APPENDIX G

Geotechnical Investigation

Proposed Residential Development
Avenue 50 east of Jefferson Street
APNs: 602-070-004, 602-080-001 & 602-080-002
Indio, California
November 20, 2019

and

Infiltration Testing for On-Site Storm Water Retention
November 8, 2019

Prepared for

Kraemer Land Company Inc.
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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET
APNS 602-070-004, 602-080-001 &
602-080-002
INDIO, CALIFORNIA

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November 20, 2019

Project No. 544-19317
19-10-496

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Subject: Geotechnical Investigation

Project: Proposed Residential Development
Avenue 50 east of Jefferson Street
APNs: 602-070-004, 602-080-001 & 602-080-002
Indio, California

Sladden Engineering is pleased to present the results of our geotechnical investigation performed for the residential development proposed for the vacant property located on the north side Avenue 50 just east of Jefferson Street (APNs: 602-070-0047, 602-080-001 & 602-080-002) in the City of Indio, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated September 17, 2019 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.

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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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET
APNS 602-070-004, 602-080-001 & 602-080-002
INDIO, CALIFORNIA

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INTRODUCTION

This report presents the results of our geotechnical investigation performed by Sladden Engineering (Sladden) for the residential development proposed for the vacant property located on the north side of Avenue 50 just east of Jefferson Street (APNs: 602-070-0047, 602-080-001 & 602-080-002) in the City of Indio, California. The site is located at approximately 33.6886 degrees north latitude and 116.2580 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

It is our understanding that the proposed project will consist of constructing a new residential development on the currently vacant property. The proposed project will also include paved roadways, concrete flatwork, underground utilities, retention basins, recreational park and various associated site improvements. For our analyses, we expect that the proposed residences will consist of relatively lightweight wood-frame structures 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 surface soil and primary foundation bearing soil within the proposed residential 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 commercial structures, we expect that isolated column loads will be less than 30 kips and continuous wall loads will be less than 3.0 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 supplemental 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 six (6) exploratory boreholes to depths of approximately 15 to 51 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.
- Advancing six (6) exploratory boreholes to depths of approximately 15 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 potential geologic hazards that may impact development.
- 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 proposed project is located on the north side of Avenue 50 just east of Jefferson Street in the City of Indio, California. The property is formally identified by the County of Riverside as APNs 602-070-0047, 602-080-001 & 602-080-002 and occupies approximately 44.5-acres of land. The property is vacant and appeared to have been previously leveled for agricultural use. Various north to south and east to west trending dirt access roads are present throughout the site. A sand dune is located near the central portion along the western property boundary. The site is bounded on the north, east and west by residential property and on the south by Avenue 50. The adjacent roadways are paved and near the elevation of the subject property. Topographic relief across the site is generally level with no discernible surface gradients.

Based on our review of the La Quinta 7.5-Minute Quadrangle Map (USGS, 2012), the site is situated at an approximate elevation of 30 feet above mean sea level (MSL).

No natural ponding of water or surface seeps were observed at or near the site during our investigation conducted on October 18, 2019. Site drainage appears to be controlled via sheet flow and surface infiltration. Regional drainage is provided by the Whitewater River that is located approximately 1 ½ mile to the north of the property.

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 – Orocopia 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 lacustrian (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 undifferentiated Quaternary-age lacustrine deposits and alluvium (Ql-Qal). The regional geologic setting for the site vicinity is presented on the Regional Geologic Map (Figure 2).

SUBSURFACE CONDITIONS

Our investigation of the site consisted of drilling six (6) exploratory boreholes to depths of approximately 15 to 51 feet bgs. The approximate locations of the boreholes are illustrated on the Borehole 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, a thin mantle of artificial fill/disturbed soil generally less than approximately three (3) feet in depth was encountered within each of the boreholes. Alluvium was encountered below the fill/disturbed soil and consisted primarily of silty sand (SM) with interbbeded layers of sand (SP). Generally, granular horizons appeared grayish brown, dry to slightly moist, medium dense to dense and fine-grained. A clay layer was encountered within BH-3.

The final logs represent our interpretation of the contents of the field logs along with 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.

Groundwater was not encountered to a maximum explored depth of approximately 51 feet bgs. As such, it is our opinion that groundwater should not be a factor during the 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 Ridgecrest M7.1 (2019).

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). 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 Andreas – Coachella	8.7	7.2
San Andreas – Southern	8.7	7.2
Burnt Mountain	30.6	6.5
Eureka Peak	31.8	6.4
San Andreas – San Bernardino	33.1	7.5
San Jacinto – Anza	33.6	7.2
San Jacinto – Coyote Creek	34.6	6.8
Pinto Mountain	51.1	7.2

2016 CBC SEISMIC DESIGN PARAMETERS

Sladden has reviewed the 2016 California Building Code (CBC) and summarized the current seismic design parameters for the proposed structures. The seismic design category for a structure may be determined in accordance with Section 1613 of the 2016 CBC or ASCE7. According to the 2016 CBC, Site Class D may be used to estimate design seismic loading for the proposed structures. The 2016 CBC Seismic Design Parameters are summarized below (USGS, 2018a). The project Design Map Reports are included within Appendix C.

Risk Category (Table 1.5-1): II
Site Class (Table 1613.3.2): D
S_s (Figure 1613.3.1): 1.579g
S_1 (Figure 1613.3.1): 0.747 g
F_a (Table 1613.3.3(1)): 1.0
F_v (Table 1613.5.3(2)): 1.5
S_{ms} (Equation 16-37 {F_a X S_s}): 1.579g
S_{m1} (Equation 16-38 {F_v X S_1}): 1.121g
S_{DS} (Equation 16-39 {2/3 X S_{ms}}): 1.053g
S_{D1} (Equation 16-40 {2/3 X S_{m1}}): 0.747g
Seismic Design Category: D

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. The subsequent discussions address potential geologic hazards and their relationship to the project site.

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), CDOC (2019), and RCPR (2019), known faults are not mapped on or projecting towards 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, 2019). Finally, no signs of active surface fault 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 "negligible".

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 probabilistic approach was employed to the estimate the peak ground acceleration (a_{max}) that could be experienced at the site. Based on the USGS Unified Hazard Tool (USGS, 2019) and shear wave velocity (Vs30) of 259 m/s, the site could be subjected to ground motions on the order of 0.49g. The peak ground acceleration at the site is judged to have a 475 year return period and a 10 percent chance of exceedance in 50 years.

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.

According to the County of Riverside, the site is situated within a "Low to Moderate" liquefaction potential zone (RCPR, 2019). Based on our review of groundwater maps of the site vicinity (>50 feet bgs; Tyley, 1974), risks associated with liquefaction and liquefaction related hazards should be 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, risk associated with tsunamis and seiches is considered negligible.

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

VI. Expansive Soil. Generally, the site near surface soil consists of silty sand (SM). Based on the results of our laboratory testing (EI=1), the surface materials underlying the site are considered to have a "very low" expansion potential.

VII. Settlement. Static settlement resulting from the anticipated foundation loads should be minimal provided that the recommendations included in this report are considered in foundation design and construction. The estimated ultimate static settlement is calculated to be approximately 1 inch when using the recommended bearing pressures. As a practical matter, differential static settlement between footings can be assumed as one-half of the total settlement over a horizontal distance of approximately 50 feet.

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.

Although recent investigations have documented significant subsidence within the Coachella Valley area (USGS, 2007), no fissures or other surficial evidence of subsidence were observed during our field investigations or during our review of black and white stereo-photo pairs. With the exception of isolated tension zones typically manifested on the ground surface as fissures and/or ground cracks, subsidence related to groundwater depletion is generally areal in nature with very little differential settlement over short distances such as across individual buildings.

The Coachella Valley Water District has publically acknowledged regional subsidence throughout the southern portion of the Coachella Valley and has indicated a commitment to groundwater replenishment programs that are intended to limit future subsidence. At this time, subsidence is considered a regional problem requiring regional mitigation not specific to the project vicinity.

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). 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 remote.

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 loose artificial fill and potentially compressible near surface native soil.

We recommend that remedial grading work within the proposed residential building areas include over-excavation and re-compaction of the primary foundation bearing soil. 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. On the basis of 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 on the basis of 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. Stripping: Areas to be graded and paved should be cleared of any existing improvements, foundation elements, vegetation, root systems 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 Building Areas: In order to achieve firm and uniform 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 3 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally, a minimum of five feet beyond the footing limits. The soil exposed by over-excavation should then be scarified, moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction. The previously removed soil may then be replaced as engineered fill soil as recommended in the following section.

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. 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 excavations 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 2
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Removal and recompaction of all artificial fill soil and loose native soil to depths of at least 3 feet below existing grade or 3 feet below the bottom of footings, whichever is deeper. Removals should extend laterally a minimum of 5 feet beyond the footing limits.
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 within 2 percent of the optimum moisture content.
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. <u>Shrinkage and Subsidence</u>: 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 15 and 20 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 tenth 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.

CONVENTIONAL SHALLOW SPREAD FOOTINGS

Conventional spread footings are expected to provide adequate support for the proposed new residential buildings. 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 engineered fill compacted 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 2500 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 50 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 0.40 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. Excavated soil generated from footing and/or utility trenches should not be stockpiled within the building envelope or in areas of exterior concrete flatwork. 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 subgrades 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 bar at 24 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. 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.

PRELIMINARY PAVEMENT DESIGN

Asphalt concrete pavements should be designed in accordance with Topic 608 of the Caltrans Highway Design Manual based on R-Value and Traffic Index. An R-Value of 63 was determined to develop the on-site roadways. On-site and any imported soil should be tested for R-Value after grading. For pavement design, a Traffic Index (TI) of 6.0 was used for light-duty pavement. We assumed Asphalt Concrete (AC) over Class II Aggregate Base (AB). The preliminary flexible pavement design is as follows:

TABLE 3
SUMMARY OF RECOMMENDATIONS

RECOMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS	
Pavement Material	**Recommended Thickness**
	Light-Duty (TI = 6.0)
Asphalt Concrete Surface Course	3.0 inches
Class II Aggregate Base Course	4.5 inches
Compacted Subgrade Soil	12.0 inches

Asphalt concrete should conform to Sections 203 and 302 of the latest edition of the Standard Specifications for Public Works Construction ("Greenbook") or Caltrans. Class II aggregate base should conform to Section 26 of the Caltrans Standard Specifications or Greenbook, latest edition. The aggregate base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Method D 1557.

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 240 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 should not be necessary. 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 levels of the surface soil was 9.2. Based on soluble chloride concentration testing (110 ppm) the soil is considered to have a "negligible" corrosion potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 3,100 ohm-cm that suggests the site soil is considered to have a "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 relative compaction of 90 percent. 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 conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project soil engineer 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 relative compaction of 90 percent. 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 operations 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 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.

REFERENCES

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FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PLAN

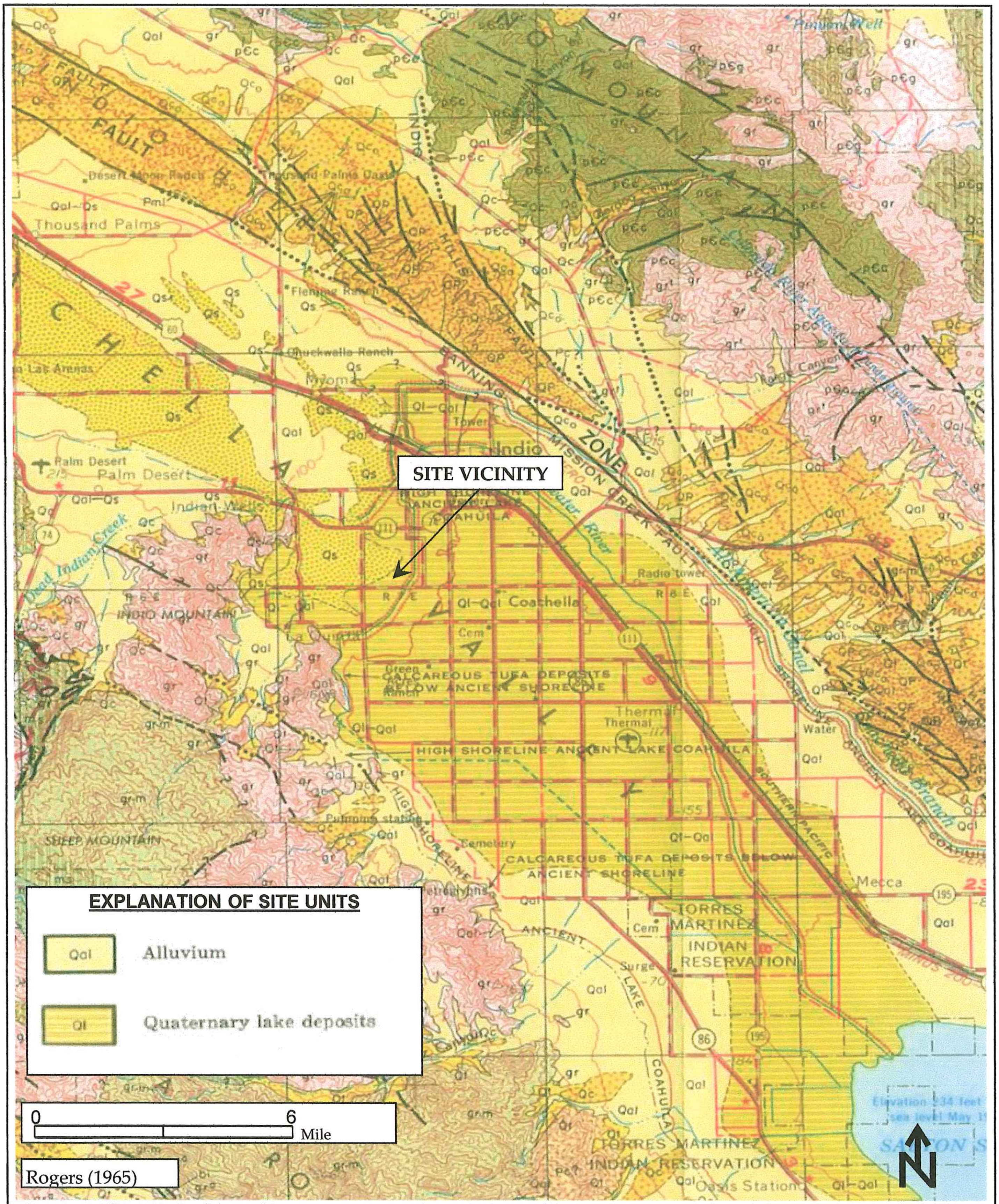
Sladden Engineering

SITE LOCATION MAP

Project Number:	544-19317
Report Number:	19-10-496
Date:	November 20, 2019

FIGURE

1



Sladden Engineering

REGIONAL GEOLOGIC MAP

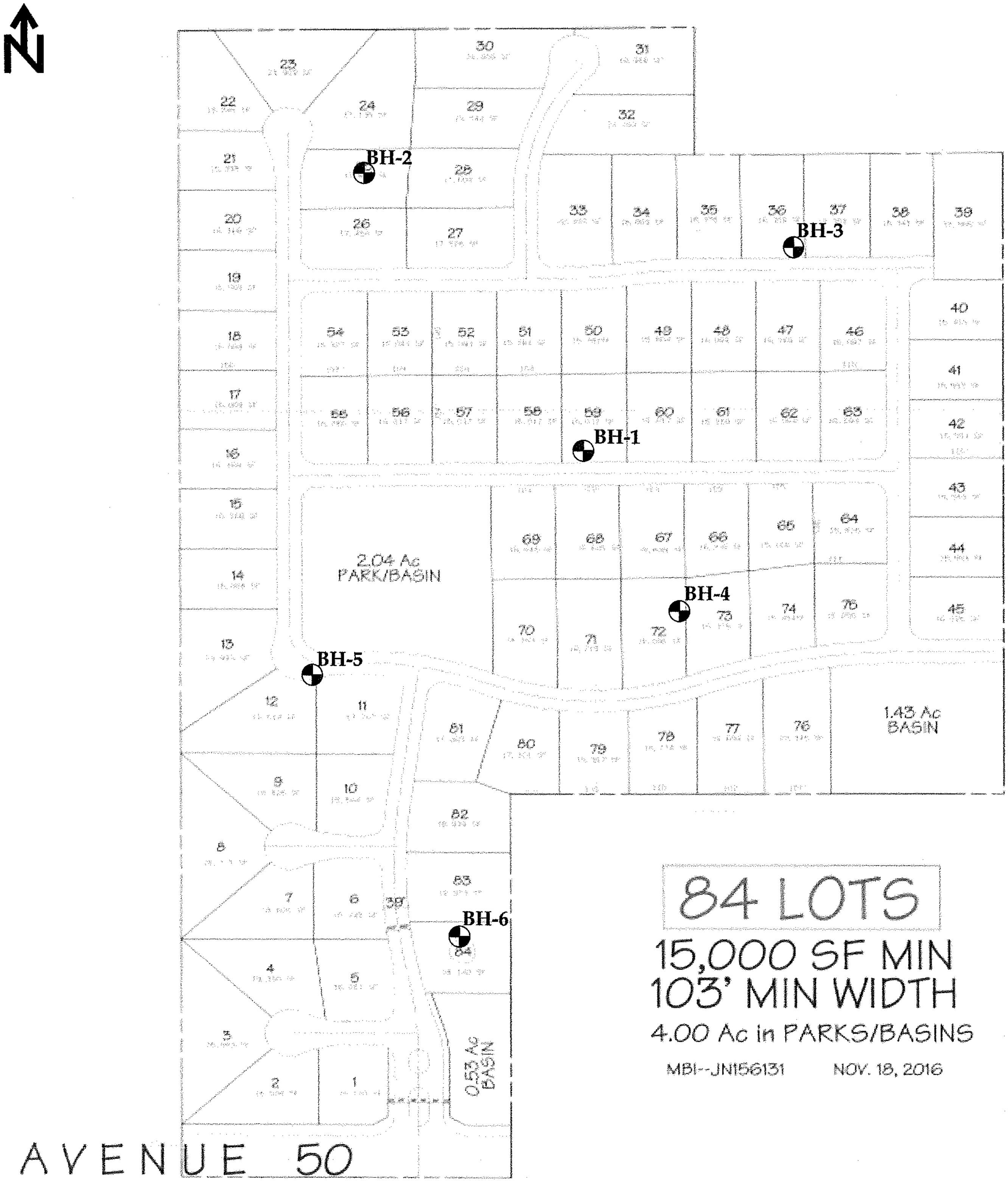
FIGURE

2

Project Number: 544-19317

Report Number: 19-10-496

Date: November 20, 2019



BOREHOLE LOCATION PLAN

Project Number:	544-19317
Report Number:	19-10-496
Date:	November 20, 2019

FIGURE

3

Sladden Engineering

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our supplemental field investigation, six (6) exploratory bores were excavated on October 8, 2019 utilizing a Mobile B-61 truck mounted hollow stem auger rig. 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 that 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.

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 Ft (MSL)	Boring No:	BH-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
■	8/9/11	1	1	13.1	2.3	113.9	2 4		Silty Sand (SM); grayish brown, dry to slightly moist, medium dense, fine-grained, micaceous (Fill/Disturbed).
■	4/8/9			7.7	1.6	101.6	6 8		Sand (SP); grayish brown, dry to slightly moist, medium dense, fine- to coarse-grained, micaceous (Qal-Ql).
▮	4/6/7			28.8	7.5		10 12 14		Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
■	5/8/10			26.9	6.1	92.4	16 18		Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
▮	4/6/7			32.7	8.2		20 22 24		Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
■	11/29/35			15.4	4.9	117.3	26 28		Silty Sand (SM); grayish brown, slightly moist, dense, fine-grained, micaceous (Qal-Ql).
▮	10/14/21			8.8	3.6		30 32 34		Sand (SP); grayish brown, slightly moist, dense, fine- to coarse-grained, micaceous (Qal-Ql).
■	16/24/36			10.3	3.0	108.8	36 38		Sand (SP); grayish brown, slightly moist, dense, fine- to coarse-grained, micaceous (Qal-Ql).
▮	8/14/20			23.4	6.8		40 42 44		Silty Sand (SM); grayish brown, slightly moist, dense, fine-grained, micaceous (Qal-Ql).
■	10/13/20			53.5	11.2	105.4	46 48		Clayey Silt (ML); grayish brown, moist, very stiff, low plasticity (Ql-Qal).
▮	10/13/18			27.0	5.1		50		Silty Sand (SM); grayish brown, slightly moist, dense, fine-grained, micaceous (Qal-Ql).

Completion Notes:
Terminated at ~51.5 Feet bgs.
No Bedrock Encountered.
No Groundwater or Seepage Encountered.

PROPOSED RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317
Report No: 19-10-496

Page 1

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 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, dry to slightly moist, fine-grained, micaceous (Fill/Disturbed).
◫	3/5/8			16.8	6.7		4 6 8	▒	Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
■	4/6/9			36.0	9.8	95.8	10 12 14	▒	Silty Sand (SM); grayish brown, slightly moist, loose, fine-grained, micaceous (Qal-Ql).
◫	3/4/6			43.8	13.3		16 18	▒	Silty Sand (SM); grayish brown, moist, loose, fine-grained, micaceous (Qal-Ql).
■	5/8/10			33.7	11.4	94.3	20 22	▒	Silty Sand (SM); grayish brown, slightly moist to moist, medium dense, fine-grained, micaceous (Qal-Ql).
							24 26 28 30 32 34 36 38 40 42 44 46 48 50		Terminated at ~21.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Completion Notes:

PROPOSED RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317
Report No: 19-10-496

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 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, dry to slightly moist, fine-grained, micaceous (Fill/Disturbed).
■	2/2/4			93.3	38.2	81.9	4 6 8		Clay (CH); olive brown, very moist to wet, soft, high plasticity (Ql-Qal).
▮	4/5/6			7.3	5.0		10 12 14		Sand (SM); grayish brown, slightly moist, medium dense, fine- to coarse-grained, micaceous (Qal-Ql).
■	8/12/12			3.1	2.4	100.3	16 18		Sand (SM); grayish brown, dry to slightly moist, medium dense, fine- to coarse-grained, micaceous (Qal-Ql).
▮	5/7/8			28.2	9.4		20 22		Silty Sand (SM); grayish brown, slightly moist to moist, medium dense, fine-grained, micaceous (Qal-Ql).
							24 26		Terminated at ~21.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.
							28 30 32 34 36 38 40 42 44 46 48 50		

Completion Notes:

PROPOSED RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317
Report No: 19-10-496

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 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, dry to slightly moist, fine-grained, micaceous (Fill/Disturbed).
■	2/3/3			24.2	8.0		4 6 8	▨	Silty Sand (SM); grayish brown, slightly moist, loose, fine-grained, micaceous (Qal-Ql).
■	8/12/15			5.8	2.2	102.6	10 12 14	▨	Sand (SM); grayish brown, slightly moist, medium dense, fine- to coarse-grained, micaceous (Qal-Ql).
■	4/6/9			46.4	9.1		16	▨	Silty Sand (SM); grayish brown, moist, medium dense, fine-grained, micaceous (Qal-Ql).
							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 RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317	Page	4
Report No: 19-10-496		

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 Ft (MSL)	Boring No:	BH-5

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2	▨	Silty Sand (SM); grayish brown, dry to slightly moist, fine-grained, micaceous (Fill/Disturbed).
■	8/9/11			15.3	0.4	99.9	4 6 8	▨	Silty Sand (SM); grayish brown, dry, medium dense, fine-grained, micaceous (Qal-Ql).
▣	6/10/11			12.2	4.6	102.6	10 12 14	▨	Silty Sand (SM); grayish brown, slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
■	6/9/13			22.1	5.0	96.8	16	▨	Silty Sand (SM); grayish brown, moist, medium dense, fine-grained, micaceous (Qal-Ql).
							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 RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317
Report No: 19-10-496

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	10/18/2019
Elevation:	25 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, dry to slightly moist, fine-grained, micaceous (Fill/Disturbed).
▮	4/5/6			19.4	2.0		4 6 8	▨	Silty Sand (SM); grayish brown, dry to slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
■	6/9/10			34.0	1.2	104.1	10 12 14	▨	Silty Sand (SM); grayish brown, dry, medium dense, fine-grained, micaceous (Qal-Ql).
▮	5/7/10			20.0	2.5		16	▨	Silty Sand (SM); grayish brown, dry to slightly moist, medium dense, fine-grained, micaceous (Qal-Ql).
							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 RESIDENTIAL DEVELOPMENT
AVENUE 50 EAST OF JEFFERSON STREET, INDIO

Project No: 544-19317
Report No: 19-10-496

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.

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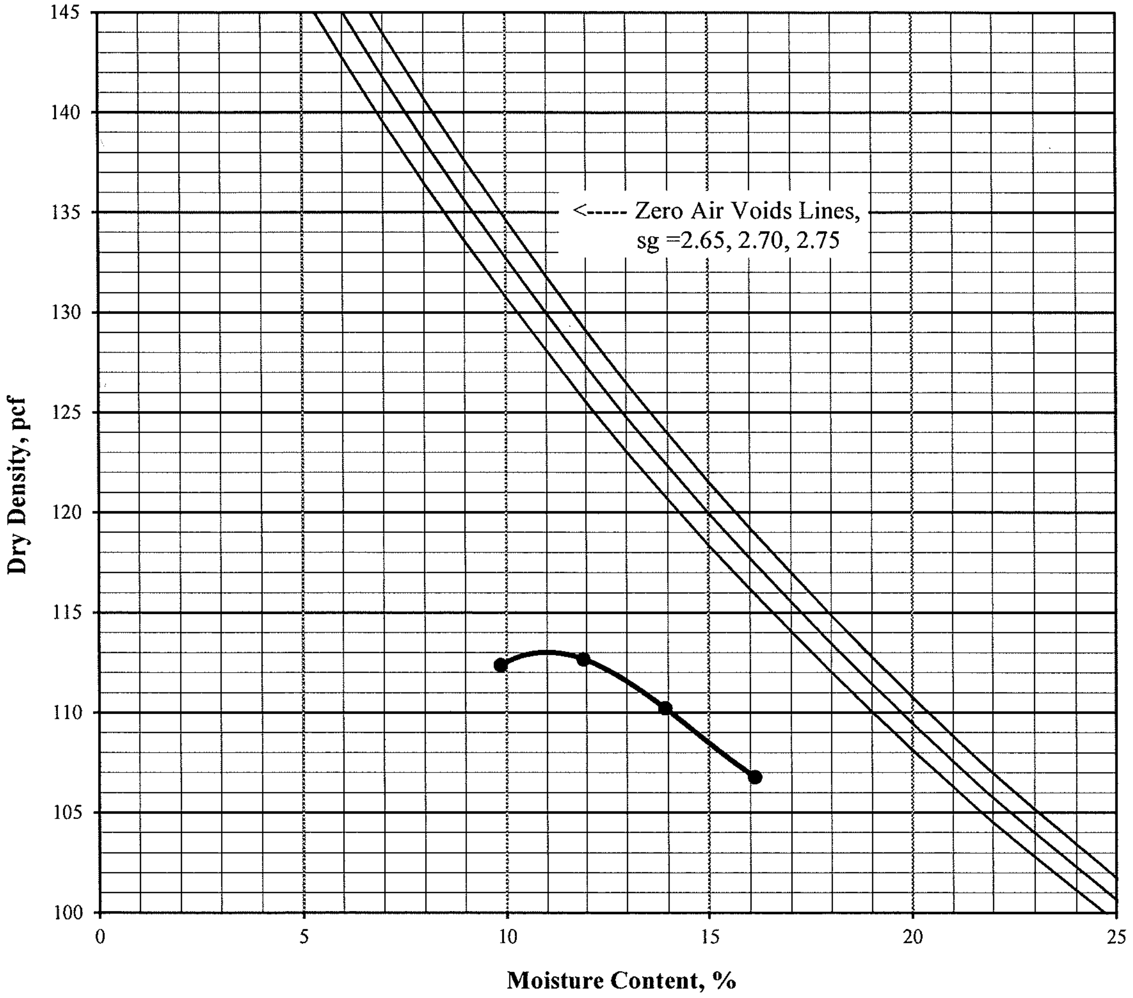
Maximum Density/Optimum Moisture

ASTM D698/D1557

Project Number:	544-19317	November 20, 2019
Project Name:	Avenue 50	
Lab ID Number:	LN6-19553	ASTM D-1557 A
Sample Location:	BH-1 Bulk 1 @ 0-5'	Rammer Type: Machine
Description:	Olive Brown Silty Sand (SM)	

Maximum Density: **113 pcf**
Optimum Moisture: **12%**

Sieve Size	% Retained
3/4"	
3/8"	
#4	0.1



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Expansion Index

ASTM D 4829

November 20, 2019

Job Number:	544-19317
Job Name:	Avenue 50
Lab ID Number:	LN6-19553
Sample ID:	BH-1 Bulk 1 @ 0-5'
Soil Description:	Olive Brown Silty Sand (SM)

Wt of Soil + Ring:	556.5
Weight of Ring:	191.2
Wt of Wet Soil:	365.3
Percent Moisture:	11.1%
Sample Height, in	0.95
Wet Density, pcf:	116.5
Dry Denstiy, pcf:	104.9

% Saturation:	49.4

Expansion **Rack # 2**

Date/Time	11/15/2019	3:35 PM
Initial Reading	0.0000	
Final Reading	0.0007	

Expansion Index **1**

(Final - Initial) x 1000

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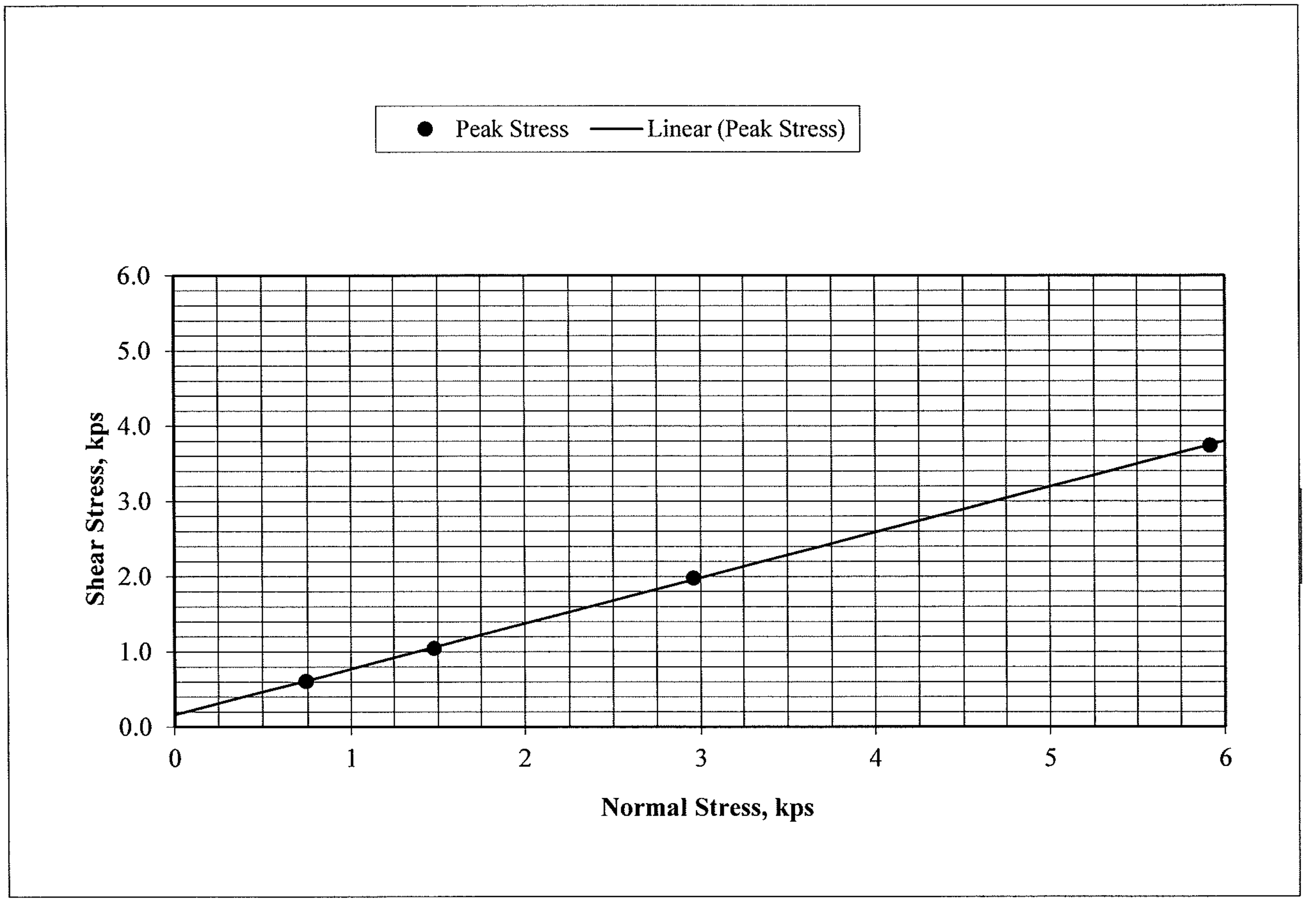
Direct Shear ASTM D 3080-04
(modified for unconsolidated condition)

Job Number: 544-19317
Job Name Avenue 50
Lab ID No. LN6-19553
Sample ID BH-1 Bulk 1 @ 0-5'
Classification Olive Brown Silty Sand (SM)
Sample Type Remolded @ 90% of Maximum Density

November 20, 2019
Initial Dry Density: 101.6 pcf
Initial Mosture Content: 12.0 %
Peak Friction Angle (Ø): 31°
Cohesion (c): 160 psf

Test Results	**1**	**2**	**3**	**4**	**Average**
Moisture Content, %	21.2	21.2	21.2	21.2	21.2
Saturation, %	86.8	86.8	86.8	86.8	86.8
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.609	1.044	1.979	3.741	



Job Number: 544-19317
Job Name: Avenue 50
Date: 11/20/2019

Moisture Adjustment		Remolded Shear Weight	
Wt of Soil:	1,000	Max Dry Density:	113.0
Moist As Is:	1.7	Optimum Moisture:	12.0
Moist Wanted:	12.0		
ml of Water to Add:	101.3	Wt Soil per Ring, g:	137.0

UBC

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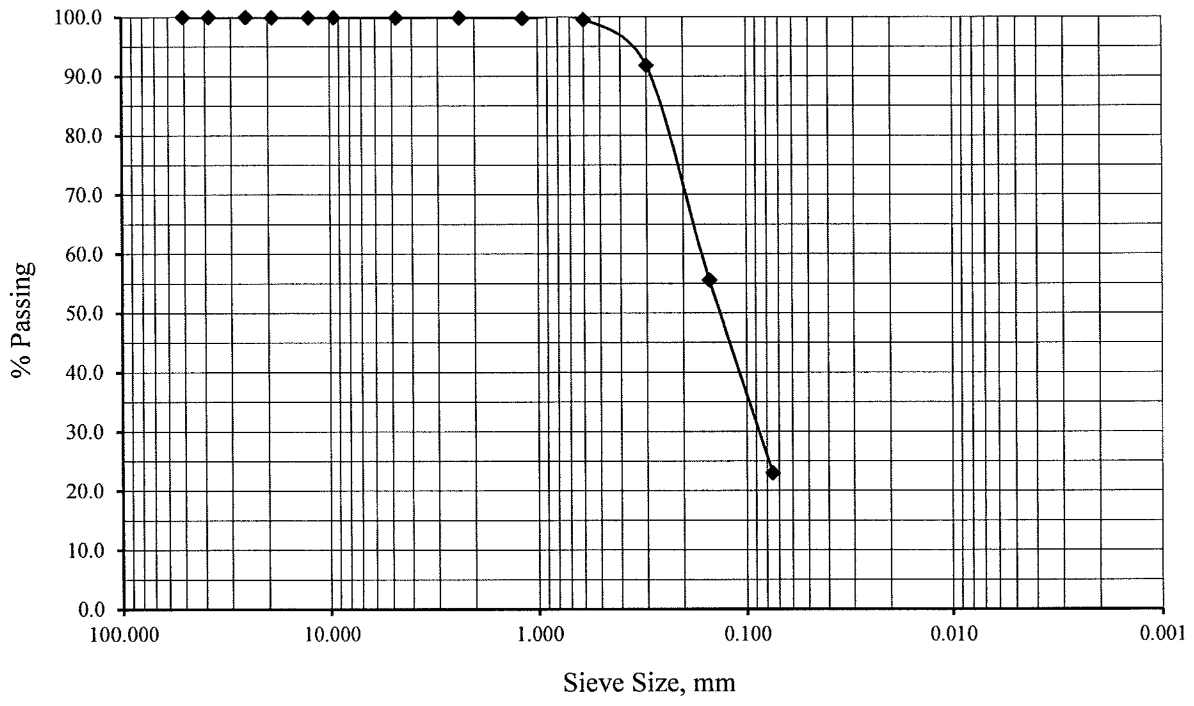
Gradation

ASTM C117 & C136

Project Number:	544-19317	November 20, 2019
Project Name:	Avenue 50	
Lab ID Number:	LN6-19553	
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	99.9
1/2"	12.7	99.9
3/8"	9.53	99.9
#4	4.75	99.9
#8	2.36	99.9
#16	1.18	99.8
#30	0.60	99.5
#50	0.30	91.8
#100	0.15	55.6
#200	0.075	23.0



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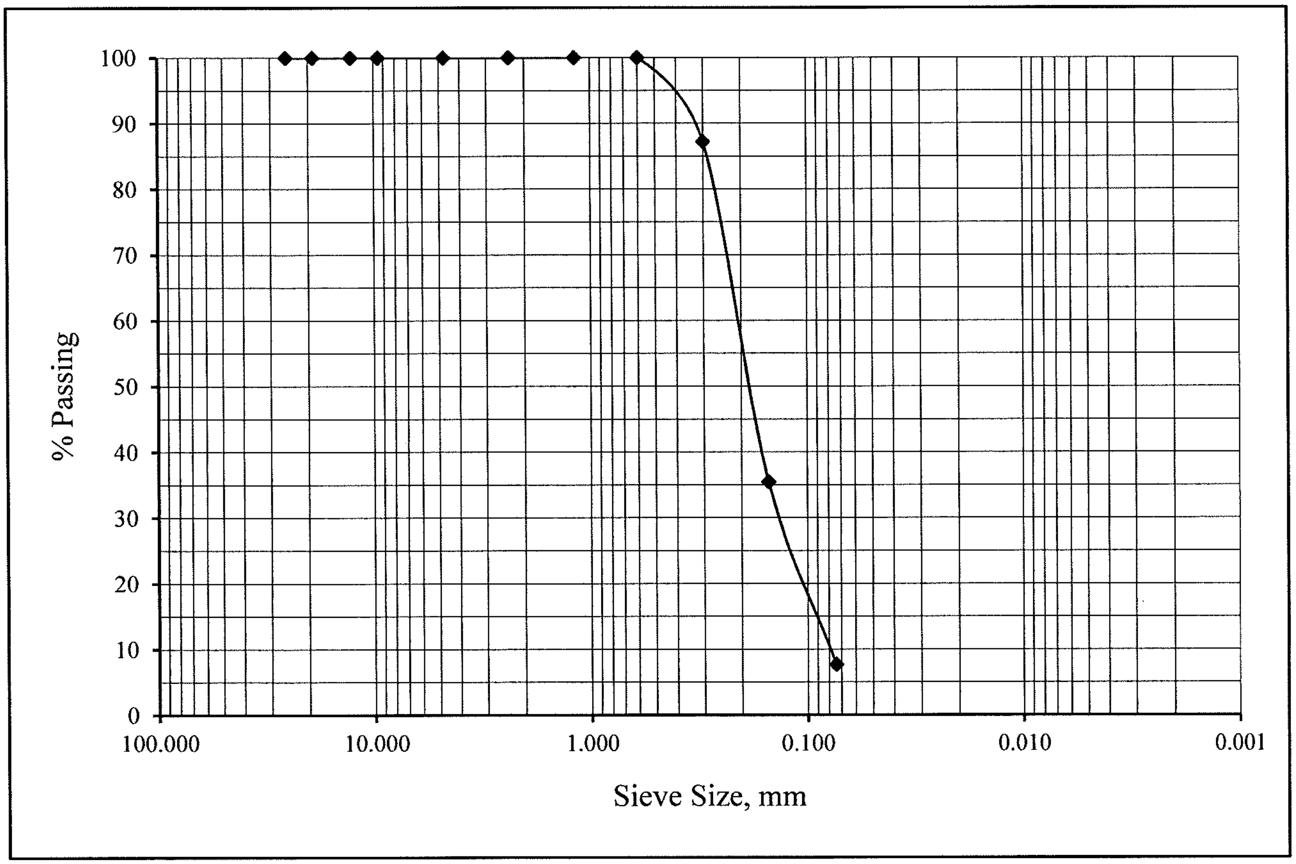
Gradation

ASTM C117 & C136

Project Number: 544-19317 November 20, 2019
Project Name: Avenue 50
Lab ID Number: LN6-19553
Sample ID: BH-1 R-2 @ 5' 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	87.2
#100	0.15	35.5
#200	0.074	7.7

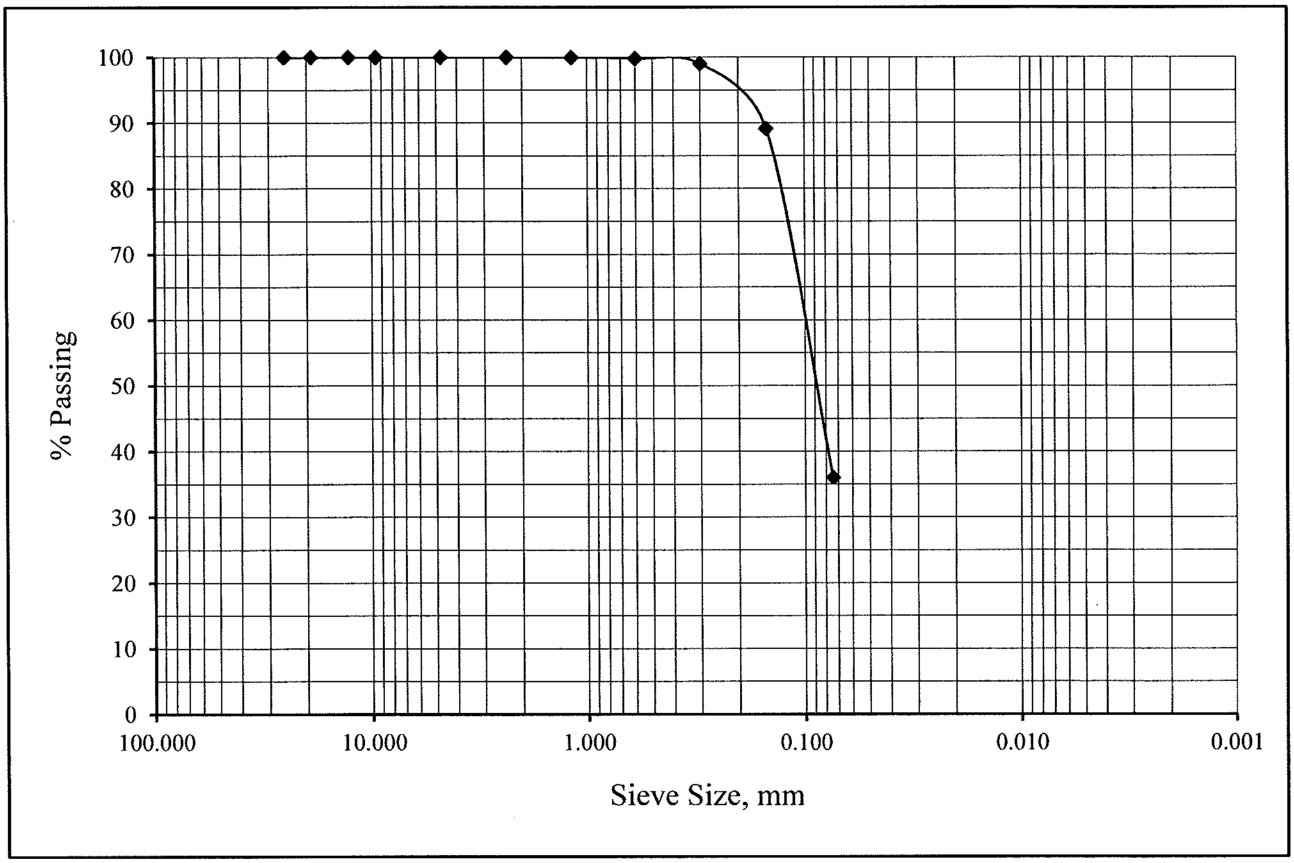


Gradation

ASTM C117 & C136

Project Number: 544-19317 November 20, 2019
Project Name: Avenue 50
Lab ID Number: LN6-19553
Sample ID: BH-2 R-2 @ 10' 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	99.9
#30	0.60	99.8
#50	0.30	99.0
#100	0.15	89.2
#200	0.074	36.0



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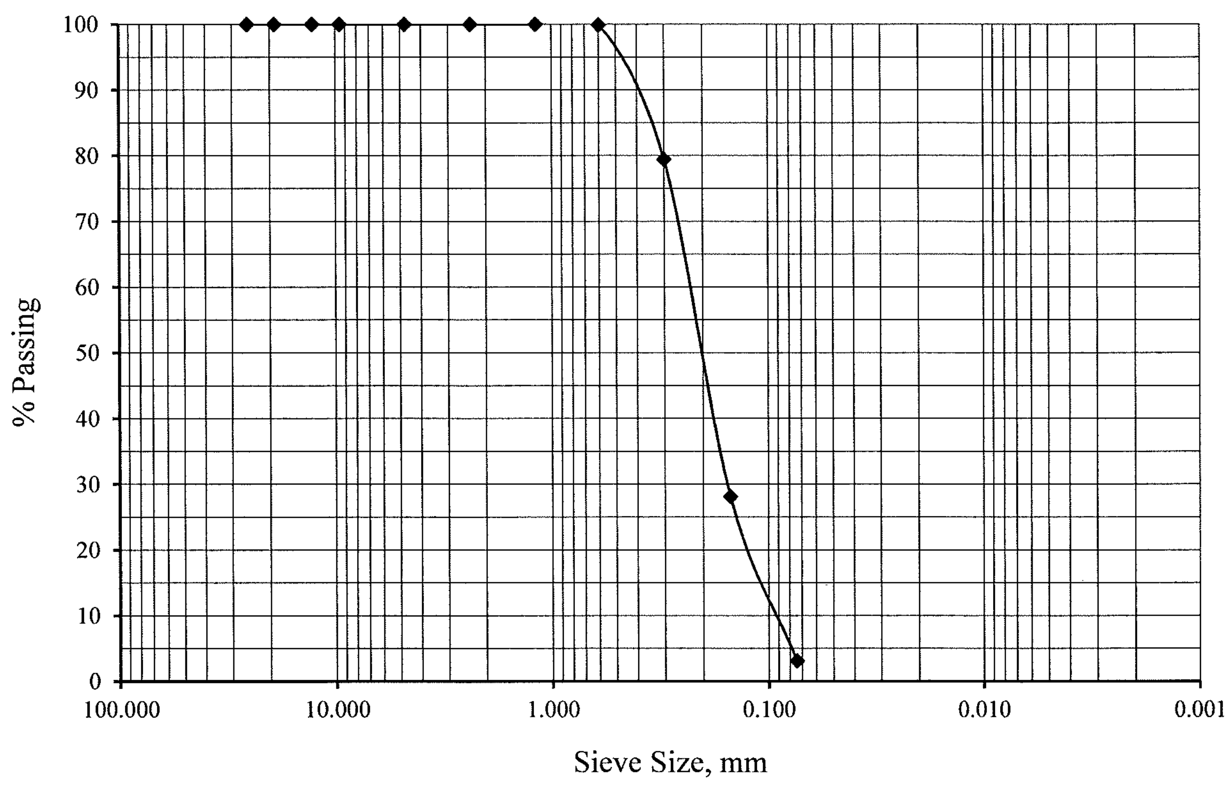
Gradation

ASTM C117 & C136

Project Number: 544-19317 November 20, 2019
Project Name: Avenue 50
Lab ID Number: LN6-19553
Sample ID: BH-3 R-3 @ 15' Soil Classification: SP

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	79.4
#100	0.15	28.1
#200	0.074	3.1



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Gradation

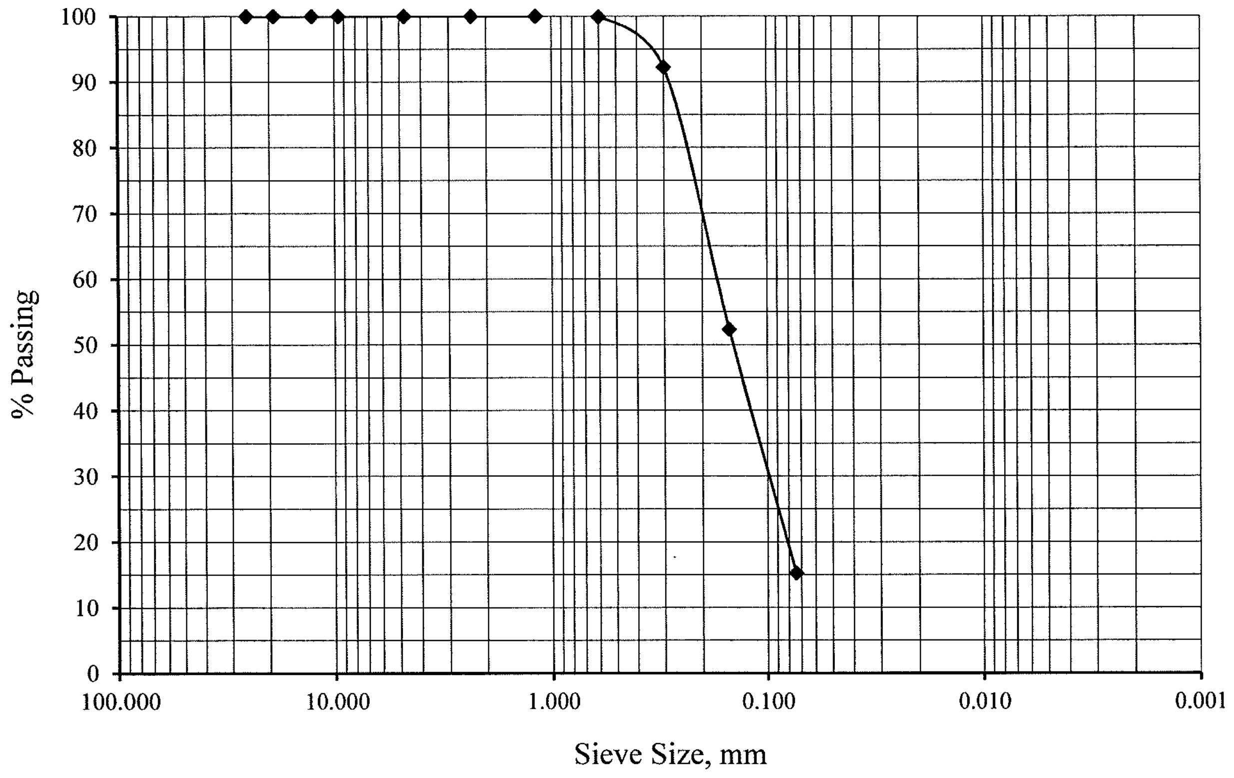
ASTM C117 & C136

Project Number: 544-19317
Project Name: Avenue 50
Lab ID Number: LN6-19553
Sample ID: BH-5 R-1 @ 5'

November 20, 2019

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	92.2
#100	0.15	52.3
#200	0.074	15.3



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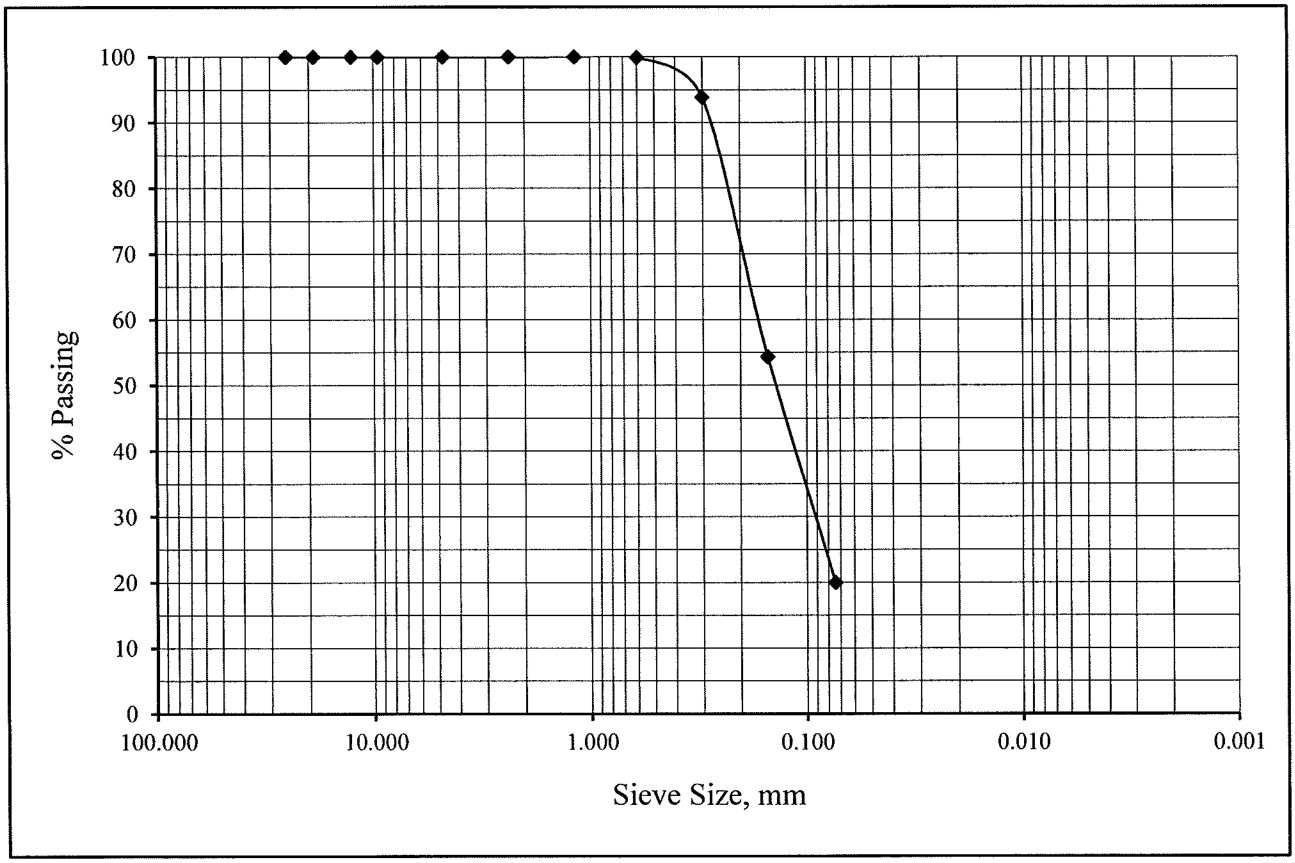
Gradation

ASTM C117 & C136

Project Number:	544-19317	November 20, 2019
Project Name:	Avenue 50	
Lab ID Number:	LN6-19553	
Sample ID:	BH-6 S-3 @ 15'	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	93.8
#100	0.15	54.3
#200	0.074	20.0



One Dimensional Consolidation

ASTM D2435 & D5333

Job Number: 544-19317

Job Name: Avenue 50

November 20, 2019

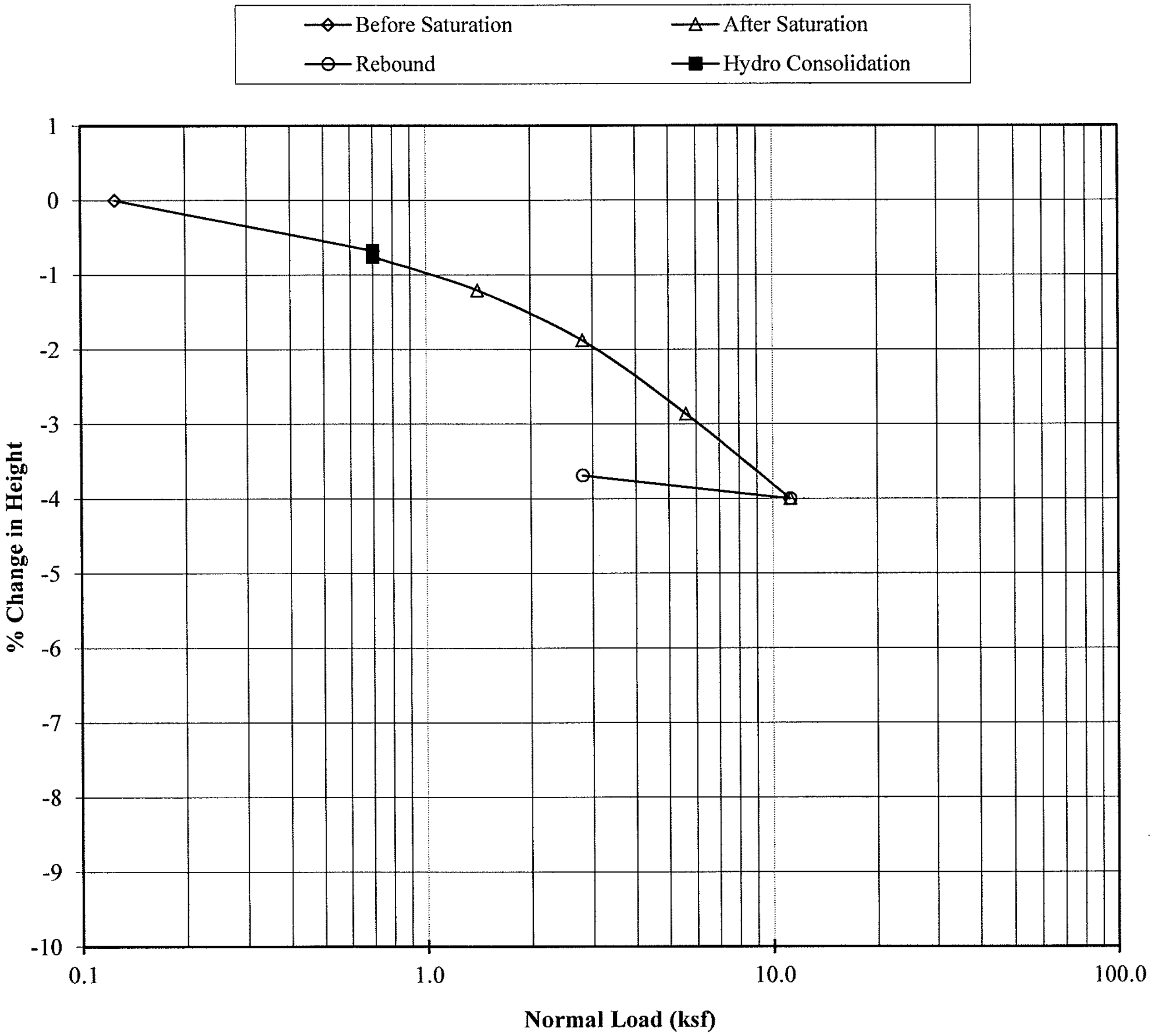
Lab ID Number: LN6-19553

Sample ID: BH-2 R-2 @ 10'

Soil Description: Dark Brown Silty Sand (SM)

Initial Dry Density, pcf:	94.0
Initial Moisture, %:	9.8
Initial Void Ratio:	0.773
Specific Gravity:	2.67

% Change in Height vs Normal Presssure Diagram



Sladden Engineering

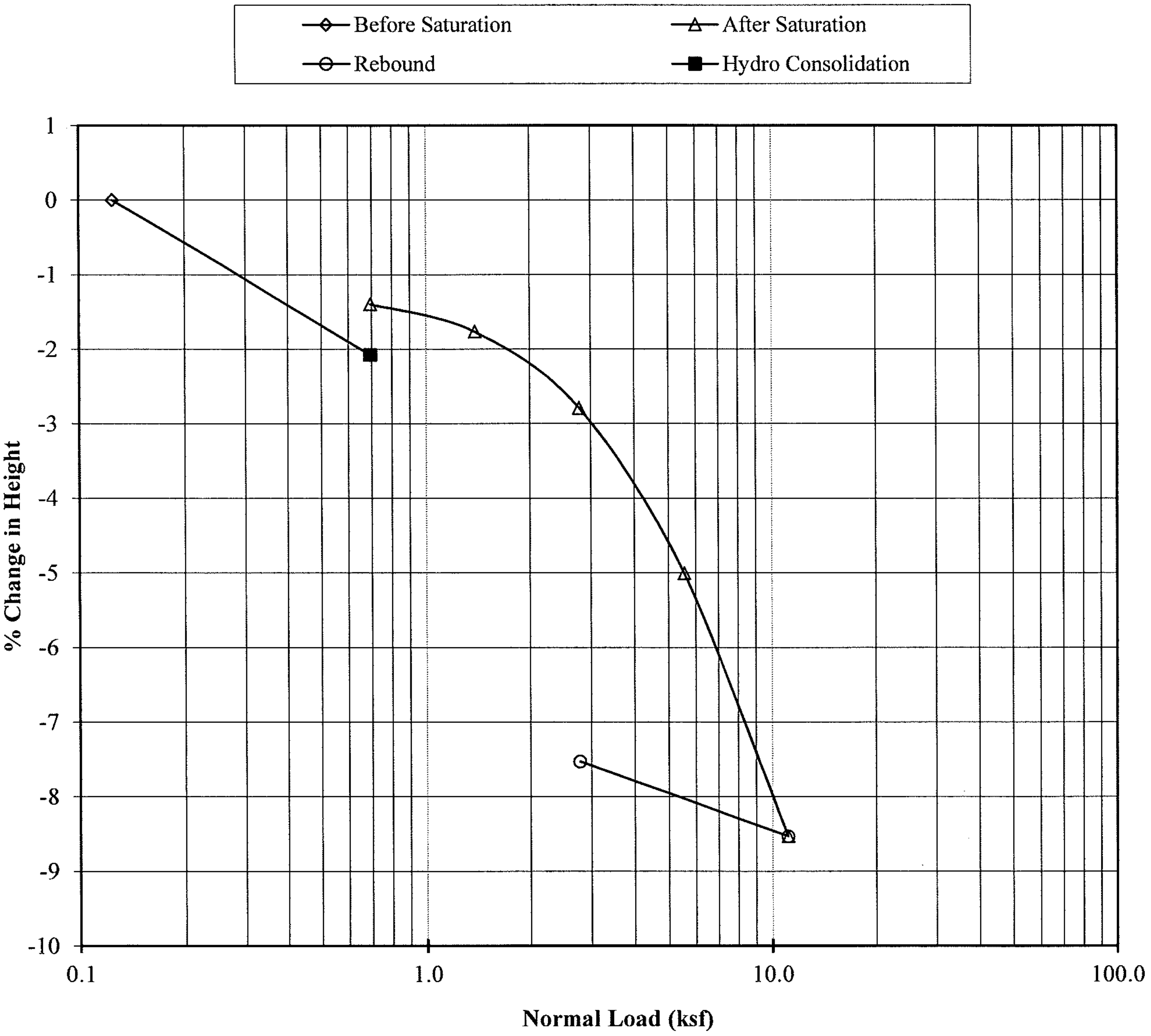
450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

One Dimensional Consolidation

ASTM D2435 & D5333

Job Number:	544-19317	November 20, 2019	
Job Name:	Avenue 50		
Lab ID Number:	LN6-19553	Initial Dry Density, pcf:	81.2
Sample ID:	BH-3 R-1 @ 5'	Initial Moisture, %:	38.2
Soil Description:	Dark Olive Brown Clay (CL)	Initial Void Ratio:	1.053
		Specific Gravity:	2.67

% Change in Height vs Normal Presssure Diagram



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6782 Stanton Ave., Suite C, 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: November 20, 2019

Account No.: 544-19317

Customer: Kraemer Land Company, Inc.

Location: Avenue 50 East of Jefferson, Indio

Analytical Report

Corrosion Series

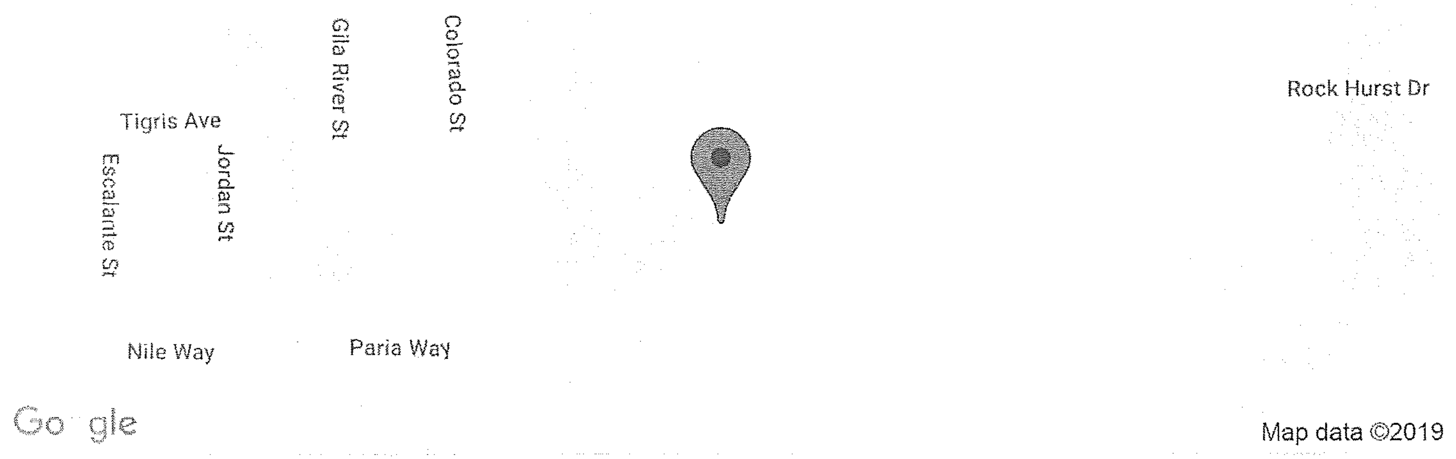
	pH per CA 643	Soluble Sulfates per CA 417 ppm	Soluble Chloride per CA 422 ppm	Min. Resistivity per CA 643 ohm-cm
BH-1 @ 0-5’	9.2	240	110	3100

APPENDIX C

USGS SEISMIC DESIGN MAP AND REPORT
DEAGGREGATION OUTPUT

OSHPD

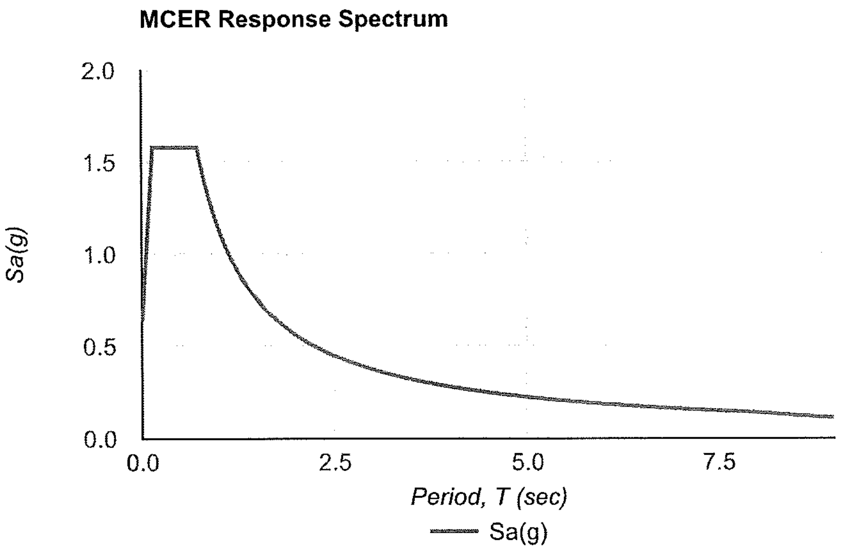
Latitude, Longitude: 33.688632, -116.258041

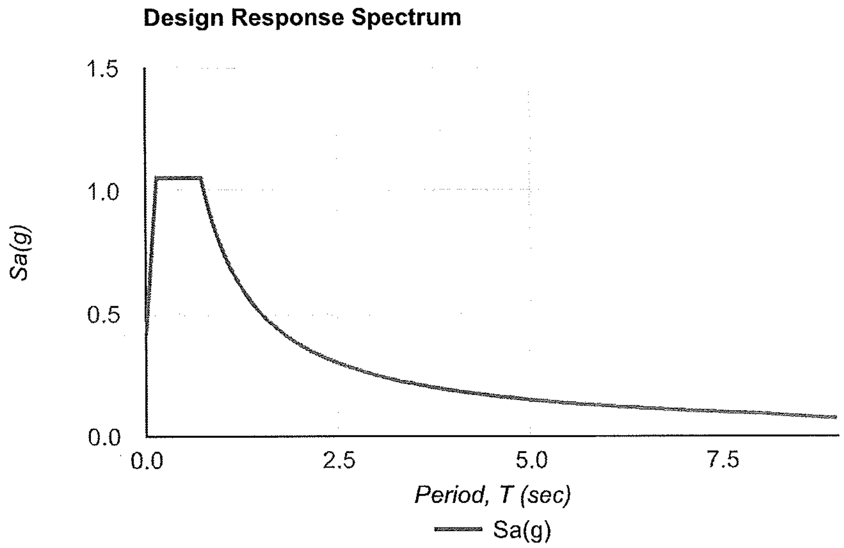


Date	11/18/2019, 10:29:22 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.579	MCE_R ground motion. (for 0.2 second period)
S_1	0.747	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.579	Site-modified spectral acceleration value
S_{M1}	1.121	Site-modified spectral acceleration value
S_{DS}	1.053	Numeric seismic design value at 0.2 second SA
S_{D1}	0.747	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.638	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.638	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.337	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.29	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.579	Factored deterministic acceleration value. (0.2 second)
S1RT	0.892	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.903	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.747	Factored deterministic acceleration value. (1.0 second)
PGAd	0.638	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	1.021	Mapped value of the risk coefficient at short periods
C_{R1}	0.988	Mapped value of the risk coefficient at a period of 1 s


MCER Response Spectrum
2.0
1.5
1.0
0.5
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)


Design Response Spectrum
1.5
1.0
0.5
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)

DISCLAIMER

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Latitude

Decimal degrees

33.688632

Longitude

Decimal degrees, negative values for western longitudes

-116.258041

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

Return period in years

475

^ Hazard Curve

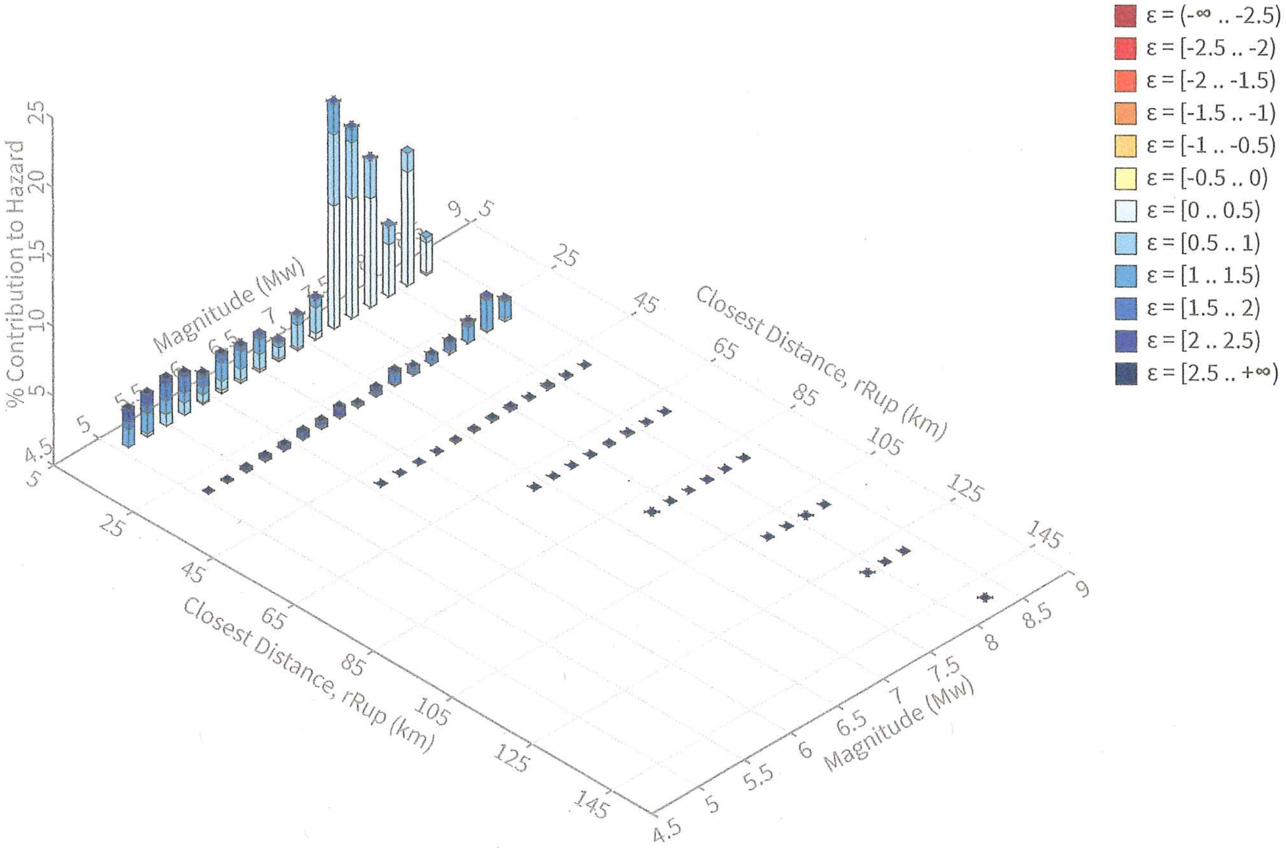
Please select “Edition”, “Location” & “Site Class” above to compute a hazard curve.

Compute Hazard Curve

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.49158173 g

Recovered targets

Return period: 506.63222 yrs
Exceedance rate: 0.0019738184 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.13 %

Mean (over all sources)

m: 7.15
r: 12.25 km
ε_0: 0.84 σ

Mode (largest m-r bin)

m: 7.34
r: 8.74 km
ε_0: 0.56 σ
Contribution: 16.21 %

Mode (largest m-r-ε_0 bin)

m: 7.34
r: 8.69 km
ε_0: 0.41 σ
Contribution: 8.82 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							38.10
San Andreas (Coachella) rev [1]		8.68	7.58	0.47	116.191°W	33.743°N	45.39	31.00
San Jacinto (Anza) rev [5]		32.39	7.99	1.40	116.513°W	33.490°N	226.96	1.91
San Jacinto (Clark) rev [0]		31.86	7.75	1.56	116.462°W	33.459°N	216.54	1.62
UC33brAvg_FM32	System							38.03
San Andreas (Coachella) rev [1]		8.68	7.58	0.47	116.191°W	33.743°N	45.39	30.89
San Jacinto (Anza) rev [5]		32.39	7.97	1.41	116.513°W	33.490°N	226.96	1.94
San Jacinto (Clark) rev [0]		31.86	7.76	1.55	116.462°W	33.459°N	216.54	1.57
UC33brAvg_FM31 (opt)	Grid							11.94
PointSourceFinite: -116.258, 33.729		6.75	5.64	0.93	116.258°W	33.729°N	0.00	1.40
PointSourceFinite: -116.258, 33.729		6.75	5.64	0.93	116.258°W	33.729°N	0.00	1.39
UC33brAvg_FM32 (opt)	Grid							11.93
PointSourceFinite: -116.258, 33.729		6.75	5.64	0.93	116.258°W	33.729°N	0.00	1.40
PointSourceFinite: -116.258, 33.729		6.75	5.64	0.93	116.258°W	33.729°N	0.00	1.39

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450 Egan Avenue, Beaumont, CA. 92223 (951) 845-7743 Fax (951) 845-8863
800 E. Florida Avenue, Hemet, CA. 92543 (951) 766-8777 Fax (951) 766-8778

November 8, 2019

Project No. 544-19317
19-11-509

Kraemer Land Company, Inc.
Attn: Mr. Matthew Ferree
180 North Riverside Drive, Suite 100
Anaheim, California 92808

Project: Proposed Residential Development
Avenue 50 East of Jefferson Street
Indio, California

Subject: Infiltration Testing for On-Site Storm Water Retention

In accordance with your request, we have performed infiltration testing on the subject site to evaluate the infiltration potential of the near surface soil to assist in storm water retention system design. It is our understanding that on-site storm water retention will be required for the subject project. The infiltration rates determined should be useful in the assessment of on-site storm water retention needs. The approximate locations of the tests are indicated on the attached Test Location Plan (Figure 1).

Infiltration testing was performed on October 24, 2019. Testing was performed utilizing double-ring infiltrometers at depths of approximately 3.5 and 4.0 feet below the existing ground surface in the areas of the proposed retention basins. Testing was performed in general accordance with the *Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer* (ASTM D-3385). The following table presents the infiltration rates determined.

Test Location	Depth (bgs)	Infiltration Rate (in/hr)
DR-1	3.5	15.6
DR-2	3.5	15.6
DR-3	4.0	10.4

The rates determined represent the ultimate field rates and an appropriate safety factor should be incorporated into design to account for long-term saturation and potential "silting" of the surface soil. The safety factor should be determined with consideration to other factors considered in the storm water retention system design (specifically storm water volume estimates) and the safety factors associated with the related design components.

If you have any questions regarding this memo or the testing summarized herein, please contact the undersigned.

Respectfully submitted,
SLADDEN ENGINEERING

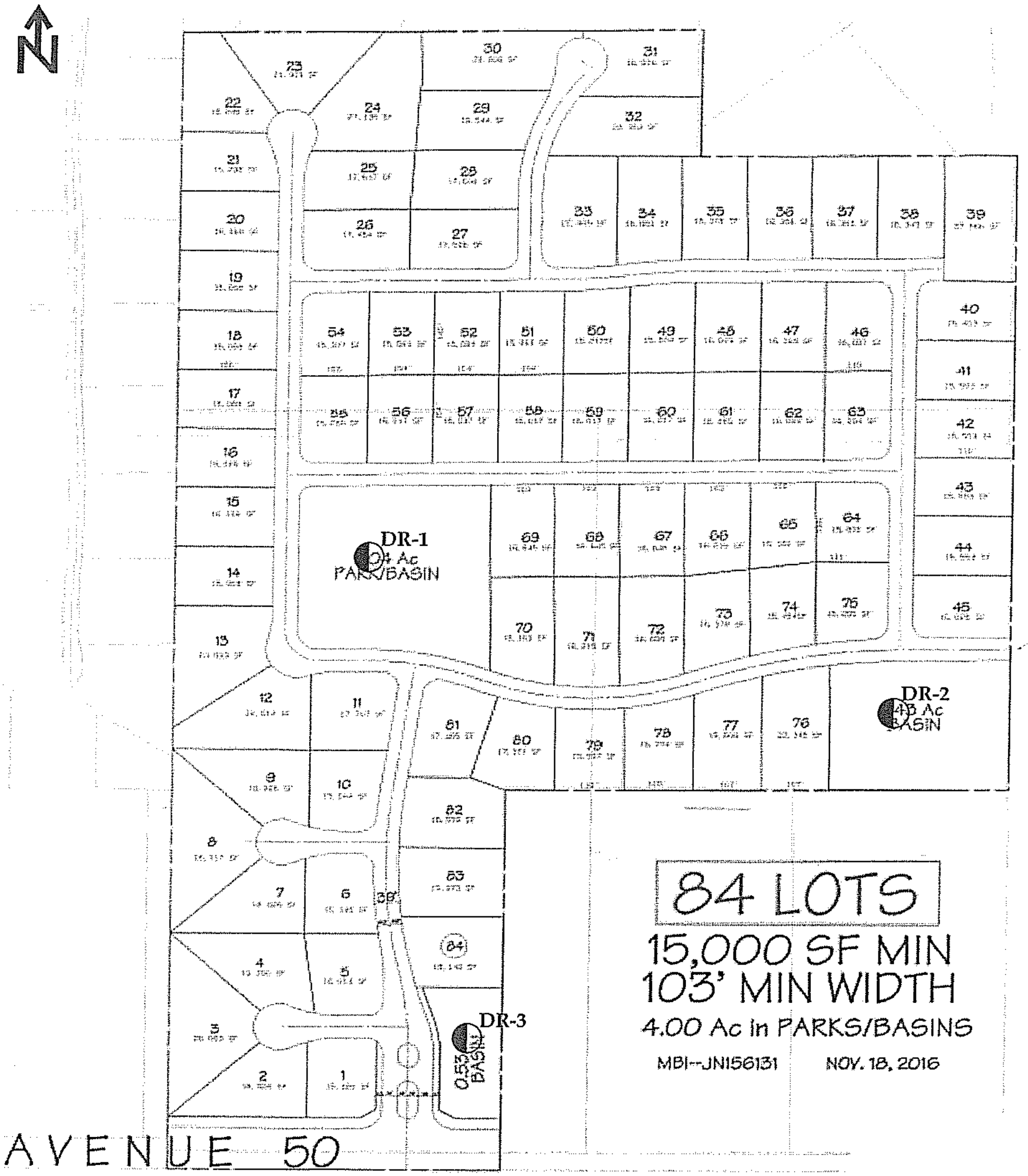
James W. Minor III
Senior Geologist

PROFESSIONAL GEOLOGIST
JAMES W.
MINOR III
No. 9735
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
BRETT L.
ANDERSON
No. C45389
CIVIL
ENGINEERING
STATE OF CALIFORNIA

Brett L. Anderson
Principal Engineer

Copies: 4 / Addressee



Sladden Engineering

TEST LOCATION PLAN

Project Number:	544-19317
Report Number:	19-11-509
Date:	November 8, 2019

FIGURE

1

DOUBLE RING INFILTRATION RATE CALCULATIONS

Job No. 544-19317

Test Hole DR-1

Depth 3.5 ft bgs

AVERAGE RATE* = 15.6 (in/hr)

INNER RING

Interval Number	Initial Water(cm)	Final Water(cm)	Con. Factor (cm to in)	Water (in)	Area Mar. (in2)	Volume (in3)	Area IR (in2)	Time (min)	Time (hr)	Vir (in/hr)
1	46.6	0.0	0.39	18.3	8.9	164.1	113.1	15	0.25	5.8
2	46.9	0.0	0.39	18.5	8.9	165.1	113.1	10	0.17	8.8
3	46.1	0.0	0.39	18.1	8.9	162.3	113.1	10	0.17	8.6
4	45.9	4.0	0.39	16.5	8.9	147.5	113.1	5	0.08	15.7
5	44.1	3.3	0.39	16.1	8.9	143.6	113.1	5	0.08	15.2
6	46.3	4.6	0.39	16.4	8.9	146.8	113.1	5	0.08	15.6
7	45.9	4.2	0.39	16.4	8.9	146.8	113.1	5	0.08	15.6
8	45.7	3.9	0.39	16.5	8.9	147.2	113.1	5	0.08	15.6
9	46.1	4.1	0.39	16.5	8.9	147.9	113.1	5	0.08	15.7
10	45.8	3.8	0.39	16.5	8.9	147.9	113.1	5	0.08	15.7
11	45.3	3.5	0.39	16.5	8.9	147.2	113.1	5	0.08	15.6
12	45.8	4	0.39	16.6	8.9	148.2	113.1	5	0.08	15.7
13										
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30										

DOUBLE RING INFILTRATION RATE CALCULATIONS

Job No. 544-19317

Test Hole DR-2

Depth 3.5 ft bgs

AVERAGE RATE* = 15.6
(in/hr)

INNER RING

Interval Number	Initial Water(cm)	Final Water(cm)	Con. Factor (cm to in)	Water (in)	Area Mar. (in2)	Volume (in3)	Area IR (in2)	Time (min)	Time (hr)	Vir (in/hr)
1	45.8	0.0	0.39	18.0	8.9	161.2	113.1	15	0.25	5.7
2	47.0	0.0	0.39	18.5	8.9	165.5	113.1	10	0.17	8.8
3	45.9	0.0	0.39	18.1	8.9	161.6	113.1	10	0.17	8.6
4	45.5	3.3	0.39	16.6	8.9	148.6	113.1	5	0.08	15.8
5	46.2	3.8	0.39	16.7	8.9	149.3	113.1	5	0.08	15.8
6	45.3	3.4	0.39	16.5	8.9	147.5	113.1	5	0.08	15.7
7	45.1	3.7	0.39	16.3	8.9	145.8	113.1	5	0.08	15.5
8	46.3	4.0	0.39	16.7	8.9	148.9	113.1	5	0.08	15.8
9	45.0	3.9	0.39	16.2	8.9	144.7	113.1	5	0.08	15.4
10	46.0	4.2	0.39	16.5	8.9	147.2	113.1	5	0.08	15.6
11	46.3	4.3	0.39	16.5	8.9	147.9	113.1	5	0.08	15.7
12	45.9	4.1	0.39	16.5	8.9	147.2	113.1	5	0.08	15.6
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DOUBLE RING INFILTRATION RATE CALCULATIONS

Job No. 544-19317

Test Hole DR-3

Depth 4 ft bgs

AVERAGE RATE* = 10.4
(in/hr)

INNER RING

Interval Number	Initial Water(cm)	Final Water(cm)	Con. Factor (cm to in)	Water (in)	Area Mar. (in2)	Volume (in3)	Area IR (in2)	Time (min)	Time (hr)	Vir (in/hr)
1	46.5	0.0	0.39	18.3	8.9	163.7	113.1	15	0.25	5.8
2	46.1	15.7	0.39	12.0	8.9	107.0	113.1	10	0.17	5.7
3	45.9	18.3	0.39	10.9	8.9	97.2	113.1	10	0.17	5.2
4	46.2	16.4	0.39	11.7	8.9	104.9	113.1	5	0.08	11.1
5	45.5	17.8	0.39	10.9	8.9	97.5	113.1	5	0.08	10.3
6	45.6	17.9	0.39	10.9	8.9	97.5	113.1	5	0.08	10.3
7	45.8	18.1	0.39	10.9	8.9	97.5	113.1	5	0.08	10.3
8	46.6	18.6	0.39	11.0	8.9	98.6	113.1	5	0.08	10.5
9	46.1	18.2	0.39	11.0	8.9	98.2	113.1	5	0.08	10.4
10	45.7	17.8	0.39	11.0	8.9	98.2	113.1	5	0.08	10.4
11	46.2	18.0	0.39	11.1	8.9	99.3	113.1	5	0.08	10.5
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