LOR GEOTECHNICAL GROUP, INC.
Soil Engineering ▲ Geology ▲ Environmental

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
ASSESSOR'S PARCEL NUMBER 488-310-012
NORTHWEST CORNER OF SPRUCE AVENUE
AND REDLANDS BOULEVARD
MORENO VALLEY, CALIFORNIA

PROJECT NO. 13358.1
AUGUST 9, 2017

Prepared For:

Anthem Oil, Inc.
2640 Camino Del Sol
Fullerton, California 92833

Attention: Mr. Chandresh Ravaliya

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Project No. 13358.1

Attention: Mr. Chandresh Ravaliya

Subject: Preliminary Geotechnical Investigation, Proposed Commercial Development, Assessor's Parcel Number 488-310-012, Northwest Corner of Spruce Avenue and Redlands Boulevard, Moreno Valley, California.

LOR Geotechnical Group, Inc. is pleased to present this report summarizing our geotechnical investigation for the proposed retail structures and associated improvements to be located at the northwest corner of Spruce Avenue and Redlands Boulevard in the City of Moreno Valley.

In summary, it is our opinion that the site can be developed from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction. The following summary reviews some of the important elements of the project, however, this summary should not be solely relied upon.

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. All undocumented fill material and all loose older alluvial materials should be removed from areas to receive engineered compacted fill and settlement prone improvements, (ie, flatwork, pavements, canopies, etc.). The data developed during this investigation indicates that removals of approximately 5 to 7 feet below existing grades within currently planned structural areas will be required.

The on-site soils were tested and found to have a very low expansion potential. Soluble sulfate content testing of the on-site soils found them to have negligible soluble sulfate content. The on-site soils were found to have a poor R-value quality.

LOR Geotechnical Group, Inc.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

INTRODUCTION

During July and August of 2017, a Preliminary Geotechnical Investigation was performed by LOR Geotechnical Group, Inc. for the proposed commercial development of Assessor's Parcel Number 488-310-012, located at the northwest corner of Spruce Avenue and Redlands Boulevard in the City of Moreno Valley, 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 residential type development. The scope of our services included:

- Review of available pertinent geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Geologic field reconnaissance mapping to verify the aerial 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;
- Infiltration testing via the double ring infiltrometer test method for the feasibility and design of infiltration systems;
- Laboratory testing of selected soil samples obtained during the field investigation;
- 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.

To orient our investigation at the site, you provided us with a Site Plan of the site. The proposed development is shown on this map. A copy of this map was utilized as a base for our investigation and is shown on Enclosure A-2, within Appendix A.

PROJECT CONSIDERATIONS

Information furnished to this firm indicates the proposed project will consist of the construction of gas station, truck stop, fast food restaurants, office space, and the associated improvements on the 6.76 ± acre site.

Based on the topography of the site and adjacent properties, grading will most likely incorporate cuts and fills on the order of a few feet. Although the type of construction for the buildings has not yet been stated, it will most likely consist of wood frame and stucco, one story structures. Light to moderate foundation loads are associated with such structures.

EXISTING SITE CONDITIONS

The subject site is a roughly 'L'-shaped parcel comprising approximately 6.76 acres located at the northwest corner of Spruce Avenue and Redlands Boulevard, in the City of Moreno Valley, California. The topography of the site is generally planar with a gentle fall to the south.

At the time of our investigation the subject site was vacant of structures. Some minor wind blown trash was present. Power poles were present along the north and east site boundaries. Hemlock Avenue, not yet existing, bounds the site on the north with vacant land beyond. Vacant land lies adjacent the site on the west. An Eastern Municipal Water District pump station lies to the east of the northern half of the site. The remainder of the eastern half of the site is bound by Redlands Boulevard and Spruce Avenue, a two lane paved roadway. Further east, large lot residential properties are present. South of the site lies a horse ranch.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on July 13 and 21, 2017 and consisted of drilling 6 exploratory borings with a truck-mounted Mobile B61B drill rig equipped with 8-inch diameter hollow stem augers and excavating 4 exploratory test pits using a rubber tire backhoe equipped with an 18-inch bucket. The borings were extended to depths ranging from approximately 21.5 feet to 51.5 feet below the existing ground surface. The test pits were excavated to depths of approximately 14 to 14.5 feet below the existing ground surface. The approximate locations of our exploratory borings and trenches are presented on the enclosed Plate, Enclosure A-2, within Appendix A.

Logs of the subsurface conditions encountered in the exploratory borings were created by a geologist from this firm. Bulk samples of the encountered materials were obtained and returned to our geotechnical laboratory in sealed containers for further testing and evaluation. Relatively undisturbed samples were obtained at maximum intervals of 5 feet and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

The subsurface conditions encountered in the exploratory trenches were logged by a geologist from this firm. In-place density determinations were conducted at selected levels within the trenches utilizing the Nuclear Gauge Method (ASTM D 2922). 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 boring and trench logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

The subject site is situated along the northeastern end of Moreno Valley that lies just south of the junction of the Box Springs Mountains and the hills of the Badlands. This area is located within the northeastern portion of the Peninsular Ranges Geomorphic Province of southern California. This province incorporates several northwest trending mountain ranges, such as the Santa Ana and San Jacinto Mountains, which extend from the Transverse Ranges Geomorphic Province, northeast of Los Angeles, into the Baja California Peninsula. Lying in between these small ranges are a series of valleys and basins, such as the Perris Plain. The Perris Plain is composed of rocks of the Peninsular Ranges batholith, a very large mass composed primarily of batholithic crystalline igneous rocks, with lesser amounts of metasedimentary and metavolcanic rocks which predate the intrusion of the batholith, such as the hills of the Box Springs Mountains to the north of the site. These rocks actually consist of numerous separate plutonic intrusions which range in composition from gabbro to granite, with tonalite the predominate lithology. Erosion of the hills has resulted in the covering of a thin to thick veneer of various ages of alluvial fan materials across the flank of the hills and out into the adjoining valley floor to the south. The current drainage pattern of the northeastern section of Moreno Valley flows to the south, then turns to the southwest where southward flow is blocked by Mount Russell. This pattern has eroded off some of the older alluvial fan materials and subsequently deposited various amounts of relatively younger, unconsolidated alluvial sediments along the lower reaches of the valley.

The interior of the Perris Plain is considered to be relatively stable with few known active faults. However, this plain is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the northeast, the San Andreas fault zone on the north, and the Agua-Tibia fault zone on the south. As the subject site is located near the northeastern margin of Perris Plain, the San Jacinto fault is the closest known active fault in relation to the site. At its closest approach, the San Jacinto fault is located approximately 0.9 kilometers (0.6 miles) northeast from the site. A complete listing of the distances to known active faults in relation to the site is given in the Faulting section of this report.

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

Site Geologic Conditions

As observed during this investigation, the subject site is underlain by fill/topsoil overlying native alluvial materials. These units are described in further detail in the following sections:

Surficial Deposits

Fill/Topsoil: The surface of the site contained a layer of materials which have been altered in some manner. These materials were noted to generally consist of silty sand which was tan, dry, and in a loose state. These units were noted to generally be on the order of two feet in thickness with a local area noted to be 5 feet in thickness.

Alluvium: Underlying the surficial materials, natural units of alluvium were encountered. These units consisted of primarily of silty sand to sandy silt with minor units of well and poorly graded sand, sandy clay, and clayey sand beneath approximately 10 feet. Typically, the fine grained alluvial materials were yellow brown to red brown in color while the sandier units were yellow brown to white. Typically, the alluvial units were dry to damp. Some pinhole porosity and secondary calcite was present within the finer grained units within the upper portions. Based on the results of the equivalent SPT blow counts, it was noted that the alluvial materials were in loose/soft to medium dense/medium within the upper 10 to 12 feet, becoming medium dense/stiff to dense/hard beneath. Consolidation testing was conducted on representative in-place samples at depths of 5, 7, 10 and 12 feet. Testing indicates a potential for hydroconsolidation of the materials at a depth of 5 feet. Samples tested beneath this depth are considered to have normal hydroconsolidation characteristics. Hydroconsolidation is the unfavorable collapse of soil under a relatively normal load upon the introduction of water. The samples tested are indicative of relatively low density, porous, dry alluvial units.

A detailed description of the subsurface soil conditions as encountered within our exploratory borings is presented on the Boring Logs within Appendix B.

Groundwater Hydrology

Groundwater was not encountered in any of our exploratory borings as advanced to a maximum depth of approximately 51.5 feet, nor was any groundwater seepage observed during our site reconnaissance on the subject site.

Records for nearby wells which were readily available from the State of California Department of Water Resources online database (DWR, 2017) and the Western Municipal Water District Cooperative Well Measurement Program (WMWD, 2016) were reviewed as a part of this investigation.

According to the State of California Department for Water resources online database, the nearest wells are EMWD12047 and EMWD14352 located approximately 0.8 kilometers (0.5 miles) to the southwest of the site. Data for these wells was present from November of 2011 through April of 2016. Groundwater levels ranged from depths of approximately 197 to 220 feet during that time. A measuring point elevation of 1,791 and 1,786 feet above mean sea level, respectively, was provided.

A review of the Cooperative Well Measuring Program, Fall 2015 database identified 4 wells nearby the site (03S/03S-02L001S,-02L002S, -02L001R, and -02L002R). These wells lie approximately 0.4 miles to the southwest. Data was available from November of 2004 to April of 2015. The depth of the water in the wells from this time period was approximately 200 feet. Measuring point elevations ranged from 1,774 to 1,791 feet above mean sea level.

Based on the information above, groundwater is anticipated to lie on the order of 200 feet beneath the site.

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 are 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 or project into the subject site, nor was any evidence of faulting or lineaments noted during our field investigation of the site. In addition, the site does not lie within a current State of California Earthquake Fault Zone (Hart, 1997) nor within a County of Riverside Fault Zone (TLMA, 2017).

As previously mentioned, the closest known fault with a well documented location is the San Jacinto fault, located approximately 0.9 kilometers (0.6 miles) to the northeast. Other active earthquake faults in the region include the San Andreas fault located approximately 18 kilometers (11 miles) to the northeast, the Elsinore fault located approximately 34.5 kilometers (21.5 miles) to the southwest, and the

Cucamonga fault located approximately 37 kilometers (23 miles) to the north-northwest.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or larger.

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.

The Elsinore fault zone is one of the largest in southern California. At its northern end it splays into two segments and at its southern end it is cut by the Yuba Wells fault. The primary sense of slip along the Elsinore fault is right lateral strike-slip. It is believed that the Elsinore fault zone is capable of producing an earthquake magnitude on the order of 6.5 to 7.5.

The Cucamonga fault is considered to be part of the Sierra Madre fault system which marks the southern boundary of the San Gabriel Mountains. This is a north dipping thrust fault which is believed to be responsible for the uplift of the San Gabriel Mountains. It is believed that the Cucamonga fault is capable of producing an earthquake magnitude on the order of 7.0.

Current standards of practice have 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 greater distance and/or smaller anticipated magnitudes.

<u>Historical Seismicity</u>

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 Jacinto and San Andreas faults trending southeast to northwest. Of these events, the closest was a magnitude 4.1 located approximately 7 kilometers (4.5 miles) north of the site.

In the second search, the micro seismicity of the area lying within a 10 kilometer (6.2 miles) radius of the site was examined by selecting an epicenter map listing events on the order of 0.0 and greater since 1977. 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 40 ± 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 nearby San Jacinto fault appears to be the source of numerous events.

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 faults described within. 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 50 plus feet, 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 absence of any large bodies of water very 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 in earthquake and effect 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.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Because the site is underlain by relatively dense alluvium at depth and grading of the site will remove any undocumented fill soils and any near surface loose alluvial soils, the potential for settlement is considered nil.

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* Site Location (USGS WGS84) 33.9419, -117.1581, Risk Category III	
Site Class Definition Chapter 20 ASCE 7	D
S_s Mapped Spectral Response Acceleration at 0.2s Period, (Figure 1613.3.1(1))	2.431
S_1 Mapped Spectral Response Acceleration at 1s Period, (Figure 1613.3.3(2))	1.113
F_a Short Period Site Coefficient at 0.2s Period, (Table 1613.3.3(1))	1.0
F_v Long Period Site Coefficient at 1s Period, (Table 1613.3.3(2))	1.5
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period, (eq .16-37)	2.431
S_{M1} Adjusted Spectral Response Acceleration at 1s Period, (eq .16-38)	1.670
S_{DS} Design Spectral Response Acceleration at 0.2s Period, (eq .16-39)	1.620
S_{D1} Design Spectral Response Acceleration at 1s Period, (eq .16-40)	1.113
Seismic Design Category - Short Period (Table 1613.3.5(1))	E
Seismic Design Category - Long Period (Table 1613.3.5(2))	E
*Values obtained from U.S.G.S. online U.S. Seismic Design Maps tool	

INFILTRATION TESTING AND TEST RESULTS

Two double ring infiltration tests were conducted at the approximate locations illustrated on Enclosure A-2. Test pits were excavated to a depth of approximately 6 feet, as requested. A 12-inch diameter casing was installed within the center of the test location with a 24-inch diameter casing centered around it. Each casing was imbedded to a depth approximately 5 inches. These liners extended approximately 15-inches above the bottom of the test location. 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 height of approximately 1 to 1.5-inches.

The testing procedure was as follows:

Both the inside and outside area of the casings were filled with water to a level of approximately 1 to 1.5-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 inner ring. See the attached Infiltration Test Data sheets, Enclosures D-1 and 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.	Depth (ft.)	Clear Water Infiltration Rate*	
		gal/sf/day	in/hr
DRI-1	3	27.6	1.8
DRI-2	3	17.1	1.1
* average of final two readings			

The clear water percolation rates obtained in our test locations were 1.1 and 1.8 inches per hour.

The borings placed during this evaluation in the general area proposed for infiltration indicates that the subsurface soils generally consists of silty sand within the tests at a depth of 6 feet. No groundwater was encountered as explored to a maximum depth of approximately 51.5 feet within Boring B-1.

CONCLUSIONS

This investigation provides a broad overview of the geotechnical and geologic factors which are expected to influence future site planning and development. 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 geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

The subsurface conditions encountered in our exploratory borings and trenches are indicative of the locations explored. The subsurface conditions presented here are not to be construed as being present the same everywhere on 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.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the existing fills and the upper native soils, will not, in their present condition, provide uniform and/or adequate support for the proposed structures. Our equivalent Standard Penetration Test (SPT) data indicated variable in-situ conditions of the existing fills and upper native soils, typically ranging from loose to medium dense and dense states. In addition, our consolidation testing indicates a potential for hydroconsolidation of the materials to a depth of 5 to 7 feet. This condition could cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads and other site improvements. Therefore, in order 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. In addition, the construction of this compacted fill mat will allow for the removal of the existing unsuitable earth materials within the building pad and other site improvement areas. 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.

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 known 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 Jacinto fault zone, as described above, it is reasonable to expect a 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 Jacinto 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

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

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 owner/developer, the contractor, and geotechnical engineer, and jurisdictional agency should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions 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. These materials should not be incorporated within engineered compacted fill. It is our recommendation that any existing fills that may be present be removed and replaced with engineered compacted fill.

Any uncontrolled fills encountered during site preparation should be completely removed, cleaned of significant deleterious materials, and may then be reused as compacted fill. Uncontrolled fills were identified at the site during this study and these will be encountered within the grading areas. Areas of deeper fill than that identified may be present locally, primarily in areas of any previous development.

It is our recommendation that all existing uncontrolled and/or undocumented fills, buried obstructions, under any proposed flatwork and paved areas should 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 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.

Initial Site Preparation

All existing fill and the upper portions of the alluvial materials should be removed from areas to receive engineered compacted fill and/or supporting site improvements that are sensitive to settlement, (ie, flatwork, pavements, canopies, etc.). The data developed during this investigation indicates that removals on the order of 5 to 7 feet will be required across the site to encounter competent alluvial materials. Removals should expose alluvial materials with a relative in-situ compaction of at least 83 percent and/or an in-situ saturation of at least 75 percent. The actual depths of removal should be verified during the grading operation by observation and in-place density testing.

Preparation of Fill Areas

Prior to placing fill, the surfaces of all areas to receive fill should be scarified to a depth of at least 12 inches. The scarified soil should be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent (ASTM D 1557).

Preparation of Foundation Areas

All footings should rest entirely upon a minimum of 24 inches of properly compacted fill material placed over competent native soils. In areas where the required fill thickness is not accomplished by the removal of the existing fil and loose native soils, 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 removals in excess of 5 feet deep are required, the removal areas 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 recompacted to at least 90 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill.

Engineered Compacted Fill

All fill materials should be 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.

If required, import fill 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 uniform, 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. The upper 12 inches of areas to be paved should be compacted to at least 95 percent (ASTM D 1557).

Based upon the relative compaction of the near surface fill and alluvial soils determined during this investigation and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage factor of approximately 20 to 25 percent. Therefore, 1.20 to 1.25 cubic yards of in-place materials would be necessary to yield one cubic yard of properly compacted fill material. The volume used in calculations should include the processed bottom. These values are for estimating purposes only, and are exclusive of losses due to stripping or the removal of subsurface obstructions. These values may vary due to differing conditions within the project boundaries and the limitations of this investigation. Shrinkage/bulkage should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

Careful evaluation of on-site soils and any import fill for their expansion potential should be conducted during the grading operation.

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 excavation 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 borings, it appears that the alluvial soils can be classified as Type C soils. These are the predominant types of soil and rock on the project and all short-term excavation should be based on these types 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 materials encountered during this investigation were tested and found 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 imported soils for their expansion potential should be conducted following completion of the grading operation.

Foundation Design

If the site is prepared as recommended, the proposed structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing either on a minimum of 24 inches of engineered compacted fill or bearing entirely on competent native materials. 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, spread foundations may be designed using an allowable bearing pressure of 1,500 psf. This bearing pressure may be increased by 300 psf for each additional foot of width or depth, up to a maximum of 3,000 psf. For example, a footing 2 feet wide and embedded 3 feet will have an allowable bearing pressure of 2,700 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. Buildings should be setback from slopes as detailed on the California Building Code.

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 400 pounds per square foot per foot of depth. Base friction may be computed at 0.45 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by 1/3 for wind or seismic.

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 settlements 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.

Slabs-On-Grade

To provide adequate support, after conducting the removals discussed above, concrete slabs-on-grade should bear on a minimum of 24 inches of compacted soil. 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, after conducting the removals discussed above, 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 retaining walls should be performed in accordance with the recommendations described earlier within under Preparation of Foundation 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 pressure of 40 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of recompacted, non-expansive, native soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

Retaining walls subject to uniform surcharge loads within a horizontal distance behind the structure equal to the structural height should be designed to resist additional lateral loads equal to 0.3 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

To avoid over stressing 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, properly drained, non-expansive 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).

Preliminary Pavement Design

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

TYPE OF TRAFFIC	TRAFFIC INDEX (T.I.)	DESIGN R-VALUE	PRELIMINARY SECTION
Light Vehicle and Incidental Truck Traffic	5.0	10	0.25' AC/0.75' AB or 0.42' JPCP/0.35' AB
Light Vehicle and Minor Truck Traffic	6.0	10	0.25' AC/1.05' AB or 0.50' JPCP/0.50' AB *
Truck Traffic	8.0	10	0.40' AC/1.35' AB 0.58'JPCP/0.50' AB**

AC - Asphalt Concrete
AB - Class 2 Aggregate Base
JPCP - Jointed Plain Concrete Pavement with MR ≥ 625 psi
* Average Daily Truck Traffic (ADTT) = 100 tractor trailer units with one or more trailers
** Average Daily Truck Traffic (ADTT) = 300 tractor trailer units with one or more trailers

The above structural section is predicated upon the removals discussed within and upon 90 percent relative compaction (ASTM 1557) of all utility trench backfills and 95 percent relative compaction (ASTM 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 0.5 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 soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented on Enclosure C.

Based on the test results, it appears that there is a negligible sulfate exposure to concrete elements in contact with the on site soils per the 2016 California Building Code. Therefore, no specific recommendations are given for concrete elements to be in contact with the on site soils.

Infiltration

Based upon our field investigation and infiltration test data, a clear water absorption rate of 1.1 inches per hour appears to be applicable for the planned infiltration areas to be placed at a depth of 6 feet in the locations tested. However, due to the hydroconsolidation potential of the on-site alluvium within the upper 5 to 7 feet, infiltration should not occur within the upper 7 feet. An appropriate factor of safety should be applied as stated within the Riverside County Flood Control and Water Conservation District Design Handbook for Low Impact Development Best Management Practices (2011).

To ensure continued infiltration capability of the infiltration area, a program to maintain the facility 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 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 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 contained within this report and any other jurisdictional agency requirements.

Systems should be constructed at a depth of at least 7 feet.

Systems should be set back at least 10 feet from foundations and walls.

Any geotextile filter fabric utilized should consist of such that it 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 this firm prior to construction to confirm that the intent of the recommendations presented herein have been incorporated into the design. Testing for on-site pavement design should be performed after the site is rough graded. In addition, additional expansion index testing should be conducted in order to evaluate the impact, if any, to the site development as proposed.

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.

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.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Anthem Oil, Inc., and their designates 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. 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.

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

John P. Leuer, GE 2030
President

Robert M Markoff, CEG 2073
Engineering Geologist

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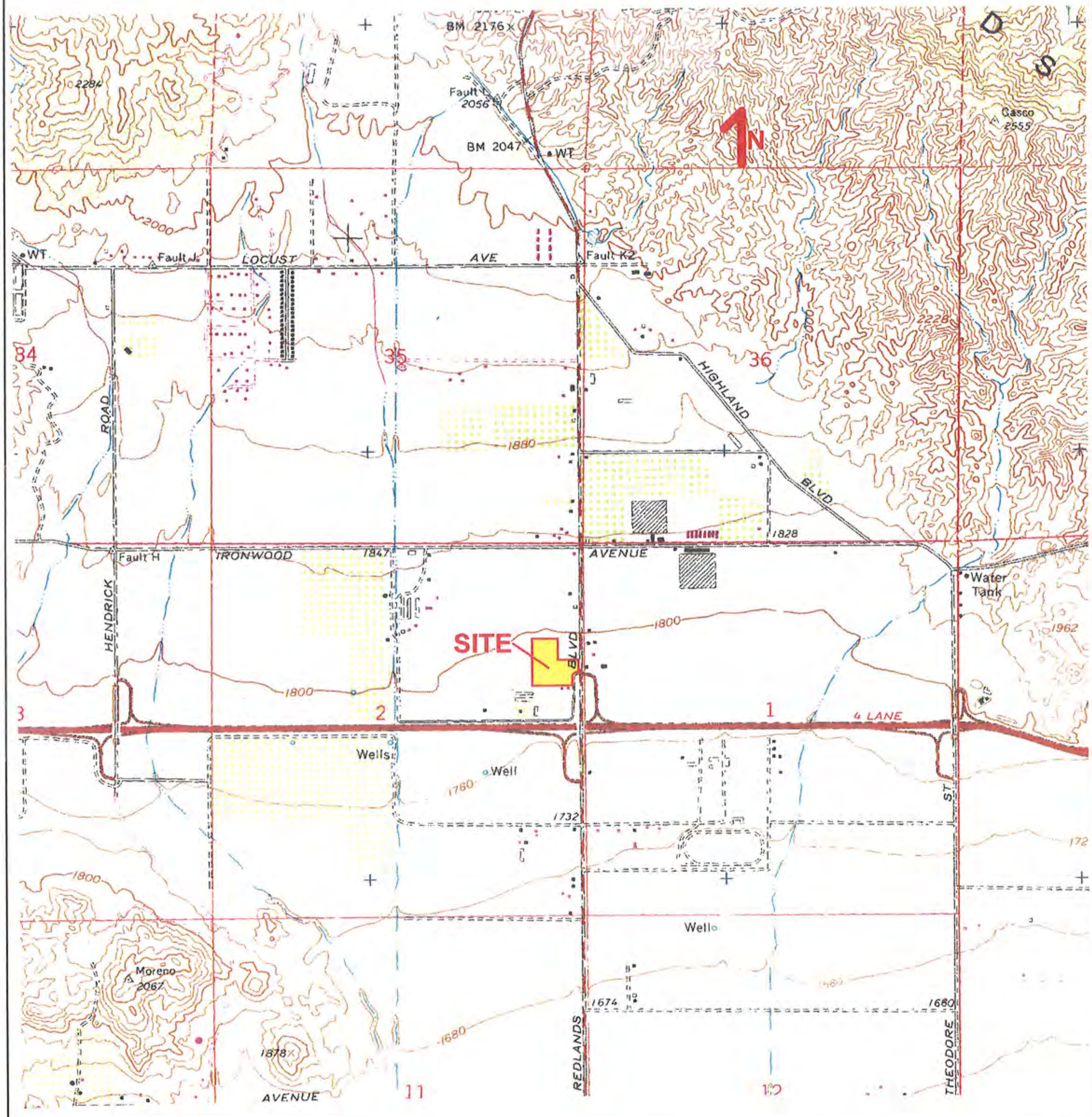
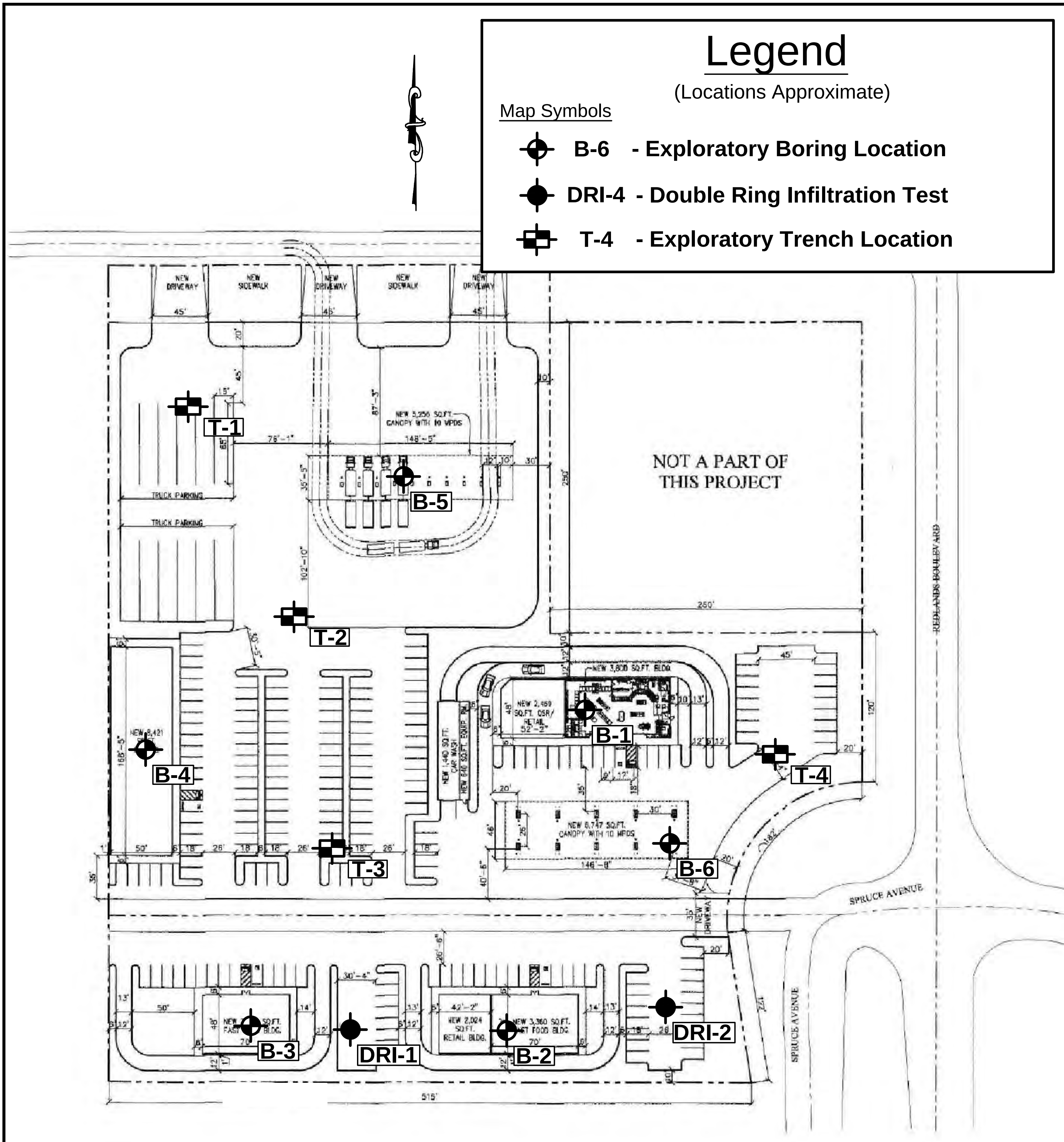
APPENDIX A

Index Map, Plate, and Regional Geologic Map,



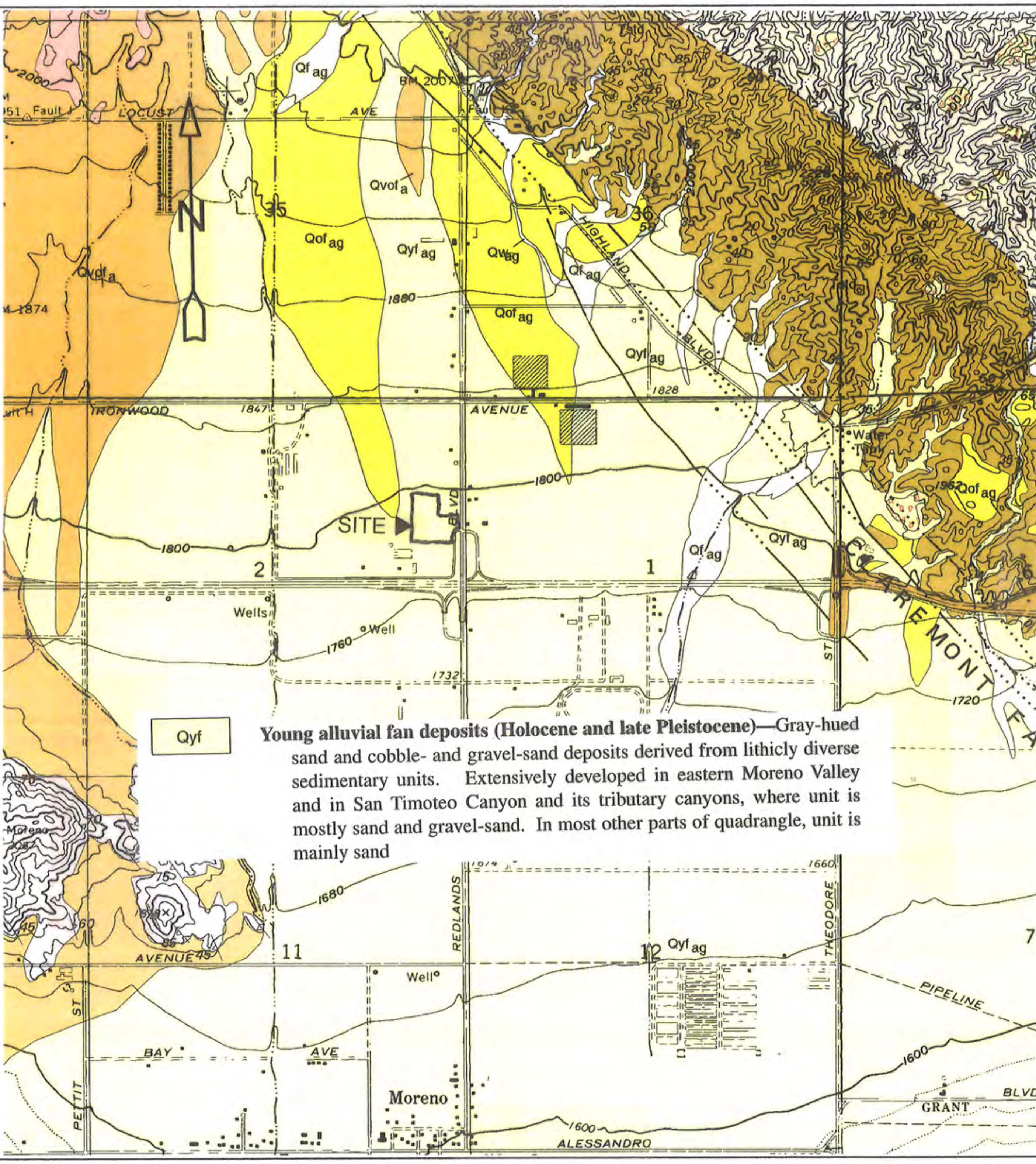
INDEX MAP

PROJECT:	**NWC SPRUCE AVE. AND REDLANDS BLVD., MORENO VALLEY, CA**	**PROJECT NO.:**	**13358.1**
CLIENT:	**ANTHEM OIL, INC.**	**ENCLOSURE:**	**A-1**
LOR Geotechnical Group, Inc.		**DATE:**	**JULY 2017**
		SCALE:	**1"=2,000'**



PLATE

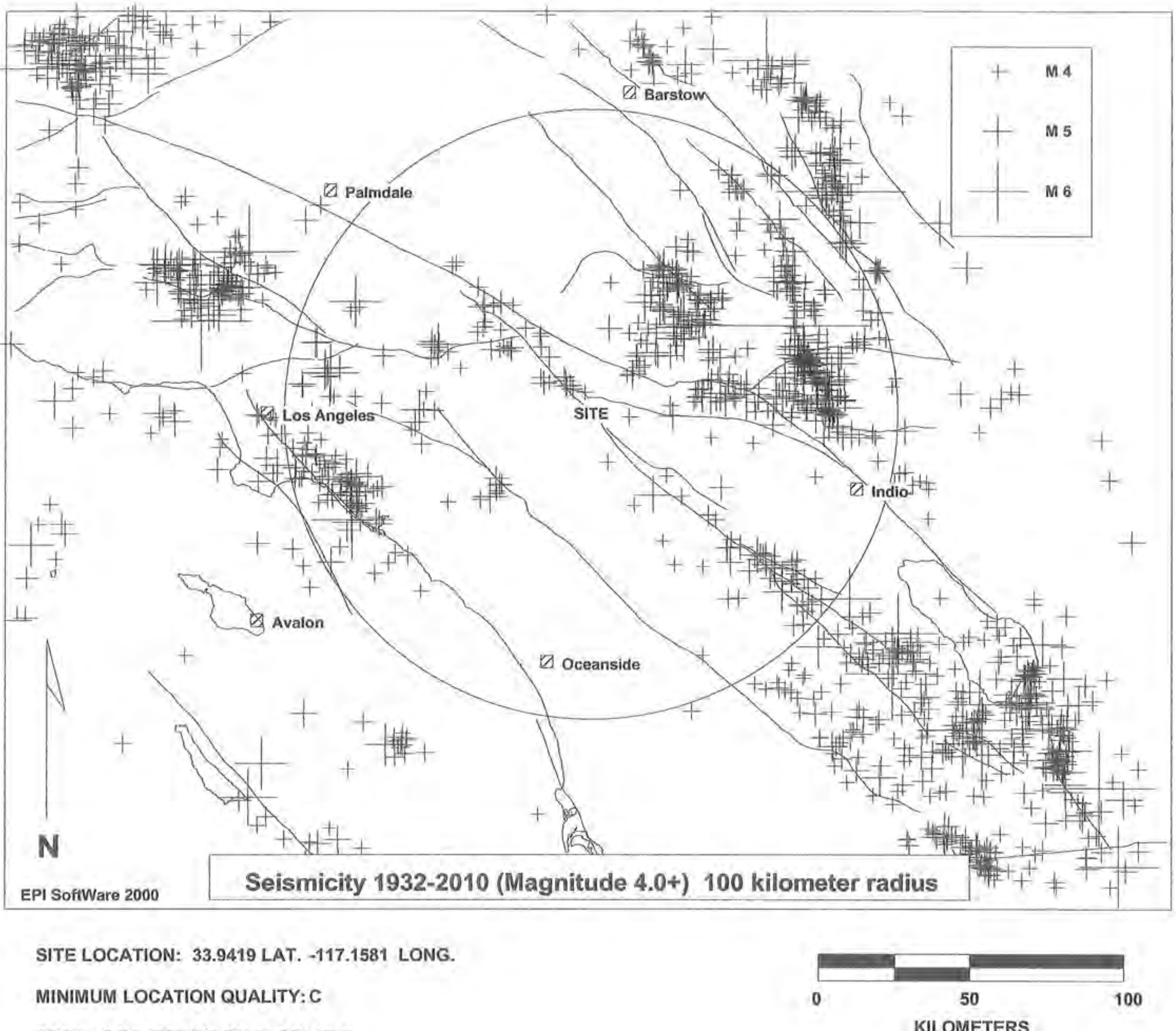
PROJECT:	NWC SPRUCE AVE. & REDLANDS BLVD., MORENO VALLEY, CALIFORNIA	**PROJECT NO:**	13358.1
CLIENT:	ANTHEM OIL	**ENCLOSURE:**	A-2
LOR Geotechnical Group, Inc.		**DATE:**	JULY 2017
		SCALE:	1" ≈ 100'



Qyf **Young alluvial fan deposits (Holocene and late Pleistocene)**—Gray-hued sand and cobble- and gravel-sand deposits derived from lithicly diverse sedimentary units. Extensively developed in eastern Moreno Valley and in San Timoteo Canyon and its tributary canyons, where unit is mostly sand and gravel-sand. In most other parts of quadrangle, unit is mainly sand

REGIONAL GEOLOGIC MAP (MORTON and MATTI, 2011)

PROJECT:	NWC SPRUCE AVE. AND REDLANDS BLVD., MORENO VALLEY, CA	PROJECT NO.:	13358.1
CLIENT:	ANTHEM OIL, INC.	ENCLOSURE:	A-3
LOR Geotechnical Group, Inc.		DATE:	JULY 2017
		SCALE:	1"=2,000'



SITE LOCATION: 33.9419 LAT. -117.1581 LONG.

MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 1517

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 605

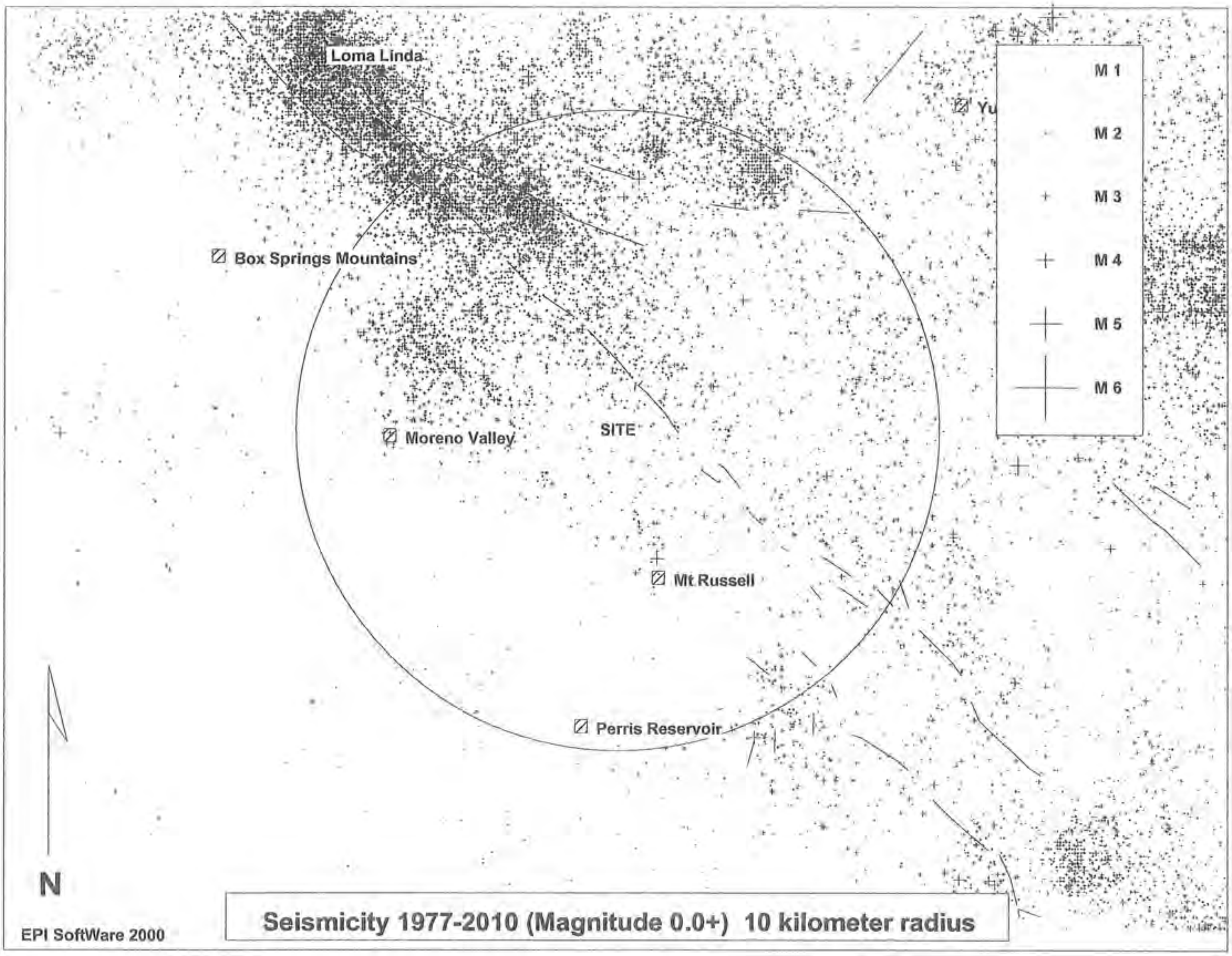
MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 544
5.0- 5.9 : 56
6.0- 6.9 : 4
7.0- 7.9 : 1
8.0- 8.9 : 0

CLOSEST EVENT: 4.1 ON SATURDAY, FEBRUARY 13, 201(LOCATED APPROX. 7 KILOMETERS NORTH OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 72 KILOMETERS NORTHEAST OF THE SITE
6.4 ON SATURDAY, MARCH 11, 193: LOCATED APPROX. 85 KILOMETERS SOUTHWEST OF THE SITE
6.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX. 42 KILOMETERS NORTHEAST OF THE SITE
6.1 ON THURSDAY, APRIL 23, 199: LOCATED APPROX. 77 KILOMETERS EAST OF THE SITE
6.0 ON SATURDAY, DECEMBER 04, 194ε LOCATED APPROX. 76 KILOMETERS EAST OF THE SITE



SITE LOCATION: 33.9419 LAT. -117.1581 LONG.

MINIMUM LOCATION QUALITY: A

TOTAL # OF EVENTS ON PLOT: 14363

TOTAL # OF EVENTS WITHIN SEARCH RADIUS: 5208

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

0.0- .9 : 592
1.0- 1.9 : 3880
2.0- 2.9 : 693
3.0- 3.9 : 42
4.0- 4.9 : 1
5.0- 5.9 : 0
6.0- 6.9 : 0
7.0- 7.9 : 0
8.0- 8.9 : 0

CLOSEST EVENT: 1.0 ON FRIDAY, MARCH 04, 1994 LOCATED APPROX. .2 KILOMETER OF THE SITE

LARGEST 5 EVENTS:

4.1 ON SATURDAY, FEBRUARY 13, 2010 LOCATED APPROX. 7 KILOMETERS NORTH OF THE SITE
3.8 ON THURSDAY, SEPTEMBER 12, 1996 LOCATED APPROX. 4 KILOMETERS SOUTH OF THE SITE
3.8 ON SATURDAY, MARCH 20, 1993 LOCATED APPROX. 9 KILOMETERS NORTHWEST OF THE SITE
3.7 ON WEDNESDAY, NOVEMBER 18, 1992 LOCATED APPROX. 8 KILOMETERS NORTHEAST OF THE SITE
3.6 ON FRIDAY, FEBRUARY 19, 2010 LOCATED APPROX. 8 KILOMETERS NORTH OF THE SITE

Enclosure A-5

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on July 13 and 21, 2017 and consisted of drilling 6 exploratory borings with a truck-mounted Mobile B61B drill rig equipped with 8-inch diameter hollow stem augers and excavating 4 exploratory test pits using a rubber tire backhoe equipped with an 18-inch bucket. The borings were extended to depths ranging from approximately 21.5 feet to 51.5 feet below the existing ground surface. The test pits were excavated to depths of approximately 14 to 14.5 feet below the existing ground surface. The approximate locations of our exploratory borings and trenches are presented on the enclosed Plate, Enclosure A-2, within Appendix A.

The drilling exploration was conducted using a truck mounted Mobile B61B drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by a staff geologist from this firm who inspected the site, created detailed logs of the borings, obtained undisturbed, as well as disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

Relatively undisturbed samples of the subsoils were obtained at a maximum interval of 5 feet. The samples were recovered by using a California split barrel sampler of 2.50-inch inside diameter and 3.00-inch outside diameter, from the ground surface to the maximum depths attained. The sampler was driven by a 140-pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches was recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), inner diameter of the hollow stem auger (3.75 in), and rod length at the test depth were considered for further computing of equivalent SPT N-values corrected for field procedures (N60) which are included in the boring logs, Enclosures B-1 through B-6.

The undisturbed soil samples were retained in brass sample rings of 2.42 inches in diameter and 1.00 inch in height, and placed in sealed plastic containers. Disturbed soil samples were obtained at selected levels within the borings and placed in sealed containers for transport to our geotechnical laboratory.

The test pit exploration was conducted using a CAT 420E backhoe with an 18-inch bucket. The soils were continuously logged by a geologist from this firm who visually inspected the site, maintained detailed logs of the trenches, obtained disturbed soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

In-place density determinations were conducted at selected levels within the trenches utilizing the Nuclear Gauge Method, in accordance with the standard ASTM D 2922. Disturbed soil samples were obtained at soil changes and other selected levels within the trenches. The samples were placed in sealed containers for transport to our geotechnical laboratory.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings and trenches are presented on the enclosed Boring Logs, Enclosures B-1 through B-10. A Boring/Trench Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented on Enclosure B-ii.

CONSISTENCY OF SOIL

SANDS

SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

COHESIVE SOILS

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

SAMPLE KEY

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

BORING LOG LEGEND

PROJECT: NWC SPRUCE AVE. AND REDLANDS BLVD., MORENO VALLEY, CA	PROJECT NO.: 13358.1
CLIENT: ANTHEM OIL, INC.	ENCLOSURE: B-i
LOR Geotechnical Group, Inc.	DATE: JULY 2017

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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*
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	*GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)*		GM	*SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES*
				GC	*CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES*
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*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*
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	*SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)*		SM	*SILTY SANDS, SAND - SILT MIXTURES*
				SC	*CLAYEY SANDS, SAND - CLAY 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*
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*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 No. 10 No. 40 200

(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	**NWC SPRUCE AVE. AND REDLANDS BLVD., MORENO VALLEY, CA**	**PROJECT NO.:**	**13358.1**
CLIENT:	**ANTHEM OIL, INC.**	**ENCLOSURE:**	**B-ii**
LOR Geotechnical Group, Inc.		**DATE:**	**JULY 2017**

LOG OF BORING B-1

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 40% silty fines, tan, dry, loose.
	7		5.9		108.9			ML	@ 2 feet, ALLUVIUM:SANDY SILT, trace gravel to 2", approximately 5% coarse grained sand, 10% medium grained sand, 25% fine grained sand, 60% silty fines, light yellow brown, dry, some thin calcite stringers.
5	9		5.6		113.0				@ 5 feet, becomes slightly coarser grained, some roothairs, thin calcite stringers remain.
	9		5.0		110.5			SM	@ 7 feet, SILTY SAND, approximately 10% coarse grained sand, 20% medium grained sand, 45% fine grained sand, 25% silty fines, light yellow brown, dry to damp.
10	12		6.4		109.0				
	19		3.3		104.0			SP SM	@ 12 feet, POORLY GRADED SAND with SILT, approximatley 90% fine grained sand, 10% silty fines, white, damp.
15	17		9.1		103.6				
20	16		7.0		110.0			SM	@ 20 feet. SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 25% silty fines, light red brown, damp.
25	25		9.5		119.6			ML SM	@ 25 feet, SILTY SAND/SANDY SILT, approximately 50% fine grained sand, 50% silty fines, light yellow brown, damp.
30	35		9.4		118.8			SC	@ 30 feet, CLAYEY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 30% clayey fines of low plasticity, red brown, damp, abundant secondary calcite.
35	33		8.2		118.7			ML	@ 35 feet, SANDY SILT, approximately 15% medium grained sand, 20% fine grained sand, 65% silty fines, light red brown, damp, some pinhole porosity, some secondary calcite as nodules.
40	48		6.4		117.8				@ 40 feet, SANDY SILT, approximately 20% fine grained sand, 80% silty fines, brown, damp, some thin calcite stringers.
45	58		5.5		121.9				@ 45 feet, becomes slighlty sandier.
50	52		10.5		122.2			CL	@ 50 feet, LEAN CLAY with SAND, approximately 15% medium grained sand, 25% fine grained sand, 60% clayey fines of low plasticity, red brown, damp, abundant calcite stringers.
55									END OF BORING
60									

END OF BORING

Fill to 2'
No groundwater
No bedrock

PROJECT: Proposed Commercial Development	PROJECT NUMBER: 13358.1
CLIENT: Anthem Oil, Inc.	ELEVATION:
	DATE DRILLED: July 13, 2017
LOR GEOTECHNICAL GROUP INC.	EQUIPMENT: Mobile B61
	HOLE DIA.: 8" ENCLOSURE: B-1

LOG OF BORING B-2

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL:SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 50% fine grained sand, 25% silty fines, tan, dry, loose.
	4		8.5		113.6				
5	3		7.5		99.8			ML SM	@ 5 feet, ALLUVIUM:SILTY SAND/SANDY SILT, trace gravel to 1/2", approximately 5% coarse grained sand, 10% medium grained sand, 35% fine grained sand, 50% silty fines, yellow brown, damp.
	9		6.1		120.3			SM	@ 7 feet, SILTY SAND, approximately 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 40% silty fines, red brown, damp.
10	15		2.8		115.3			SP SM	@ 10 feet, POORLY GRADED SAND with SILT, approximately 15% coarse grained sand, 25% medium grained sand, 50% fine grained sand, 10% silty fines, yellow brown, dry.
	47		1.7		122.4			SW	@ 12 feet, WELL GRADED SAND, approximately 25% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 5% silty fines, yellow brown, dry.
15	53		1.5		123.0				
20	36		8.7		123.5			ML	@ 20 feet, SANDY SILT, approximately 10% medium grained sand, 25% fine grained sand, 65% silty fines, red brown, damp, some thin calcite stringers.
									END OF BORING Fill to 2' No groundwater No bedrock
25									
30									

PROJECT:	**Proposed Commercial Development**	PROJECT NUMBER:	**13358.1**
CLIENT:	**Anthem Oil, Inc.**	ELEVATION:	
		DATE DRILLED:	**July 13, 2017**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Mobile B61**
		HOLE DIA.: **8"**	ENCLOSURE: **B-2**

LOG OF BORING B-3

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% silty fines, tan, dry, loose.
	5		9.7		101.3			ML	@ 2 feet, ALLUVIUM:SANDY SILT, approximately 5% medium grained sand, 10% fine grained sand, 85% silty fines, yellow brown, damp, some pinhole porosity, some thin calcite stringers.
5	8		8.9		101.9				
	7	2	9.1		100.9				
10	9	2	9.0		113.0				
	13		6.3		108.8			SM	@ 12 feet, SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 25% silty fines, red brown, damp.
15	26		6.1		122.2				
20	17		4.3		107.6			SP	@ 20 feet, POORLY GRADED SAND, trace gravel to 1/2" approximately 20% coarse grained sand, 30% medium grained sand, 45% fine grained sand, 5% silty fines, yellow brown, dry.
25	31		6.2		109.2			SM	@ 25 feet, SILTY SAND, approximately 85% fine grained sand, 15% silty fines, yellow brown, dry.
									END OF BORING Fill to 2' No groundwater No bedrock
30									

PROJECT: **Proposed Commercial Development**	PROJECT NUMBER: **13358.1**
CLIENT: **Anthem Oil, Inc.**	ELEVATION:
	DATE DRILLED: **July 13, 2017**
LOR GEOTECHNICAL GROUP INC.	EQUIPMENT: **Mobile B61**
	HOLE DIA.: **8"** ENCLOSURE: **B-3**

LOG OF BORING B-4

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 65% fine grained sand, 30% silty fines, tan, dry, loose.
	8		5.4		108.3			ML	@ 2 feet, ALLUVIUM:SANDY SILT, aproximately 10% medium grained sand, 35% fine grained sand, 55% silty fines, yellow brown, dry, trace pinhole porosity, abundant thin calcite stringers.
5	6	2	6.9		97.6				@ 5 feet, slight increase in porosity diameter.
	10		9.5		116.3				@ 7 feet, slight decrease in porosity diameter to pinhole.
10	12		9.2		107.8				@ 10 feet, trace pinhole porosity, trace thin calcite stringers, slightly finer grained, damp.
15	38		5.9		123.4				@ 15 feet, porosity no longer visible, red brown, dry.
20	42		0.9		110.4			SP	@ 20 feet, POORLY GRADED SAND, approximately 5% gravel to 1/2", 10% coarse grained sand, 25% medium grained sand, 55% fine grained sand, 5% silty fines, white, dry.
									END OF BORING Fill to 2' No groundwater No bedrock
25									
30									

PROJECT: Proposed Commercial Development	PROJECT NUMBER: 13358.1		
CLIENT: Anthem Oil, Inc.	ELEVATION:		
	DATE DRILLED: July 13, 2017		
LOR GEOTECHNICAL GROUP INC.	EQUIPMENT: Mobile B61		
	HOLE DIA.: 8"	ENCLOSURE: B-4	

LOG OF BORING B-5

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL:SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 55% fine grained sand, 25% silty fines, tan, dry, loose.
	4		5.0		103.4			ML SM	@ 2 feet, ALLUVIUM:SILTY SAND/SANDY SILT, approximately 5% coarse grained sand, 15% medium grained sand, 30% fine grained sand, 50% silty fines, yellow brown, dry, abundant pinhole and slightly larger porosity.
5	5		5.8		107.7			SM	@ 5 feet, SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 20% silty fines, yellow brown, dry.
	13		2.9		113.2				
10	11		7.1		105.4			ML	@ 10 feet, SANDY SILT, approximately 15% fine grained sand, 85% silty fines, tan, dry.
	13		3.0		102.6				
								SP	@ 13 feet, POORLY GRADED SAND, approximately 5% coarse grained sand, 25% medium grained sand, 65% fine grained sand, 5% silty fines, white, dry.
15	17		8.4		121.5			SM	@ 15 feet, SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% silty fines with trace clay, red brown, damp.
20	29		4.3		115.7			SP SM	@ 20 feet, POORLY GRADED SAND with SILT, approximately 90% fine grained sand, 10% silty fines, yellow brown, dry to damp.
									END OF BORING Fill to 2' No groundwater No bedrock
25									
30									

PROJECT:	Proposed Commercial Development	PROJECT NUMBER:	13358.1
CLIENT:	Anthem Oil, Inc.	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	July 13, 2017
		EQUIPMENT:	Mobile B61
		HOLE DIA.: 8"	ENCLOSURE: B-5

LOG OF BORING B-6

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL:**SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 35% silty fines, tan, dry, loose.**
	8		4.1		98.9	▮		ML	@ 2 feet, ALLUVIUM:**SANDY SILT, approximately 15% medium grained sand, 20% fine grained sand, 65% silty fines, yellow brown, dry, some pinhole and slighlty larger porosity, trace roothairs.**
5	9		5.9		116.1	▮			**@ 5 feet, contains thin calcite stringers, no roothairs.**
	7	2	8.1		102.8	▮		SM	**@ 7 feet, SILTY SAND, trace gravel to 1/2", approximately 5% coarse grained sand, 5% medium grained, 70% fine grained sand, 20% silty fines, yellow brown, damp.**
10	9		9.3		100.8	▮			**@ 10 feet, becomes slightly coarser grained.**
	8	2	10.4		97.2	▮			**@ 12 feet, becomes red brown.**
15	15		7.9		118.6	▮			**@ 15 feet, SILTY SAND, approximately 5% coarse grained sand, 20% medium grained, 45% fine grained sand, 30% silty fines with trace clay, red brown, moist.**
20	27		4.8		116.4	▮			
									END OF BORING **Fill to 2'** **No groundwater** **No bedrock**
25									
30									

PROJECT: **Proposed Commercial Development**	PROJECT NUMBER: **13358.1**
CLIENT: **Anthem Oil, Inc.**	ELEVATION:
	DATE DRILLED: **July 13, 2017**
LOR GEOTECHNICAL GROUP INC.	EQUIPMENT: **Mobile B61**
	HOLE DIA.: **8"** ENCLOSURE: **B-6**

LOG OF TRENCH T-1

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.8				SM	@ 0 feet, FILL/TOPSOIL SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 40% fine grained sand, 35% silty fines, tan, dry, loose.
	3, 4, 7		71	4.5	92.5			ML	@ 2 feet, ALLUVIUM:SANDY SILT, approximately 5% coarse grained sand, 15% medium grained sand, 15% fine grained sand, 65% silty fines, yellow brown, dry, abundant pinhole porosity, roothairs, massive. @ 3 feet, some porosity slightly larger than pinhole.
			70	7.8	90.9				@ 4 feet, slight increase in moisture content, some secondary calcite.
5									@ 5 feet, roothairs, porosity no longer visible.
								SM	@ 9 feet, SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% silty fines, red brown, dry.
10									
15									END OF TRENCH Fill to 2' No caving No groundwater No bedrock

PROJECT:	**Proposed Commercial Development**	PROJECT NUMBER:	**13358.1**
CLIENT:	**Anthem Oil, Inc.**	ELEVATION:	
		DATE EXCAVATED:	**July 21, 2017**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**CAT 420E**
		BUCKET W.: **36"**	ENCLOSURE: **B-7**

LOG OF TRENCH T-2

DEPTH IN FEET	TEST DATA: LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	9, 10, 11			0.5				SM	@ 0 feet, <u>FILL/TOPSOIL</u>:**SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 50% fine grained sand, 35% silty fines, tan, dry, loose.**
								ML	**@ 2 feet, <u>ALLUVIUM</u>:SANDY SILT, approximately 5% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 55% silty fines, yellow brown, dry, pinhole and slighlty porosity, roothairs, massive.**
			72	3.0	93.8				**@ 4 feet, roothairs no longer present, pinhole porosity remains, slight increase in moisture.**
5			73	7.0	95.1				
10									
									@ 12 feet, becomes red brown, damp, abundant pinhole porosity and thin calcite stringers.
									END OF TRENCH **Fill to 2'** **No caving** **No groundwater** **No bedrock**
15									

PROJECT: **Proposed Commercial Development**	PROJECT NUMBER: **13358.1**
CLIENT: **Anthem Oil, Inc.**	ELEVATION:
LOR GEOTECHNICAL GROUP INC.	DATE EXCAVATED: **July 21, 2017**
	EQUIPMENT: **CAT 420E**
	BUCKET W.: **36"** ENCLOSURE: **B-8**

LOG OF TRENCH T-3

DEPTH IN FEET	TEST DATA: LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0				0.1				SM	@ 0 feet, FILL/TOPSOIL SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 45% fine grained sand, 35% silty fines, tan, dry, loose.
			70	3.5	91.0			ML	@ 2 feet, ALLUVIUM:SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 25% fine grained sand, 60% silty fines, yellow brown, dry, abundant pinhole porosity, some roothairs, massive.
	7		74	7.0	95.5				@ 4 feet, slight decrease in porosity, slight increase in moisture.
5									
10									
									@ 12 feet, becomes red brown, damp, some pinhole porosity and secondary calcite.
15									**END OF TRENCH** Fill to 2' No caving No groundwater No bedrock

PROJECT: **Proposed Commercial Development**	PROJECT NUMBER: **13358.1**
CLIENT: **Anthem Oil, Inc.**	ELEVATION:
	DATE EXCAVATED: **July 21, 2017**
LOR GEOTECHNICAL GROUP INC.	EQUIPMENT: **CAT 420E**
	BUCKET W.: **36"** ENCLOSURE: **B-9**

LOG OF TRENCH T-4

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	8, 9, 10, 11			0.1				SM	@ 0 feet, FILL/TOPSOIL:SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% silty fines, tan, dry, loose.
								ML	@ 2 feet, ALLUVIUM:SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% silty fines, yellow brown, dry, some porosity and roothairs, massive.
			73	4.3	94.3				
5			75	4.2	96.8				@ 5 feet, slight increase in moisture.
								SM	@ 8 feet, SILTY SAND, approximately 25% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, yellow brown, dry, caving.
10									
									@ 12 feet, trace fine gravel.
15									**END OF TRENCH** Fill to 2' Caving from 8 to 14.5' No groundwater No bedrock

PROJECT:	**Proposed Commercial Development**	PROJECT NUMBER:	**13358.1**
CLIENT:	**Anthem Oil, Inc.**	ELEVATION:	
		DATE EXCAVATED:	**July 21, 2017**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**CAT 420E**
		BUCKET W.: **36"**	ENCLOSURE: **B-10**

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. Laboratory testing included, in-place moisture content and density, laboratory compaction, direct shear, expansion index, consolidation, 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 2216, and ASTM D 2937 and the results are shown on our boring logs, Enclosures B-1 through B-4, 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	2-3	(ML) Sandy Silt	130.0	10.0
T-3	4-5	(ML) Sandy Silt	129.5	9.0

Direct Shear Tests

Shear tests are performed with a direct shear machines in general accordance with ASTM D 3080 at a constant rate-of-strain (usually 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 a remolded condition (90 percent per ASTM D 1557) and soaked, according to conditions expected in the field.

The results of the sheared 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	2-3	(ML) Sandy Silt	23	500

Expansion Index Test

Remolded samples are tested to determine their expansion potential in accordance with the Expansion Index (EI) test. The test is performed in accordance with the Uniform Building Code Standard 18-2. The test results are presented on the following table:

EXPANSION INDEX TEST				
Trench Number	**Sample Depth (feet)**	**Material Description (U.S.C.S)**	**Expansion Index (EI)**	**Expansion Potential**
T-1	2-3	(ML) Sandy Silt	15	Very Low
Expansion Index:	0-20 Very Low	21-50 Low	51-90 Medium	91-130 High

Consolidation Tests

The apparatus used for the consolidation tests (odometer) is designed to test a one-inch high portion of the undisturbed soil sample as contained in a sample ring. Porous stones and filler paper are placed in contact with the top and bottom of the specimen to permit the addition or release of water. Loads are applied to the test specimen in specified increments, and the resulting axial deformations are recorded. The results are plotted as log of axial pressure versus consolidation or compression, expressed as strain or sample height.

Samples are tested at field and greater-than field moisture contents. The results are shown on Enclosures C-1 through C-5.

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with 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 analysis are presented graphically on Enclosure C-6.

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 analysis on Enclosure C-6.

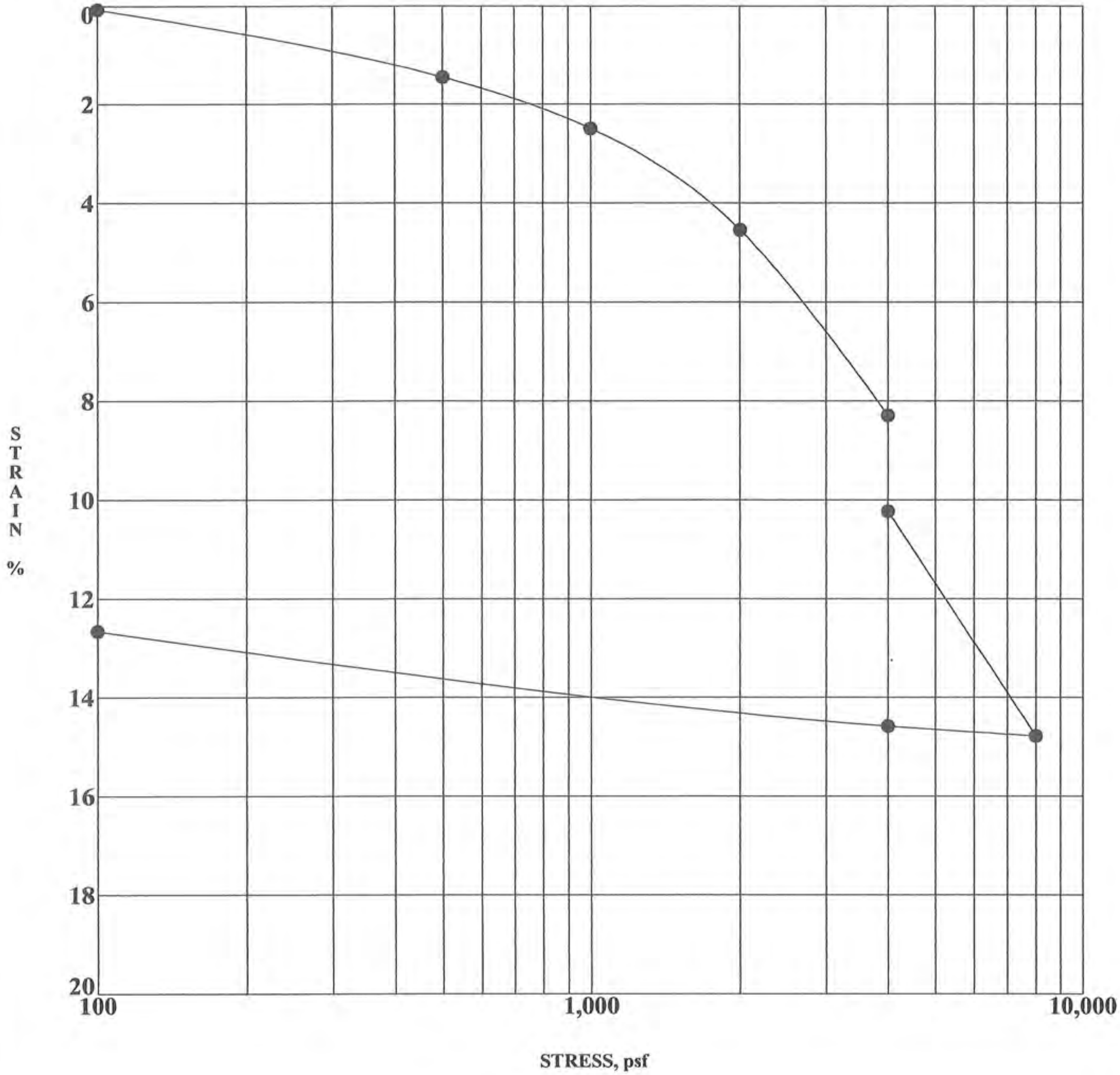
R-Value Test

Soil samples were obtained at probable pavement subgrade level and sieve analysis and sand equivalent tests were conducted. 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-6.

Soluble Sulfate Content Tests

The soluble sulfate content of selected subgrade soils was 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			
Trench Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S)**	**Sulfate Content (percent by weight)**
T-1	2-3	(ML) Sandy Silt	< 0.005
T-3	4-5	(ML) Sandy Silt	< 0.005



	Specimen I.D.		Classification	DD	MC%
●	**B-3**	**@ 7 ft**	**(ML) Sandy Silt**	*97*	*9*

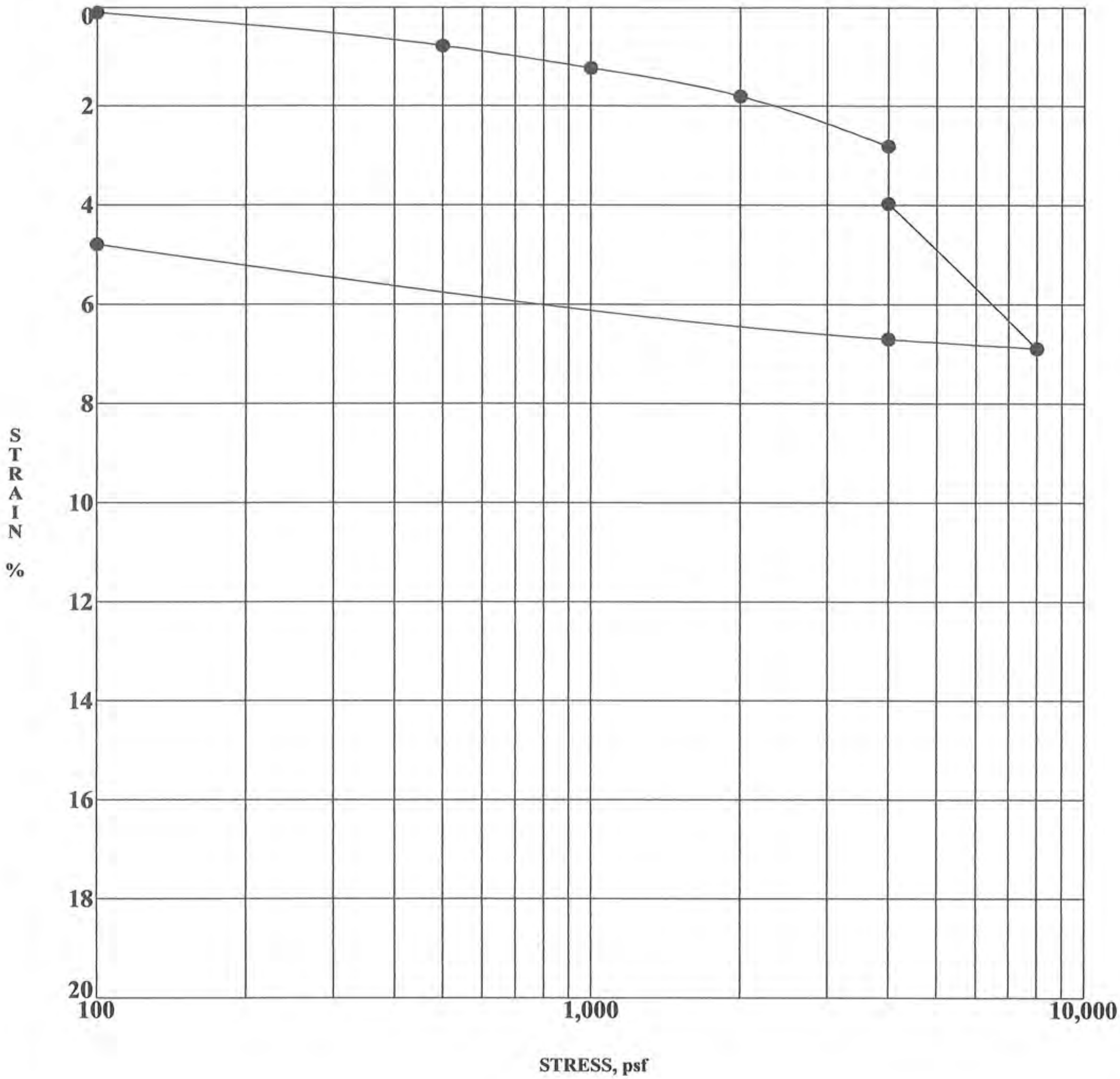
PROJECT *Proposed Commercial Development* **PROJECT NO.** *13358.1*

DATE *8/8/17*

CONSOLIDATION TEST

LOR Geotechnical Group, Inc.

ENCLOSURE C-1

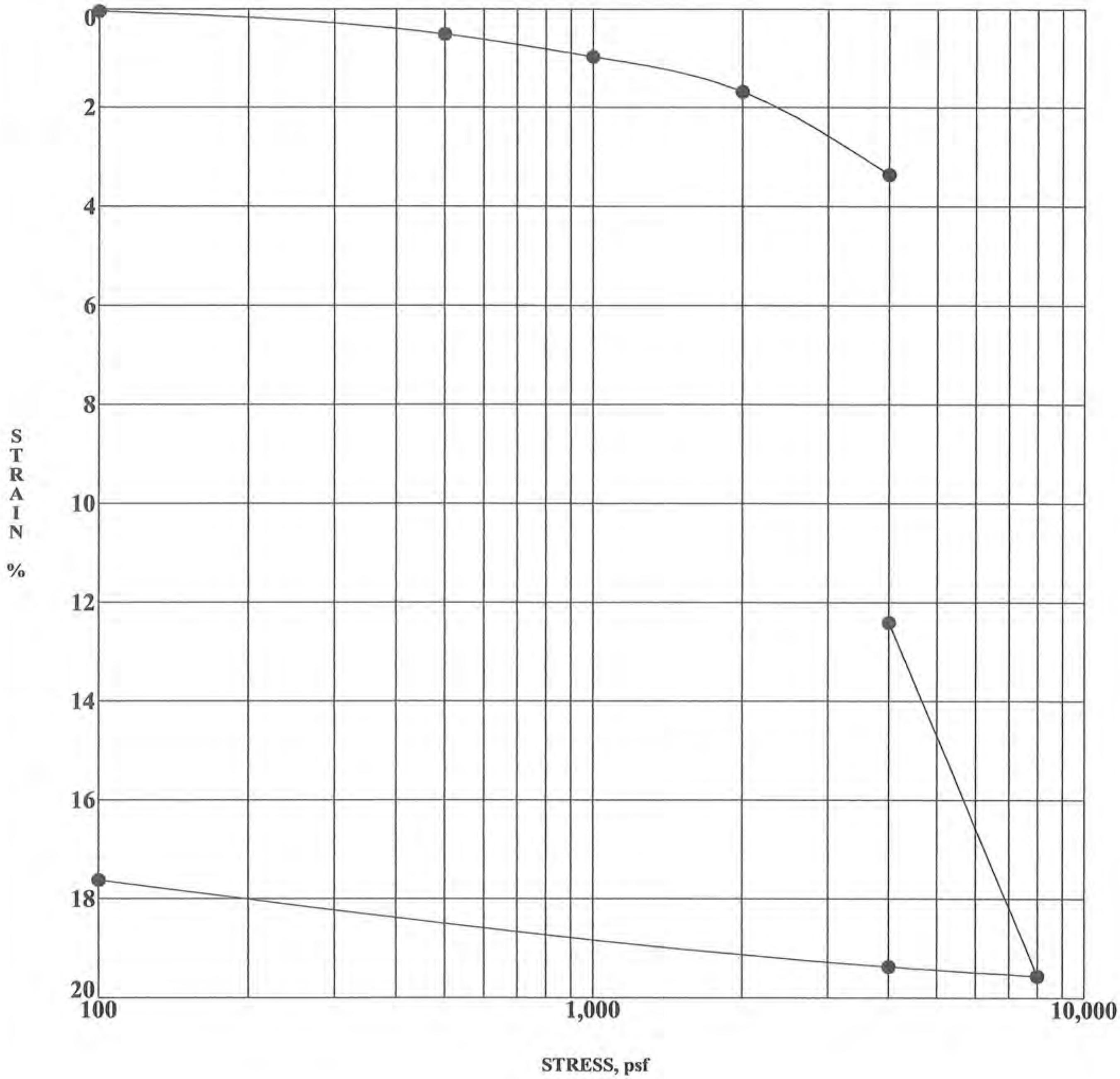


	Specimen I.D.		Classification	DD	MC%
●	**B-3**	**@ 10 ft**	**(ML) Sandy Silt**	*104*	*9*

PROJECT *Proposed Commercial Development* **PROJECT NO.** *13358.1*
DATE *8/8/17*

CONSOLIDATION TEST
LOR Geotechnical Group, Inc.

ENCLOSURE C-2



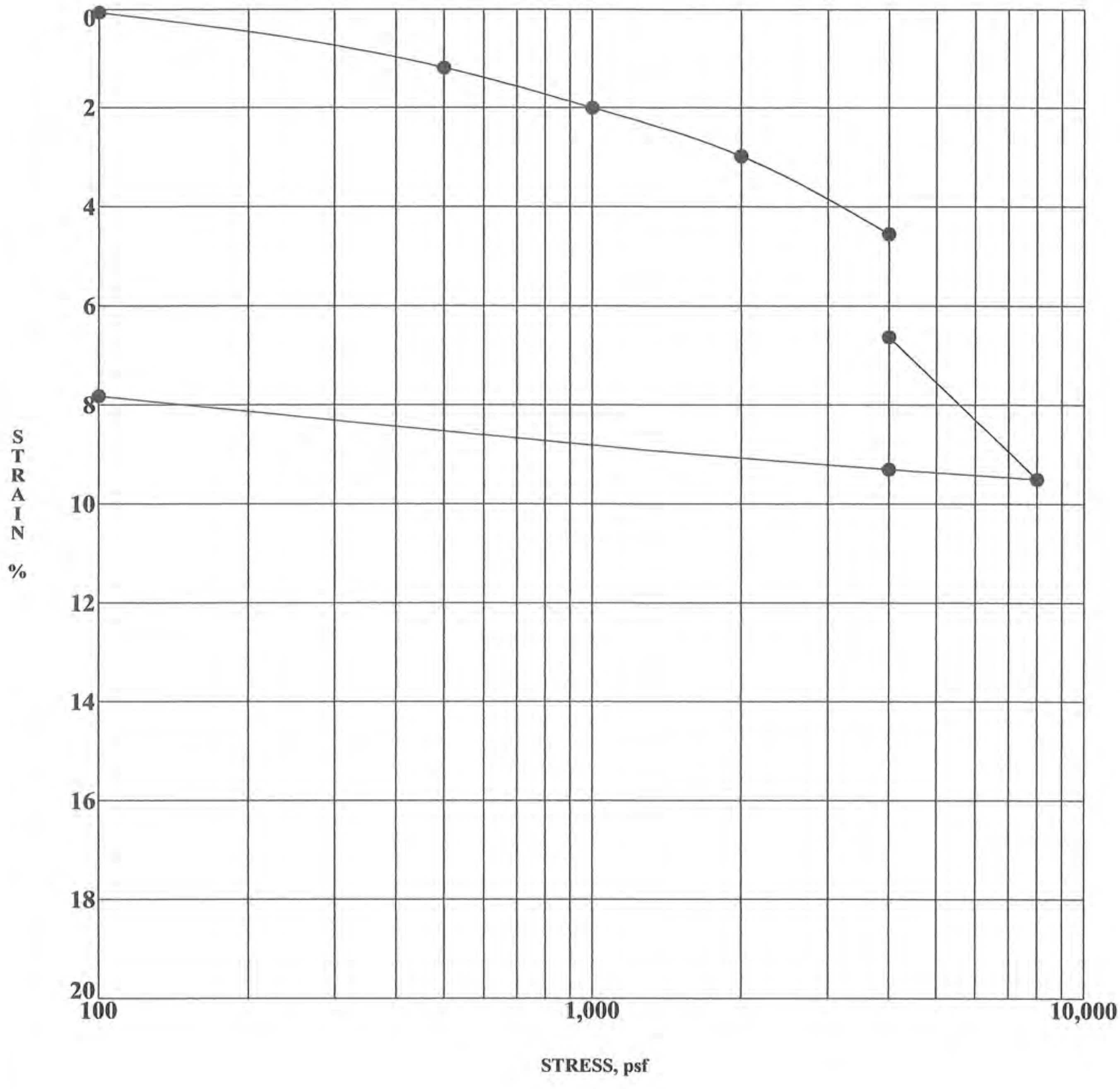
	Specimen I.D.		Classification	DD	MC%
●	**B-4**	**@ 5 ft**	**(SM) Silty Sand**	*92*	*7*

PROJECT *Proposed Commercial Development*

PROJECT NO. *13358.1*
DATE *8/8/17*

CONSOLIDATION TEST
LOR Geotechnical Group, Inc.

ENCLOSURE C-3



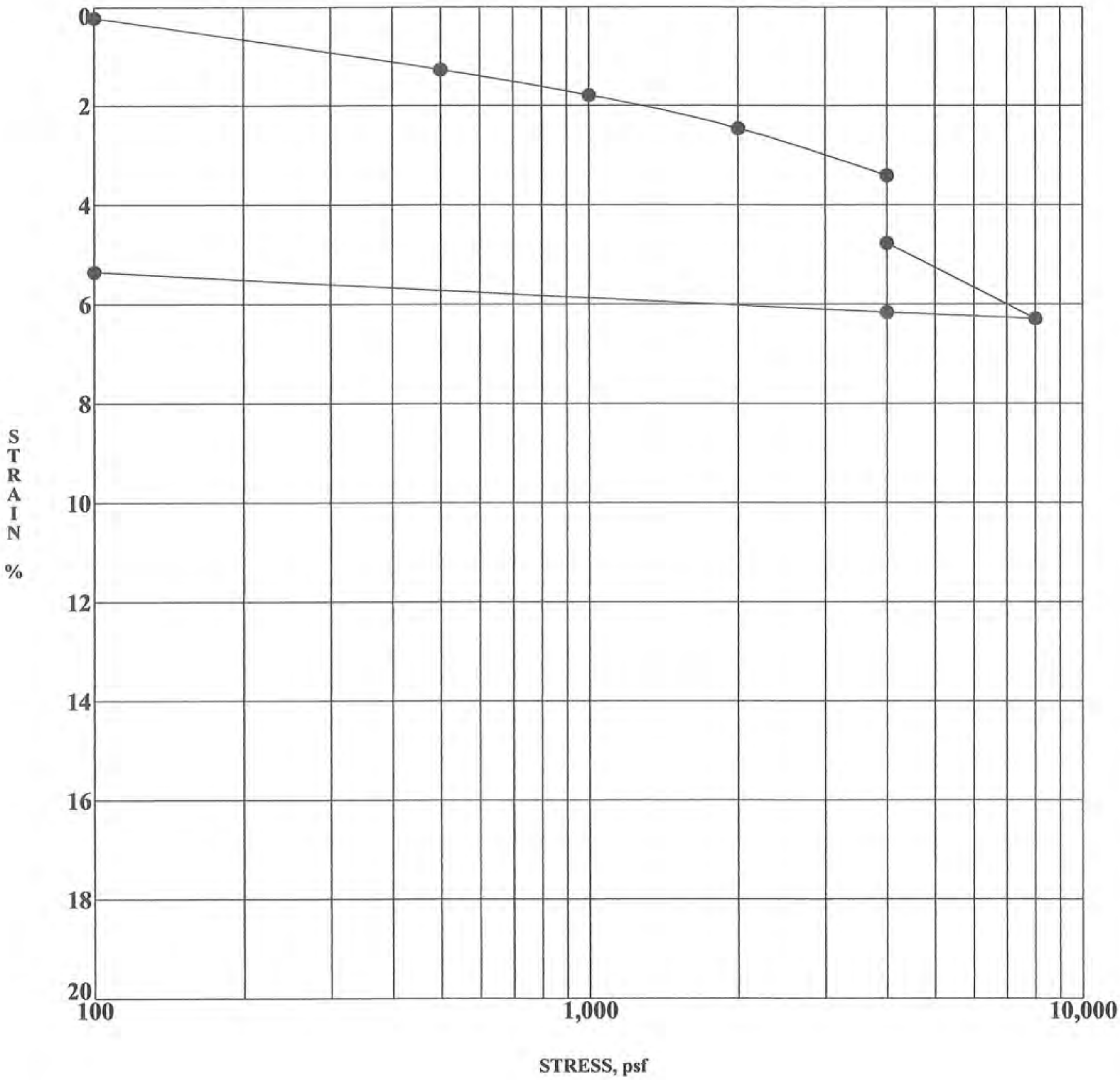
	Specimen I.D.		Classification	DD	MC%
●	**B-6**	**@ 7 ft**	**(SM) Silty Sand**	*102*	*8*

PROJECT *Proposed Commercial Development*

PROJECT NO. *13358.1*
DATE *8/8/17*

CONSOLIDATION TEST
LOR Geotechnical Group, Inc.

ENCLOSURE C-4



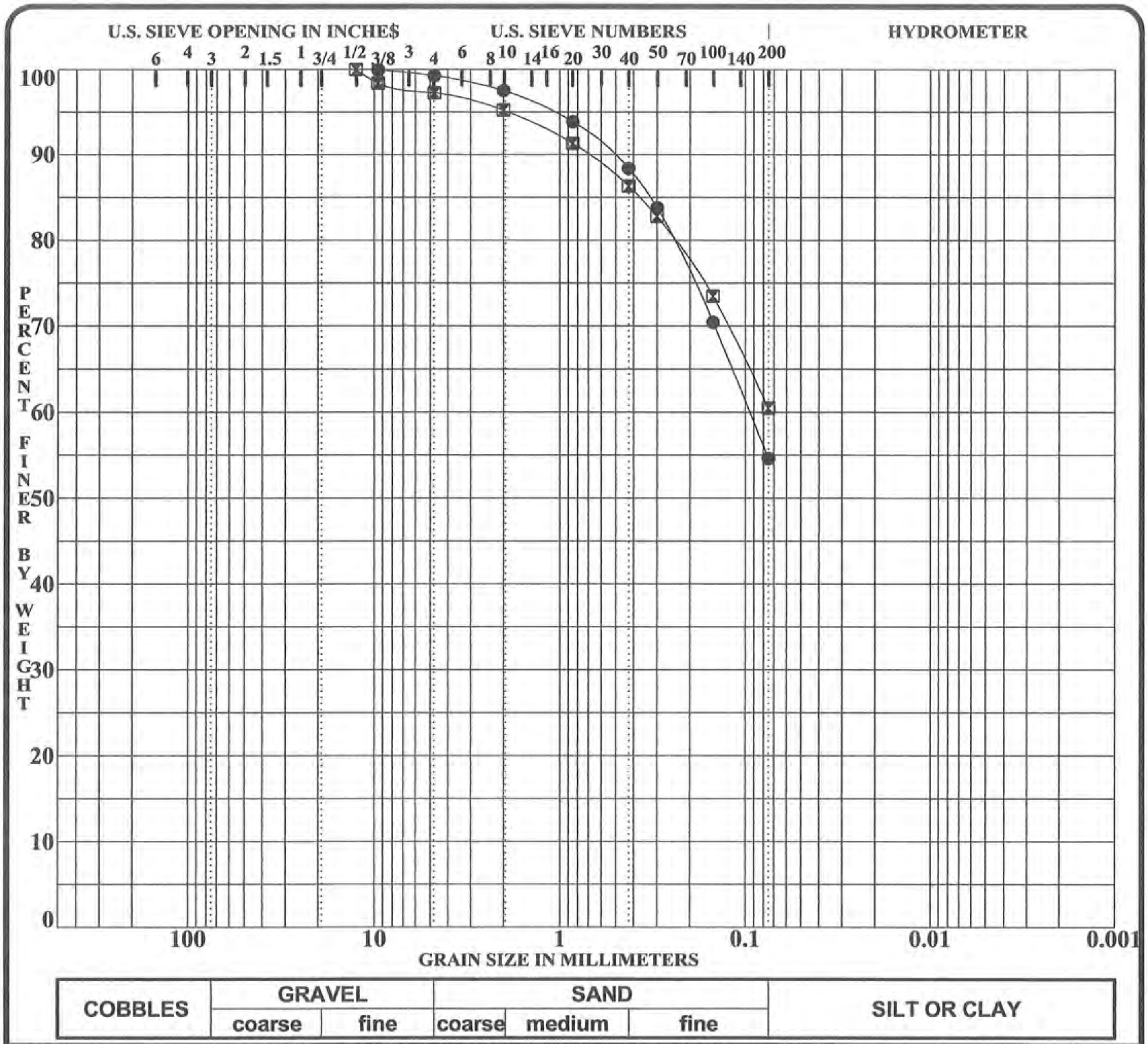
	Specimen I.D.		Classification	DD	MC%
●	**B-6**	**@ 12 ft**	**(SM) Silty Sand**	*94*	*10*

PROJECT *Proposed Commercial Development*

PROJECT NO. *13358.1*
DATE *8/8/17*

CONSOLIDATION TEST
LOR Geotechnical Group, Inc.

ENCLOSURE C-5



Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
● T-2	@ 0-3 ft	(ML) Sandy Silt	9	--				
⊠ T-4	@ 0-3 ft	(ML) Sandy Silt	7	10				

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
● T-2	@ 0-3 ft	9.50	0.09			0.7	44.7	54.6
⊠ T-4	@ 0-3 ft	12.50				2.8	36.8	60.5

PROJECT *Proposed Commercial Development*

PROJECT NO. *13358.1*
DATE *8/8/17*

GRADATION CURVES
LOR Geotechnical Group, Inc.

ENCLOSURE C-6

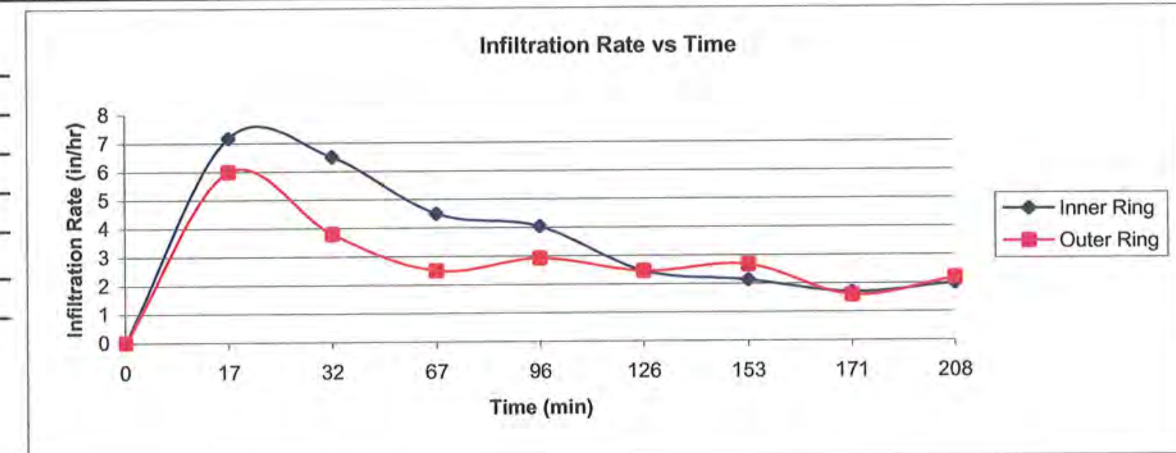
APPENDIX D

Infiltration Test Results

DOUBLE RING INFILTROMETER TEST DATA

Project: Anthem Oil
Project No.: 13358.1
Soil Classification: (SM) Silty Sand
Depth of Test Hole: 6 ft
Liquid Used: Tap Water
Area of Rings: Inner = 0.785 ft^2 , Annular 2.36 ft^2
Tested By: A.L.
Liquid Level Maintained Using: Vacuum Seal
Depth to Water Table: ~ 200 ft

Test Date: July 21, 2017
Test Hole No.: DRI-1
Test Hole Diameter: 12 in. inner, 24 in. annular
Date Excavated: July 21, 2017
pH: 7.8
Depth of Water in Rings: 1.5 in
Ring Penetration: 4 in

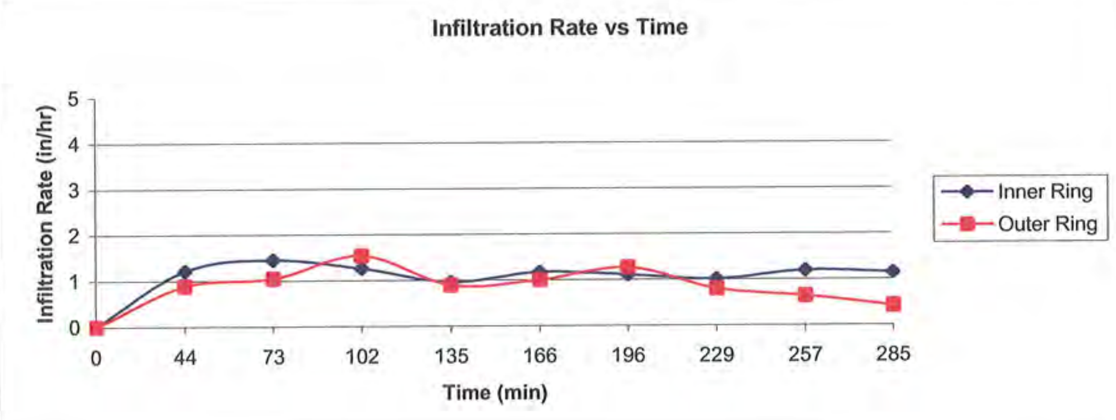


TEST PERIOD

TRIAL NO.		INNER TIME	INNER TIME INTERVAL (minutes)	INNER TOTAL ELASPED TIME (minutes)	ANNULAR SPACE TIME	ANNULAR SPACE TIME INTERVAL (minutes)	ANNULAR SPACE TOTAL ELASPED TIME (minutes)	WATER USED (lbs.) inner	WATER USED (lbs.) annular space	WATER USED (gal) inner	WATER USED (gal) annular space	INFILTRATION RATE (gal/sf.day) inner	INFILTRATION RATE (gal/sf.day) annular space	INFILTRATION RATE (in/hr) inner	INFILTRATION RATE (in/hr) annular space	LIQUID TEMP (°F)	REMARKS
1	S	11:08	17	17	11:08	17	17	8.27	20.78	0.993	2.495	107.1	89.5	7.2	6.0	82	
	E	11:25			11:25											82	
2	S	11:25	15	32	11:25	15	32	6.62	11.61	0.795	1.394	97.2	56.7	6.5	3.8	82	
	E	11:40			11:40											82	
3	S	11:40	35	67	11:40	35	67	10.69	17.82	1.283	2.139	67.3	37.3	4.5	2.5	83	
	E	12:15			12:15											83	
4	S	12:15	29	96	12:15	29	96	7.96	17.28	0.956	2.074	60.4	43.6	4.0	2.9	84	
	E	12:44			12:44											84	
5	S	12:55	30	126	12:55	30	126	5.01	15.09	0.601	1.812	36.8	36.8	2.5	2.5	85	refilled both
	E	13:25			13:25											85	
6	S	13:25	27	153	13:25	27	153	3.92	14.74	0.471	1.770	32.0	40.0	2.1	2.7	85	
	E	13:52			13:52											85	
7	S	13:52	18	171	13:52	18	171	2.08	5.85	0.250	0.702	25.4	23.8	1.7	1.6	86	
	E	14:10			14:10											86	
8	S	14:10	37	208	14:10	37	208	5.03	16.60	0.604	1.993	29.9	32.9	2.0	2.2	86	
	E	14:47			14:47											86	

DOUBLE RING INFILTROMETER TEST DATA

Project:	Anthem Oil	**Test Date:**	July 21, 2017
Project No.:	13358.1	**Test Hole No.:**	DRI-2
Soil Classification:	(SM) Silty Sand	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Depth of Test Hole:	6 ft	**Date Excavated:**	July 21, 2017
Liquid Used:	Tap Water	**pH:**	7.8
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**Depth of Water in Rings:**	1 in
Tested By:	A.L.	**Ring Penetration:**	4 in
Liquid Level Maintained Using:	Vacuum Seal		
Depth to Water Table:	~ 200 ft		



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 ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	innner	annular space		
1	S	10:14	44	44	10:14	44	44	3.63	7.95	0.436	0.954	18.2	13.2	1.2	0.9	81	
	E	10:58			10:58											81	
2	S	10:58	29	73	10:58	29	73	2.86	6.19	0.343	0.743	21.7	15.6	1.5	1.0	81	
	E	11:27			11:27											82	
3	S	11:27	29	102	11:27	29	102	2.49	9.15	0.299	1.098	18.9	23.1	1.3	1.5	82	
	E	11:56			11:56											82	
4	S	11:56	33	135	11:56	33	135	2.16	6.03	0.259	0.724	14.4	13.4	1.0	0.9	83	
	E	12:29			12:29											83	
5	S	12:29	31	166	12:29	31	166	2.47	6.35	0.297	0.762	17.5	15.0	1.2	1.0	83	
	E	13:00			13:00											84	
6	S	13:00	30	196	13:00	30	196	2.27	7.81	0.273	0.938	16.7	19.1	1.1	1.3	84	
	E	13:30			13:30											84	
7	S	13:30	33	229	13:30	33	229	2.27	5.43	0.273	0.652	15.1	12.1	1.0	0.8	85	
	E	14:03			14:03											85	
8	S	14:03	28	257	14:03	28	257	2.27	3.66	0.273	0.439	17.9	9.6	1.2	0.6	86	
	E	14:31			14:31											86	
9	S	14:31	28	285	14:31	28	285	2.18	2.44	0.262	0.293	17.1	6.4	1.1	0.4	86	
	E	14:59			14:59											86	