INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists
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Project No. E080-055

ENGINEERING RESOURCES OF SOUTHERN CALIFORNIA, INC.
1861 West Redlands Boulevard
Redlands, California 92373

Attention: Mr. Matt Brudin, P.E.

Subject: Geotechnical Investigation
Soboba Horseshoe Service Center, Phase I
SWC Lake Park Drive and Soboba Road
Soboba Nation, San Jacinto, California

Dear Mr. Brudin:

We are pleased to submit this geotechnical report prepared for the subject project. The project site is located southwest of and adjacent to the intersection of Lake Park Drive and Soboba Road on the Soboba Nation in San Jacinto, California.

The proposed development is feasible from a geotechnical engineering standpoint. The following report includes design recommendations along with the field and laboratory data. We have also included recommendations for site grading.

We appreciate being of service to you on this project. If you have any questions, please contact our office.

Respectfully,
INLAND FOUNDATION ENGINEERING, INC.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

This report presents the results of the geotechnical investigation conducted for the proposed Soboba Horseshoe Service Center, Phase I development. The project site is located southwest of and adjacent to the intersection of Lake Park Drive and Soboba Road on the Soboba Nation in San Jacinto, California. The following references were provided for our use during this investigation.

- Conceptual Site Plan, Scheme 2, Soboba Horseshoe Service Center, Phase I, prepared by KTGY Architecture & Planning, dated March 17, 2020.
- Conceptual Rough Grading Plan, Soboba Horseshoe Service Center, Phase I, prepared by Engineering Resources of Southern California, Inc., undated.
- Rough Grading Plan for the Construction of Horseshoe Property Commercial/Retail Development, prepared by Engineering Resources of Southern California, Inc., dated April 19, 2020.

This report provides geotechnical engineering recommendations for design and construction of the proposed development.

SCOPE OF SERVICES

The purpose of this preliminary geotechnical investigation is to provide geotechnical parameters for design and construction of the proposed project. The scope of the geotechnical services included:

- *Review of the general geologic conditions and specific subsurface conditions of the project site.*
- *Seismicity evaluation of the site.*
- *Evaluation of the engineering and geologic data collected for the project site.*
- *Preparation of this report with geotechnical conclusions and recommendations for design and construction.*

The tasks performed to achieve these objectives included:

- *Collection and review of new and existing data relative to the site.*
- *Subsurface exploration to evaluate the nature and stratigraphy of the subsurface soil and to obtain representative samples for laboratory testing.*

- *Laboratory testing of representative samples to evaluate the classification and engineering properties of the soils.*
- *Analysis of the data collected and the preparation of this report with our geotechnical conclusions and recommendations.*

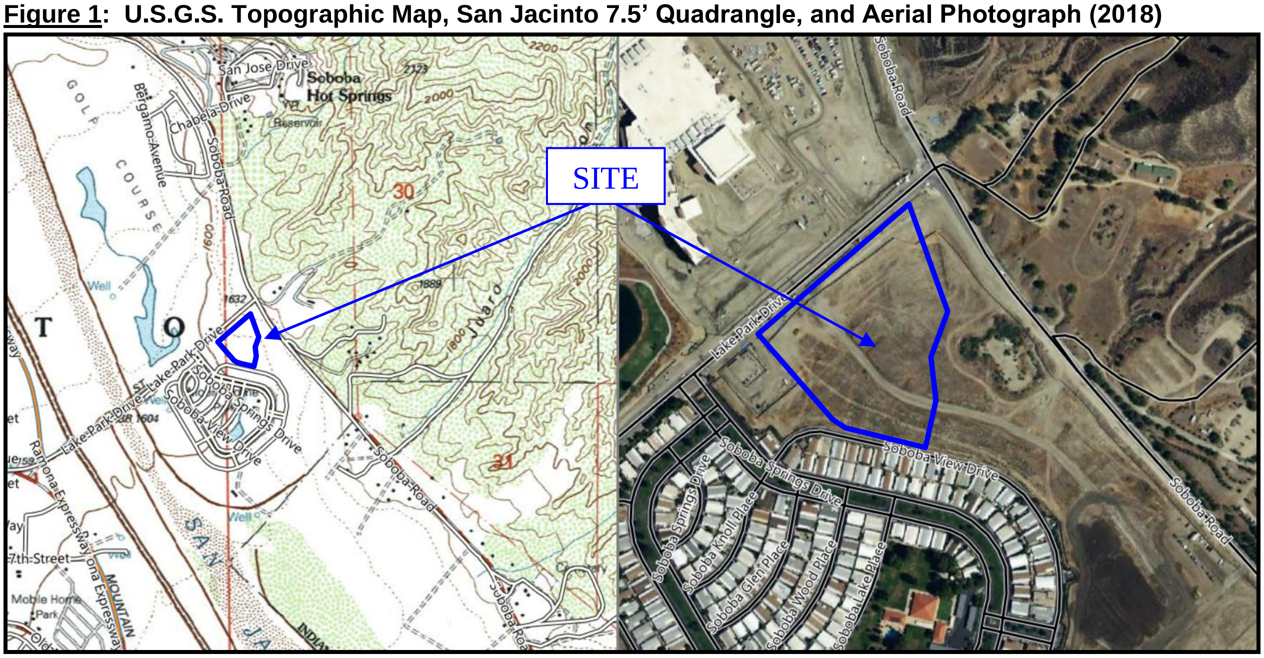
We understand that a geologic fault study was performed for this site by others and that a non-buildable use fault zone has been established on the northeasterly portion of the site. Our scope of service did not include a review/verification of earthquake faulting at the site or any subsurface fault exploration.

Evaluation of hazardous waste was not within the scope of services provided. The evaluation of seismic hazards (excluding surface fault rupture potential) was based upon field mapping, literature review and limited subsurface exploration.

SITE AND PROJECT DESCRIPTION

The proposed development is located southwest of and adjacent to the intersection of Lake Park Drive and Soboba Road on the Soboba Nation in San Jacinto, California. This study focused only on the Soboba Horseshoe Service Center, Phase I portion of the site. Figure 1 below shows the location of the project site.

<u>Figure 1</u>: U.S.G.S. Topographic Map, San Jacinto 7.5' Quadrangle, and Aerial Photograph (2018)



At the time of our field investigation, the site was vacant. Site clearing and grading activities were in progress. Soil trucking and import stockpiling operations were also being conducted on the site during our field investigation.

The project site was previously graded as a residential subdivision. An existing buried storm drain is present through a portion of the site. Portions of the site had standing water present due to recent rains. A large stormwater basin is present to the southeast of the proposed Phase I development. Figure 2 below is a photograph of the project site taken during our investigation, looking toward the northwest.

Figure 2: Site Photograph, Looking Northwest

The proposed development primarily consists of construction of a 5,000 square foot (sf) convenience store/gas station, 2,000 sf retail building, fuel dispensing island area, and 3,590 sf car wash structure. Paved access drives and parking areas are planned. Future retail space is also planned. We understand the proposed structures are to be supported by continuous and isolated spread footings.

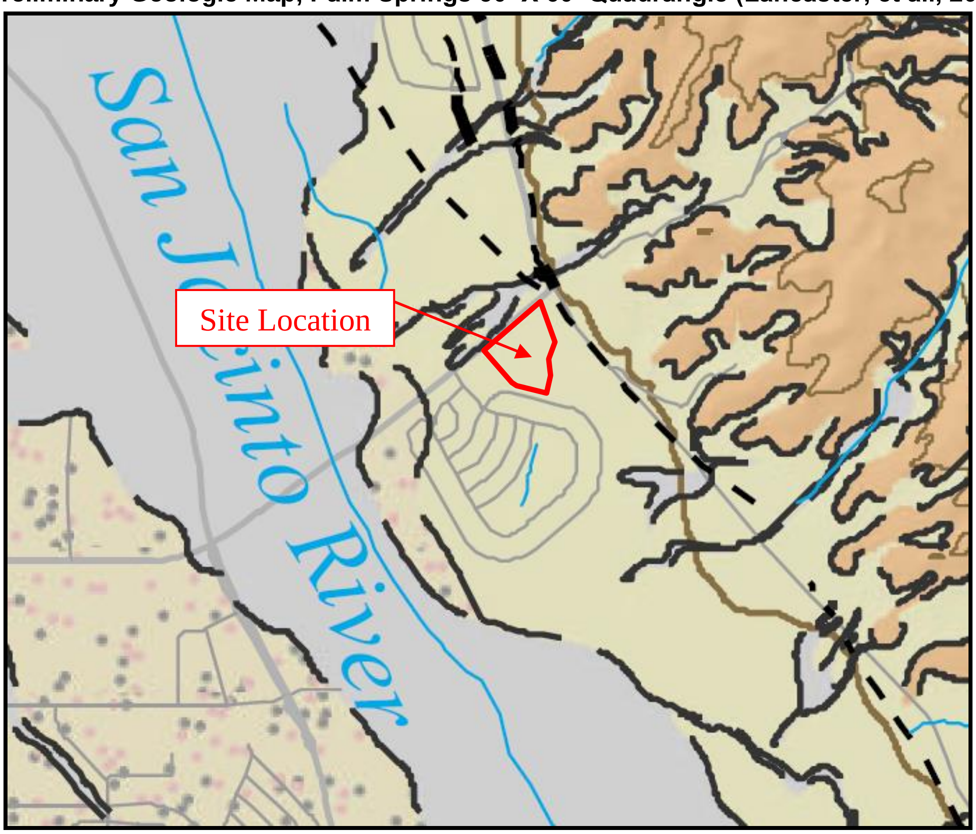
Site grades at the time of our field exploration were on the order of 10 to 13 feet below final design grades. Final site grades will be achieved by filling with soil imported from the EMWD Mountain Avenue West Replenishement Basin site at Mountain Avenue and Ramona Expressway in San Jacinto, California.

GEOLOGIC SETTING

Regional Geology: The subject site is situated within a natural geomorphic province in southwestern California known as the Peninsular Ranges, which is characterized by steep, elongated ranges and valleys that trend northwesterly. This geomorphic province encompasses an area that extends 125 miles, from the Transverse Ranges and the Los Angeles Basin, south to the Mexican border, and beyond another 795 miles to the tip of Baja California (Norris & Webb, 1990; Harden, 1998). This province is believed to have originated as a thick accumulation of predominantly marine sedimentary and volcanic rocks during the late Paleozoic and early Mesozoic. Following this accumulation, in mid-Cretaceous time, the province underwent a pronounced episode of mountain building. The accumulated rocks were then complexly metamorphosed and intruded by igneous rocks, known locally as the Southern California Batholith. A period of erosion followed the mountain building, and during the late Cretaceous and Cenozoic time, sedimentary and subordinate volcanic rocks were deposited upon the eroded surfaces of the batholithic and pre-batholithic rocks.

Local Geology: According to the Preliminary Geologic Map of Quaternary Deposits in Southern California Palm Springs 30' x 60' Quadrangle (CGS, 2012), the site is underlain by young (Holocene to late Pleistocene) alluvial fan deposits described as unconsolidated boulders, cobbles, gravel, sand and silt (map symbol Qyf). Figure 3 below is a portion of the referenced map depicting the mapped geologic units in the vicinity of the site.

Figure 3: Preliminary Geologic Map, Palm Springs 30' X 60' Quadrangle (Lancaster, et al., 2012)

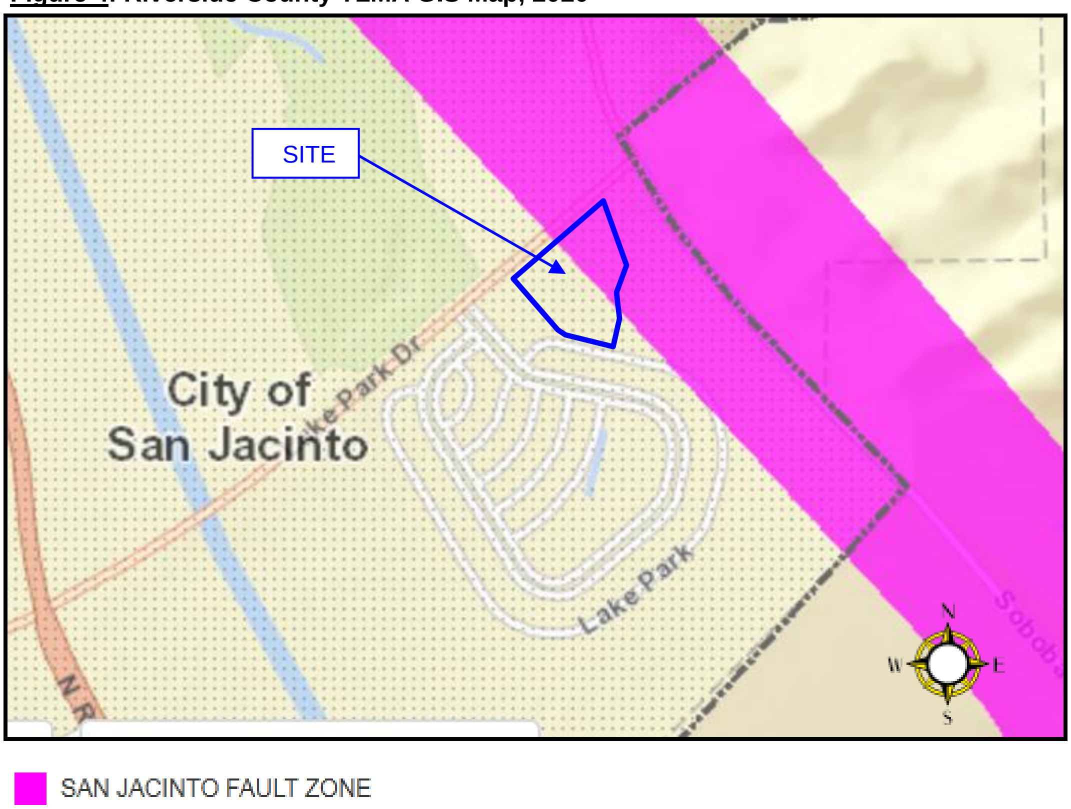


Qyf	**Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Groundwater: Groundwater was not encountered within our exploratory borings, which extended to a maximum depth of approximately 51.5 feet below the existing ground surface. Water well data provided by the Eastern Municipal Water District (2010) and Watermaster Support Services (2019) indicate that groundwater in the vicinity of the site is greater than 180 feet below the existing ground surface. Perched groundwater may be encountered at shallower zones and could be encountered during periods of high precipitation and or irrigation.

Faulting: There are at least 38 major late Quaternary active/potentially active faults that are within a 100-kilometer radius of the site (Blake, 2000). The northeasterly portion of the site is located within a State of California "Alquist-Priolo Earthquake Fault Zone" for fault rupture hazard (Hart and Bryant, 2007) associated with the Claremont fault (branch of the San Jacinto Valley segment of the San Jacinto Fault Zone). The San Jacinto fault (San Jacinto Valley Segment, U.S.G.S., 2008) is a right-lateral, strike-slip fault, approximately 43 kilometers in length, with an estimated maximum moment magnitude (M_w) earthquake of M_w7.0 and an associated slip-rate of 18 mm/year. Mapping and fault studies by others (Geocon, 2017) have identified the presence of the Claremont Fault on the northeasterly portion of the site and have established a *Building Setback Zone* that has been used and relied upon for the current project design. Figure 4 shows a portion of the Riverside County GIS map showing the location of the site and mapped State of California Alquist-Priolo Earthquake Fault Zone.

<u>Figure 4</u>: Riverside County TLMA GIS Map, 2020



The site and surrounding area have been subjected to strong ground shaking related to active faults that traverse the region. The major faults influencing the site include the San Jacinto fault (San Jacinto Valley and Anza segments). The approximate distances to the faults and published maximum earthquake magnitudes are shown in Table 1:

Table 1: Major Fault Parameters (USGS, 2008)

Fault Zone	Approximate Distance (km)	Earthquake Magnitude (M_w)
San Jacinto - San Jacinto Valley (Claremont)	0.01	7.0
San Jacinto - Anza	5.0	7.2
San Andreas - Southern	20.3	7.4
San Jacinto - San Bernardino	30.9	6.7

Seismic Parameters: The site coordinates (WGS 84) are 33.7892°N / -116.9278°W. A site-specific ground motion analysis was conducted for this project in accordance with the 2019 California Building Code and ASCE 7-16. The details of the site-specific ground motion analysis are presented within Appendix C. Additionally, a seismic shear-wave survey was conducted for this study for the purpose of determining the Site Classification and Vs_{30} input values for the ground motion analysis. The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps (OSHPD, 2020) and the California Building Code criteria (CBC, 2019), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of the site-specific analysis are summarized in Table 2 below.

Table 2: Summary of Seismic Design Parameters

Factor or Coefficient	Value
S_S	2.153g
S_1	0.862g
S_{DS}	1.37g
S_{D1}	2.41g
S_{MS}	2.058
S_{M1}	3.617
T_S	1.76
T_L	8 Seconds
$MCE_G PGA$	0.85g
Site Class	D

Secondary Seismic Hazards: Other than the ground rupture hazard on the site, which has been addressed by others, the primary geologic hazard affecting the project is that of ground shaking. Secondary permanent or transient seismic hazards generally associated with severe ground shaking during an earthquake include, but are not necessarily limited to; ground rupture, liquefaction, seismically-induced settlement, seiches or tsunamis, landsliding, rockfalls and debris flows. These are discussed below:

Ground Rupture: Ground rupture is generally considered most likely to occur along pre-existing faults. The northeasterly portion of the site is located within a State of California "Alquist-Priolo Earthquake Fault Zone" for fault rupture hazard (Hart and Bryant, 2007) associated with the Claremont fault (branch of the San Jacinto Valley segment of the San Jacinto Fault Zone). Our scope of service did not include a review/verification of earthquake faulting at the site or any subsurface fault exploration. Mapping and fault studies by others (Geocon, 2017) has identified the presence of the Claremont Fault on the northeasterly portion of the site and have established a *Building Setback Zone* that has been used and relied upon for the current project design.

Liquefaction: In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soils that can result in the settlement of buildings, ground failure, or other hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soils having relatively low density (usually of Holocene age); 2) shallow ground water (generally less than 50 feet); and 3) moderate to high seismic ground shaking. Based on the groundwater depth beneath the site of more than 100 feet, it is our opinion that the potential for liquefaction at the site is nil.

Seismically-Induced Settlement: Seismically-induced settlement includes post-liquefaction settlement of saturated soil and “dry sand” settlement of partially saturated soil. Inland valleys and desert areas underlain by loose alluvial deposits are particularly prone to dry sand settlement, as is the case with much of the San Jacinto Valley.

The potential for “dry sand” seismically-induced settlement was evaluated using the GeoSuite® computer program, based on Pradel’s method (1998). The results of our analysis are appended and indicate a total estimated settlement of approximately 1.8 inches due to seismic shaking. The estimated differential settlement due to a seismic event is approximately 1.0 inch in 30 feet.

Seiches/Tsunamis: A seiche is a standing wave in an enclosed or partially enclosed body of water. In order for a seiche to form, the body of water needs to be at least partially bounded, allowing the formation of the standing wave. Tsunamis are very large ocean waves that are caused by an underwater earthquake or volcanic eruption, often causing extreme destruction when they strike land.

There are no bodies of water on or adjacent to the project site. Based on the distance to large, open bodies of water and the elevation of the site with respect to sea level, it is our opinion that the potential for seiches/tsunamis does not present a hazard to this project.

Landsliding: Due to the relatively low-lying relief of the site and adjacent areas, the potential for landsliding due to seismic shaking is considered very low.

Rockfalls: Since no large rock outcrops are present at or adjacent to the site, the possibility of rockfalls during seismic shaking is nil.

Debris Flows: Debris flows are composed of a slurry-like mass of liquefied debris (ranging up to boulder size) that moves downhill under the force of gravity. Such slurries are dense enough to support very large particles but not solid enough to resist flowing downhill. Debris flows are most common in steep mountain canyons when a mass of mud and debris becomes saturated during a heavy rainstorm and suddenly begins to flow down the canyons (Prothero & Schwab, 1996). Based on the location of the site and the relatively planar topography of the property up-gradient of site, it is our opinion that the hazard of debris flow should be considered low.

Other Geologic Hazards: There are other geologic hazards not necessarily associated with seismic activity that occur statewide. These hazards include methane gas, hydrogen-sulfide gas, tar seeps, Radon-222 gas, regional subsidence, and naturally occurring asbestos. Of these hazards, there are none that appear to impact the site.

SUBSURFACE CONDITIONS

The field and laboratory exploration and testing indicate that the site is underlain by younger alluvial soils comprised predominately of interbedded layers of silty sand (SM), silty clayey sand (SM-SC), clayey sand (SC), sandy clay (CL) and sand with silt (SP-SM) to at least a depth of 51.5 feet, locally. As much as approximately four (4) feet of artificial fill consisting of fine- to medium silty sand (SM), clayey sand (SC) and sandy clay (CL) was encountered within our exploratory borings. Due to previous grading on the site, variable depths of existing artificial fill across the site are expected.

The density and moisture content of the samples obtained from the borings varied considerably. Fine-grained silt and clay soils ranged in consistency from soft to very stiff throughout the range of depths explored. Coarse-grained sand deposits ranged from loose to very dense.

Near surface samples obtained were generally slightly moist to moist. Moisture content of fine-grained silt and clay soils ranged from slightly moist to very moist. A sample of clay (CL) obtained from a depth of about 12 feet in boring B-2 had a very high moisture content of 30 percent. The moisture content of coarse-grained soil generally varied from slightly moist to moist. Density and moisture content data from the exploratory borings are shown on the boring logs.

Groundwater was not encountered within our subsurface exploration, which extended to a depth of approximately 51.5 feet below the existing ground surface. Based on our review of groundwater data, it appears that the depth to groundwater exceeds 130 feet below the existing ground surface.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of our field and laboratory exploration and testing, the proposed construction is feasible from a geotechnical engineering standpoint. Existing site grades are 10 to 13 feet below final design grades. Final site grades will be achieved by filling with imported soil.

IFE obtained soil samples from the EMWD import site prior to importing to evaluate the suitablity of the proposed import soil for use as site fill material, including its suitability for foundation and pavement support. The results of the testing and evaluation were presented in our report dated November 19, 2019.

The following conclusions and recommendations are based on the soil conditions encountered in the exploratory borings and the results of the import fill evaluation. *The recommendations should be confirmed at the completion of site grading based on testing of the actual foundation and pavement subgrade soil placed.*

Foundation Design: Shallow spread footings (continuous and isolated) should be designed using an allowable soil bearing pressure of 1,200 pounds per square foot (psf). Footings should have a minimum width of 12 inches and be founded a minimum depth of 12 inches below the lowest adjacent grade. The allowable bearing pressure can be increased by 300 psf for each additional foot of width and by 800 psf for each additional foot of depth, to a maximum allowable bearing pressure of 2,400 psf. The allowable bearing pressure may be increased by ⅓ for short-term transient wind and seismic loads.

Static settlement of foundations properly designed and constructed as recommended herein is expected to be less than 1.0 inch. Static differential settlement between foundations of similar size and load is expected to be less than one-half inch.

Seismically-induced differential settlement is estimated to be on the order of 1.8 inches, with estimated differential settlement of 1.0 inch in 30 feet.

Lateral Resistance: Resistance to lateral loads will be provided by a combination of friction acting at the base of the slab or foundation and passive earth pressure. A coefficient of friction of 0.40 between soil and concrete may be used with dead load forces only. A passive earth pressure of 240 psf, per foot of depth, may be used for the sides of footings poured against recompacted or suitably dense native material. Passive earth pressure should be ignored within the upper one foot except where confined as beneath a floor slab, for example.

Lateral Earth Pressure: Retaining walls should be designed for an active earth pressure equivalent to that exerted by a fluid weighing not less than 40 pcf. Any applicable construction or seismic surcharges should be added to this pressure. Retaining wall backfill should have an expansion index of less than 20.

Excavation and Trench Wall Stability: All excavations should be configured in accordance with the requirements of CalOSHA. The soil should be classified as Type C. The classification of the soil and the shoring and/or slope configuration should be the responsibility of the contractor on the basis of the excavation depth and the soil encountered. The contractor should have a "competent person" onsite for the purpose of assuring safety within and about all construction excavations.

Concrete Slabs-on-Grade: Concrete slabs-on-grade should have a minimum thickness of four inches. During final grading and prior to the placement of concrete, all surfaces to receive concrete slabs-on-grade should be compacted to maintain a minimum compacted fill thickness of 12 inches. Load bearing slabs should be designed using a modulus of subgrade reaction not exceeding 100 pounds per square inch per inch.

Slabs should be designed and constructed in accordance with the provisions of the American Concrete Institute (ACI). Shrinkage of concrete should be anticipated and will result in cracks in all concrete slabs-on-grade. Shrinkage cracks may be directed to saw-cut "control joints" spaced on the basis of slab thickness and reinforcement. ACI typically recommend control joint spacings in unreinforced concrete at maximum intervals equal to the slab thickness times 24.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder/barrier designed and constructed according to the American Concrete Institute 302.1 R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder/barrier construction. At a minimum, the vapor retarder/barrier should comply with ASTM El745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage.

Expansive Soil: One sample of near surface silty-clayey sand (SC-SM) was tested for expansion index. The results indicate an expansion index of 23. All soil with an expansion index higher that 20 is considered to be expansive and is not recommended for placement in foundation or pavement areas without special design consideration.

Soil is likely present on-site with significantly higher expansion potential than that tested, particularly samples of clay encountered intermittently in the borings. Potentially expansive on-site soil should not be placed within five feet of finish design grade. All samples of proposed import soil should be tested for expansion potential.

Preliminary Concrete Pavement Design: The following Portland cement concrete pavement sections are based on the American Concrete Institute (ACI) Guide for Design and Construction of Concrete Parking Lots (ACI 330R-08), and an assumed design R-value of 18.

TABLE 3: Portland Cement Pavement Designs

Service	Concrete Thickness (ft.)
Car parking and access lanes (Category A)	5.0
Entrance and service lanes (Category B)	7.0

The concrete should have a minimum 28-day modulus of rupture of 500 psi. This corresponds to a compressive strength of approximately 2,500 psi. The upper 12 inches of pavement subgrade soil should be compacted to a minimum relative compaction of 95 percent.

Construction joints should be sawcut in the pavement at a maximum spacing of 30 times the thickness of the slab, up to a maximum of 15 feet. Pavement sawcutting should be performed within 12 hours of concrete placement, preferably sooner. Sawcut depths should be equal to approximately ¼ of the slab thickness for conventional saws or one inch when early-entry saws are utilized on slabs nine inches thick or less. Construction joints should not be placed near flow lines. The use of plastic strips for formation of jointing is not recommended. The use of expansion joints is not recommended, except where the pavement will adjoin structures.

Preliminary Flexible Pavement Design: The following recommended structural sections are based on an R-value of 18 and the traffic index (TI) values shown.

TABLE 4: Preliminary AC Pavement Designs

Service	Asphalt Concrete Thickness (ft.)	Base Course Thickness (ft.)
Light traffic (autos, parking areas, T.I. = 5.0)	0.25	0.60
Heavy traffic (trucks, driveways, bus lanes, T.I. =7.0)	0.30	1.10

Inland Foundation Engineering, Inc. does not practice traffic engineering. The TI values used to develop the recommended pavement sections are typical for projects of this type. We recommend that the project civil engineer or traffic engineer review the TIs to verify that they are appropriate for this project.

Corrosion: Analytical testing indicates that sulfate concentrations are less than 0.10 percent. In accordance with ACI 201.2R, Table 6.1.4.1a, the soil can be classified as Class S0 with respect to sulfate exposure.

Chloride concentrations tested range from 510 to 1,200 ppm and are not at levels high enough to be of concern with respect to corrosion of ferrous metals. They are, however, high enough to be of concern with respect to corrosion of concrete reinforcing steel. The results should be considered in combination with the chloride content of the hardened concrete in determining the effect of chloride on reinforcing steel.

The soil is slightly alkaline with pH values ranging from 7.7 to 7.9.

Tested saturated resistivity values ranged from 4,400 to 5,500 ohm-cm, indicating that the soil is moderately corrosive with respect to buried ferrous metal. Specific corrosion control measures, such as coating of pipe with non-corrosive material or alternative non-metallic pipe material, are considered to be necessary if there is a potential for saturated soil.

IFE does not practice corrosion engineering. We recommend that a qualified corrosion engineer be consulted for additional guidance.

General Site Grading: All site grading for the subject project should be performed in accordance with applicable provisions of the 2019 California Building Code, the Riverside County grading ordinance, and the following recommendations.

1. Clearing and Grubbing: All building and pavement areas and all surfaces to receive compacted fill should be cleared of vegetation, debris, and other unsuitable materials. All such material should be disposed of off-site.

All undocumented fill and loose alluvial soils encountered during site grading should be completed removed. Such material is suitable for replacement as compacted fill as recommended herein. Although fill material was encountered in our borings and trench excavations to a maximum depth of approximately four feet, deeper undocumented fill is expected as a result of previous grading performed on the site.

In the area of the proposed <u>convenience store/retail building</u>, <u>fuel island/ dispensary area</u>, <u>carwash building area</u>, and <u>future building areas</u>, existing fill and native soil should be removed to a depth of at least 10 feet below existing site grades. Existing soil in proposed pavement and street areas should be removed to a depth of at least six (6) feet below existing grades. The limits of removal within building and fuel island/dispensary areas should extend at least five (5) outside of exterior footing lines, at the excavation bottom. Deeper removals may be necessary depending on the conditions exposed during site excavation.

We have recommended that the limits of the recent exploratory trenches on the site be surveyed as well as the projection of the former fault trench backfill. The recent exploratory trenches conducted during this investigation ranged in depth from approximately 6 to 8 feet below the existing ground surface.

The outer edges of the former fault trench backfill associated with historical fault trench "LTF-5" have been staked in three of the recent exploratory trenches for survey purposes. Based on our findings, the minimum depth of removal within the limits of the former fault trench backfill should be 7 feet below the existing ground level.

2. Preparation of Surfaces to Receive Compacted Fill: All surfaces to receive compacted fill should be tested for compaction prior to processing. Testing should indicate a relative compaction of at least 85 percent within the unprocessed native soils. If roots or other deleterious materials are encountered or if the relative compaction fails to meet the acceptance criterion, deeper excavation may be required until satisfactory conditions are encountered. Upon approval, surfaces to receive fill should be scarified, brought to near optimum moisture content, and compacted to a minimum of 90 percent relative compaction.

3. Placement of Compacted Fill: Fill materials consisting of on-site soils or approved imported granular soil should be spread in shallow lifts and compacted at near optimum moisture content to a minimum of 90 percent relative compaction, based on ASTM D1557.

4. Preparation of Slab and Paving Areas: During final grading and immediately prior to the placement of concrete or aggregate base, the slab or pavement subgrade should be processed and compacted to a depth of at least of 12 inches. Compaction below concrete slabs should be to a minimum of 90 percent relative compaction. Compaction within pavement areas should be to a minimum of 95 percent relative compaction for both subgrade and aggregate base.

5. Utility Trench Backfill: Utility trench backfill consisting of on-site soils or approved imported granular soil should be mechanically compacted to at least 90 percent relative compaction. This is with the exception of the upper 12 inches under pavement areas where the minimum relative compaction is 95 percent. Jetting of utility trench backfill is not recommended.

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6. Testing and Observation: During all grading and backfilling, tests and observations should be performed by a representative of IFE to verify that the exposed subsurface conditions are as expected and that grading is performed in accordance with the project requirements. Field density testing should be performed in accordance with the current ASTM D1556 or ASTM D6938 test methods.

At or near completion of site grading, actual near-surface soil placed in foundation areas should be sampled and tested to confirm the recommendations presented herein remain valid and appropriate. Testing should include sieve analysis, expansion index, corrosivity and R-value.

GENERAL

The findings and recommendations presented in this report are based upon the soil conditions encountered. Should conditions be encountered during grading that appear to be different than those indicated by this report, this office should be notified.

This report was prepared for Engineering Resources of Southern California, Inc. for use in the design of the proposed Soboba Horseshoe Service Center, Phase I development. This report may only be used by Engineering Resources of Southern California, Inc. for this purpose. The use of this report by other parties other or for other purposes is not authorized without written permission by Inland Foundation Engineering, Inc. Inland Foundation Engineering, Inc. will not be liable for any projects connected with the unauthorized use of this report.

The recommendations of this report are considered to be preliminary. The final design parameters may only be determined or confirmed at the completion of site grading on the basis of observations made during the site grading operation. To this extent, this report is not considered to be complete until the completion of both the design process and the site preparation.

LIMITATIONS

The findings and recommendations of this report are based upon an interpolation of soil conditions between test locations. It is possible that conditions may be encountered that are different than those indicated in this report. Should such conditions be encountered during construction, our office should be notified in order to determine if revisions or retesting are warranted.

Evaluation of the potential of ground rupture due to faulting or evaluation of hazardous waste was not within the scope of services provided. The information in this report represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, either expressed or implied, is made as to the professional advice included in this report.

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APPENDIX A – Field Exploration

APPENDIX A

FIELD EXPLORATION

Five exploratory borings were excavated using a truck mounted rotary auger rig at the approximate locations shown on Figure No. A-8. The materials encountered in the borings were logged on the site by a staff geologist. These logs are presented on Figure Nos. A-3 through A-7.

Representative relatively undisturbed samples were obtained within the borings by driving an 18-inch long thin-walled steel penetration sampler (SPT) with successive 30-inch drops of a 140-pound hammer. The number of blows required to achieve each six inches of penetration were recorded on our boring logs and used for estimating the relative consistency of the subsoils. Two different samplers were used. The first sampler used was a Standard Penetration Test Sampler (SPT) for which published correlations relating the number of hammer blows to the strength of the soil are available. The second sampler type was a Modified California split barrel sampler, which is larger in diameter, carrying brass sample rings having inner diameters of 2.41 inches. Relatively undisturbed samples were removed from the sampler and placed in moisture sealed containers in order to preserve the natural soil moisture content. They were then transported to our laboratory for further observations and testing. Representative bulk samples were obtained from the trenches and transported to our laboratory for further evaluation and testing.

Laboratory test results are discussed and presented in Appendix B.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASES ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL

RELATIVE DENSITY	SPT * (# BLOWS/FT)	RELATIVE DENSITY (%)
VERY LOOSE	<4	0-15
LOOSE	4-10	15-35
MEDIUM DENSE	10-30	35-65
DENSE	30-50	65-85
VERY DENSE	>50	85-100

CONSISTENCY – FINE-GRAIN SOIL

CONSISTENCY	SPT* (# BLOWS/FT)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET ** PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2-4	0.13-0.25	0.25-0.5
Medium Stiff	4-8	0.25-0.5	0.5-1.0
Stiff	8-15	0.5-1.0	1.0-2.0
Very Stiff	15-30	1.0-2.0	2.0-4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM -1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp but no visible water
WET	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbled or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

EXPLANATION OF LOGS

LOG OF BORING B-01

DRILLING RIG	CME-75	DATE DRILLED	4/3/20
DRILLING METHOD	Rotary Auger	HAMMER TYPE	Auto-Trip
LOGGED BY	FWC	HAMMER WEIGHT	140-lb.
GROUND ELEVATION	+/- 1602 ft	HAMMER DROP	30-inches
		BORING DIAMETER	8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	SM	**ARTIFICIAL FILL,** SILTY SAND, with trace clay, very fine- to fine, olive (5Y 4/3), moist, loose.	AU			
	SM		SS	7 8	6	114
5	SM	**ARTIFICIAL FILL,** SILTY SAND, with trace clay, very fine- to fine, olive gray (5Y 4/2), moist, loose.	AU			
			SS	12 15	9	101
	SM	**SILTY SAND,** fine- to coarse, olive-gray (5Y 4/2), moist, medium dense.	SS	9 10	9	97
10	SC	**SILTY SAND,** very fine- to fine, olive-gray (5Y 4/2), moist, loose to medium dense, with thin interbeds of silt.	SS	8 9	12	98
	CL	**CLAYEY SAND,** very fine- to fine, olive (5Y 4/3), moist, loose.	AU			
15		**SANDY CLAY,** very fine- to fine, olive (5Y 4/3), moist, stiff.				
	SM	**SILTY SAND,** fine- to coarse, olive (5Y 4/3), slightly moist, loose to medium dense, with thin interbeds of sandy silt with clay.	SS	5 10	4	106
20	CL	**SANDY CLAY,** olive gray (5Y 4/2), moist, stiff.	AU			
			SPT	3 7	19	
25	SM	**SILTY SAND,** fine- to medium, olive gray (5Y 4/2), moist, medium dense.	SPT	10 17	16	
	SC	**CLAYEY SAND,** very fine- to fine, olive gray (5Y 4/2), moist, medium dense.				
30	SW-SM	**SAND with SILT,** fine- to coarse, olive (5Y 4/4), slightly moist, medium dense.	SS	8 11	25	94
	MH	**SANDY ELASTIC SILT,** olive (5Y 4/3), moist, stiff.		4 11	29	
	CL	**SANDY CLAY,** olive gray (5Y 4/2), moist, stiff.	AU			
35	SM	**SILTY SAND,** fine- to medium, olive gray (5Y 4/2), moist, loose to medium dense.	SPT	4 11	23	
40	CL	**SANDY CLAY,** olive gray (5Y 4/2), moist, stiff, with thin interbeds of silty sand.	SS	28 50	7	124
	SM	**SILTY SAND,** with trace clay, fine- to medium-grained, olive (5Y 4/3), moist, very dense, moderately cemented.	SPT	18 28	6	
45	SC	**CLAYEY SAND,** fine- to medium, olive-gray (5Y 4/2), moist, medium dense, weakly cemented.	SPT	9 13	18	
50	SC-SM	**SILTY, CLAYEY SAND,** very fine- to fine, olive gray (5Y 4/2), moist, dense.				
	SM	**SILTY SAND,** fine- to very coarse, olive-gray (5Y 4/2), slightly moist, dense, moderately cemented.	SPT	31 45	15	

End of boring at 51.5 feet. No groundwater encountered. Backfilled with native soils.

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CLIENT	Engineering Resources
PROJECT NAME	Soboba Horeshoe Service Center
PROJECT LOCATION	Soboba Rd San Jacinto, CA
PROJECT NUMBER	E080-055

FIGURE NO. A-3

LOG OF BORING B-02

DRILLING RIG	CME-75	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger	HAMMER WEIGHT	140-lb.
LOGGED BY	FWC	HAMMER DROP	30-inches
GROUND ELEVATION	+/- 1602 ft	BORING DIAMETER	8-inches

DATE DRILLED 4/3/20

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	SM		**SILTY SAND,** with trace clay, fine- to medium, olive-brown, moist, loose.			AU		10	109
	SC-SM		**SILTY, CLAYEY SAND,** very fine- to fine, very dark olive-gray (5Y 3/2), moist, loose.			AU			
						SS	3 4	14	104
5						SS	5 5	5	117
	SM		**SILTY SAND,** with trace clay, very fine- to fine, olive-gray (5Y 4/2), moist, loose.			SS	6 8	12	110
10	CL		**SANDY CLAY,** dark gray (5Y 4/1), very moist, soft.			SS	3 5	30	91
15	SM		**SILTY SAND,** fine- to medium, olive (5Y 4/3), slightly moist, medium dense, with interbeds of sand with silt.			SS	13 20	3	108
			End of boring at 16.5 feet. No groundwater encountered. Backfilled with native soils.						

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PROJECT NAME	Soboba Horeshoe Service Center
PROJECT LOCATION	Soboba Rd San Jacinto, CA
PROJECT NUMBER	E080-055

FIGURE NO. A-4

LOG OF BORING B-03

DRILLING RIG	CME-75	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger	HAMMER WEIGHT	140-lb.
LOGGED BY	FWC	HAMMER DROP	30-inches
GROUND ELEVATION	+/- 1602 ft	BORING DIAMETER	8-inches

DATE DRILLED 4/3/20

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	DESCRIPTION	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	SC-SM	**ARTIFICIAL FILL,** SILTY, CLAYEY SAND, very fine- to fine, gray-brown, moist, loose.	AU SS	3 3		
	SC	**CLAYEY SAND,** very fine- to fine, very dark gray (5Y 3/1), moist, loose.				
	SM	**SILTY SAND,** fine- to coarse, olive (5Y 4/3), slightly moist, medium dense.	SS	10 14	16	108
5	SC	**CLAYEY SAND,** very fine- to fine, dark gray (5Y 4/1), moist, loose to medium dense.	SS	11 17	17	104
	SM	**SILTY SAND,** with trace clay, very fine- to fine, olive-gray (5Y 4/2), slightly moist, medium dense, moderately cemented.	AU			
10			SS	7 9	4	113
	SC	**CLAYEY SAND,** very fine- to fine, olive-gray (5Y 4/2), moist, medium dense.	SS	8 9	19	100
15	SM	**SILTY SAND,** with trace clay, fine- to medium, olive (5Y 4/3), moist, medium dense, moderately cemented.	SS	11 16	6	108
	SP-SM	**SAND with SILT,** fine- to coarse, olive (5y 4/3), slightly moist, medium dense.	SPT	4 8	6	
20			SPT	8 15	3	
		End of boring at 21.5 feet. No groundwater encountered. Backfilled with native soils.				

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PROJECT LOCATION	Soboba Rd San Jacinto, CA
PROJECT NUMBER	E080-055

FIGURE NO. A-5

IFE BORING - GINT STD US LAB.GDT - 5/12/20 13:44 - P:\E080\E080-055 PRELIM SOBOBA HORSESHOE\GINT.GPJ

LOG OF BORING B-04

DRILLING RIG	CME-75	DATE DRILLED 4/3/20	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger		HAMMER WEIGHT	140-lb.
LOGGED BY	FWC		HAMMER DROP	30-inches
GROUND ELEVATION	+/- 1599 ft		BORING DIAMETER	8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	DESCRIPTION	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	SC	**ARTIFICIAL FILL,** CLAYEY SAND, very fine- to fine, dark gray (5Y 4/1), moist, loose.	AU SS	6 7	13	108
	CL	**ARTIFICIAL FILL,** SANDY CLAY, dark gray (5Y 4/1), moist, soft.	SS AU	5 5	19	103
5	CL	**SANDY CLAY,** very dark gray (5Y 3/1), moist, soft.				
	SM	**SILTY SAND,** fine- to medium, olive (5Y 4/3), moist, medium dense.	AU SS	11 11	10	111
10	SM	**SILTY SAND,** with trace clay, fine- to medium, olive (5Y 4/4), moist, medium dense.	SS	8 10	9	106
	ML	**SANDY SILT,** with trace clay, olive (5Y 4/4), moist, very stiff.				
	SM	**SILTY SAND,** fine- to coarse, olive (5Y 4/4), moist, medium dense to dense.	SS	16 26	15	105
15	SP-SM	**SAND with SILT,** with trace gravel, fine- to very coarse, olive (5Y 4/4), slightly moist, medium dense.	SS	11 13	1	113
20			SPT	8 16	2	
25	SC	**CLAYEY SAND,** fine- to coarse, olive-gray (5Y 4/2), very moist, loose to medium dense, with thin interbeds of silt with sand.	SPT	4 5	29	
		End of boring at 26.5 feet. No groundwater encountered. Backfilled with native soils.				

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PROJECT NAME	Soboba Horeshoe Service Center
PROJECT LOCATION	Soboba Rd San Jacinto, CA
PROJECT NUMBER	E080-055

FIGURE NO. A-6

LOG OF BORING B-05

DRILLING RIG	CME-75	DATE DRILLED	4/3/20	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger			HAMMER WEIGHT	140-lb.
LOGGED BY	FWC			HAMMER DROP	30-inches
GROUND ELEVATION	+/- 1600 ft			BORING DIAMETER	8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS
0	SC-SM	**SILTY CLAYEY SAND,** very fine- to fine, olive-gray (5y 4/2), moist, dense.
5	SC	**CLAYEY SAND,** very fine- to fine, dark gray (5Y 4/1), moist, medium dense.
	CL	**SANDY CLAY,** very fine- to fine, dark gray (5Y 4/1), moist, very stiff.
10	SC	**CLAYEY SAND,** very fine- to fine, olive-gray (5Y 4/2), moist, medium dense.
	SC-SM	**SILTY, CLAYEY SAND,** fine- to medium, olive-gray (5y 4/2), moist, medium dense, moderately cemented
15	SC	**CLAYEY SAND,** with trace gravel, fine- to coarse, olive-gray (5Y 4/2), moist, medium dense.
20	SM	**SILTY SAND,** fine- to very coarse, olive-gray (5Y 4/2), moist, loose to medium dense, with thin interbeds of sand with silt.
	CL	**SANDY CLAY,** very fine, olive-gray (5Y 4/2), moist, stiff.
25	SM	**SILTY SAND,** fine- to medium, olive-gray (5Y 4/2), moist, medium dense.
	CL	**SANDY CLAY,** dark gray (5Y 4/1), moist, stiff, interbedded with silty sand.
30	SM	**SILTY SAND,** fine- to medium, olive (5Y 4/3), moist, medium dense to dense, with very thin interbeds of sandy clay.
35	SP	**SAND,** fine- to coarse-grained, olive (5y 4/4), slightly moist, medium dense.
40	SM	**SILTY SAND,** with trace clay, fine- to medium, olive-gray (5Y 4/2), moist, dense, moderately cemented.
45	CL	**SANDY CLAY,** very dark grayish-brown (2.5Y 3/2), moist, stiff, with thin interbeds of silty, clayey sand.
50	SC	**CLAYEY SAND,** fine- to medium, olive (5Y 4/3), moist, dense, weakly cemented.
		End of boring at 51.5 feet. No groundwater encountered. Backfilled with native soils.

SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
AU		5	120
SS	23	10	115
AU	28		
SS	24 22		
SS	14	12	100
AU	17		
SS	16	12	112
AU	20		
SS	18	5	122
AU	25		
SPT	5 4	4	
SPT	5 5	18	
SS	24 23	5	97
SPT	7 10	18	
SPT	13 14	2	
SS	32 22	8	122
SPT	5 8	26	
SPT	8 11	16	
SPT	15 15	11	

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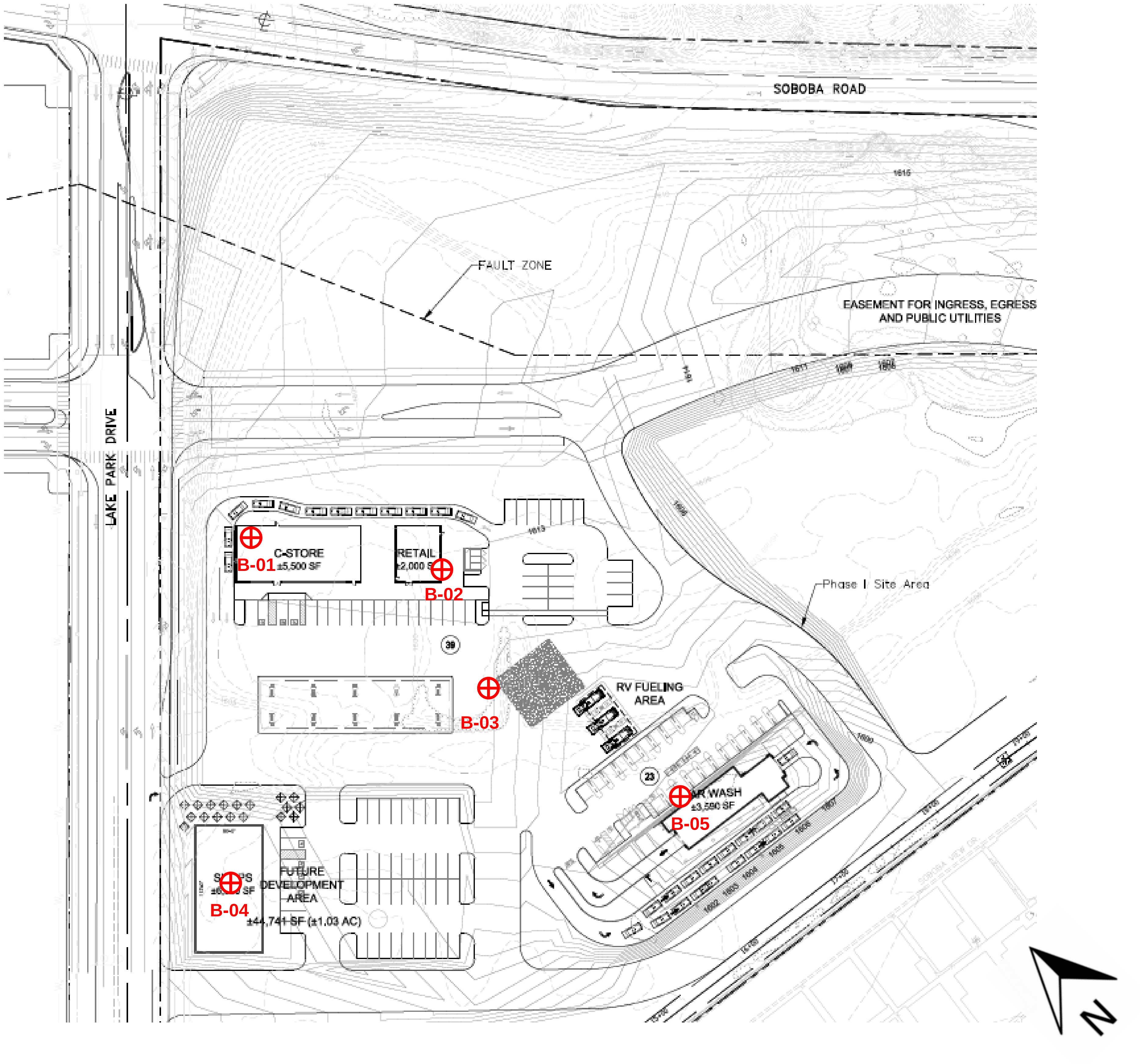
CLIENT	Engineering Resources
PROJECT NAME	Soboba Horeshoe Service Center
PROJECT LOCATION	Soboba Rd San Jacinto, CA
PROJECT NUMBER	E080-055

FIGURE NO. A-7

IFE BORING - GINT STD US LAB.GDT - 5/12/20 13:45 - P:\E080\E080-055 PRELIM SOBOBA HORSESHOE\GINT.GPJ

SITE PLAN



⊕ Approximate Location of Exploratory Boring

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A-8	Geotechnical Investigation Soboba Horseshoe Service Center, Phase 1 San Jacinto, California	
	Drawn By: DRL	Project No. E080-055
	Scale: 1" = 100' ±	Date: April 2020

APPENDIX B – Laboratory Testing

APPENDIX B

LABORATORY TESTING

Representative soil samples obtained from our borings were returned to our laboratory for additional evaluation and testing. Descriptions of the tests performed are provided below.

Unit Weight and Moisture Content: Ring samples were weighed and measured to evaluate their unit weight. A small portion of each sample was then tested for moisture content. The testing was performed per ASTM D2937 and D2216. The results of this testing are shown on the boring logs (Figure Nos. A-3 through A-7).

Maximum Density-Optimum Moisture: Two soil samples were selected for maximum density testing in accordance with ASTM D1557. The maximum density is compared to the field density of the soil to evaluate the existing relative compaction to the soil. This is useful in estimating the strength and compressibility of the soil. The results of this testing are presented graphically on Figure No. B-3.

Sieve Analysis: Six soil samples were selected for sieve analysis testing in accordance with ASTM D6913. These tests provide information for classifying the soil in accordance with the Unified Classification System. This classification system categorizes the soil into groups having similar engineering characteristics. The results of this testing are shown on Figure No. B-4 and B-5.

Plastic Index: Three samples were selected for plastic index testing in accordance with ASTM D4318. These tests provide information regarding soil plasticity and are also used for developing classifications for the soil in accordance with the Unified Classification System. The results are shown on Figure No. B-4 and B-5.

Analytical Testing: Two samples were selected to evaluate the concentration of soluble sulfates and chlorides, pH level, and resistivity of and within the on-site soils. The following table presents the results of this testing.

SAMPLE LOCATION	SAMPLE DEPTH (FT.)	WATER-SOLUBLE SULFATES (%)	CHLORIDES (PPM)	MINIMUM RESISTIVITY (OHM-CM)	PH
B-01	0.0-2.0	<.01	1,200	5,500	7.7
B-05	0.0-3.0	<.01	500	4,400	7.9

Expansion Index: One sample was selected for expansion index in accordance with ASTM D4829. This test provides information regarding the expansive characteristics of soil under standardized test conditions. The following table presents the results of this testing.

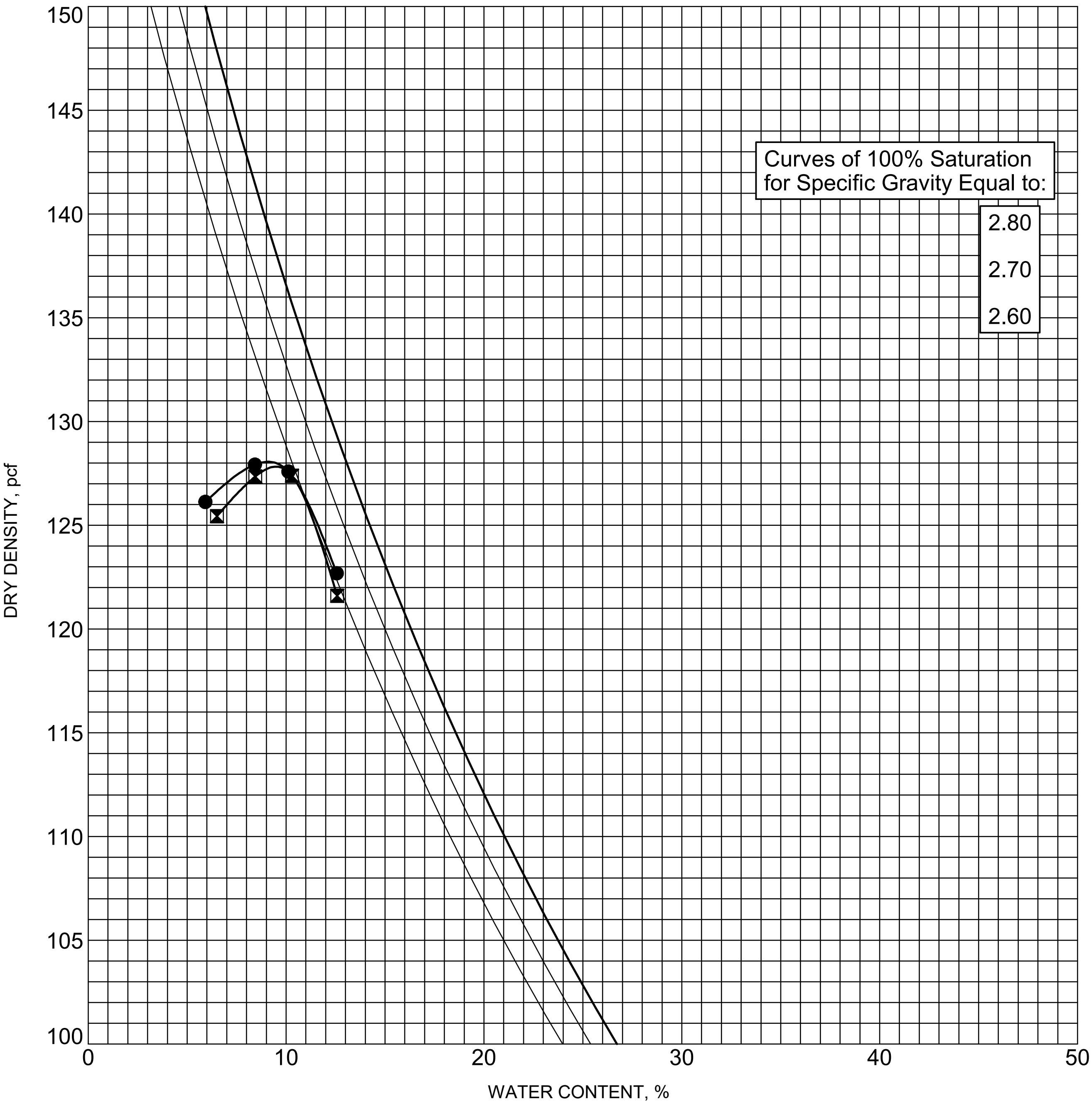
SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL DRY DENSITY (PCF)	INITIAL MOISTURE CONTENT (%)	EXPANSION INDEX	EXPANSION CLASS
B-02	2.0-6.3	108.3	10.3	23	Low

Direct Shear Strength: One sample was selected for direct shear strength testing in accordance with ASTM D3080. This testing measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation bearing capacity and lateral earth pressure. Test results are shown on Figure No. B-6.

Consolidation Testing: Three in-situ samples were selected for consolidation testing in general accordance with ASTM D2435. A fourth sample was remolded to 90 percent relative compaction and tested. The results are shown graphically on Figure Nos. B-7 through B-10.

GENERAL

All laboratory testing has been conducted in conformance with the applicable ASTM test methods by personnel trained and supervised in conformance with our QA/QC policy. Our test data only relates to the specific soils tested. Soil conditions typically vary and any significant variations should be reported to our laboratory for review and possible testing. The data presented in this report are for the use of the Engineering Resources of Southern California, Inc. only and may not be reproduced or used by others without written approval of Inland Foundation Engineering, Inc.



	BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
●	**B-01**	**2.0**	**SILTY SAND (SM)**	**128.1 PCF**	**9.0 %**
⧗	**B-05**	**0.0**	**SILTY, CLAYEY SAND (SC-SM)**	**127.8 PCF**	**9.5 %**

Inland Foundation Engineering, Inc.

MOISTURE-DENSITY CURVES (ASTM D1557)

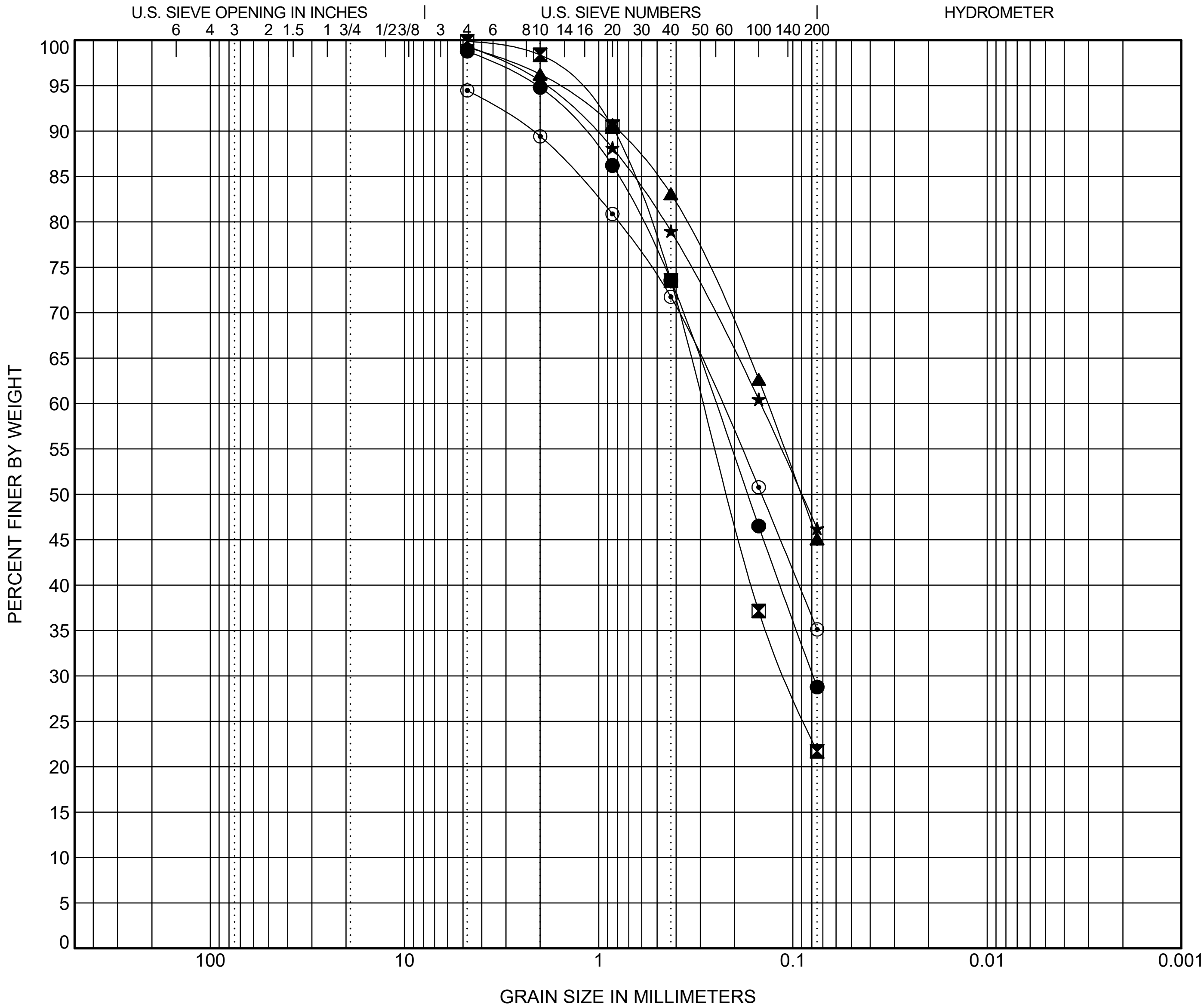
FIGURE NO. B-3

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd
San Jacinto, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**2.0**	**SILTY SAND (SM)**	**22**	**18**	**4**		
⧗	**B-01**	**15.5**	**SILTY SAND (SM)**					
▲	**B-02**	**2.0**	**SILTY, CLAYEY SAND (SC-SM)**	**25**	**20**	**5**		
★	**B-03**	**0.0**	**SILTY, CLAYEY SAND (SC-SM)**					
⊙	**B-05**	**0.0**	**SILTY, CLAYEY SAND (SC-SM)**	**26**	**21**	**5**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**2.0**	**4.75**	**0.252**	**0.079**			**70.0**	**28.8**	
⧗	**B-01**	**15.5**	**4.75**	**0.288**	**0.109**			**78.2**	**21.7**	
▲	**B-02**	**2.0**	**4.75**	**0.135**				**54.1**	**45.1**	
★	**B-03**	**0.0**	**4.75**	**0.147**				**53.1**	**46.2**	
⊙	**B-05**	**0.0**	**4.75**	**0.237**				**59.4**	**35.1**	

GRADATION CURVES (ASTM D422, ASTM D4318)

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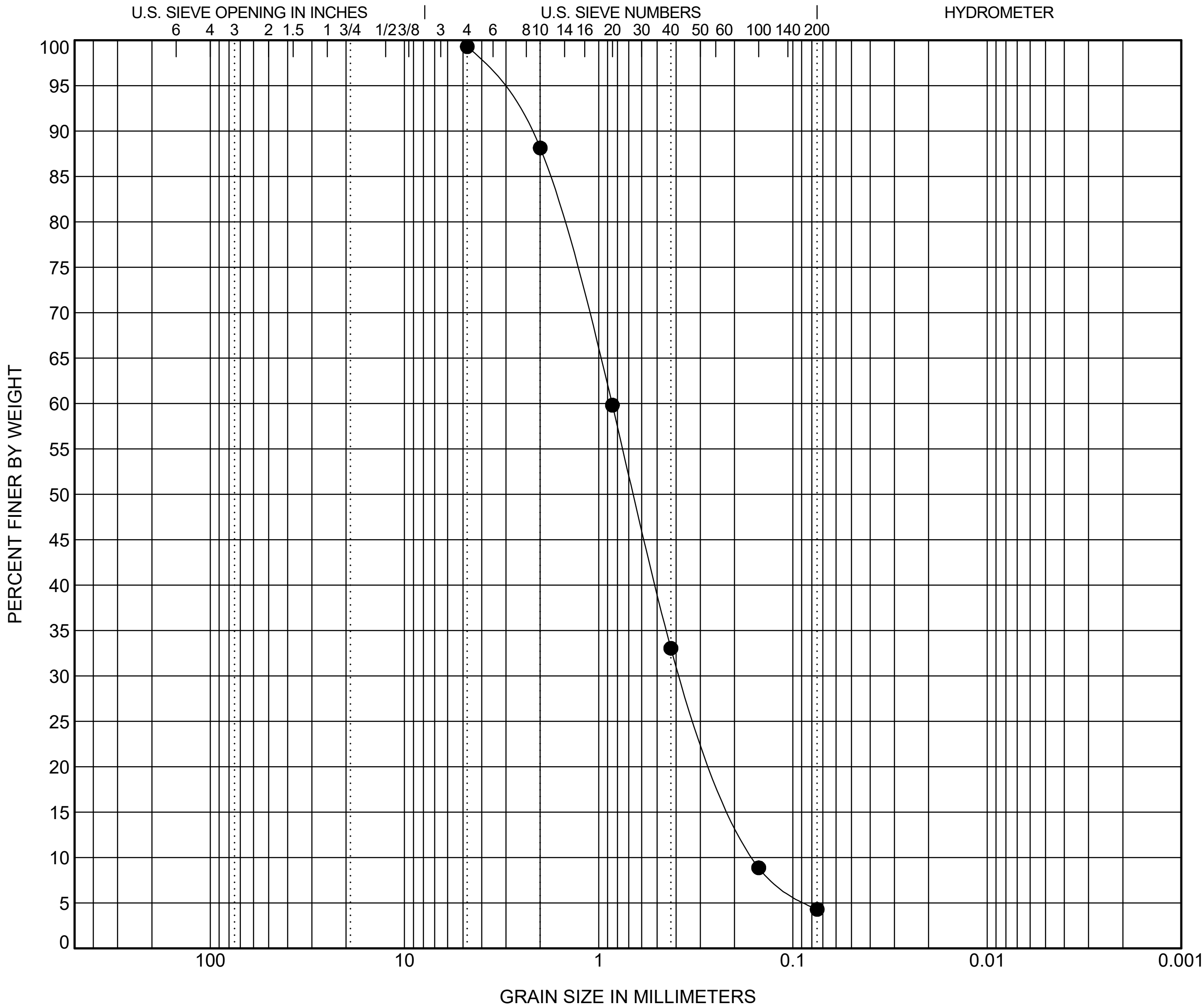
FIGURE NO. B-4

CLIENT Engineering Resources
PROJECT NAME Soboba Horeshoe Service Center
PROJECT NUMBER E080-055
PROJECT LOCATION Soboba Rd
San Jacinto, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-05**	**35.5**	**POORLY GRADED SAND (SP)**				**1.03**	**5.43**

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-05**	**35.5**	**4.75**	**0.855**	**0.373**	**0.157**		**95.0**	**4.3**	

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

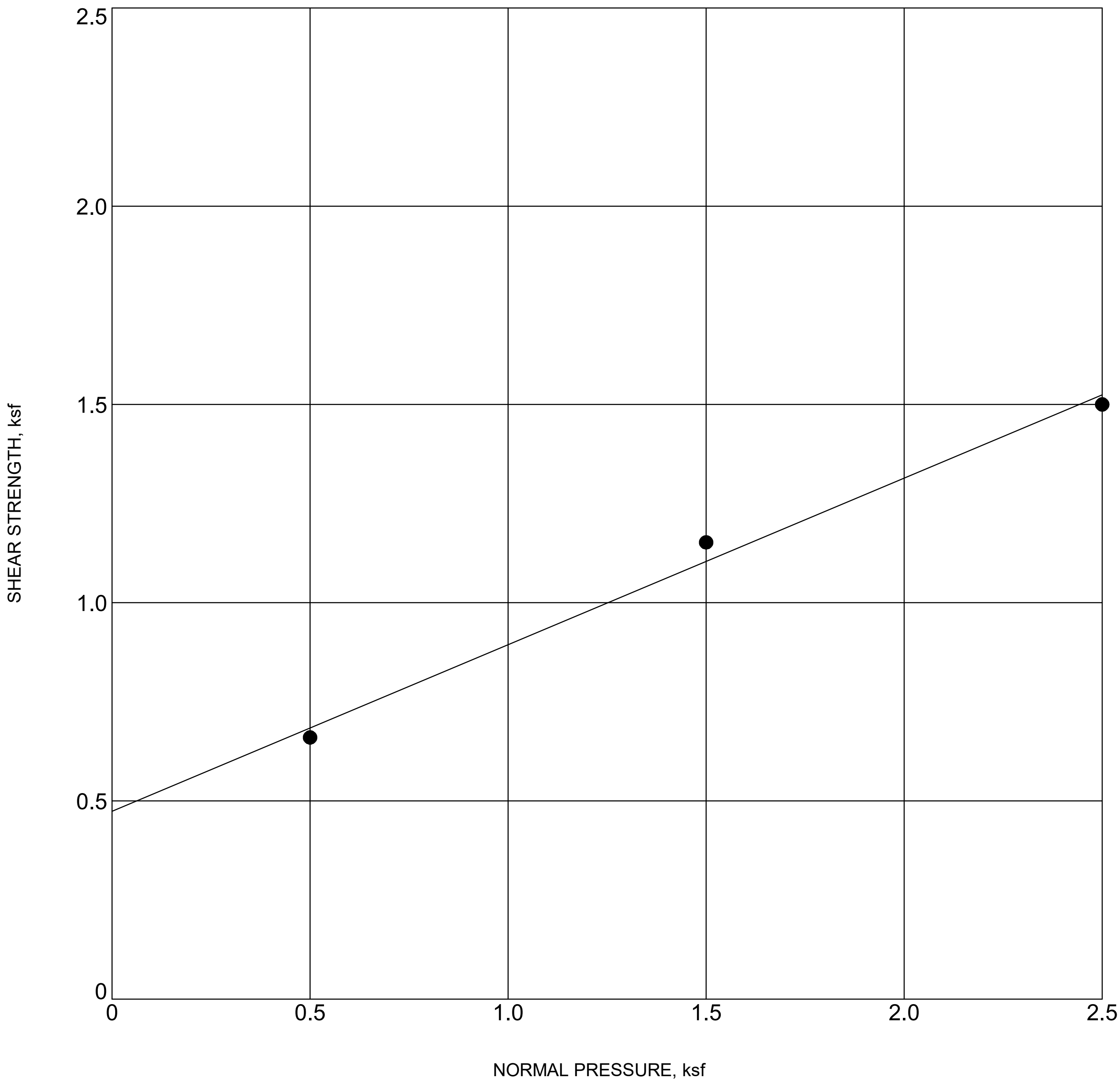
FIGURE NO. B-5

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd
San Jacinto, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-02	**2.5**	**SILTY, CLAYEY SAND (SC-SM)**	**105**	**18**	**0.5**	**23**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

Inland Foundation Engineering, Inc.

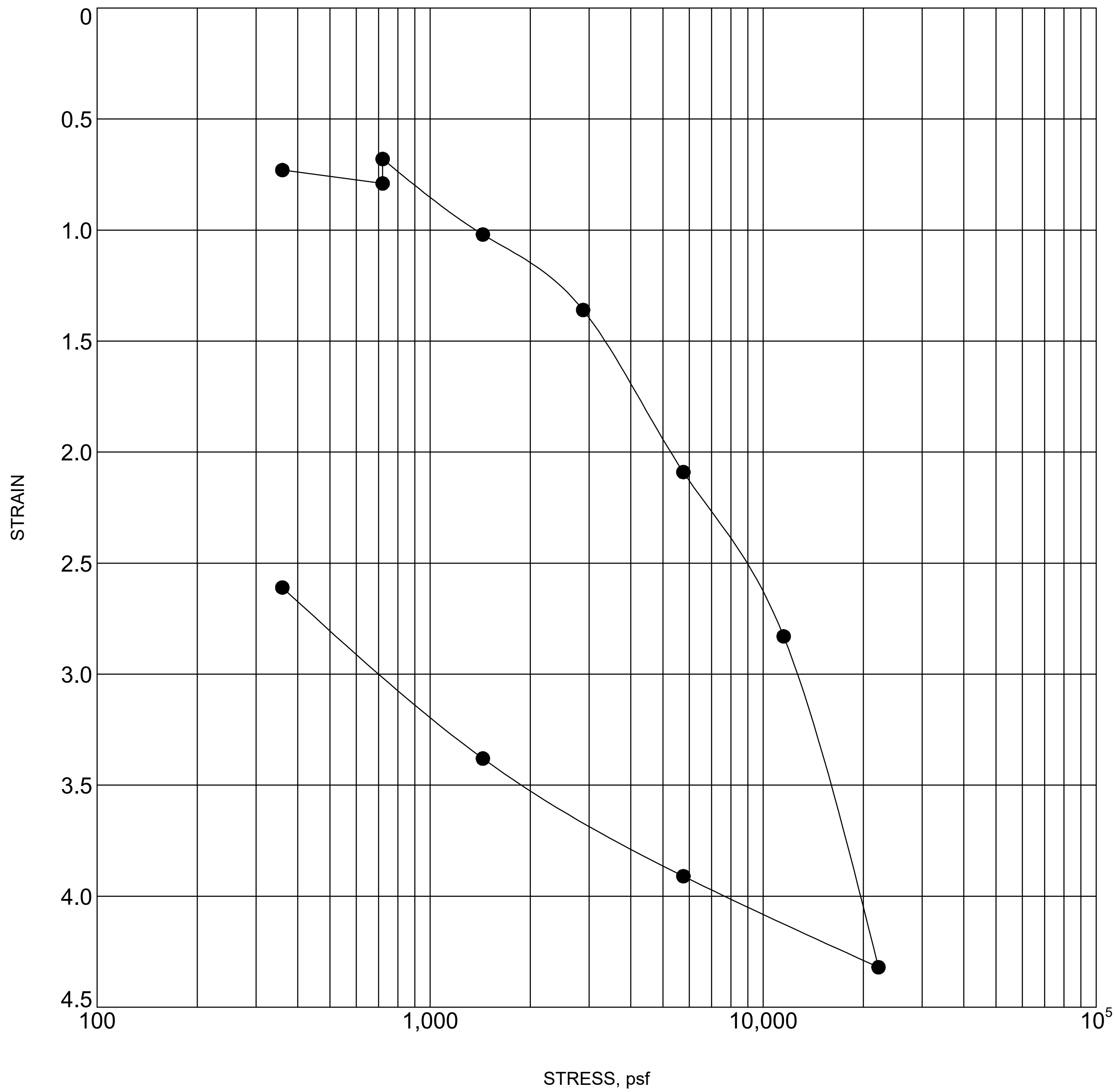
FIGURE NO. B-6

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd, San Jacinto, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-01**	**2.0**	**SILTY SAND (SM)**	**116**	**10**

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

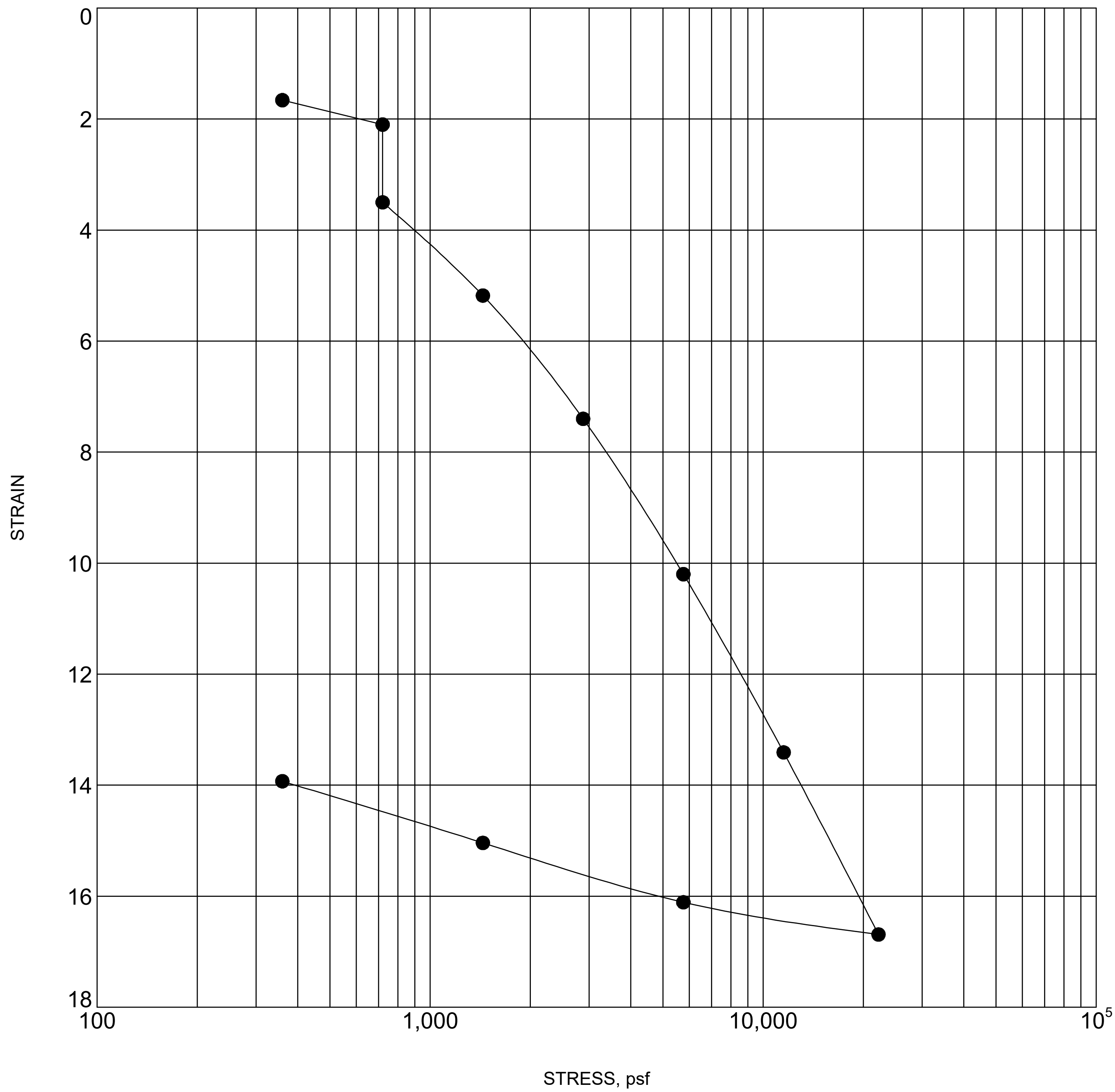
FIGURE NO. B-7

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd
San Jacinto, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-02**	**2.5**	**SILTY, CLAYEY SAND (SC-SM)**	**97**	**14**

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

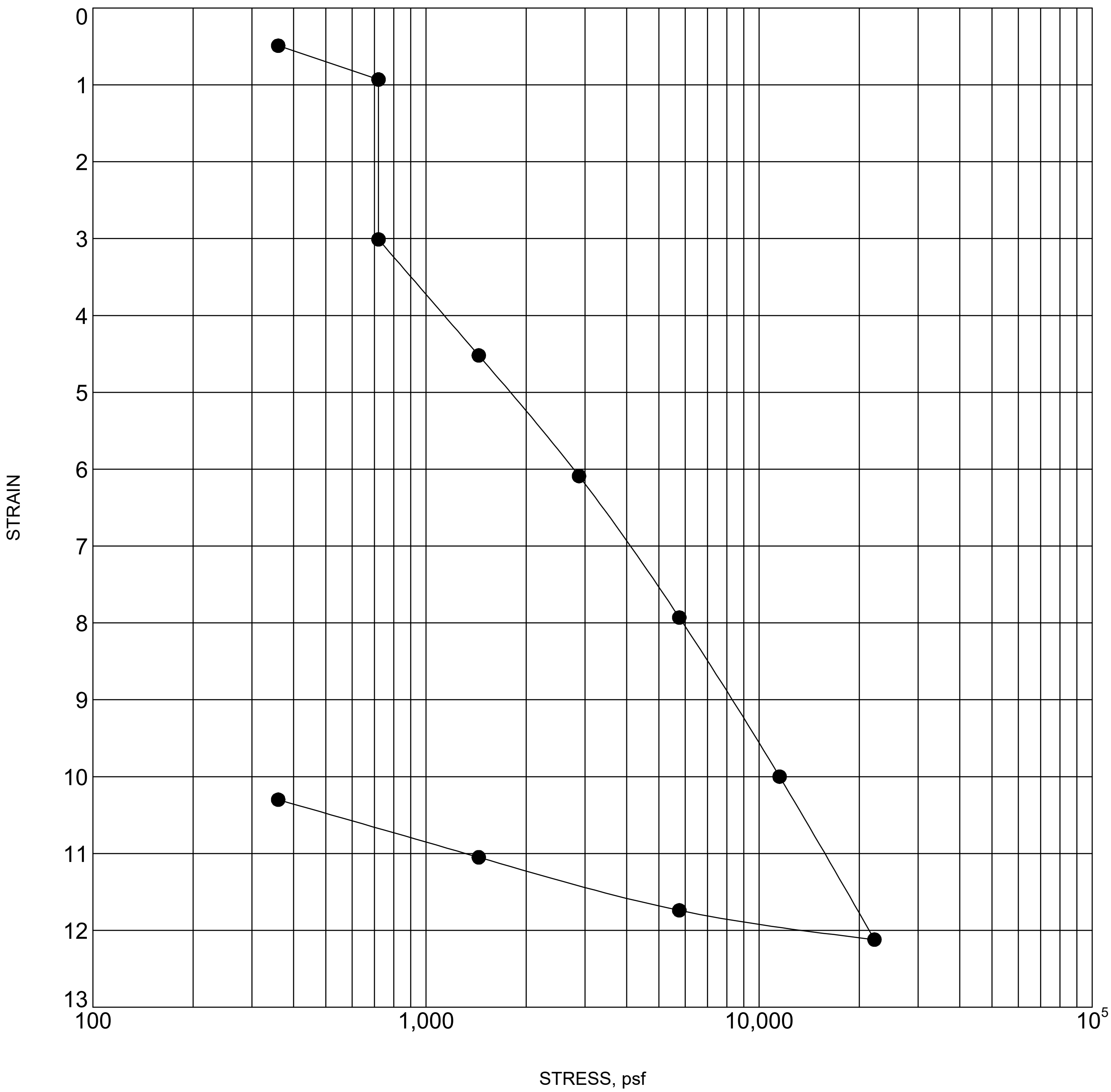
FIGURE NO. B-8

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd
San Jacinto, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-04**	**6.5**	**SILTY SAND (SM)**	**106**	**9**

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

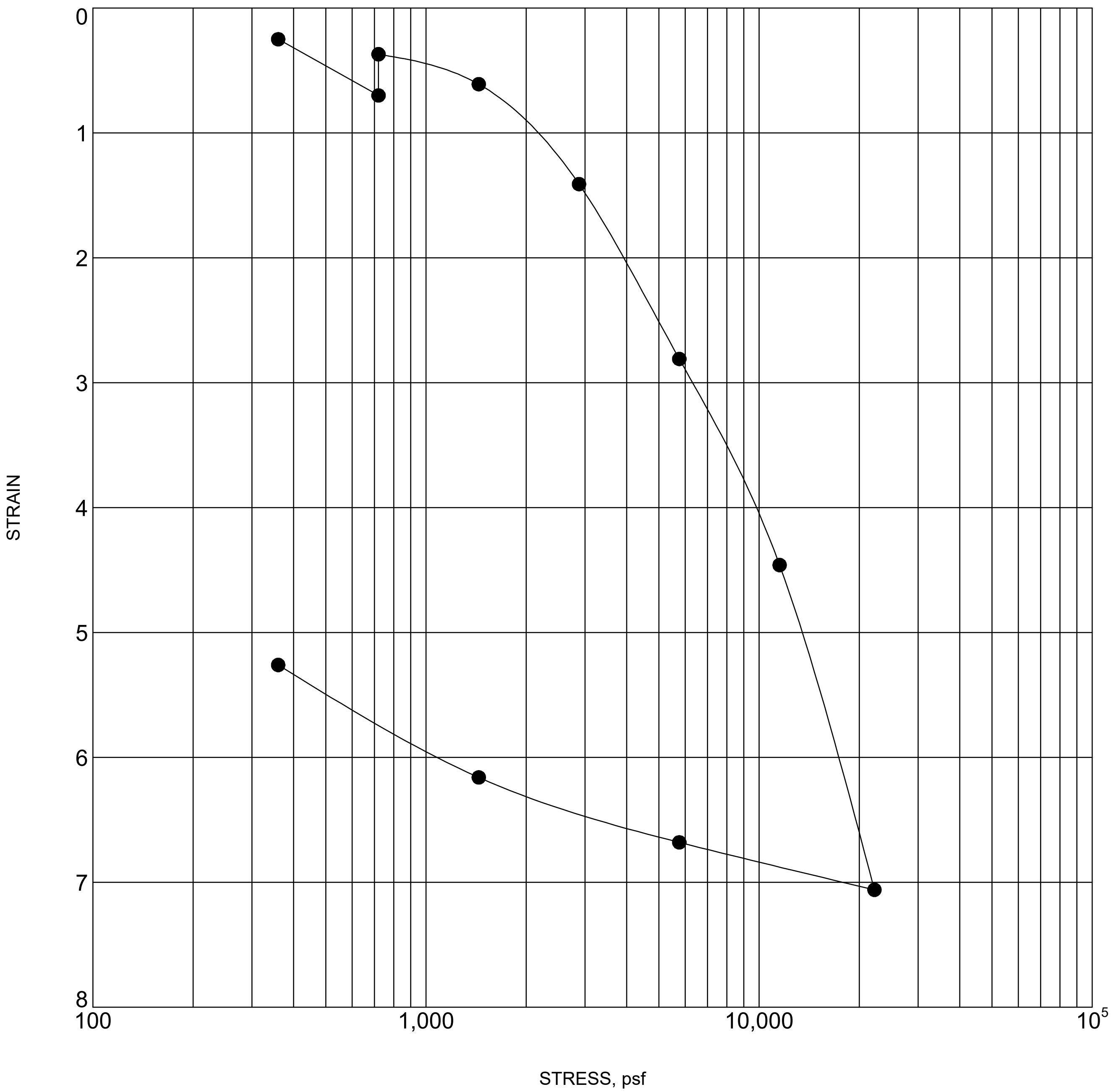
FIGURE NO. B-9

CLIENT Engineering Resources

PROJECT NAME Soboba Horeshoe Service Center

PROJECT NUMBER E080-055

PROJECT LOCATION Soboba Rd
San Jacinto, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-05**	**0.0**	**SILTY, CLAYEY SAND (SC-SM)**	**116**	**9**
			Note: Sample was remolded to 90% relative compaction		

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

FIGURE NO. B-10

CLIENT Engineering Resources
PROJECT NAME Soboba Horeshoe Service Center
PROJECT NUMBER E080-055
PROJECT LOCATION Soboba Rd
San Jacinto, CA

APPENDIX C –
Site-Specific Ground Motion Analysis

APPENDIX C

SITE-SPECIFIC GROUND MOTION ANALYSIS

Seismic Parameters: A site-specific ground motion analysis was conducted for this project in accordance with the 2019 California Building Code and ASCE 7-16. Additionally, a seismic shear-wave survey was conducted for this study for the purpose of determining the Site Classification and Vs_{30} input values for the ground motion analysis. The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps (OSHPD, 2020) and the California Building Code criteria (CBC, 2019), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017).

A summary of the site-specific analysis is shown on Figures C2 through C6.

SEISMIC DESIGN PARAMETERS SUMMARY

Project:	Soboba Gas Station	Lattitude:	33.7892
Project #:	0	Longitude:	-116.9278
Date:	5/5/20		

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	2.153	Figure 22-1
S_1=	0.862	Figure 22-2

Site Class per Table 20.3-1

Site Class=	D - Stiff Soil

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1	Table 11.4-1	=	1	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.7	Table 11.4-2	=	2.50	For Site Specific Analysis per ASCE7-16 21.3

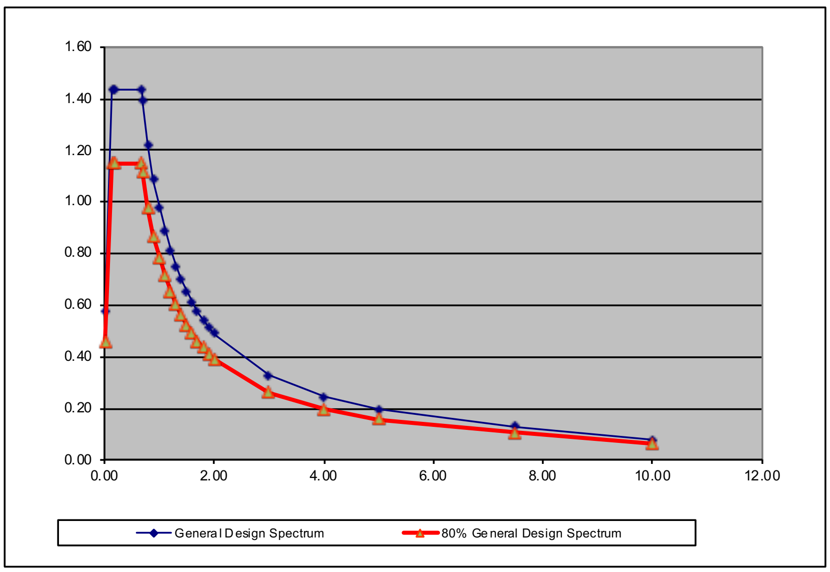
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	2.153	Equation 11.4-1	2.153	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.465	Equation 11.4-2	2.155	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}=	1.435	Equation 11.4-3
S_{D1}=	0.977	Equation 11.4-4

T_0=	0.136	sec	
T_S=	0.681	sec	
T_L=	8	sec	From Fig 22-12
PGA	0.916	g	
F_{PGA}=	1.1		From Table 11.8-1
C_{RS}=	0.898		Figure 22-17
C_{R1}=	0.881		Figure 22-18

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.58	0.460
0.14	1.44	1.148
0.20	1.44	1.148
0.68	1.44	1.148
0.70	1.40	1.116
0.80	1.22	0.977
0.90	1.09	0.868
1.00	0.98	0.782
1.10	0.89	0.710
1.20	0.81	0.651
1.30	0.75	0.601
1.40	0.70	0.558
1.50	0.65	0.521
1.60	0.61	0.488
1.70	0.57	0.460
1.80	0.54	0.434
1.90	0.51	0.411
2.00	0.49	0.391
3.00	0.33	0.261
4.00	0.24	0.195
5.00	0.20	0.156
7.50	0.13	0.104
10.00	0.08	0.063



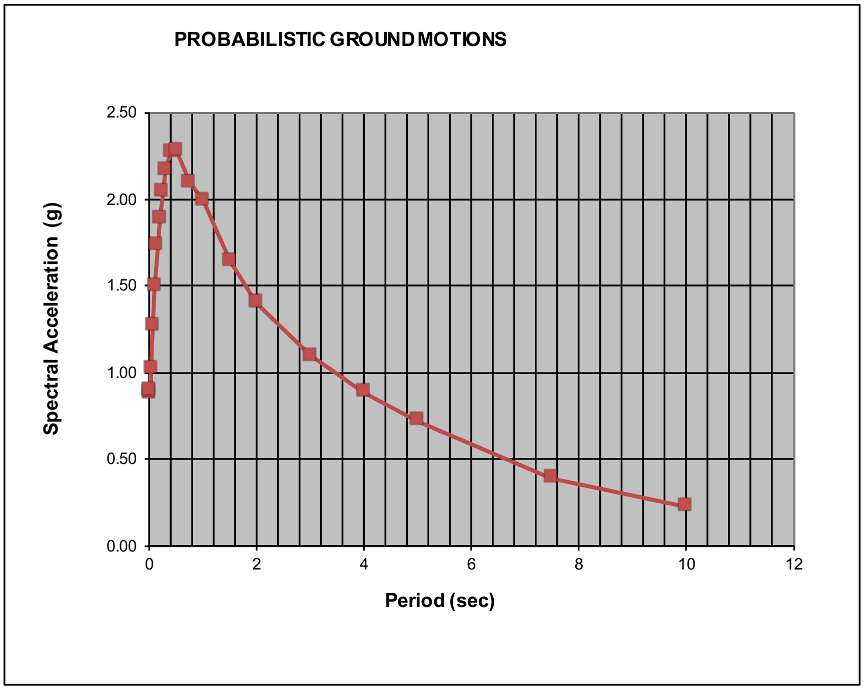
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

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

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships
Earthquake Rupture Forecast - UCERF3

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

T	Sa 2% in 50	MCER
0.01	0.98	0.88
0.02	0.99	0.89
0.03	1.00	0.90
0.05	1.14	1.02
0.08	1.42	1.28
0.10	1.67	1.50
0.15	1.94	1.74
0.20	2.11	1.90
0.25	2.28	2.05
0.30	2.43	2.17
0.40	2.54	2.27
0.50	2.57	2.29
0.75	2.37	2.10
1.00	2.27	2.00
1.50	1.87	1.65
2.00	1.59	1.40
3.00	1.25	1.10
4.00	1.01	0.89
5.00	0.82	0.72
7.50	0.45	0.40
10.00	0.26	0.23



S_s=	2.11	1.90
S_1=	2.27	2.00
PGA	0.86 g	

Risk Coefficients:			
C_{RS}	0.898	Figure 22-18	Get from Mapped Values
C_{R1}	0.881	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Input Parameters		San Jacinto - Claremont Fault
Fault		
M	= Moment magnitude	7.8
R_{RUP}	= Closest distance to coseismic rupture (km)	0.1
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	0.1
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	0.1
U	= Unspecified Faulting Flag (Boore et.al.)	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	1
δ	= Average dip of rupture plane (degrees)	89.99999
V_{S30}	= Average shear-wave velocity in top 30m of site profile	245.4
$F_{Measured}$		1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	1
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	2.4
Site Class		D
W (km)	= Fault rupture width (km)	15
F_{AS}	= 0 for mainshock; 1 for aftershock	0
σ	=Standard Deviation	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

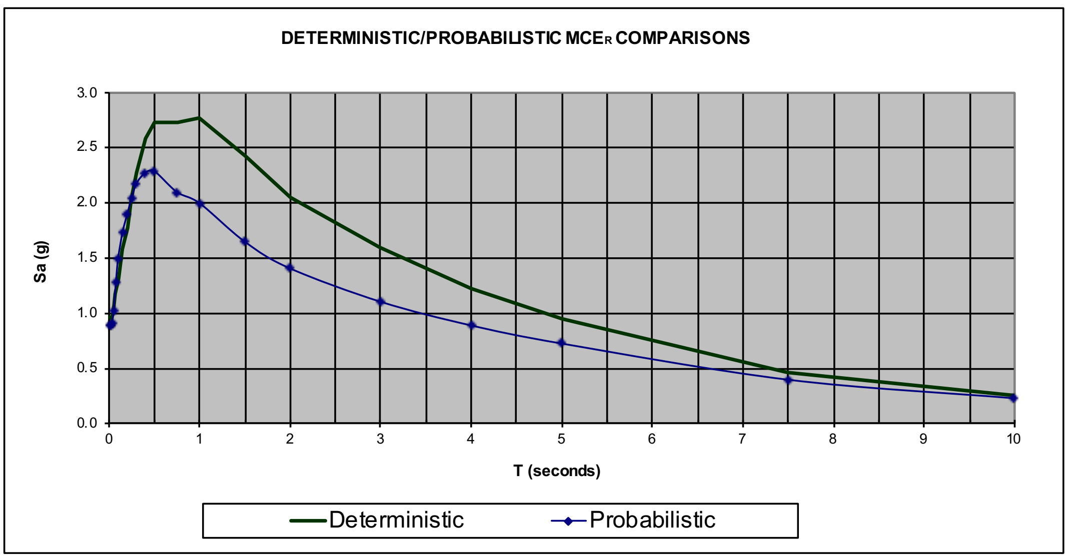
T	Median S_a (Average)	*Corrected* S_a (per ASCE7-16)*	*Scaled $S_{a(Average)}$*
0.010	0.87	0.96	0.96
0.020	0.87	0.96	0.96
0.030	0.87	0.96	0.96
0.050	0.92	1.01	1.01
0.075	1.06	1.17	1.17
0.100	1.21	1.33	1.33
0.150	1.43	1.57	1.57
0.200	1.62	1.78	1.78
0.250	1.83	2.04	2.04
0.300	2.02	2.27	2.27
0.400	2.25	2.59	2.59
0.500	2.32	2.73	2.73
0.750	2.21	2.73	2.73
1.000	2.13	2.77	2.77
1.500	1.82	2.42	2.42
2.000	1.52	2.06	2.06
3.000	1.14	1.60	1.60
4.000	0.84	1.22	1.22
5.000	0.63	0.94	0.94
7.500	0.30	0.46	0.46
10.000	0.17	0.25	0.25
PGA	0.87		0.87 g
Max Sa=	2.77		
Fa =	1.00	Per ASCE7-16 21.2.2	
1.5XFa=	1.5		
Scaling Factor=	1.00		

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.96	0.88	0.88	Probabilistic Governs
0.020	0.96	0.89	0.89	Probabilistic Governs
0.030	0.96	0.90	0.90	Probabilistic Governs
0.050	1.01	1.02	1.01	Deterministic Governs
0.075	1.17	1.28	1.17	Deterministic Governs
0.100	1.33	1.50	1.33	Deterministic Governs
0.150	1.57	1.74	1.57	Deterministic Governs
0.200	1.78	1.90	1.78	Deterministic Governs
0.250	2.04	2.05	2.04	Deterministic Governs
0.300	2.27	2.17	2.17	Probabilistic Governs
0.400	2.59	2.27	2.27	Probabilistic Governs
0.500	2.73	2.29	2.29	Probabilistic Governs
0.750	2.73	2.10	2.10	Probabilistic Governs
1.000	2.77	2.00	2.00	Probabilistic Governs
1.500	2.42	1.65	1.65	Probabilistic Governs
2.000	2.06	1.40	1.40	Probabilistic Governs
3.000	1.60	1.10	1.10	Probabilistic Governs
4.000	1.22	0.89	0.89	Probabilistic Governs
5.000	0.94	0.72	0.72	Probabilistic Governs
7.500	0.46	0.40	0.40	Probabilistic Governs
10.000	0.25	0.23	0.23	Probabilistic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE$_R$	80% General Design Response Spectrum (per ASCE 7-16 Figure 11.4-1)	Design Response Spectrum	TXSa
0.01	0.59	0.51	0.59	
0.02	0.59	0.56	0.59	
0.03	0.60	0.61	0.61	
0.05	0.68	0.71	0.71	
0.08	0.78	0.84	0.84	
0.10	0.88	0.97	0.97	
0.15	1.05	1.15	1.15	
0.20	1.18	1.15	1.18	
0.25	1.36	1.15	1.36	
0.30	1.45	1.15	1.45	
0.40	1.52	1.15	1.52	
0.50	1.52	1.15	1.52	
0.75	1.40	1.15	1.40	
1.00	1.33	1.15	1.33	1.33
1.50	1.10	0.77	1.10	1.65
2.00	0.94	0.57	0.94	1.87
3.00	0.73	0.38	0.73	2.20
4.00	0.59	0.29	0.59	2.37
5.00	0.48	0.23	0.48	2.41
7.50	0.26	0.15	0.26	
10.00	0.15	0.09	0.15	

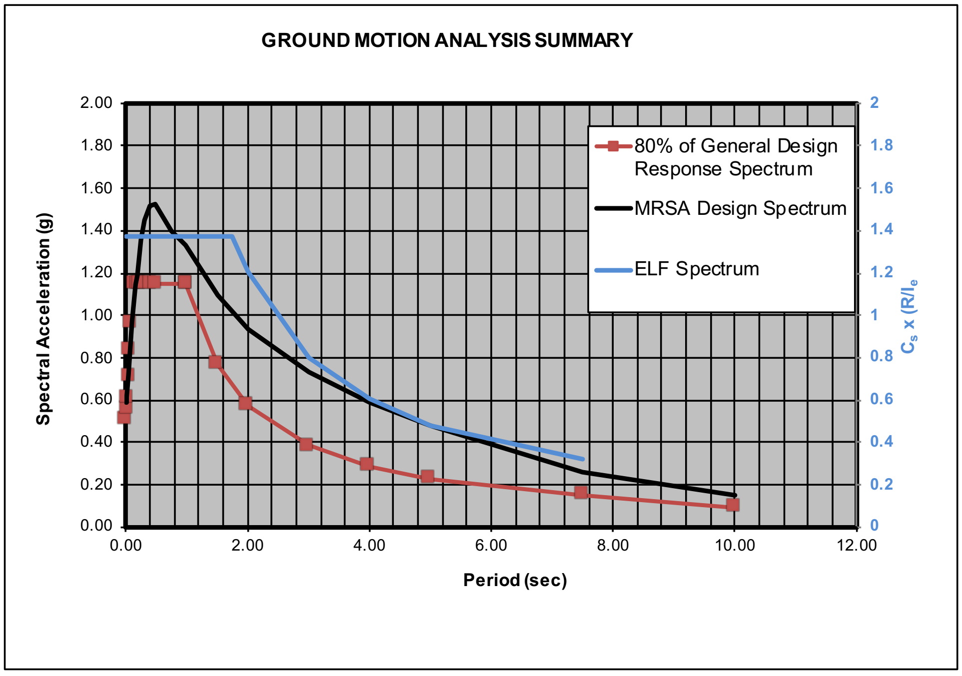
Highest value of S_a for any period exceeding 0.2 sec.=	1.52
90%of Highest Value =	1.37
Maximum TSa from T=1s-5s =	2.41

S_{DS}=	1.37
S_{D1}=	2.41
T_s =	1.76

S_{MS}=	2.058
S_{M1}=	3.617

PGA Determination:

Site Coefficient F_{PGA}=	1.1	
Mapped PGA=	0.92	Figure 22-7
PGA_M =	1.01	g
Deterministic PGA =	0.87	g
Probabilistic PGA =	0.86	g
Lesser of Deterministic/Probabilistic =	0.86	g
80% of $PGA_{M=}$	0.81	g
MCE_G PGA=	0.86	g



APPENDIX D –
Seismic Settlement Analysis

APPENDIX D

SEISMIC SETTLEMENT ANALYSIS

Seismic settlement potential was evaluated using the GeoSuite® computer program (version 2.2.2.14). The seismic parameters included a horizontal acceleration of 0.85g and a Moment Magnitude of 7.80. This is based on recent published parameters for faults in California from the *Working Group on Earthquake Probabilities* (Field and others, 2008; Willis and others, 2008), considering a cascading effect of rupture along the entire length of the San Jacinto Fault Zone. We analyzed the soil profile logged for exploratory boring B-01. The potential for "dry sand" seismically-induced settlement was evaluated using the GeoSuite® computer program, based on Pradel's method (1998). The program calculates corrected normalized SPT N-values $(N_1)_{60}$ using the following formula (SCEC, 1999).

$$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$$

Where; N_M = measured standard penetration resistance. Modified California sample blowcounts were converted to SPT blowcounts using Burmister's formula (1948) prior to input in the program. The modified California sample blowcounts were also corrected to account for lined samplers, as described in the C_S factor discussion below.

C_N = depth correction factor. GeoSuite® calculates C_N for each layer in the soil profile using the relationship suggested by Idriss and Boulanger (2008)

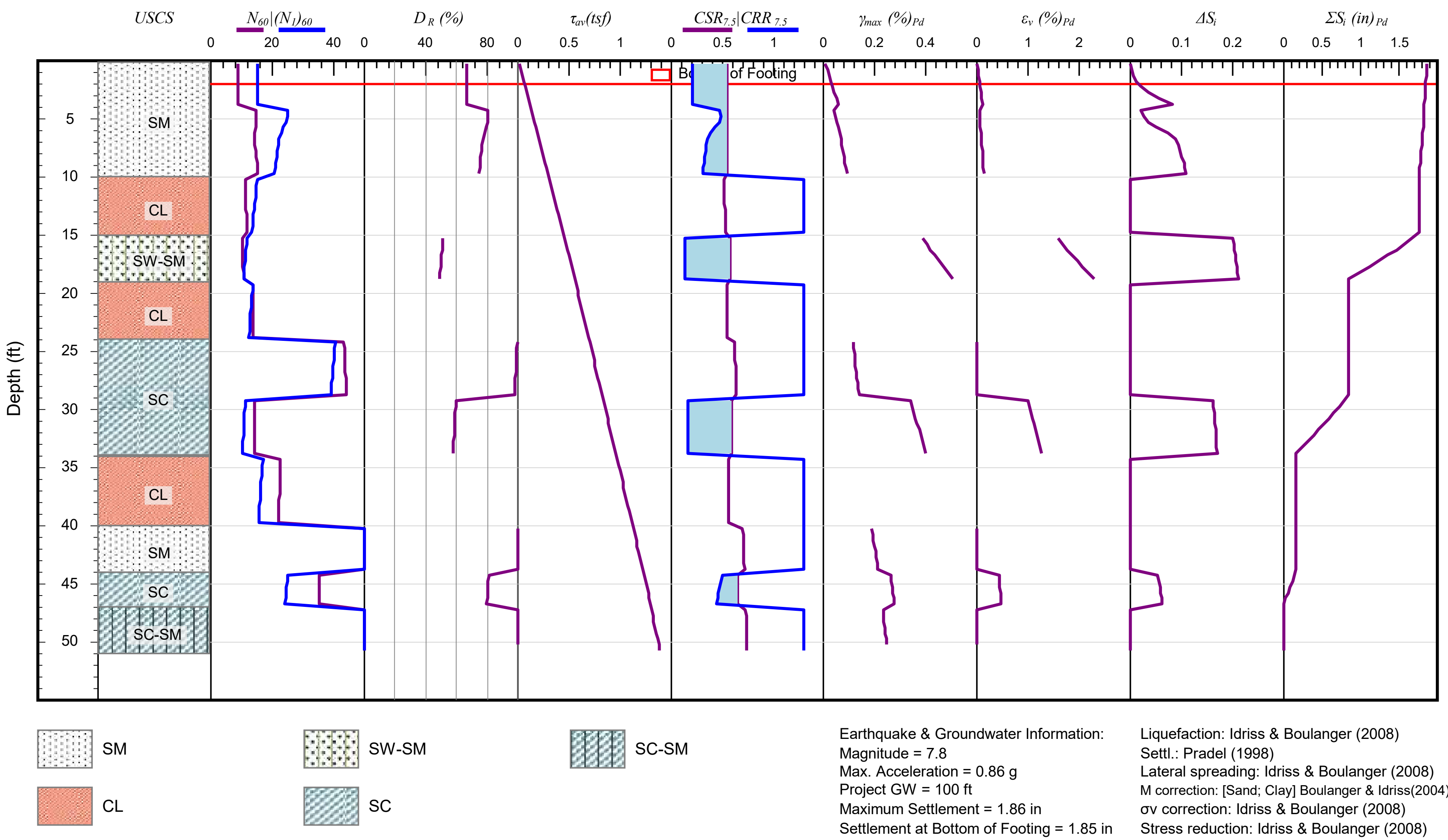
C_E = hammer energy ratio (ER) correction factor. A C_E factor of 1.3 was applied for the automatic trip hammer used during drilling. This was calculated using the relationship suggested by Idriss and Boulanger (2008) and SPT hammer energy measurements provided by the drilling subcontractor.

C_B = borehole diameter correction factor. A C_B factor of 1.0 was applied for the 8-inch diameter hollow-stem augers with inside diameters of four (4) inches (SCEC 1999).

C_R = rod length correction factor. GeoSuite® applies a C_R factor for each layer in the soil profile using the values in Table 5.2 of the 1999 SCEC guidelines, and assuming a rod stick up length (above the ground surface) of 3 feet.

C_S = correction factor for samplers with or without liners. SPT samplers without liners were used for this project. For SPT samplers without liners, GeoSuite® applies a C_S factor for each layer in the soil profile using the relationships from Seed et al. (1984) and suggested by Idriss and Boulanger (2008). Since GeoSuite® applies a C_S factor to all layers in the soil profile, it is necessary to adjust blowcounts for modified California samplers with liners. This was done through an iterative process by initially dividing the modified California sampler blowcounts by an assumed C_S value of 1.2 prior to input in the program. Calculated C_S values were then checked against the assumed values and adjusted where necessary, so that the actual applied C_S value for modified California samples is 1.0.

The results of our analysis are shown on Figure D-3.



Earthquake & Groundwater Information:
Magnitude = 7.8
Max. Acceleration = 0.86 g
Project GW = 100 ft
Maximum Settlement = 1.86 in
Settlement at Bottom of Footing = 1.85 in

Liquefaction: Idriss & Boulanger (2008)
Settl.: Pradel (1998)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand; Clay] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists

Seismic Settlement Potential - SPT Data

Project:	Soboba Horseshoe Service Center				
Location:	San Jacinto, California				
Project No.:	E080-055	Boring No.:	B-01	Figure:	D-3

P:\E080\E080-055 Prelim Soboba Horseshoe\GeoSuite\GeoSuite_E080-055_B-01.csv

GeoSuite© Version 2.2.2.23. Developed by Fred Yi, PhD, PE, GE

Prepared at 5/7/2020 10:47:31 AM