GEOTECHNICAL INVESTIGATION
PROPOSED FACILITIES EXPANSION
PHASE 2.75 & PHASE 3.0
THE LIVING DESERT
47900 PORTOLA AVENUE
PALM DESERT, CALIFORNIA

-Prepared By-

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Project No. 544-21530
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Subject: Geotechnical Investigation

Project: Proposed Facilities Expansion – Phase 2.75 & Phase 3.0
The Living Desert
47900 Portola Avenue
Palm Desert, California

Sladden Engineering is pleased to present the results of the geotechnical investigation performed for the Phase 2.75 and Phase 3.0 facilities expansion project proposed for The Living Desert Zoo complex located at 47900 Portola Avenue in the City of Palm Desert, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated November 18, 2022 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site to provide recommendations for foundation design and the design of the various site improvements. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

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

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

Respectfully submitted,
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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the Phase 2.75 and Phase 3.0 facilities expansion projects proposed for The Living Desert Zoo complex located at 47900 Portola Avenue in the City of Palm Desert, California. The site is located at approximately 33.7007 degrees north longitude and 116.3730 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 plan (Prest Vuksic, Architects, 2022), it is our understanding that Phase 2.75 will consist of constructing a new 8,000 square foot (sf) building with a splash pad feature and accessory structures. Phase 3 will consist of constructing a new 30,000 sf two-story building and a lion habitat with accessory structures. Concrete flatwork, landscape areas and various associated site improvements are also proposed. For our analyses we expect that the proposed new structures will be of relatively lightweight wood-frame, steel-frame or reinforced concrete construction supported on conventional shallow spread footings and concrete slabs-on-grade.

Based on the relatively level nature of the site, Sladden expects that grading will be limited to minor cuts and fills in order to accomplish the desired elevations and to provide adequate gradients for site drainage. This does not include the removal and re-compaction of the near surface soil within the proposed building pad areas. Upon completion of the precise grading plans, Sladden should be retained to verify that the recommendations presented within in this report are properly incorporated into the design of the proposed project.

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

SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- Drilling four (4) exploratory boreholes to depths between approximately 5 to 29 feet bgs to characterize the subsurface soil conditions. The presence of cobbles and boulders resulted in practical auger refusal for BH-2 & BH-3. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- Performing laboratory testing on selected samples to evaluate their engineering characteristics.
- Reviewing geologic literature and discussing geologic hazards.
- Performing site-specific ground motion procedures for the subject property.
- Performing engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

SITE CONDITIONS

The proposed new construction areas are within the northern portion of The Living Desert Zoo complex located at 47900 Portola Avenue in the City of Palm Desert, California. At the time of our investigation, the proposed work areas were occupied by existing animal enclosures, existing structures, landscaped areas, unpaved pathways and paved walkways. The subject property is near the elevation of the adjacent properties and roadways and is bounded on the north, east, west and south by zoo attractions.

Based on our review of the La Quinta Quadrangle (USGS, 2015) and Google Earth (2022), the site is situated at an approximate elevation of 215 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 December 13, 2022. Site drainage appears to be controlled via sheet flow and surface infiltration. Regional drainage is provided by the Whitewater River that is located to the north of the subject site.

GEOLOGIC SETTING

The project site is located within the Colorado Desert Physiographic Province (also referred to as the Salton Trough) that is characterized as a northwest-southeast trending structural depression extending from the Gulf of California to the Banning Pass. The Salton Trough is dominated by several northwest trending faults, most notably the San Andreas Fault system. The Salton Trough is bounded by the Santa Rosa – San Jacinto Mountains on the southwest, the San Bernardino Mountains on the north, the Little San Bernardino - Chocolate – 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 Quaternary-age alluvium (Qal) and dune sand (Qs) deposits. The regional geologic setting for the site vicinity is presented on the Regional Geologic Map (Figure 2).

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling four (4) exploratory boreholes to depths between approximately 5 and 29 feet bgs. The approximate locations of the boreholes are illustrated on the Exploration Location Plan (Figure 3). The boreholes were advanced using a truck-mounted Mobile B-61 drill-rig equipped with 8-inch outside diameter hollow stem augers. A representative of Sladden was on-site to log the materials encountered and retrieve samples for laboratory testing and engineering analysis.

During our field investigation, a thin mantle of artificial fill/disturbed soil was encountered to depths of approximately two (2) to three (3) feet bgs. Underlying the fill/disturbed soil and extending to the maximum depths explored, native alluvium was encountered. The native soil throughout the site consists primarily of fine-to coarse -grained gravelly sand (SW). The native soil was found to be dry to slightly moist, medium dense to very dense, fine-to coarse-grained and yellowish brown in *in-situ* color. The presence of cobbles and boulders resulted in practical auger refusal within BH-2 & BH-3.

The presence of existing site improvements limited access for our geotechnical investigation. However, Sladden has reviewed previous geotechnical engineering reports prepared by Sladden Engineering (2022) and Earth Systems (2019) for The Living Desert Phase 2.5 Facilities Expansion and Living Desert Crossroads of Conservation Project reports, respectively. These reports document soil conditions consistent with the conditions summarized in this report.

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

Groundwater was not encountered to the maximum explored depth of 51.5 feet bgs during our field investigation. Based upon our review of the groundwater levels in the site vicinity (Tyley, 1974), it is our opinion that groundwater will not be a factor in the design and construction of the proposed project.

SEISMICITY AND FAULTING

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

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

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

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

TABLE 1
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
San Andreas - Coachella	15.2	7.2
San Andreas - Southern	15.2	7.2
San Andreas - San Bernardino	27.0	7.5
Burnt Mountain	27.6	6.5
San Jacinto - Anza	28.4	7.2
San Jacinto - Coyote Creek	29.4	6.8
Eureka Peak	30.3	6.4
Pinto Mountain	45.9	7.0

SITE SPECIFIC GROUND MOTION PARAMETERS

Sladden has reviewed the 2022 California Building Code (CBC) and ASCE7-16 and developed site specific ground motion parameters for the subject site. The project Seismic Design Maps and site-specific ground motion parameters are summarized in the following table and included within Appendix C. The project Structural Engineer should verify that all design parameters provided are applicable for the subject project.

TABLE 2
GROUND MOTION PARAMETERS

Latitude / Longitude	33.7007/ -116.3730
Risk Category	II
Site Class	D
Code Reference Documents	ASCE 7-16; Chapter 11 & 21

Description	**Type**	**Map Based**	**Site-Specific**
MCE$_R$ Ground Motion (0.2 second period)	S_S	1.500	---
MCE$_R$ Ground Motion (1.0 second period)	S_1	0.600	---
Site-Modified Spectral Acceleration Value	S_{MS}	1.500	**1.441**
Site-Modified Spectral Acceleration Value	S_{M1}	null	**1.200**
Numeric Seismic Design Value at 0.2 second SA	S_{DS}	1.000	**0.961**
Numeric Seismic Design Value at 1.0 second SA	S_{D1}	null	**0.800**
Site Amplification Factor at 0.2 second	F_a	1.0	**1.0**
Site Amplification Factor at 1.0 second	F_v	null	**2.5**
Site Peak Ground Acceleration	PGA_M	0.594	**0.550**

GEOLOGIC HAZARDS

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

I. Surface Rupture. Surface rupture is expected to occur along preexisting, known active fault traces. However, surface rupture could potentially splay or step from known active faults or rupture along unidentified traces. Based on our review of Rogers (1965), Jennings (1994), and CGS (2023) known active faults are not mapped on the site. In addition, no signs of active surface faulting were observed during our review of non-stereo digitized photographs of the site and site vicinity (Google Earth, 2023). Finally, no signs of active surface fault rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified during our field investigation. Therefore, it is our opinion that risks associated with primary surface ground rupture should be considered "low".

II. Ground Shaking. The site has been subjected to past ground shaking by faults that traverse through the region. Strong seismic shaking from nearby active faults is expected to produce strong seismic shaking during the design life of the proposed project. Based on site-specific ground motion parameters developed for the property (Appendix C), the site modified peak ground acceleration (PGAm) is estimated to be 0.550g.

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

Based on the depth to groundwater in the project vicinity (Tyley, 1974), risks associated with liquefaction are considered "negligible".

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

V. Slope Failure, Landsliding, Rock Falls. Slope instability in the form of landslides and rock falls were not observed at or near the subject site. The site is situated on relatively flat ground and is not located immediately adjacent to any slopes. As such, risks associated with slope instability (landslides, mass wasting and rock falls) are considered "negligible".

VI. Expansive Soil. Generally, the near surface soil consists of Gravelly Sand (SW/SP). Based on the results of our laboratory testing, the materials underlying the site are considered "non-expansive".

VII. Static Settlement. Static settlement resulting from the anticipated foundation loads should be tolerable provided that the recommendations included in this report are considered in foundation design and construction. The ultimate static settlement is estimated to be less than 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 static settlement.

VIII. Subsidence. Land subsidence can occur in valleys where aquifer systems have been subjected to extensive groundwater pumping, such that groundwater pumping exceeds groundwater recharge. Generally, pore water reduction can result in a rearrangement of skeletal grains and could result in elastic (recoverable) or inelastic (unrecoverable) deformation of an aquifer system.

Locally, no fissures or other surficial evidence of subsidence were observed at or near the subject site. However, site specific effects resulting from long term regional subsidence is beyond the scope of our investigation.

IX. Debris Flows. Debris flows are viscous flows consisting of poorly sorted mixtures of sediment and water and are generally initiated on slopes steeper than approximately six horizontal to one vertical (6H:1V) (Boggs, 2001). Based on the flat nature of the site and the composition of the surface soil, we judge that the risks associated with debris flows should be considered "negligible".

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

CONCLUSIONS

Based on the results of our investigation and our review of previous geotechnical reports prepared for The Living Desert, 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 existing surface improvements, existing underground utilities, presence of artificial fill soil and loose and potentially compressible near surface soil.

The proposed new structures may be supported upon conventional shallow spread footings. We recommend that remedial work within the proposed new building areas include over-excavation and re-compaction of the artificial fill soil and 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. Based on our observations of the materials encountered, we anticipate that the subsoil will conform to that described by CalOSHA as Type C. Soil conditions should be verified in the field by a "Competent person" employed by the Contractor.

EARTHWORK AND GRADING

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

a. Site Clearing: Areas to be graded should be cleared of any existing building elements, surface improvements, vegetation and trees, associated root systems, existing utilities and debris. All areas scheduled to receive fill should be cleared of old fills and any irreducible matter. The unsuitable materials should be removed off site. 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 provide firm and uniform foundation bearing conditions, we recommend over-excavation and re-compaction throughout the proposed building areas. All disturbed and/or low density near surface soil should be removed to a depth of at least 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 5 feet beyond the footing limits. The native soil exposed by over-excavation should be scarified, moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction prior to fill placement. The previously removed soil may then be replaced as engineered fill as recommended below.

c. Compaction: Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances. All fill material should be placed in thin lifts, not exceeding six inches in a loose condition. If import fill is required, the material should be of a low to non-expansive nature and should meet the following criteria:

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

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

TABLE 3
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Excavation and re-compaction within the building envelope and extending laterally for 5 feet beyond the building limits and to a minimum of 3 feet below existing grade 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, at near optimum moisture content and compact to a minimum of 90 percent relative compaction.
Concrete Sections	Compact the top 12 inches to at least 95 percent compaction at near optimum moisture content.

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

d. <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 3 tenths of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

CONVENTIONAL SHALLOW SPREAD FOOTINGS

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

Based on the recommended allowable bearing pressures, the total static settlement of the shallow footings is anticipated to be less than one-inch, provided foundation area preparation conforms with the recommendations included 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 0.45 may be used for dead and sustained live loads to compute the frictional resistance of the footing placed directly on compacted fill. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

All footing excavations should be observed by a representative of the project geotechnical consultant to verify adequate embedment depths prior to placement of forms, steel reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, disturbed, sloughed or moisture-softened soils and/or any construction debris should be removed prior to concrete placement. 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 foundation support, concrete slabs-on-grade must be placed on properly compacted engineered fill soil as outlined in the previous sections of this report. The slab subgrade should remain near optimum moisture content and should not be permitted to dry prior to concrete placement. Slab subgrade should be firm and unyielding. Disturbed soil should be removed and replaced with engineered fill soil compacted to a minimum of 90 percent relative compaction.

Slab thickness and reinforcement should be determined by the Structural Engineer. We recommend a minimum slab thickness of 4.0 inches and minimum reinforcement of #3 bars at 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. Final floor slab design and reinforcement should be determined by the Structural Engineer.

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

RETAINING WALLS

Minor retaining walls may be necessary to complete the proposed construction. Cantilever retaining walls may be designed using "active" pressures. Active pressures may be estimated using an equivalent fluid weight of 35 pcf for level native backfill soil acting in a triangular pressure distribution with drained backfill conditions. "At Rest" pressures should be utilized for restrained walls. At rest pressures may be estimated using an equivalent fluid weight of 55 pcf for native backfill soil with level drained backfill conditions.

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

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

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 20 parts per million (ppm). The soil is considered to have a "negligible – S0" corrosion potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes should not be necessary. The soluble sulfate content of the surface soil should be reevaluated after grading and appropriate concrete mix designs should be established based upon post-grading test results.

The pH level of the surface soil was 7.6. Based on soluble chloride concentration testing (60 ppm) the soil is considered to have "negligible-S0" corrosion potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 4,650 ohm-cm which suggests the site soil is considered to have "moderate" corrosion potential with respect to ferrous metal installations.

UTILITY TRENCH BACKFILL

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

EXTERIOR CONCRETE FLATWORK

In order to provide uniform support and minimize settlement related cracking of concrete flatwork, the subgrade soil within concrete flatwork areas should be compacted to a minimum of 90 percent relative compaction. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soil prior to concrete placement.

DRAINAGE

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

LIMITATIONS

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

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

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

ADDITIONAL SERVICES

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

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

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Tyley, S.J., 1974, Analog Model Study of the Ground-Water Basin of the Upper Coachella Valley, California, Geological Survey Water-Supply Paper 2027.

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United States Geological Survey (USGS), 2023b, Unified Hazard Tool; available at: https://geohazards.usgs.gov/hazards/interactive/

United States Geological Survey (USGS), 2023c, Risk Targeted Ground Motion Calculator; available at: https://earthquake.usgs.gov/designmaps/rtgm/

FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
EXPLORATION LOCATION PLAN

Source: USGS (2015)

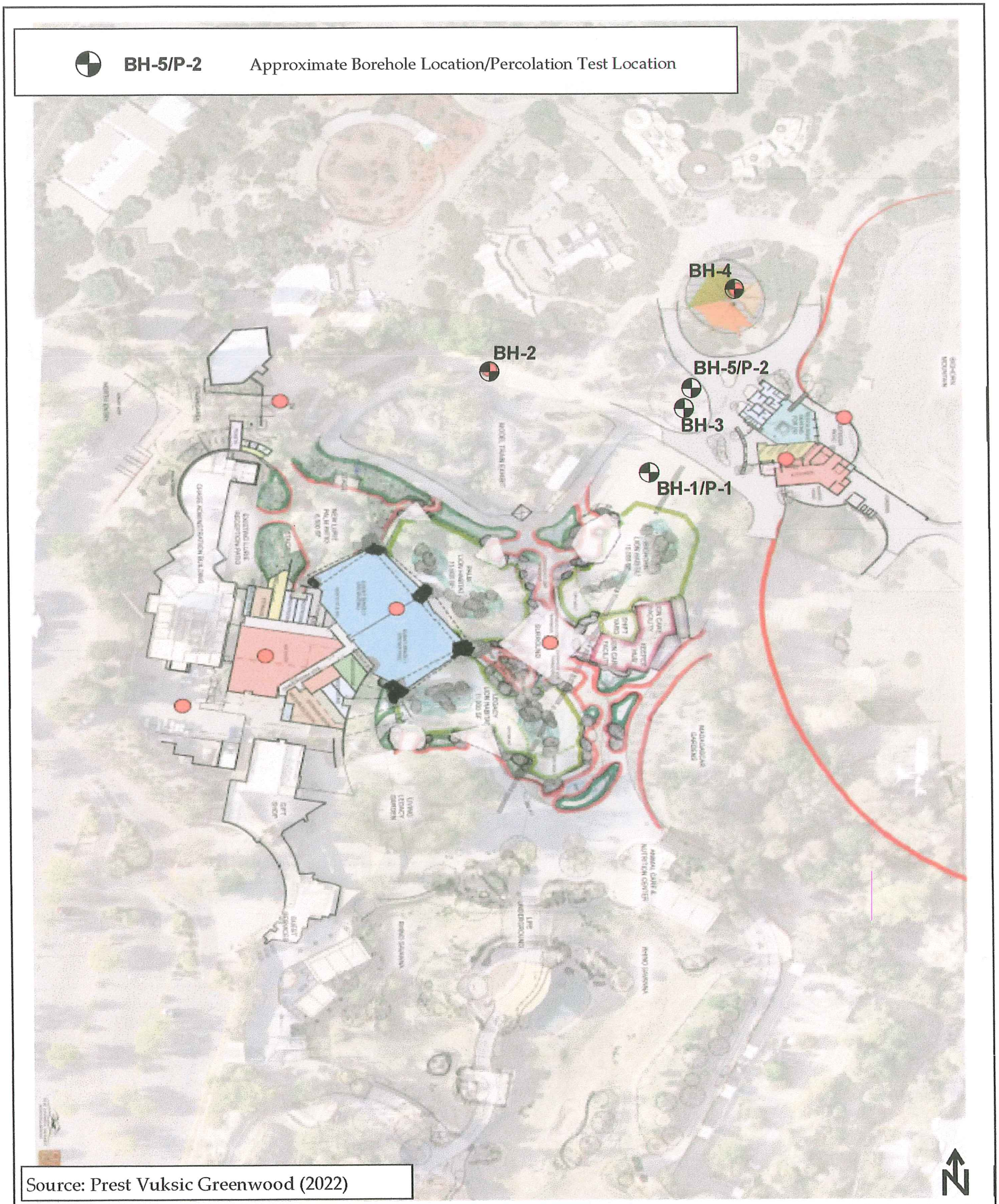
Sladden Engineering

SITE LOCATION MAP

Project Number:	544-21530
Report Number:	23-01-022
Date:	January 18, 2022

FIGURE

1



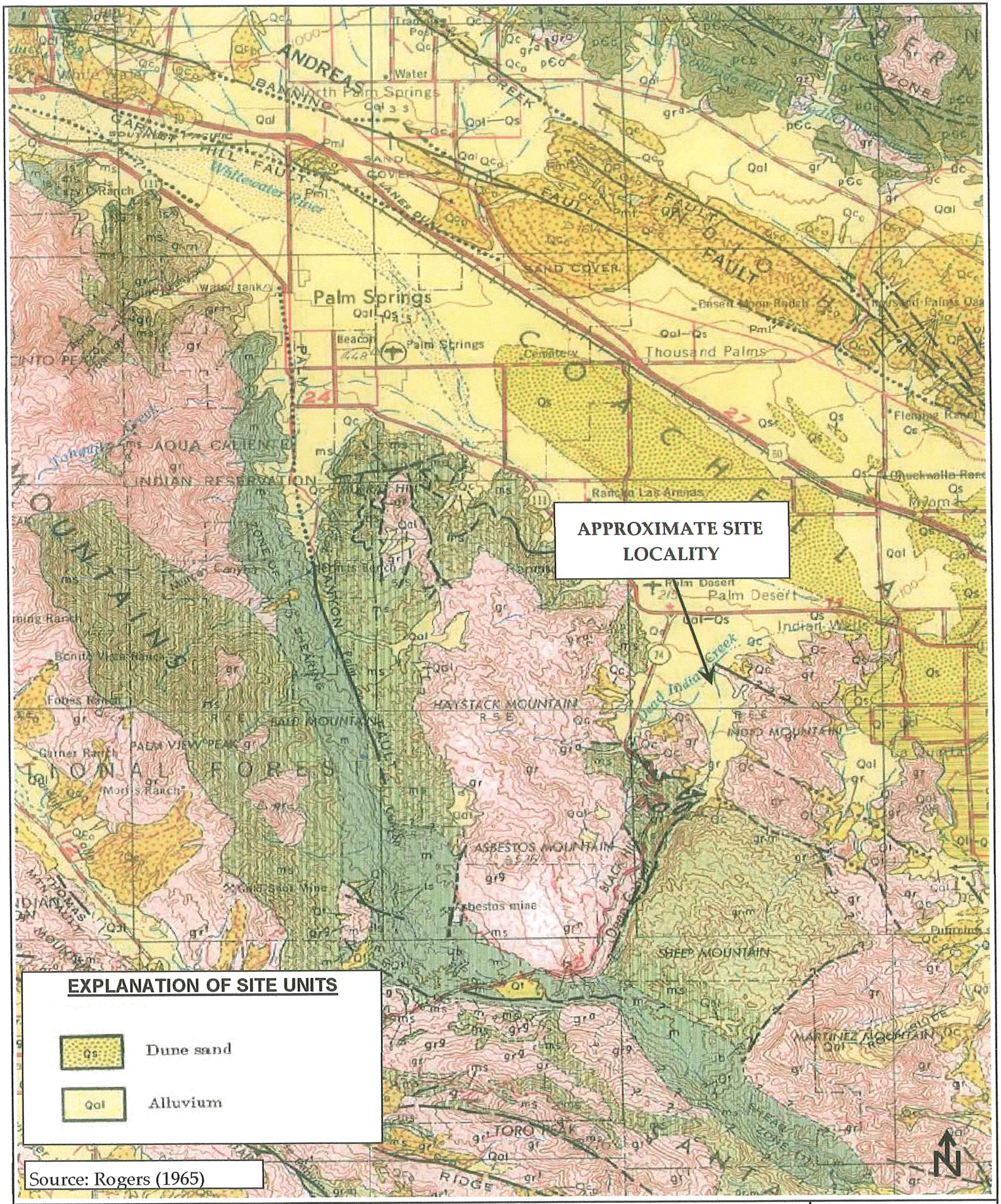
Source: Prest Vuksic Greenwood (2022)

EXPLORATION LOCATION PLAN

Sladden Engineering

Project Number:	544-21530
Report Number:	23-01-022
Date:	January 18, 2022

FIGURE 3



EXPLANATION OF SITE UNITS

Qs	Dune sand
Qal	Alluvium

Source: Rogers (1965)

Sladden Engineering

REGIONAL GEOLOGIC MAP

Project Number:	544-21530
Report Number:	23-01-022
Date:	January 18, 2022

FIGURE

2

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation four (4) exploratory boreholes were excavated on December 13, 2022 utilizing a truck mounted hollow stem auger rig (Mobile B-61). Continuous logs of the materials encountered were made by a representative of Sladden Engineering. Materials encountered in the boreholes were classified in accordance with the Unified Soil Classification System which is presented in this appendix.

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

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

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	12/13/2022
Elevation:	385 Ft (MSL)	Boring No:	BH-1/P-1

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2	░	Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained (Fill/Disturbed).
							4	░	Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained (Qal-Qs).
							6		
							8		Terminated at ~ 5.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered. Borehole Cased with Perforated Pipe for Percolation Testing.
							10		
							12		
							14		
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PHASE 2.75 & 3.0
THE LIVING DESERT

Project No: 544-21530
Report No: 23-01-022

Page 1

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	12/13/2022
Elevation:	385 Ft (MSL)	Boring No:	BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2	░	Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained with gravel (Fill/Disturbed).
							4	░	
▮	10/13/13			8.4	1.0		6	░	Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained (Qal-Qs).
							8	░	
■	13/14/17			4.5	0.5	114.4	10	░	Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained (Qal-Qs).
							12		
							14		Practical Auger Refusal at ~ 12.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PHASE 2.75 & 3.0
THE LIVING DESERT

Project No: 544-21530
Report No: 23-01-022

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	12/13/2022
Elevation:	385 Ft (MSL)	Boring No:	BH-3

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
■	9/14/23	1	0	3.3	0.5	120.6	2	░	Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with gravel (Fill/Disturbed).
■	14/24/27			7.5	1.0	119.7	4 6 8	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
◨	13/17/17			6.9	1.3		10 12 14	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
■	38/41/42			3.6	1.0	132.8	16 18	░	Gravelly Sand (SW); yellowish brown, dry, very dense, fine-to-coarse grained (Qal-Qs).
◨	17/19/25			9.7	2.0		20 22 24	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
☒	32/50-5"						26 28	░	No Recovery.
							30 32 34 36 38 40 42 44 46 48 50		Practical Auger Refusal at ~ 29.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.

Completion Notes:

PHASE 2.75 & 3.0
THE LIVING DESERT

Project No: 544-21530
Report No: 23-01-022

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	12/13/2022
Elevation:	385 Ft (MSL)	Boring No:	BH-4

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2	░	Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with gravel (Fill/Disturbed).
■	50-6"			4.1	0.3		4 6 8	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
▣	8/14/21			6.8	2.6		10 12 14	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
■	19/31/50-6"			4.7	2.3	124.8	16 18	░	Gravelly Sand (SW); yellowish brown, dry, very dense, fine-to-coarse grained (Qal-Qs).
▣	17/23/25			8.1	2.4		20 22	░	Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained (Qal-Qs).
							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:

PHASE 2.75 & 3.0
THE LIVING DESERT

Project No: 544-21530
Report No: 23-01-022

SLADDEN ENGINEERING

BORE LOG

Drill Rig:	Mobile B-61	Date Drilled:	12/13/2022
Elevation:	385 Ft (MSL)	Boring No:	BH-5/P-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
							2	▒	Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained (Fill/Disturbed).
							4	▒	
■	13/18/22			5.4	0.8	126.8	6	▒	Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained (Qal-Qs).
							8	▒	
▮	14/15/15			5.4	0.69		10		
							12		Terminated at ~ 10.0 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered. Borehole Cased with Perforated Pipe for Percolation Testing.
							14		
							16		
							18		
							20		
							22		
							24		
							26		
							28		
							30		
							32		
							34		
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PHASE 2.75 & 3.0
THE LIVING DESERT

Project No: 544-21530
Report No: 23-01-022

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

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

CLASSIFICATION AND COMPACTION TESTING

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

Maximum Density-Optimum Moisture Determinations: Representative soil types were selected for maximum density determinations. This testing was performed in accordance with the ASTM Standard D1557-91, Test Method A. Graphic representations of the results of this testing are presented in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

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

SOIL MECHANIC'S TESTING

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

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

Consolidation/Hydro-Collapse Testing: One (1) relatively undisturbed sample was selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load.

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

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Maximum Density/Optimum Moisture

ASTM D698/D1557

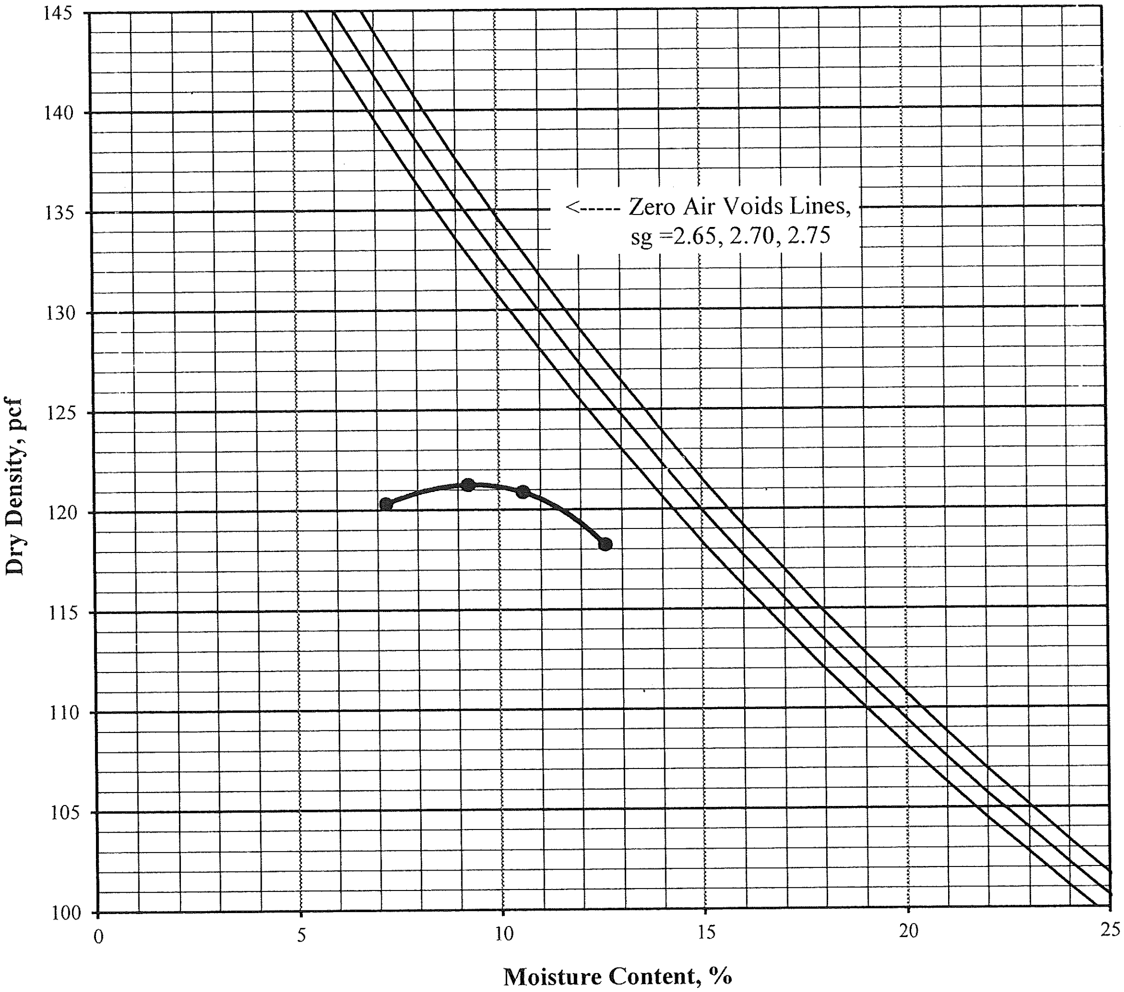
January 17, 2023

Project Number:	544-21530
Project Name:	The Living Desert
Lab ID Number:	LN6-22600
Sample Location:	BH-3 Bulk 1 @ 0-5'
Description:	Brown Sand w/Silt & Gravel (SW-SM)

ASTM D-1557 A
Rammer Type: Machine

Maximum Density: **126.5 pcf**
Optimum Moisture: **8.5%**
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	
3/8"	
#4	15.4



Expansion Index

ASTM D 4829

January 17, 2023

Job Number:	544-21530
Job Name:	The Living Desert
Lab ID Number:	LN6-22600
Sample ID:	BH-3 Builk 1 @ 0-5'
Soil Description:	Brown Sand w/Silt & Gravel (SW-SM)

Wt of Soil + Ring:	577.8
Weight of Ring:	191.9
Wt of Wet Soil:	385.9
Percent Moisture:	8.8%
Sample Height, in	0.95
Wet Density, pcf:	123.5
Dry Denstiy, pcf:	113.5

% Saturation:	49.0

Expansion **Rack # 2**

Date/Time	1/16/2023	2:15 PM
Initial Reading	0.0000	
Final Reading	0.0000	

Expansion Index **0**

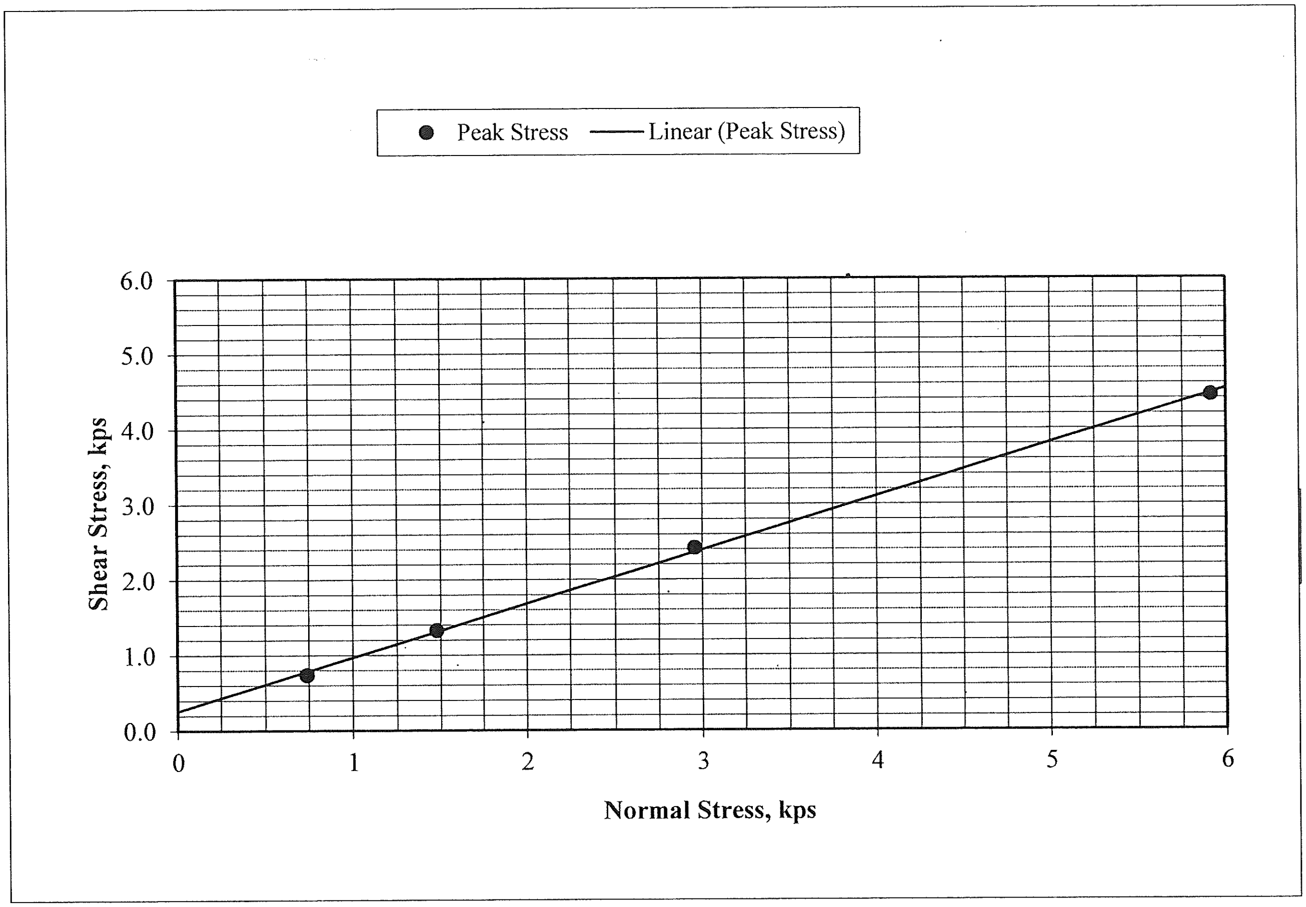
(Final - Initial) x 1000

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

Job Number: 544-21530
Job Name The Living Desert
Lab ID No. LN6-22600
Sample ID BH-3 Bulk 1 @ 0-5'
Classification Brown Sand w/Silt & Gravel (SW-SM)
Sample Type Remolded @ 90% of Maximum Density

January 17, 2023
Initial Dry Density: 109.4 pcf
Initial Mosture Content: 10.0 %
Peak Friction Angle (Ø): 35°
Cohesion (c): 260 psf

Test Results	1	2	3	4	Average
Moisture Content, %	15.3	15.3	15.3	15.3	15.3
Saturation, %	76.6	76.6	76.6	76.6	76.6
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.741	1.330	2.420	4.447	



Job Number: 544-21530
Job Name: The Living Desert
Date: 1/17/2023

Moisture Adjustment		Remolded Shear Weight	
Wt of Soil:	1,000	Max Dry Density:	121.5
Moist As Is:	1.3	Optimum Moisture:	10.0
Moist Wanted:	10.0		
ml of Water to Add:	85.9	Wt Soil per Ring, g:	144.6

UBC

Gradation

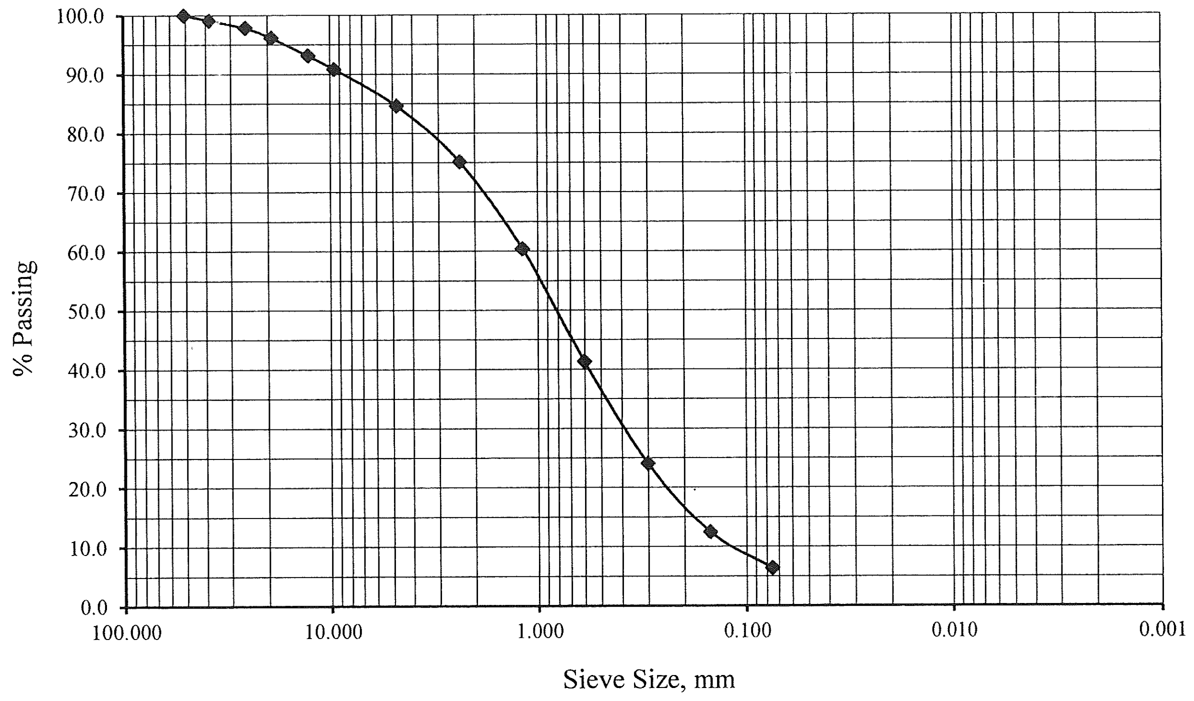
ASTM C117 & C136

January 17, 2023

Project Number: 544-21530
Project Name: The Living Desert
Lab ID Number: LN6-22600
Sample ID: BH-3 Bulk 1 @ 0-5'

Soil Classification: SW-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
2"	50.8	100.0
1 1/2"	38.1	99.2
1"	25.4	97.9
3/4"	19.1	96.2
1/2"	12.7	93.2
3/8"	9.53	90.9
#4	4.75	84.6
#8	2.36	75.2
#16	1.18	60.4
#30	0.60	41.3
#50	0.30	24.1
#100	0.15	12.5
#200	0.075	6.4



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Gradation

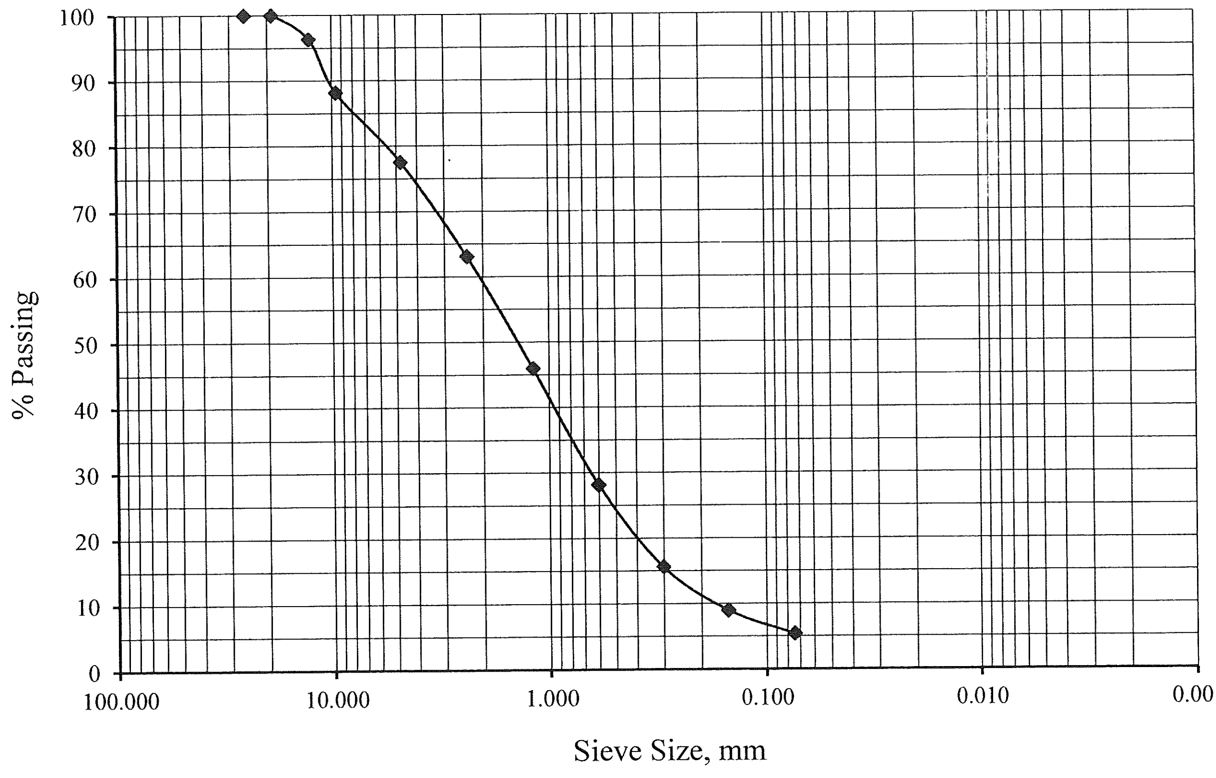
ASTM C117 & C136

January 17, 2023

Project Number:	544-21530
Project Name:	The Living Desert
Lab ID Number:	LN6-22600
Sample ID:	BH-1 R-1 @ 5'

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	96.3
3/8"	9.53	88.2
#4	4.75	77.5
#8	2.36	63.1
#16	1.18	45.9
#30	0.60	28.2
#50	0.30	15.6
#100	0.15	9.0
#200	0.074	5.4



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Gradation

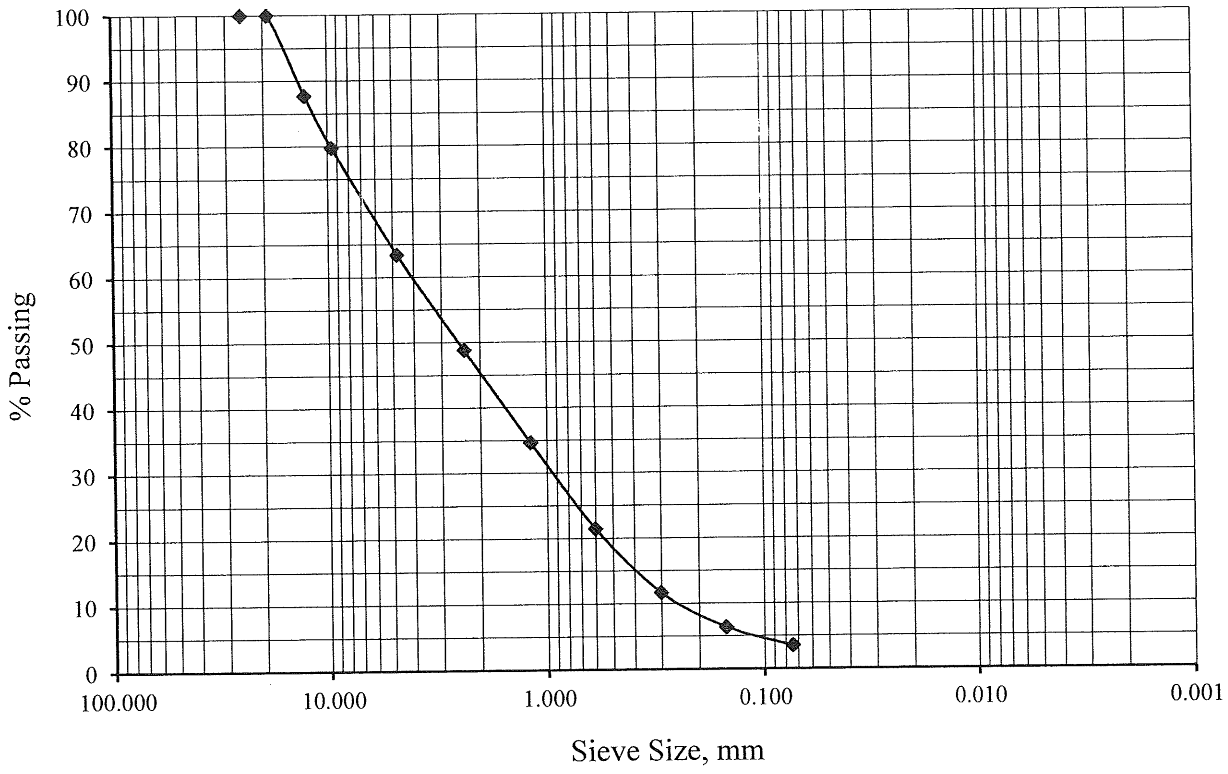
ASTM C117 & C136

January 17, 2023

Project Number:	544-21530
Project Name:	The Living Desert
Lab ID Number:	LN6-22600
Sample ID:	BH-3 R-4 @ 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	87.7
3/8"	9.53	79.8
#4	4.75	63.4
#8	2.36	48.8
#16	1.18	34.7
#30	0.60	21.5
#50	0.30	11.7
#100	0.15	6.4
#200	0.074	3.6



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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: January 17, 2023

Account No.: 544-21530

Customer: PVG Architects

Location: 47900 Portola Avenue, Palm Desert

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-3 @ 0-5'	7.6	20	60	4650

APPENDIX C

SEISMIC DESIGN PARAMETERS
SEISMIC DESIGN MAPS

SITE-SPECIFIC GROUND MOTION ANALYSIS
(ASCE 7-16)

Project: The Living Desert Phase 2.75 & 3.0
Project Number: 544.2153
Client: PVG Architects
Site Lat/Long: 33.7007/ -116.3730
Controlling Seismic Source: San Andreas

REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.3(ii)	F_v	2.5
$0.2*(S_{D1}/S_{DS})$	T_0	0.200
S_{D1}/S_{DS}	T_S	1.000
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	1.0000
Equation 11.4-2 - F_V*S_1[1]	S_{M1}	1.5000

[1] - F_V as determined by Section 21.3

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	1.500
Design Maps	S_1	0.600
Equation 11.4-1 - F_A*S_S	S_{MS}	1.500*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.00*
Design Maps	PGA	0.54
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - $F_{PGA}*PGA$	PGA_M	0.594*
Section 21.5.3	80% of PGA_M	0.475
Design Maps	C_{RS}	0.922
Design Maps	C_{R1}	0.9

REFERENCE	NOTATION	VALUE
$0.2*(S_{D1}/S_{DS})$	T_0	0.136*
S_{D1}/S_{DS}	T_S	0.68*
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.680*
Equation 11.4-2 - F_V*S_1	S_{M1}	1.020*

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.922
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.9

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

Period	Cr
0.200	0.922
0.300	0.919
0.400	0.917
0.500	0.914
0.600	0.911
0.680	0.909
1.000	0.9

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

Mapped values from https://hazards.atcouncil.org/
https://www.seismicmaps.org/

Sladden Engineering

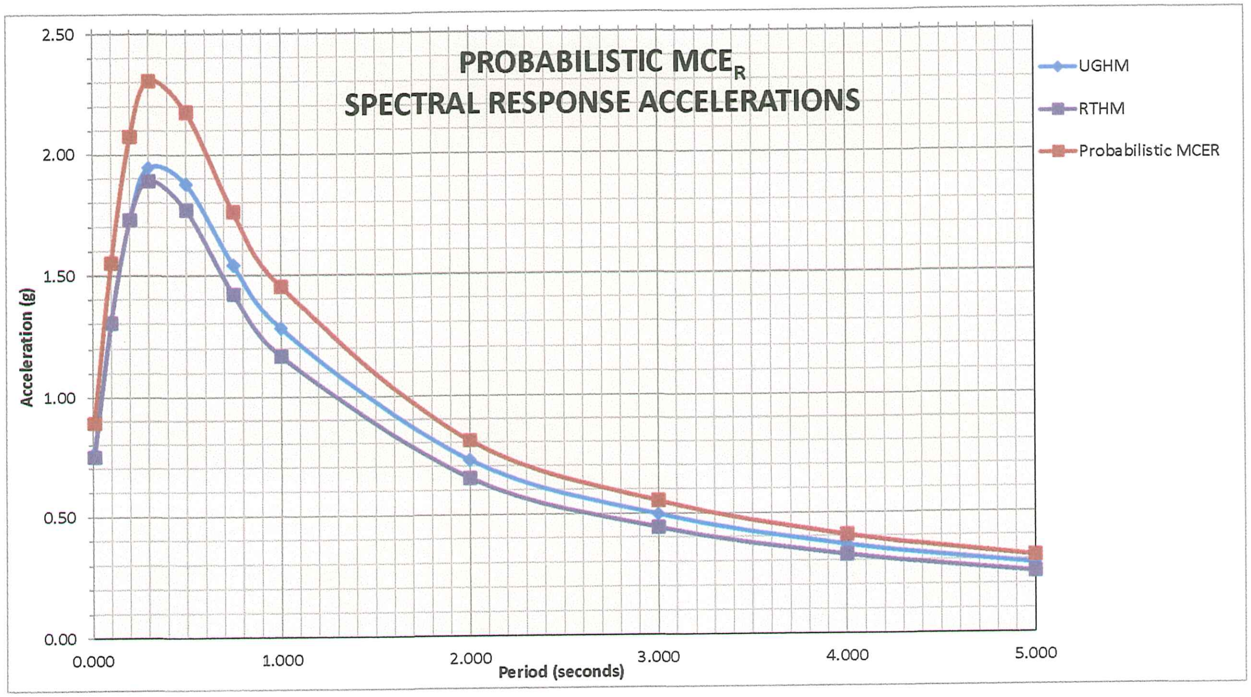
PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

Project No: 544.2153

Period	UGHM	RTGM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.765	0.748	1.19	0.890
0.100	1.310	1.302	1.19	1.549
0.200	1.728	1.728	1.20	2.074
0.300	1.945	1.890	1.22	2.306
0.500	1.876	1.769	1.23	2.176
0.750	1.540	1.419	1.24	1.760
1.000	1.280	1.170	1.24	1.451
2.000	0.726	0.651	1.24	0.807
3.000	0.495	0.441	1.25	0.551
4.000	0.365	0.324	1.25	0.405
5.000	0.285	0.250	1.26	0.315

Probabilistic PGA: 0.765

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



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

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

DETERMINISTIC SPECTRUM

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

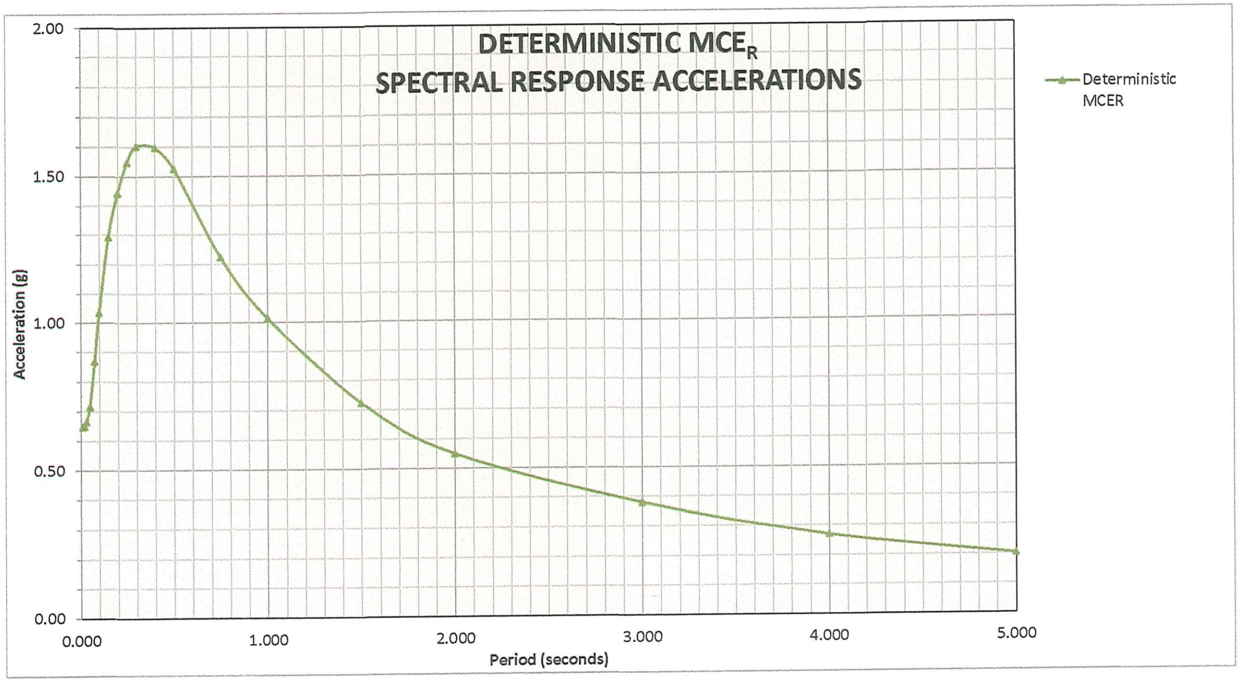
Controlling Source: San Andreas

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

Project No: 544.2153

Period	Deterministic PSa Median + 1.σ for 5% Damping	Max Directional Scale Factor[2]	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.546	1.19	0.650	0.650
0.020	0.548	1.19	0.652	0.652
0.030	0.559	1.19	0.665	0.665
0.050	0.602	1.19	0.716	0.716
0.075	0.729	1.19	0.868	0.868
0.100	0.870	1.19	1.036	1.036
0.150	1.077	1.20	1.293	1.293
0.200	1.203	1.20	1.443	1.443
0.250	1.278	1.21	1.546	1.546
0.300	1.312	1.22	1.601	1.601
0.400	1.298	1.23	1.596	1.596
0.500	1.240	1.23	1.525	1.525
0.750	0.987	1.24	1.224	1.224
1.000	0.818	1.24	1.014	1.014
1.500	0.584	1.24	0.724	0.724
2.000	0.443	1.24	0.550	0.550
3.000	0.303	1.25	0.379	0.379
4.000	0.216	1.25	0.270	0.270
5.000	0.161	1.26	0.203	0.203

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	~~0.546~~
Is Deterministic PGA >=F_{PGA}*0.5?	NO
Deterministic PGA:	0.550



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

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

Sladden Engineering

SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.890	0.650	0.650	0.433
0.100	1.549	1.036	1.036	0.690
0.200	2.074	1.443	1.443	0.962
0.300	2.306	1.601	1.601	1.067
0.500	2.176	1.525	1.525	1.017
0.750	1.760	1.224	1.224	0.816
1.000	1.451	1.014	1.014	0.800
2.000	0.807	0.550	0.550	0.400
3.000	0.551	0.379	0.379	0.267
4.000	0.405	0.270	0.270	0.200
5.000	0.315	0.203	0.203	0.160

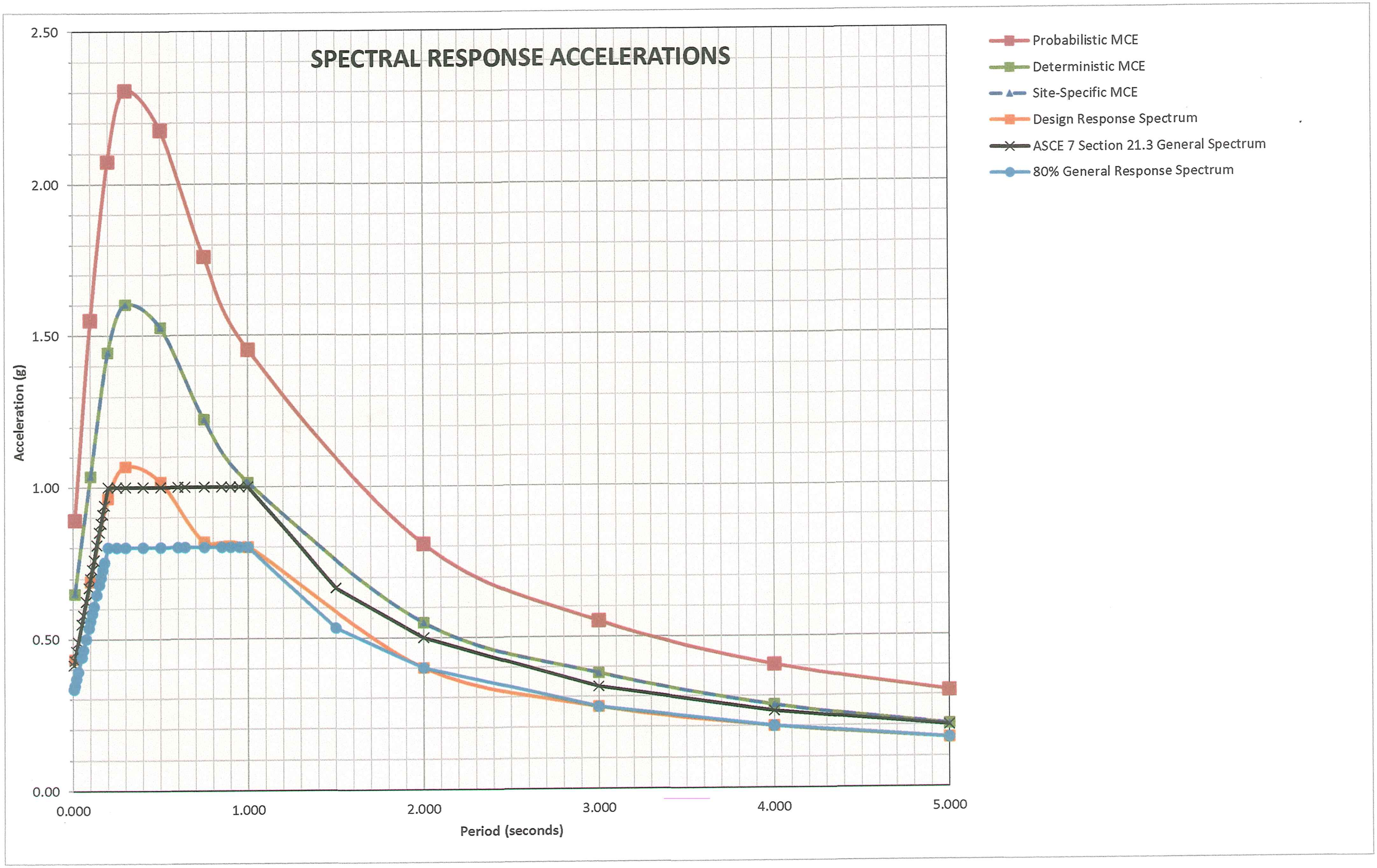
Period	ASCE 7 SECTION 21.3 General Spectrum	80% General Response Spectrum
0.005	0.415	0.332
0.010	0.430	0.344
0.020	0.460	0.368
0.030	0.490	0.392
0.050	0.550	0.440
0.060	0.580	0.464
0.075	0.625	0.500
0.090	0.670	0.536
0.100	0.700	0.560
0.110	0.730	0.584
0.120	0.760	0.608
0.136	0.808	0.646
0.150	0.850	0.680
0.160	0.880	0.704
0.170	0.910	0.728
0.180	0.940	0.752
0.200	1.000	0.800
0.250	1.000	0.800
0.300	1.000	0.800
0.400	1.000	0.800
0.500	1.000	0.800
0.600	1.000	0.800
0.640	1.000	0.800
0.750	1.000	0.800
0.850	1.000	0.800
0.900	1.000	0.800
0.950	1.000	0.800
1.000	1.000	0.800
1.500	0.667	0.533
2.000	0.500	0.400
3.000	0.333	0.267
4.000	0.250	0.200
5.000	0.200	0.160

ASCE 7-16: Section 21.4
Site Specific

	Calculated Value	Design Value
SDS:	0.961	0.961
SD1:	0.800	0.800
SMS:	1.441	1.441
SM1:	1.200	1.200
Site Specific PGAm:	0.550	0.550
Site Class:	D measured	

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

* Risk Categories I, II, or III


SPECTRAL RESPONSE ACCELERATIONS
Acceleration (g)
2.50
2.00
1.50
1.00
0.50
0.00
0.000
1.000
2.000
3.000
4.000
5.000
Period (seconds)
Probabilistic MCE
Deterministic MCE
Site-Specific MCE
Design Response Spectrum
ASCE 7 Section 21.3 General Spectrum
80% General Response Spectrum

OSHPD

Latitude, Longitude: 33.7007, -116.3730

Date	1/11/2023, 8:46:00 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.54	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.594	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.605	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.741	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.608	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.675	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.54	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.686	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.9	Mapped value of the risk coefficient at a period of 1 s
C_V	1.4	Vertical coefficient

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