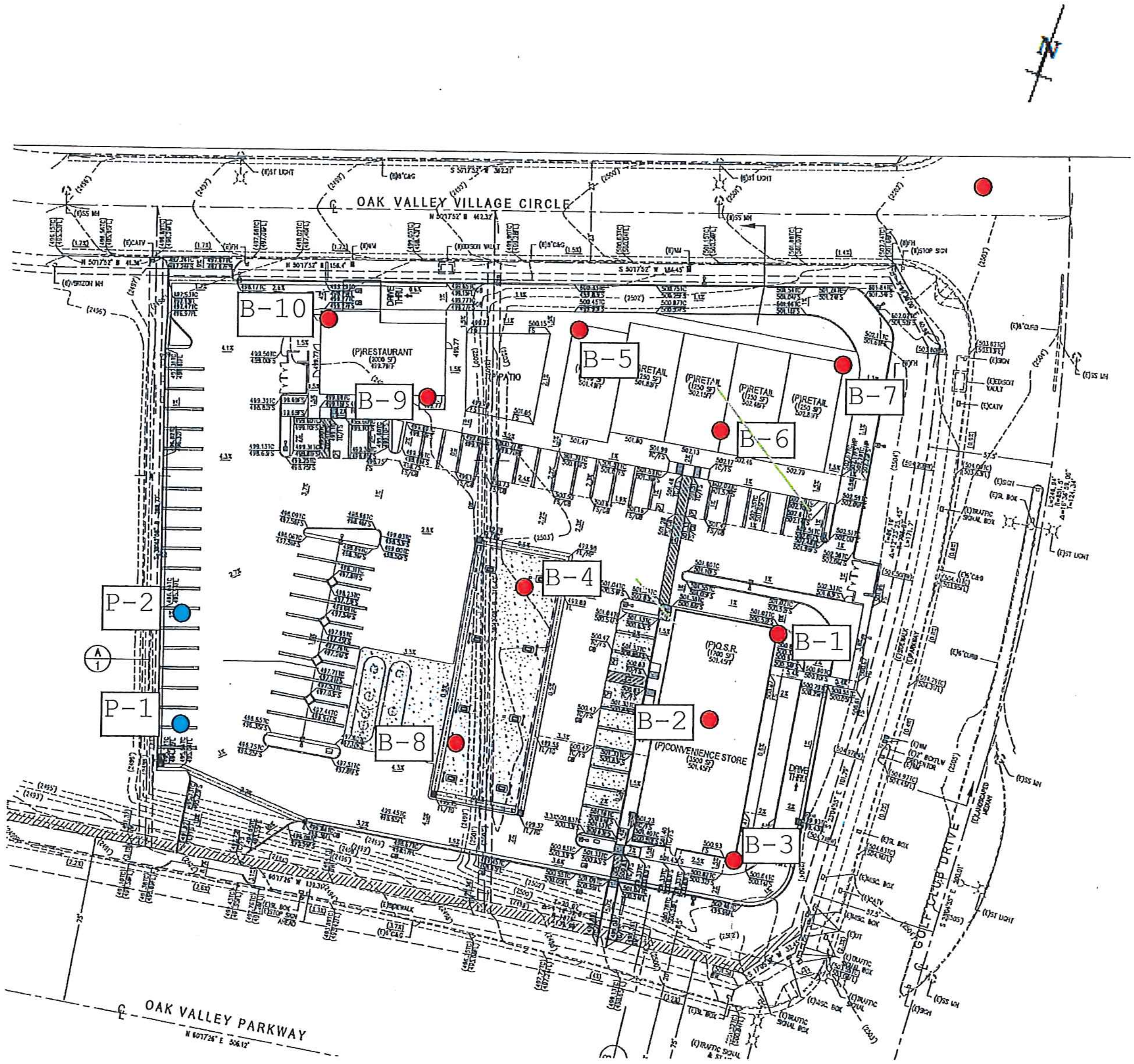
A pregrading meeting between grading contractor and soils engineer is recommended prior to construction preferably at the site, to discuss the grading procedures to be implemented and other requirements described in this report to be fulfilled.

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 during construction, our professional liability and responsibility shall be limited to the extent that Soils Southwest, Inc. would not be the geotechnical engineer of record. In addition, a Letter of Transfer of Responsibility will be required indemnifying Soils Southwest, Inc. from any liability for structural distress that may arise during lifetime use of the development planned.

PLOT PLAN AND TEST LOCATIONS
(Not to Scale)



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

7.0 APPENDIX A

Field Explorations

Field evaluations included site reconnaissance and seventeen (10) soil test borings and two (2) WQMP-BMP infiltration test borings using a hollow-stem auger drill-rig. During site reconnaissance, the surface conditions were noted and test exploration 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.

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

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

LOG OF BORING B-1

Project: Beaumont Gas Station **Job No.:** 18059-F
Logged By: John F. **Boring Diam.:** 8" HSA **Date:** 11-13-18

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			tilled weeds
								SAND- engineered fill, light yellow, silty, fine, scattered rock and cobbles, dry
	California sampler	5.4	121.5	96.4	SP-SM		5	- color change to yellow to gray-brown, slightly silty, fine to medium, pebbles, rock fragments, occasional 1/2" rock - dense to very dense
21	Standard penetration test				SM-ML		10	- color change to light yellowish brown, silty, fine, occasional pebbles, dry to damp - medium dense
							15	
					SP-SM			- color change to orangish light brown, slightly silty, fine to medium, pebbles, scattered rock fragments
50	Standard penetration test				SP		20	- color change to light yellowish brown, occasional rocks and cobbles, fine to medium coarse, dry to damp, very dense
							25	
							30	
30	Standard penetration test				SM-ML			- silty, fine, 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 commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, California

Plate #

California sampler

Standard penetration test

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

LOG OF BORING B-2

Project: Beaumont Gas Station **Job No.:** 18059-F
Logged By: John F. **Boring Diam.:** 8" HSA **Date:** 11-13-18

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			gravels, tilled weeds
								SAND - engineered fill, light yellowish brown, silty, fine, pebbles, rock fragments, scattered rock, dry
	California sampler	5.4	125.7	99.8	SM			- color change to orangish light brown, silty, fine to medium, pebbles, occasional rock fragments and rocks 1/2"
30	Standard penetration test						5	- dense
	California sampler	6.1	114.6	90.9				- color change to light yellowish brown, silty, fine to medium, pebble, occasional rock fragments and 1/2" rock, occasional d.g. origin material
							10	
							15	
32	Standard penetration test				SM-ML			- color change to light brown, silty, fine occasional pebble and rock fragments
								- End of test boring @ 16 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 commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, California

Plate #

California sampler   Standard penetration test

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

LOG OF BORING B-3

Project: Beaumont Gas Station		**Job No.:**	18059-F
Logged By: John F.	**Boring Diam.:** 8" HSA	**Date:**	11-13-18

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			gravels, tilled weeds
46	Standard penetration test							SAND - engineered fill, light yellowish brown, silty, fine, pebbles, rock fragments, scattered rock, dry
					SM			- color change to light brown, dense to very dense rock fragments and rocks 1/2"
							5	- color change to light yellowish gray-brown silty, fine to medium, pebbles, dry
30	Standard penetration test				SM-ML			- color change to yellow to greenish yellow brown, silty, fine, scattered pebbles, rock fragments, damp, dense
	California sampler	6.5	107.1	85	SM		10	- color change to light yellowish brown, silty, fine to medium, pebble, occasional rock fragments and 1/2" rock, occasional d.g. origin material
							15	- End of test boring @ 11.0 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 commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	**Plate #**

■ California sampler ◩ Standard penetration test

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

LOG OF BORING B-4

Project: Beaumont Gas Station | **Job No.:** 18059-F
Logged By: John F. | **Boring Diam.:** 8" HSA | **Date:** 11-13-18

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			gravels, tilled weeds
								SAND - engineered fill, light yellowish brown, silty, fine, pebbles, rock fragments, scattered 1/2" rock, dry
							5	
32	Standard penetration test				SM-ML			- color change to light brown, damp, dense
	California sampler	6.5	125.4	99.5	SP			- fine to medium, pebble, rock fragments and some d.g. origin material, damp
							10	
							15	
32	Standard penetration test				SM-ML			- silty, fine, scattered pebble and rock fragments, damp
								- End of test boring @ 16 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 commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, California

Plate #

California sampler   Standard penetration test

Soils Southwest, Inc.
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Colton, CA 92324
(909) 370-0474 Fax (909) 370-3156

LOG OF BORING B-5

Project: Beaumont Gas Station | **Job No.:** 18059-F
Logged By: John F. | **Boring Diam.:** 8" HSA | **Date:** 11-13-18

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			tilled weeds
								SAND - engineered fill sands, yellowish brown, silty, fine, pebbles, rock fragments, occasional rock 1"
							5	
	California sampler	5.8	119.3	94.7	SM			- color change to light brown, silty, fine to medium, pebbles, occasional rock fragments 3/4", very dense
							10	
25	Standard penetration test							- color change return to yellowish brown, pebbles, rock fragments, medium dense damp
							15	
							20	
40	Standard penetration test							- silty, fine to medium coarse, pebbles, rock fragments, rock 1"
							25	
							30	
30	Standard penetration test				SM-ML			- color change to orangish brown, silty, fine, scattered pebbles and rock fragments dense, damp
								- 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 commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, California

Plate #

California sampler   Standard penetration test

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

LOG OF BORING B-6

Project: Beaumont Gas Station		**Job No.:**	18059-F
Logged By: John F.	**Boring Diam.:** 8" HSA	**Date:**	11-13-18

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			some gravels, tilled weeds
								SAND - engineered fill, light yellowish brown, silty, fine, pebbles, rock fragments, occasional 1/2" rock, dry
					SM		5	- fine to medium, pebbles, occasional rock fragments
60	Standard penetration test				SM-ML			- color change to gray brown, silty, fine, pebbles, rock fragments, scattered rock damp
	California sampler	7.0	115.4	91.6				- color change to light yellowish brown, silty, fine, damp
					SP		10	- fine to medium, pebble, occassional rock fragments and 1/4" rock, and some d.g. origin material, damp
24	Standard penetration test				SM-ML		15	- silty, fine
	California sampler	8.9	113.3	90.0	SM			- color change to orangish light brown, silty, fine to medium, pebbles and scattered rock fragments and 1/4" rock
								- End of test boring @ 16 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 commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	**Plate #**

California sampler   Standard penetration test

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

LOG OF BORING B-7

Project: Beaumont Gas Station		Job No.: 18059-F
Logged By: John F.	Boring Diam.: 8" HSA	Date: 11-13-18

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			gravels, tilled weeds
								SAND - engineered fills, light yellow brown, silty, fine, pebbles, rock fragments dry
		4.5	116.8	92.7	SP		5	- color change to light brown, fine to medium, pebbles, occasional rock fragments and 1/4" rock
					SM-ML			- color change to yellow, silty, fine, scattered pebbles and rock fragments,
					SP		10	- fine to medium coarse, occasional rock 1"-2"
26					SM-ML			- silty, fine, medium dense to dense, damp
							15	- color change to brown
							20	- color change to yellow
38					SP-SM			- color change to light yellowish gray-brown gravely, slightly silty, fine to medium coarse, dense, dry
								- End of test boring @ 21.0 ft. - no bedrock - no groundwater
							25	
							30	

	Site Location	Plate #
Groundwater: n/a Approx. Depth of Bedrock: n/a Datum: n/a Elevation: n/a	proposed commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	

California sampler   Standard penetration test   Bulk/Grab sample

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

LOG OF BORING B-8

Project: Beaumont Gas Station		**Job No.:**	18059-F
Logged By: John F.	**Boring Diam.:** 8" HSA	**Date:**	11-13-18

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			gravels, tilled weeds
								SAND - engineered fills, light yellow brown, silty, fine, pebbles, rock fragments dry
41	Standard penetration test				SP-SM		5	- color change to grayish light brown, slightly silty, fine to medium, pebbles, rock fragments, and scattered rocks, damp dense
25	Standard penetration test							- color change to yellow brown, silty, fine damp, medium dense to dense
	California sampler	5.4	110.9	88.0	SM		10	- color change to orangish light brown, silty, fine to medium, pebbles, damp
								- End of test boring @ 11.0 ft. - no bedrock - no groundwater
							15	
							20	
							25	
							30	

Groundwater: n/a **Approx. Depth of Bedrock:** n/a **Datum:** n/a **Elevation:** n/a	**Site Location** proposed commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	**Plate #**

Standard penetration test California sampler

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

LOG OF BORING B-9

Project: Beaumont Gas Station		**Job No.:**	18059-F
Logged By: John F.	**Boring Diam.:** 8" HSA	**Date:**	11-13-18

Standard Penetration (Blows per Ft.)	Water Content in %	Dry Density in PCF	Percent Compaction	Unified Classification System	Depth in Feet	Description and Remarks
				FILL		gravels, tilled weeds
						SAND - engineered fills, light yellow brown, silty, fine, with gravels
						- (Max Dry Density = 126 @ 9.5%)
				SP-SM	5	- slightly silty sands with gravels (pebbles and small rocks), fine to medium coarse, dry, dense
45				SP		- traces of silts, fine to medium coarse, rock fragments, dry
	7.7	109.3	86.75	SM	10	- color change to orangish light brown, silty, fine to medium, occasional rock fragments and 1" rock, d.g. origin, dense, damp material.
				SM-ML		- color change to light brown, silty, fine damp
					15	- color change to reddish light brown, silty fine,
28				SM	20	- color change to light yellowish brown, silty, fine to medium, occasional pebbles, medium dense to dense, damp
				SP	25	- color change to light gray-brown, gravely, medium to medium coarse, rock fragments,
42				SM-ML		- color change to orangish light brown, silty fine, damp
					30	- End of test boring @ 26.0 ft. - no bedrock - no groundwater

Groundwater: n/a **Approx. Depth of Bedrock:** n/a **Datum:** n/a **Elevation:** n/a	**Site Location** proposed commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	**Plate #**

Bulk/Grab sample   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-10

Project: Beaumont Gas Station		Job No.: 18059-F
Logged By: John F.	Boring Diam.: 8" HSA	Date: 11-13-18

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			gravels, tilled weeds
								SAND - engineered fills, light yellow brown, silty, fine, with gravels, dry
43	Standard penetration test				SM			- color change to grau brown, fine to medium, pebbles, occasional rock fragments, dry to damp and 1/4" rock
	California sampler	6.1	120.5	95.6			5	- color change to yellow light brown, silty, fine to medium, pebbles, rock fragments, and rock 1"
							10	
28	Standard penetration test							- color change to yellowish light brown, silty, fine to medium coarse, pebbles, rock fragments, medium dense to dense, dry to damp
								- End of test boring @ 11.0 ft. - no bedrock - no groundwater
							15	
							20	
							25	
							30	

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

Site Location
proposed commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, 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 P-1

Project: Beaumont Gas Station		Job No.: 18059-F
Logged By: John F.	Boring Diam.: 8" HSA	Date: 11-13-18

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
								tilled weeds
					FILL			SAND - engineered fill sands, yellow brown, silty, fine, gravels, dry
								- scattered cobbles
					SM-ML		5	- color change to gray brown
								- End of infiltration test boring @ 5.0 ft. - no bedrock - no groundwater - 3" perforated sleeved pvc pipe installed with gravel at bottom
							10	
							15	
							20	
							25	
							30	

Groundwater: n/a Approx. Depth of Bedrock: n/a Datum: n/a Elevation: n/a	**Site Location** proposed commercial development Oak Valley Parkway and Gold Club Drive Beaumont, California	**Plate #**

California sampler   Standard penetration test   Bulk/Grab sample

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

LOG OF BORING P-2

Project: Beaumont Gas Station		**Job No.:** 18059-F
Logged By: John F.	**Boring Diam.:** 8" HSA	**Date:** 11-13-18

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			tilled weeds
								SAND - engineered fill sands, yellow brown, silty, fine, with gravels, dry
					SP-SM		5	- color change to gray-brown, slightly silty fine to medium, pebbles, rock fragments, occasional rock 1/2'-1", damp
								- End of infiltration test boring @ 5.0 ft. - no bedrock - no groundwater - 3" perforated pvc pipe installed with gravel at bottom
							10	
							15	
							20	
							25	
							30	

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

Site Location
proposed commercial development
Oak Valley Parkway and Gold Club Drive
Beaumont, California

Plate #

California sampler | Standard penetration test | Bulk/Grab sample

KEY TO SYMBOLS

Symbol Description

Strata symbols

Fill

Poorly graded sand with silt

Poorly graded silty fine sand

Poorly graded sand

Silty sand

Soil Samplers

California sampler

Standard penetration test

Bulk/Grab sample

Notes:

1. Exploratory borings were drilled on 11-13-18 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.

8.0 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.

Potential Expansion (ASTM Standard D4829-88)

Silty sandy in nature, the site soils are considered 'very low' in expansion characteristic. Supplemental testing for soil expansion should be performed following mass grading completion.

Laboratory Test Results

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

Test Boring No.	Sample Depth, ft.	Dry Density, pcf.	Moisture Content, %
1	5.0	121.5	5.4
2	3.0	125.7	5.4
2	8.0	114.6	6.1
3	10.0	107.1	6.5
4	7.0	125.4	6.5
5	5.0	119.3	5.8
6	8.0	115.4	7.0
6	15.0	113.3	8.9
7	3.0	116.8	4.5
8	10.0	110.9	5.4
9	8.0	109.3	7.7
10	5.0	120.5	6.1

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

Sample Location	Max. Dry Density, pcf	Opt. Moisture (%)
(A) B-9 @ 0-5 ft. (light orangish brown, silty, fine to med. occasional pebbles & rock 1.5")	126.0	9.5

C. Table III: Sand Equivalent

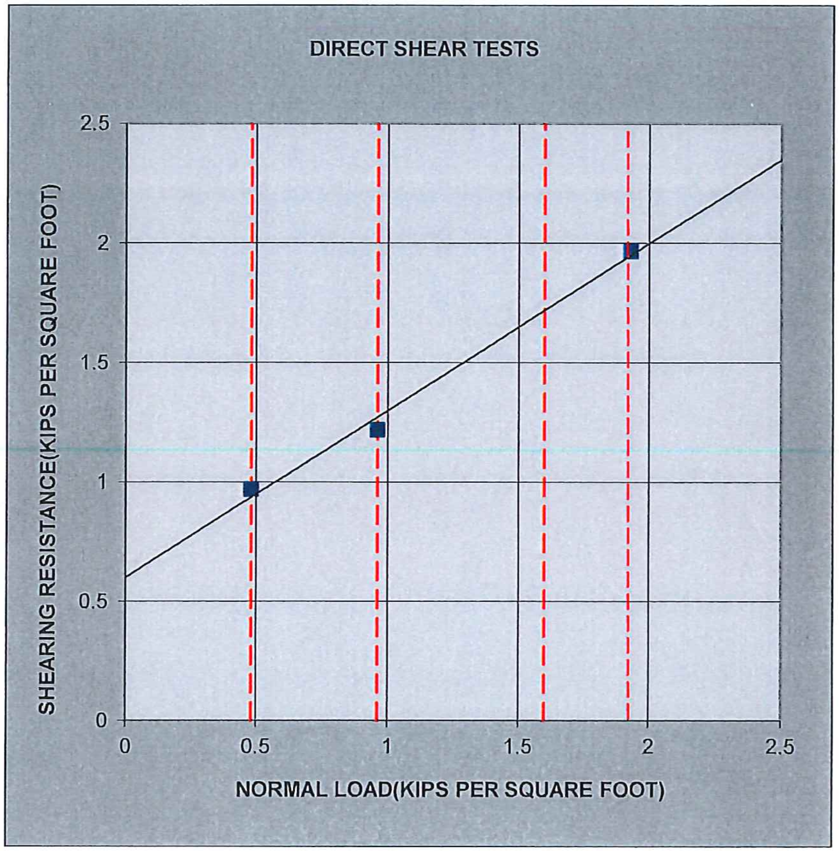
Sample Location @ depth, ft.	Sand Equivalent Average
B-9 @ 0-5	39.55

D. Table IV: Consolidation (D2835)

Boring B #	Depth (ft.)	Consolidation prior to saturation (@ 2 kips)	Hydro collapse (%)	Total Consolidation (%@ 8 kips) (saturated)
1 (remolded)	0 - 5	0.7	0.1	1.8
1 (undisturbed)	5.0	0.9	0.7	2.9
6 (undisturbed)	8.0	0.7	0.4	3.3

E. Table V: Direct Shear (ASTM D3080)

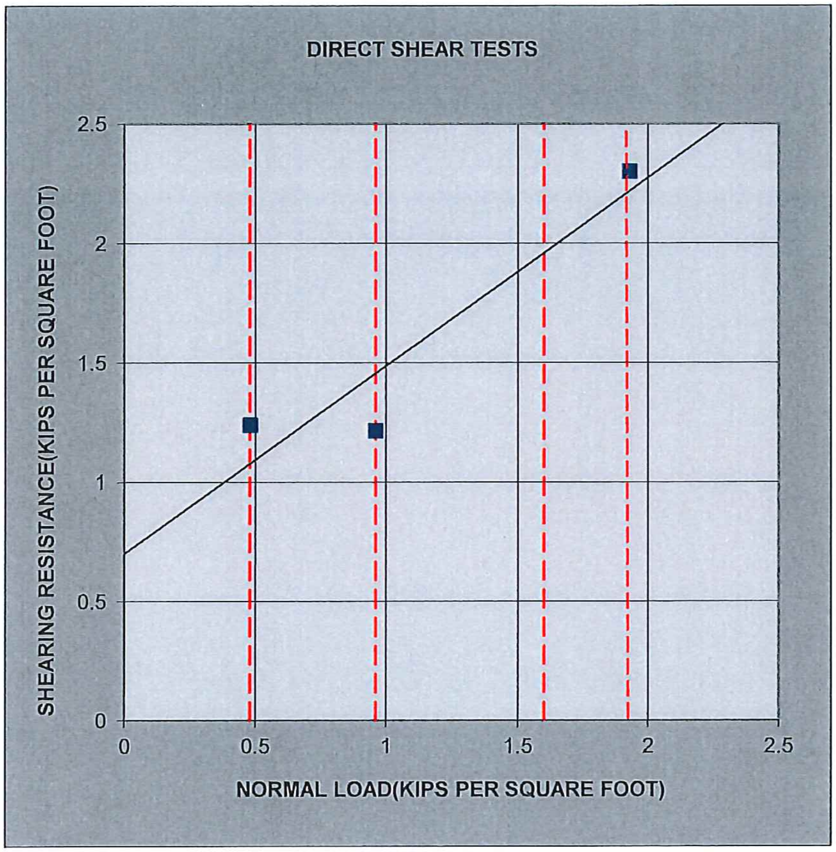
Test Trench & Sample Depth	Test Condition	Cohesion (PSF)	Friction (Degree)
B-9@ 0-5 ft	Remolded to 90%	600.0	34.96
B-5 @ 5.0	Undisturbed	700.40	38.22
B-2 @ 8.0	Undisturbed	500.74	30.84



SYMBOL	LOCATION	DEPTH (FT)	TEST CONDITION	COHESION (psf)	FRICTION (degree)
■	B-9	0 to 5	Remolded to 90%	600.02	34.96

Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway and Golf Club Drive Beaumont, California	PROJECT NO.	18059-F
	PLATE	B-1

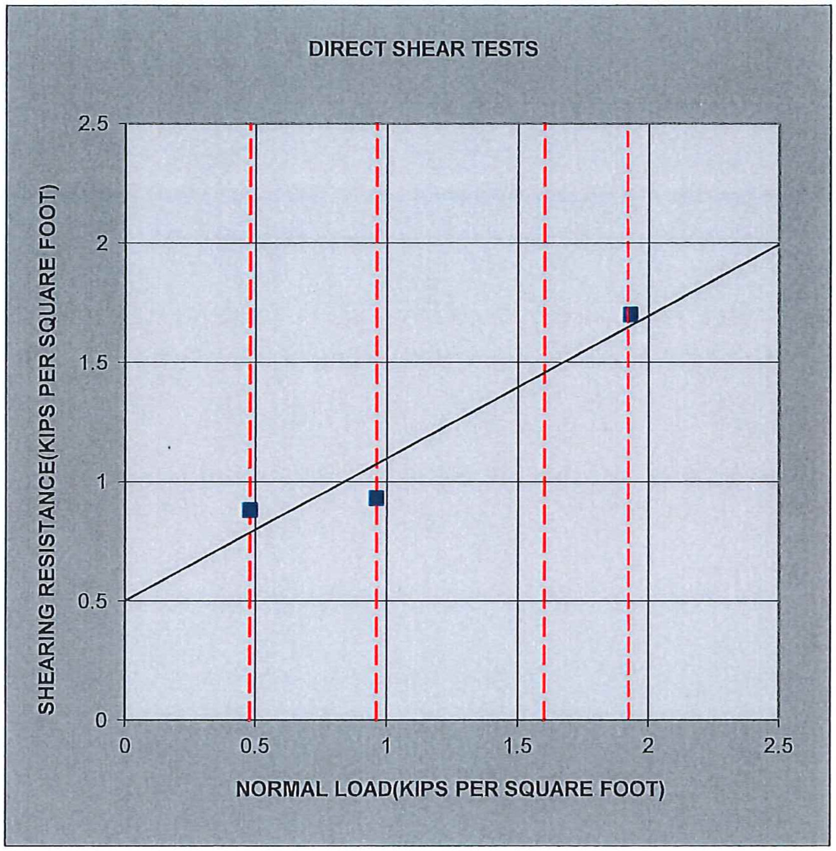
SOILS SOUTHWEST, INC.
Consulting Foundation Engineers



SYMBOL	LOCATION	DEPTH (FT)	TEST CONDITION	COHESION (psf)	FRICTION (degree)
■	B-5	5.0	Undisturbed	700.40	38.22

Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway and Golf Club Drive Beaumont, California	PROJECT NO.	18059-F
	PLATE	B-1-1

SOILS SOUTHWEST, INC.
Consulting Foundation Engineers

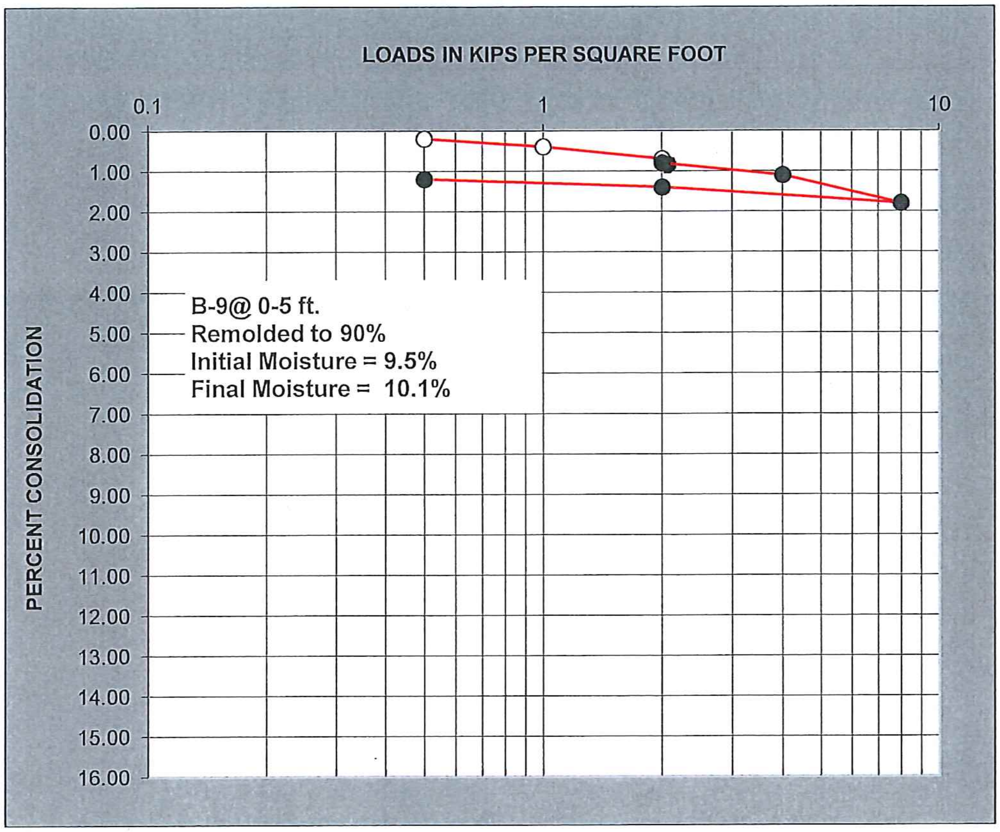


SYMBOL	LOCATION	DEPTH (FT)	TEST CONDITION	COHESION (psf)	FRICTION (degree)
■	B-2	8.0	Undisturbed	500.74	30.84

Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway and Golf Club Drive Beaumont, California	PROJECT NO.	18059-F
	PLATE	B-1-2

SOILS SOUTHWEST, INC.
Consulting Foundation Engineers

CONSOLIDATION TESTS

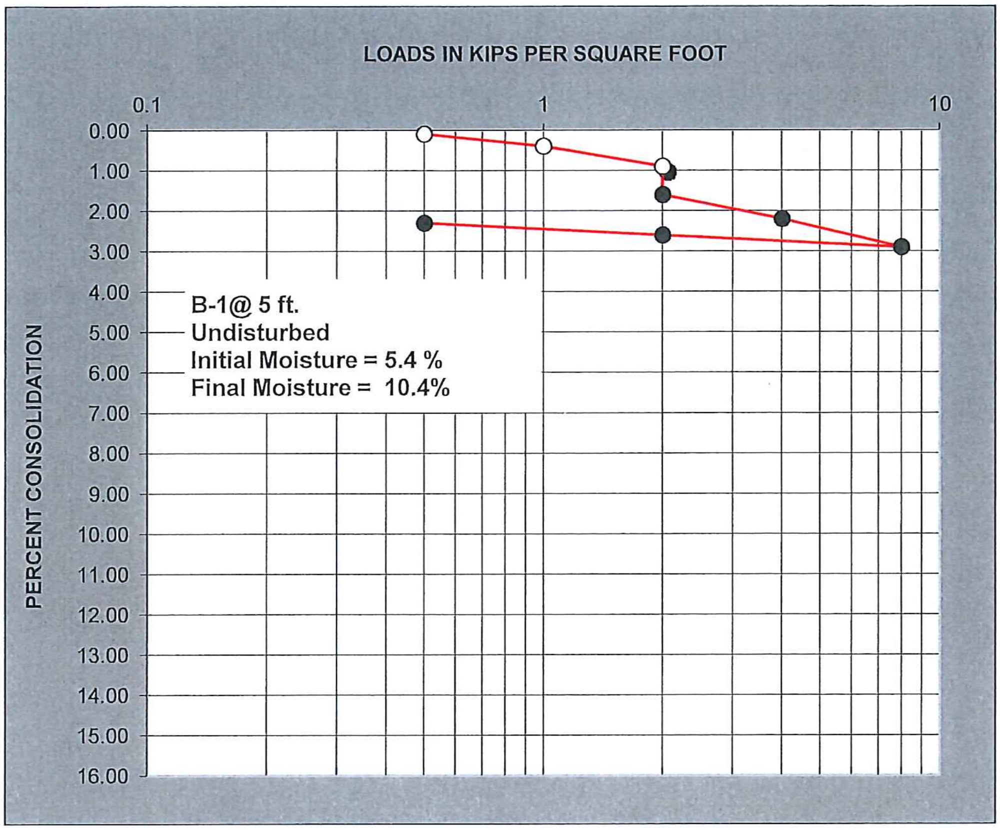


● WATER PERMITTED TO CONTACT SAMPLE

	PROJECT	Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway & Golf Club Drive, Beaumont		
	PROJECT NO.	18059-F	PLATE	B-2

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

CONSOLIDATION TESTS



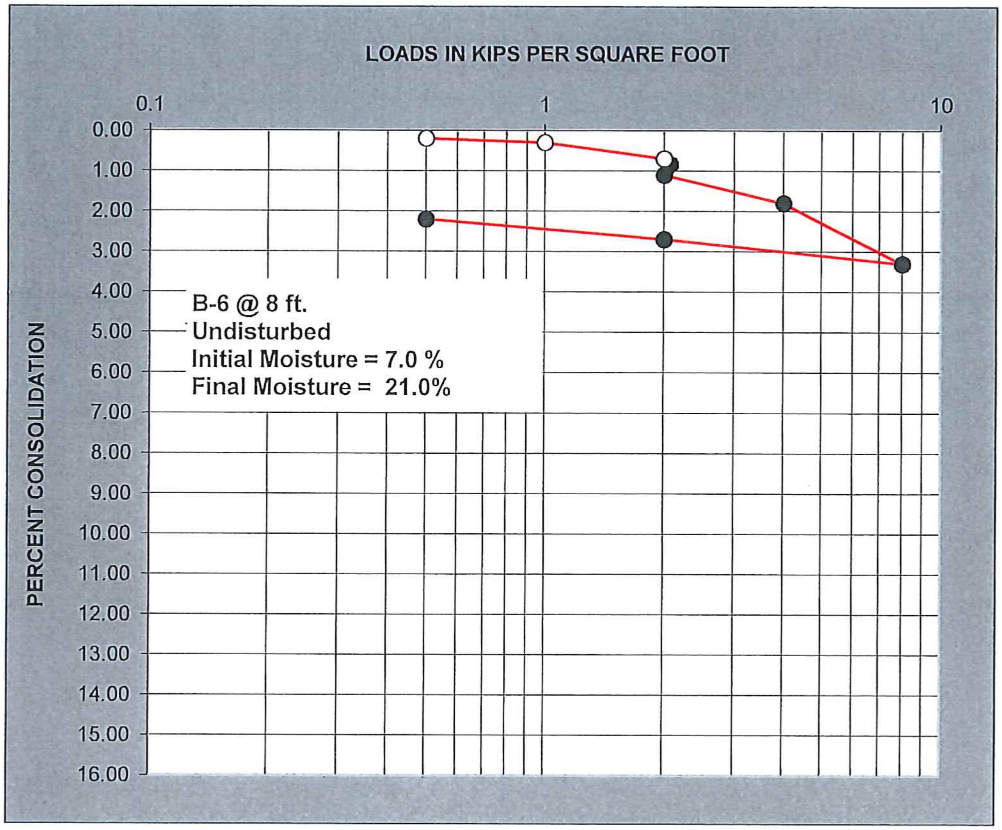
● WATER PERMITTED TO CONTACT SAMPLE

PROJECT	Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway & Golf Club Drive, Beaumont			
PROJECT NO.	18059-F	PLATE	B-2-1	

SOILS SOUTHWEST INC.

Consulting Foundation Engineers

CONSOLIDATION TESTS



● **WATER PERMITTED TO CONTACT SAMPLE**

PROJECT	Proposed Gas Station and Commercial Retail NWC Oak Valley Parkway & Golf Club Drive, Beaumont			
PROJECT NO.	18059-F	PLATE	B-2-2	

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

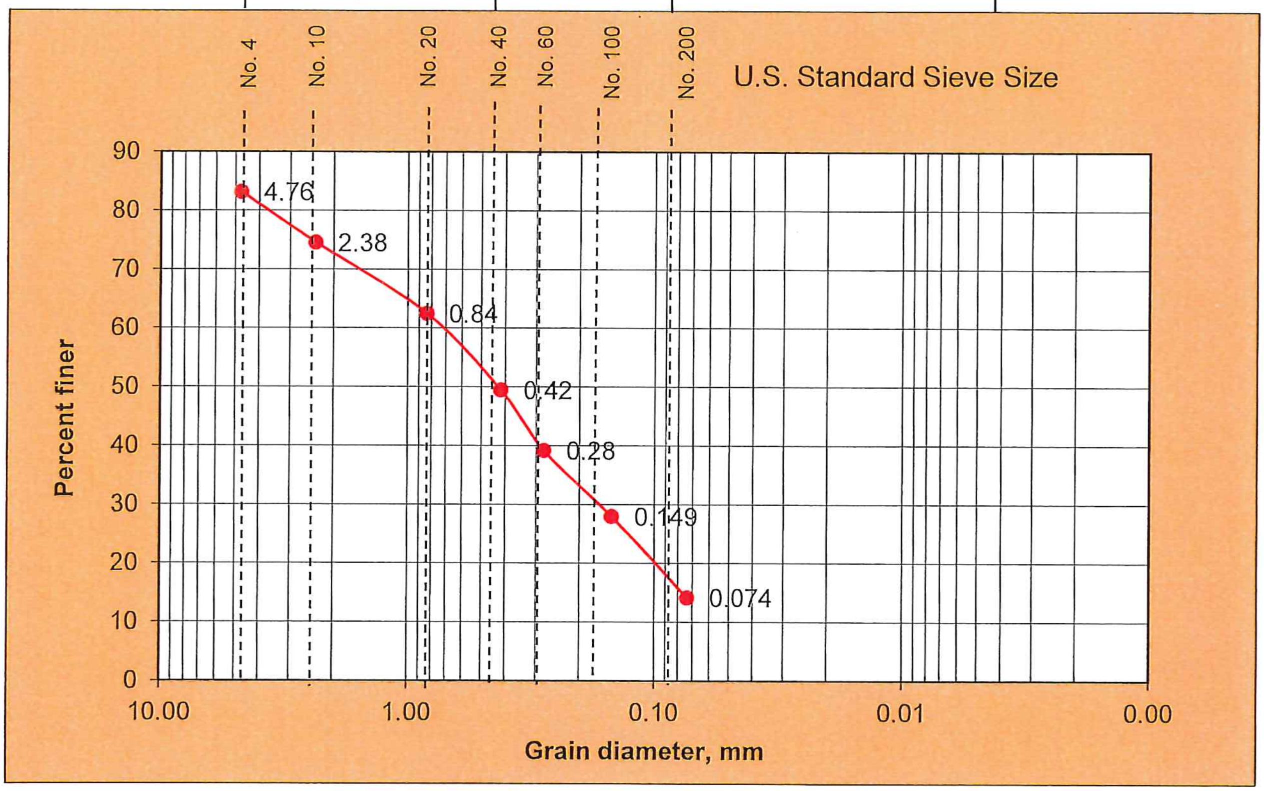
GRAIN SIZE DISTRIBUTION ASTM D422

Project: Beaumont Gas Station & Retail **Job #** 18059-F
Location: NWC Oak Valley Pkwy & Golf Club Dr. **Boring No:** B-9@0-5 **Sample No:** 3
Description of Soil: SP-SM- slightly silty fine to medium coarse sands with occasional gravels
Date of Sample: 11/13/2018
Tested By: RM **Date of Testing:** 11/29/2018

Sieve No.	Sieve Openings in mm	Percent Finer
4	4.76	83.20
10	2.38	74.60
20	0.84	62.50
40	0.42	49.50
60	0.28	39.20
100	0.149	28.00
200	0.074	14.20

Grain Size	% Retained
Gravel	17
Med. to Crs	32
Fines	33
Silts	18

Gravel	Sand			
	Coarse to Medium	Fine	Silt	Clay



Visual Soil Description : SAND - slightly silty fine to medium coarse with some gravels

Soil Classification: SP-SM

System: USC

SOILS SOUTHWEST INC.
Consulting Foundation Engineers

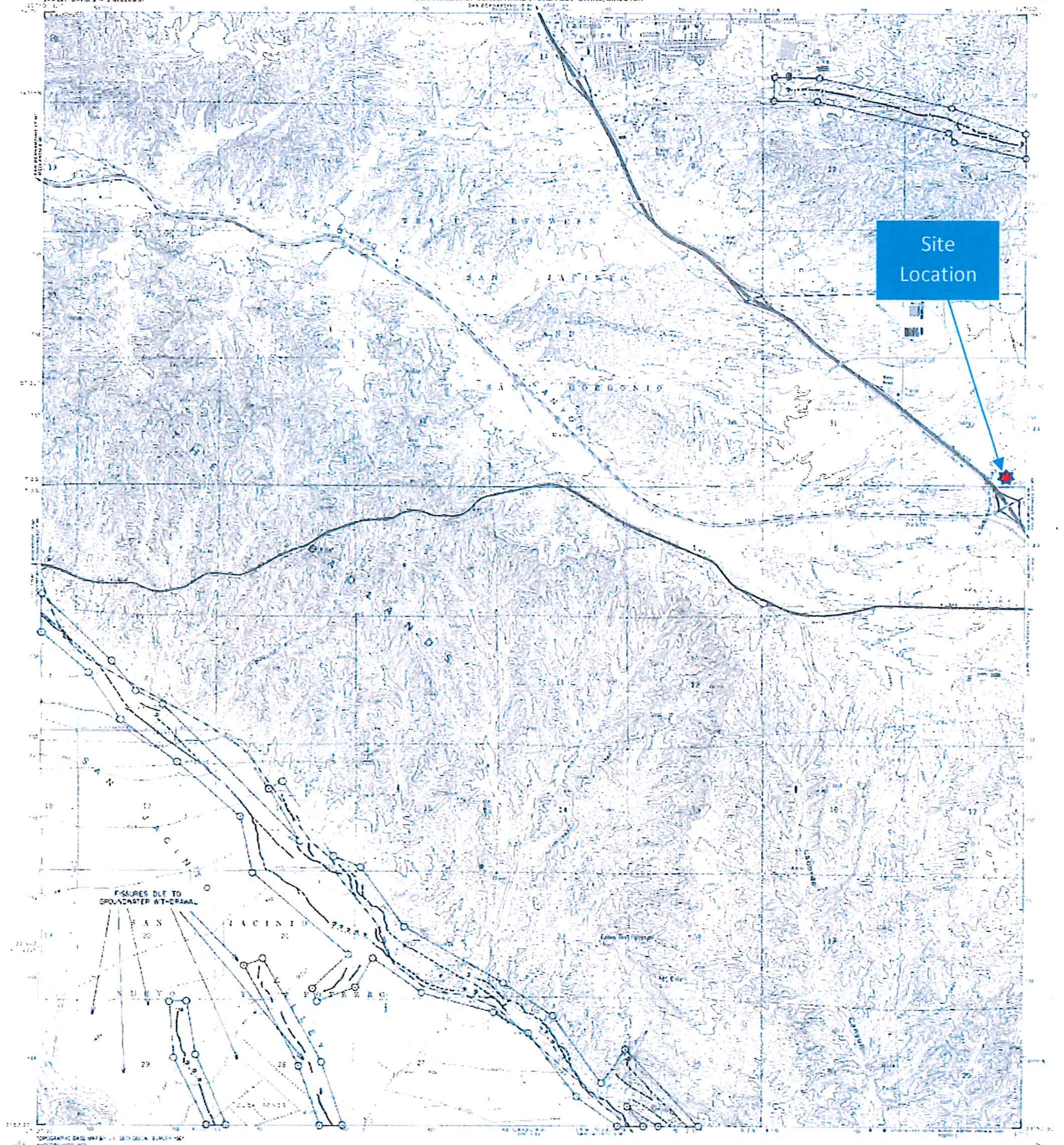
APPENDIX C

Supplemental Seismic Design Parameters

DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, STATE GEOLOGIST

STATE OF CALIFORNIA - PETE WILSON, GOVERNOR
THE RESOURCES AGENCY - DOUGLAS P. WHEELER, SECRETARY
DEPARTMENT OF CONSERVATION - MICHAEL F. BYRNE, DIRECTOR

EL CASCO QUADRANGLE
CALIFORNIA - RIVERSIDE CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)



SCALE 1:24 000

CONTOUR INTERVAL 40 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

STATE OF CALIFORNIA
EARTHQUAKE FAULT ZONES

Delineated in compliance with
Chapter 7.5, Division 2 of the California Public Resources Code
(Alquist-Priolo Earthquake Fault Zoning Act)

EL CASCO QUADRANGLE

REVISED OFFICIAL MAP

Effective: June 1, 1995

James F. Davis State Geologist

MAP EXPLANATION

Active Faults

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

Earthquake Fault Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define Earthquake Fault Zone segments.

Seaward projection of zone boundary

REFERENCES USED TO COMPILE FAULT DATA

IMPORTANT - PLEASE NOTE

1) *This map may not show all faults that have the potential for surface fault rupture, either within the Earthquake Fault Zones or outside their boundaries.*
2) Faults shown are the basis for establishing the boundaries of the Earthquake Fault Zones.
3) The identification and location of these faults are based on the best available data. However, the quality of data used is varied. Traces have been drawn as accurately as possible at this map scale.
4) Fault information on this map is not sufficient to serve as a substitute for the geologic site investigations required under Chapter 7.5 of Division 2 of the California Public Resources Code.

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

Ground Motion Interpolator (2008)

Longitude: -116.9999

Latitude: 33.946959

VS30: 270 (180-1050 m/sec)

Return Period:

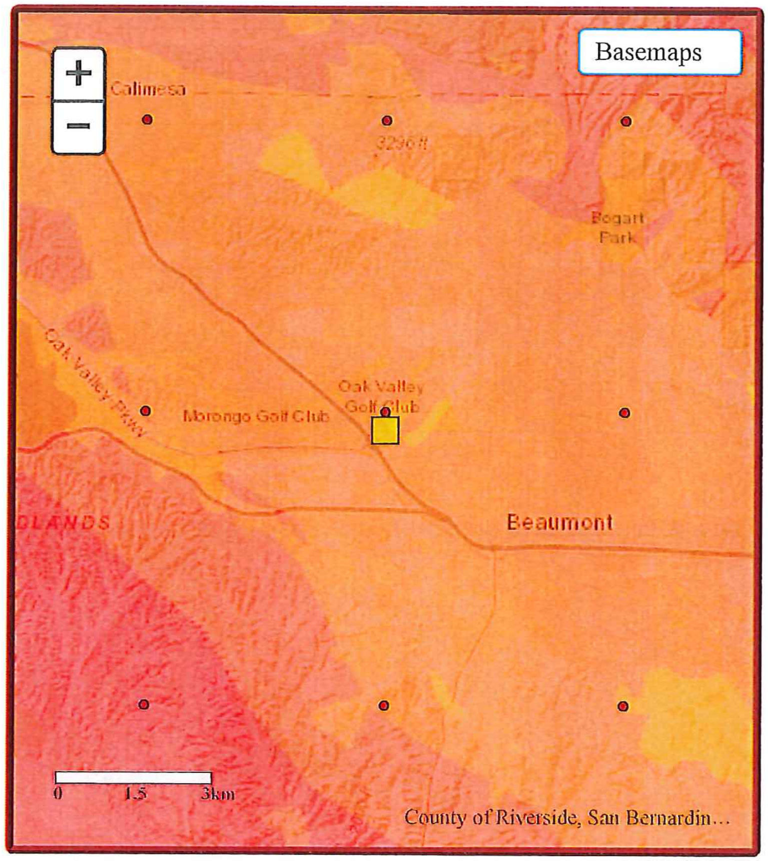
2% in 50 years   10% in 50 years

Spectral Acceleration:

PGA   0.2 second SA   1.0 second SA

Submit

Inputs:	**Result:**
-116.999985, 33.946959 vs30: 270 m/sec 10% in 50 years PGA	**0.565 g**



Information and Disclaimer

USGS Design Maps Summary Report

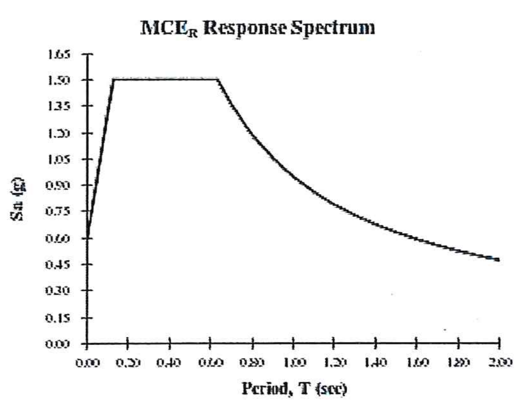
User–Specified Input

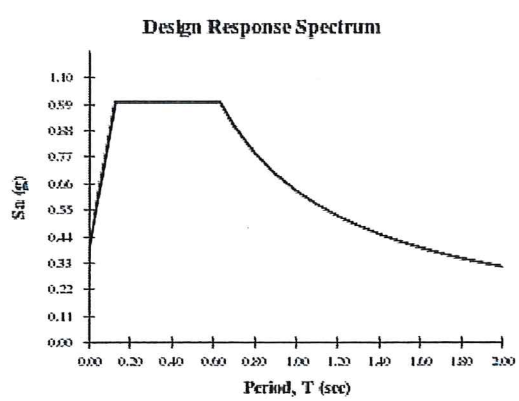
Report Title	Project 18059 - F/BMP Fri November 9, 2018 17:55:04 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.94696°N, 116.99985°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

$S_S =$ 1.500 g	$S_{MS} =$ 1.500 g	$S_{DS} =$ 1.000 g
$S_1 =$ 0.630 g	$S_{M1} =$ 0.945 g	$S_{D1} =$ 0.630 g

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 return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

Section 11.6 — Seismic Design Category

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

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.000 g, Seismic Design Category = D

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

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.630 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

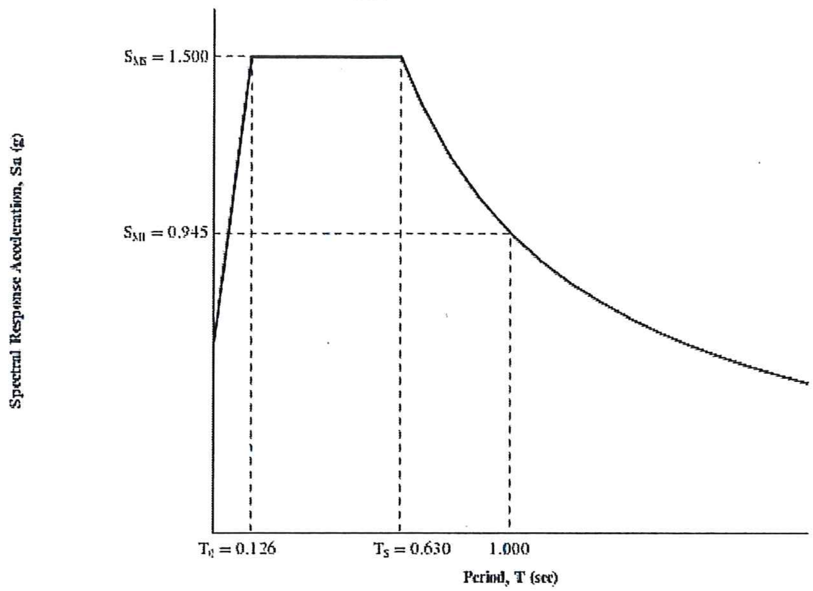
Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

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

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

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
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 = 1.500 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.630 g, F_v = 1.500

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 Project Soils Engineer must evaluate the changed conditions, 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. (SSI) 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.