GEOTECHNICAL INVESTIGATION
PROPOSED 104-UNIT CONDOMINIUM COMPLEX
LAKESHORE DRIVE
TRACT MAP NO. 37280
LAKE ELSINORE, CALIFORNIA

DRAFT

-Prepared By-

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August 3, 2017

Project No. 644-16063
17-08-068

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Subject: Geotechnical Investigation

Project: Proposed 104-Unit Condominium Complex
Lakeshore Drive
Tract Map No. 37280
Lake Elsinore, California

Sladden Engineering is pleased to present the results of our geotechnical investigation for the proposed condominium complex to be constructed on the site located east of Machado Street on the south side of Lakeshore Drive (APN: 379-230-001) in the City of Lake Elsinore, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated July 5, 2017 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site in order to provide recommendations for foundation design and site preparation. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

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

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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the new condominium complex proposed for the site located east of Machado Street on the south side of Lakeshore Drive (APN: 379-230-001) in the City of Lake Elsinore, California. The site is situated at approximately 33.6865 degrees North latitude and 117.3724 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 site plans (ANG Designs, 2017), it is our understanding that the proposed project will consist of constructing a new 104-unit condominium complex on the subject property. In addition, a swimming pool, playground and club house will be constructed on the site. Sladden anticipates that associated site improvements will also include concrete flatwork landscape areas, underground utilities and various other site improvements.

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 pad elevation and provide adequate gradients for site drainage. This does not include the removal and/or recompaction of foundation bearing soil within the building envelope.

Structural foundation loads were not available at the time of this report. Based on our experience with relatively lightweight wood-frame structures, we expect that isolated column loads will be less than 50 kips and continuous wall loads will be less than 4.0 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.

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SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Exploration of the site was achieved by drilling five (5) exploratory boreholes to depths of approximately 11.5 to 28.0 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 five (5) exploratory boreholes to depths of 11.5 and 28.0 feet bgs in order to characterize the subsurface soil conditions. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- The performance of laboratory testing on selected samples to evaluate their engineering characteristics.
- The review of available geologic literature and discuss potential geologic hazards.
- The performance of engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

SITE CONDITIONS

The site is located east of Machado Street on the south side of Lakeshore Drive in the City of Lake Elsinore, California. The site is formally identified by the County of Riverside as APN: 379-230-001 and occupies approximately 7.50 acres. At the time of our investigation, the project site was vacant and covered by sparse vegetation and gravel.

The site is near the elevation of the adjacent properties and roadways. The site is bound by Lakeshore Drive to the north and east and west and to the south by existing residential structures.

In general the site is relatively level with no discernable trends in surface gradients. According to the USGS 7.5' Lake Elsinore Quadrangle map (USGS, 2012), the site is situated at an elevation of approximately 1,310 feet above mean sea level (MSL).

No natural ponding water or surface seepage was observed at or near the site during our field investigation conducted on July 18, 2017. Site drainage appears to be controlled via sheet flow and surface infiltration.

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GEOLOGIC SETTING

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

The site has been mapped by Morton & Weber (1998) to be underlain by young alluvial deposits (Qyv). The geologic setting for the site and site vicinity is illustrated on the Regional Geologic Map, Figure 2.

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling five (5) exploratory boreholes on the subject property. 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 (O.D.) 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 fill/disturbed soil was encountered overlying native alluvium. The fill/disturbed soil was encountered to a maximum depth of generally less than three (3) feet below the existing ground surface. Underlying the fill soil and extending to the maximum depth explored, native alluvium was encountered. The site soil consists of interbedded silty sand (SM), sandy silt (ML) and gravelly sand (SP/SW) that was encountered to the maximum depths explored. Generally the native earth materials appeared near optimum moisture content with densities increasing with depth. Boring refusal was encountered within two of our bores (BH-3 & BH-5) due to the presence of coarse gravel and cobbles.

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/or variable across the site.

Groundwater was not encountered to a maximum depth explored of 28.0 feet bgs during our field investigation on July 18, 2017. Based on our exploratory bores and our review of groundwater level data (CDWR, 2017), groundwater should not be a factor during construction of the proposed developments. However, following periods of heavy or prolonged rainfall, perched water may be encountered, but would likely dissipate.

SEISMICITY AND FAULTING

The southwestern United States is a tectonically active and structurally complex region, dominated by northwest trending dextral faults. Faults in the region are often part of complex fault systems composed of numerous subparallel faults that 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.

Sladden considers the most significant geologic hazard to the project to be the potential for moderate to severe 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).

Based on our research, the site is currently not located within a State of California designated fault zone (CDMG, 1980) or a County of Riverside designated fault zone (RCPR, 2017). However, according to RCPR (2017) the subject site is located within 0.5 miles of a fault zone. 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 other faults in the region.

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
Elsinore – Glen Ivy	2.1	6.8
Elsinore – Temecula	5.3	6.8
Chino – Central Avenue (Elsinore)	24.0	6.7
Whittier	30.8	6.8
San Jacinto – San Jacinto Valley	34.6	6.9
San Jacinto – San Bernardino	38.4	6.7
San Jacinto – Anza	42.6	7.2
Newport – Inglewood (Offshore)	42.8	7.1

2016 CBC SEISMIC DESIGN PARAMETERS

Sladden has reviewed the 2016 California Building Code (CBC) and summarized the current seismic design parameters for any 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 any proposed structures. The 2016 CBC Seismic Design Parameters are summarized below (USGS, 2017a). The project Design Map Reports are included within Appendix C.

Risk Category (Table 1.5-1): I/II/III
Site Class (Table 1613.3.2): D
S_S (Figure 1613.3.1): 2.387g
S_1 (Figure 1613.3.1): 0.962g
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}): 2.387g
S_{M1} (Equation 16-38 {F_v X S_1}): 1.443g
S_{DS} (Equation 16-39 {2/3 X S_{MS}}): 1.592g
S_{D1} (Equation 16-40 {2/3 X S_{M1}}): 0.962g
Seismic Design Category: E

GEOLOGIC HAZARDS

The subject site is not located in an active seismic fault zone however, 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), CDMG (1980) RCPR (2017) and Morton & Weber (1998), 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, 2017). Finally, no signs of active surface fault rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified on-site during our field investigation. However, the Lake Elsinore fault zone projects towards the project site. Therefore, it is our opinion that additional fault evaluation should be considered.

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 Beta – Unified Hazard Tool (USGS, 2017b) and shear wave velocity (Vs30) of 259 m/s the site could be subjected to ground motions on the order of 0.666g. The peak ground acceleration at the site is judged to have a 475 year return period and a 10 percent chance of exceedence in 50 years.

III. Liquefaction/ Dry Sand Settlement. Liquefaction is the process in which loose, saturated granular soil loses strength as a result of cyclic loading. The strength loss is a result of a decrease in granular sand volume and a positive increase in pore pressures. Generally, liquefaction can occur if all of the following conditions apply: liquefaction-susceptible soil, groundwater within a depth of 50 feet or less, and strong seismic shaking.

According to the County of Riverside, the site is situated within a "Moderate" liquefaction potential zone (RCPR, 2017). Groundwater data available at CDWR (2017) indicates groundwater depths greater than 30 feet bgs in the vicinity of the site. Based on the depth to groundwater and the generally dense nature of the material underlying the site, it is Sladden's professional opinion that risks associated with liquefaction should be considered "low".

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

V. Slope Failure, Landsliding, Rock Falls. The site is located on relatively flat ground and not immediately adjacent to any slopes or hillsides. Therefore, it is our professional opinion that risks associated with slope instability should be considered "negligible".

VI. Expansive Soil. Expansion Index testing of select samples was performed in order to evaluate expansive potential of the materials underlying the site. Based the results of our laboratory testing (EI=3), the materials underlying the site were determined to be in the low expansion category and the risk of structural damage caused by volumetric changes in the subgrade soil is considered "low".

VII. Settlement. 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 ultimate settlement is estimated to be less than one inch when using the recommended bearing pressures. As a practical matter, differential settlement between footings can be assumed as one-half of the total 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. However, site specific effects resulting from long term regional subsidence is beyond the scope of our investigation.

IX. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation conducted on April 6, 2017. However, risks associated with flooding and erosion should be evaluated and mitigated by the project design Civil Engineer.

X. 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 remote.

CONCLUSIONS

Based on the results of our geotechnical investigation, it is our opinion that the project should be feasible from a geotechnical perspective provided that the recommendations provided in this report are incorporated into design and carried out through construction. The main geotechnical concerns in the design and construction of the proposed project are the presence of artificial fill soil, the loose condition of some of the near surface soil, presence of cobble material at depth and the projection of the Lake Elsinore fault zone towards the project site.

Because of the somewhat loose and potentially compressible condition of the near surface soil and the presence of artificial fill, remedial grading including overexcavation and recompaction is recommended for the proposed slab and foundation areas unless documentation of the fill soil placement and compaction can be provided or verified by testing. We recommend that remedial grading within the building areas include the over-excavation and re-compaction of the artificial fill and the primary foundation bearing soil. Specific recommendations for site 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 subgrade 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. <u>Stripping</u>. Areas to be graded should be cleared of, vegetation, existing and associated 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. <u>Preparation of the Building Areas</u>. In order to achieve firm and uniform bearing conditions, we recommend overexcavation and recompaction throughout the building areas. All artificial fill and native low density near surface soil should be removed to a depth of approximately 3 feet below existing grade or 2 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally, a minimum of five feet beyond the foundation limits. The exposed surface should then be scarified, moisture conditioned to within two percent of optimum moisture content, and compacted to at least 90 percent relative compaction. If adequate documentation of the existing fill soil is provided or adequate compaction of the existing fill soil is verified by testing, overexcavation may not be necessary.

c. <u>Compaction</u>. 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 six (6) inches in maximum dimension. All fill materials should be placed in thin lifts, not exceeding six inches in their loose state. 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 material should be compacted with acceptable compaction equipment, to at least 90 percent relative compaction. The bottom of the exposed subgrade should be observed by a representative of Sladden Engineering prior to fill placement. Compaction testing should be performed to verify proper placement of the fill materials. Table 2 provides a summary of the excavation and compaction recommendations.

TABLE 2
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Excavation and/or recompaction within the building envelopes and extending laterally for 5 feet beyond the building limits and to competent native soil or 2 feet below the bottom of the footings, whichever is deeper.
Native / Import Engineered Fill	Place in thin lifts not exceeding 6 inches in a loose condition, compact to a minimum of 90 percent relative compaction.
Asphalt Concrete Sections	Compact the top 12 inches to at least 95 percent compaction within 2 percent of optimum moisture content.

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

d. Shrinkage and Subsidence. Volumetric shrinkage of the material that is excavated and replaced as controlled compacted fill should be anticipated. We estimate that this shrinkage could vary from 10 to 20 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 and 2 tenths of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

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 44 was determined to develop the following preliminary pavement sections. Any import soil should be tested for R-Value after grading. Actual R-Value of subgrade soil should be consistent with the pavement design. For Pavement design, a Traffic Index (TI) of 6.0 was used for the light duty pavements. We assumed Asphalt Concrete (AC) over Class II Aggregate Base (AB). The preliminary flexible pavement design is as follows:

RECOMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS

Pavement Material	Recommended Thickness	
	TI=5.0 (Light Duty)	TI=6.5 (Heavy Duty)
Asphalt Concrete Surface Course	3.0 inches	4.0 inches
Class II Aggregate Base Course	6.0 inches	6.0 inches
Compacted Subgrade Soil	12.0 inches	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"). Class II aggregate base should conform to Section 26 of the Caltrans Standard Specifications, 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 was determined to be 120 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 determined to be 8.2. Based on soluble chloride concentration testing (60 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 2,800 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.

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 conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should test the backfill to verify adequate compaction.

EXTERIOR CONCRETE FLATWORK

To minimize cracking of concrete flatwork, the subgrade soil below concrete flatwork areas should first be compacted to a minimum 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 pouring concrete.

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

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

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FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PLAN

Sladden Engineering	SITE VICINITY MAP		FIGURE
	Project Number:	644-16063	1
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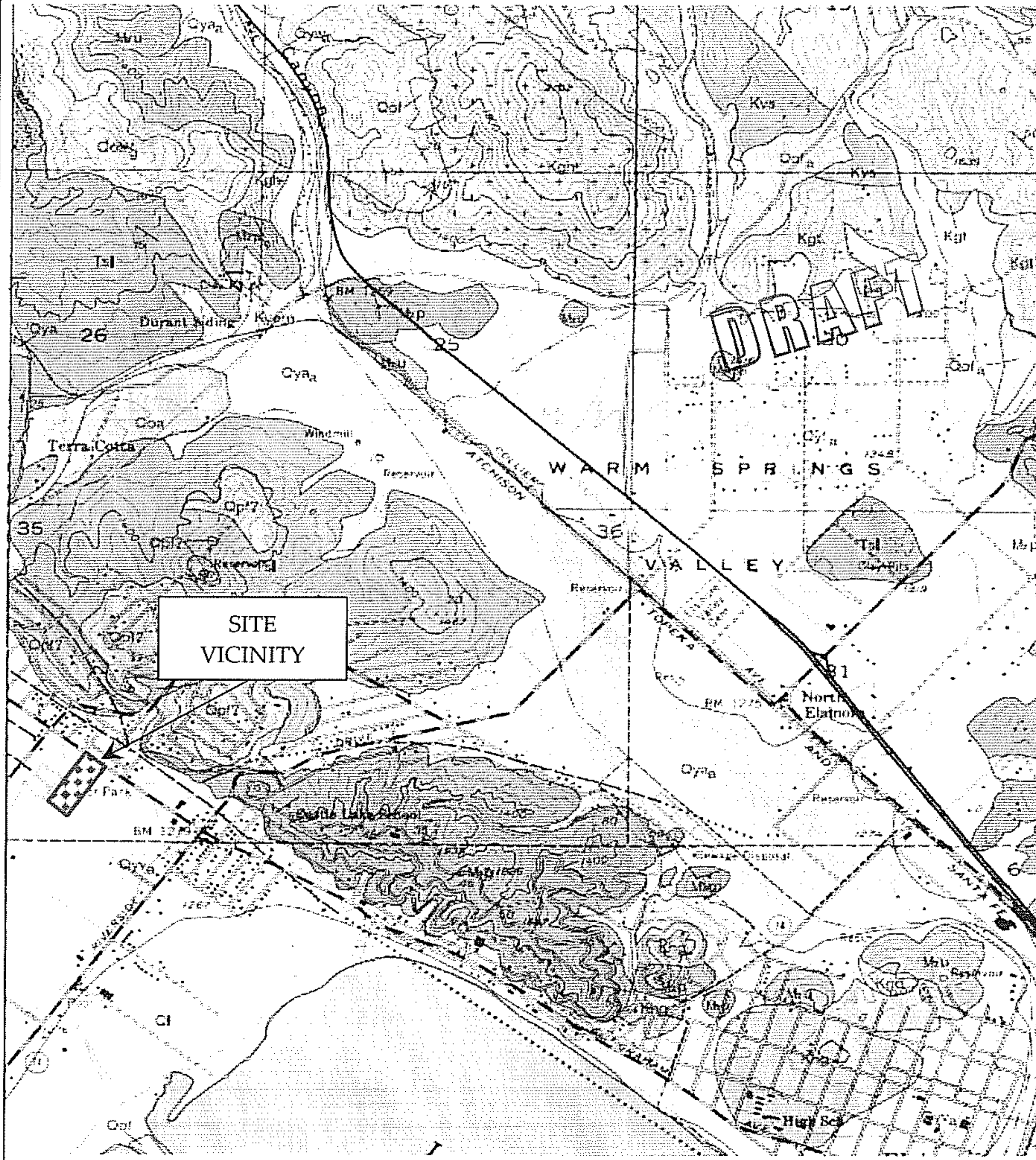

Qyv	Young alluvial-valley deposits (Holocene and late Pleistocene)—Fluvial deposits along valley floors. Consists of unconsolidated sand, silt, and clay-bearing alluvium

Morton & Weber, 1998

L A K E

Sladden Engineering	REGIONAL GEOLOGIC MAP		FIGURE
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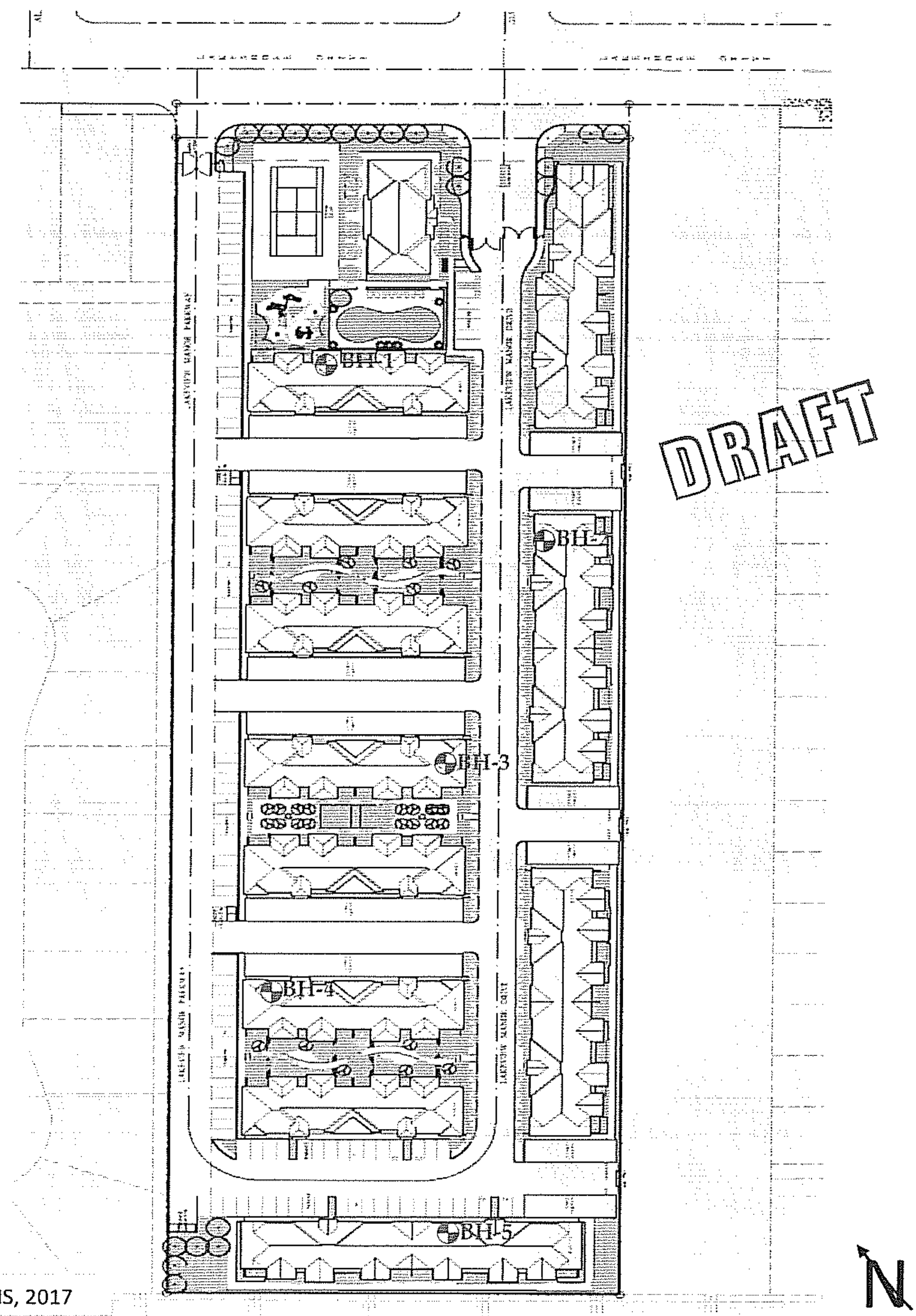

BOREHOLE LOCATION PLAN

Sladden Engineering

Project Number:	644-16063
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FIGURE

3

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation five (5) exploratory bores were excavated on July 18, 2017 utilizing a truck mounted rig (Mobile B-61) equipped with 8-inch outside diameter (O.D.) augers. 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.

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UNIFIED SOIL CLASSIFICATION SYSTEM

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MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS	MORE THAN HALF IS LARGER THAN No.200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN No.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVEL-SAND MIXTURES
				GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY-GRADED GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
		SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN No.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
				SP	POORLY GRADED SANDS, GRAVELLY SANDS
			SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
				SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS	MORE THAN HALF IS SMALLER THAN No.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, CLEAN CLAYS
				OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
		SILTS AND CLAYS: LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

EXPLANATION OF BORE LOG SYMBOLS

California Split-spoon Sample

Unrecovered Sample

Standard Penetration Test Sample

Groundwater depth

Note: The stratification lines on the borelogs represent the approximate boundaries between the soil types; the transitions may be gradual.

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	7/18/2017
Elevation:	1310 Ft (MSL)	Boring No:	BH-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–3		Silty Sand (SM); yellowish brown, slightly moist, fine- to coarse-grained with gravel (Fill).
	15/31/50-3"			51.9	6.1	122.6	3–10		Sandy Silt (ML); grayish brown, slightly moist, hard, low plasticity with trace gravel (Qyw)
	9/10/9			32.6	5.1		10–15		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).
	7/21/24			8.1	1.7	124.2	15–20		Gravelly Sand (SP); yellowish brown, slightly moist, medium dense, fine- to coarse-grained. (Qyw).
	7/11/11			22.2	5.0		20–21.5		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).

Terminated at ~ 21.5 Feet bgs.
No Bedrock Encountered.
No Groundwater or Seepage Encountered.

DRAFT

Completion Notes:

PROPOSED LAKEVIEW MANOR CONDOMINIUM COMPLEX
APN: 379-230-001

Project No:	644-16063	Page	1
Report No:	17-08-068		

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	7/18/2017
Elevation:	1310 Ft (MSL)	Boring No:	BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–3		Silty Sand (SM); yellowish brown, slightly moist, fine- to coarse-grained with gravel (Fill).
■	10/16/19			49.4	8.3		5		Silty Sand (SM); yellowish brown, moist, dense, fine- to coarse-grained with gravel (Qyw).
■	16/16/15			46.8	8.4	118.7	10		Silty Sand (SM); yellowish brown, moist, medium dense, fine- to coarse-grained with gravel (Qyw).
							12		Terminated at ~ 11.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Depth scale: 2, 4, 6, 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 LAKEVIEW MANOR CONDOMINIUM COMPLEX
APN: 379-230-001

Project No: 644-16063
Report No: 17-08-068

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	7/18/2017
Elevation:	1310 Ft (MSL)	Boring No:	BH-3

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
	15/25/36	1	3	56.6	8.3	131.0	2		Sandy Silt (ML); grayish brown, slightly moist, hard, low plasticity with trace gravel (Fill)
	17/24/30			50.5	6.3	122.8	6		Sandy Silt (ML); grayish brown, slightly moist, hard, low plasticity with trace gravel (Qyw)
	7/6/6			37.4	4.4		10		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).
	6/9/9			41.4	6.2	115.1	15		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).
	7/9/11			31.7	7.2		20		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).
	24/50-6"			41.0	6.3		25		Silty Sand (SM); yellowish brown, slightly moist, very dense, fine- to coarse-grained with gravel and cobbles (Qyw).
							28		Practical Auger Refusal at ~ 28.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Depth scale (feet): 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

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Completion Notes:

PROPOSED LAKEVIEW MANOR CONDOMINIUM COMPLEX
APN: 379-230-001

Project No: 644-16063	Page	3
Report No: 17-08-068		

SLADDEN ENGINEERING

Drill Rig:	Mobil B-61	Date Drilled:	7/18/2017
Elevation:	1310 Ft (MSL)	Boring No:	BH-4

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–3		Silty Sand (SM); yellowish brown, slightly moist, fine- to coarse-grained with gravel (Fill).
	10/12/22			48.4	8.2		3–10		Silty Sand (SM); yellowish brown, moist, dense, fine- to coarse-grained with gravel (Qyw).
	16/50-5"			20.0	2.8	119.6	10–12		Silty Sand (SM); yellowish brown, slightly moist, very dense, fine- to coarse-grained with gravel (Qyw).
							12–50		Terminated at ~ 11.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

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Completion Notes:

PROPOSED LAKEVIEW MANOR CONDOMINIUM COMPLEX
APN: 379-230-001

Project No: 644-16063
Report No: 17-08-068

BORE LOG

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobil B-61	Date Drilled:	7/18/2017
Elevation:	1310 Ft (MSL)	Boring No:	BH-5

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							0–3		Silty Sand (SM); yellowish brown, slightly moist, fine- to coarse-grained with gravel (Fill).
	19/50-6"			44.4	5.6	126.1	5		Silty Sand (SM); yellowish brown, slightly moist, very dense, fine- to coarse-grained with gravel (Qyw).
	10/10/8			15.2	2.7		10		Silty Sand (SM); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with gravel (Qyw).
	18/24/29			10.7	1.8		15		Gravelly Sand (SP); yellowish brown, slightly moist, medium dense, fine- to coarse-grained with cobbles. (Qyw).
							18		Practical Auger Refusal at ~ 18.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

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Completion Notes:

PROPOSED LAKEVIEW MANOR CONDOMINIUM COMPLEX
APN: 379-230-001

Project No: 644-16063
Report No: 17-08-068

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Representative bulk and relatively undisturbed soil samples were obtained in the field and returned to our laboratory for additional observations and testing. Laboratory testing was generally performed in two phases. The first phase consisted of testing in order to determine the compaction of the existing natural soil and the general engineering classifications of the soil 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. The results of this testing are presented graphically in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

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

SOIL MECHANIC'S TESTING

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

Direct Shear Tests: One (1) 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 Test: One (1) relatively undisturbed samples were selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load. The specimens were saturated at 575 psf or 720 psf load increment.

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

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Maximum Density/Optimum Moisture

ASTM D698/D1557

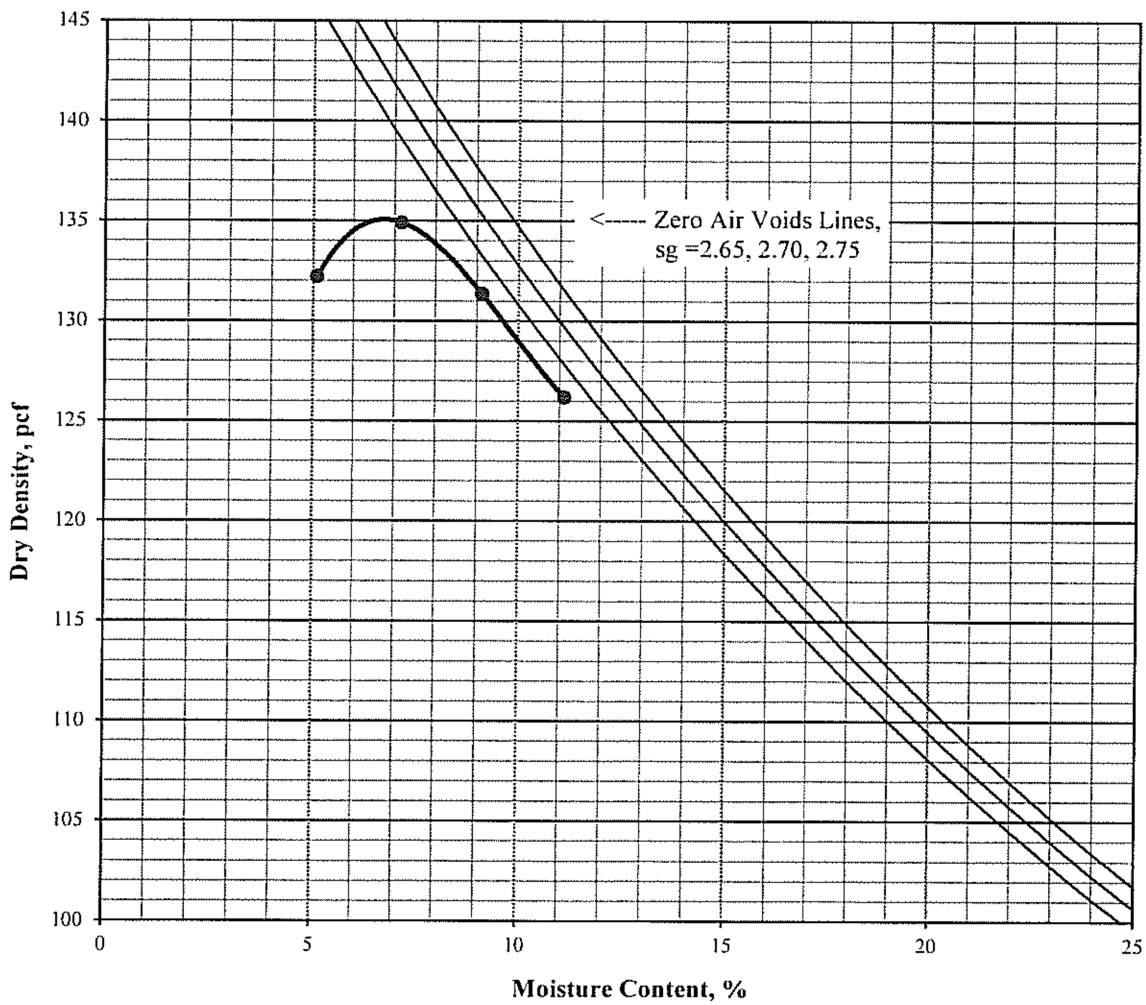
Project Number: 644-16063
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample Location: Bulk 1 @ 0-5'
Description: Dark Brown Silty Sand (SM)

August 2, 2017
ASTM D-1557 A
Rammer Type: Machine

DRAFT

Maximum Density: 137 pcf
Optimum Moisture: 7%
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	
3/8"	
#4	8.7



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Expansion Index

ASTM D 4829

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August 2, 2017

Job Number: 644-16063

Job Name: Lakeview Manor Condominium Complex

Lab ID Number: LN6-17322

Sample ID: Bulk 1 @ 0-5'

Soil Description: Dark Brown Silty Sand (SM)

Wt of Soil + Ring:	599.9
Weight of Ring:	191.1
Wt of Wet Soil:	408.8
Percent Moisture:	7.1%
Sample Height, in	0.95
Wet Density, pcf:	130.4
Dry Denstiy, pcf:	121.8

% Saturation:	49.9

Expansion **Rack # 1**

Date/Time	7/31/2017	2:35 PM
Initial Reading	0.0000	
Final Reading	0.0033	

Expansion Index 3

(Final - Initial) x 1000

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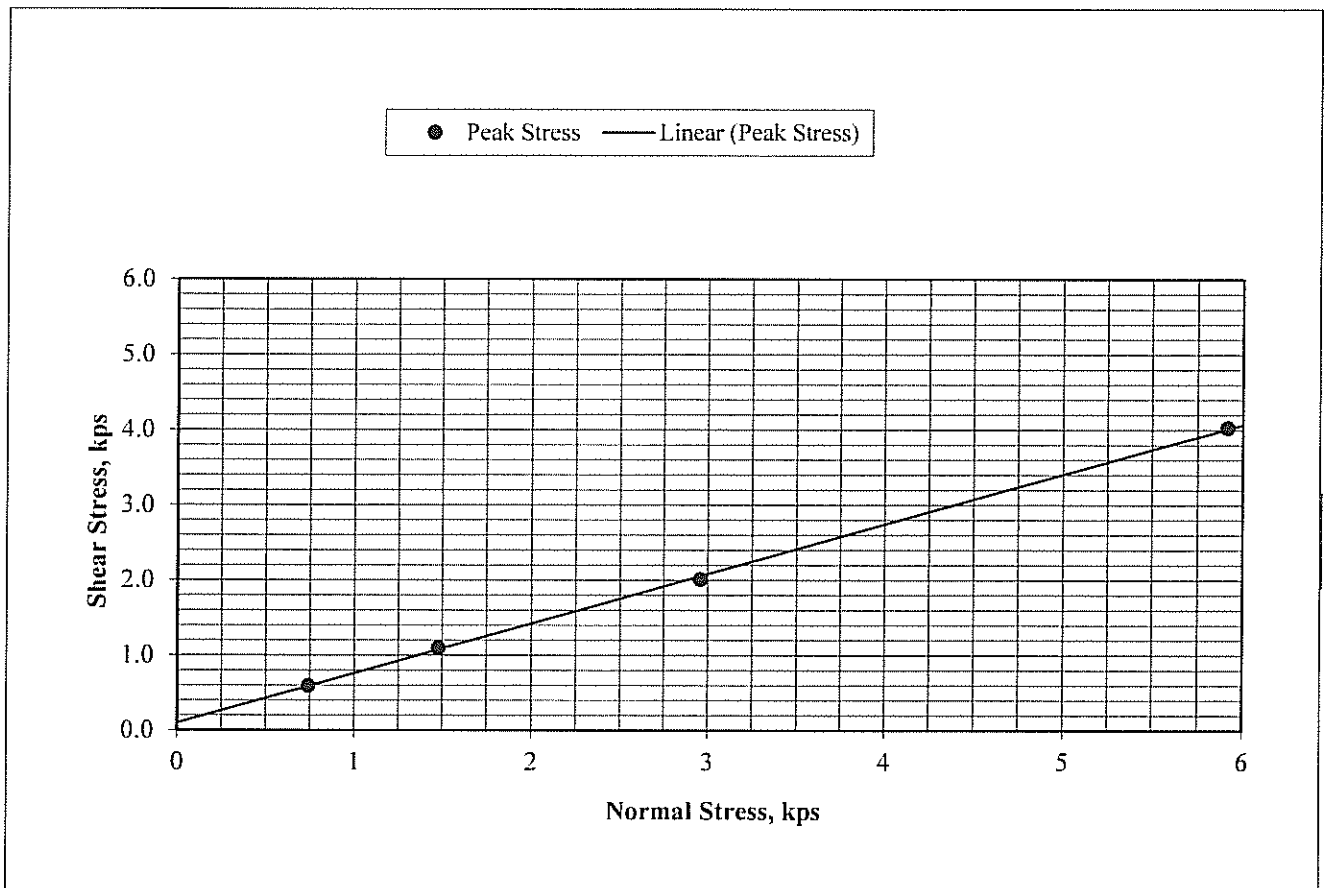
Direct Shear ASTM D 3080-04
(modified for unconsolidated condition)

DRAFT

Job Number:	644-16063		August 2, 2017
Job Name	Lakeview Manor Condominium Complex	Initial Dry Density:	121.4 pcf
Lab ID No.	LN6-17322	Initial Mosture Content:	7.6 %
Sample ID	Bulk 1 @ 0-5'	Peak Friction Angle (Ø):	33°
Classification	Dark Brown Silty Sand (SM)	Cohesion (c):	100 psf
Sample Type	Remolded @ 90% of Maximum Density		

Test Results	1	2	3	4	Average
Moisture Content, %	12.6	12.6	12.6	12.6	12.6
Saturation, %	87.8	87.8	87.8	87.8	87.8
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.594	1.096	2.012	4.028	



Job Number: 644-16063
Job Name: Lakeview Manor Condominium Complex
Date: 8/2/2017

DRAFT

Moisture Adjustment		Remolded Shear Weight	
Wt of Soil:	1,000	Max Dry Density:	135.0
Moist As Is:	4.9	Optimum Moisture:	7.5
Moist Wanted:	7.5		
ml of Water to Add:	24.8	Wt Soil per Ring, g:	157.1

UBC

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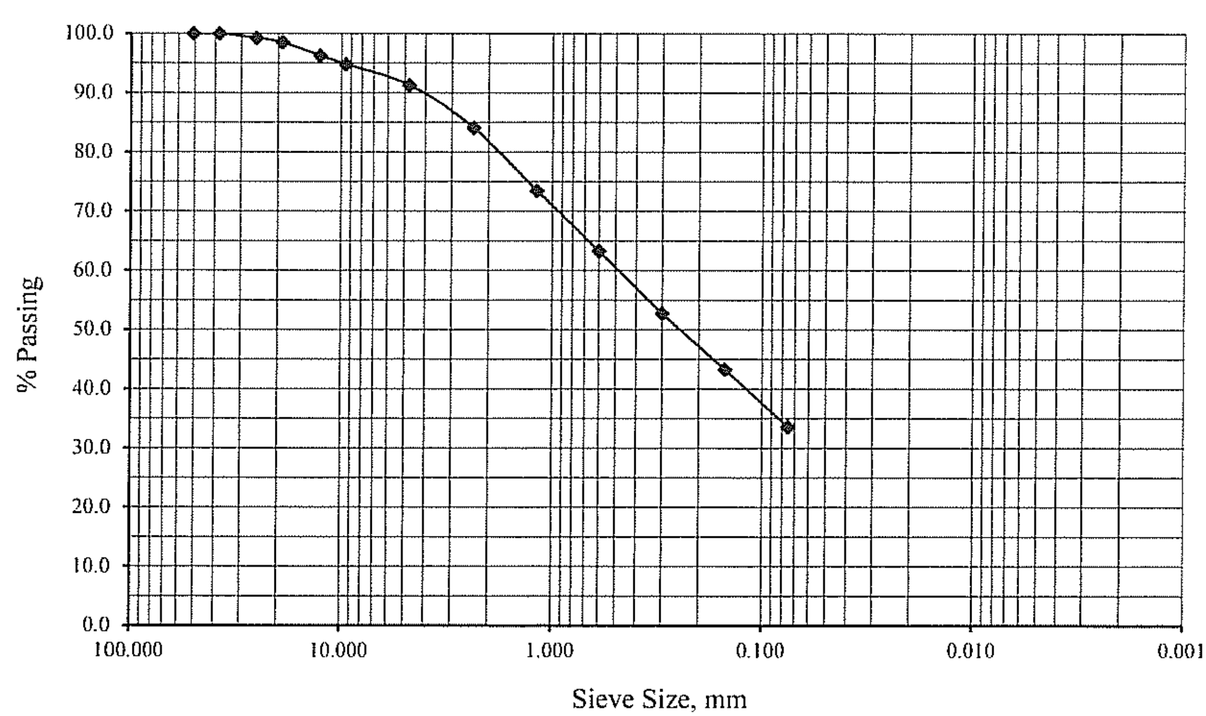
Gradation

ASTM C117 & C136

Project Number: 644-16063 August 2, 2017
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: Bulk 1 @ 0-5' Soil Classification: SM

DRAFT

Sieve Size, in	Sieve Size, mm	Percent Passing
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25.4	99.3
3/4"	19.1	98.5
1/2"	12.7	96.3
3/8"	9.53	94.8
#4	4.75	91.3
#8	2.36	84.1
#16	1.18	73.5
#30	0.60	63.3
#50	0.30	52.8
#100	0.15	43.3
#200	0.075	33.6



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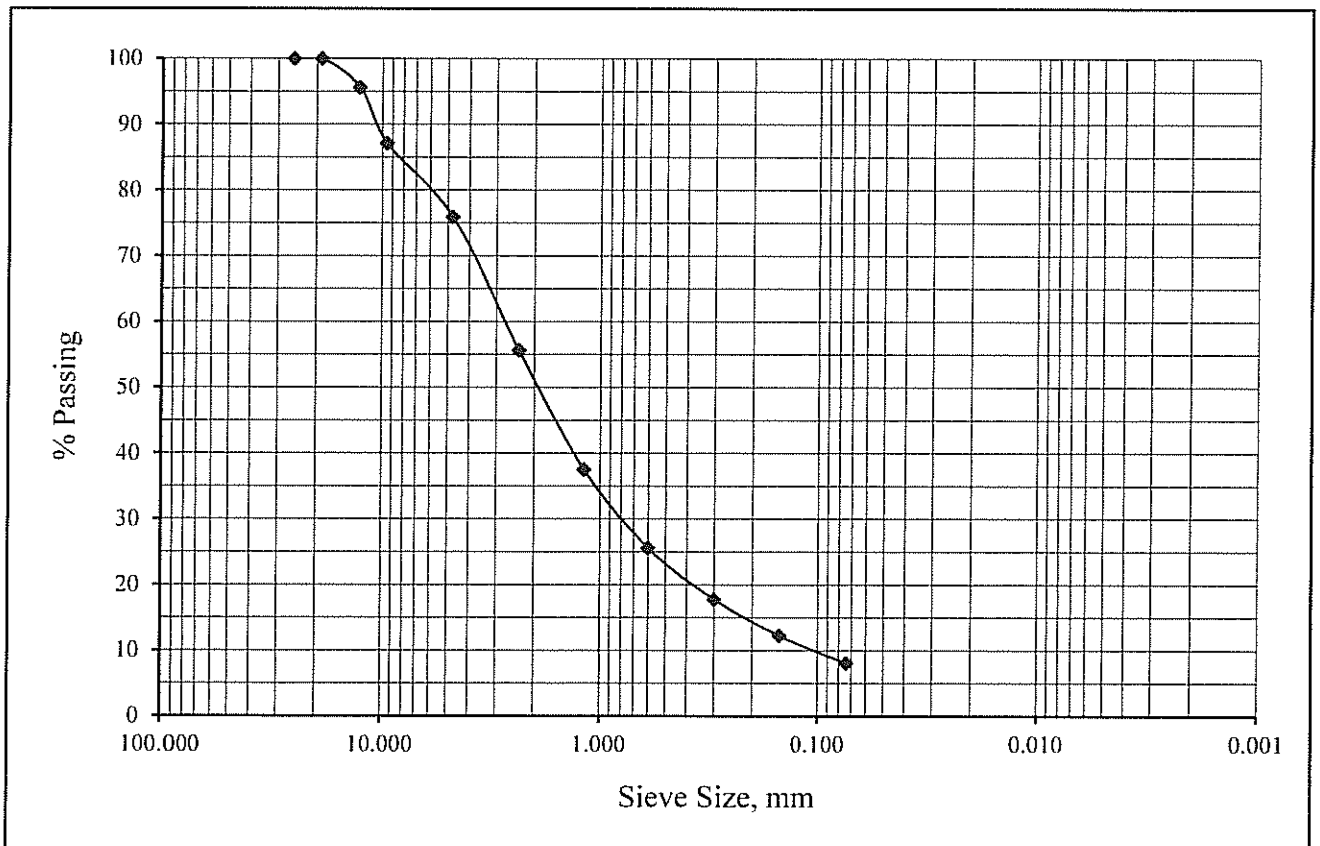
Gradation

ASTM C117 & C136

Project Number: 644-16063 August 2, 2017
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: BH-1 R-3 @ 15' Soil Classification: SW-SM

DRAFT

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	95.6
3/8"	9.53	87.1
#4	4.75	75.9
#8	2.36	55.7
#16	1.18	37.5
#30	0.60	25.6
#50	0.30	17.8
#100	0.15	12.3
#200	0.074	8.1



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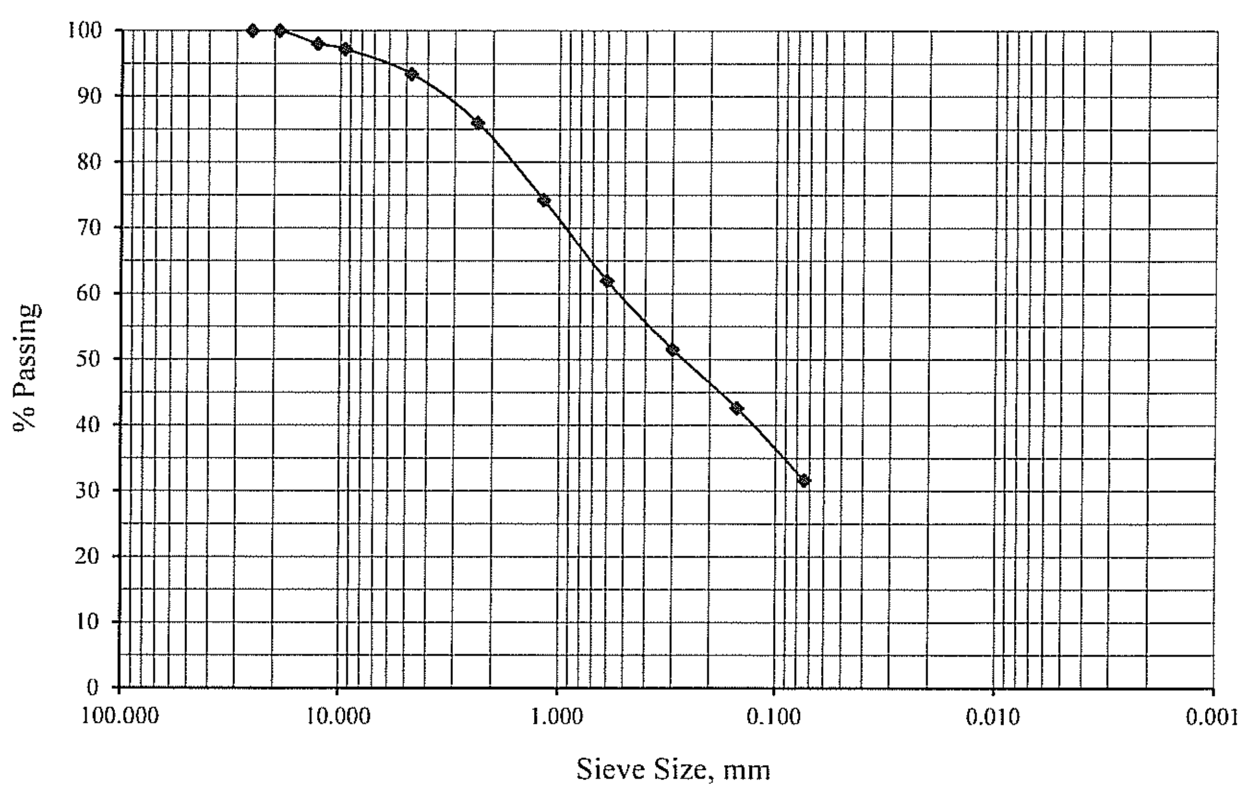
Gradation

ASTM C117 & C136

Project Number: 644-16063 August 2, 2017
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: BH-3 S-5 @ 20' Soil Classification: SM

DRAFT

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	98.0
3/8"	9.53	97.2
#4	4.75	93.4
#8	2.36	86.0
#16	1.18	74.3
#30	0.60	62.0
#50	0.30	51.6
#100	0.15	42.6
#200	0.074	31.7



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Gradation

ASTM C117 & C136

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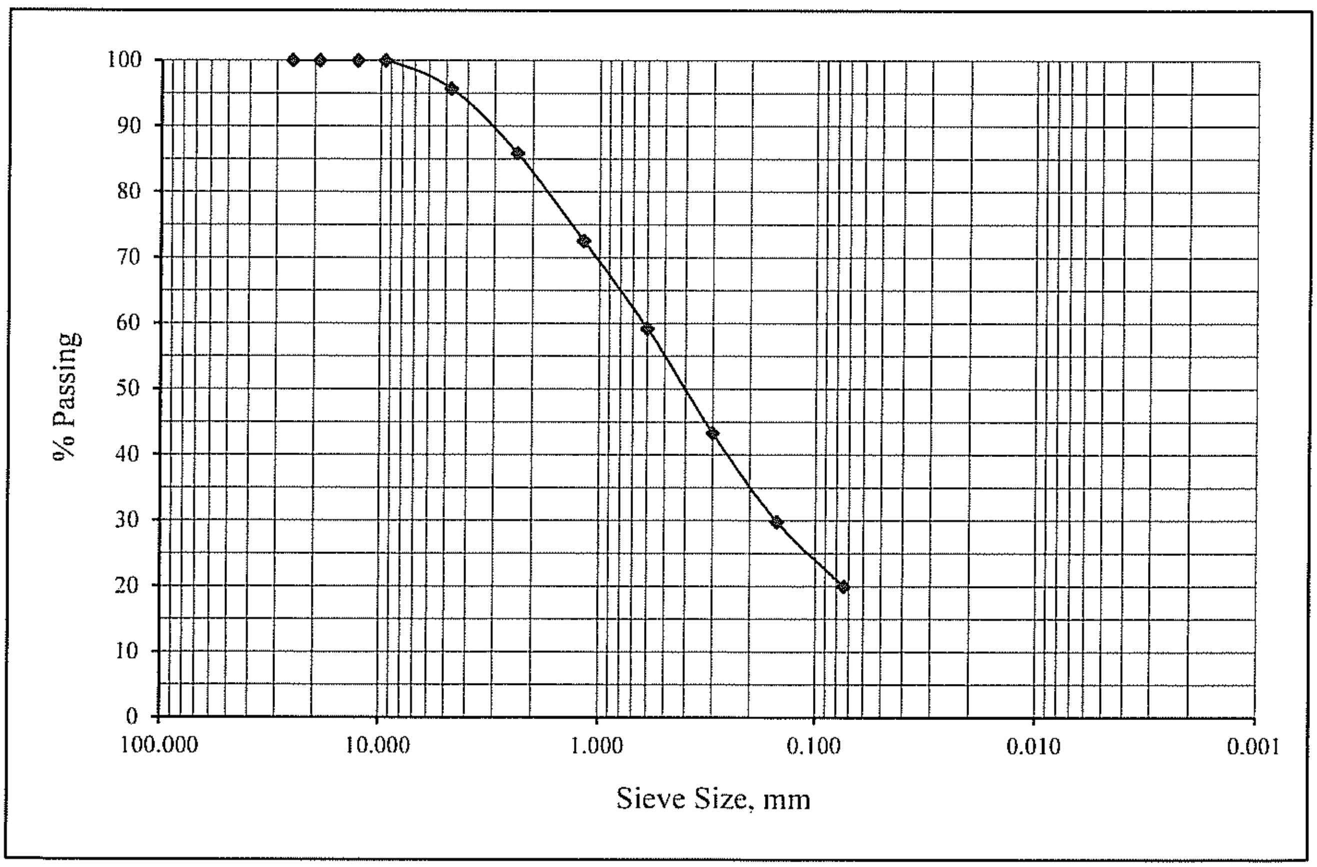
August 2, 2017

Project Number: 644-16063
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: BH-4 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	95.7
#8	2.36	85.8
#16	1.18	72.6
#30	0.60	59.2
#50	0.30	43.3
#100	0.15	29.8
#200	0.074	20.0



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Gradation

ASTM C117 & C136

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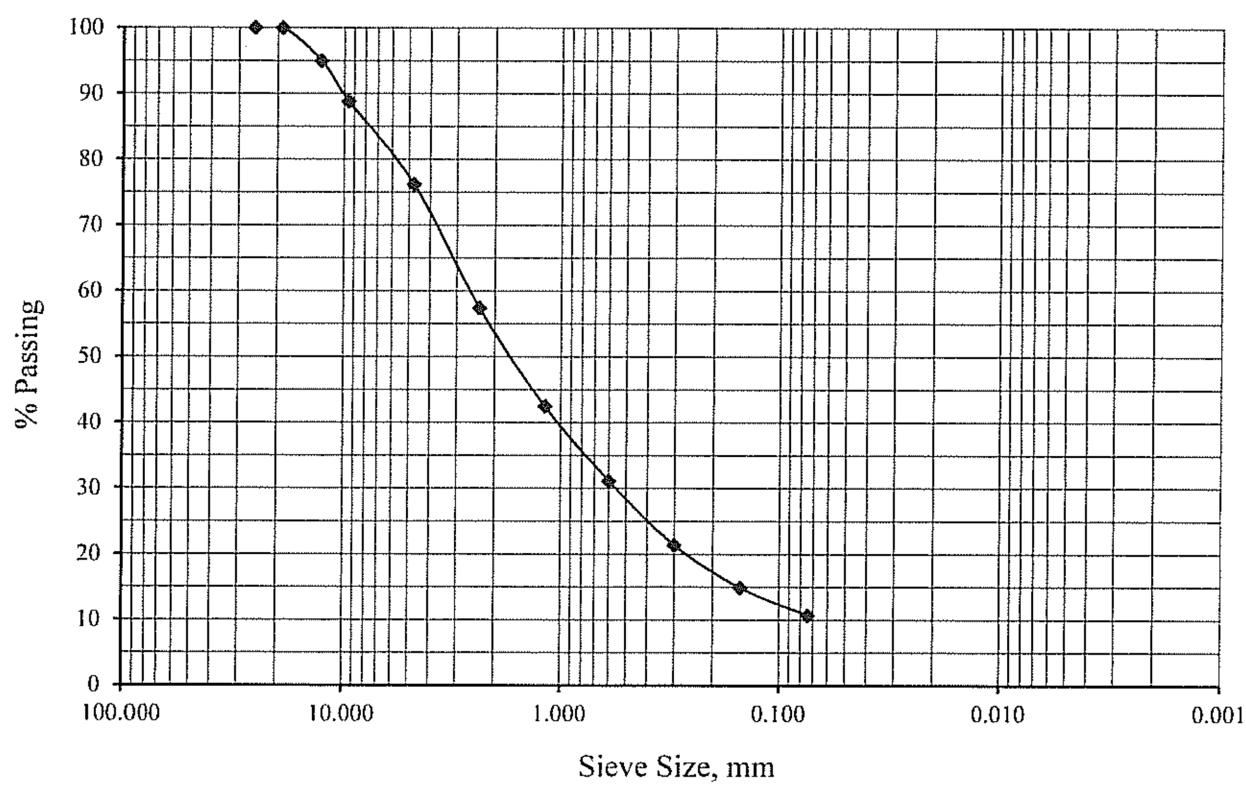
Project Number: 644-16063 August 2, 2017
Project Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: BH-5 R-3 @ 15' Soil Classification: SW-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	95.0
3/8"	9.53	88.8
#4	4.75	76.2
#8	2.36	57.4
#16	1.18	42.5
#30	0.60	31.1
#50	0.30	21.4
#100	0.15	14.9
#200	0.074	10.7



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One Dimensional Consolidation

ASTM D2435 & D5333

DRAFT

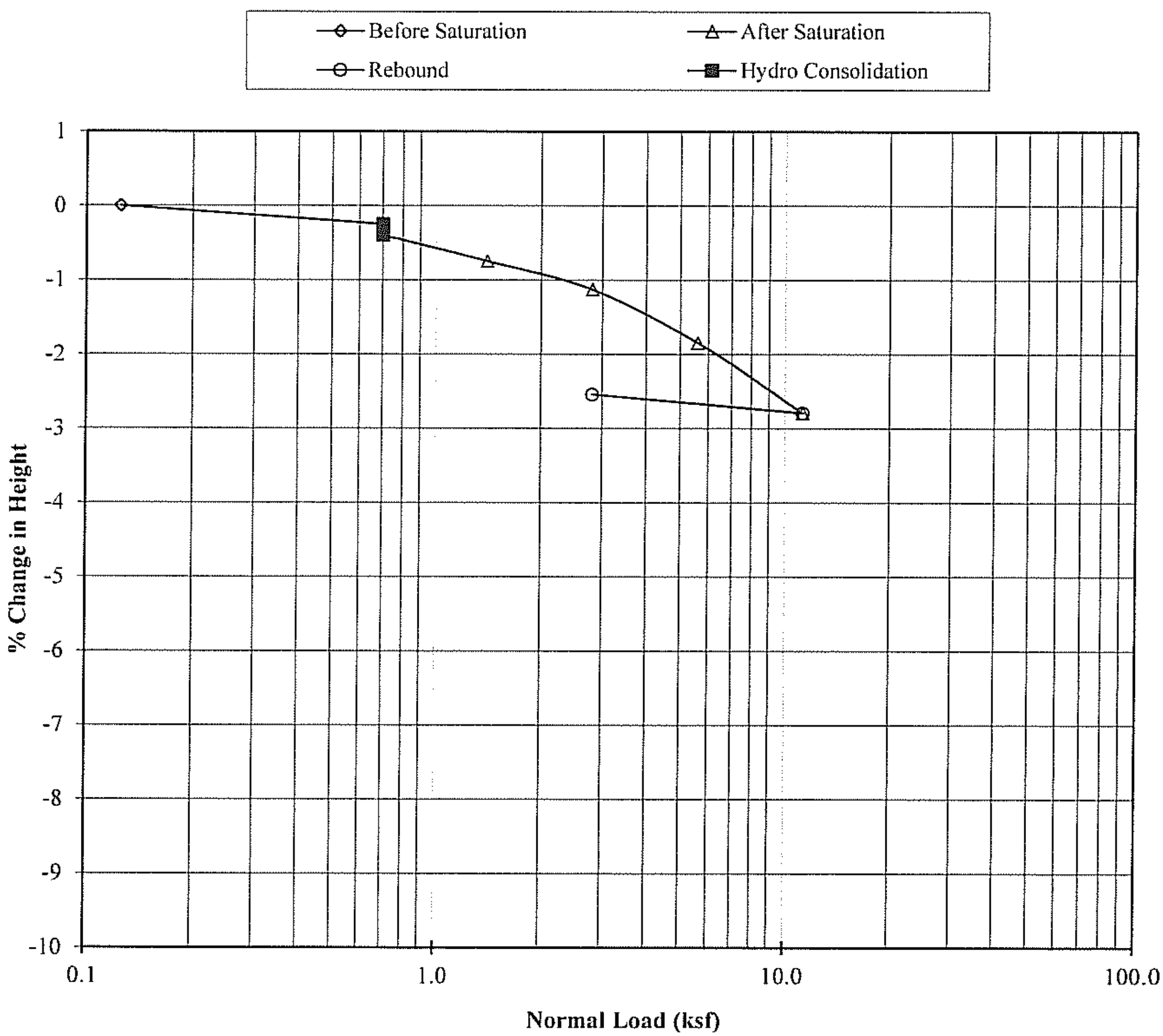
August 2, 2017

Job Number: 644-16063
Job Name: Lakeview Manor Condominium Complex
Lab ID Number: LN6-17322
Sample ID: BH-1 R-1 @ 5'
Soil Description: Brown Sandy Silt (ML)

Initial Dry Density, pcf: 121.9
Initial Moisture, %: 6.1
Initial Void Ratio: 0.368
Specific Gravity: 2.67

Hydrocollapse: 0.2% @ 0.702 ksf

% Change in Height vs Normal Presssure Diagram



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One Dimensional Consolidation

ASTM D2435 & D5333

Job Number: 644-16063

August 2, 2017

Job Name: Lakeview Manor Condominium Complex

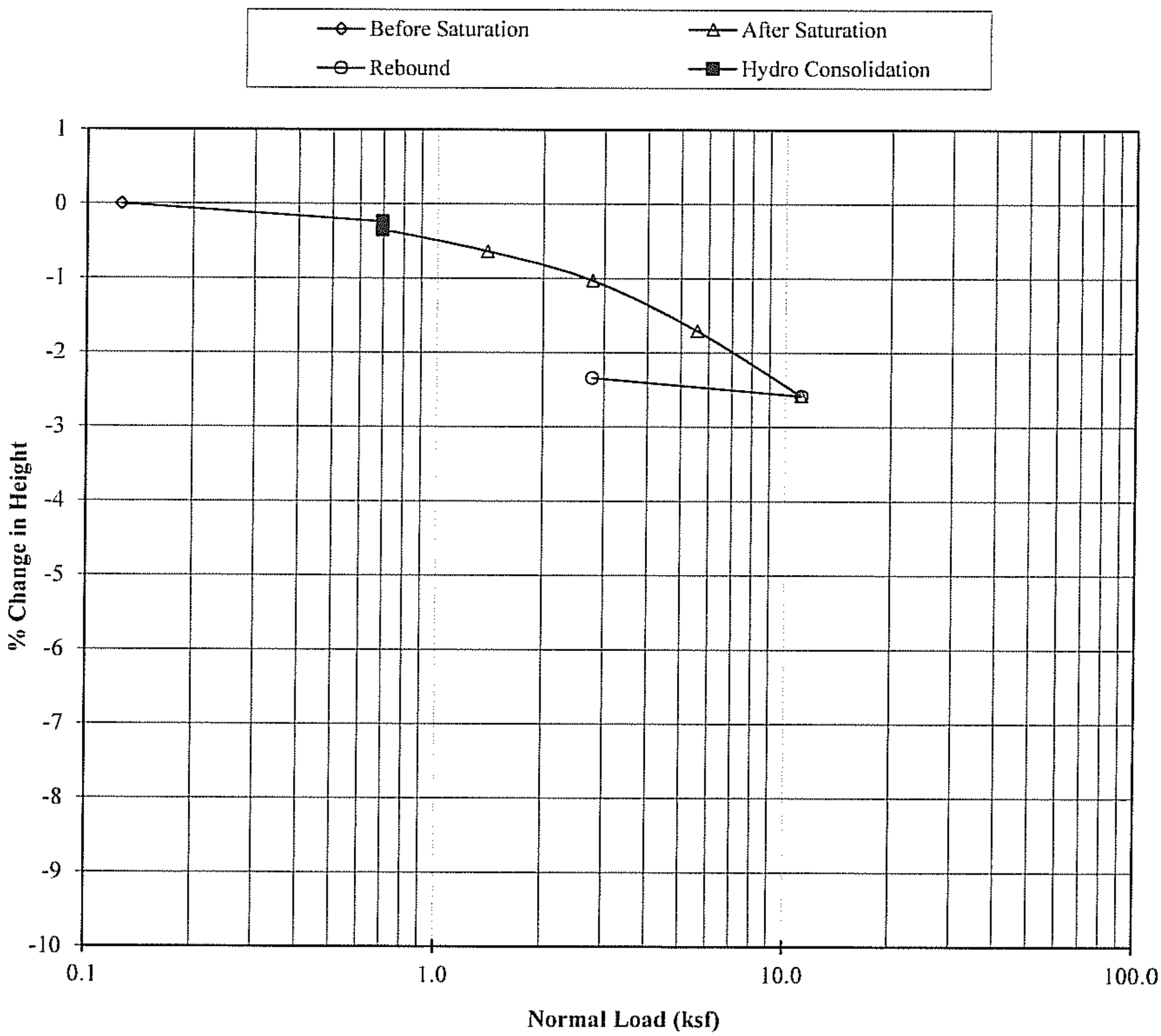
DRAFT

Lab ID Number: LN6-17322
Sample ID: BH-2 R-2 @ 10'
Soil Description: Brown Silty Sand (SM)

Initial Dry Density, pcf:	114.7
Initial Moisture, %:	8.4
Initial Void Ratio:	0.453
Specific Gravity:	2.67

Hydrocollapse: 0.1% @ 0.694 ksf

% Change in Height vs Normal Presssure 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: August 2, 2017

Account No.: 644-16063

Customer: Hong Guan, LLC

Location: Lakeshore Drive, Lake Elsinore

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
Bulk @ 0-5'	8.2	120	60	2800

APPENDIX C

SEISMIC DESIGN MAP AND REPORT
DEAGGREGATION OUTPUT

USGS Design Maps Summary Report

User–Specified Input

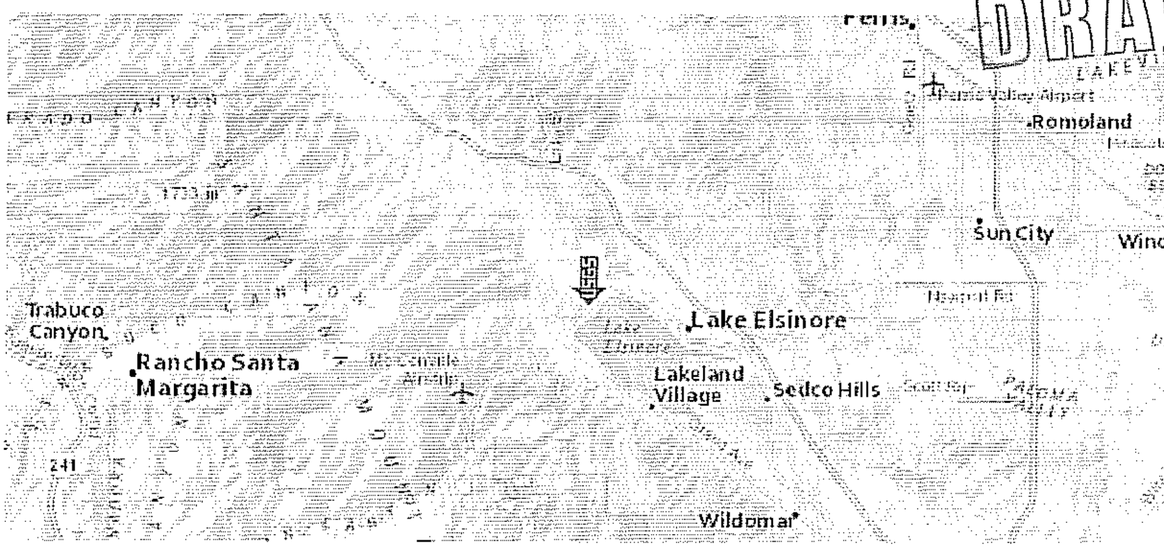
Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.68655°N, 117.3724°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

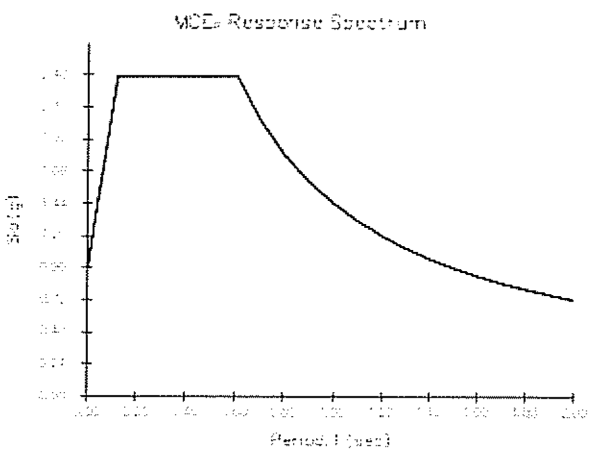
DRAFT

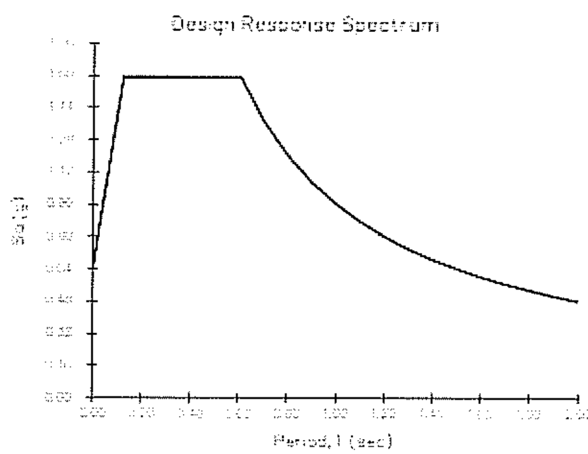


USGS–Provided Output

S_S = 2.387 g	S_{MS} = 2.387 g	S_{DS} = 1.592 g
S_1 = 0.962 g	S_{M1} = 1.443 g	S_{D1} = 0.962 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.68655°N, 117.3724°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.387$ g

From Figure 22-2 [2] $S_1 = 0.962$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

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

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.387 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

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

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

For Site Class = D and S_1 = 0.962 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_a S_S = 1.000 \times 2.387 = 2.387$ g

Equation (11.4-2): $S_{M1} = F_v S_1 = 1.500 \times 0.962 = 1.443$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

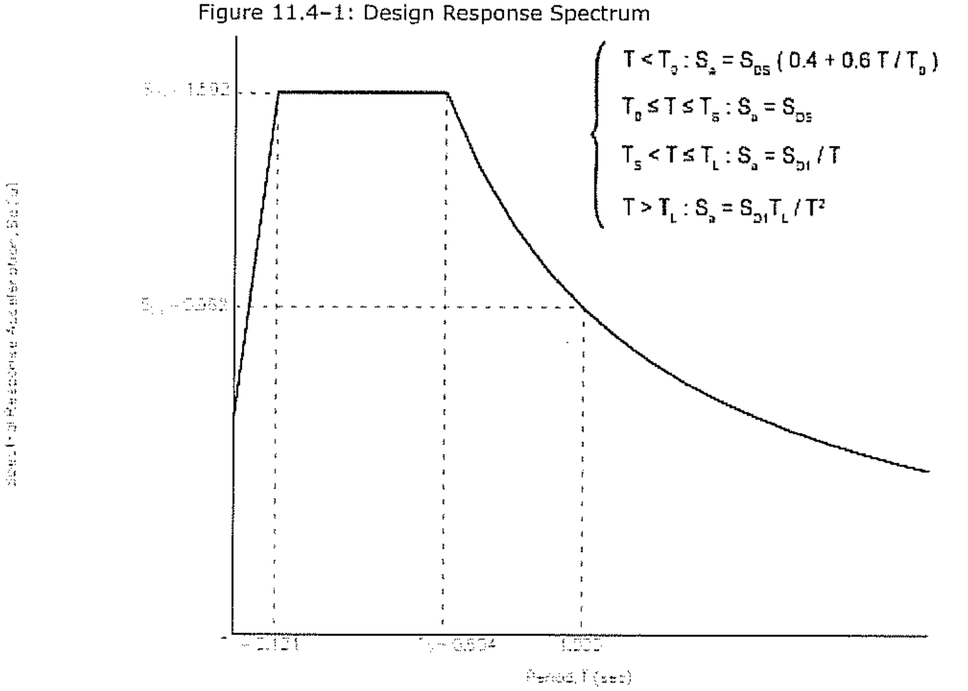
Equation (11.4-3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.387 = 1.592$ g

Equation (11.4-4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.443 = 0.962$ g

Section 11.4.5 — Design Response Spectrum

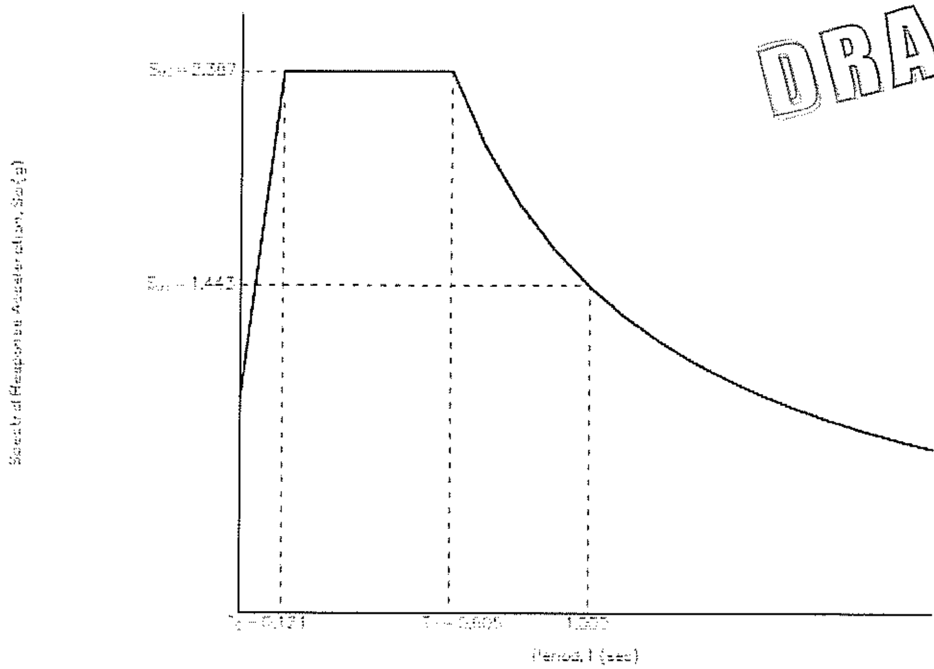
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



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

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



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.953

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.953 = 0.953$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

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

For Site Class = D and PGA = 0.953 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.908

From Figure 22-18 [6] C_{R1} = 0.895

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 1.592 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.962 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

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

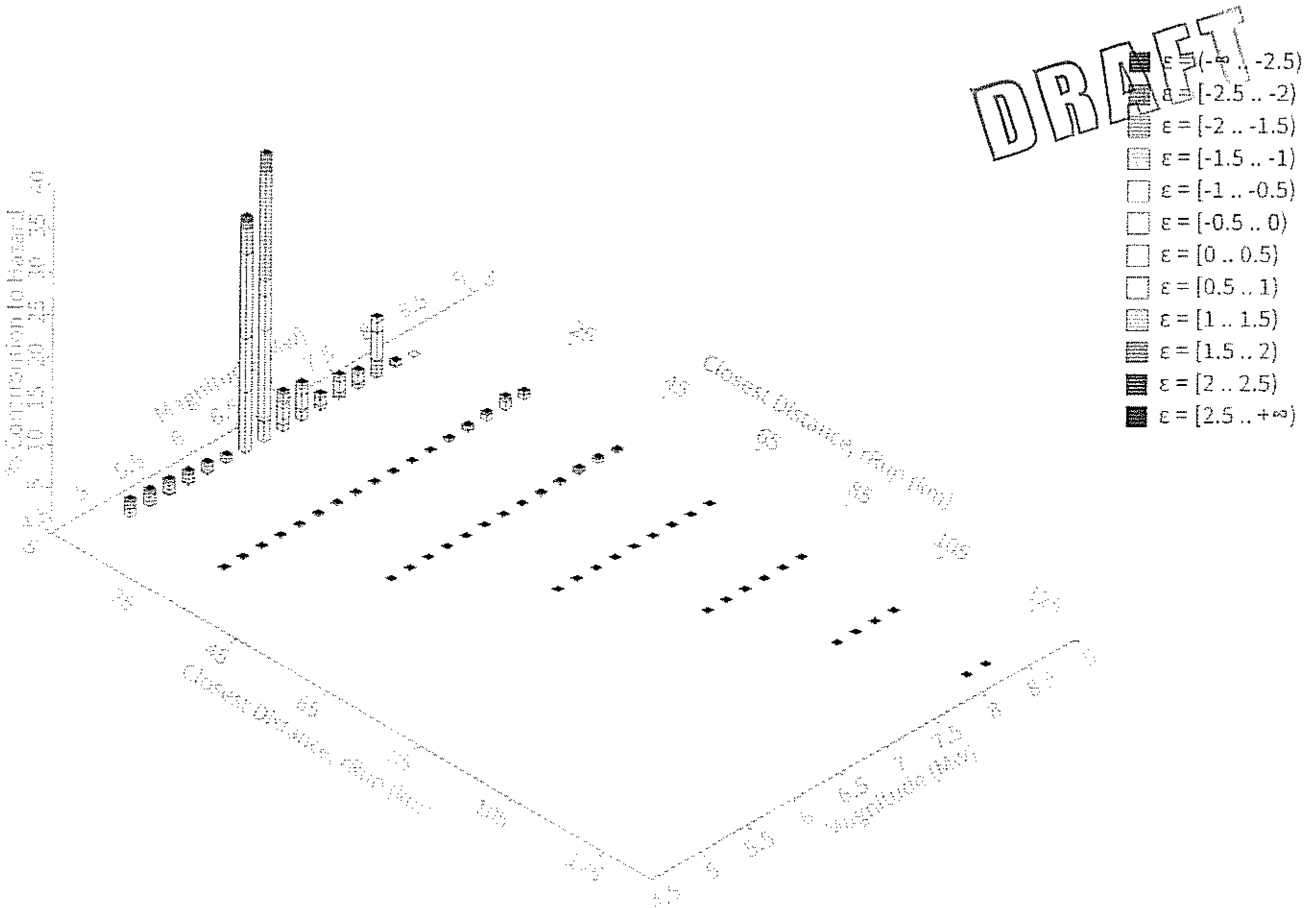
References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.66680255 g

Recovered targets

Return period: 502.63239 yrs
Exceedance rate: 0.0019895256 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.22 %

Mean (for all sources)

r: 6.98 km
m: 6.59
ε_0: 0.52 σ

Mode (largest r-m bin)

r: 5.29 km
m: 6.48
ε_0: 0.54 σ
Contribution: 33.29 %

Mode (largest ε_0 bin)

r: 6.26 km
m: 6.45
ε_0: 1.2 σ
Contribution: 12.51 %

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_FM32	System							44.36
Elsinore (Glen Ivy) rev [3]		1.33	6.70	0.05	117.373°W	33.685°N	190.70	23.14
Elsinore (Glen Ivy) rev [2]		6.53	6.47	0.68	117.428°W	33.721°N	306.61	14.17
Elsinore (Stepovers Combined) [0]		1.38	6.89	-0.02	117.373°W	33.685°N	190.70	1.86
Elsinore (Glen Ivy) rev [1]		12.90	6.43	1.20	117.481°W	33.759°N	308.85	1.75
San Jacinto (San Jacinto Valley) rev [2]		35.55	8.04	1.18	117.111°W	33.921°N	42.83	1.73
UC33brAvg_FM31	System							43.90
Elsinore (Glen Ivy) rev [3]		1.33	6.71	0.05	117.373°W	33.685°N	190.70	22.55
Elsinore (Glen Ivy) rev [2]		6.53	6.46	0.69	117.428°W	33.721°N	306.61	14.05
Elsinore (Stepovers Combined) [0]		1.38	6.83	0.00	117.373°W	33.685°N	190.70	2.06
Elsinore (Glen Ivy) rev [1]		12.90	6.41	1.21	117.481°W	33.759°N	308.85	1.77
San Jacinto (San Jacinto Valley) rev [2]		35.55	8.04	1.17	117.111°W	33.921°N	42.83	1.75
UC33brAvg_FM31 (opt)	Grid							5.88
UC33brAvg_FM32 (opt)	Grid							5.86

DRAFT

*** Deaggregation of Seismic Hazard at One Period of Spectral Acceleration ***
*** Data from Dynamic: Conterminous U.S. 2014 (v4.1.1) ****
PSHA Deaggregation. %contributions.
site: Test
longitude: 117.372°W
latitude: 33.687°E
imt: Peak ground acceleration
vs30 = 259 m/s (Site class D)
return period: 475 yrs.
#This deaggregation corresponds to: Total
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:

Deaggregation targets:
  Return period: 475 yrs
  Exceedance rate: 0.0021052632 yr^{-1}
  PGA ground motion: 0.66680255 g
Recovered targets:
  Return period: 502.63239 yrs
  Exceedance rate: 0.0019895256 yr^{-1}
Totals:
  Binned: 100 %
  Residual: 0 %
  Trace: 0.22 %
Mean (for all sources):
  r: 6.98 km
  m: 6.59
  ε_0: 0.52 σ
Mode (largest r-m bin):
  r: 5.29 km
  m: 6.48
  ε_0: 0.54 σ
  Contribution: 33.29 %
Mode (largest ε_0 bin):
  r: 6.26 km
  m: 6.45
  ε_0: 1.2 σ
  Contribution: 12.51 %
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)