

Appendix D
Preliminary Geotechnical Report



**PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED STORAGE CENTER
APN: 657-220-003
DESERT HOT SPRINGS
RIVERSIDE COUNTY, CALIFORNIA**

**PROJECT NO. 13548.1
MAY 31, 2019**

Prepared For:

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May 31, 2019

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Project No. 13548.1

Subject: Preliminary Geotechnical Investigation, Proposed Storage Center, APN: 657-220-003, Riverside County, California.

LOR Geotechnical Group, Inc., is pleased to present this report detailing our geotechnical investigation for the subject project. In summary, it is our opinion that the proposed development is feasible from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction.

Much of the project site is covered by a variable thickness of undocumented fill materials. The underlying natural alluvial soils were found to be loose to medium dense in their upper portions. It is our opinion that the existing fill materials and the upper alluvial soils will not provide uniform and/or adequate support for the proposed development. Thus, we recommend that the undocumented materials and loose alluvial. All on-site soils should be suitable for use as engineered compacted fill. Excluding removal of the undocumented materials, removals on the order of 1 to 3 feet into the natural alluvium are anticipated to be required within the proposed development areas.

Very low expansive soils and very good R-value quality soils were encountered on the site. A negligible sulfate content was found for the natural soils, however, higher sulfate content was determined for the undocumented fill soils tested. Near completion and/or at the completion of site grading, additional foundation and subgrade soils should be tested to verify their expansion potential, soluble sulfate content, and R-value quality.

LOR Geotechnical Group, Inc.

Table of Contents

Page No.

INTRODUCTION	1
PROJECT CONSIDERATIONS	1
AERIAL PHOTOGRAPH ANALYSIS	2
EXISTING SITE CONDITIONS	3
SUBSURFACE FIELD INVESTIGATION	3
LABORATORY TESTING PROGRAM	4
GEOLOGIC CONDITIONS	4
Regional Geologic Setting	4
Site Geologic Conditions	4
Fill:	5
Alluvium	5
Groundwater Hydrology	5
Mass Movement	5
Faulting	6
Historical Seismicity	6
Secondary Seismic Hazards	7
Liquefaction	7
Seiches/Tsunamis	7
Flooding (Water Storage Facility Failure)	8
Seismically-Induced Landsliding	8
Rockfalls	8
SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2016)	8
CBC Earthquake Design Summary	8
INFILTRATION TESTING PROGRAM	9
CONCLUSIONS	10
Soil Expansiveness	11
Infiltration	11
Geologic Mitigations	11
Seismicity	11

Table of Contents

	<u>Page No.</u>
RECOMMENDATIONS.	12
General Site Grading.	12
Initial Site Preparation.	13
Preparation of Fill Areas.	13
Preparation of Building Pad Areas.	13
Engineered Compacted Fill.	14
Short-Term Excavations.	14
Soil Expansiveness.	15
Foundation Design.	15
Settlement.	16
Building Pad Slab-On-Grade Design.	16
Exterior Flatwork.	16
Wall Pressures.	17
Preliminary Pavement Design.	18
Sulfate Protection.	19
Infiltration Basin.	19
Construction Monitoring.	20
LIMITATIONS.	21
TIME LIMITATIONS.	22
CLOSURE.	23
REFERENCES.	24
AERIAL PHOTOGRAPHS.	25
APPENDICES	
<u>Appendix A</u>	
Index Map.	A-1
Trench Location Maps.	A-2a and A-2b
Regional Geologic Map.	A-3
Historical Seismicity Maps.	A-4 and A-5

Table of Contents

Page No.

Appendix B

Field Investigation Program. B
Trench and Boring Log Legend. B-i
Soil Classification Chart. B-ii
Trench and Boring Logs. B-1 through B-7

Appendix C

Laboratory Testing Program. C
Gradation Curves. C-1

Appendix D

Infiltration Test Results. D-1 through D-2

INTRODUCTION

During May of 2019, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed commercial development of approximately 4.69 acres located on the south side of Dillon Road, approximately one-quarter mile west of Palm Drive, in the city of Desert Hot Springs, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photograph pairs of the site and surrounding regions dated 1948 through 2018;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Infiltration testing via the double ring test method within the approximate area proposed for the infiltration of onsite runoff waters;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

The approximate location of the site is shown on the attached Index Map, Enclosure A-1 within Appendix A.

PROJECT CONSIDERATIONS

To orient our investigation at the site, a Preliminary Site Plan was provided for our use. This plan illustrated the current site conditions as well as the currently proposed improvements. As illustrated, the site will be developed with 186 paved storage spaces,

paved access routes, and associated street and landscaping improvements. In addition, an infiltration basin is proposed for the area located in the southeast corner of the storage area and a 12' by 40' office building with three paved parking spaces is proposed. While not shown on the plans provided, it is our understanding that structural improvements in the form of additional buildings are likely to be proposed in the future. Cuts and fills on the order of 5 to 10 feet are anticipated based on the current site topography and that of the adjoining areas. A copy of the Preliminary Site Plan was utilized as a base for presenting our data as shown on Enclosure A-2a within the Appendix A. An image from Google Earth was also used as a base map to show the site and our areas of investigation and the surficial site conditions and this information is shown on Enclosure A-2b.

The types of structures that may be proposed for construction at the site in the future is currently unknown. However, for the purposes of this report, we have assumed that the structures will have light to moderate loads.

AERIAL PHOTOGRAPH ANALYSIS

To obtain a comprehensive history of previous site usage, a search was conducted for available online aerial photographs by NETRONLINE, Google Earth, and Riverside County Flood Control and Water Conservation District by a geologist from this firm. The search reviewed aerial photographs taken of the site and surrounding area.

Due to the relatively large photographic scales involved, the analysis and subsequent interpretation of detail from aerial photographs sometimes requires a degree of subjective judgment. The degree of certainty on the interpretation of details depends upon such factors as the scale and the quality of the photograph. However, an analysis of aerial photographs will reveal the general site history as to the relative use of the land, possible ground disturbance, activities, etc.

The earliest available photographs reviewed were from 1948 and these show the site in a natural condition with no indications for previous site usage. The next available site images are from the early 1970's and these show that a large amount of imported material was, by that time, spread across the northern approximately two-thirds of the property and that concrete slabs and some structures had been built within the northeast portion. It is difficult to discern what type of development previously existed onsite through review of the photographs, but based upon the presence of numerous stored items, pieces of construction equipment, and other miscellaneous items on the property, particularly during the 1980's and 1990's, some form of commercial business appears to have been the previous use. By the late 1990's, the site was cleared of most items and has appeared

much as it is at the present time. No evidence for the presence of faults traversing the site area or mass movement features were noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The site consists of approximately 4.69 acres of land located on the south side of Dillon Road, approximately one-quarter mile west of Palm Drive. The northern roughly two-thirds of the site is currently covered by fill materials that, as based upon the findings of our subsurface investigation, range from less than 2 feet to a maximum of approximately 7 feet in thickness. The fill material was apparently brought onto the property prior to development of the site in the early 1970's, at which time concrete slabs and a couple of structures were built. Only the concrete slabs and footings remain after demolition of the onsite structures sometime before the late 1990's. Northeast of the remaining concrete slabs is a wooden pole and to the southwest, in the north-central portion of the property, is an abandoned water well.

The lower, southern one-third of the site is in a relatively natural condition. It contains native plants as well as some surficial trash and debris. Along the northern side of the site is a partial chain-link fence and some Tamarisk trees. Chain link fencing also separates the site from a plant nursery to the west. On the east is an existing self-storage facility that, based upon review of aerial imagery, was constructed in the early to mid 1990's. Interestingly, it appears that when a 6-inch wide, concrete block wall was built on the property line between the site and this storage facility, the onsite debris was first removed, the wall built, and the debris replaced against the wall (which does not appear to have been designed as a retaining wall but may have been). To the south of the site property is open, vacant land. The site slopes gently to the south with drainage occurring mostly as sheet flow.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on May 16, 2019. The work consisted of excavating a total of 7 exploratory trenches using a tractor mounted backhoe equipped with a 36-inch bucket. The approximate locations of our exploratory trenches are presented on Enclosures A-2a and A-2b, within Appendix A.

The subsurface conditions encountered in the exploratory trenches were logged by a geologist from this firm. Our exploratory trenches were excavated to a maximum depth of approximately 14.5 feet below the ground surface. In-place density determinations were

conducted at selected levels within the trenches utilizing the Sand Cone Test Method (ASTM D 1557). Bulk samples were obtained at selected intervals and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the trench logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

Selected soil samples obtained during the field investigation were subjected to geotechnical laboratory testing to evaluate their physical and engineering properties. Laboratory testing included moisture content, dry density, laboratory compaction, direct shear, sieve analysis, sand equivalent, R-Value, and soluble sulfate content. A detailed description of the geotechnical laboratory testing program and the test results are presented in Appendix C.

GEOLOGIC CONDITIONS

Regional Geologic Setting

The site is located on a broad, coalescing alluvial fan that emanates from the Little San Bernardino Mountains to the north of the site. The sediments that make up this fan fill the northwestern portion of a structural depression known as the Coachella Valley. This basin is defined by small valleys and intervening low hills and these are often separated by faults. The Coachella Valley is bound by the Little San Bernardino Mountains and Mecca Hills area on the northeast, the Salton Sea and Imperial Valley on the southeast, the Santa Rosa and San Jacinto Mountains on the southwest and the Banning Pass area to the northwest.

The site is shown within the regional geologic setting as mapped by the C.D.M.G. on the enclosed Regional Geologic Map, Enclosure A-3, within Appendix A.

Site Geologic Conditions

As observed during this investigation, the majority of the subject site contains a variable thickness of imported fill materials overlying native alluvial soils. These units are described in further detail in the following sections:

Fill: The northern approximately two-thirds of the site is covered by a variable thickness of undocumented fill that was imported to the site some time prior to the early 1970's. The majority of the fill encountered consists of sand and silty sand soils, concrete, asphalt, rocks (cobbles and boulders), and trace to minor amounts of deleterious materials including metal, plastic, wood and other items. As encountered within our excavations and as based upon our observations, the fill material ranges from less than 2 to 7 or more feet in thickness. Please note that our trenching operations were conducted within and exposed a very small fraction of the fill materials. The actual, overall composition of the fill materials onsite may not be, or is likely not, clearly represented through the relatively small amount of excavation and classification work that we completed.

Alluvium: Underlying the fill materials and exposed across the southern portion of the property are natural alluvial soils. These soils generally consisted of silty sand and well graded sand with well graded sand with gravel, cobbles and boulders at depth. In general, the alluvial soils are in a relatively dense state. The soils are granular and based upon observation and classification, have very low expansion potential.

Detailed descriptions of the subsurface soil conditions as encountered within our exploratory trenches and borings are presented on the Trench Logs within Appendix B.

Groundwater Hydrology

Groundwater was not encountered in any of our excavations at the site. In order to estimate the approximate depth to groundwater in this area, a search of the water well database provided in the State of California Department of Water Resources website (CDWR, 2019) was conducted. This search indicated that the depth to groundwater in two water wells located approximately one-quarter to one-half of a mile to the north-northwest of the site ranged from 193 to 232 feet during the time period from 2011 through 2018. Based on the available information, groundwater at the site appears to be at a depth of greater than 100 feet.

Mass Movement

The majority of the site lies on a relatively flat surface. The occurrence of mass movement failures such as landslides, rockfalls, or debris flows within such areas is generally not considered common and no evidence of mass movement was observed on the site.

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2018).

The closest known active earthquake fault with a documented location is the South Branch of the San Andreas fault located approximately 1.8 kilometers (1.1 miles) to the southwest. In addition, the North Branch of San Andreas fault is located approximately 3.2 kilometers (2.0 miles) to the northeast.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific Plate and the North American Plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24mm/yr and capable of generating large magnitude events on the order of 7.5.

Current standards of practice included a discussion of all potential earthquake sources within a 100 kilometer (62 mile) radius. However, while there are other large earthquake faults within a 100 kilometer (62-mile) radius of the site, none of these are considered as relevant to the site as the faults described above, due to their closer distance and larger anticipated magnitudes.

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region, a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search program by EPI Software, Inc. (Reeder, 2000). This program conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto an overlay map of known faults. For this investigation the database of seismic events utilized by the EPI program was obtained from the Southern California Seismic Network (SCSN) available from the Southern California Earthquake Center. At the time of our search the data base contained data from January 1, 1932 through December 31, 2010.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of

moderate to large events. As depicted on Enclosure A-4, within Appendix A, the site lies within a relatively active region associated with the San Andreas fault trending southeast to northwest. Of these events, the closest was a magnitude 4.7 located approximately 1.7 kilometers (1 mile) southwest of the site.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.2 miles) radius of the site was examined by selecting an epicenter map listing events on the order of 1.0 and greater since 1978. In addition, only the "A" events, or most accurate events were selected. Caltech indicates the accuracy of the "A" events to be approximately 1 kilometer. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 30 ± years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the San Andreas fault appears to be the source of numerous events, roughly coinciding with the surface traces of this fault.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring in the region around the subject site, predominately associated with the presence of the San Andreas fault. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seismic-induced settlement, seiches and tsunamis, earthquake induced flooding, landsliding, and rockfalls.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within loose, granular sediments where the groundwater is usually less than 50 feet. As the depth to groundwater is on the order of 100 feet or more and the onsite earth materials are dense, the potential for liquefaction is considered nil.

Seiches/Tsunamis: The potential for the site to be affected by a seiche or tsunami (earthquake generated wave) is considered nil due to the absence of any large bodies of water near the site.

Flooding (Water Storage Facility Failure): There are no large water storage facilities located on or near the site which could possibly rupture during an earthquake and affect the site by flooding.

Seismically-Induced Landsliding: Due to the low relief of the site and surrounding region, the potential for landslides to occur at the site is considered nil.

Rockfalls: No large, exposed, loose or unrooted boulders are present above the site that could affect the integrity of the site.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2016)

Section 1613 of Chapter 16 of the 2016 California Building Code (CBC) contains the procedures and definitions for the calculations of the earthquake loads on structures and non structural components that are permanently attached to structures and their supports and attachments.

It should be noted that the classification of use and occupancy of all proposed structures at the site, and thus design requirements, shall be the responsibility of the structural engineer and the building official.

CBC Earthquake Design Summary

The following earthquake design criteria have been formulated for the site utilizing the source referenced above.

However, these values should be reviewed by the building official (Risk Category) and structural engineer and the final design should be performed by a qualified structural engineer familiar with the region.

CBC 2016 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 33.9240, -116.5077, Risk Category II	
Site Class Definition Chapter 20 ASCE 7	D
S_s Mapped Spectral Response Acceleration at 0.2s Period	2.887
S₁ Mapped Spectral Response Acceleration at 1s Period	1.065
F_a Short Period Site Coefficient at 0.2s Period	1.0
F_v Long Period Site Coefficient at 1s Period	1.5

CBC 2016 SEISMIC DESIGN SUMMARY (ASCE 7-10)* Site Location (USGS WGS84) 33.9240, -116.5077, Risk Category II	
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period	2.887
S_{M1} Adjusted Spectral Response Acceleration at 1s Period	1.598
S_{DS} Design Spectral Response Acceleration at 0.2s Period	1.925
S_{D1} Design Spectral Response Acceleration at 1s Period	1.065
Seismic Design Category - Short Period	E
Seismic Design Category - Long Period	E
*Values obtained from OSHPD online Seismic Design Maps tool	

INFILTRATION TESTING AND TEST RESULTS

Two double ring infiltration tests were conducted at the locations requested and illustrated on Enclosure A-2. As requested, test pits were excavated to depths of approximately 3 feet below the existing ground surface and a 12-inch diameter casing was installed within the center of the test locations with a 24-inch diameter casing centered around it. Each 20-inch tall casing was imbedded to a depth of approximately 2-inches. The test locations were tested immediately after the casings were installed by filling both the inside and outside casings and maintaining a water level to a depth of approximately 2-inches.

The testing procedure was as follows:

Both the inside and outside areas of the casings were filled with water to a level of approximately 2-inches above the ground surface. Water was then metered to maintain this water level within both rings. The volume of water use in a given time period was recorded at various time intervals to establish the infiltration rate of the water within the inner ring. See the attached Infiltration Test Data sheets, Enclosures D-1 through D-2 within Appendix D for the test information and measurements.

The infiltration rate is measured as the drop in water level compared to the permeability of the bottom surface area soils in the bottom of the test hole. If casing is not used, the water column in the test hole is allowed to seep into both the bottom and sidewalls of the hole, for which the drop in water level must be corrected and reduced for the volume of water seeping into the sidewall and for the diameter of the test hole. As described above, the tests described herein were conducted using a 12-inch diameter inner casing and 24-inch diameter outer casing.

The test holes were found to have the following measured clear water infiltration rates:

Infiltration Test No.	Infiltration Rate*	
	gal/sf/day	in/hr
DRI-1	113	7.5
DRI-2	296	20.0
* Rounded final reading		

Our test data indicates infiltration rates of 7.5 to 20 inches per hour for the soils tested.

CONCLUSIONS

On the basis of our field investigation and testing program, it is the opinion of LOR Geotechnical Group, Inc., that the proposed development is feasible from a soil engineering standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

Based upon the field investigation and test data, it is our opinion that the undocumented fill soils and the upper portions of the alluvial soil will not, in their present conditions, provide uniform and/or adequate support for the proposed improvements. However, complete removal of the fill soils, removal of the near surface alluvial soils, and the construction of engineered fill will provide an acceptable solution. Our in-place density results indicated variable conditions for the upper portions of the alluvial materials ranging from loose to medium dense states.

To provide adequate support for the proposed improvements, we recommend that a compacted fill mat be constructed beneath footings and slabs. This compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. The construction of this compacted fill mat should incorporate the removal of the existing fill soils and the upper, loose to medium dense alluvial soil. Conventional foundation systems, using either individual spread footings and/or continuous wall footings, will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat.

Soil Expansiveness

As determined through our subsurface explorations and laboratory testing, the site surficial soils primarily consist of sand and silty sand soils that possess a very low expansion potential. Therefore, design for resistance to expansive soils is not required.

Infiltration

As noted during this investigation, our infiltration testing resulted in good infiltration rates of the soils tested.

Geologic Mitigations

No special mitigation methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

Seismicity

Seismic ground rupture is generally considered most likely to occur along pre-existing active faults. Since no faults are known to exist at, or project into the site, the probability of ground surface rupture occurring at the site is considered nil.

Due to the site's close proximity to the San Andreas fault described above, it is reasonable to expect a very strong ground motion seismic event to occur during the lifetime of the proposed development on the site. Large earthquakes could occur on other faults in the general area, but because of their lesser anticipated magnitude and/or greater distance, they are considered less significant than the San Andreas fault zone from a ground motion standpoint.

The effects of ground shaking anticipated at the subject site should be mitigated by the seismic design requirements and procedures outlined in Chapter 16 of the California Building Code. However, it should be noted that the current building code requires the minimum design to allow a structure to remain standing after a seismic event, in order to allow for safe evacuation. A structure built to code may still sustain damage which might ultimately result in the demolishing of the structure (Larson and Slosson, 1992).

RECOMMENDATIONS

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor, the jurisdictional agency, and the geotechnical engineer should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusion of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials. Such materials may not be used as engineered fill.

All uncontrolled fills encountered during site preparation should be completely removed, cleaned of significant deleterious materials, and may then be reused as compacted fill. Deleterious items include metal, plastic, wood, organic matter, and trash and debris. No oversized materials (greater than 6 inches in maximum dimension) should be incorporated within the fills unless approved by the geotechnical consultant.

It is also our recommendation that all existing uncontrolled and/or undocumented fills under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

Cavities created by removal of subsurface obstructions, such as foundations, effluent disposal systems, etc., that may be associated with past development, should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

If the onsite water well is to be abandoned, this work should be done in accordance with jurisdictional agency requirements.

Initial Site Preparation

All existing uncontrolled fills and loose alluvial materials should be removed from structural areas and areas to receive structural fills. The data developed during this investigation indicates that removals on the order of 1 to 3 feet will be required to encounter competent alluvium. Competent alluvium is defined as native materials with an in-place relative compaction of at least 85 percent. The actual depths of removal should be verified during the grading operation by observation and in-place density testing and the above estimated removal depths do not include required removals of undocumented fill materials.

Preparation of Fill Areas

After conducting the removals discussed above and prior to placing fill, the surfaces of all areas to receive fill should be scarified to a depth of at least 6 inches. The scarified soil should be brought to near optimum moisture content and recompact to a relative compaction of at least 90 percent (ASTM D 1557).

Preparation of Building Pad Areas

All footings should rest upon a minimum of 24 inches of properly compacted fill material placed over competent alluvium. In areas where the required fill thickness is not accomplished by the removal of the existing fill and loose alluvial materials and site rough grading, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines. Where deeper removals in excess of 5 feet are required, these removals should extend laterally at a 1:1 ratio. The bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to near optimum moisture content, and recompact to at least 90 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill.

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 24 inches of compacted soil. During rough grading, the remedial removals recommended above will most likely provide the recommended 24 inches of compacted soil for adequate support of concrete slabs-on-grade.

Engineered Compacted Fill

The on-site natural soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6-inches should not be buried or placed in fills. The undocumented fill soils do contain a significant percentage of oversize materials in the form of concrete, asphalt and rocks and also contains deleterious man-made debris and/or organic matter. The oversize materials should be either reduced to less than 6 inches in maximum diameter prior to incorporation as fill material or hauled from the site. Any significant quantities of man-made debris and/or organic matter will also require removal from the site

Import fill, if required, should be inorganic, non-expansive granular soils free from rocks or lumps greater than 6-inches in maximum dimension. Sources for import fill should be approved by the geotechnical engineer prior to their use.

Fill should be spread in maximum 8-inch loose lifts, each lift brought to near optimum moisture content, and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557.

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations 5-feet deep and greater should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements.

Short-term excavations of 5-feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our exploratory trenches it appears that Type C soil is the predominant type of soil on the project and all short-term excavations should be based on this type of soil. Deviation from the standard short-term slopes are permitted using Option 4, Design by a Registered Professional Engineer (Section 1541.1).

Short-term slope construction and maintenance are the responsibility of the contractor, and should be a consideration of his methods of operation and the actual soil conditions encountered.

Soil Expansiveness

The upper materials encountered during this investigation were observed and classified and determined to have a very low expansion potential. Therefore, specialized construction procedures to specifically resist expansive soil activity are not anticipated at this time. In order to verify this, additional evaluation of on-site and any imported soils for their expansion potential should be conducted during and following completion of the grading operation.

Foundation Design

If the site is prepared as recommended, structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 24 inches of engineered compacted fill. All foundations should have a minimum width of 12 inches and should be established a minimum of 12 inches below lowest adjacent grade.

For the minimum width and depth, footings may be designed using a maximum soil bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads. This value may be increased by 300 psf for each additional foot of width and by 300 psf for each additional foot of depth, to a maximum of 3,000 psf.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 300 pounds per square foot per foot of depth. Base friction may be computed at 0.30 times the normal load. Base friction and passive earth pressure may be combined without reduction.

Settlement

Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Maximum settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlement between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Building Pad Slab-On-Grade Design

Concrete floor slabs should bear on a minimum of 24 inches of engineered fill compacted to at least 90 percent (ASTM D 1557) placed over competent alluvium. This will most likely be accomplished during the recommended removals previously mentioned. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork

To provide adequate support, exterior flatwork improvements should rest on a minimum of 12 inches of soil compacted to at least 90 percent (ASTM D 1557). Flatwork surface should be sloped a minimum of 1 percent away from buildings and slopes, to approved drainage structures.

Wall Pressures

The design of footings for walls below grade (basement or pit walls, etc.) and retaining structures should be performed in accordance with the recommendations described earlier under Preparation of Building Pad Areas and Foundation Design. For design of retaining wall footings, the resultant of the applied loads should act in the middle one-third of the footing, and the maximum edge pressure should not exceed the basic allowable value without increase.

For design of retaining walls unrestrained against movement at the top, we recommend an equivalent fluid density of 35 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of recompacted, non-expansive, native soils placed against the structures and with the backcut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

To avoid overstressing or excessive tilting during placement of backfill behind walls, heavy compaction equipment should not be allowed within the zone delineated by a 45 degree line extending from the base of the wall to the fill surface. The backfill directly behind the walls should be compacted using light equipment such as hand operated vibrating plates and rollers. No material larger than 3-inches in diameter should be placed in direct contact with the wall.

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, non-expansive, properly drained backfill (with no additional surcharge loadings). If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters. Toe bearing pressure for non-structural walls on soils, not prepared as described earlier under Preparation of Foundation Areas, should not exceed California Building Code values, (CBC Table 18-1.A).

The existing block wall along the east side of the site does not appear to have been designed as a retaining wall yet it retains 4 to 5 feet of material along its northern half. The type of wall that was actually built and its ability to support the existing backfill and/or additional loading should be evaluated by the project engineer during site design.

Preliminary Pavement Design

Testing and design for preliminary on-site and off-site pavements was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing, and upon assumed Traffic Indices, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject improvements:

T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
5.0 and 6.0 (Light Vehicle and Incidental Truck Traffic)	50	0.25' AC/0.35' AB
8.0 (Truck Access/Drive Isles)	50	0.40' AC/0.45' AB
AC - Asphalt Concrete AB - Aggregate Base (Caltrans Class 2)		

The above structural sections are predicated upon 90 percent relative compaction (ASTM D 1557) of all utility trench backfills and 95 percent relative compaction (ASTM D 1557) of the upper 12 inches of street subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 aggregate base.

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 0.42 foot thick concrete, with a 0.35 foot thick aggregate base. Concrete pads are also recommended in areas adjacent to trash storage areas where heavier loads will occur due to operation of trucks lifting trash dumpsters.

Transverse joints should be sawcut in the pavement at approximately 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other.

The above pavement designs were based upon the results of preliminary sampling and testing, and should be verified by additional sampling and testing when the actual subgrade soils are exposed.

Sulfate Protection

The results of the sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented in Appendix C. Based on our test results it appears that there is a negligible sulfate exposure to concrete elements in contact with natural alluvial soils at the site. The 2016 CBC, therefore, does not recommend special design criteria for concrete elements in contact with these soils. However, higher sulfate content soil materials were determined through testing of a random sample from the onsite undocumented fill soils. Therefore, additional sulfate testing and evaluation of the best methods for mitigating these conditions should be conducted at the time of site preparation for grading. If the bulk of these materials are to remain on the property, special design criteria for concrete elements in contact with them will likely be required.

Infiltration Basin

Based upon our infiltration test data, the clear water infiltration rate ranges from 7.5 inches per hour (113 gal/sf/day) to 20 inches per hour (296 gal/sf/day). The sandy alluvial soils that were logged as extending to a depth of at least 14.5 feet within our exploratory trench T-1. We recommend that a clear water design rate of 7.5 inches per hour be utilized for design.

A factor of safety should be applied to the design clear water rate as indicated by the Riverside County Flood Control and Water Conservation District, Design Handbook for Low Impact Development Best Management Practices (2011). The design clear water infiltration rate should be adjusted using a factor of safety of 3.0 (RCFCWCD, 2011).

To ensure continued infiltration capability of the infiltration areas, a program to maintain the facilities should be considered. This program should include periodic removal of accumulated materials, which can slow the infiltration and decrease the water quality. Materials to be removed from the catch basin areas typically consist of litter, dead plant matter, and soil fines (silts and clays). Proper maintenance of the system is critical. A maintenance program which meets or exceeds those developed by the local governing agency should be prepared and properly executed. At a minimum, the program should be

as outlined in the Riverside County Flood Control and Water Conservation District, Design Handbook for Low Impact Development Best Management Practices (2011).

The program should also incorporate the recommendations presented below and any other jurisdictional agency requirements.

Systems should be set back at least 10 feet from foundations or as required by the design engineer.

Systems should be set back at least 100 feet horizontally from wells.

Any geotextile filter fabric utilized should consist of material that prevents soil piping but has greater permeability than the existing soil.

During site development, care should be taken to not disturb the area(s) proposed for infiltration as changes in the soil structure could occur resulting in a change of the soil infiltration characteristics.

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by the project geotechnical consultant prior to construction to confirm that the intent of the recommendations presented in this report have been incorporated into the design. Additional R-Value, expansion index, and soluble sulfate testing will be required after/during site rough grading.

During construction, sufficient and timely geotechnical observation and testing should be provided to correlate the findings of this investigation with the actual subsurface conditions exposed during construction. Items requiring observation and testing include, but are not necessarily limited to, the following:

1. Site preparation-stripping and removals.
2. Excavations, including approval of the bottom of excavation prior to backfilling.
3. Scarifying and recompacting prior to fill placement.

4. Subgrade preparation for pavements and slabs-on-grade.
5. Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Real Estate Investments, and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. If conditions are encountered during the construction of the project, which differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant. If parties other than LOR Geotechnical Group, Inc. provide construction monitoring services, they must be notified that they will be required to assume responsibility for the geotechnical phase of the project being completed by concurring with the recommendations provided in this report or by providing alternative recommendations.

The report was prepared using generally accepted geotechnical engineering practices under the direction of a state licensed geotechnical engineer. No warranty, expressed or implied, is made as to conclusions and professional advice included in this report. Any persons using this report for bidding or construction purposes should perform such independent investigations as deemed necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on this project.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Governmental Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a significant amount of time without a review by LOR Geotechnical Group, Inc., verifying the suitability of the conclusions and recommendations.

CLOSURE

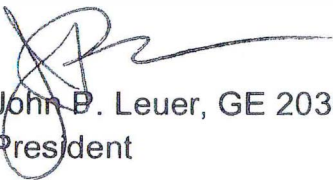
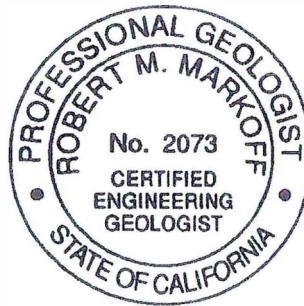
It has been a pleasure to assist you with this project. We look forward to being of further assistance to you as construction begins. Should conditions be encountered during construction that appear to be different than indicated by this report, please contact this office immediately in order that we might evaluate their effect.

Should you have any questions regarding this report, please do not hesitate to contact our office at your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.



Robert M. Markoff, CEG 2073
Engineering Geologist



John P. Leuer, GE 2030
President



RMM:JPL/ss

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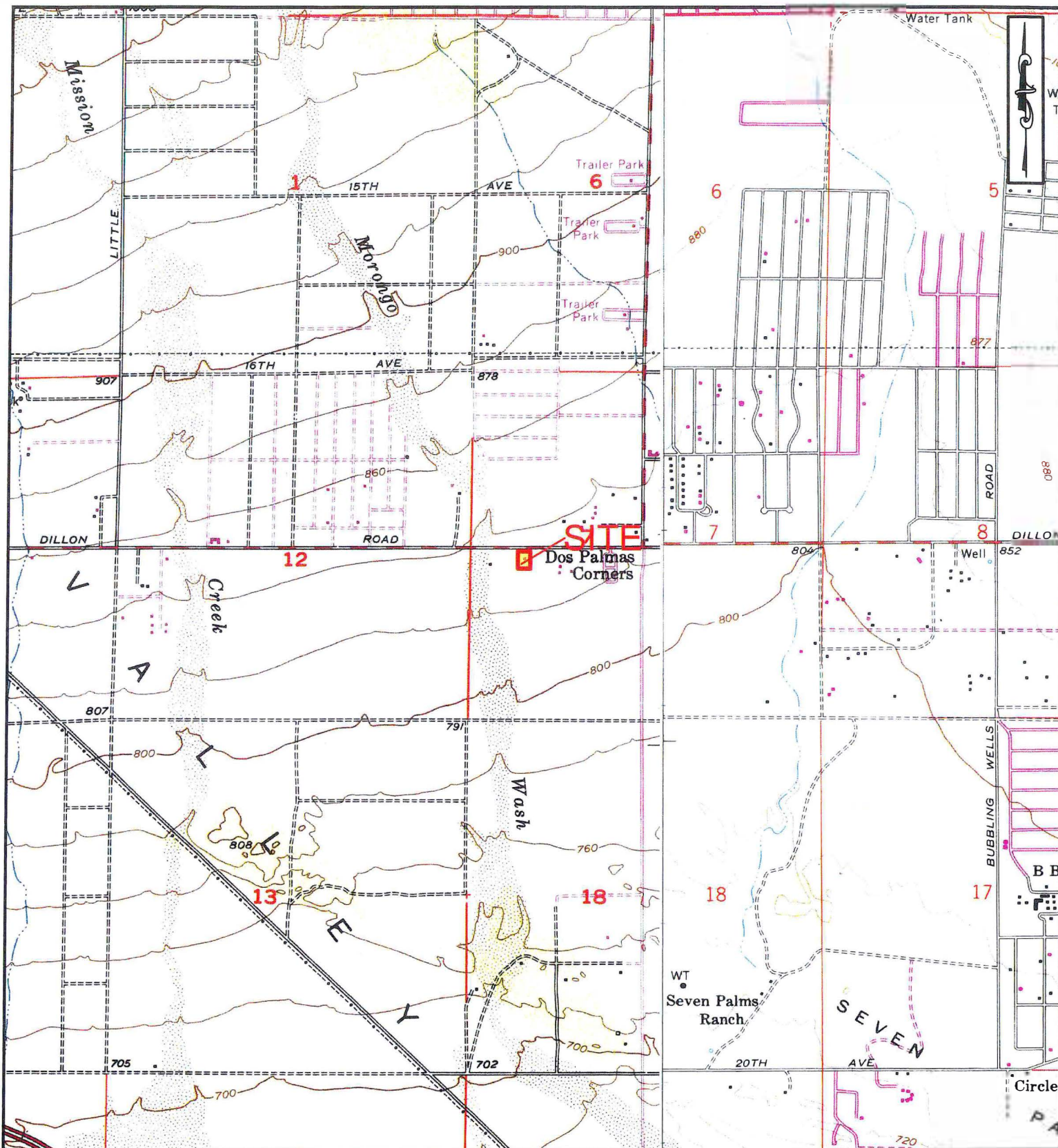
AERIAL PHOTOGRAPHS

(Source: Riverside County Flood Control and Water Conservation District)

Date	Photo No(s)	Scale 1" =
November, 1948	44	3,300'
May 24, 1974	196 and 197	2,000'
February 1, 1980	203 and 204	1,600'
February 7, 1984	1438 and 1439	1,600'
January 19, 1990	5-77 and 5-78	1,600'
January 28, 1995	5-87 and 5-88	1,600'
March 11, 2000	5-67, 5-68	1,600'
April 18, 2005	5-65 and 5-66	1,600'
March 12, 2010	5-68 and 5-69	1,600'

APPENDIX A

Index Map, Trench Location Maps, Regional Geologic Map and Historical Seismicity Maps



INDEX MAP

PROJECT:

APN: 657-220-003, DESERT HOT SPRINGS, CALIFORNIA

PROJECT NO:

13548.1

CLIENT:

REAL ESTATE INVESTMENTS

ENCLOSURE:

A-1

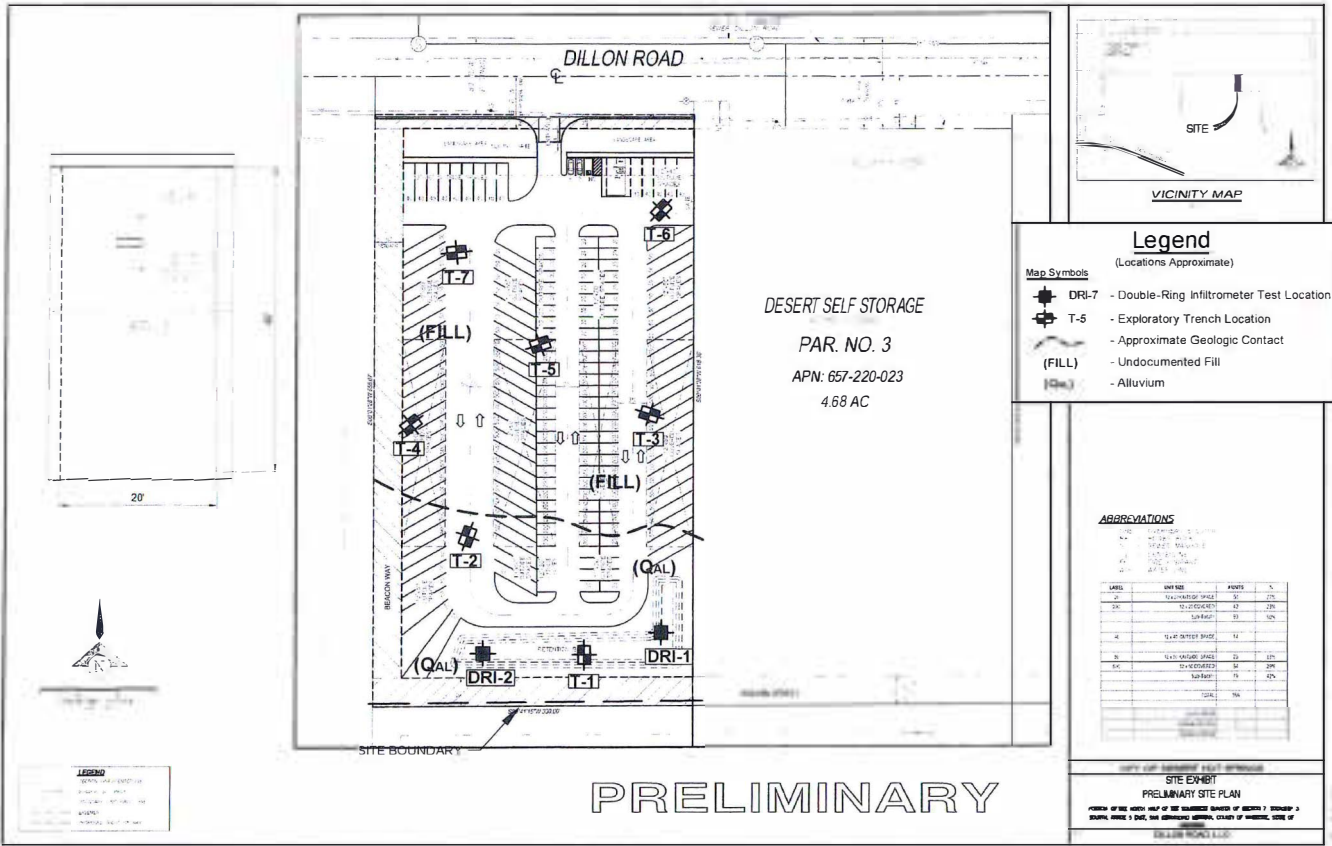
LOR Geotechnical Group, Inc.

DATE:

MAY 2019

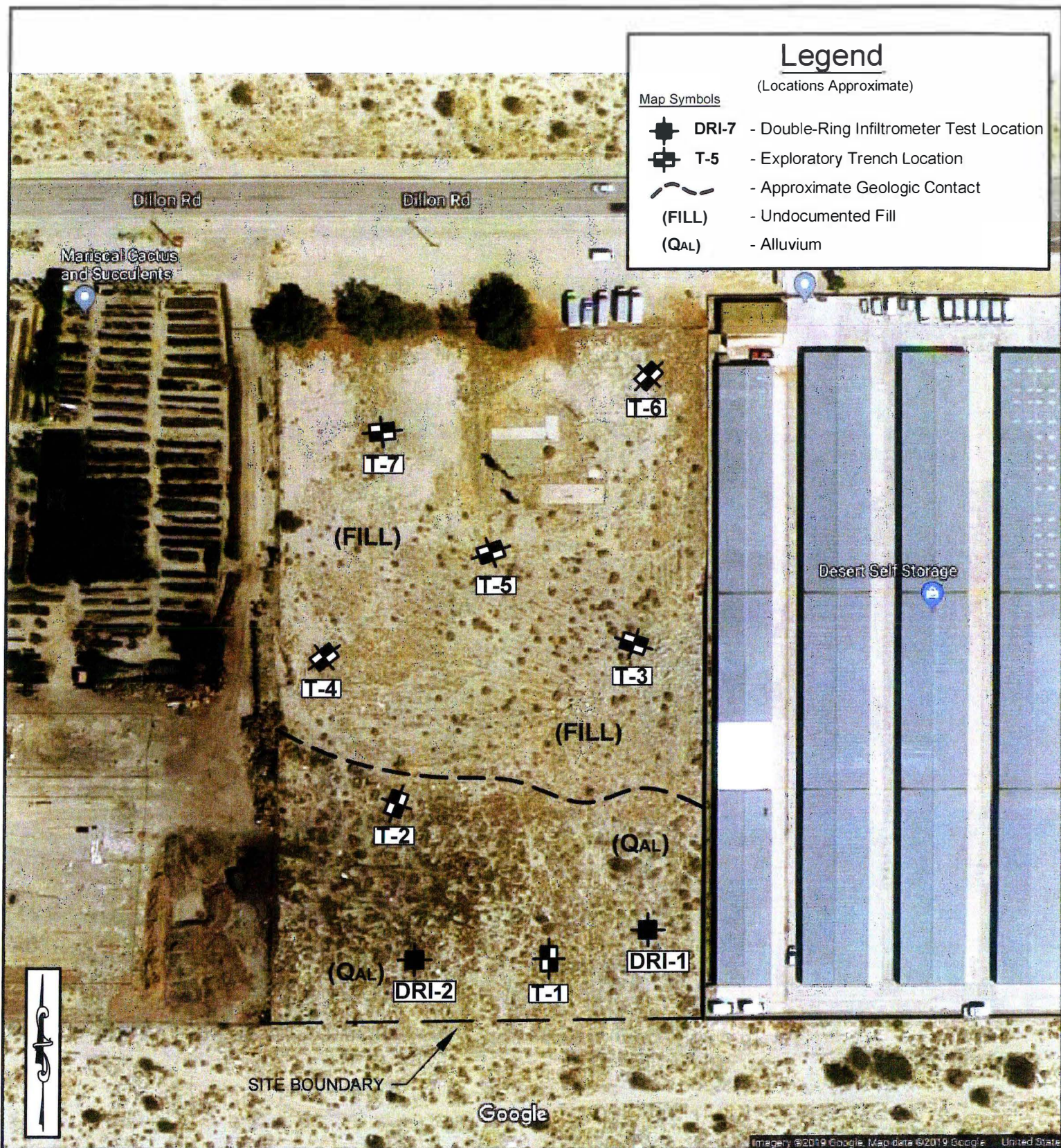
SCALE:

1" = 2,000'



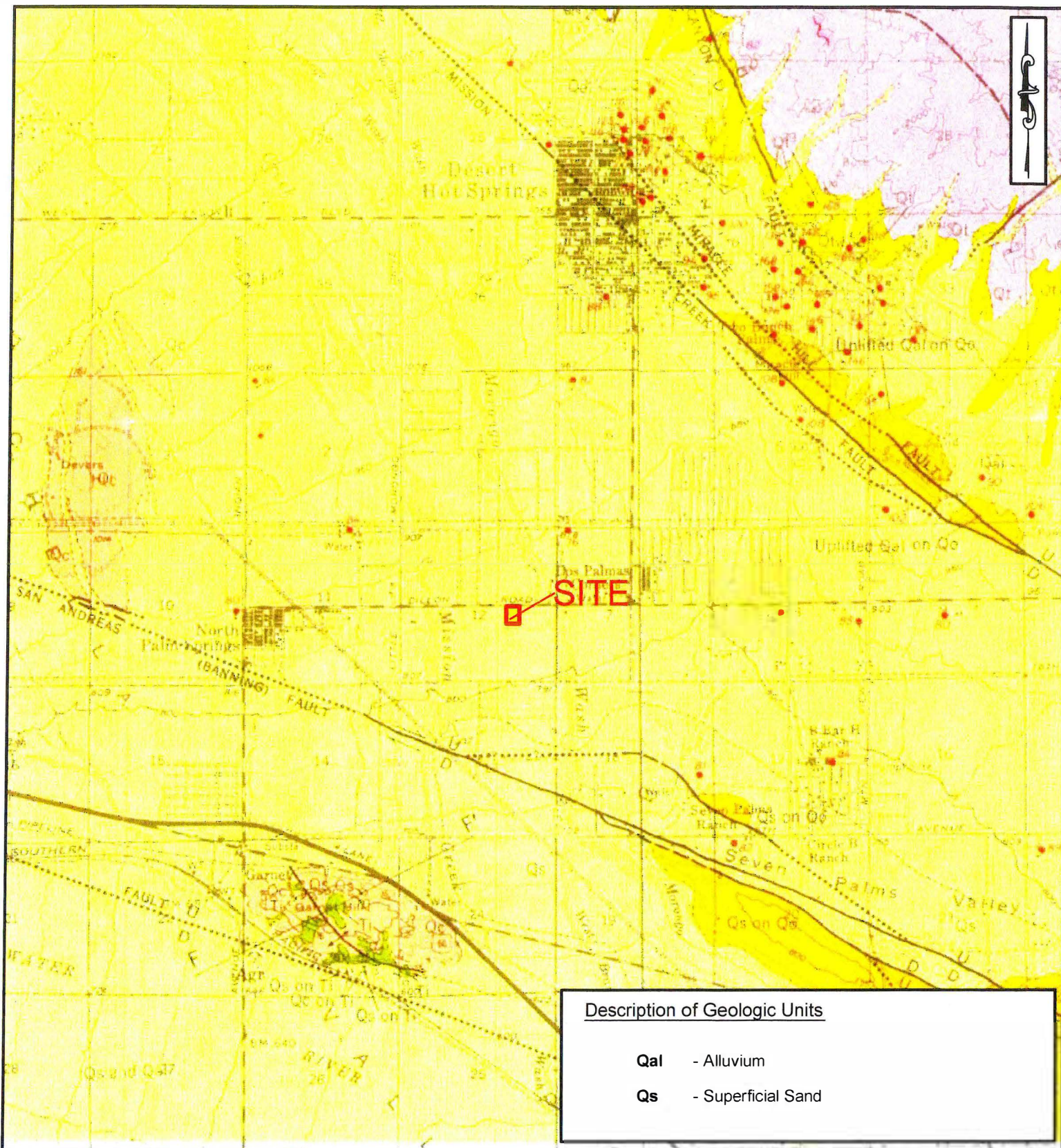
TRENCH LOCATION MAP

PROJECT:	APN: 657-220-023	DESERT HOT SPRINGS, CALIFORNIA	PROJECT NO:	11546.1
CLIENT:	LOR Geotechnical Group, Inc.		ENCLOSURE:	A-3
			DATE:	MAY 2019
			SCALE:	AS SHOWN



TRENCH LOCATION MAP

PROJECT:	APN: 657-220-003, DESERT HOT SPRINGS, CALIFORNIA	PROJECT NO:	13548.1
CLIENT:	REAL ESTATE INVESTMENTS	ENCLOSURE:	A-2b
LOR Geotechnical Group, Inc.		DATE:	MAY 2019
		SCALE:	1" ≈ 80'



REGIONAL GEOLOGIC MAP

(Proctor, 1968)

PROJECT: APN: 657-220-003, DESERT HOT SPRINGS, CALIFORNIA

PROJECT NO: 13548.1

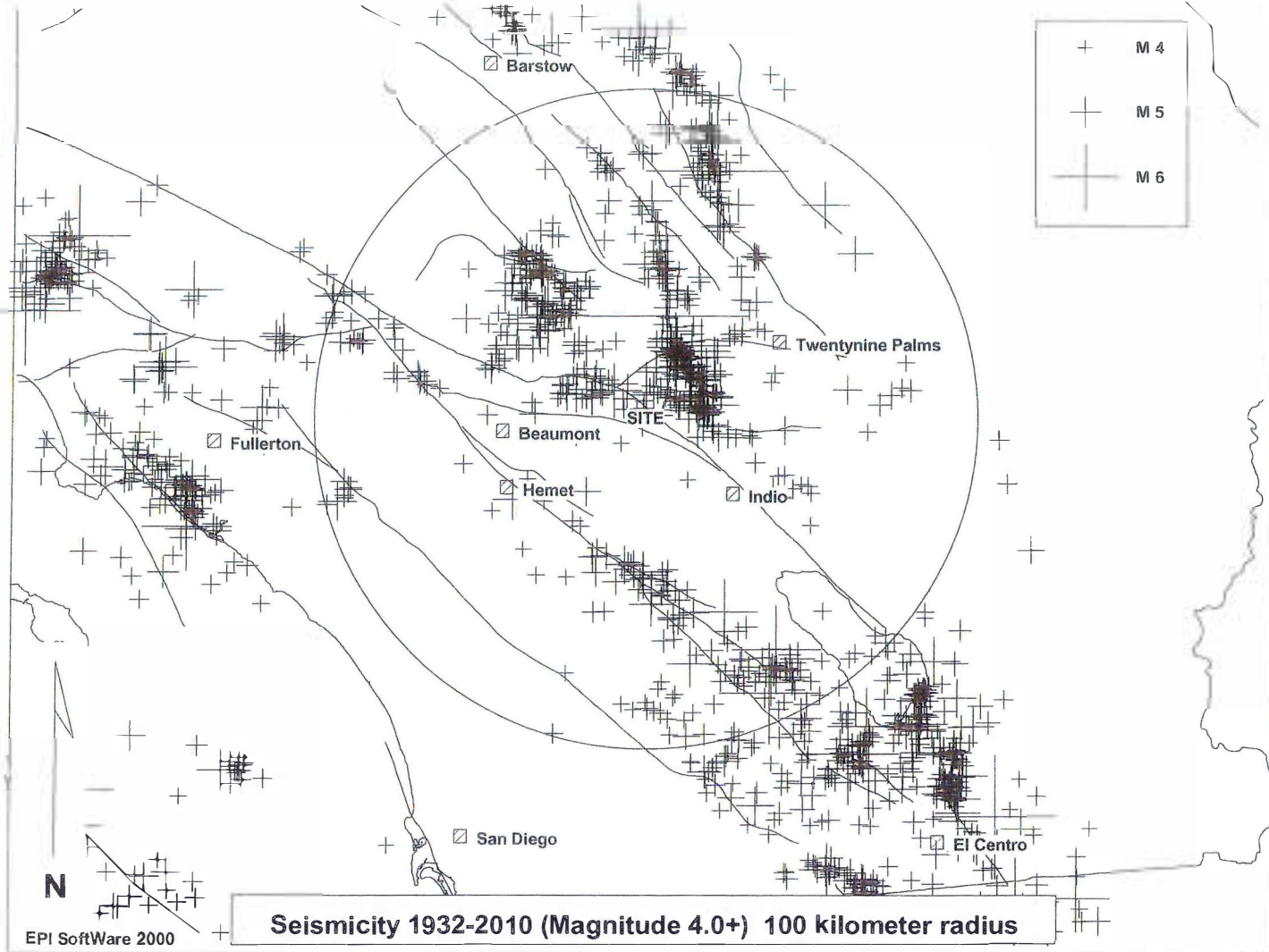
CLIENT: REAL ESTATE INVESTMENTS

ENCLOSURE: A-3

LOR Geotechnical Group, Inc.

DATE: MAY 2019

SCALE: 1" = 2,000'



SITE LOCATION: 33.9240 LAT. -116.5077 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1333

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 638

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 572

5.0- 5.9 : 58

6.0- 6.9 : 6

7.0- 7.9 : 2

8.0- 8.9 : 0

CLOSEST EVENT: 4.7 ON THURSDAY, JULY 24, 1947 LOCATED APPROX. 1.7 KILOMETERS SOUTHWEST OF THE SITE

LARGEST 5 EVENTS:

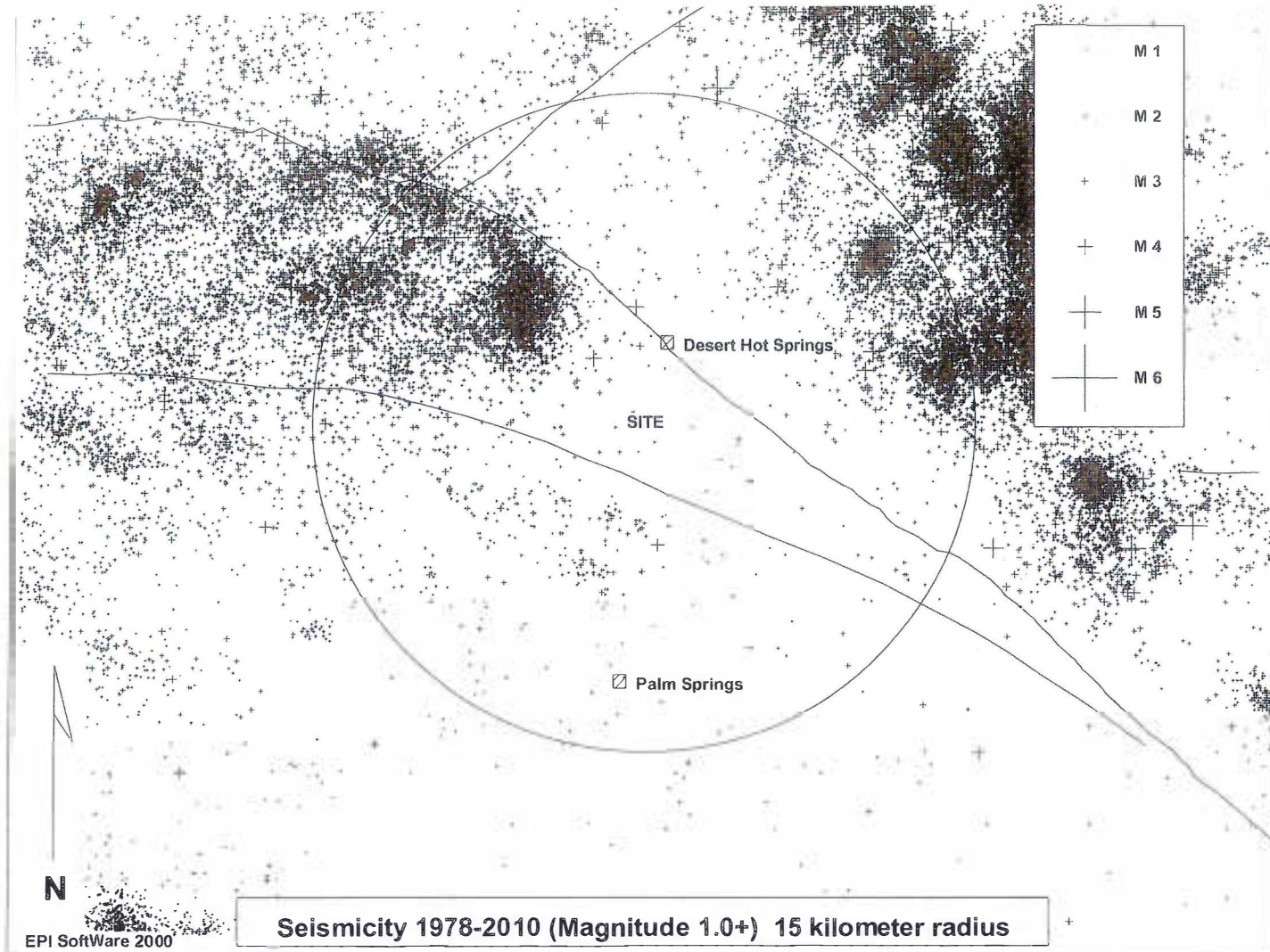
7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 31 KILOMETERS NORTH OF THE SITE

7.1 ON SATURDAY, OCTOBER 16, 1999 LOCATED APPROX. 77 KILOMETERS NORTH OF THE SITE

6.6 ON TUESDAY, APRIL 09, 1968 LOCATED APPROX. 90 KILOMETERS SOUTHEAST OF THE SITE

6.4 ON FRIDAY, MARCH 19, 1954 LOCATED APPROX. 80 KILOMETERS SOUTHEAST OF THE SITE

6.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 42 KILOMETERS NORTHWEST OF THE SITE



SITE LOCATION: 33.9240 LAT. -116.5077 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 28408

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 6191

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

1.0- 1.9 : 4023
 2.0- 2.9 : 2029
 3.0- 3.9 : 124
 4.0- 4.9 : 14
 5.0- 5.9 : 1
 6.0- 6.9 : 0
 7.0- 7.9 : 0
 8.0- 8.9 : 0

CLOSEST EVENT: 1.9 ON THURSDAY, JANUARY 19, 1989 LOCATED APPROX. .1 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

5.7 ON TUESDAY, JULY 08, 1986 LOCATED APPROX. 12 KILOMETERS NORTHWEST OF THE SITE
 4.5 ON THURSDAY, JULY 17, 1986 LOCATED APPROX. 14 KILOMETERS NORTHWEST OF THE SITE
 4.4 ON THURSDAY, JULY 17, 1986 LOCATED APPROX. 14 KILOMETERS NORTHWEST OF THE SITE
 4.3 ON WEDNESDAY, JANUARY 12, 2005 LOCATED APPROX. 10 KILOMETERS EAST OF THE SITE
 4.3 ON TUESDAY, JULY 08, 1986 LOCATED APPROX. 8 KILOMETERS NORTHWEST OF THE SITE



APPENDIX B

Field Investigation Program and Trench Logs

APPENDIX B

FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of excavation of 7 exploratory trenches to depths of between 7 and 14.5 feet below the existing ground surface. The trenches were excavated using a JD-410C backhoe. The approximate locations of the trenches are shown on Enclosure A-2.

The soils encountered within the trenches were continuously logged by a geologist from this firm who inspected the site, created detailed logs of the trenches, obtained soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the trenches are presented on the enclosed Trench Logs, Enclosures B-1 through B-7. A Trench Log Legend is presented on Enclosure B-I. A Soil Classification Chart is presented as Enclosure B-ii.

CONSISTENCY OF SOIL

SANDS

SPT BLOWS

0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

CONSISTENCY

COHESIVE SOILS

SPT BLOWS

0-2	Very Soft
2-4	Soft
4-8	Medium
8-15	Stiff
15-30	Very Stiff
30-60	Hard
Over 60	Very Hard

CONSISTENCY

Symbol



Description

INDICATES CALIFORNIA
SPLIT SPOON SOIL
SAMPLE

INDICATES BULK
SAMPLE

INDICATES SAND CONE
OR NUCLEAR DENSITY
TEST

INDICATES STANDARD
PENETRATION TEST
(SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

- 1 Atterberg Limits
- 2 Consolidation
- 3 Direct Shear (undisturbed or remolded)
- 4 Expansion Index
- 5 Hydrometer
- 6 Organic Content
- 7 Proctor (4", 6", or Cal216)
- 8 R-value
- 9 Sand Equivalent
- 10 Sieve Analysis
- 11 Soluble Sulfate Content
- 12 Swell
- 13 Wash 200 Sieve

TRENCH LOG LEGEND

PROJECT: PROPOSED STORAGE CENTER, DESERT HOT SPRINGS, CALIFORNIA

PROJECT NO.: 13548.1

CLIENT: REAL ESTATE INVESTMENTS

ENCLOSURE: B-i

LOR Geotechnical Group, Inc.

DATE: MARCH 2019

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND 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, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	
12"	3"	3/4"	No. 4 (U.S. STANDARD SIEVE SIZE)	No. 10	No. 40	200	

SOIL CLASSIFICATION CHART

PROJECT: PROPOSED STORAGE CENTER, DESERT HOT SPRINGS, CALIFORNIA				PROJECT NO.: 13548.1	
CLIENT: REAL ESTATE INVESTMENTS				ENCLOSURE: B-ii	
LOR Geotechnical Group, Inc.				DATE: MARCH 2019	

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C

LABORATORY TESTING

General

Selected soil samples obtained from the trenches were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction, direct shear, sieve analysis, sand equivalent, R-value, and soluble sulfate content. Descriptions of the laboratory tests are presented in the following paragraphs:

Moisture Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, in accordance with ASTM D 2921 and ASTM D 2216, respectively, and ASTM D 2922 and the results are shown on the trench logs, Enclosures B-1 through B-7, for convenient correlation with the soil profile.

Laboratory Compaction

Selected soil samples were tested in the laboratory to determine compaction characteristics using the ASTM D 1557 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Trench Number	Sample Depth (feet)	Soil Description U.S.C.S.	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
T-1	2-4	(SW/SM) Sand with Silt	120.0	9.5
T-2	1-3	(SW) Well Graded Sand	128.0	8.0

Direct Shear Tests

Shear tests are performed in general accordance with ASTM D 3080 with a direct shear machine at a constant rate-of-strain (0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in remolded condition (90 percent relative compaction per ASTM D 1557) and soaked, to represent the worse case conditions expected in the field.

The results of the shear tests are presented in the following table:

DIRECT SHEAR TESTS				
Trench Number	Sample Depth (feet)	Soil Description U.S.C.S.	Angle of Internal Friction (degrees)	Apparent Cohesion (psf)
T-1	2-4	(SW/SM) Sand with Silt	45	24
T-2	1-3	(SW) Well Graded Sand	41	240

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with the ASTM D 422 laboratory test procedure. The determination is performed by passing the soil through a series of sieves, and recording the weights of retained particles on each screen. The results of the sieve analyses are presented graphically on Enclosure C-1.

Sand Equivalent

The sand equivalent of selected soils were evaluated using the California Sand Equivalent Test Method, Caltrans Number 217. The results of the sand equivalent tests are presented with the grain size distribution analyses on Enclosure C-1.

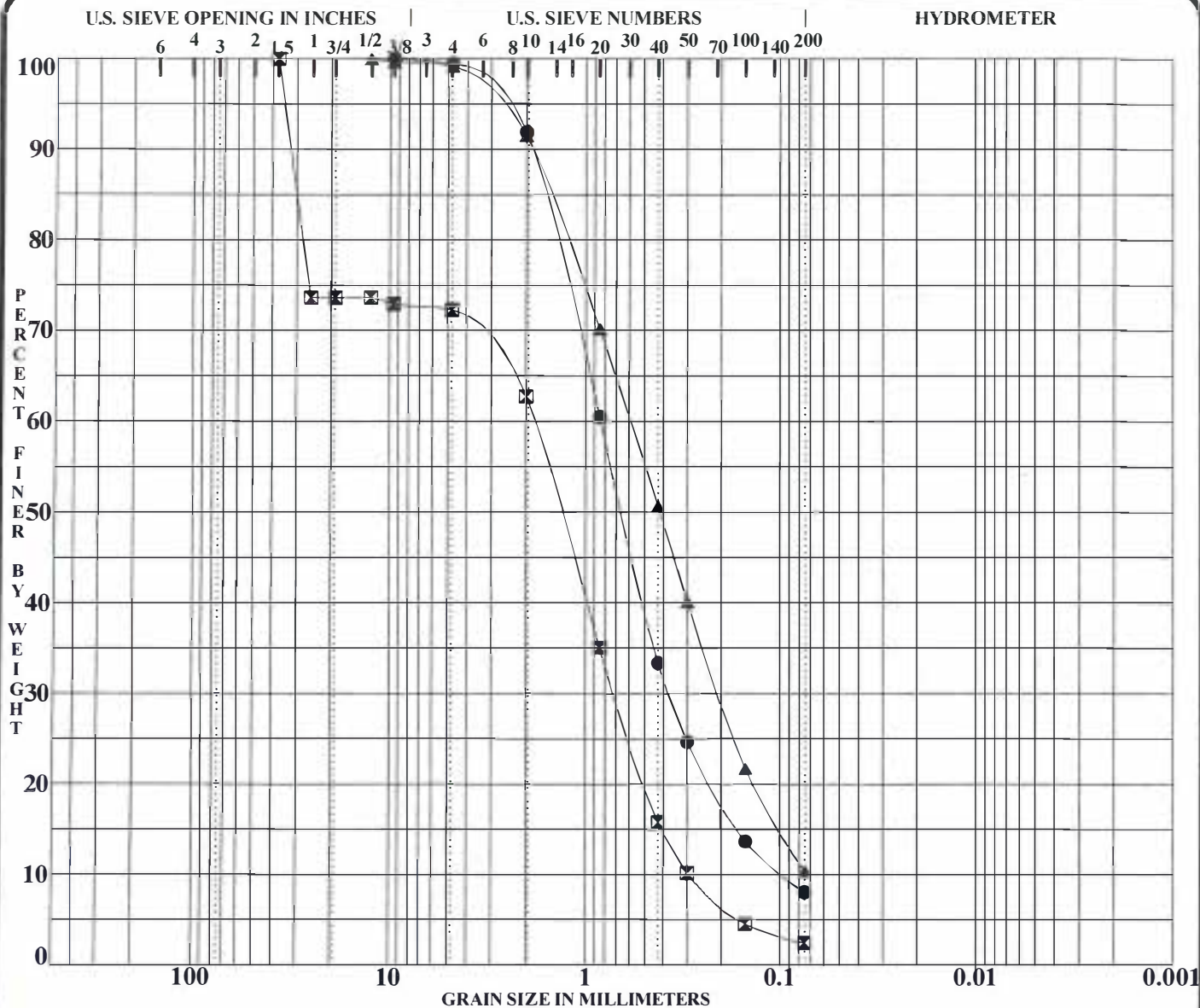
R-Value Test

Soil samples were obtained at probable pavement subgrade level and sieve analysis and sand equivalent tests were conducted. Based on these indicator tests, a selected soil sample was tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The results of the sieve analysis, sand equivalent, and R-value tests are presented on Enclosure C-1.

Soluble Sulfate Content Tests

The soluble sulfate content of selected subgrade soils were evaluated. The concentration of soluble sulfates in the soils was determined by measuring the optical density of a barium sulfate precipitate. The precipitate results from a reaction of barium chloride with water extractions from the soil samples. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test results are presented on the following table:

SOLUBLE SULFATE CONTENT TESTS			
Boring Number	Sample Depth (feet)	Soil Description U.S.C.S.	Sulfate Content (percent by weight)
T-1	2-4	(SW/SM) Sand with Silt	<0.005
T-2	1-3	(SW) Well graded Sand	<0.005
T-4	3-5	(SM) Silty Sand	>0.02
T-6	4.5-6	(SW) Well graded Sand	<0.005



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	Soil Classification					SE	RV	PL	PI	Cc	Cu
● T-1 @ 2-4 ft	(SW-SM) Well Graded Sand with Silt					68	--			1.72	8.8
☒ T-2 @ 1-3 ft	(SP) Poorly Graded Sand with Gravel					81	--			0.93	6.3
▲ T-6 @ 4.5-6 ft	(SW-SM) Well Graded Sand with Silt					59	70			0.97	8.1

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● T-1 @ 2-4 ft	9.50	0.84	0.371	0.0956	0.5	91.4	8.0	
☒ T-2 @ 1-3 ft	37.50	1.84	0.709	0.2937	27.7	69.9	2.4	
▲ T-6 @ 4.5-6 ft	12.50	0.59	0.205		0.8	88.8	10.4	

PROJECT Proposed Storage Center Desert Hot Spings

PROJECT NO. 13548.1

DATE 5/28/19

GRADATION CURVES
LOR Geotechnical Group, Inc.

ENCLOSURE C-1

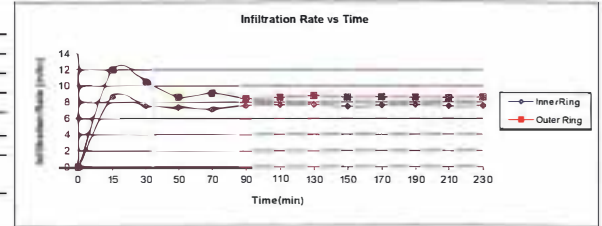
APPENDIX D

Infiltration Test Results

DOUBLE RING INFILTROMETER TEST DATA

Project: Desert Hot Springs Storage Center
Project No.: 13548.1
Soil Classification: (SW-SM) Well Graded Sand with Silt
Depth of Test Hole: 3 ft
Liquid Used: Tap Water
Area of Rings: Inner = 0.785 ft², Annular 2.36 ft²
Tested By: A. L.
Liquid Level Maintained Using: Vacuum Seal
Depth to Water Table: > 100 ft

Client: Real Estate Investments
Test Date: May 16, 2019
Test Hole No.: DRI-1
Test Hole Diameter: 12 in. inner, 24 in. annular
Date Excavated: May 16, 2019
pH: 7.8
Depth of Water in Rings: 2 in
 4 in

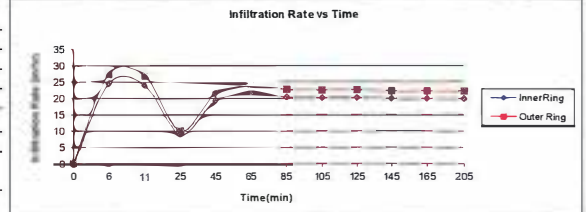


TRIAL NO.	TEST PERIOD															LIQUID TEMP (°F)	REMARKS
	INNER			ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)				
	TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space			
1	S 8:39	15	15	8:39	15	15	8.82	36.75	1.059	4.412	129.5	179.5	8.7	12.0	60		
	E 8:54			8:54											60		
2	S 8:57	15	30	8:57	15	30	7.68	32.19	0.922	3.864	112.8	157.2	7.6	10.5	60	refilled outer	
	E 9:12			9:12											60		
3	S 9:15	20	50	9:15	20	50	9.97	35.22	1.197	4.228	109.8	129.0	7.4	8.6	60	refilled outer	
	E 9:35			9:35											60		
4	S 9:37	20	70	9:37	20	70	9.66	37.00	1.160	4.442	106.4	135.5	7.1	9.1	62	refilled outer	
	E 9:57			9:57											62		
5	S 10:13	20	90	10:13	20	90	10.31	34.53	1.238	4.145	113.5	126.5	7.6	8.5	62	refilled both	
	E 10:33			10:33											62		
6	S 10:35	20	110	10:35	20	110	10.42	35.00	1.251	4.202	114.7	128.2	7.7	8.6	64	refilled outer	
	E 10:55			10:55											64		
7	S 10:58	20	130	10:58	20	130	10.49	35.91	1.259	4.311	115.5	131.5	7.7	8.8	64	refilled outer	
	E 11:18			11:18											64		
8	S 11:20	20	150	11:20	20	150	10.11	34.96	1.214	4.197	111.3	128.0	7.5	8.6	64	refilled outer	
	E 11:40			11:40											66		
9	S 11:41	20	170	11:41	20	170	10.32	35.21	1.239	4.227	113.6	129.0	7.6	8.6	66	refilled outer	
	E 12:01			12:01											66		
10	S 12:05	20	190	12:05	20	190	10.41	35.06	1.250	4.209	114.6	128.4	7.7	8.6	66	refilled both	
	E 12:25			12:25											68		
11	S 12:27	20	210	12:27	20	210	10.17	34.84	1.221	4.182	112.0	127.6	7.5	8.5	68	refilled outer	
	E 12:47			12:47											66		
12	S 12:50	20	230	12:50	20	230	10.26	35.13	1.232	4.217	113.0	128.7	7.6	8.6	68		
	E 13:10			13:10											70		

DOUBLE RING INFILTROMETER TEST DATA

Project: Desert Hot Springs Storage Center
Project No.: 13548.1
Soil Classification: (SW) Well Graded Sand
Depth of Test Hole: 3ft
Liquid Used: Tap Water
Area of Rings: Inner = 0.785 ft², Annular 2.36 ft²
Tested By: A. L.
Liquid Level
Maintained Using: Vacuum Seal
Depth to Water Table: > 100 ft

Client: Real Estate Investments
Test Date: May 16, 2019
Test Hole No.: DRI-2
Test Hole Diameter: 12 in. inner, 24 in. annular
Date Excavated: May 16, 2019
pH: 7.8
Depth of Water in Rings: 1 in
Ring Penetration: 4 in



TEST PERIOD

TRIAL NO.	INNER			ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	inner	annular space*	inner	annular space	inner	annular space	inner	annular space		
1	S 9:40	6	6	9:40	6	6	9.98	33.54	1.198	4.026	366.3	409.5	24.5	27.4	62	outer ring metered manually entire test
	E 9:46			9:46											62	
2	S 9:46	5	11	9:46	5	11	8.15	27.39	0.978	3.288	359.0	401.3	24.0	26.9	62	
	E 9:51			9:51											62	
3	S 9:51	14	25	9:51	14	25	8.53	28.67	1.024	3.442	134.2	150.0	9.0	10.1	62	
	E 10:05			10:05											64	
4	S 10:08	20	45	10:08	20	45	26.17	87.94	3.142	10.557	288.2	322.1	19.3	21.6	64	inner refilled
	E 10:28			10:28											64	
5	S 10:31	20	65	10:31	20	65	29.11	97.84	3.495	11.745	320.5	358.4	21.5	24.0	64	inner refilled
	E 10:51			10:51											64	
6	S 10:53	20	85	10:53	20	85	27.64	92.80	3.318	11.140	304.3	339.9	20.4	22.8	64	inner refilled
	E 11:13			11:13											66	
7	S 11:15	20	105	11:15	20	105	27.33	91.85	3.281	11.026	300.9	336.4	20.2	22.5	66	inner refilled
	E 11:35			11:35											66	
8	S 11:38	20	125	11:38	20	125	27.43	92.18	3.293	11.066	302.0	337.6	20.2	22.6	66	inner refilled
	E 11:58			11:58											66	
9	S 12:00	20	145	12:00	20	145	26.94	90.54	3.234	10.869	296.6	331.6	19.9	22.2	66	inner refilled
	E 12:20			12:20											68	
10	S 12:24	20	165	12:24	20	165	27.01	90.77	3.242	10.897	297.4	332.5	19.9	22.3	68	inner refilled
	E 12:44			12:44											68	
11	S 12:47	20	185	12:47	20	185	27.13	91.18	3.257	10.946	298.7	334.0	20.0	22.4	68	inner refilled
	E 13:07			13:07											70	
12	S 13:10	20	205	13:10	20	205	26.93	90.50	3.233	10.864	296.5	331.5	19.9	22.2	70	
	E 13:30			13:30											71	

*estimated