GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET
APN 314-170-009 & 314-170-010
PERRIS, CALIFORNIA

-Prepared By-

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May 31, 2022

Project No. 644-22018
22-05-068

Dedeaux Properties
100 Wilshire Boulevard, Suite 250
Santa Monica, California 90401

Subject: Geotechnical Investigation

Project: Proposed Industrial Building
945-995 West Markham Street
APN 314-170-009 & 314-170-010
Perris, California

Sladden Engineering is pleased to present the results of our geotechnical investigation performed for the proposed new industrial building to be constructed on the site (APN 314-170-009 & 010) located at 945-995 West Markham Street in the City of Perris, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated March 7, 2022 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site in order to provide recommendations for foundation design and site preparation. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

The opinions, recommendations and design criteria presented in this report are based on our field exploration program, laboratory testing and engineering analyses. Based on the results of our investigation, it is our professional opinion that the proposed project should be feasible from a geotechnical perspective provided that the recommendations presented in this report are implemented into design and carried out through construction.

We appreciate the opportunity to provide service to you on this project. If you have any questions regarding this report, please contact the undersigned.

Respectfully submitted,
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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the proposed industrial building to be constructed on the site (APN 314-170-009 & 010) located at 945-995 West Markham Street in the City of Perris, California. The site is located at approximately 33.8513 degrees North latitude and 117.2471 degrees West longitude. The approximate location of the site is indicated on the Site Location Map (Figure 1).

Our investigation was conducted in order to evaluate the engineering properties of the subsurface materials, to evaluate their *in-situ* characteristics, and to provide engineering recommendations and design criteria for site preparation, foundation design and the design of various site improvements. This study also includes a review of published and unpublished geotechnical and geological literature regarding seismicity at and near the subject site.

PROJECT DESCRIPTION

Based on the conceptual site plan (AO, 2022), it is our understanding that the proposed project will consist of constructing a new 83,000 square foot (ft^2) industrial building. The new building will consist of 79,000 ft^2 of warehouse space and 4,000 ft^2 of mezzanine and office space. The project will also include paved parking areas, truck loading docks, exterior concrete flatwork, underground utilities, landscape areas and various other improvements. For our analyses, we expect that the proposed industrial building will be of reinforced concrete tilt-up construction supported on conventional shallow spread footings and concrete slabs-on-grade.

We anticipate that grading will be limited to minor cuts and fills in order to accomplish the desired pad elevations and provide adequate gradients for site drainage. This does not include the removal and re-compaction of foundation bearing soil within the building envelope. Upon completion of precise grading plans, Sladden should be retained in order to ensure that the recommendations presented within in this report are incorporated into the design of the proposed project.

Structural foundation loads were not available at the time of this report. Based on our experience with relatively lightweight concrete tilt-up structures, we expect that isolated column loads will be less than 50 kips and continuous wall loads will be less than 5.0 kips per linear foot. If these assumed loads vary significantly from the actual loads, we should be consulted to verify the applicability of the recommendations provided.

SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Exploration of the site was achieved by drilling seven (7) exploratory boreholes to depths between approximately 5 and 48 feet below the existing ground surface (bgs). Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- The excavation of seven (7) exploratory boreholes to depths between approximately 5 and 48 feet bgs in order to characterize the subsurface soil conditions. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- The performance of laboratory testing on selected samples to evaluate their engineering characteristics.
- The review of geologic literature with respect to potential geologic hazards.
- The performance of engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

SITE CONDITIONS

The site is located on the south side of West Markham Street between Patterson Avenue and North Webster Avenue in the City of Perris, California. The property consists of two (2) parcels that are formally identified by the County of Riverside as APN 314-170-009 & 010. The site occupies approximately 4.08 acres of land. At the time of our investigation, the project site was occupied by scattered residential structures and automobile/truck parking areas. The subject site is bounded by West Markham Street to the north, and developed properties to the east, south and west.

The project site is relatively level with minimal surface gradients. According to the USGS 7.5′ Perris Quadrangle map (2015), the site is at an approximate elevation of 1,490 feet above mean sea level (MSL).

No ponding water or surface seeps were observed at or near the site during our investigation conducted on April 4, 2022. Site drainage appears to be controlled via sheet flow and surface infiltration.

GEOLOGIC SETTING

The project site is located in the Peninsular Ranges Physiographic Province of California. The Peninsular Ranges are mountainous areas that extend from the western edge of the continental borderland to the Salton Trough and from the Transverse Ranges Physiographic Province in the north to the tip of Baja California in the south. The Peninsular Ranges Physiographic Province is characterized by northwest-trending topographic and structural features. The province is characterized by elongated, northwest-southeast trending mountain ranges and valleys and is truncated at its northern margin by the east-west trending Transverse Ranges. Mountainous areas of the Peninsular Ranges Physiographic Province generally consist of Igneous, metasedimentary and metavolcanic rocks. However, plutonic rocks of the Southern California Batholith are the dominant basement rock exposed (Jahns, 1954).

The site has been mapped by Morton (2003) to be immediately underlain by young alluvial valley deposits consisting of well-indurated sand deposits (Qvof). The geologic setting for the site and site vicinity is illustrated on the Regional Geologic Map, Figure 2.

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling seven (7) exploratory boreholes on the site. The approximate locations of the boreholes are illustrated on the Exploration Location Plan (Figure 3). The boreholes were advanced using a truck-mounted Mobile B-61 drill-rig equipped with 8-inch outside diameter hollow stem augers. A representative of Sladden was on-site to log the materials encountered and retrieve samples for laboratory testing and engineering analyses.

During our field investigation, disturbed soil was encountered to a maximum depth of approximately two (2) feet below the (existing) ground surface (bgs) within our bore locations. Underlying the disturbed soil, native alluvial materials were encountered to the maximum explored depth of approximately 48 feet bgs. The native soil consists primarily of clayey sand (SC) and sandy clay (CL). Sampler penetration resistance as measured by field blow counts indicates that density generally increases with depth.

The final logs represent our interpretation of the contents of the field logs, and the results of the laboratory observations and tests of the field samples. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradual and variable across the site.

Groundwater was not encountered during our field investigation. Based on our experience in the project vicinity, and our review of groundwater elevations in the project vicinity (CDWR, 2022), it is our opinion that groundwater should not be a factor during construction of the proposed project.

SEISMICITY AND FAULTING

The southwestern United States is a tectonically active and structurally complex region, dominated by northwest trending dextral faults. The faults of the region are often part of complex fault systems, composed of numerous subparallel faults that splay or step from the main fault traces. Strong seismic shaking could be produced by any of these faults during the design life of the proposed project.

We consider the most significant geologic hazard to the project to be the potential for moderate to strong seismic shaking that is likely to occur during the design life of the project. The proposed project is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene epoch (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

Table 1 lists the closest known potentially active faults that was generated in part using the EQFAULT computer program (Blake, 2000), as modified using the fault parameters from The Revised 2002 California Probabilistic Seismic Hazard Maps (Cao et al, 2003), Southern Earthquake Data Center (SCEDC, 2022), Building Seismic Safety Council (BSSC, 2014) and the Quaternary Fault and Fold Database of the United States (USGS, 2022a). This table does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any of the other faults in the region.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
San Jacinto – San Jacinto Valley	13.3	7.03*
San Jacinto – San Bernardino	18.4	6.7
Elsinore – Glen Ivy	23.5	6.8
Elsinore – Temecula	25.0	6.8
Chino – Central Avenue (Elsinore)	29.8	6.7
San Andreas – Southern	32.1	7.5
San Andreas – San Bernardino	32.1	7.5
San Jacinto – Anza	32.9	7.2
Whittier	36.0	6.8
Cucamonga	40.8	6.9

*BSSC (2014)

SITE SPECIFIC GROUND MOTION PARAMETERS

Sladden has reviewed the 2019 California Building Code (CBC) and ASCE7-16 and developed site specific ground motion parameters for the subject site. The project Seismic Design Maps and site-specific ground motion parameters are summarized in the following table and included within Appendix C. The project Structural Engineer should verify that all design parameters provided are applicable for the subject project.

TABLE 2
GROUND MOTION PARAMETERS

Latitude / Longitude	33.8513/-117.2471
Risk Category	II
Site Class	D
Code Reference Documents	ASCE 7-16; Chapter 11 & 21

Description	**Type**	**Map Based**	**Site-Specific**
MCE$_R$ Ground Motion (0.2 second period)	S_S	1.5	---
MCE$_R$ Ground Motion (1.0 second period)	S_1	0.572	---
Site-Modified Spectral Acceleration Value	S_{MS}	1.5	**1.369**
Site-Modified Spectral Acceleration Value	S_{M1}	null	**0.898**
Numeric Seismic Design Value at 0.2 second SA	S_{DS}	1	**0.913**
Numeric Seismic Design Value at 1.0 second SA	S_{D1}	null	**0.599**
Site Amplification Factor at 0.2 second	F_a	1	**1.0**
Site Amplification Factor at 1.0 second	F_v	null	**2.5**
Site Peak Ground Acceleration	PGA_M	0.55	**0.55**

GEOLOGIC HAZARDS

The subject site is located in an active seismic zone and will likely experience strong seismic shaking during the design life of the proposed project. In general, the intensity of ground shaking will depend on several factors including: the distance to the earthquake focus, the earthquake magnitude, the response characteristics of the underlying materials, and the quality and type of construction. Geologic hazards and their relationship to the site are discussed below.

I. Surface Rupture. Surface rupture is expected to occur along preexisting, known active fault traces. However, surface rupture could potentially splay or step from known active faults or rupture along unidentified traces. Based on review of Jennings (1994), CGS (2022) and Morton (2003), known faults are not mapped on the site. In addition, no signs of active surface faulting were observed during our review of non-stereo digitized photographs of the site and site vicinity (Google, 2022). Finally, no signs of active surface rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified on-site during our field investigation. Therefore, it is our opinion that risks associated with primary surface ground rupture should be considered "low".

II. Ground Shaking. The site has been subjected to past ground shaking by faults that traverse through the region. Strong seismic shaking from nearby active faults is expected to produce strong seismic shaking during the design life of the proposed project. A site-specific approach determined the peak ground acceleration (PGAm) at the site to be 0.55g.

III. Liquefaction/Seismic Settlement. Liquefaction is the process in which loose, saturated granular soil loses strength as a result of cyclic loading. The strength loss is a result of a decrease in granular sand volume and a positive increase in pore pressures. Generally, liquefaction can occur if all of the following conditions apply; liquefaction-susceptible soil, groundwater within a depth of 50 feet or less, and strong seismic shaking. The site is located within a "low" liquefaction potential zone (RCMMC, 2022). Based on the relatively dense nature of the underlying native earth materials, risks associated with liquefaction are considered "low".

IV. Tsunamis and Seiches. Because the site is situated at an elevated inland location and is not immediately adjacent to any impounded bodies of water, risk associated with tsunamis and seiches is considered "negligible".

V. Slope Failure, Landslides, Rock Falls. The site is situated on relatively level ground and is not immediately adjacent to any slopes or hillsides that could be potentially susceptible to slope instability. No signs of slope instability in the form of landslides, rock falls, earthflows or slumps were observed at or near the subject site during our investigation. As such, risks associated with slope instability should be considered "negligible".

VI. Expansive Soil. Expansion Index testing of a bulk sample was performed in order to evaluate the expansive potential of the materials underlying the site. Based the results of our laboratory testing (EI = 15), the materials underlying the site are considered to have a "very low" expansion potential.

VII. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation. However, risks associated with flooding and erosion should be evaluated and mitigated by the project design Civil Engineer.

CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project should be feasible from a geotechnical perspective provided that the recommendations provided in this report are incorporated into design and carried out through construction. The main geotechnical concerns in the design and construction of the proposed project are the presence of the existing structures and improvements, and potentially compressible surface and near surface soil.

Because of the somewhat soft and compressible condition of the near surface native soil, remedial grading including overexcavation and recompaction is recommended for the proposed building and foundation areas. We recommend that remedial grading within the proposed building areas include over-excavation and/or re-compaction of the artificial fill and primary foundation bearing soil. Specific recommendations for site preparation are presented in the Earthwork and Grading section of this report.

Groundwater was encountered during our field investigation. Based on the conditions encountered during our field investigation, groundwater should not be a factor in design or during the construction of the proposed project.

Caving did occur to varying degrees within each of our exploratory bores and the surface soil may be susceptible to caving within deeper excavations. All excavations should be constructed in accordance with the normal CalOSHA excavation criteria. Based on our observations of the materials encountered, we anticipate that the subsoil will conform to that described by CalOSHA as Type C. Soil conditions should be verified in the field by a "Competent person" employed by the Contractor.

The following recommendations present more detailed design criteria that have been developed based on our field investigation and laboratory testing.

EARTHWORK AND GRADING

All earthwork including excavation, backfill and preparation of the surface soil, should be performed in accordance with the geotechnical recommendations presented in this report and portions of the local regulatory requirements, as applicable. All earth work should be performed under the observation and testing of a qualified soil engineer. The following geotechnical engineering recommendations for the proposed project are based on observations from the field investigation program, laboratory testing and geotechnical engineering analyses.

a. Site Clearing. Areas to be graded should be cleared of the existing structures, surface improvements, debris and underground utilities. All areas scheduled to receive fill should be cleared of surface improvements, artificial fill and any unsuitable matter. The unsuitable materials should be removed off-site. Existing fill soil should be removed in its entirety and replaced as engineering fill. Voids left by obstructions should be properly backfilled in accordance with the compaction recommendations of this report.

b. Preparation of Building Areas. In order to achieve firm and uniform foundation bearing conditions, we recommend over-excavation and re-compaction throughout the building areas. All artificial fill soil and low density near surface native soil should be removed to competent native soil expected at depths of approximately 3 feet below the existing ground surface or to a minimum depth of 3 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally a minimum of five feet beyond the building foundations. The soil exposed by over-excavation should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.

c. Compaction. Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain irreducible matter greater than three inches in maximum dimension. All fill materials should be placed in thin lifts, not exceeding six inches in a loose condition at near optimum moisture content. If import fill is required, the material should be of a low to non-expansive nature and should meet the following criteria:

Plastic Index	Less than 12
Liquid Limit	Less than 35
Percent Soil Passing #200 Sieve	Between 15% and 35%
Maximum Aggregate Size	3 inches

The subgrade soil and all fill material should be compacted with acceptable compaction equipment to at least 90 percent relative compaction. The bottom of the exposed subgrade should be observed by a representative of Sladden Engineering prior to fill placement. Compaction testing should be performed in order to verify proper compaction. Table 3 provides a summary of the excavation and compaction recommendations.

TABLE 3
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Over-excavation and re-compaction within the building envelope and extending laterally at least 5 feet beyond the building limits and to a minimum depth of 3 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper.
Native / Import Engineered Fill	Place in thin lifts not exceeding 6 inches in a loose condition, compact to a minimum of 90 percent relative compaction.
Asphalt Concrete Sections	Compact the top 12 inches to at least 95 percent compaction within 2 percent of optimum moisture content.

*Actual depth may vary and should be determined by a representative of Sladden Engineering in the field during construction.

d. Shrinkage and Subsidence. Volumetric shrinkage of the material that is excavated and replaced as controlled compacted fill should be anticipated. We estimate that this shrinkage could vary from 10 to 15 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 and 2 tenths of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

FOUNDATIONS: CONVENTIONAL SHALLOW SPREAD FOOTINGS

Exterior footings should extend at least 18 inches beneath lowest adjacent grade and interior footings should extend at least 12 inches below slab subgrade. Isolated square or rectangular footings at least 2 feet square and continuous footings at least 12 inches wide may be designed using allowable bearing pressures of 2200 and 2000 pounds per square foot, respectively. The allowable bearing pressure may be increased by approximately 250 psf for each additional 1 foot of width and 250 psf for each additional 6 inches of depth, if desired. The maximum allowable bearing pressure should be limited to 3000 psf unless confirmed by Sladden Engineering subsequent to performing specific settlement calculations. The allowable bearing pressures are for dead and frequently applied live loads and may be increased by 1/3 to resist wind, seismic or other transient loading. All footings should be reinforced in accordance with the project structural engineer's recommendations.

Based on the allowable bearing pressures recommended above the total static settlement of conventional shallow spread footings is anticipated to be less than one inch, provided that foundation preparation conforms to the recommendations provided in this report. Differential static settlement is anticipated to be approximately one-half the total static settlement for similarly loaded footings spaced approximately 40 feet apart.

Resistance to lateral loads may be provided by a combination of friction acting at the base of the slabs or foundations and passive earth pressure along the sides of the foundations. A coefficient of friction of 0.40 between soil and concrete may be used for dead load forces only. A passive earth pressure of 250 pounds per square foot, per foot of depth, may be used for the sides of footings that are placed against properly compacted native soil. Passive earth pressure should be ignored within the upper 1 foot except where confined.

All footing excavations should be observed by a representative of the project geotechnical consultant to verify adequate embedment depths prior to placement of forms, steel reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, disturbed, sloughed or moisture-softened soils and/or any construction debris should be removed prior to concrete placement. Excavated soil generated from footing and/or utility trenches should not be stockpiled within the building envelope or in areas of exterior concrete flatwork.

SLABS-ON-GRADE

In order to reduce the risk of heave, cracking and settlement, concrete slabs-on-grade must be placed on properly compacted fill as outlined in the previous sections. The slab subgrades should remain near optimum moisture content and should not be permitted to dry prior to concrete placement. All slab subgrades should be firm and unyielding. Disturbed soil should be removed and then replaced and compacted to a minimum of 90 percent relative compaction.

Slab thickness and reinforcement should be determined by the structural engineer. All slab reinforcement should be supported on concrete chairs to ensure that reinforcement is placed at slab mid-height. Considering the expected uses, we recommend a minimum slab thickness of 6.0 inches within warehouse areas and 4.0 inches within office areas along with minimum reinforcement of #4 bars at 24 inches on center in both directions in warehouse areas and #3 bars at 24 inches on center in both directions for office areas.

Slabs with moisture sensitive surfaces should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 10-mil Visqueen. All laps within the membrane should be sealed and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete and to limit damage. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface can not be achieved by grading, consideration should be given to placing a thin leveling course of sand across the pad surface prior to placement of the membrane.

RETAINING WALLS

Minor retaining walls may be required to accomplish the proposed construction. Cantilever retaining walls may be designed using "active" pressures. Active pressures may be estimated using an equivalent fluid weight of 40 pcf for level native backfill soil acting in a triangular pressure distribution with drained backfill conditions. "At Rest" pressures should be utilized for restrained walls. At rest pressures may be estimated using an equivalent fluid weight of 60 pcf for native backfill soil with level drained backfill conditions.

We recommend that a back drain system be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The back drains should consist of a heavy walled, four inch diameter, perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining, three-quarter to one and one-half inch crushed rock or gravel. The crushed rock or gravel should extend to within one foot of the surface. The upper one foot should be backfilled with compacted, fine-grained soil to exclude surface water. A Mirafi 140N (or equivalent) filter cloth should be placed between the on-site native material and the drain rock.

ON-SITE PAVEMENT DESIGN

Asphalt concrete pavements should be designed in accordance with the Caltrans Highway Design Manual based on R-Value and Traffic Index. The R-Value of the near surface soil determined to be 36 at equilibrium. For preliminary pavement design, Traffic Indices (TI) of 6.0 and 7.5 were used for the light duty and heavy duty pavements, respectively. We assumed Asphalt Concrete (AC) over Class II Aggregate Base (AB). The preliminary flexible pavement layer thickness is as follows:

TABLE 4
RECOMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS

Pavement Material	**Recommended Thickness**	
	TI = 6.0	**TI = 7.5**
Asphalt Concrete Surface Course	3.0 inches	4.0 inches
Class II Aggregate Base Course	6.0 inches	8.0 inches
Compacted Subgrade Soil	12.0 inches	12.0 inches

Asphalt concrete and Class II aggregate base should conform to the latest edition of the Standard Specifications for Public Works Construction ("Greenbook") or CalTrans Standard Specifications. The aggregate base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Method D 1557.

We expect that concrete pavement may also be considered for on-site pavement areas. A concrete pavement section of 6.0 inches of Portland Cement Concrete (PCC) on compacted native soil should be adequate for the on-site concrete pavement limited to automobile and light truck traffic. In areas where heavy truck traffic is expected, the concrete pavement section should be increased to 7.0 inches of PCC on compact native soil. Properly spaced and constructed control joints including expansion joints and contraction joints should be incorporated into concrete pavement design to accommodate temperature and shrinkage related cracking. Joint spacing and joint patterns should be established based upon Portland Cement Association (PCA) and American Concrete Institute (ACI) guidelines.

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 140 parts per million (ppm). The soil is considered to have a "negligible" corrosion potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes will be required. The soluble sulfate content of the surface soil should be reevaluated after grading and appropriate concrete mix designs should be established based upon post-grading test results.

The pH level of the surface soil was determined to be 7.8. Based on soluble chloride concentration testing (170 ppm), the soil is considered to have a "low" corrosion potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 640 ohm-cm, which suggests that the site soil is considered to have a "sever" corrosion potential with respect to ferrous metal installations. A corrosion expert should be consulted regarding appropriate corrosion protection measures for corrosion sensitive installations.

UTILITY TRENCH BACKFILL

All utility trench backfill should be compacted to a minimum of 90 percent relative compaction. Trench backfill materials should be placed in lifts no greater than six inches in a loose condition, moisture conditioned (or air-dried) as necessary to achieve near optimum moisture content and then mechanically compacted in place to a minimum of 90 percent relative compaction. A representative of the project geotechnical consultant should test the backfill to verify adequate compaction.

EXTERIOR CONCRETE FLATWORK

To minimize cracking of concrete flatwork, the subgrade soil below concrete flatwork areas should first be compacted to a minimum of 90 percent relative compaction. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soil.

DRAINAGE

All final grades should be provided with positive gradients away from foundations to provide rapid removal of surface water runoff to an adequate discharge point. No water should be allowed to be pond on or immediately adjacent to foundation elements. In order to reduce water infiltration into the subgrade soil, surface water should be directed away from building foundations to an adequate discharge point. Subgrade drainage should be evaluated upon completion of the precise grading plans and in the field during grading.

LIMITATIONS

The findings and recommendations presented in this report are based upon an interpolation of the soil conditions between the exploratory boring locations and extrapolation of these conditions throughout the proposed building area. Should conditions encountered during grading appear different than those indicated in this report, this office should be notified.

The use of this report by other parties or for other projects is not authorized. The recommendations of this report are contingent upon monitoring of the grading operation by a representative of Sladden Engineering. All recommendations are considered to be tentative pending our review of the grading operation and additional testing, if indicated. If others are employed to perform any soil testing, this office should be notified prior to such testing in order to coordinate any required site visits by our representative and to assure indemnification of Sladden Engineering.

We recommend that a pre-job conference be held on the site prior to the initiation of site grading. The purpose of this meeting will be to assure a complete understanding of the recommendations presented in this report as they apply to the actual grading performed.

ADDITIONAL SERVICES

Once completed, final project plans and specifications should be reviewed by use prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the Soil Engineer during construction to document that foundation elements are founded on/or penetrate into the recommended soil, and that suitable backfill soil is placed upon competent materials and properly compacted at the recommended moisture content.

Tests and observations should be performed during grading by the Soil Engineer or his representative in order to verify that the grading is being performed in accordance with the project specifications. Field density testing shall be performed in accordance with acceptable ASTM test methods. The minimum acceptable degree of compaction should be 90 percent for subgrade soils and 95 percent for Class II aggregate base as obtained by the ASTM Test Method D1557. Where testing indicates insufficient density, additional compactive effort shall be applied until retesting indicates satisfactory compaction.

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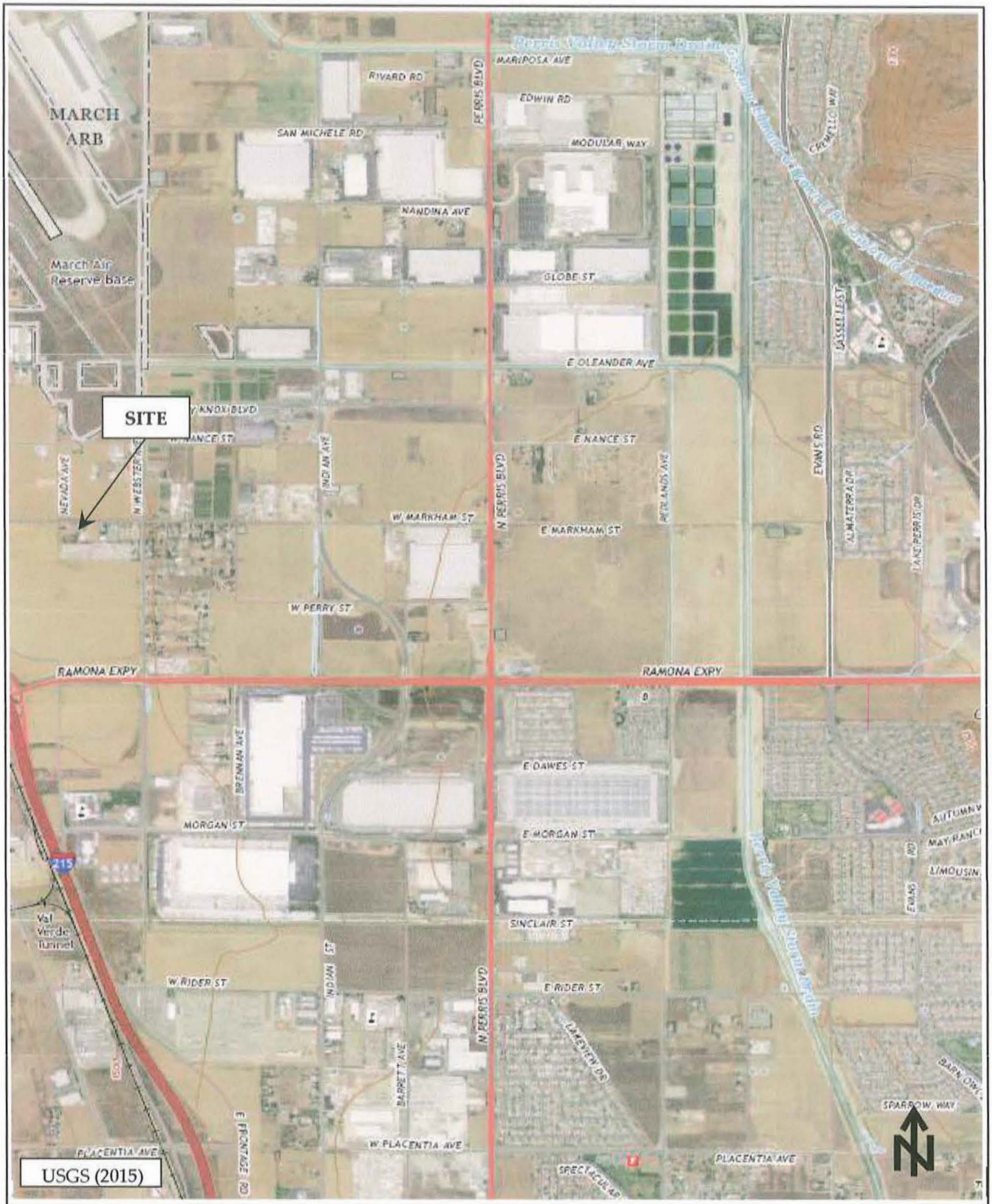
FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
EXPLORATION LOCATION PLAN



USGS (2015)

SITE LOCATION MAP

Project Number:	644-22018
Report Number:	22-05-068
Date:	May 31, 2022

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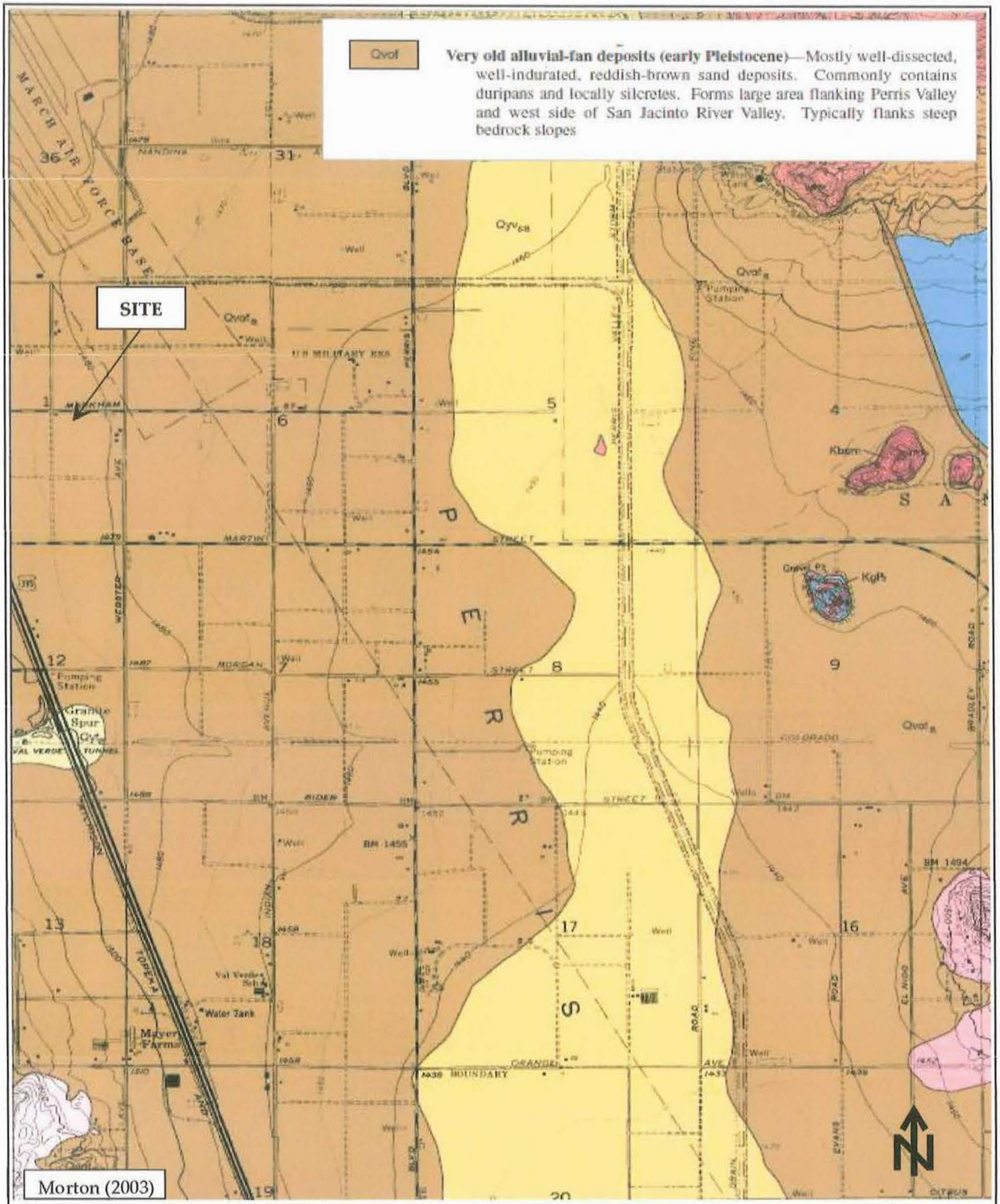
FIGURE

1



Qvof **Very old alluvial-fan deposits (early Pleistocene)**—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Forms large area flanking Perris Valley and west side of San Jacinto River Valley. Typically flanks steep bedrock slopes

REGIONAL GEOLOGIC MAP

FIGURE 2

Project Number:	644-22018
Report Number:	22-05-068
Date:	May 31, 2022

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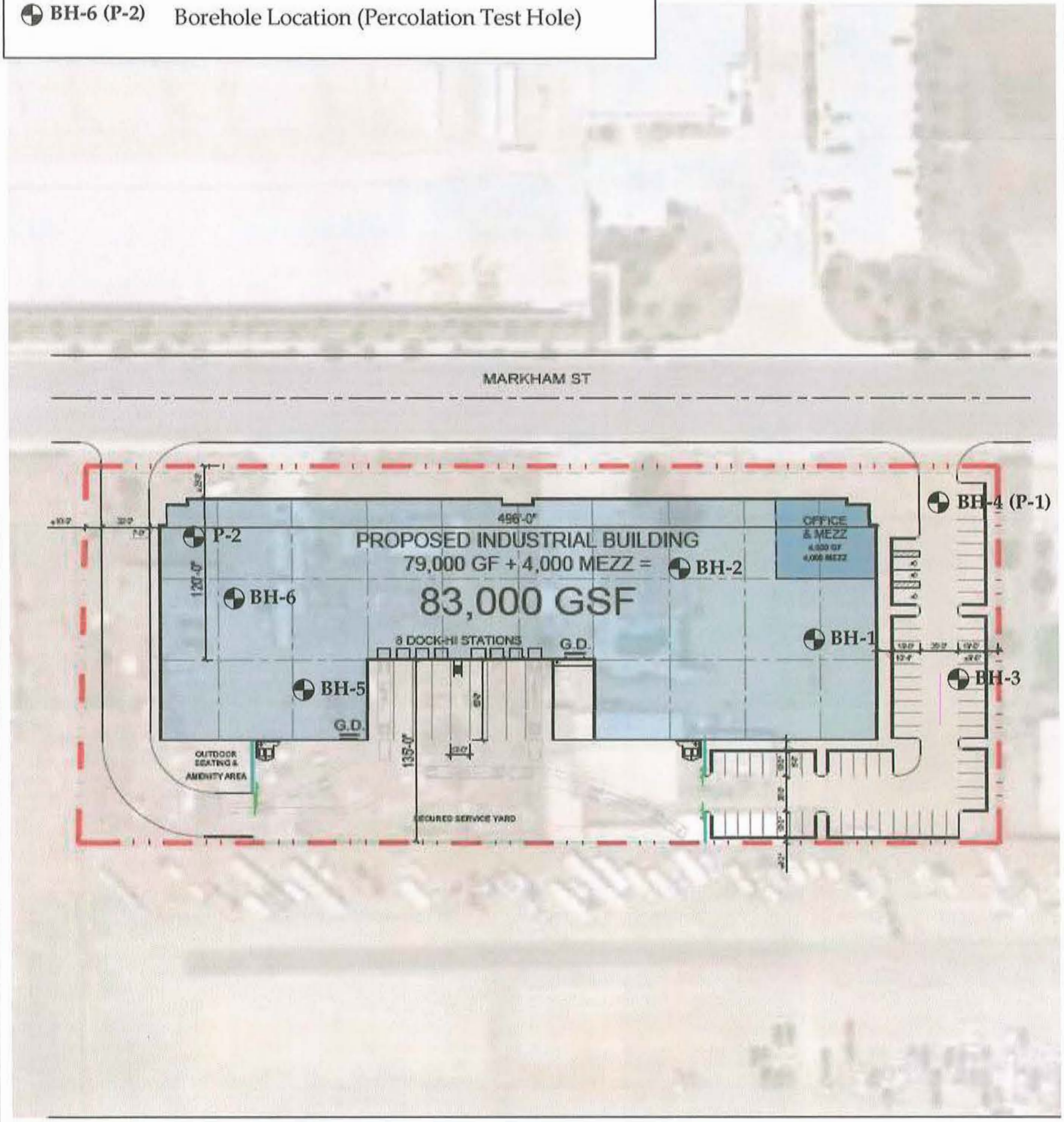
LEGEND

● BH-6 (P-2) Borehole Location (Percolation Test Hole)



N

EXPLORATION LOCATION PLAN

Project Number:	644-22018
Report Number:	22-05-068
Date:	May 31, 2022

FIGURE

3

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation seven (7) exploratory bores were excavated on April 4, 2022 utilizing a truck mounted hollow stem auger rig (Mobile B-61). Continuous logs of the materials encountered were made by a representative of Sladden Engineering. Materials encountered in the boreholes were classified in accordance with the Unified Soil Classification System which is presented in this appendix.

Representative undisturbed samples were obtained within our bores by driving a thin-walled steel penetration sampler (California split spoon sampler) or a Standard Penetration Test (SPT) sampler with a 140 pound automatic-trip hammer dropping approximately 30 inches (ASTM D1586). The number of blows required to drive the samplers 18 inches was recorded in 6-inch increments and blowcounts are indicated on the boring logs.

The California samplers are 3.0 inches in diameter, carrying brass sample rings having inner diameters of 2.5 inches. The standard penetration samplers are 2.0 inches in diameter with an inner diameter of 1.5 inches. Undisturbed samples were removed from the sampler and placed in moisture sealed containers in order to preserve the natural soil moisture content. Bulk samples were obtained from the excavation spoils and samples were then transported to our laboratory for further observations and testing.

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BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▨	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
■	12 18 25	1	15	41.6	3.4	118.9	4	▨	Clayey Sand (SC); dark brown, dry, medium dense, fine-grained with gravel (Qvof).
■	18 34 39			33.8	4.0	134.5	6 8	▨	Clayey Sand (SC); dark brown, dry, medium dense, fine-grained with gravel (Qvof).
□■□	12 15 15			46.6	6.5		10 12 14	▨	Clayey Sand (SC); reddish brown, dry, medium dense, fine-grained with gravel (Qvof).
■	50-3			34.5	7.7	125.6	16	▨	Clayey Sand (SC); reddish brown, dry, very dense, fine-grained with gravel (Qvof).
							18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50		Terminated at ~15.25 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No:	644-22018	Page	1
Report No:	22-05-068		

Sladden Engineering

BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▒	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
◧	8 10 12			56.7	11.4	129.5	4 6 8	▨	Sandy Clay (CL); dark yellowish brown, slightly moist, very stiff, low plasticity (Qvof).
■	20 27 33			37.2	10.3		10 12	▒	Clayey Sand (SC); dark yellowish brown, slightly moist, dense, fine-grained (Qvof).
◧	18 21 24			35.9	8.6	130.8	14 16	▒	Clayey Sand (SC); dark yellowish brown, slightly moist, very dense, fine-grained (Qvof).
■	15 50-5			43.8	10.6		18 20	▒	Clayey Sand (SC); dark yellowish brown, slightly moist, very dense, fine-grained (Qvof).
							22 24 26 28 30 32 34 36 38 40 42 44 46 48 50		Terminated at ~21.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No: 644-22018
Report No: 22-05-068

Sladden Engineering

BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-3

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
								▨	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
							2	▨	
							4	▨	
☒	50-3							▨	No Recovery.
							6	▨	
							8	▨	
							10	▨	
■	9 14 17			47.9	8.1			▨	Clayey Sand (SC); dark yellowish brown, dry to slightly moist, dense, fine-grained (Qvof)>
							12		
							14		Terminated at ~11.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No: 644-22018
Report No: 22-05-068

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BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-4 (P-1)

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▨	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
							4	▨	
◨	8 10 11			43.1	6.6		6	▨	Clayey Sand (SC); dark yellowish brown to reddish brown, dry, medium dense, fine-grained (Qvof).
							8	▨	
■	18 24 50-6			46.0	8.5	134.3	10	▨	Clayey Sand (SC); dark yellowish brown to reddish brown, dry, medium dense, fine-grained with gravel (Qvof).
							12	▨	
							14		Terminated at ~11.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered. Cased to Facilitate Percolation Testing.
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No: 644-22018
Report No: 22-05-068

Sladden Engineering

BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-5

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▒	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
							4	▒	
■	8 13 17			49.9	7.2	125.7	6	▒	Clayey Sand (SC); yellowish brown, dry, medium dense, fine-grained (Qvof).
							8	▒	
▣	9 10 12			17.3	4.3		10	▒	Clayey Sand (SC); yellowish brown, dry, medium dense, fine-grained (Qvof).
							12	▒	
							14	▒	
■	12 50-5			38.2	10.4	125.6	16	▒	Clayey Sand (SC); yellowish brown, dry, very dense, fine-grained with gravel (Qvof).
							18	▒	
▣	7 10 11			45.3	10.2		20	▒	Clayey Sand (SC); yellowish brown, dry, medium dense, fine-grained (Qvof).
							22	▒	
							24	▒	
■	11 20 25			48.1	9.5	122.6	26	▒	Clayey Sand (SC); reddish brown, dry, medium dense, fine-grained (Qvof).
							28	▒	
▣	6 11 15			42.7	9.6		30	▒	Clayey Sand (SC); reddish brown, dry, medium dense, fine-grained (Qvof).
							32	▒	
							34	▒	
■	14 17 52			29.3	5.5	128.9	36	▒	Clayey Sand (SC); reddish brown, dry, medium dense, fine-grained (Qvof).
							38	▒	
▣	17 19 22			29.1	6.3		40	▒	Clayey Sand (SC); reddish brown, dry, dense, fine-grained (Qvof).
							42	▒	
							44	▒	
■	22 35 50-6			44.6	8.3	133.6	46	▒	Clayey Sand (SC); reddish brown, dry, very dense, fine-grained (Qvof).
							48	▒	
							50		

Completion Notes:
Practical Auger Refusal at ~48.0 Feet bgs.
No Bedrock Encountered.
No Groundwater or Seepage Encountered.

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No: 644-22018
Report No: 22-05-068

Sladden Engineering

BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	BH-6

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▨	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
■	14 18 27			50.6	7.0	133.7	4 6 8	▨	Sandy Clay (CL); reddish brown, dry to slighlty moist, very stiff, low plasticity (Qvof).
□■□	9 18 29			50.4	9.5		10 12	▨	Sandy Clay (CL); reddish brown, dry to slighlty moist, hard, low plasticity (Qvof).
■	11 26 41			51.5	10.1	131.0	14 16	▨	Sandy Clay (CL); reddish brown, slighlty moist, hard, low plasticity (Qvof).
							18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50		Terminated at ~16.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No:	644-22018	Page 6
Report No:	22-05-068	

Sladden Engineering

BORE LOG

Equipment:	MOBILE B-61	Date Drilled:	4/4/2022
Elevation:	1,490 Ft. MSL	Boring No:	P-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Density, pcf	Depth (Feet)	Graphic Lithology	Description
							2	▨	Silty Sand to Clayey Sand (SM/SC); yellowish brown, dry, fine-grained (Disturbed/Fill).
							4	▨	Clayey Sand (SC); reddish brown, dry, fine-grained (Qvof).
							6		Termianted at ~5.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered. Cased to Facilitate Percolation Testing
							8		
							10		
							12		
							14		
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PROPOSED INDUSTRIAL BUILDING
945-995 WEST MARKHAM STREET, PERRIS

Project No: 644-22018
Report No: 22-05-068

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Representative bulk and relatively undisturbed soil samples were obtained in the field and returned to our laboratory for additional observations and testing. Laboratory testing was generally performed in two phases. The first phase consisted of testing in order to determine the compaction of the existing natural soil and the general engineering classifications of the soils underlying the site. This testing was performed in order to estimate the engineering characteristics of the soil and to serve as a basis for selecting samples for the second phase of testing. The second phase consisted of soil mechanics testing. This testing including consolidation, shear strength and expansion testing was performed in order to provide a means of developing specific design recommendations based on the mechanical properties of the soil.

CLASSIFICATION AND COMPACTION TESTING

Unit Weight and Moisture Content Determinations: Each undisturbed sample was weighed and measured in order to determine its unit weight. A small portion of each sample was then subjected to testing in order to determine its moisture content. This was used in order to determine the dry density of the soil in its natural condition. The results of this testing are shown on the Bore Logs.

Maximum Density-Optimum Moisture Determinations: Representative soil types were selected for maximum density determinations. This testing was performed in accordance with the ASTM Standard D1557, Test Method A. The results of testing are presented graphically in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

Classification Testing: Soil samples were selected for classification testing. This testing consists of mechanical grain size analyses. This provides information for developing classifications for the soil in accordance with the Unified Soil Classification System which is presented in the preceding appendix. This classification system categorizes the soil into groups having similar engineering characteristics. The results of this testing is very useful in detecting variations in the soils and in selecting samples for further testing.

SOIL MECHANIC'S TESTING

Expansion Testing: One (1) bulk sample was selected for Expansion testing. Expansion testing was performed in accordance with the UBC Standard 18-2. This testing consists of remolding 4-inch diameter by 1-inch thick test specimens to a moisture content and dry density corresponding to approximately 50 percent saturation. The samples are subjected to a surcharge of 144 pounds per square foot and allowed to reach equilibrium. At that point the specimens are inundated with distilled water. The linear expansion is then measured until complete.

Direct Shear Testing: One (1) sample was selected for Direct Shear testing. This test measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation design and lateral design. Tests were performed using a recompacted test specimen that was saturated prior to tests. Tests were performed using a strain controlled test apparatus with normal pressures ranging from 800 to 2300 pounds per square foot.

Consolidation Testing: Two (2) relatively undisturbed samples were selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load. The specimens were saturated at 575 psf or 720 psf load increment.

Corrosion Series Testing: The soluble sulfate concentrations of the surface soil were determined in accordance with California Test Method Number (CA) 417. The pH and Minimum Resistivity were determined in accordance with CA 643. The soluble chloride concentrations were determined in accordance with CA 422.

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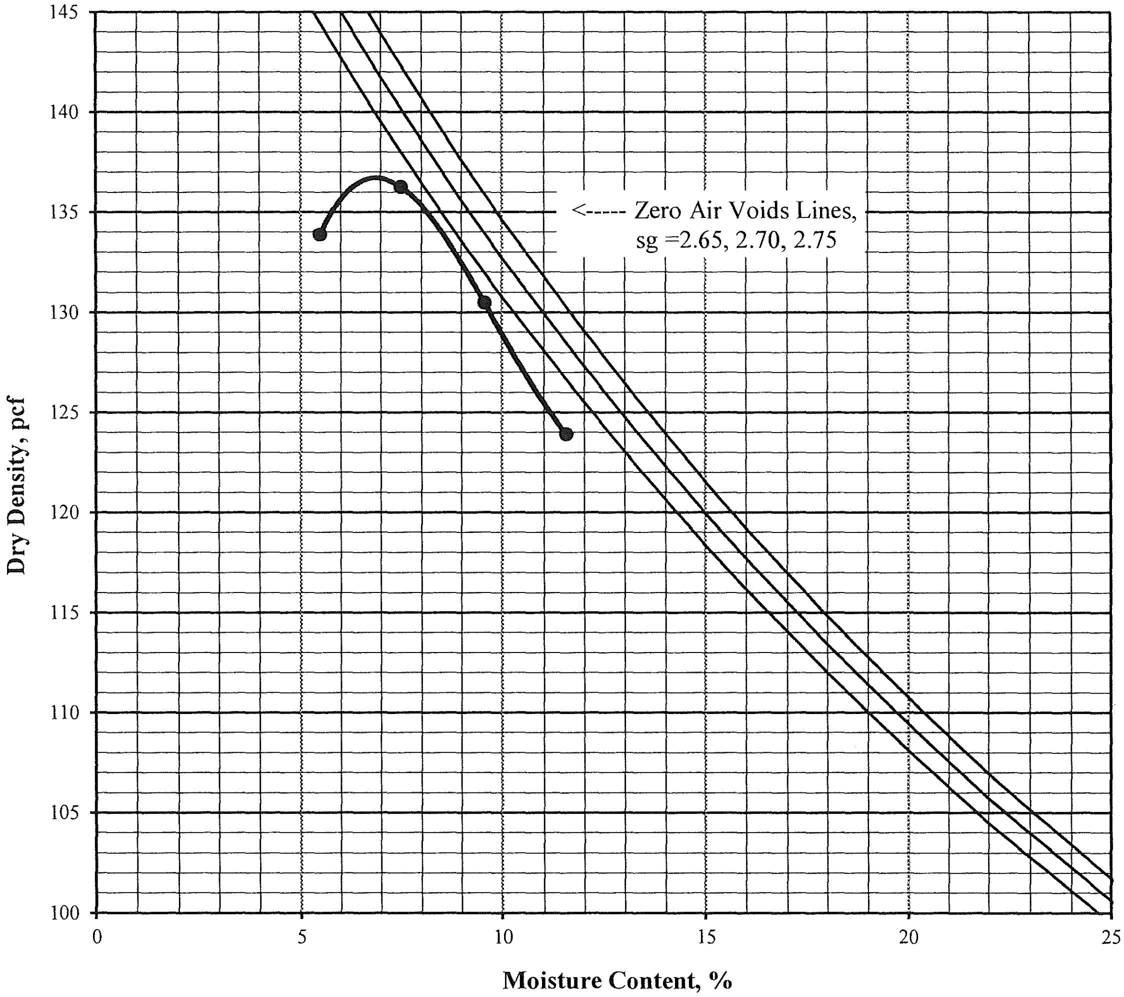
Maximum Density/Optimum Moisture

ASTM D698/D1557

Project Number:	644-22018	May 23, 2022
Project Name:	945-995 West Markham Street	
Lab ID Number:	LN6-22169	ASTM D-1557 A
Sample Location:	BH-1 Bulk 1 @ 0-5'	Rammer Type: Machine
Description:	Dark Brown Clayey Sand (SC)	

Maximum Density: **136.5 pcf**
Optimum Moisture: **7.5%**

Sieve Size	% Retained
3/4"	
3/8"	
#4	0.4



Expansion Index

ASTM D 4829

Job Number:	644-22018	May 23, 2022
Job Name:	945-995 West Markham Street	
Lab ID Number:	LN6-22169	
Sample ID:	BH-1 Bulk 1 @ 0-5'	
Soil Description:	Dark Brown Clayey Sand (SC)	

Wt of Soil + Ring:	603.8
Weight of Ring:	188.6
Wt of Wet Soil:	415.2
Percent Moisture:	6.3%
Sample Height, in	0.95
Wet Density, pcf:	132.9
Dry Denstiy, pcf:	125.0

% Saturation:	48.9

Expansion **Rack # 2**

Date/Time	5/19/2022	3:30 PM
Initial Reading	0.0000	
Final Reading	0.0145	

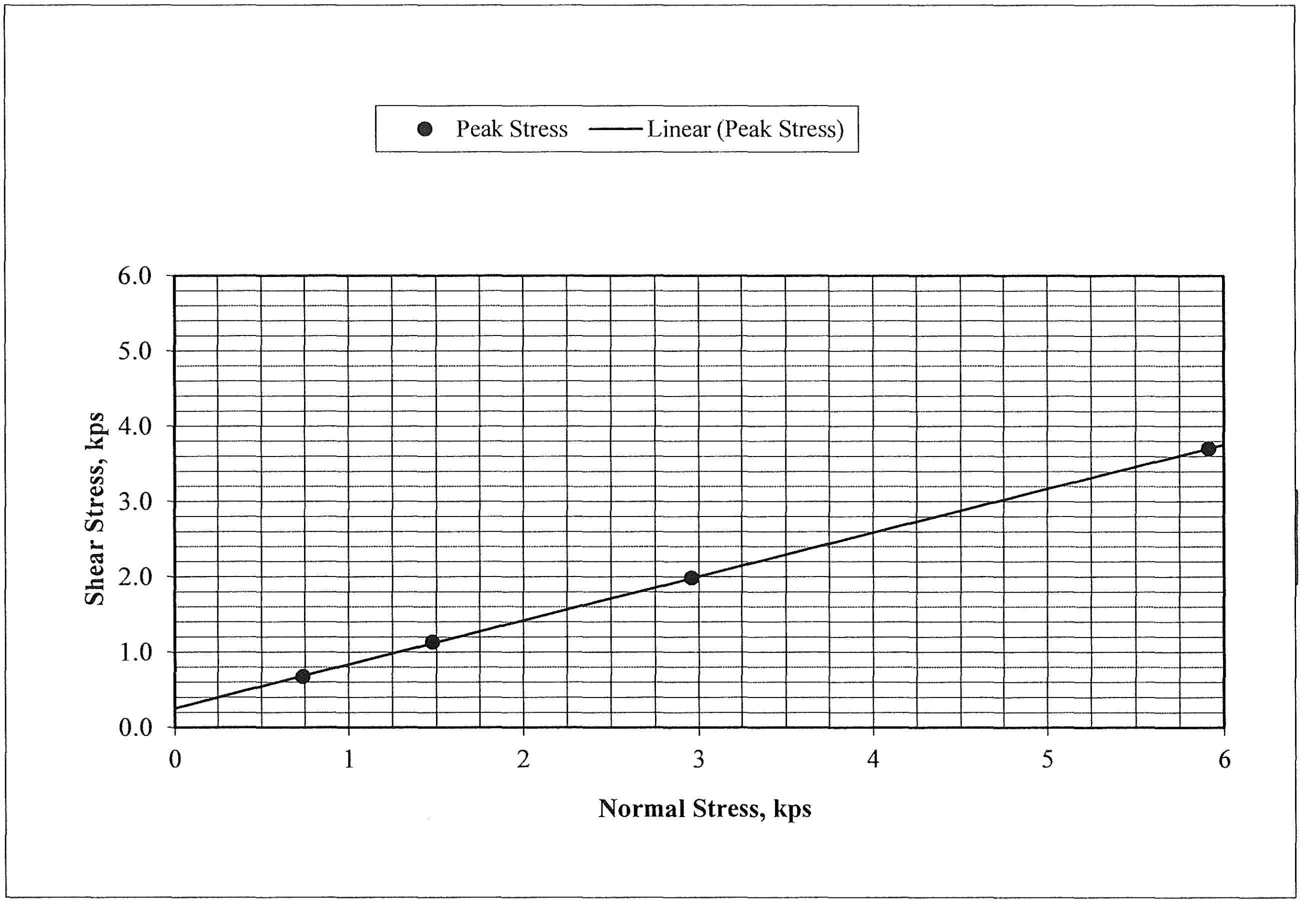
Expansion Index **15**

(Final - Initial) x 1000

Direct Shear ASTM D 3080-04
(modified for unconsolidated condition)

Job Number:	644-22018	May 23, 2022
Job Name	945-995 West Markham Street	Initial Dry Density: 122.8 pcf
Lab ID No.	LN6-22169	Initial Mosture Content: 7.5 %
Sample ID	BH-1 Bulk 1 @ 0-5'	Peak Friction Angle (Ø): 30°
Classification	Dark Brown Clayey Sand (SC)	Cohesion (c): 260 psf
Sample Type	Remolded @ 90% of Maximum Density	

Test Results	1	2	3	4	Average
Moisture Content, %	12.8	12.8	12.8	12.8	12.8
Saturation, %	92.7	92.7	92.7	92.7	92.7
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.676	1.134	1.984	3.706	



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Gradation

ASTM C117 & C136

Project Number: 644-22018

May 23, 2022

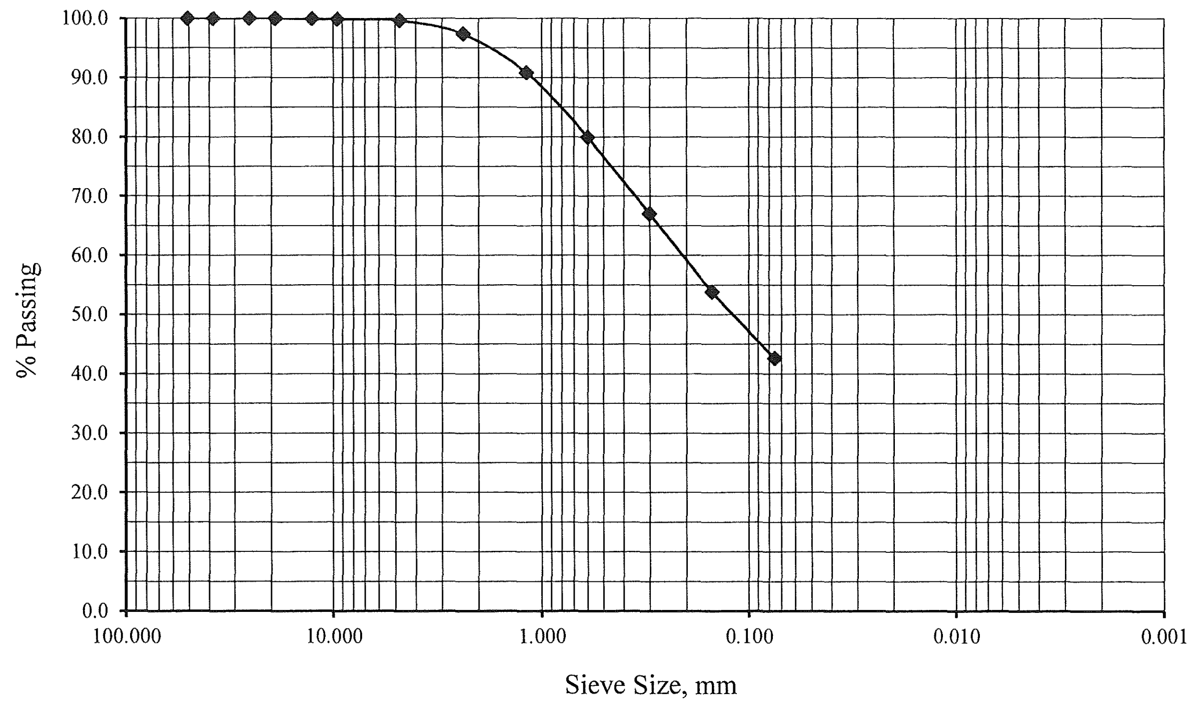
Project Name: 945-995 West Markham Street

Lab ID Number: LN6-22169

Sample ID: BH-1 Bulk 1 @ 0-5'

Soil Classification: SC

Sieve Size, in	Sieve Size, mm	Percent Passing
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	99.9
3/8"	9.53	99.9
#4	4.75	99.7
#8	2.36	97.4
#16	1.18	90.9
#30	0.60	80.0
#50	0.30	67.0
#100	0.15	53.9
#200	0.075	42.6

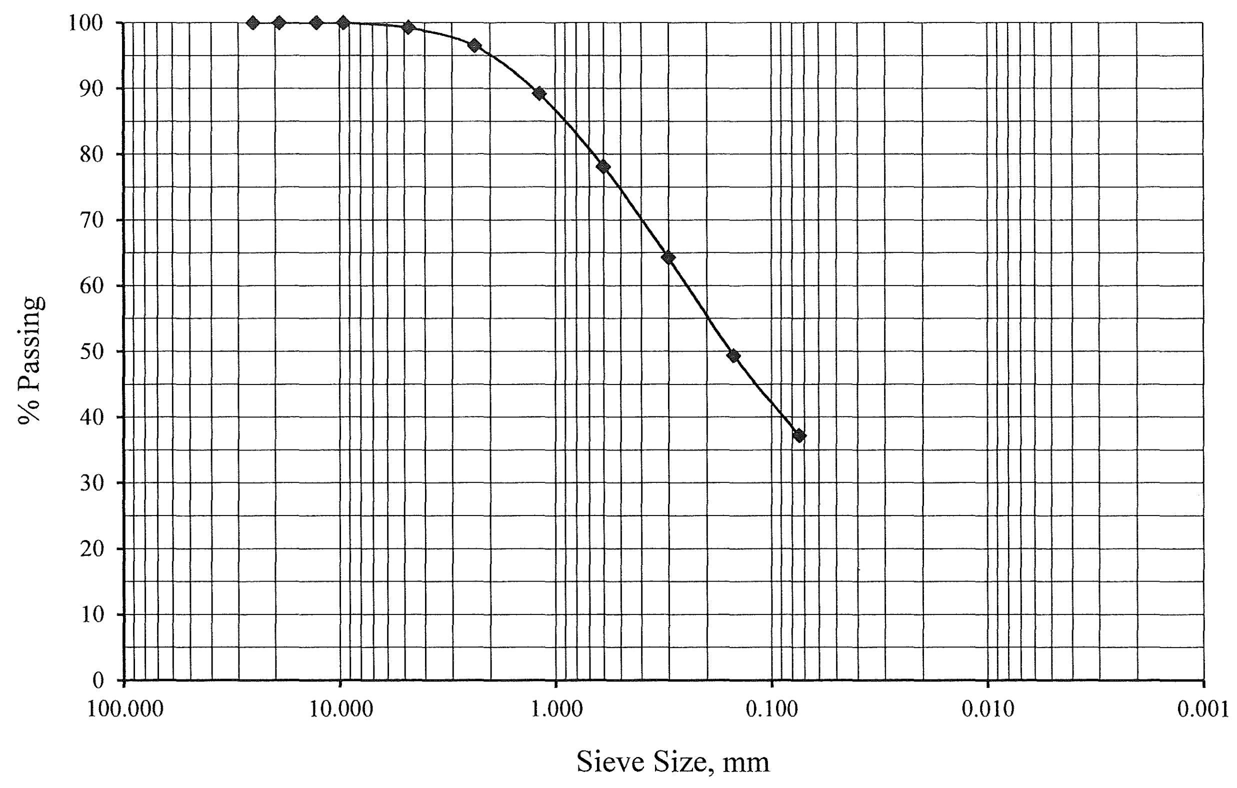


Gradation

ASTM C117 & C136

Project Number:	644-22018	May 23, 2022
Project Name:	945-995 West Markham Street	
Lab ID Number:	LN6-22169	
Sample ID:	BH-2 R-2 @ 10'	Soil Classification: SC

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	99.3
#8	2.36	96.5
#16	1.18	89.2
#30	0.60	78.1
#50	0.30	64.3
#100	0.15	49.4
#200	0.074	37.2



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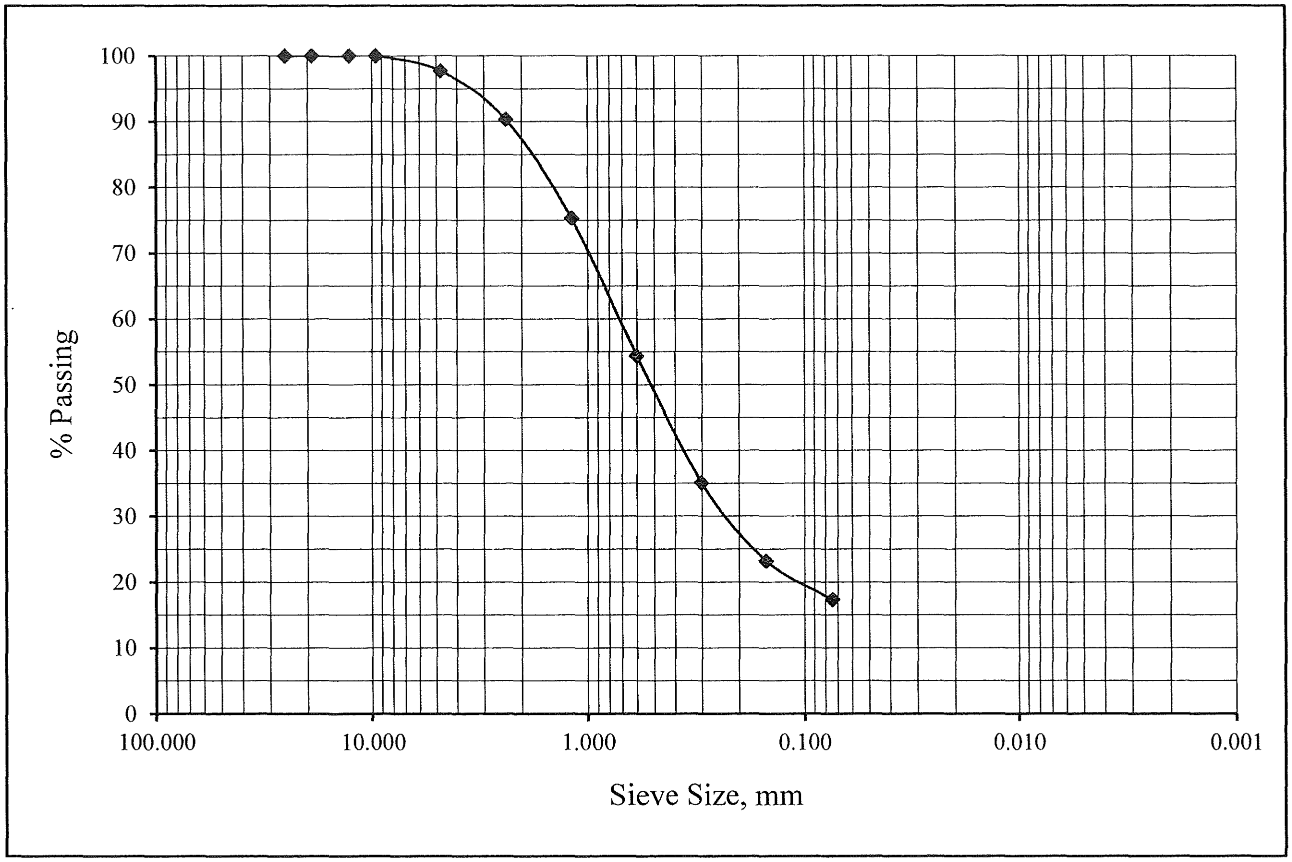
Gradation

ASTM C117 & C136

May 23, 2022

Project Number: 644-22018
Project Name: 945-995 West Markham Street
Lab ID Number: LN6-22169
Sample ID: BH-5 S-2 @ 10'
Soil Classification: SC

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	97.7
#8	2.36	90.3
#16	1.18	75.3
#30	0.60	54.4
#50	0.30	35.1
#100	0.15	23.2
#200	0.074	17.3



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Gradation

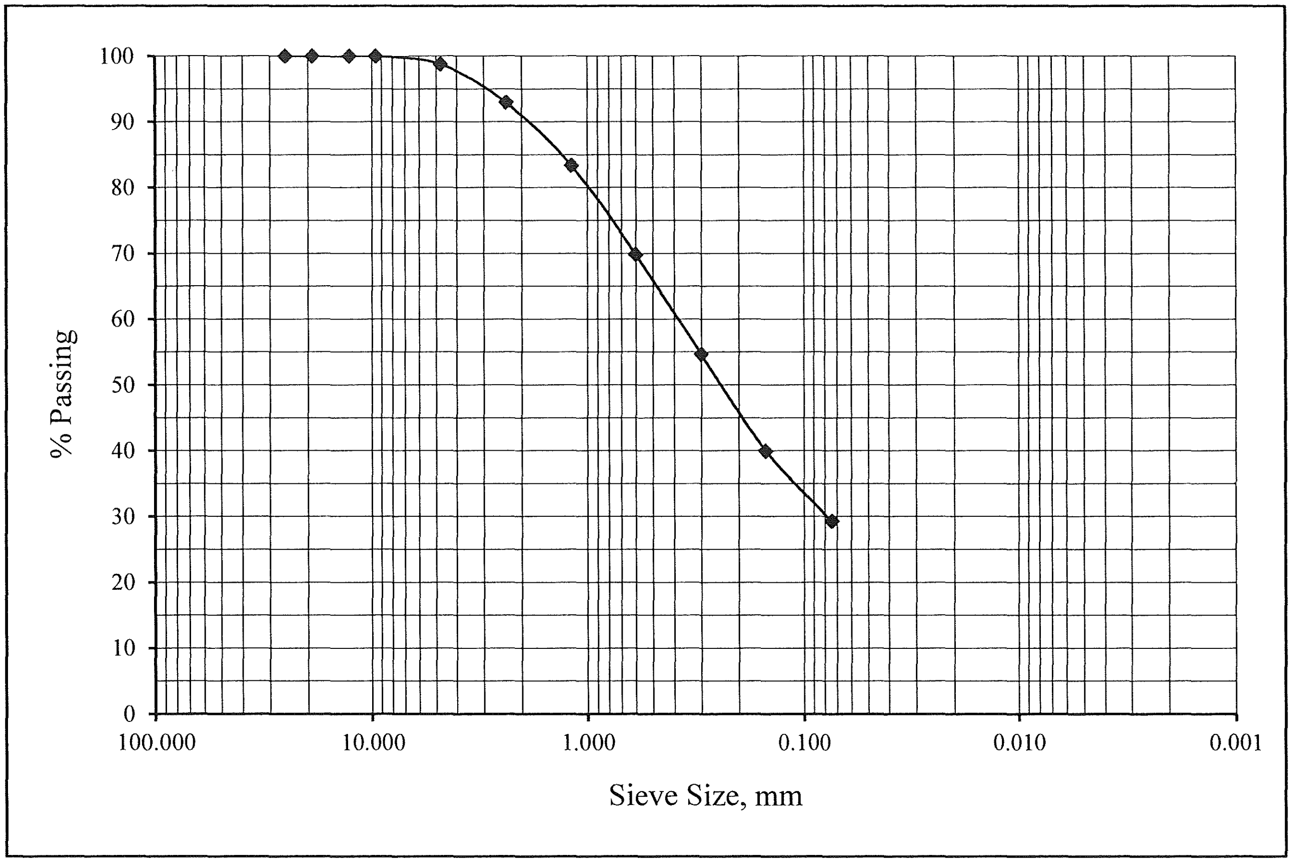
ASTM C117 & C136

May 23, 2022

Project Number:	644-22018
Project Name:	945-995 West Markham Street
Lab ID Number:	LN6-22169
Sample ID:	BH-5 R-7 @ 35'

Soil Classification: SC

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	100.0
#4	4.75	98.8
#8	2.36	93.0
#16	1.18	83.4
#30	0.60	69.8
#50	0.30	54.7
#100	0.15	40.0
#200	0.074	29.3



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One Dimensional Consolidation

ASTM D2435 & D5333

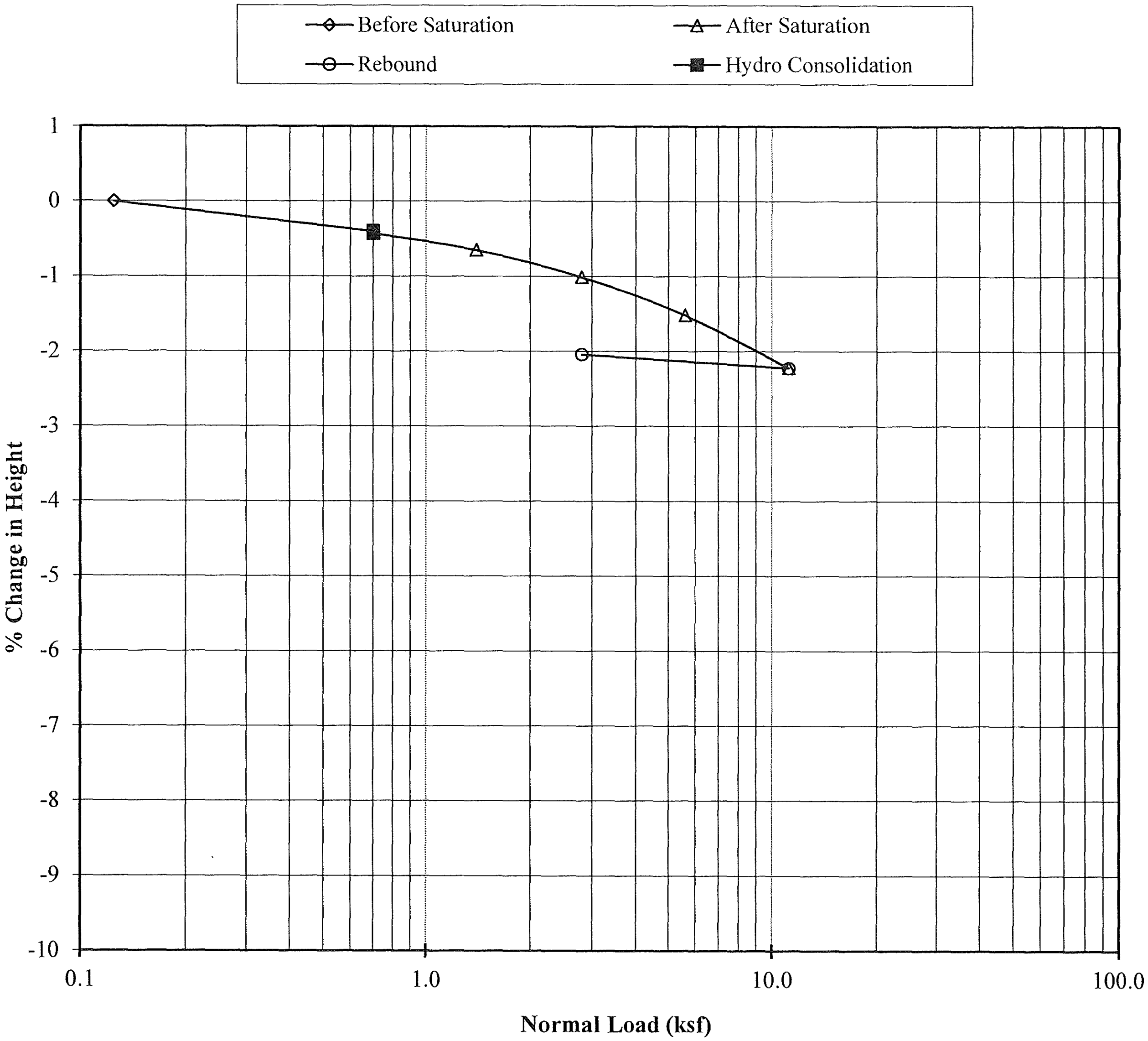
May 23, 2022

Job Number: 644-22018
Job Name: 945-995 West Markham Street

Lab ID Number: LN6-22169
Sample ID: BH-2 R-2 @ 10'
Soil Description: Brown Clayey Sand (SC)

Initial Dry Density, pcf: 128.0
Initial Moisture, %: 10.3
Initial Void Ratio: 0.303
Specific Gravity: 2.67

% Change in Height vs Normal Presssure Diagram



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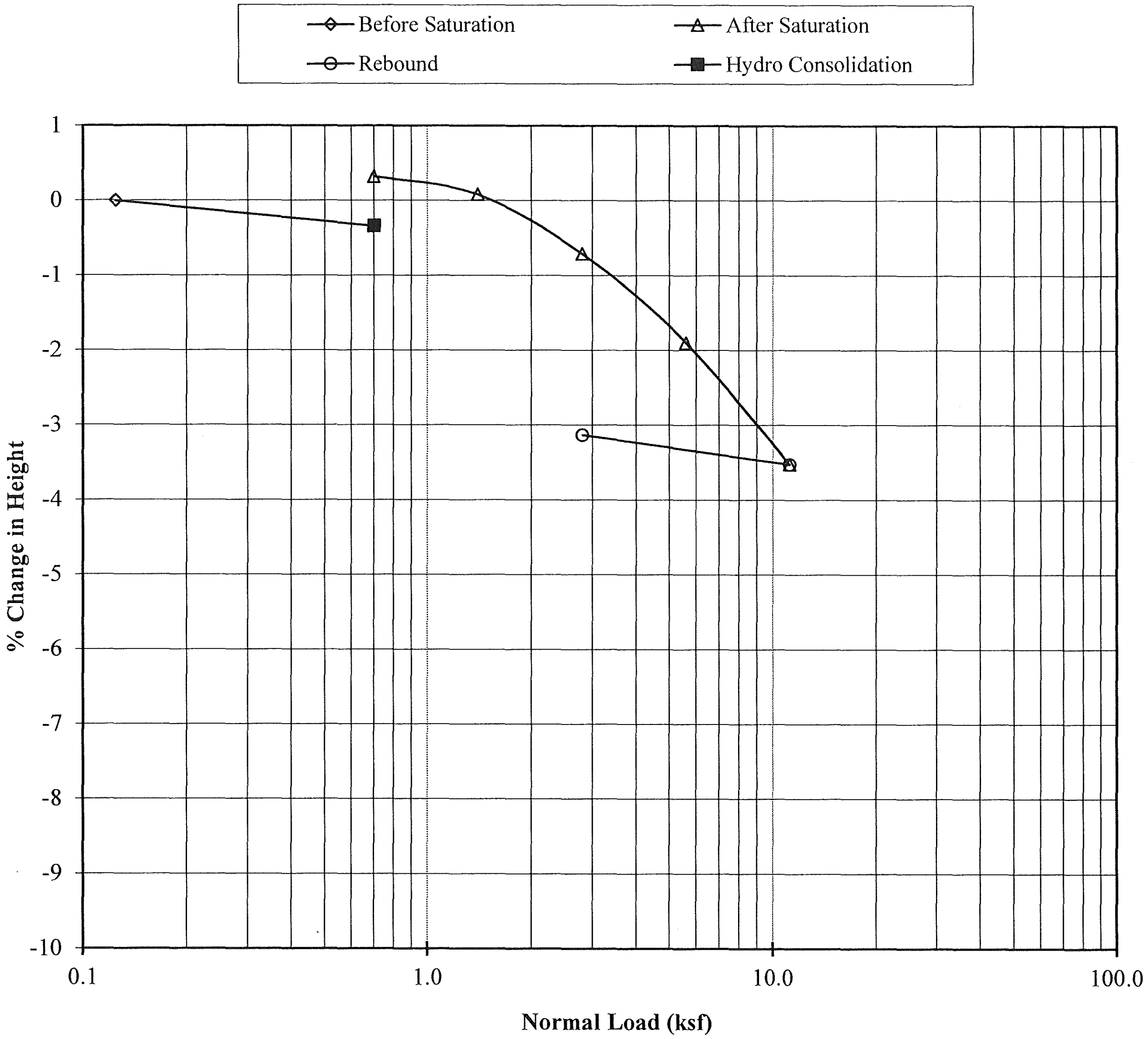
One Dimensional Consolidation

ASTM D2435 & D5333

Job Number:	644-22018	May 23, 2022
Job Name:	945-995 West Markham Street	
Lab ID Number:	LN6-22169	
Sample ID:	BH-5 R-1 @ 5'	
Soil Description:	Brown Clayey Sand (SC)	

Initial Dry Density, pcf:	126.5
Initial Moisture, %:	7.2
Initial Void Ratio:	0.318
Specific Gravity:	2.67

% Change in Height vs Normal Presssure Diagram



RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

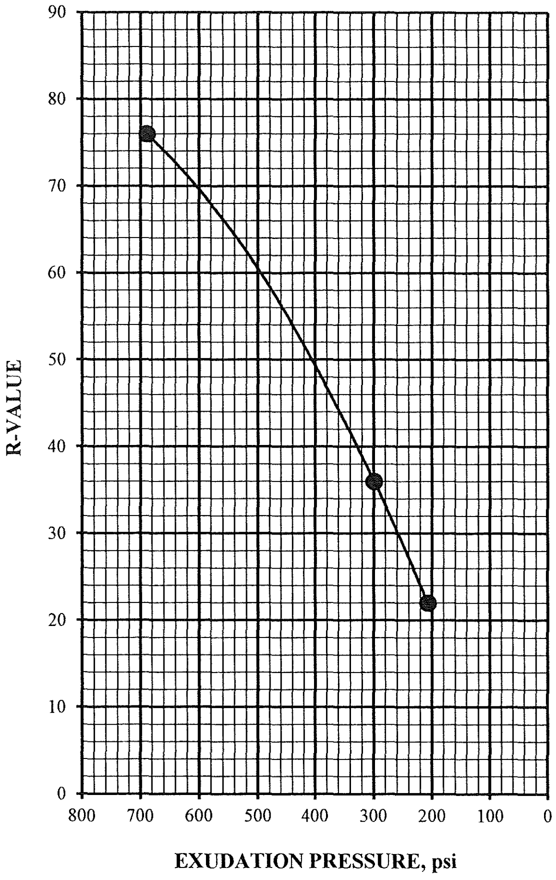
CTM 301

May 23, 2022

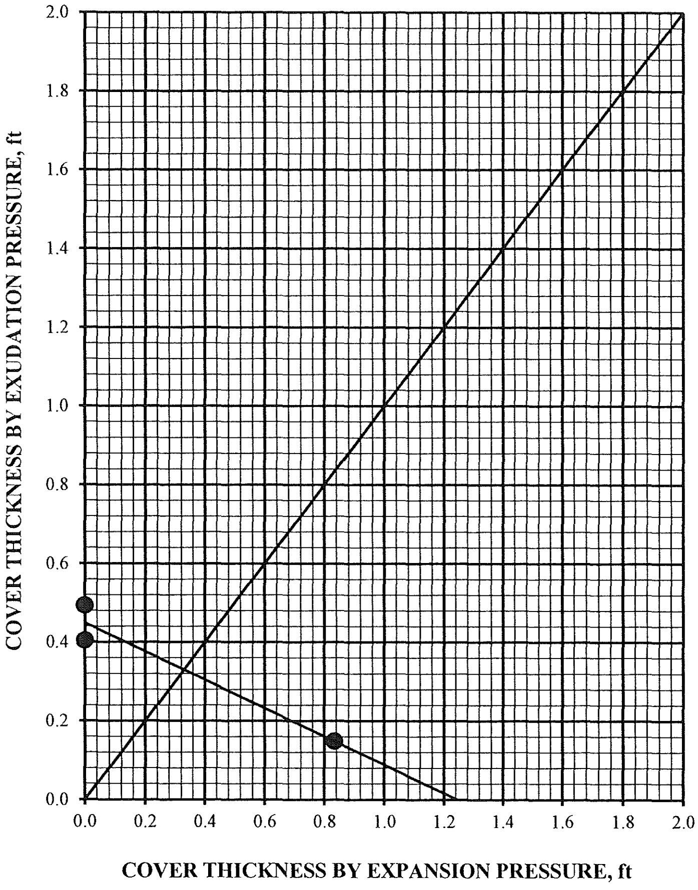
Project Number: 644-22018
Project Name: 945-995 West Markham Street
Lab ID Number: LN6-22169
Sample ID: BH-1 Bulk 1 @ 0-5'
Sample Description: Dark Brown Clayey Sand (SC)
Specified Traffic Index: 5.0

Dry Density @ 300 psi Exudation Pressure: 129.6-pcf
%Moisture @ 300 psi Exudation Pressure: 9.4%
R-Value - Exudation Pressure: 36
R-Value - Expansion Pressure: 48
R-Value @ Equilibrium: 36

EXUDATION PRESSURE CHART



EXPANSION PRESSURE CHART



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6782 Stanton Ave., Suite A, Buena Park, CA 90621 (714) 523-0952 Fax (714) 523-1369
45090 Golf Center Pkwy, Suite F, Indio CA 92201 (760) 863-0713 Fax (760) 863-0847
450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

Date: May 23, 2022

Account No.: 644-22018

Customer: Dedeaux Properties

Location: APN's 314-17-0009 & 0010, 945-995 West Markham Street, Perris

Analytical Report

Corrosion Series

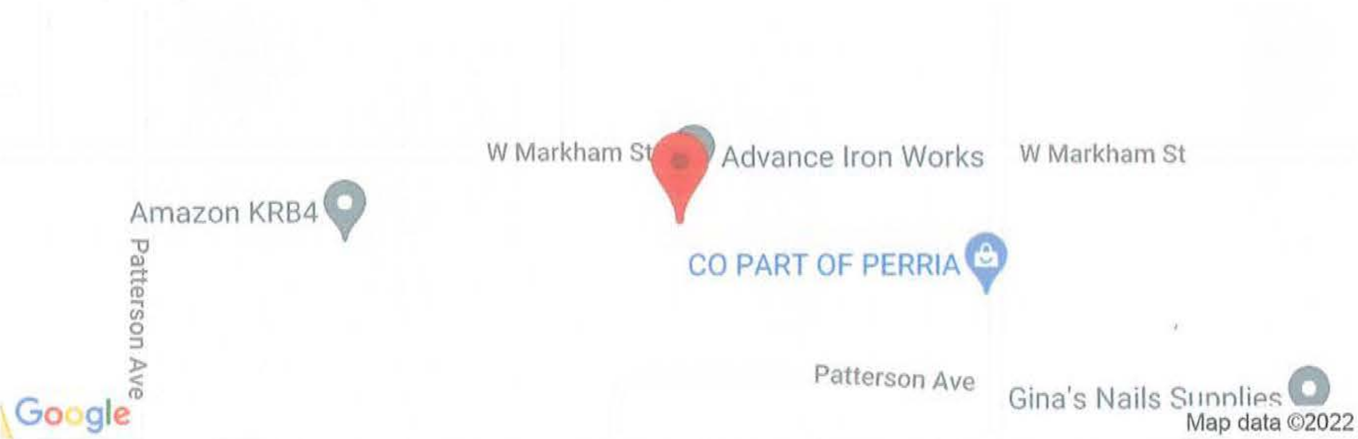
	pH per CA 643	Soluble Sulfates per CA 417 ppm	Soluble Chloride per CA 422 ppm	Min. Resistivity per CA 643 ohm-cm
BH-1 @ 0-5'	7.8	140	170	640

APPENDIX C

SEISMIC DESIGN MAP AND REPORT
SITE SPECIFIC GROUND MOTION PARAMETERS

945-995 West Markham Street, Perris

Latitude, Longitude: 33.8513, -117.2471



Date	5/19/2022, 10:34:11 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE$_R$ ground motion. (for 0.2 second period)
S_1	0.572	MCE$_R$ ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.5	MCE$_G$ peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.535	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.642	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.572	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.627	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.935	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s

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Project: 945-995 West Markham Street, Perris
Project Number: 644-22018
Client: Dedeaux Properties
Site Lat/Long: 33.8513/-117.2471
Controlling Seismic Source: San Jacinto

REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.3(ii)	F_v	2.5
$0.2*(S_{D1}/S_{DS})$	T_0	0.132
S_{D1}/S_{DS}	T_S	0.659
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.6589*
Equation 11.4-2 - F_V*S_1	S_{M1}	0.9884*

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	1.500
Design Maps	S_1	0.572
Equation 11.4-1 - F_A*S_S	S_{MS}	1.500*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.00*
Design Maps	PGA	0.5
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.55*
Section 21.5.3	80% of PGA_M	0.440
Design Maps	C_{RS}	0.935
Design Maps	C_{R1}	0.912

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.935
Cr - At Perods >=1.0, Cr=C_{R1}	C_{R1}	0.912

Cr - At Periods between 0.2 and 1.0 use trendline formula to complete

Period	Cr
0.200	0.935
0.300	0.932
0.400	0.929
0.500	0.926
0.600	0.924
0.680	0.921
1.000	0.912

* Code based design value. See accompanying data for Site Specific Design values.

Mapped values from https://seismicmaps.org/

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PROBABILISTIC SPECTRA[1]

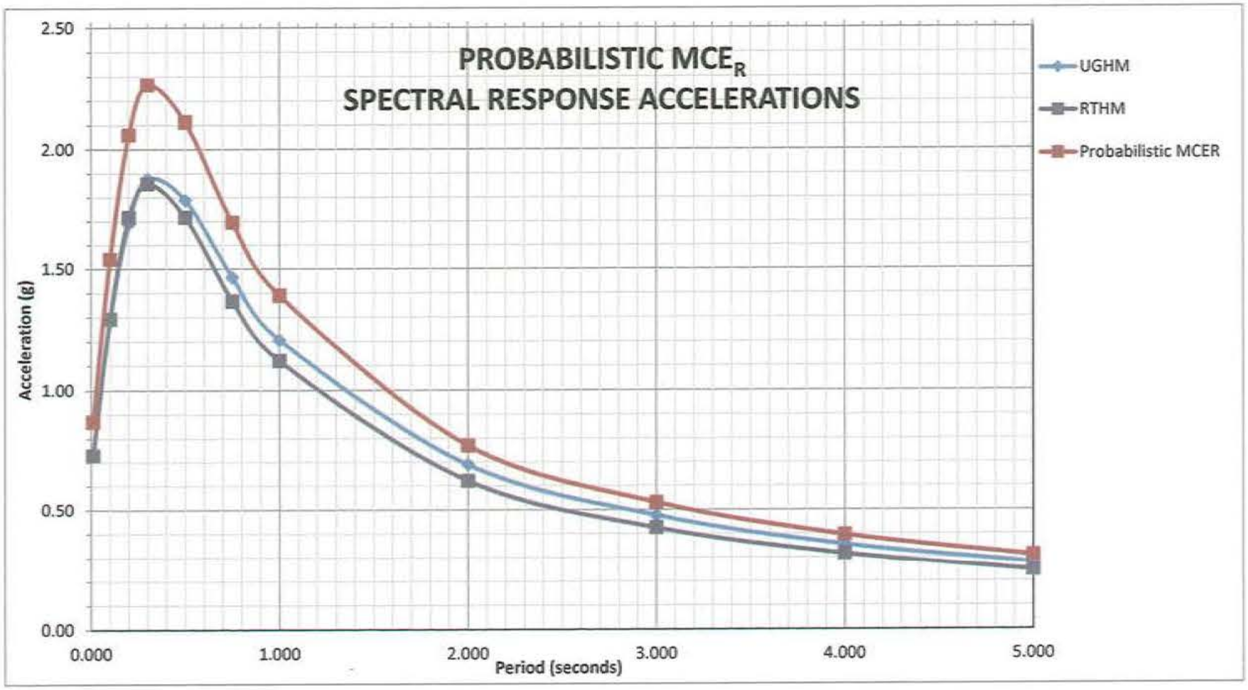
2% in 50 year Exceedence

Project No: 644-22018

Period	UGHM	RTHM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.737	0.731	1.19	0.870
0.100	1.286	1.294	1.19	1.540
0.200	1.691	1.717	1.20	2.060
0.300	1.878	1.858	1.22	2.267
0.500	1.789	1.717	1.23	2.112
0.750	1.466	1.367	1.24	1.695
1.000	1.208	1.121	1.24	1.390
2.000	0.691	0.622	1.24	0.771
3.000	0.477	0.425	1.25	0.531
4.000	0.355	0.316	1.25	0.395
5.000	0.279	0.246	1.26	0.310

Probabilistic PGA: 0.737

Is Probabilistic $Sa_{(max)}<1.2F_a$? NO



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[1] Data Sources:
https://earthquake.usgs.gov/hazards/interactive/
https://earthquake.usgs.gov/designmaps/rtgm/

[2] Shahi-Baker RotD100/RotD50 Factors (2014)

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DETERMINISTIC SPECTRUM

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations[1]

Controlling Source: San Jacinto

Is Probabilistic $Sa_{(max)}$<1.2Fa? NO

Project No: 644-22018

Period	Deterministic PSa Median + 1.σ for 5% Damping	Max Directional Scale Factor[2]	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.506	1.19	0.602	0.602
0.020	0.507	1.19	0.604	0.604
0.030	0.518	1.19	0.616	0.616
0.050	0.563	1.19	0.670	0.670
0.075	0.688	1.19	0.819	0.819
0.100	0.830	1.19	0.987	0.987
0.150	1.036	1.20	1.243	1.243
0.200	1.159	1.20	1.391	1.391
0.250	1.224	1.21	1.481	1.481
0.300	1.247	1.22	1.521	1.521
0.400	1.214	1.23	1.493	1.493
0.500	1.141	1.23	1.404	1.404
0.750	0.887	1.24	1.100	1.100
1.000	0.720	1.24	0.893	0.893
1.500	0.492	1.24	0.610	0.610
2.000	0.362	1.24	0.449	0.449
3.000	0.233	1.25	0.291	0.291
4.000	0.158	1.25	0.198	0.198
5.000	0.114	1.26	0.143	0.143

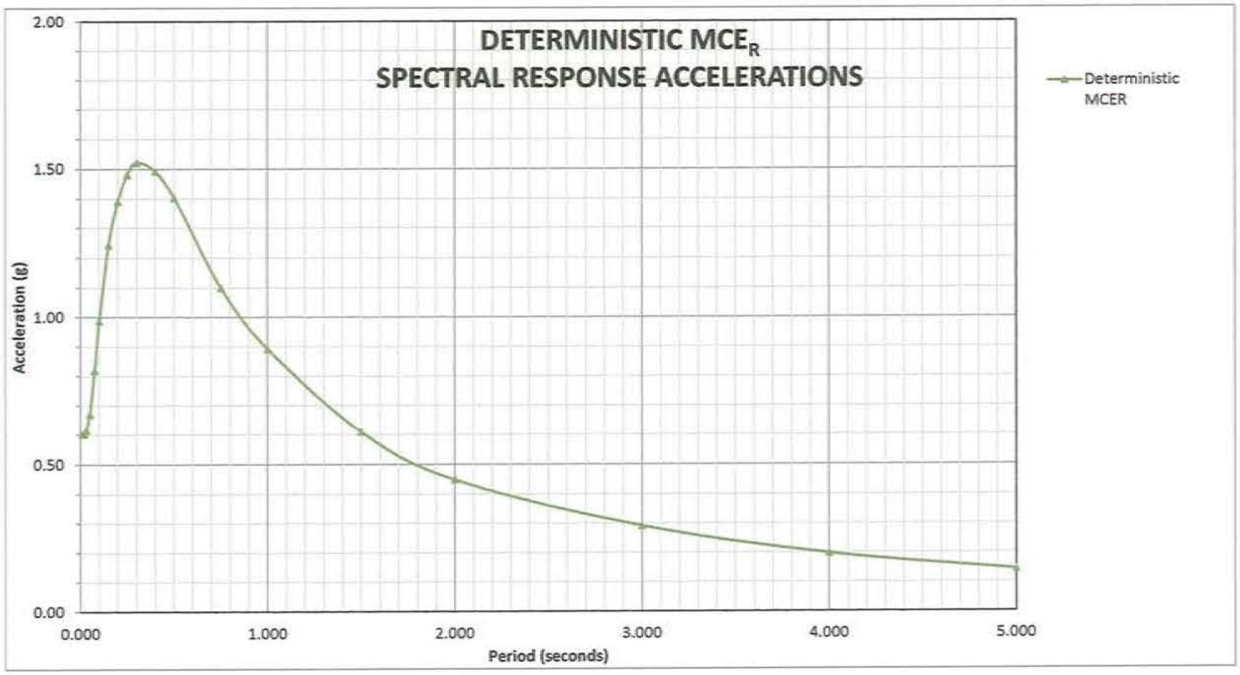
Is Determinstic $Sa_{(max)}$<1.5*Fa? NO

Section 21.2.2 Scaling Factor: N/A

Deterministic PGA: ~~0.506~~

Is Deterministic PGA >=F_{PGA}*0.5? NO

Deterministic PGA: 0.550



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[1] NGAWest 2 GMPE worksheet and Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) - Time Dependent Model

[2] Shahi-Baker RotD100/RotD50 Factors (2014)

SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.870	0.602	0.602	0.402
0.100	1.540	0.987	0.987	0.684
0.200	2.060	1.391	1.391	0.927
0.300	2.267	1.521	1.521	1.014
0.500	2.112	1.404	1.404	0.936
0.750	1.695	1.100	1.100	0.733
1.000	1.390	0.893	0.893	0.595
2.000	0.771	0.449	0.449	0.299
3.000	0.531	0.291	0.291	0.194
4.000	0.395	0.198	0.198	0.132
5.000	0.310	0.143	0.143	0.105

ASCE 7-16: Section 21.4

Site Specific

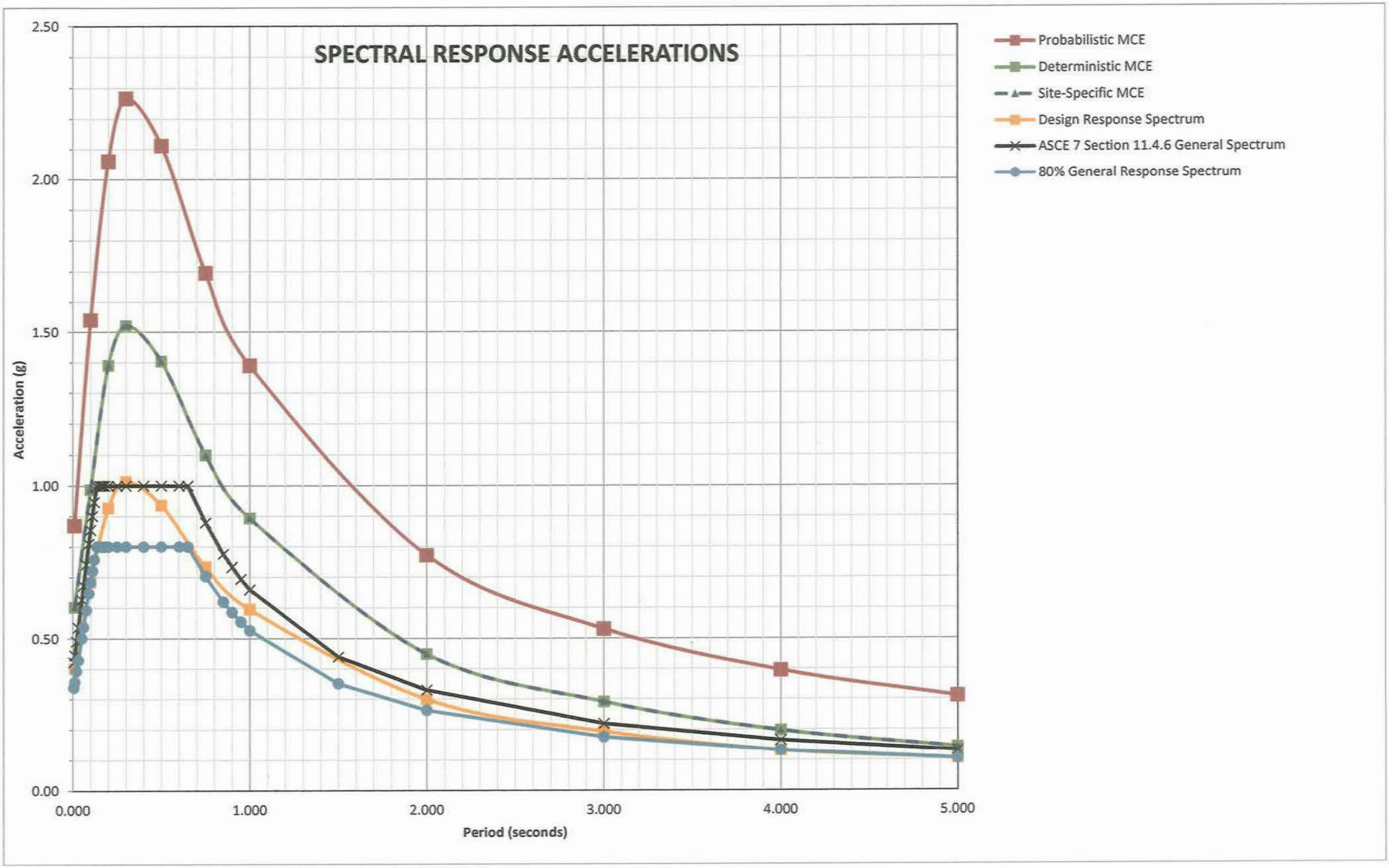
	Calculated Value	Design Value
SDS:	0.913	0.913
SD1:	0.599	0.599
SMS:	1.369	1.369
SM1:	0.898	0.898
Site Specific PGAm:	0.550	0.550
Site Class:	D measured	

Seismic Design Category - Short*	D
Seismic Design Category - 1s*	D

* Risk Categories I, II, or III

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.423	0.338
0.010	0.446	0.356
0.020	0.491	0.393
0.030	0.537	0.429
0.050	0.628	0.502
0.060	0.673	0.539
0.075	0.741	0.593
0.090	0.810	0.648
0.100	0.855	0.684
0.110	0.901	0.721
0.120	0.946	0.757
0.136	1.000	0.800
0.150	1.000	0.800
0.160	1.000	0.800
0.170	1.000	0.800
0.180	1.000	0.800
0.200	1.000	0.800
0.250	1.000	0.800
0.300	1.000	0.800
0.400	1.000	0.800
0.500	1.000	0.800
0.600	1.000	0.800
0.650	1.000	0.800
0.750	0.879	0.703
0.850	0.775	0.620
0.900	0.732	0.586
0.950	0.694	0.555
1.000	0.659	0.527
1.500	0.439	0.351
2.000	0.329	0.264
3.000	0.220	0.176
4.000	0.165	0.132
5.000	0.132	0.105

Project No: 644-22018


SPECTRAL RESPONSE ACCELERATIONS
Probabilistic MCE
Deterministic MCE
Site-Specific MCE
Design Response Spectrum
ASCE 7 Section 11.4.6 General Spectrum
80% General Response Spectrum
Acceleration (g)
2.50
2.00
1.50
1.00
0.50
0.00
Period (seconds)
0.000
1.000
2.000
3.000
4.000
5.000