

APPENDIX J



Geotechnical Investigation

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
APN'S 780-020-001, 002, 003 & 780-010-012
MONROE STREET
LA QUINTA AREA
RIVERSIDE COUNTY, CALIFORNIA

-Prepared By-

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October 17, 2022

Project No. 544-22319
22-10-613

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Subject: Geotechnical Investigation

Project: Proposed Residential Development
APN's 780-020-001 to 003 & 780-010-012
Monroe Street
La Quinta Area, Riverside County, California

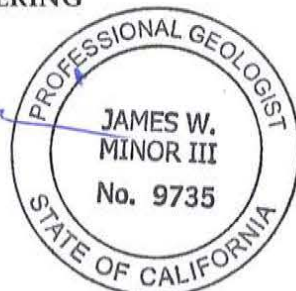
Sladden Engineering is pleased to present the results of the geotechnical investigation performed for the new residential development to be constructed on the currently vacant parcels located on the east side of Monroe Street and on the north and south side of Avenue 53 (APN's 780-020-001, 002, 003 & 780-010-012) in the La Quinta area of Riverside County, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated July 1, 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 the design of the various site improvements. 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 in 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.

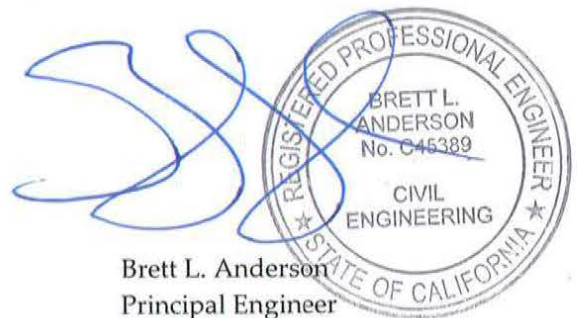
Respectfully submitted,
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 APN'S 780-020-001, 002, 003 & 780-010-012
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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the new residential development to be constructed on the currently vacant parcels located on the east side of Monroe Street and on the north and south side of Avenue 53 (APN's 780-020-001, 002, 003 & 780-010-012) in the La Quinta area of Riverside County, California. The site is located at approximately 33.6626 degrees north longitude and 116.2296 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 provided Conceptual Master Plan, it is our understanding that the proposed project will consist of constructing 31 new residences on the subject property. In addition, the proposed project will also include a lagoon with a beach, a barn, a racquet club and a golf practice range. For our analyses we expect that the proposed residences will be of relatively lightweight wood-frame construction supported on conventional shallow spread footings and concrete slabs-on-grade.

Based on the relatively level nature of the site, Sladden expects that grading will be limited to minor cuts and fills in order to accomplish the desired elevations and to provide adequate gradients for site drainage. This does not include the removal and re-compaction of the loose compressible near surface soil within the proposed building pad areas. Upon completion of precise grading plans, Sladden should be retained in order to verify that the recommendations presented within in this report are properly 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 structures, we expect that isolated column loads will be less than 20 kips and continuous wall loads will be less than 2.0 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. Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- Drilling ten (10) exploratory boreholes to depths of approximately 5 to 51 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.
- Performing laboratory testing on selected samples to evaluate their engineering characteristics.
- Reviewing geologic literature and discussing geologic hazards.
- Performing site-specific ground motion procedures for the subject property.
- Performing 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 subject property is located on the east side of Monroe Street between Avenue 52 and Avenue 54 in the La Quinta area of Riverside County, California. The property is formally identified by the County of Riverside as APN's 780-020-001, 780-020-002, 780-020-003 and 780-010-012 and occupies a total area of approximately 135.04 acres. At the time of our investigation, the ground surface appeared to have been recently disked. A concrete slab was observed within the southern portion of the property. Three (3) irrigation ponds were observed near the northwest corner, the southern side and the northeast corner of the property. CVWD irrigation standpipes were observed along the northern side of the property. Ongoing farming operations were evident within the eastern portion of the property. Avenue 53 transects the subject property in the east to west direction. The site is generally level throughout and near the elevation of the adjacent properties and streets. The site is bounded by a date palm tree orchard on the north, by agricultural property on the east, by residential property on the south and by Monroe Street on the west.

Based on our review of the La Quinta 7.5-Minute Quadrangle Map (USGS, 2018), the site is situated at an approximate elevation of 30 feet below mean sea level (MSL).

No natural ponding of water or surface seeps were observed at or near the sites during our investigation conducted on August 9, 2022. Site drainage appears to be controlled via sheet flow and surface infiltration. Regional drainage is provided by the Whitewater River to the north of the project site.

GEOLOGIC SETTING

The project site is located within the Colorado Desert Physiographic Province (also referred to as the Salton Trough) that is characterized as a northwest-southeast trending structural depression extending from the Gulf of California to the Banning Pass. The Salton Trough is dominated by several northwest trending faults, most notably the San Andreas Fault system. The Salton Trough is bounded by the Santa Rosa – San Jacinto Mountains on the southwest, the San Bernardino Mountains on the north, the Little San Bernardino - Chocolate – Orocochia Mountains on the east and extends through the Imperial Valley into the Gulf of California on the south.

A relatively thick sequence (20,000 feet) of sediment has been deposited in the Coachella Valley portion of the Salton Trough from Miocene to present times. These sediments are predominately terrestrial in nature with some lacustrine (lake) and minor marine deposits. The major contributor of these sediments has been the Colorado River. The mountains surrounding the Coachella Valley are composed primarily of Precambrian metamorphic and Mesozoic “granitic” rock.

The Salton Trough is an internally draining area with no readily available outlet to Gulf of California and with portions well below sea level (-253' msl). The region is intermittently blocked from the Gulf of California by the damming effects of the Colorado River delta (current elevation +30' msl). Between about 300AD and 1600 AD (to 1700) the Salton Trough has been inundated by the River's water, forming ancient Lake Cahuilla (max. elevation +58' msl). Since that time the floor of the Trough has been repeatedly flooded with other “fresh” water lakes (1849, 1861, and 1891), the most recent and historically long lived being the current Salton Sea (1905). The sole outlet for these waters is evaporation, leaving behind vast amounts of terrestrial sediment materials and evaporite minerals.

The site has been mapped by Rogers (1965) to be immediately underlain by Quaternary-age alluvium (Qal) and lake deposits (Ql). The regional geologic setting for the site vicinity is presented on the Regional Geologic Map (Figure 2).

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling ten (10) exploratory boreholes on the property to depths between approximately 5 to 51 feet bgs. The approximate locations of the boreholes are illustrated on the Borehole Location Photograph (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 analysis.

During our field investigation, a thin mantle of artificial fill/disturbed soil was encountered to a depth of approximately three (3) feet bgs. Underlying the fill soil and extending to the maximum depths explored, native alluvium was encountered. The native soil throughout the site consists primarily of fine-grained sand/silty sand (SP/SM) and sandy clay (CL/CH) encountered throughout the depth of our bores. Generally, granular soil was found to be slightly moist to moist, loose to dense and grayish brown in in-situ color. Cohesive layers was found to be moist to wet, soft to stiff, olive brown in in-situ color and exhibited medium to high plasticity characteristics.

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 at the maximum explored depth of 51 feet bgs during our field investigation. Based upon the depth to groundwater in the project vicinity (CDWR, 2022), it is our opinion that groundwater should not be a factor in the design or 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 which splay or step from 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).

As previously stated, the site has been subjected to strong seismic shaking related to active faults that traverse through the region. Some of the more significant seismic events near the subject site within recent times include: M6.0 North Palm Springs (1986), M6.1 Joshua Tree (1992), M7.3 Landers (1992), M6.2 Big Bear (1992), M7.1 Hector Mine (1999), and M7.1 Ridgecrest (2019).

Table 2 lists the closest known potentially active faults that was generated in part using fault parameters from The Revised 2002 California Probabilistic Seismic Hazard Maps (Cao et al, 2003) and the Fault and Fold Database of the United States (USGS, 2021a). This table does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any faults in the region.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
San Andreas - Coachella	8.9	7.2
San Andreas - Southern	8.9	7.2
San Jacinto - Anza	32.5	6.5
Burnt Mountain	34.2	6.4
San Jacinto - Coyote Creek	34.4	7.2
Eureka Peak	35.3	7.5
San Andreas - San Bernardino	37.0	6.8

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	33.6626/ -116.2296
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	1.595	---
MCE _R Ground Motion (1.0 second period)	S ₁	0.656	---
Site-Modified Spectral Acceleration Value	S _{MS}	1.595	1.786
Site-Modified Spectral Acceleration Value	S _{M1}	null	1.549
Numeric Seismic Design Value at 0.2 second SA	S _{DS}	1.063	1.190
Numeric Seismic Design Value at 1.0 second SA	S _{D1}	null	1.033
Site Amplification Factor at 0.2 second	F _a	1.0	1.0
Site Amplification Factor at 1.0 second	F _v	null	2.5
Site Peak Ground Acceleration	PGA _M	0.767	0.689

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 our review of Rogers (1965), Jennings (1994), and CDOC (2022) known active 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 Earth, 2022). Finally, no signs of active surface fault rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified 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. Based on site-specific ground motion parameters developed for the property (Appendix C), the site modified peak ground acceleration (PGAm) is estimated to be 0.689g.
- III. Liquefaction. 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. Based on the depth to groundwater in the project vicinity, risks associated with liquefaction are considered "negligible".
- IV. Tsunamis and Seiches. Because the site is situated at an inland location and is not immediately adjacent to any impounded bodies of water, risks associated with tsunamis and seiches are considered "negligible".
- V. Slope Failure, Landsliding, Rock Falls. Slope instability in the form of landslides and rock falls were not observed at or near the subject site. The site is situated on relatively flat ground and is not located immediately adjacent to any slopes. As such, risks associated with slope instability (landslides, mass wasting and rock falls) are considered "negligible".
- VI. Expansive Soil. Generally, the near surface soil consists of silty sand (SM). Based on the results of our laboratory testing (EI=0 & EI=1 for BH-1 & BH-8, respectively), the materials underlying the site are considered to be "Non Expansive".
- VII. Static Settlement. Static settlement resulting from the anticipated foundation loads should be tolerable provided that the recommendations included in this report are considered in foundation design and construction. The ultimate static settlement is expected to be less than 1 inch when using the recommended allowable bearing pressures. As a practical matter, differential static settlement between footings can be assumed as one-half of the total static settlement.
- VIII. Subsidence. Land subsidence can occur in valleys where aquifer systems have been subjected to extensive groundwater pumping, such that groundwater pumping exceeds groundwater recharge. Generally, pore water reduction can result in a rearrangement of skeletal grains and could result in elastic (recoverable) or inelastic (unrecoverable) deformation of an aquifer system. Locally, no fissures or other surficial evidence of subsidence were observed at or near the subject site.
- IX. Debris Flows. Debris flows are viscous flows consisting of poorly sorted mixtures of sediment and water and are generally initiated on slopes steeper than approximately six horizontal to one vertical (6H:1V) (Boggs, 2001). Based on the flat nature of the site and the composition of the surface soil, we judge that risks associated with debris flows should be considered "negligible".

- X. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation. 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 included in this report are incorporated into design and carried out through construction. The main geotechnical concerns are the presence of artificial fill soil and the loose and potentially compressible near surface native soil throughout the project site.

We recommend that remedial work within the proposed building areas include over-excavation and re-compaction of the primary foundation bearing soil to provide uniform foundation support and to help mitigate potential differential settlements. Specific recommendations for foundation area preparation are presented in the Earthwork and Grading section of this report.

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 and laboratory investigation.

EARTHWORK AND GRADING

All earthwork including excavation, backfill and preparation of the primary foundation and/or slab bearing soil should be performed in accordance with the geotechnical recommendations presented in this report and portions of the local regulatory requirements, as applicable. All earthwork 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 vegetation, associated root systems, underground utilities and debris. All areas scheduled to receive fill should be cleared of old fills and any irreducible matter. The strippings should be removed off-site, or stockpiled for later use in landscape areas. Voids left by obstructions should be properly backfilled in accordance with the compaction recommendations of this report.

- b. Preparation of the Building Areas. In order to provide firm and uniform foundation bearing conditions, we recommend over-excavation and re-compaction throughout the proposed building areas. All artificial fill soil and native low density near surface native soil should be removed to a depth of at least 5 feet below existing grade or 4 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally, a minimum of five feet beyond the building limits. The native soil exposed by over-excavation should be scarified, moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction prior to fill placement. The previously removed soil may then be replaced as compacted engineered soil in accordance with the recommendations below.
- c. Compaction: Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances. All fill material should be placed in thin lifts, not exceeding six inches in a loose condition. 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 and all fill 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 on all lifts in order to ensure proper placement of the fill materials. 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 for 5 feet beyond the building limits and to a minimum of 5 feet below existing grade or 4 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, at near optimum moisture content and compact to a minimum of 90 percent relative compaction.
Asphalt Concrete Sections	Compact the top 12 inches to at least 95 percent compaction at near 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 should be between 10 and 20 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 tenth and 3 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.

CONVENTIONAL SHALLOW SPREAD FOOTINGS

Conventional shallow spread footings are expected to provide adequate support for the proposed residential structures. All footings should be founded upon properly compacted engineered fill soil and should have a minimum embedment depth of 12 inches measured from the lowest adjacent finished grade. Continuous and isolated footings should have minimum widths of 12 inches and 24 inches, respectively. Continuous and isolated footings supported upon properly compacted engineered fill soil may be designed using allowable (net) bearing pressures of 1800 and 2000 pounds per square foot (psf), respectively. Allowable increases of 200 psf for each additional 1 foot of width and 250 psf for each additional 6 inches of depth may be used if desired. The maximum allowable bearing pressure should be 3000 psf. The allowable bearing pressures apply to combined dead and sustained live loads. The allowable bearing pressures may be increased by one-third when considering transient live loads, including seismic and wind forces.

Based on the recommended allowable bearing pressures, the total static settlement of the shallow footings is anticipated to be less than one-inch provided foundation preparations conform to the recommendations described in this report. Static differential settlement is anticipated to be approximately one-half of the total settlement for similarly loaded footings spaced up to approximately 40 feet apart.

Lateral load resistance for the spread footings will be developed by passive pressure against the sides of the footings below grade and by friction acting at the base of the footings. An allowable passive pressure of 250 psf per foot of depth may be used for design purposes. An allowable coefficient of friction 0.45 may be used for dead and sustained live loads to compute the frictional resistance of the footing placed directly on compacted fill. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

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. All footings should be reinforced in accordance with the project Structural Engineer's recommendations.

SLABS-ON-GRADE

In order to provide uniform and adequate support, concrete slabs-on-grade must be placed on properly compacted engineered fill soil as outlined in the previous sections of this report. The slab subgrade should remain near optimum moisture content and should not be permitted to dry prior to concrete placement. Slab subgrade should be firm and unyielding. Disturbed soil should be removed and replaced with engineered fill soil compacted to a minimum of 90 percent relative compaction.

Slab thickness and reinforcement should be determined by the Structural Engineer. We recommend a minimum slab thickness of 4.0 inches and minimum reinforcement of #3 bars at 18 inches on center in both directions. All slab reinforcement should be supported on concrete chairs to ensure that reinforcement is placed at slab mid-height. Final floor slab design and reinforcement should be determined by the Structural Engineer.

Slabs with moisture sensitive surfaces should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 10-mil visqueen, or equivalent. 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. 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 1-inch thick leveling course of sand across the pad surface prior to placement of the membrane.

RETAINING WALLS

Minor retaining walls may be necessary to complete the proposed construction. Cantilever retaining walls may be designed using "active" pressures. Active pressures may be estimated using an equivalent fluid weight of 35 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 55 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.

We recommend that the ground surface behind retaining walls be sloped to drain. Under no circumstances should the surface water be diverted into back drains. Where migration of moisture through walls would be detrimental, the walls should be waterproofed.

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 80 & 40 parts per million (ppm) for BH-1 & BH-8, respectively. The soil is considered to have a "negligible-S0" corrosion potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes may not be necessary. 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 8.7 & 9.1 for BH-1 & BH-8, respectively. Based on soluble chloride concentration testing (80 & 50 ppm for BH-1 & BH-8, respectively) the soil is considered to have "negligible" corrosive potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 3,600 & 4,600 ohm-cm for BH-1 & BH-8, respectively that suggests the site soil is considered to have "moderate" 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 mechanically compacted to a minimum of 90 percent relative compaction. A representative of the project soil engineer should test the backfill to verify adequate compaction.

EXTERIOR CONCRETE FLATWORK

In order to provide uniform support and minimize settlement related cracking of concrete flatwork, the subgrade soil within concrete flatwork areas should 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 prior to concrete placement.

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 bore locations and extrapolation of these conditions throughout the proposed building areas. 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 ensure 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 extend into the properly compacted 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 engineered fill soil and 95 percent for Class II aggregate base as obtained by ASTM Test Method D1557. Where testing indicates insufficient density, additional compactive effort shall be applied until retesting indicates satisfactory compaction.

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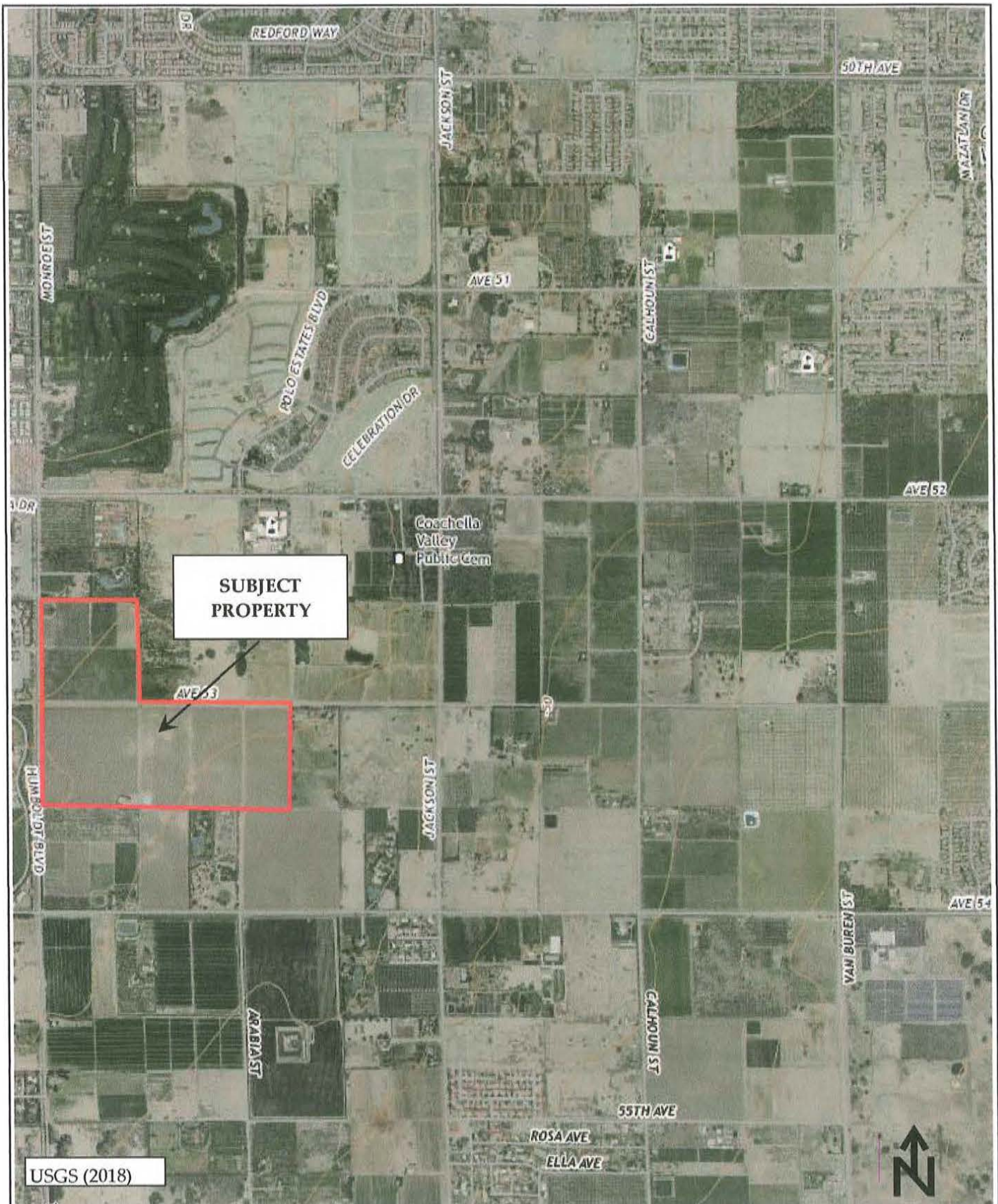
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FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PHOTOGRAPH
CONCEPTUAL MASTER PLAN



SITE LOCATION MAP

FIGURE

1



Sladden Engineering

Project Number:

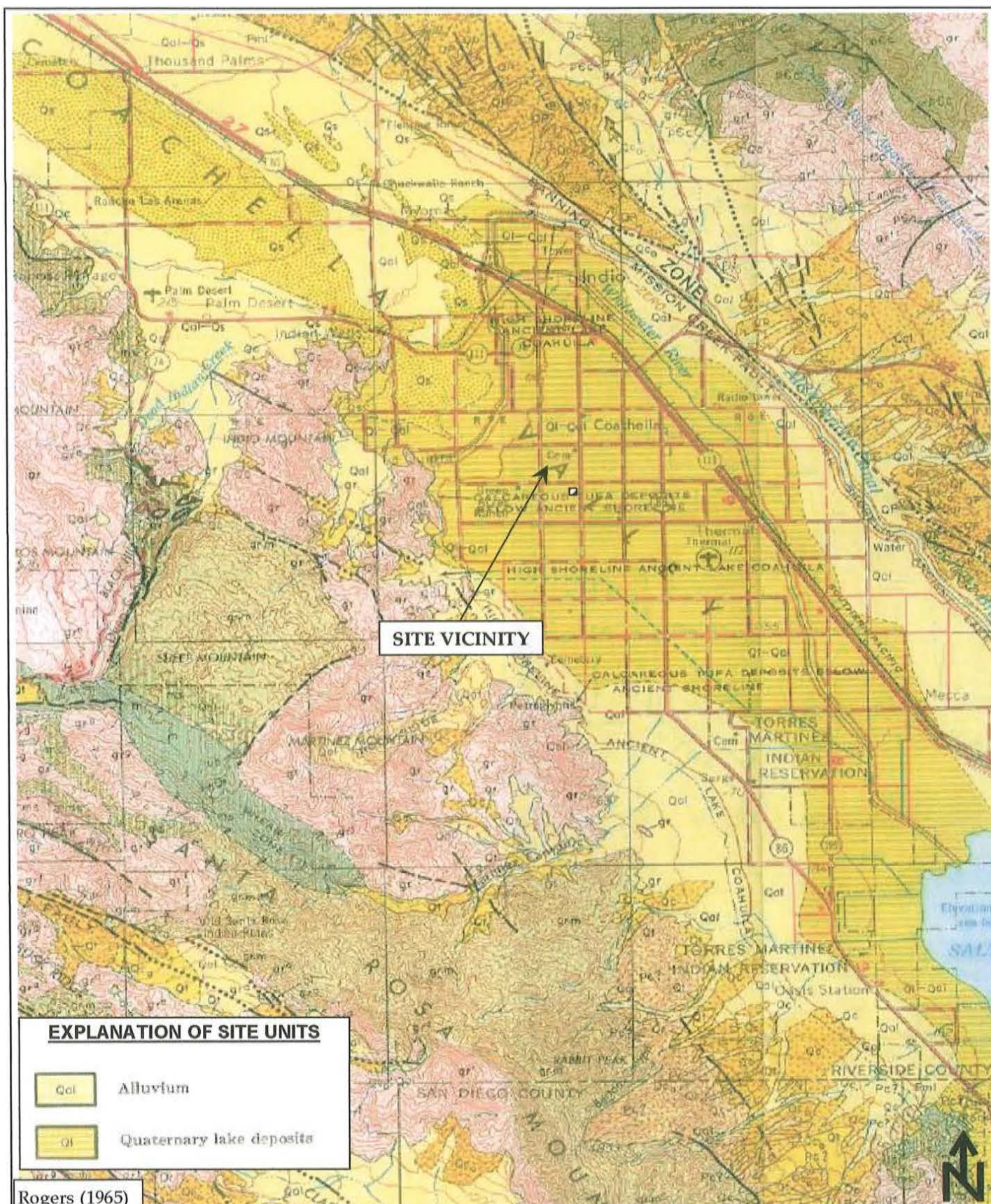
544-22319

Report Number:

22-10-613

Date:

October 18, 2022



REGIONAL GEOLOGIC MAP

FIGURE

2



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Project Number:

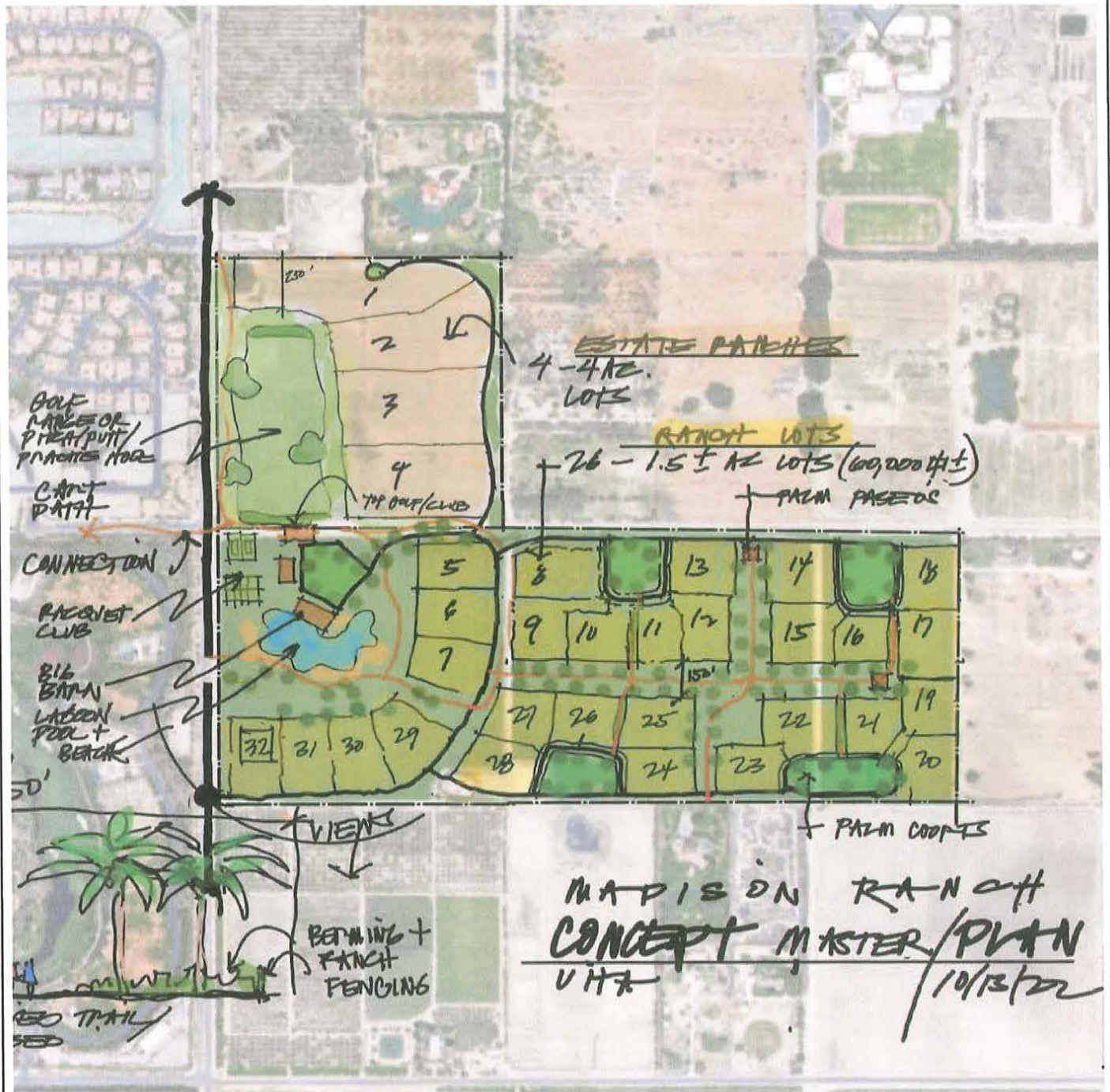
544-22319

Report Number:

22-10-613

Date:

October 18, 2022



CONCEPT MASTER PLAN

FIGURE

4



Sladden Engineering

Project Number:

544-22319

Report Number:

22-10-613

Date:

October 18, 2022

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation ten (10) exploratory boreholes were excavated on August 9, 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 borings 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.

**SLADDEN ENGINEERING****BORE LOG**

Drill Rig: Mobile B-61 Date Drilled: 8/9/2022

Elevation: -30 Ft (MSL) Boring No: BH-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
	4/5/9	1	0	26.0	7.5	110.3	2		Silty Sand (SM); grayish brown, slightly moist, loose, fine-grained (Fill/Disturbed).
	5/6/8			12.1	4.0	96.5	4		Silty Sand (SM); grayish brown, slightly moist, loose, fine-grained (Qal/Ql).
							6		
	4/6/7			10.0	6.1		10		
							12		Sand (SP/SM); grayish brown, moist, loose, fine-grained (Qal/Ql).
							14		
	8/9/11			46.0	10.6	96.8	16		Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
							18		
							20		
	2/2/2			91.5	40.0		22		Sandy Clay (CL/CH); olive brown, very moist to wet, soft, medium to high plasticity (Qal/Ql).
							24		
	5/8/11			56.6	17.0	105.9	26		
							28		
	8/10/10			10.4	3.9		30		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
							32		
							34		Clayey Sand (SC); grayish brown, moist, medium dense, fine-grained (Qal/Ql).
	6/10/16			49.2	12.2	97.3	36		
							38		
	7/9/12			15.8	6.2		40		Clayey Sand (SC); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
							42		
							44		Clayey Sand (SC); grayish brown, slightly moist, dense, fine-grained (Qal/Ql).
	11/19/30			16.5	5.6	110.9	46		
							48		
	9/9/10			26.2	8.9		50		Clayey Sand (SC); grayish brown, slightly moist to moist, medium dense, fine-grained (Qal/Ql).

Completion Notes:

Terminated at ~51.5 Feet bgs.

No Bedrock Encountered.

No Groundwater Encountered.

PROPOSED RESIDENTIAL DEVELOPMENT

APNS 780-020-001, 002, 003 & 780-010-012

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SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	8/9/2022
Elevation:	-30 Ft (MSL)	Boring No:	BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).
	3/4/6			64.1	16.3		4		Sandy Clay (CL/CH); olive brown, very moist, stiff, medium to high plasticity (Qal/Ql).
	6/9/13			10.4	4.1	101.3	10		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
	4/5/6			49.3	9.0		14		Silty Sand (SM); grayish brown, moist, medium dense, fine-grained (Qal/Ql).
	4/7/10			39.5	10.6	105.0	20		Silty Sand (SM); grayish brown, moist, medium dense, fine-grained (Qal/Ql).
							22		
							24		Terminated at ~ 21.5 Feet bgs.
							26		No Bedrock Encountered.
							28		No Groundwater or Seepage Encountered.
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PROPOSED RESIDENTIAL DEVELOPMENT
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2

BORE LOG

Drill Rig:	Mobile B-61
------------	-------------

Date Drilled: 8/9/2022

Elevation: -30 Ft (MSL)

Boring No: BH-3

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).
	3/4/5			71.3	10.0	86.1	4		
							6		Sandy Clay (CL/CH); olive brown, moist, medium stiff, medium to high plasticity (Qal/Ql).
	3/4/5			97.4	32.5		10		Sandy Clay (CL/CH); olive brown, very moist, stiff, medium to high plasticity (Qal/Ql).
							12		
	5/8/15			38.9	6.1	101.4	14		Clayey Sand (SC); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
							16		
	7/9/12			20.7	2.8		20		Clayey Sand (SC); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
							22		
							24		Terminated at ~ 21.5 Feet bgs.
							26		No Bedrock Encountered.
							28		No Groundwater or Seepage Encountered.
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		
Completion Notes:								PROPOSED RESIDENTIAL DEVELOPMENT	
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**SLADDEN ENGINEERING****BORE LOG**

Drill Rig:	Mobile B-61	Date Drilled:	8/9/2022
Elevation:	-30 Ft (MSL)	Boring No:	BH-4

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).
	2/3/3			96.3	34.5		4		Sandy Clay (CL/CH); olive brown, very moist, medium stiff, medium to high plasticity (Qal/Ql).
	7/13/18			7.1	2.7	99.9	10		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).
	4/5/5			40.1	11.0		16		Silty Sand (SM); grayish brown, moist, loose, fine-grained (Qal/Ql).
	3/4/7			91.7	32.5	85.9	20		Sandy Clay (CL/CH); olive brown, very moist, medium stiff, medium to high plasticity (Qal/Ql).
							24		Terminated at ~ 21.5 Feet bgs.
							26		No Bedrock Encountered.
							28		No Groundwater or Seepage Encountered.
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		
Completion Notes:								PROPOSED RESIDENTIAL DEVELOPMENT	
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Drill Rig:	Mobile B-61	Date Drilled:	8/9/2022
Elevation:	-30 Ft (MSL)	Boring No:	BH-5

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description	
	2/3/4			97.5	29.0		2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).	
							4		Sandy Clay (CL/CH); olive brown, very moist, medium stiff, medium to high plasticity (Qal/QI).	
							6			
							8		Terminated at ~ 6.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.	
							10			
							12			
							14			
							16			
							18			
							20			
							22			
							24			
							26			
							28			
							30			
							32			
							34			
							36			
							38			
							40			
							42			
							44			
							46			
							48			
							50			
Completion Notes:								PROPOSED RESIDENTIAL DEVELOPMENT		
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**BORE LOG**

Drill Rig: Mobile B-61

Date Drilled: 8/9/2022

Elevation: -30 Ft (MSL)

Boring No: BH-6

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description				
							2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).				
	7/8/9			11.0	3.1	101.5	4		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).				
						6							
						8							
						10							
	4/5/7			12.7	3.4		12		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/Ql).				
						14							
	4/7/9			83.5	20.1	88.7	16		Sandy Clay (CL/CH); olive brown, very moist, stiff, medium to high plasticity (Qal/Ql).				
							18		Terminated at ~ 16.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.				
							20						
							22						
							24						
							26						
							28						
							30						
							32						
							34						
							36						
							38						
							40						
							42						
							44						
							46						
							48						
							50						
Completion Notes:										PROPOSED RESIDENTIAL DEVELOPMENT			
										APNS 780-020-001, 002, 003 & 780-010-012			
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**BORE LOG**

Drill Rig: Mobile B-61

Date Drilled: 8/9/2022

Elevation: -30 Ft (MSL)

Boring No: BH-7

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description	
							2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).	
							4			
5/6/7				7.8	3.9		6		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/QI).	
							8			
5/7/11				24.4	7.3	96.3	10			
							12		Sand (SP/SM); grayish brown, slightly moist, medium dense, fine-grained (Qal/QI).	
							14			
3/5/6				67.3	22.5		16		Sandy Clay (CL/CH); olive brown, very moist, stiff, medium to high plasticity (Qal/QI).	
							18		Terminated at ~ 16.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.	
							20			
							22			
							24			
							26			
							28			
							30			
							32			
							34			
							36			
							38			
							40			
							42			
							44			
							46			
							48			
							50			
Completion Notes:								PROPOSED RESIDENTIAL DEVELOPMENT		
								APNS 780-020-001, 002, 003 & 780-010-012		
								Project No: 544-22319		
								Report No: 22-10-613		
								Page	7	

**SLADDEN ENGINEERING****BORE LOG**

Drill Rig:	Mobile B-61	Date Drilled:	8/9/2022
Elevation:	-30 Ft (MSL)	Boring No:	BH-8

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description		
		2	1				2		Silty Sand (SM); grayish brown, slightly moist, fine-grained (Fill/Disturbed).		
							4		Sand (SP/SM); grayish brown, slightly moist, fine-grained (Qal/Ql).		
							6		Terminated at ~ 5.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.		
							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 RESIDENTIAL DEVELOPMENT	
										APNS 780-020-001, 002, 003 & 780-010-012	
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								Report No: 22-10-613			
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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 Boring 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 D1557-91, Test Method A. Graphic representations of the results of this testing are presented 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 soil and in selecting samples for further testing.

SOIL MECHANIC'S TESTING

Expansion Testing: Two (2) 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: Two (2) bulk 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/Hydro-Collapse Testing: Two (2) relatively undisturbed sample was 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.

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

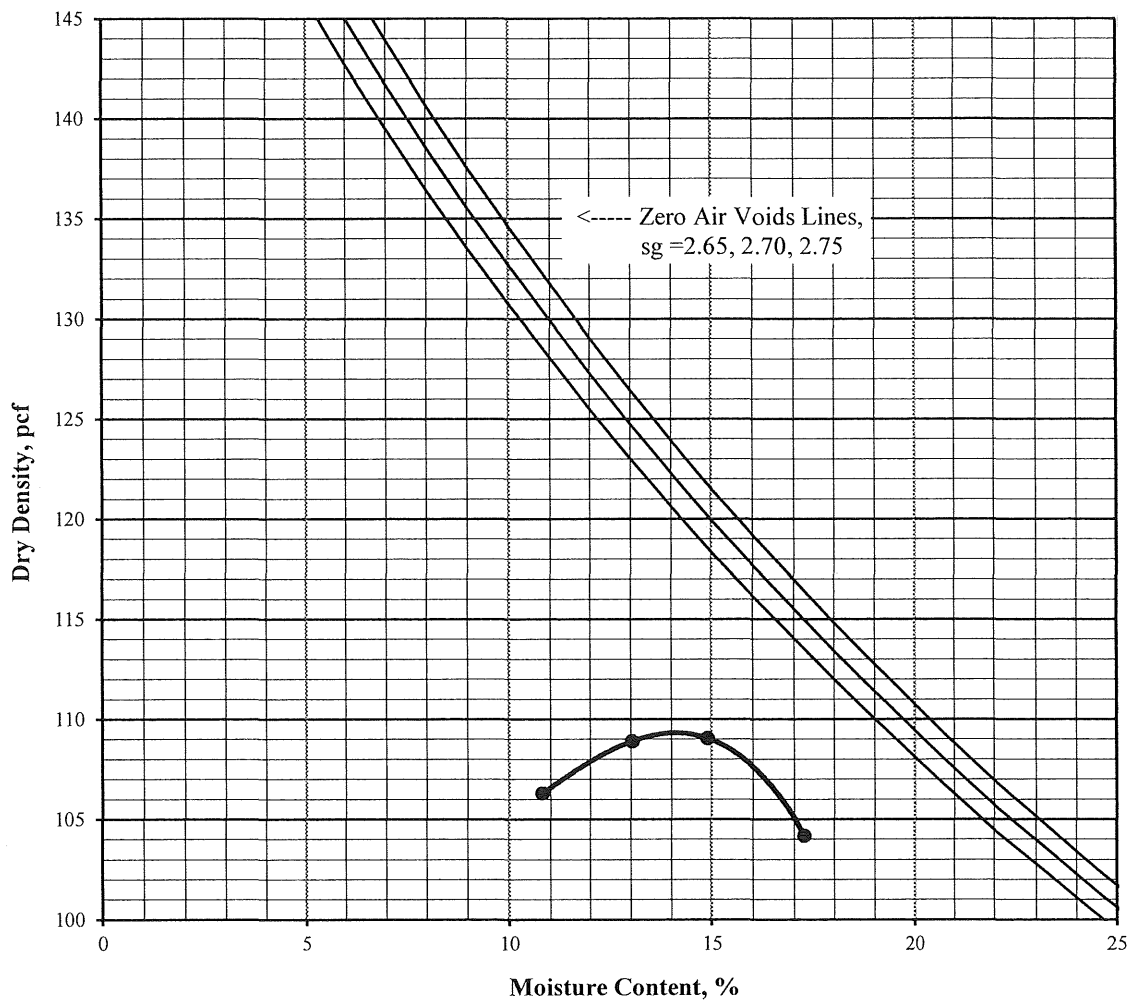
Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample Location: BH-1 Bulk 1 @ 0-5'
Description: Olive Brown Silty Sand (SM)

September 7, 2022

ASTM D-1557 A
Rammer Type: Machine

Maximum Density: 110 pcf
Optimum Moisture: 14%

Sieve Size	% Retained
3/4"	
3/8"	
#4	0.2





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Maximum Density/Optimum Moisture

ASTM D698/D1557

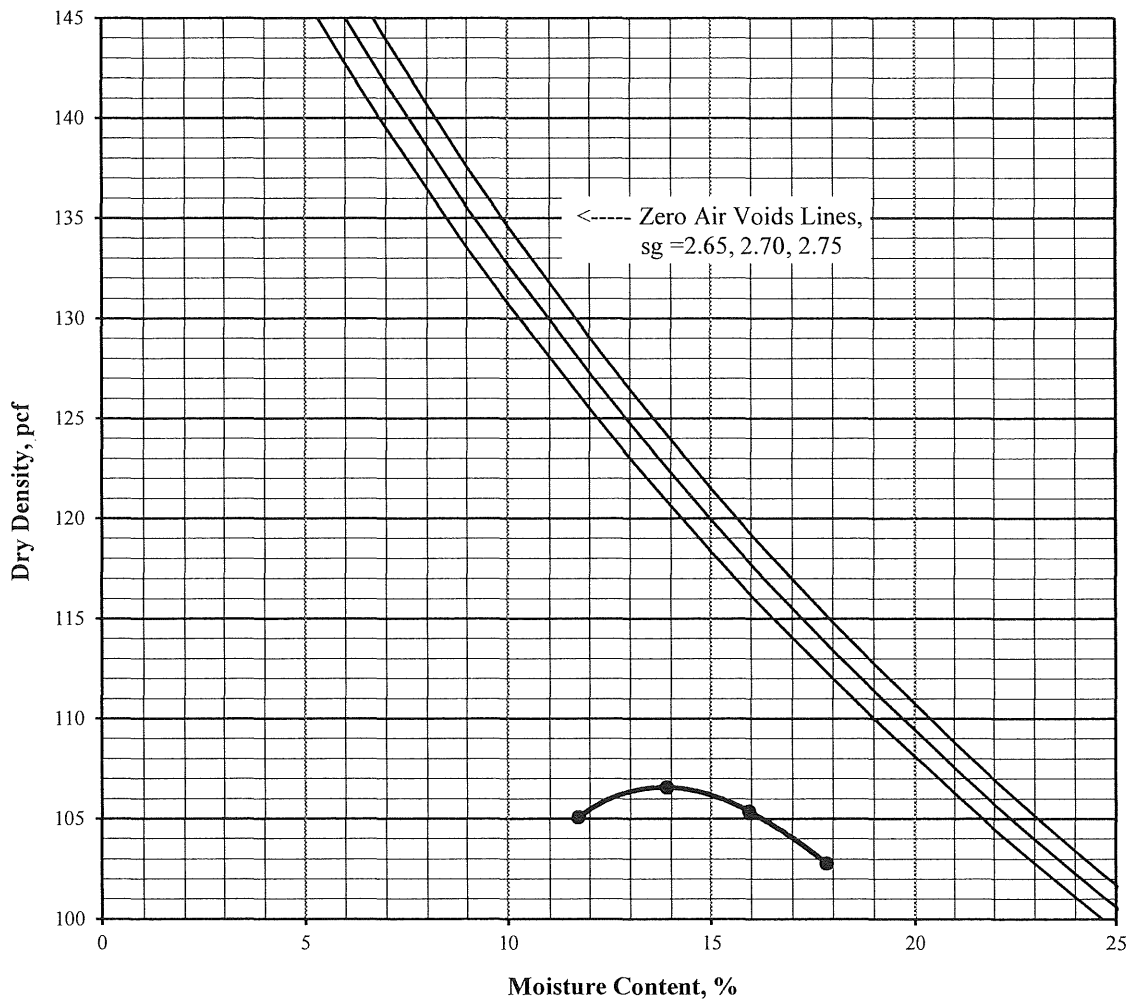
Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample Location: BH-8 Bulk 2 @ 0-5'
Description: Olive Brown Silty Sand (SM)

September 7, 2022

ASTM D-1557 A
Rammer Type: Machine

Maximum Density: 107 pcf
Optimum Moisture: 14.5%

Sieve Size	% Retained
3/4"	
3/8"	
#4	0.0





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Expansion Index

ASTM D 4829

Job Number: 544-22319
Job Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-1 Bulk 1 @ 0-5'
Soil Description: Olive Brown Silty Sand (SM)

September 7, 2022

Wt of Soil + Ring:	560.7
Weight of Ring:	191.0
Wt of Wet Soil:	369.7
Percent Moisture:	11.2%
Sample Height, in	0.95
Wet Density, pcf:	118.3
Dry Denstiy, pcf:	106.4

% Saturation:	51.8
---------------	------

Expansion

Rack # 3

Date/Time	8/30/2022	11:09 AM
Initial Reading	0.0000	
Final Reading	0.0004	

Expansion Index

0

(Final - Initial) x 1000



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Expansion Index

ASTM D 4829

Job Number: 544-22319
Job Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-8 Bulk 2 @ 0-5'
Soil Description: Olive Brown Silty Sand (SM)

September 7, 2022

Wt of Soil + Ring:	542.7
Weight of Ring:	191.0
Wt of Wet Soil:	351.7
Percent Moisture:	12.3%
Sample Height, in	0.95
Wet Density, pcf:	112.5
Dry Density, pcf:	100.2

% Saturation:	48.7
---------------	------

Expansion

Rack # 3

Date/Time	9/1/2022	2:10 PM
Initial Reading	0.0000	
Final Reading	0.0005	

Expansion Index

1

(Final - Initial) x 1000



Sladden Engineering

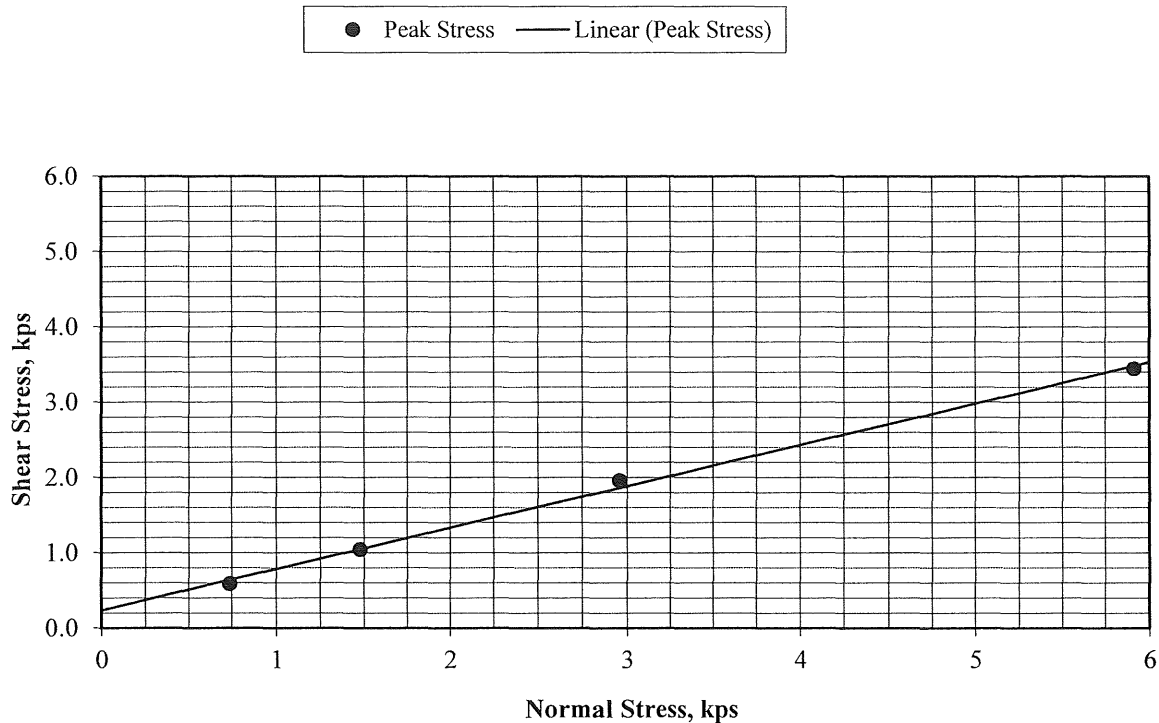
450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

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

Job Number: 544-22319
Job Name: Monroe Street
Lab ID No. LN6-22370
Sample ID: BH-1 Bulk 1 @ 0-5'
Classification: Olive Brown Silty Sand (SM)
Sample Type: Remolded @ 90% of Maximum Density

September 7, 2022
Initial Dry Density: 99.1 pcf
Initial Moisture Content: 13.7 %
Peak Friction Angle (ϕ): 29°
Cohesion (c): 240 psf

Test Results	1	2	3	4	Average
Moisture Content, %	21.8	21.8	21.8	21.8	21.8
Saturation, %	83.9	83.9	83.9	83.9	83.9
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.589	1.046	1.962	3.444	





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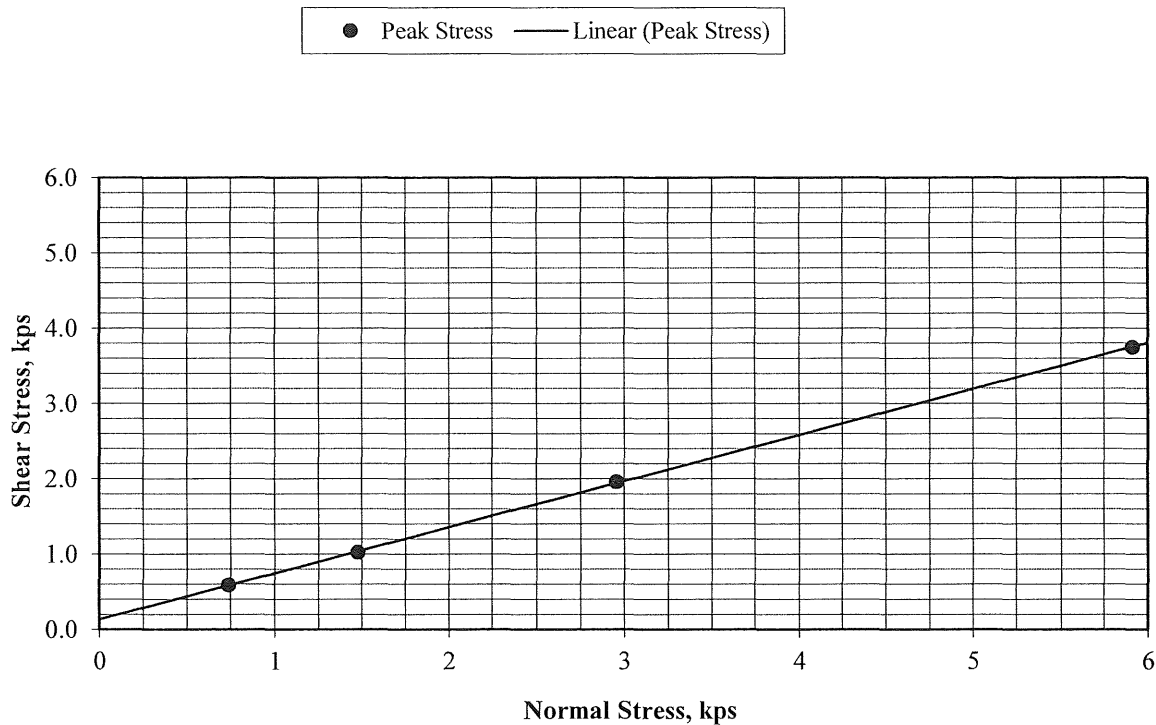
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Direct Shear ASTM D 3080-04 (modified for unconsolidated condition)

Job Number: 544-22319
Job Name: Monroe Street
Lab ID No. LN6-22370
Sample ID: BH-8 Bulk 2 @ 0-5'
Classification: Olive Brown Silty Sand (SM)
Sample Type: Remolded @ 90% of Maximum Density

September 7, 2022
Initial Dry Density: 96.3 pcf
Initial Moisture Content: 14.6 %
Peak Friction Angle (ϕ): 31°
Cohesion (c): 130 psf

Test Results	1	2	3	4	Average
Moisture Content, %	24.9	24.9	24.9	24.9	24.9
Saturation, %	89.6	89.6	89.6	89.6	89.6
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.589	1.025	1.962	3.750	



Job Number: 544-22319
Job Name: Monroe Street
Date: 9/7/2022

Moisture Adjustment	
Wt of Soil:	<u>1,000</u>
Moist As Is:	<u>6.5</u>
Moist Wanted:	<u>14.0</u>

ml of Water to Add: 70.4

Remolded Shear Weight	
Max Dry Density:	110.0
Optimum Moisture:	14.0

Wt Soil per Ring, g: 135.7

UBC



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Gradation

ASTM C117 & C136

Project Number: 544-22319

September 7, 2022

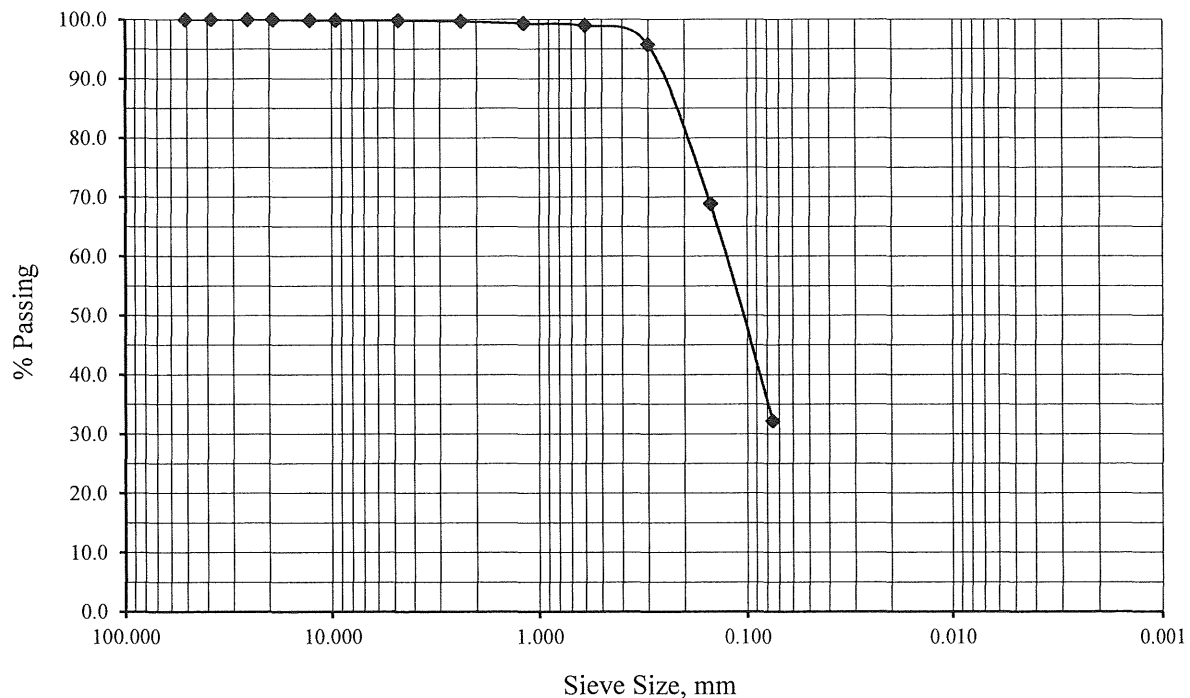
Project Name: Monroe Street

Lab ID Number: LN6-22370

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

Soil Classification: SM

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	99.8
#8	2.36	99.7
#16	1.18	99.3
#30	0.60	99.0
#50	0.30	95.8
#100	0.15	68.9
#200	0.075	32.2





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ASTM C117 & C136

Project Number: 544-22319

September 7, 2022

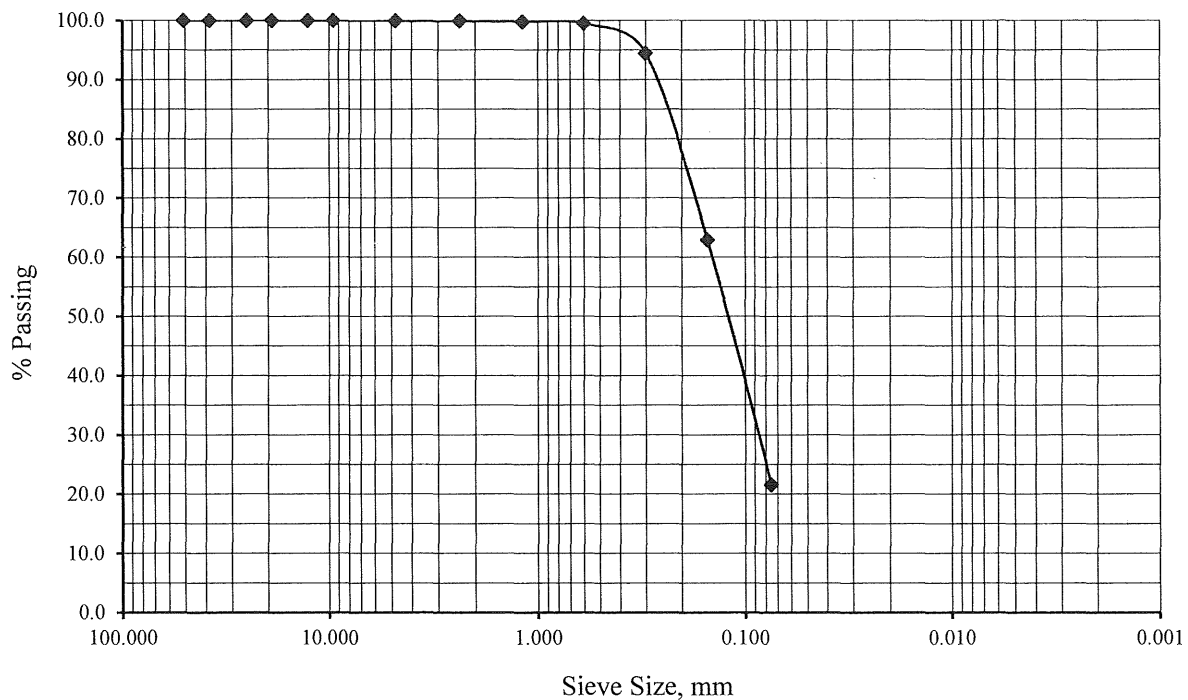
Project Name: Monroe Street

Lab ID Number: LN6-22370

Sample ID: BH-8 Bulk 2 @ 0-5'

Soil Classification: SM

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	100.0
3/8"	9.53	100.0
#4	4.75	100.0
#8	2.36	99.9
#16	1.18	99.8
#30	0.60	99.5
#50	0.30	94.5
#100	0.15	63.0
#200	0.075	21.6





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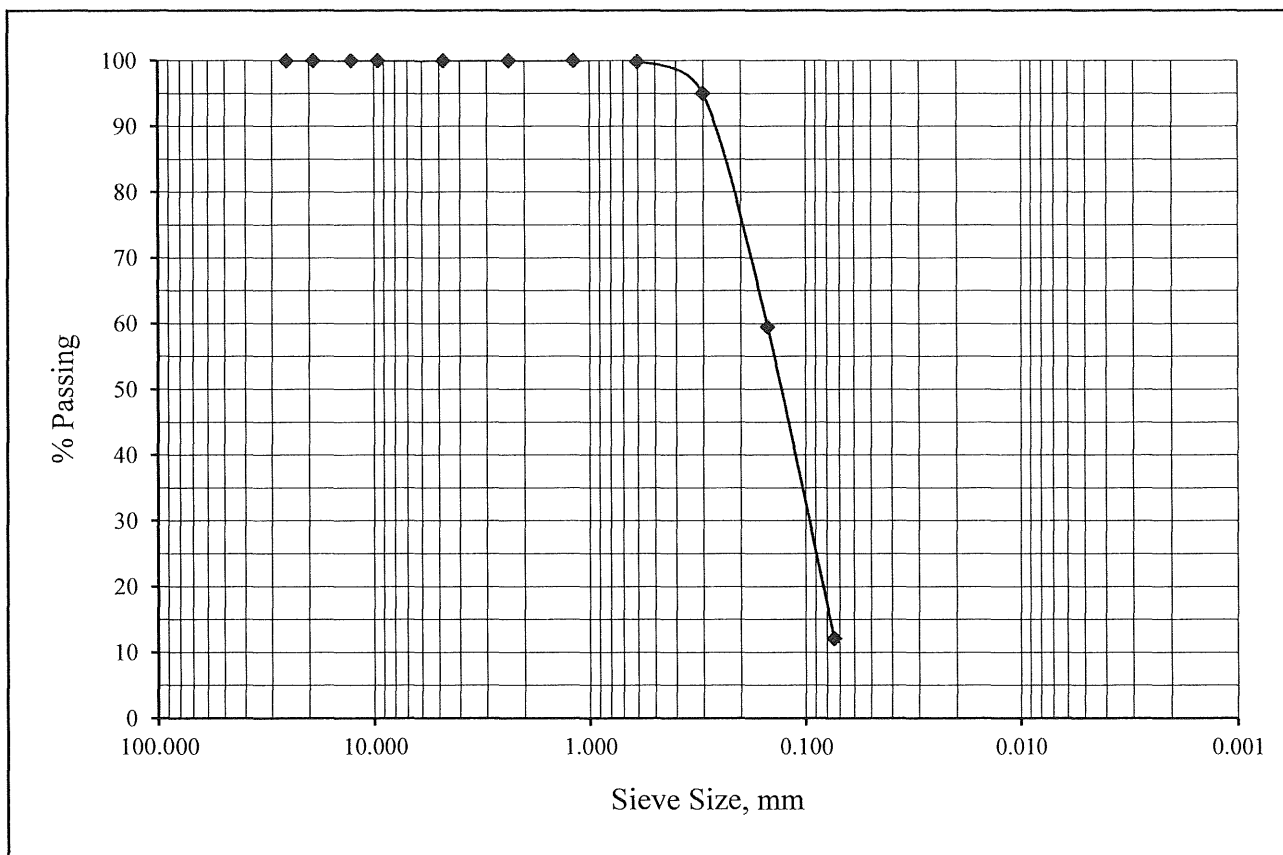
ASTM C117 & C136

Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-1 R-2 @ 5'

September 7, 2022

Soil Classification: SM

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	100.0
#8	2.36	100.0
#16	1.18	100.0
#30	0.60	99.9
#50	0.30	95.0
#100	0.15	59.5
#200	0.074	12.1





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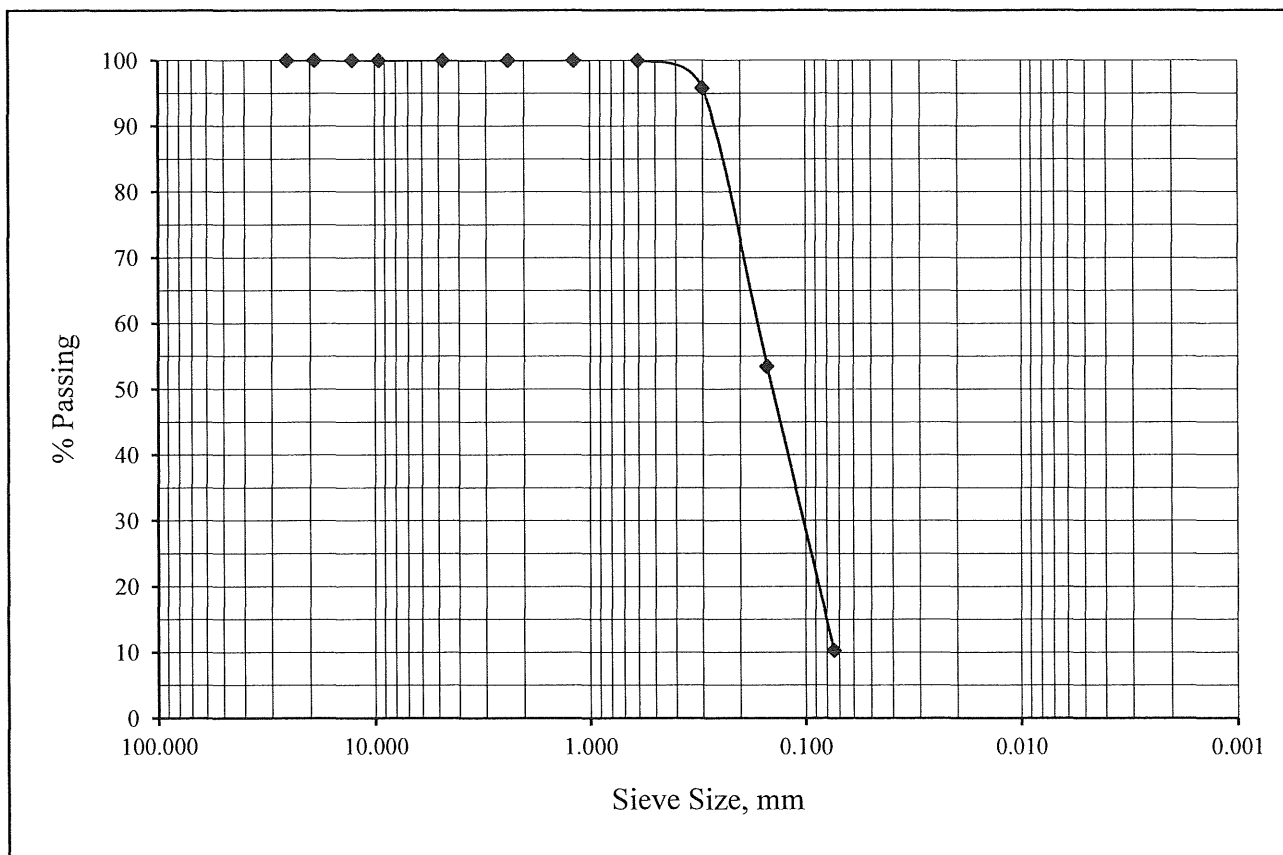
ASTM C117 & C136

Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-2 R-2 @ 10'

September 7, 2022

Soil Classification: SP-SM

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	100.0
#8	2.36	100.0
#16	1.18	100.0
#30	0.60	100.0
#50	0.30	95.8
#100	0.15	53.5
#200	0.074	10.4





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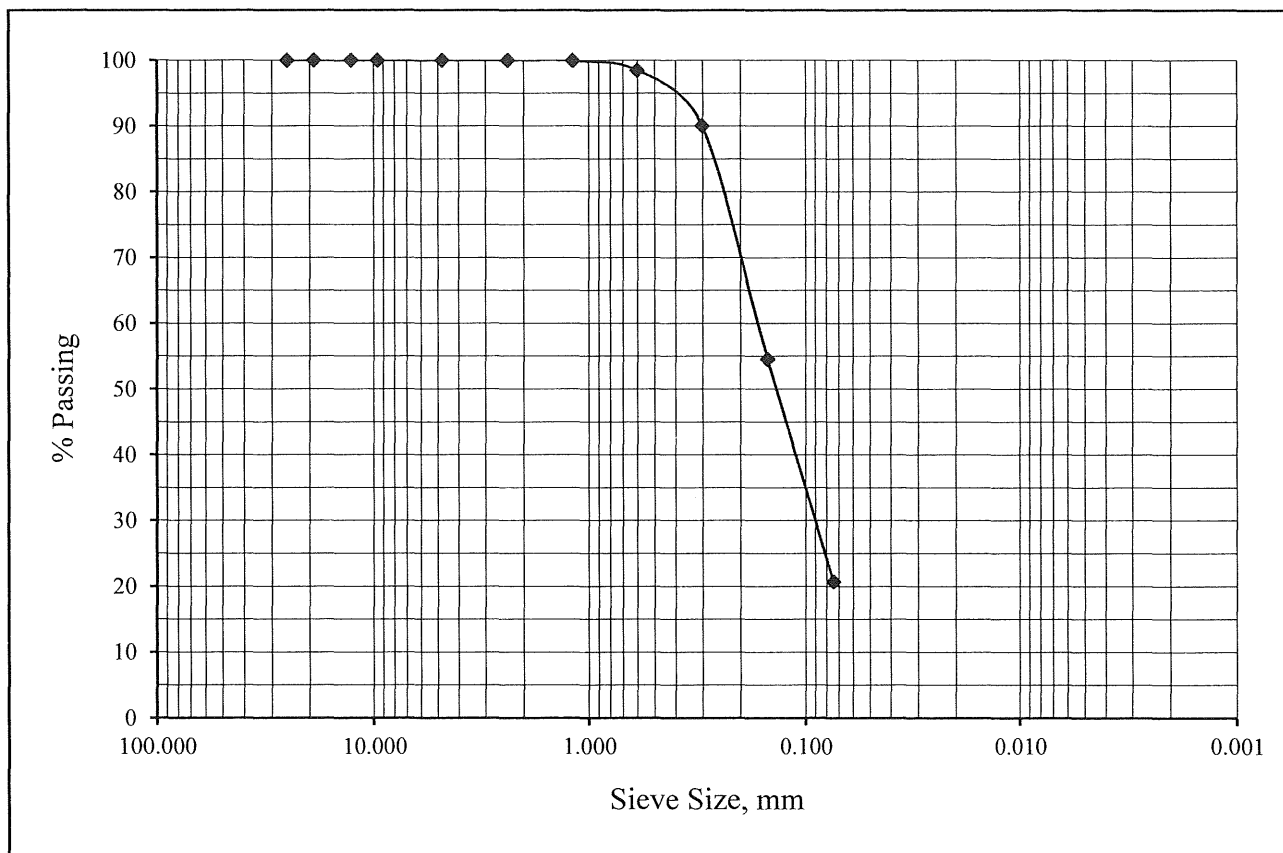
ASTM C117 & C136

Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-3 S-4 @ 20'

September 7, 2022

Soil Classification: SM

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	100.0
#8	2.36	100.0
#16	1.18	100.0
#30	0.60	98.5
#50	0.30	90.1
#100	0.15	54.5
#200	0.074	20.7





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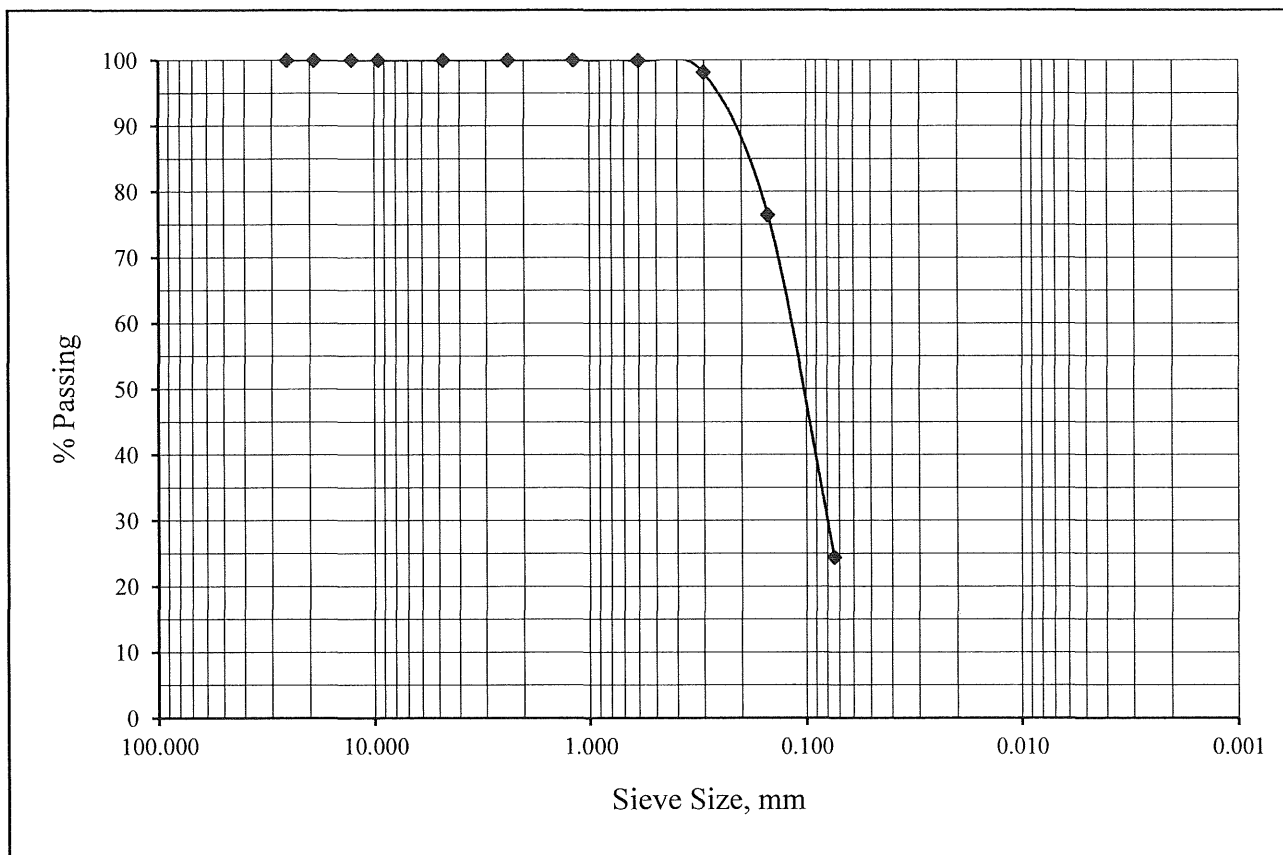
ASTM C117 & C136

Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-7 R-2 @ 10'

September 7, 2022

Soil Classification: SM

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	100.0
#8	2.36	100.0
#16	1.18	100.0
#30	0.60	99.9
#50	0.30	98.1
#100	0.15	76.5
#200	0.074	24.4





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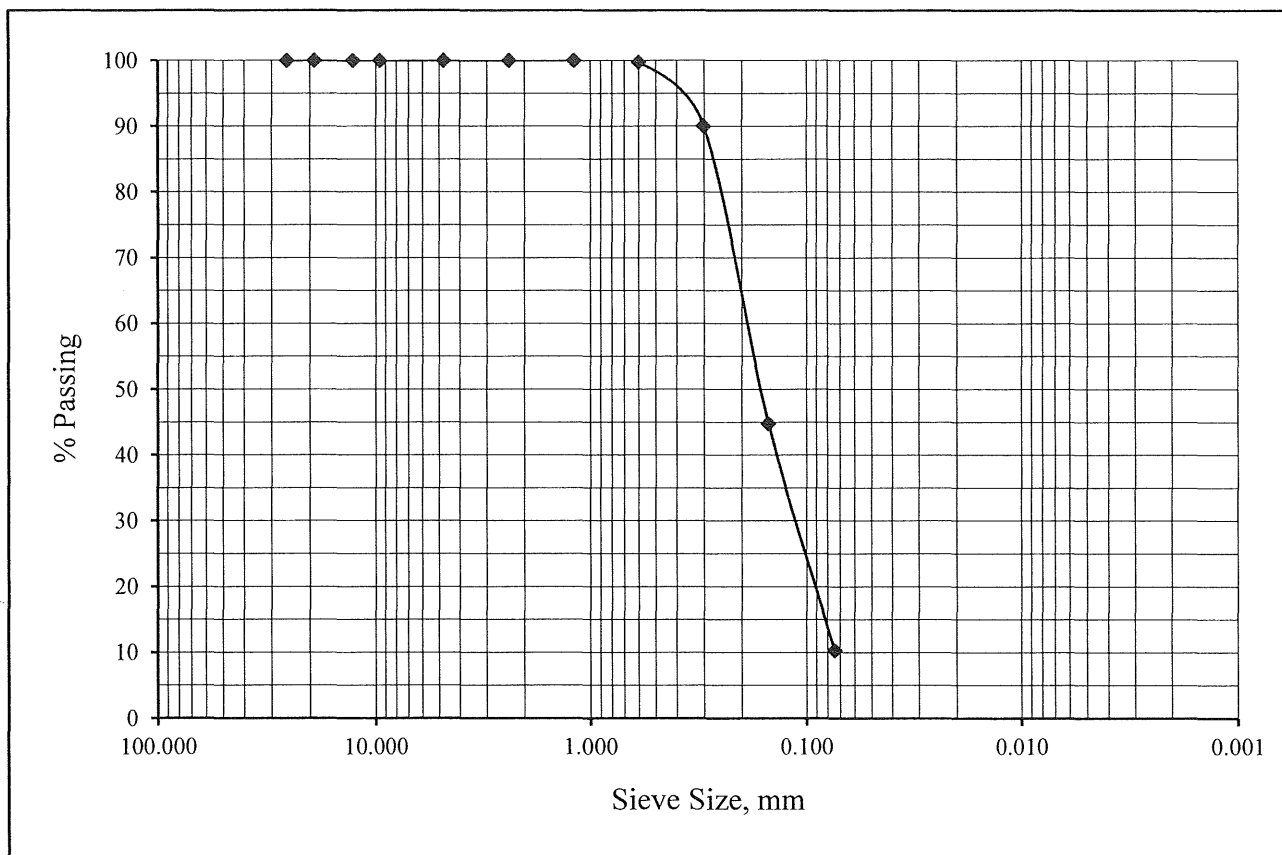
ASTM C117 & C136

Project Number: 544-22319
Project Name: Monroe Street
Lab ID Number: LN6-22370
Sample ID: BH-9 S-3 @ 15'

September 7, 2022

Soil Classification: SP-SM

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	100.0
#8	2.36	100.0
#16	1.18	100.0
#30	0.60	99.7
#50	0.30	90.1
#100	0.15	44.8
#200	0.074	10.4





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One Dimensional Consolidation

ASTM D2435 & D5333

Job Number: 544-22319
Job Name: Monroe Street

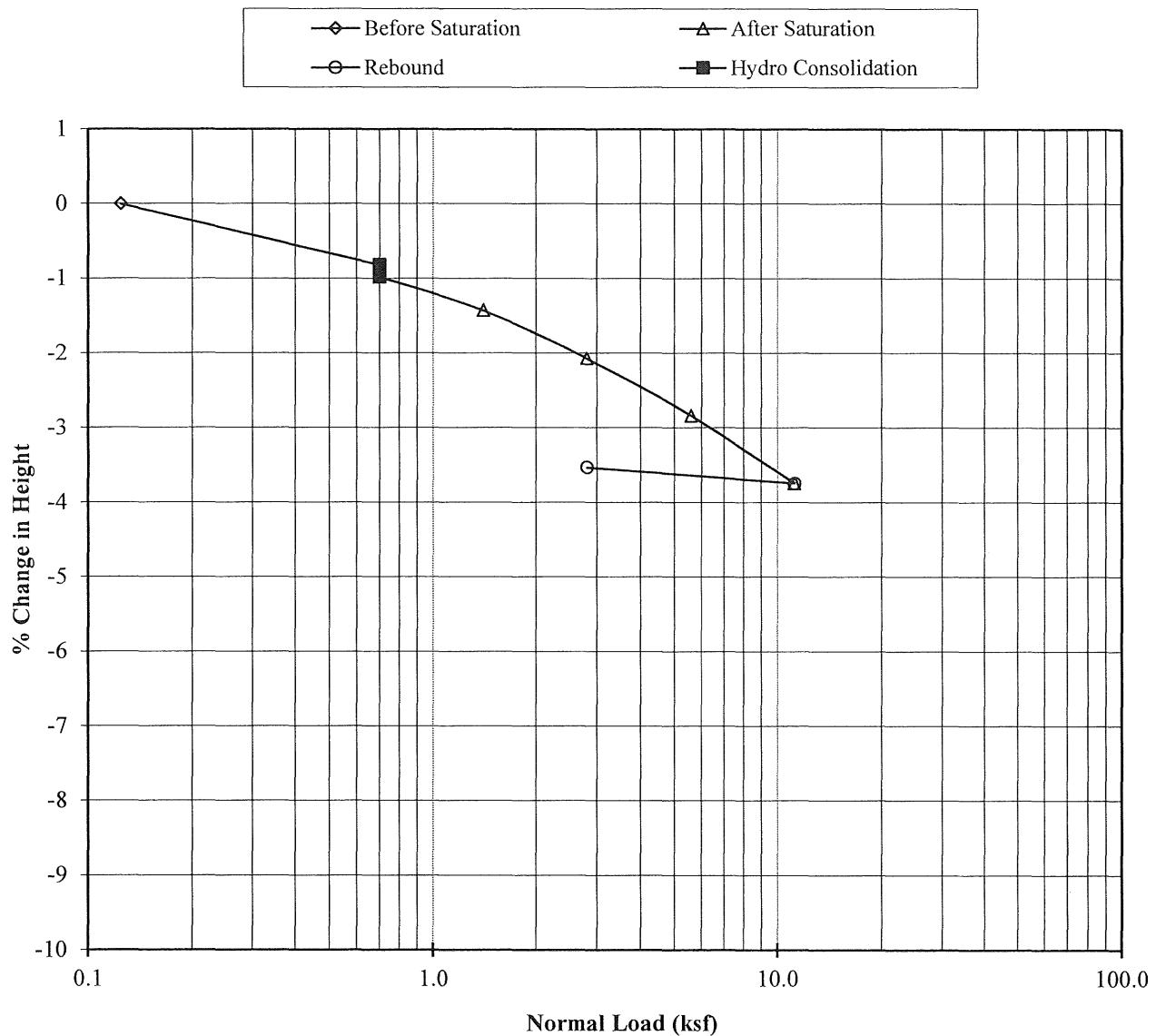
September 7, 2022

Lab ID Number: LN6-22370
Sample ID: BH-1 R-2 @ 5'
Soil Description: Gray Brown Silty Sand (SM)

Initial Dry Density, pcf: 95.7
Initial Moisture, %: 4.0
Initial Void Ratio: 0.742
Specific Gravity: 2.67

Hydrocollapse: 0.2% @ 0.702 ksf

% Change in Height vs Normal Pressure Diagram





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One Dimensional Consolidation

ASTM D2435 & D5333

Job Number: 544-22319
Job Name: Monroe Street

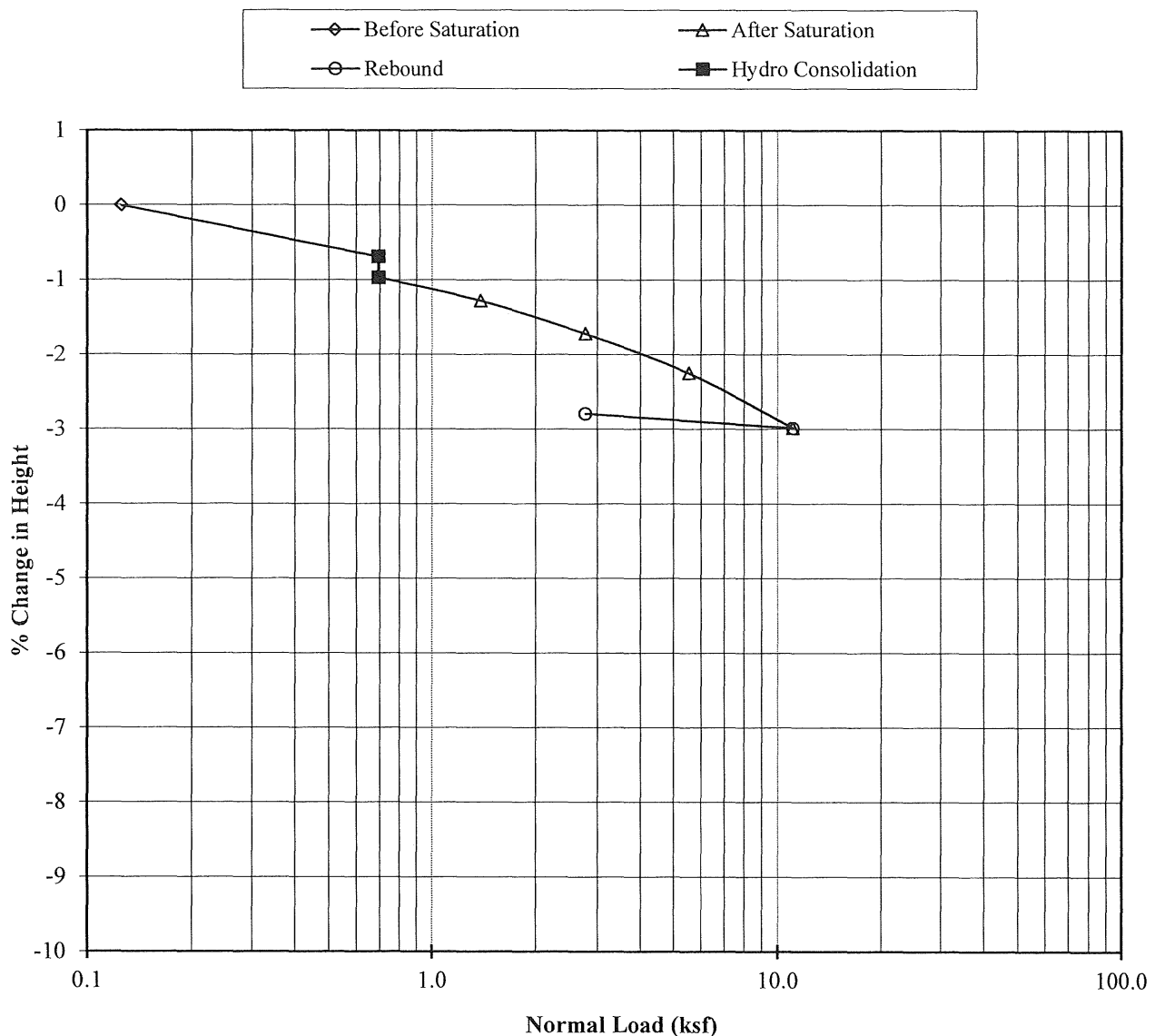
September 7, 2022

Lab ID Number: LN6-22370
Sample ID: BH-2 R-2 @ 10'
Soil Description: Olive Brown Sand w/Silt (SP-SM)

Initial Dry Density, pcf: 98.3
Initial Moisture, %: 4.1
Initial Void Ratio: 0.696
Specific Gravity: 2.67

Hydrocollapse: 0.3% @ 0.694 ksf

% Change in Height vs Normal Pressure Diagram





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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: September 7, 2022

Account No.: 544-22319

Customer: Discovery Land Company

Location: APN's 780-020-001, 002, 003 & 780-010-012, Monroe Street, La Quinta

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'	8.7	80	80	3600
BH-8 @ 0-5'	9.1	40	50	4600

APPENDIX C

SEISMIC DESIGN PARAMETERS SEISMIC DESIGN MAPS

Project: Residential Development
 Project Number: 544-22319
 Client: Discovery Land Company
 Site Lat/Long: 33.6626/ -116.2296
 Controlling Seismic Source: San Andreas

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	0.140
S_{D1}/S_{DS}	T_s	0.699
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}	0.7435*
Equation 11.4-2 - F_v*S_1	S_{M1}	1.1152*

RISK COEFFICIENT

Cr - At Periods ≤ 0.2 , $Cr=C_{RS}$	C_{RS}	0.899
Cr - At Periods ≥ 1.0 , $Cr=C_{R1}$	C_{R1}	0.884

REFERENCE	NOTATION	VALUE
F_v (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	1.595
Design Maps	S_1	0.656
Equation 11.4-1 - F_a*S_s	S_{MS}	1.595*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.063*
Design Maps	PGA	0.697
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - $F_{PGA}*PGA$	PGA_M	0.767*
Section 21.5.3	80% of PGA_M	0.613
Design Maps	C_{RS}	0.899
Design Maps	C_{R1}	0.884

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

Period	Cr
0.200	0.899
0.300	0.897
0.400	0.895
0.500	0.893
0.600	0.892
0.680	0.890
1.000	0.884

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

Mapped values from <https://seismicmaps.org/>



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PROBABILISTIC SPECTRA¹
2% in 50 year Exceedence

Project No: 544-22319

Period	UGHM	RTHM	Max Directional Scale Factor ²	Probabilistic MCE
0.010	0.842	0.799	1.19	0.951
0.100	1.380	1.337	1.19	1.591
0.200	1.838	1.791	1.20	2.149
0.300	2.135	2.002	1.22	2.442
0.500	2.156	1.958	1.23	2.408
0.750	1.793	1.618	1.24	2.006
1.000	1.541	1.369	1.24	1.698
2.000	0.906	0.798	1.24	0.990
3.000	0.624	0.547	1.25	0.684
4.000	0.455	0.397	1.25	0.496
5.000	0.345	0.303	1.26	0.382

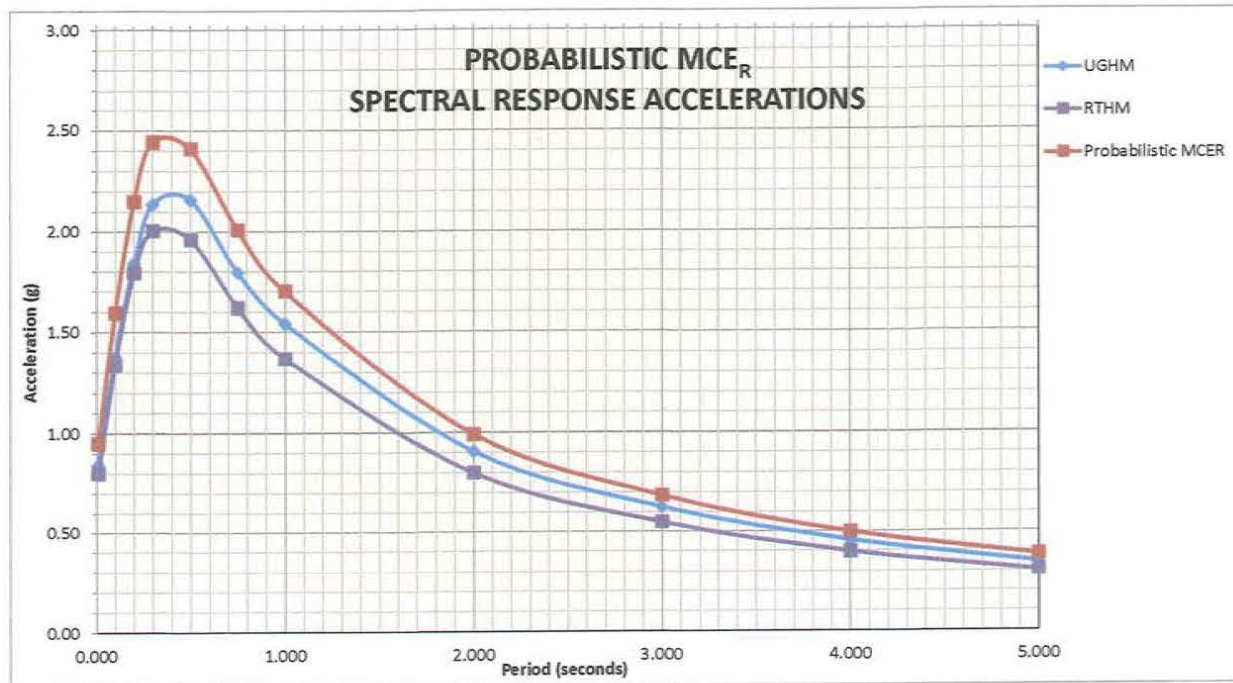
¹ Data Sources:

<https://earthquake.usgs.gov/hazards/interactive/>

<https://earthquake.usgs.gov/designmaps/rtgm/>

² Shahi-Baker RotD100/RotD50 Factors (2014)

Probabilistic PGA: 0.842
Is Probabilistic $S_{a(max)} < 1.2F_a$? **NO**



DETERMINISTIC SPECTRUM

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations¹

Controlling Source: San Andreas

Is Probabilistic $S_{a(max)} < 1.2F_a$?

NO

Project No: 544-22319

Period	Deterministic PS_a Median ± 1.0 for 5% Damping	Max Directional Scale Factor ²	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.689	1.19	0.819	0.819
0.020	0.691	1.19	0.823	0.823
0.030	0.703	1.19	0.836	0.836
0.050	0.746	1.19	0.888	0.888
0.075	0.893	1.19	1.063	1.063
0.100	1.060	1.19	1.262	1.262
0.150	1.304	1.20	1.565	1.565
0.200	1.460	1.20	1.753	1.753
0.250	1.569	1.21	1.898	1.898
0.300	1.626	1.22	1.984	1.984
0.400	1.640	1.23	2.018	2.018
0.500	1.587	1.23	1.953	1.953
0.750	1.289	1.24	1.598	1.598
1.000	1.086	1.24	1.347	1.347
1.500	0.785	1.24	0.973	0.973
2.000	0.601	1.24	0.745	0.745
3.000	0.413	1.25	0.516	0.516
4.000	0.291	1.25	0.364	0.364
5.000	0.217	1.26	0.273	0.273

Is Deterministic $S_{a(max)} < 1.5 * F_a$?

NO

Section 21.2.2 Scaling Factor:

N/A

Deterministic PGA:

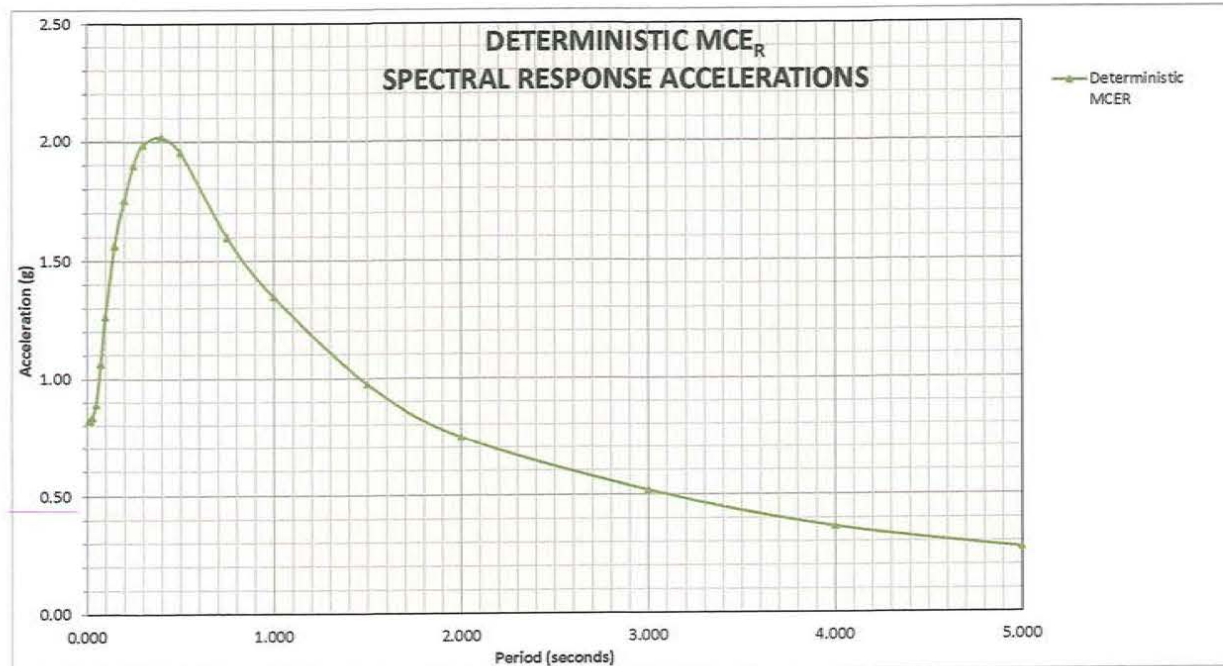
0.689

Is Deterministic PGA $\geq F_{PGA} * 0.5$?

YES

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	0.951	0.819	0.819	0.546
0.100	1.591	1.262	1.262	0.841
0.200	2.149	1.753	1.753	1.168
0.300	2.442	1.984	1.984	1.323
0.500	2.408	1.953	1.953	1.302
0.750	2.006	1.598	1.598	1.066
1.000	1.698	1.347	1.347	0.898
2.000	0.990	0.745	0.745	0.496
3.000	0.684	0.516	0.516	0.344
4.000	0.496	0.364	0.364	0.243
5.000	0.382	0.273	0.273	0.182

ASCE 7-16: Section 21.4

Site Specific

	Calculated Value	Design Value
SDS:	1.190	1.190
SD1:	1.033	1.033
SMS:	1.786	1.786
SM1:	1.549	1.549
Site Specific PGAm:	0.689	0.689
Site Class:	D measured	

Seismic Design Category - Short*

D

Seismic Design Category - 1s*

D

* Risk Categories I, II, or III

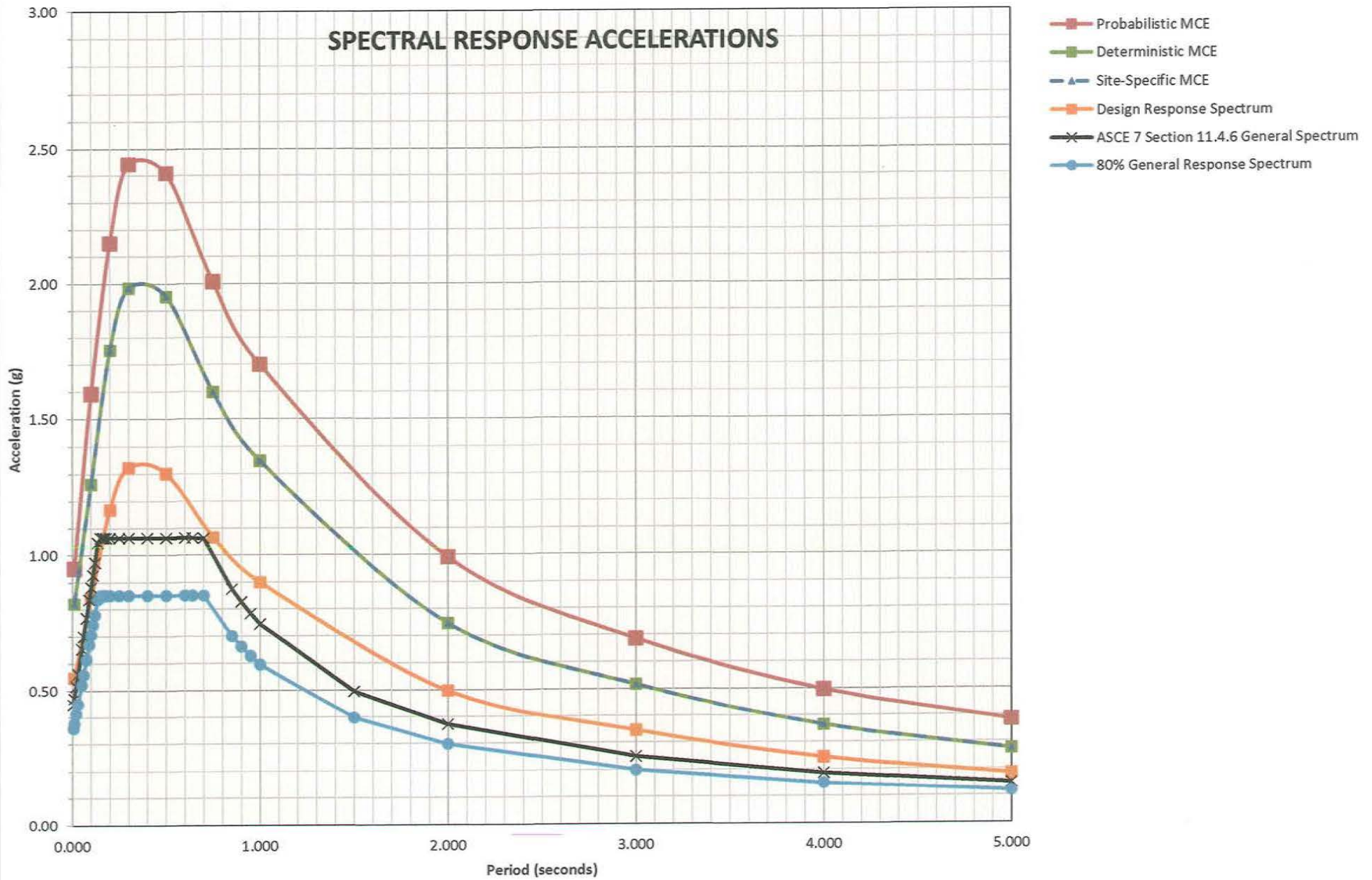
Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.448	0.359
0.010	0.471	0.377
0.020	0.517	0.413
0.030	0.562	0.450
0.050	0.653	0.523
0.060	0.699	0.559
0.075	0.768	0.614
0.090	0.836	0.669
0.100	0.882	0.705
0.110	0.927	0.742
0.120	0.973	0.778
0.136	1.046	0.837
0.150	1.063	0.851
0.160	1.063	0.851
0.170	1.063	0.851
0.180	1.063	0.851
0.200	1.063	0.851
0.250	1.063	0.851
0.300	1.063	0.851
0.400	1.063	0.851
0.500	1.063	0.851
0.600	1.063	0.851
0.640	1.063	0.851
0.700	1.062	0.850
0.850	0.875	0.700
0.900	0.826	0.661
0.950	0.783	0.626
1.000	0.743	0.595
1.500	0.496	0.397
2.000	0.372	0.297
3.000	0.248	0.198
4.000	0.186	0.149
5.000	0.149	0.119

Project No: 544-22319



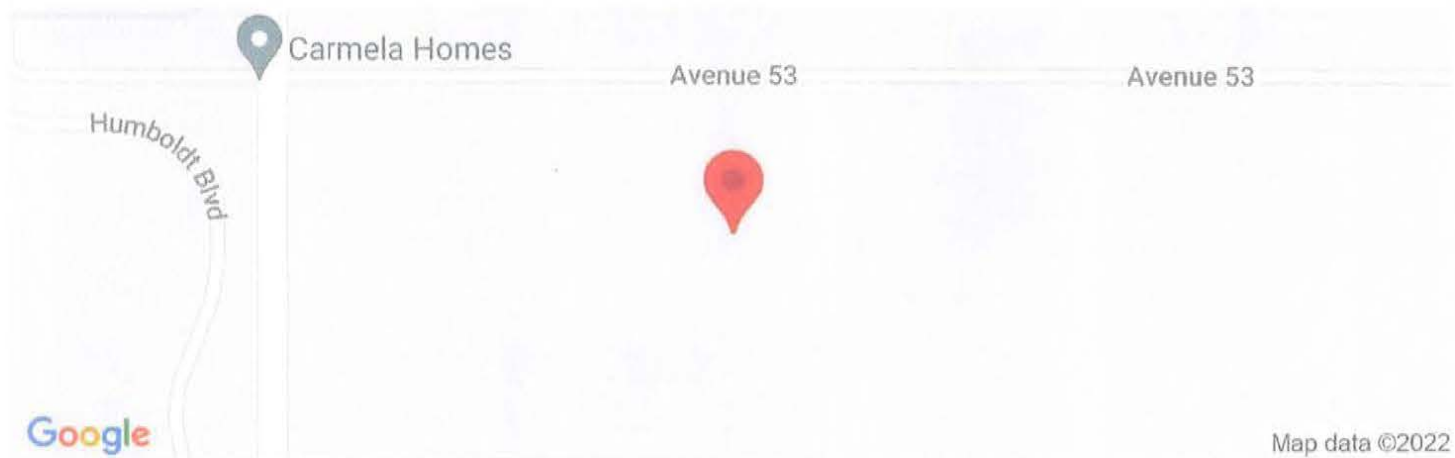
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SPECTRAL RESPONSE ACCELERATIONS





Latitude, Longitude: 33.6626, -116.2296



Map data ©2022

Date	10/17/2022, 10:55:02 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.595	MCE_R ground motion. (for 0.2 second period)
S_1	0.656	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.595	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.063	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	0.697	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.767	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	1.888	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	2.1	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	1.595	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.74	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.836	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.656	Factored deterministic acceleration value. (1.0 second)
PGA_d	0.697	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.83	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.899	Mapped value of the risk coefficient at short periods
C_{R1}	0.884	Mapped value of the risk coefficient at a period of 1 s
C_v	1.419	Vertical coefficient

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