Appendix D

Preliminary Geotechnical and Infiltration Feasibility Investigation
Proposed Tamarisk Apartments, Phase 2 APN 3057-121-08

LOR Geotechnical

June 5, 2024

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED TAMARISK APARTMENTS, PHASE 2 APN 3057-121-08 HESPERIA, CALIFORNIA

PROJECT NO. 34002.1
JUNE 5, 2024

Prepared For:

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Project No. 34002.1

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Tamarisk Apartments, Phase 2, APN 3057-121-08 Hesperia, California.

LOR Geotechnical Group, Inc., is pleased to present this report of 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. However, the contents of 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 existing loose alluvial materials and undocumented fill material should be removed from structural areas and areas to receive engineered compacted fills. The data developed during this investigation indicates that removals ranging from approximately 2 to 7 feet will be required from currently planned development areas. The given removal depths are preliminary and the actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

Very low expansion potential and good R-value quality content generally characterize the upper onsite materials tested. Near completion and/or at the completion of site grading, additional foundation and subgrade soils should be tested, as necessary, to verify their expansion potential, soluble sulfate content, and R-value quality.

Favorable infiltration rates were obtained for the soils tested.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During May 2024, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed Tamarisk Apartments, Phase 2, APN 3057-121-08 in the city of Hesperia, 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 photographs of the site and surrounding regions dated 1938 through 2023;
- 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 borehole percolation test method at four locations 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 site plan, prepared by Steeno Design Studio, Inc., undated, was furnished for our use. The existing site conditions and proposed development are illustrated on this plan. As illustrated, the site is proposed to be developed with 8, two-story structures, a clubhouse, and the associated landscape, parking, and driveways.

This plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A.

The proposed structures are anticipated to consist of two story wood frame and stucco or similar type construction. Conventional foundation systems with light to moderate foundation loads are anticipated with such structures. Excluding removals and over-excavation, site grading is anticipated to involve minimal cuts and fills.

AERIAL PHOTO ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photographs of varying scales. We reviewed imagery available from Google Earth (2024) and from Historic Aerials (2024).

To summarize briefly, the site was vacant natural land, since the earliest photograph available in 1938. Around 2020, construction of Tamarisk Apartments Phase 1 adjacent to the site on the north began, and by 2022 appears to be completed. Around this time, end dump piles of fill appear within the eastern portion of the site. It appears that the end dumped piles of fill most likely originated during the grading of the adjacent development. The site remained essentially as seen today since that time. No evidence for the presence of faults traversing the site area or mass movement features was noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The subject site generally consists of a roughly rectangular parcel of land comprising approximately 4.9 acres. At the time of our investigation, the site is vacant desert land generally in a natural state. The topography of the site is relatively planar with a very gentle fall toward the east. Approximately one third of the site, from the center to the eastern portion is overlain with a very gently sloping mound of fill approximately 5 feet higher in elevation relative to the ground surface, and approximately 200 feet long by 100 feet wide. A row of end dump piles of fill are located along the northern site boundary and is approximately 300 feet long. Vegetation across the majority of the site generally consists of a light growth of desert brush, field weeds, and occasional Joshua trees.

The site lies at the northwest corner of the intersection of Orange Street and Tamarisk Avenue. Orange Street, a paved roadway, bounds the property on the south followed by a tract of single family residential homes. The site is bound by Tamarisk Avenue, also a paved roadway, on the east followed by large lot out single family residential homes.

Relatively vacant land in natural desert state bounds the site on the west. Phase 1 of the Tamarisk Apartments Development bounds the site on the north.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on May 16, 2024. The work consisted of advancing a total of 6 exploratory borings on the site using a Mobile B-61 truck mounted drill rig equipped with an 8-inch diameter hollow stem augers. In addition, 4 borehole percolation tests were conducted in general accordance with the Deep Percolation Test procedure as outlined in the Technical Guidance Document for Water Quality Management Plans (CDM Smith, 2013). The approximate locations of our exploratory borings and percolation tests are presented on Enclosure A-2, within Appendix A.
Logs of the subsurface conditions encountered in the exploratory borings were maintained by a geologist from this firm. The borings were drilled to depths ranging from approximately 16.5 to 51.5 feet below the existing ground surface. Relatively undisturbed and bulk samples were generally obtained at a typical maximum depth interval of 5 feet and returned to the laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the boring logs are presented in Appendix B, while a detailed description of our borehole percolation testing program and the test results are presented in Appendix C.

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 in-place moisture content and dry density, laboratory compaction characteristics, direct shear, sieve analysis, sand equivalent, R-value, expansion index, and corrosion screening. Physical testing was conducted in our geotechnical laboratory and chemical testing was conducted by our subconsultant, Project X Corrosion Engineering. A detailed description of the geotechnical laboratory testing program and the test results are presented in Appendix D.

GEOLOGIC CONDITIONS

Regional Geologic Setting

The site is situated along the southern edge of the Mojave Desert on a series of coalescing alluvial fans and terraces collectively referred to as the Cajon Fan. These fans and terraces have formed from sediment eroded from the San Gabriel and San Bernardino Mountains in Pleistocene and Recent times. The subject site is generally located on a large, wide fan region within the Cajon Fan series, referred to as the Baldy Mesa Fan. The Baldy Mesa Fan slopes to the northeast and is composed predominantly of silty sand and poorly graded to well graded sand, with lesser amounts of clayey sand. These fans lie on a very thick sequence of terrestrial sedimentary rocks, which in turn overlie crystalline bedrock (Dibblee, 1965).

This area north of the San Gabriel Mountains lies along the southeastern portion of a larger geomorphic province in southern California known as the Mojave Desert. The Mojave Desert geomorphic province is essentially a very large, wedge shaped, alluviated plain of comparatively low relief, containing irregularly trending bedrock hills and low mountains.

The Mojave Desert province is bounded on the southwest by the San Andreas fault zone and on the north by the Garlock fault zone. The eastern boundary of the Mojave Desert geomorphic province is not distinct, but gradually converges with the Basin and Range geomorphic province east of Death Valley and into Arizona and Nevada. The province is broken by many internal, major but discontinuous faults, predominately trending to the northwest showing rough parallelism with the trend of the San Andreas. Most of these faults have been active within the last 1.6 million years and many are still considered to be active or potentially active.

The closest known fault to the subject site noted in the documents reviewed during our study is the North Frontal fault located approximately 12.9 kilometers (8.0 miles) southeast of the site. The North Frontal fault is considered to be an active fault. 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 regional geology as mapped by the U.S.G.S. (Dibblee, 1965) and partial legend is shown on Enclosure A-3, within Appendix A.

Site Geologic Conditions

As observed and encountered during this investigation, the eastern portion of the property is underlain by a thin veneer of fill overlying alluvium while alluvium is present at the surface of the site. These units are described in further detail in the following sections:

Fill: Fill materials, believed to be derived from the adjacent multi-family residential development that bounds the site on the north, are present within the eastern portion of the site. The fill was noted be comprised of silty sand which was brown in color, dry, and in a loose state.

Alluvium: Alluvial soils were encountered underlying the fill material described above and were observed at the surface. The alluvial soils consist mostly of silty sand and well graded sand with minor units of well graded sand with silt, poorly graded sand with silt, and silty sand/sandy silt. In general, the alluvial materials were in a medium dense state upon first encounter, generally becoming increasingly dense with increasing depth based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. The alluvium was typically white to red brown in color and dry to damp.

A geological map of the site is presented as Enclosure A-4, within Appendix A. 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 below the existing ground surface of 3,418± feet, above mean sea level.

According to the Department of Water Resources Water Data Library (2024), the nearest water well to the site for which groundwater level data is available is Local Well No. 04N05W13J001S, located approximately 1.1 kilometers (0.7 miles) to the north. Data indicates this well had a depth to groundwater of 595 feet beneath the ground surface elevation of 3,368 feet above mean sea level in 2006, the only date for which data is available.

State Well No. 04N05W21H001S, located approximately 4.7 kilometers (3.0 miles) to the east, had data available from 1995 to 2023. During that time, groundwater ranged from approximately 648 to 660 feet below the ground surface elevation of 3,535 feet above mean sea level.

Based on this information and the lack of groundwater encountered during our field exploration, groundwater under the site is anticipated to lie in excess of 500 feet and does not appear to be a factor in the development of the site.

Mass Movement

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, 1997).

As previously mentioned, the closest known active fault is the North Frontal fault, located approximately 12.9 kilometers (8.0 miles) to the southeast. In addition, other relatively close active faults include the Cleghorn fault, located approximately 13.2 kilometers (8.2 miles) to the south, the San Andreas fault located approximately 19.8 kilometers (12.3 miles) to the southwest, the San Jacinto fault located 21.8 kilometers (13.5 miles) to the southwest, and Helendale fault located approximately 32.4 kilometers (20.2 miles) to the northeast.

The North Frontal fault zone of the San Bernardino Mountains is a zone consisting of numerous fault segments, many of which have their own names. The primary sense of slip is south dipping thrust. This fault seems to be offset (right-laterally) by the Helendale fault. It is believed that the North Frontal fault zone is capable of producing an earthquake magnitude on the order of 6.0 to 7.1.

The Cleghorn fault of the San Bernardino Mountains is a left-lateral strike-slip fault. The exact nature of the activity of this fault is questionable. The local landscape does not seem to express the reported slip rate (0.3 mm/yr) and some have dismissed Holocene displacement and rupture surfaces as caused by landsliding, not faulting. However, it is believed that the Cleghorn fault is capable of producing an earthquake magnitude on the order of 6.5.

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 24 mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

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 Helendale fault is a right-lateral strike slip fault. This fault has been active very recently. It is believed that the Helendale fault is capable of producing an earthquake magnitude on the order of 6.5 to 7.3.

Current standards of practice often include 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 is considered as relevant to the site as the faults described above, due to their greater distance and/or smaller 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 website of the U.S.G.S. (2024). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from January 1, 1932 through May 28, 2024.

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-5, within Appendix A, the site lies within a relatively active region associated with the San Andreas fault and various Mojave Desert faults to the east. The 7.3 magnitude Landers earthquake and associated aftershocks including the 6.3 magnitude Big Bear earthquake are illustrated on this map, located to the northeast of the site. In addition, the 6.6 magnitude 1971 San Fernando Earthquake is also illustrated.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 1.0 and greater since 1978. 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 time period for the events on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid to late1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-6, within Appendix A, a few events are present in the area, some of which are associated with the North Frontal and Cleghorn faults to the south.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring in the region around the subject site. 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 granular loose sediments where the groundwater is usually less than 50 feet below the ground surface. As groundwater is anticipated to lie greater than 50 feet beneath the site and the site is underlain by relatively dense alluvial materials, the possibility of liquefaction at the site 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 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 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.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the site is underlain by relatively dense alluvial materials, the potential for settlement is considered very low. In addition, the recommended earthwork operations to be conducted during the development of the site should mitigate any near surface loose soil conditions.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2022)

Design requirements for structures can be found within Chapter 16 of the 2022 California Building Code (CBC) based on building type, use, and/or occupancy. The classification of use and occupancy of all proposed structures at the site, shall be the responsibility of the building official.

Site Classification

Chapter 20 of the ASCE 7-16 defines six possible site classes for earth materials that underlie any given site. Our investigation, mapping by others, and our experience in the site region indicates that the materials beneath the site are considered Site Class D stiff soils.

CBC Earthquake Design Summary

Earthquake design criteria have been formulated in accordance with the 2022 CBC and ASCE 7-16 for the site based on the results of our investigation to determine the Site Class and an assumed Risk Category II. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region. In addition, the building official should confirm the Risk Category utilized in our design (Risk Category II). Our design values are provided below:

CBC 2022/ASCE 7-16 SEISMIC DESIGN SUMMARY* Site Location (USGS WGS84) 34.4235, -117.3519, Risk Category II	
Site Class Definition Chapter 20 ASCE 7	D
$\mathbf{S_s}$ Mapped Spectral Response Acceleration at 0.2s Period	1.474
$\mathbf{S_1}$ Mapped Spectral Response Acceleration at 1s Period	0.571
$\mathbf{S_{MS}}$ Adjusted Spectral Response Acceleration at 0.2s Period	1.494
$\mathbf{S_{M1}}$Adjusted Spectral Response Acceleration at 1s Period	1.142
$\mathbf{S_{DS}}$ Design Spectral Response Acceleration at 0.2s Period	0.996
$\mathbf{S_{D1}}$ Design Spectral Response Acceleration at 1s Period	0.761
$\mathbf{F_a}$ Short Period Site Coefficient at 0.2s Period	1.0
$\mathbf{F_v}$ Long Period Site Coefficient at 1s Period	1.7
$\mathbf{PGA_M}$ Site Modified Peak Ground Acceleration	0.553
Seismic Design Category	D
*See Appendix E for detailed calculations	

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 of the site for the proposed use is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

It should be noted that the subsurface conditions encountered in our exploratory borings are indicative of the locations explored and the subsurface conditions may vary. If conditions are encountered during the construction of the project that 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

To provide adequate support for the proposed structure, 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.

Conventional foundation systems utilizing 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

The upper materials encountered during this investigation are relatively granular and were tested and found to have a very low expansion potential. Therefore, specialized construction procedures to specifically resist expansive soil activity for this type of soil are not anticipated at this time.

Corrosion Screening

Select representative samples from our borings were taken to Project X Corrosion Engineering for full corrosion series testing. Results from soil corrosivity testing completed by Project X Corrosion Engineering are presented within Appendix D.

The corrosivity test results indicate that soluble sulfate concentrations in the samples were less than 0.10 percent by weight. These concentrations indicate an exposure class S0 for sulfate (ACI 318). No special mitigation methods are considered necessary.

The corrosivity test results indicate that chloride concentrations were below 500 ppm. This concentration indicates an exposure class C1 for chloride (ACI 318). Special mitigation measures are not considered necessary.

Soil pH for the samples was 7.6, neutral. Therefore, the need for specialized design is not anticipated.

Concentrations of ammonium and nitrate indicate the soil may be aggressive towards copper.

Resistivity results for the samples indicate the soil is mildly to moderately corrosive to ferrous metals.

LOR Geotechnical does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, is required, then a competent corrosion engineer should be consulted.

Infiltration

The results of our field investigation and test data indicate the soils tested have an average clear water infiltration rate of approximately 3.5 inches per hour for the proposed northern chamber system and approximately 4.7 inches per hour for the proposed southern chamber system.

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 the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2010) nor within a County of San Bernardino fault zone (San Bernardino County, 2024), the probability of ground surface rupture occurring at the site is considered nil.

Due to the site's close proximity to the faults described above, it is reasonable to expect a relatively 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 faults described above from a ground motion standpoint.

The affects 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

Geologic Recommendations

No special geologic recommendations 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 onsite, 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 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. All undocumented fill encountered during grading should be completely removed, cleaned of significant deleterious materials and may then be reused as compacted fill. It is our recommendation that any existing 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.

Although not anticipated, cavities created by the removal of any subsurface obstructions such as foundations, utilities, and septic systems 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

The existing loose alluvial soils and existing fill materials should be removed from all proposed structural and/or fill areas. The data developed during this investigation indicates that removals on the order of 2 to 7 feet deep will be required from proposed development areas in order to encounter competent alluvium upon which engineered compacted fill can be placed. The given removal depths are preliminary. Deeper fills may be present locally. Removals should expose alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557). The actual depths of the removals should be determined during the grading operation by observation and/or 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 minimum depth of 12 inches. The scarified soil should be brought to near optimum moisture content and compacted to a relative compaction of at least 90 percent (ASTM D 1557).

Engineered Compacted Fill

The onsite soils should provide adequate quality fill material, provided they are free from oversized and/or 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.

Preparation of Foundation Areas

All footings should rest upon at least 24 inches of properly compacted fill material placed over competent alluvium. In areas where the required fill thickness is not accomplished by the recommended removals or by 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. The bottom of all excavations should be scarified to a depth of 12 inches, brought to near optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557) prior to the placement of compacted fill.

Concrete floor slabs should bear on a minimum of 24 inches of compacted soil. This should be accomplished by the recommendations provided above. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

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 will conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on the findings from our exploratory borings, it appears that Type C soils are 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 four, Design by a Registered Professional Engineer (Section 1541.1).

Short-term excavation 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.

Slope Construction

Preliminary data indicates that cut and fill slopes should be constructed no steeper than two horizontal to one vertical. Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction, then roll the final slopes to provide dense, erosion-resistant surfaces.

Slope Protection

Since the site soil materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after the completion of grading. The use of succulent ground covers such as iceplant or sedum is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the irrigation system and to prevent over watering.

Soil Expansiveness

The upper 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 anticipated at this time and are provided within the following sections of this report.

Additional evaluation of on-site and any 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 supported on conventional shallow foundations, either individual spread footings and/or continuous wall footings, bearing entirely on a minimum of 24 inches of engineered compacted fill placed over competent alluvial materials. All foundations should have a minimum width of 12 inches. Footings placed upon very low expansive soils 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 2,000 psf. This bearing pressure may be increased by 200 psf for each additional foot of width, and by 500 psf for each additional foot of depth, up to a maximum of 4,000 psf. For example, a footing 2 feet wide and embedded 2 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.

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 0.40 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 one-third for wind or seismic loading.

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.

Building Area Slab-on-Grade

To provide adequate support, concrete floor slabs-on-grade should bear on a minimum of 24 inches of engineered fill compacted soil. The final pad surfaces should be rolled to provide smooth, dense surfaces.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder/barrier. We recommend that a vapor retarder/barrier be designed and constructed according to the American Concrete Institute 302.1R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder/barrier construction. At a minimum, the vapor retarder/barrier should comply with ASTM E1745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage. Per the Portland Cement Association, for slabs with vapor-sensitive coverings, a layer of dry, granular material (sand) should be placed under the vapor retarder/barrier.

For slabs in humidity-controlled areas, a layer of dry, granular material (sand) should be placed above the vapor retarder/barrier.

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 retaining walls should be performed in accordance with the recommendations described earlier 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 active pressure of 40 pounds per square foot (psf) per foot of depth be used. This assumes level backfill consisting of compacted, non-expansive, on-site 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 structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.40 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 three 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.

Preliminary Pavement Design

Testing and design for preliminary onsite pavement was conducted in accordance with the California Highway Design Manual and the Guide for the Design and Construction of Concrete Parking Lots (ACI33OR).

Based upon our preliminary sampling and testing, and upon an assumed Traffic Index generally used for similar projects, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject on-site pavement improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
On site vehicular parking with occasional truck traffic (ADTT=1)	5.0	50	0.25’ AC / 0.35' AB or 0.35' AC / Native or 0.33'PCC/Native
On site vehicular parking with occasional truck traffic (ADTT=10)	6.0	50	0.25’ AC / 0.35' AB or 0.45' AC / Native or 0.42' PCC / Native
AC - Asphalt Concrete AB - Class 2 Aggregate Base PCC - Portland Cement Concrete			

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 pavement subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 Aggregate Base.

The recommended Portland Cement (PCC) concrete pavement should have a minimum modulus of rupture (MR) of 550 pounds per square inch (psi). 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.

It should be noted that all of the above pavement design was based upon the results of preliminary sampling and testing, and should be verified by additional sampling and testing during construction when the actual subgrade soils are exposed.

<u>Infiltration</u>

Based upon our field investigation and infiltration test data, an average clear water absorption rate of approximately 3.5 inches per hour for the proposed northern chamber system and approximately 4.7 inches per hour for the proposed southern chamber system. It is our opinion that a design clear water rate of 3.5 inches per hour is appropriate for the planned infiltration for the northern chamber at the depth tested and design clear water rate of 4.7 inches per hour is appropriate for the planned infiltration for the southern chamber at the depth tested.

A factor of safety should be applied as indicated by the Technical Guidance Document for Water Quality Management Plans (CDM Smith, 2013). The design infiltration rate should be adjusted using Worksheet H, using the following factor values in determination of the suitability assessment, S_A:

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment method	0.25	1	0.25
		Predominant soil texture	0.25	1	0.25
		Site soil variability	0.25	2	0.50
		Depth to groundwater/impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \sum p$			1.25

The project design engineer should determine the suitability assessment S_B.

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 considerably 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 should be prepared and properly executed. At a minimum, the program should be as outlined in the Technical Guidance Document for Water Quality Management Plans (CDM Smith, 2013).

The program should also incorporate the recommendations contained within this report and any other jurisdictional agency requirements.

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

- 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.

Corrosion Protection

Based on the test results, this soil is classified as exposure class S0 for sulfate and exposure class C1 for chloride (ACI 318), mildly to moderately corrosive to ferrous metals, and potentially aggressive towards copper. The laboratory data above should be reviewed and corrosion design should be completed by a qualified corrosion engineer.

In lieu of corrosion design for metal piping, ABS/PVC may be used. Soil corrosion is not considered a factor with ABS/PVC materials. ABS/PVC is considered suitable for use due to the corrosion potential of the on-site soils with respect to metals.

LOR Geotechnical does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, is required, then a competent corrosion engineer should be consulted.

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, and soluble sulfate content testing should be conducted 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 excavations prior to the processing and preparation of the bottom areas for fill placement.

3. Scarifying and compacting prior to fill placement.

4. Foundation excavations.

5. Subgrade preparation for pavements and slabs-on-grade.

6. 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 Mark Maida, and the 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.

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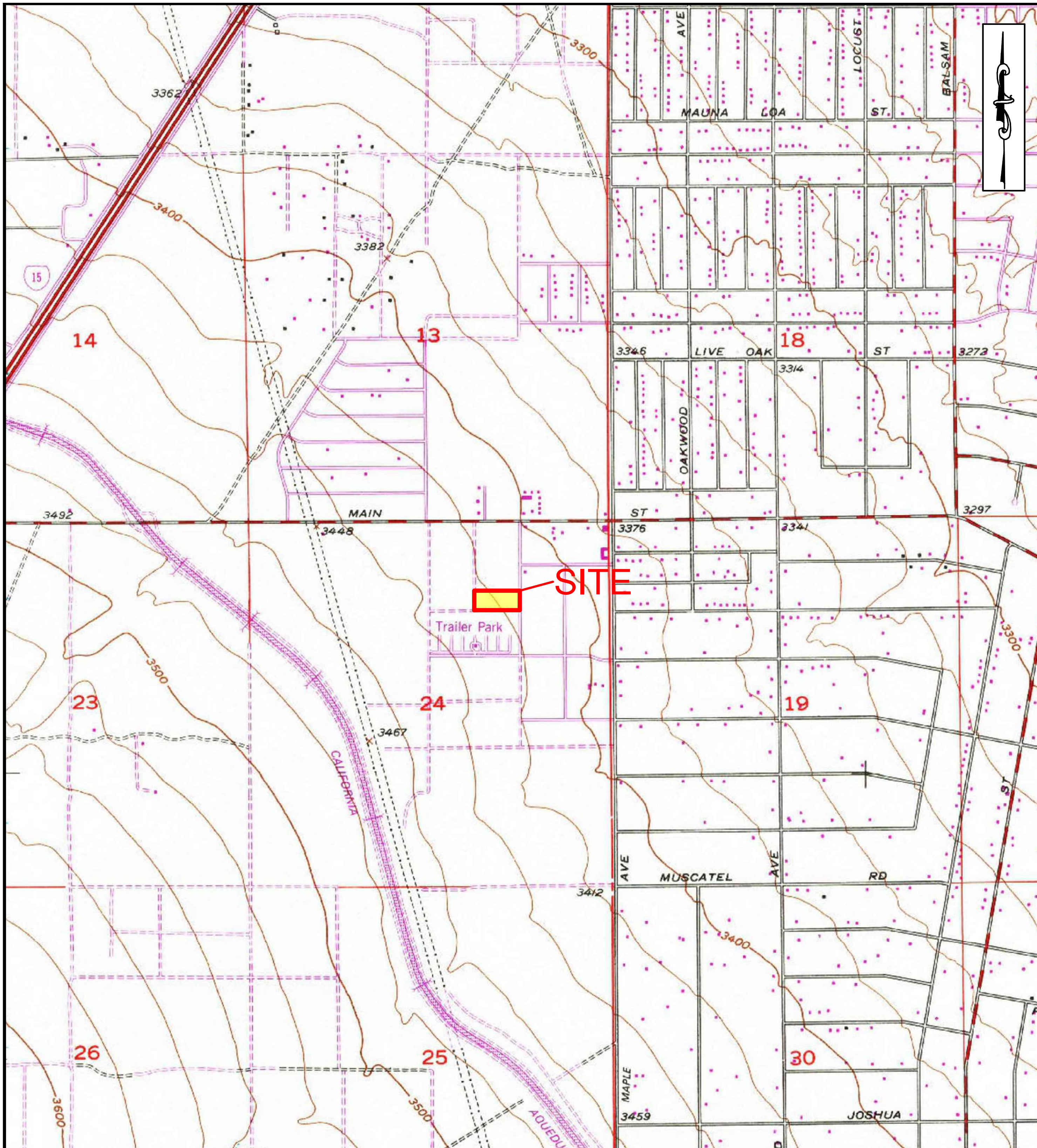
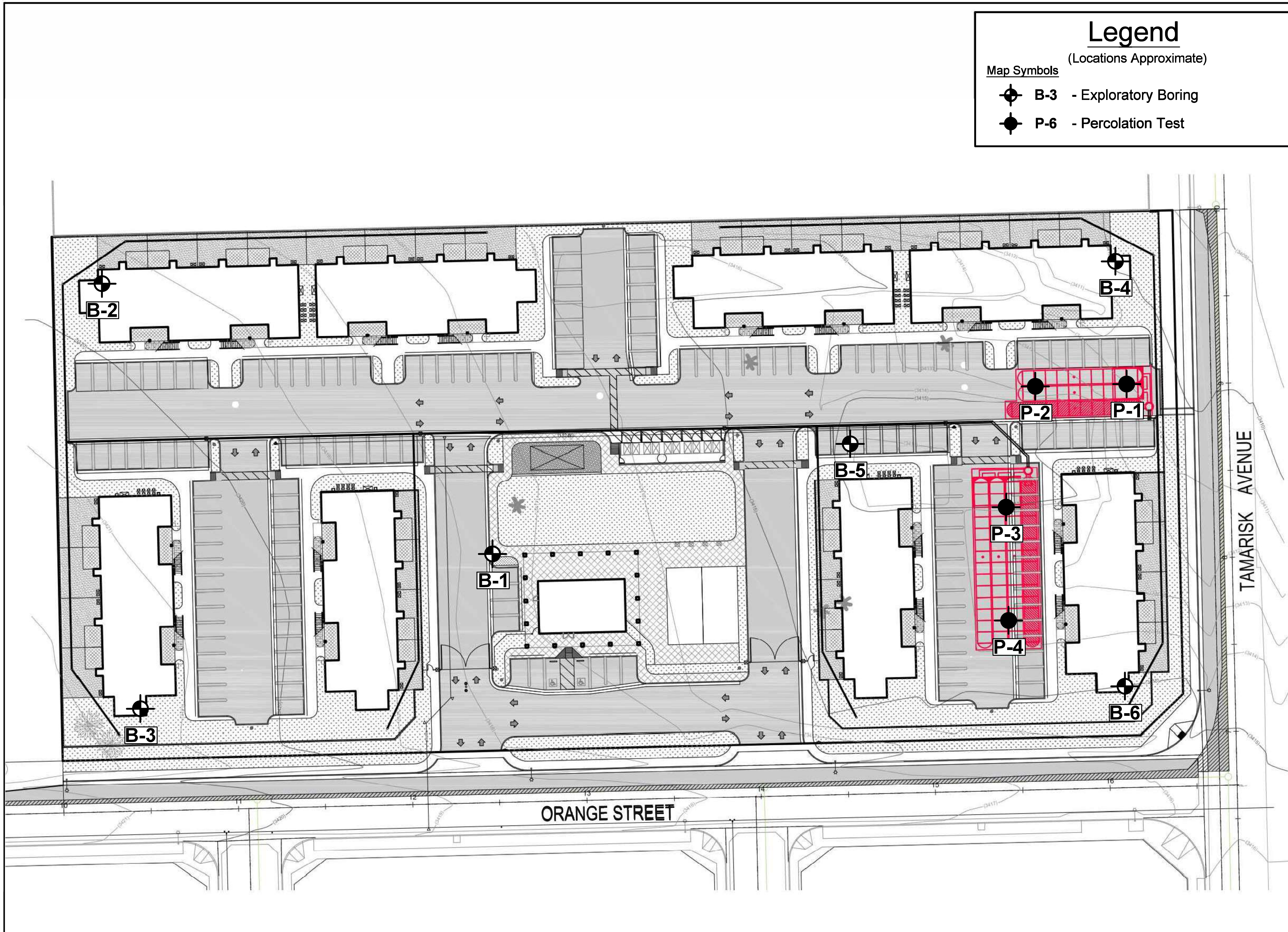
APPENDIX A

Index Map, Site Plan, Regional Geologic Map, and Historical Seismicity Maps



INDEX MAP

PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2024
		SCALE:	1" ≈ 2,000'



SITE PLAN

PROJECT: Tamarisk Apartments Phase 2, Hesperia, California

CLIENT: Mark Maida

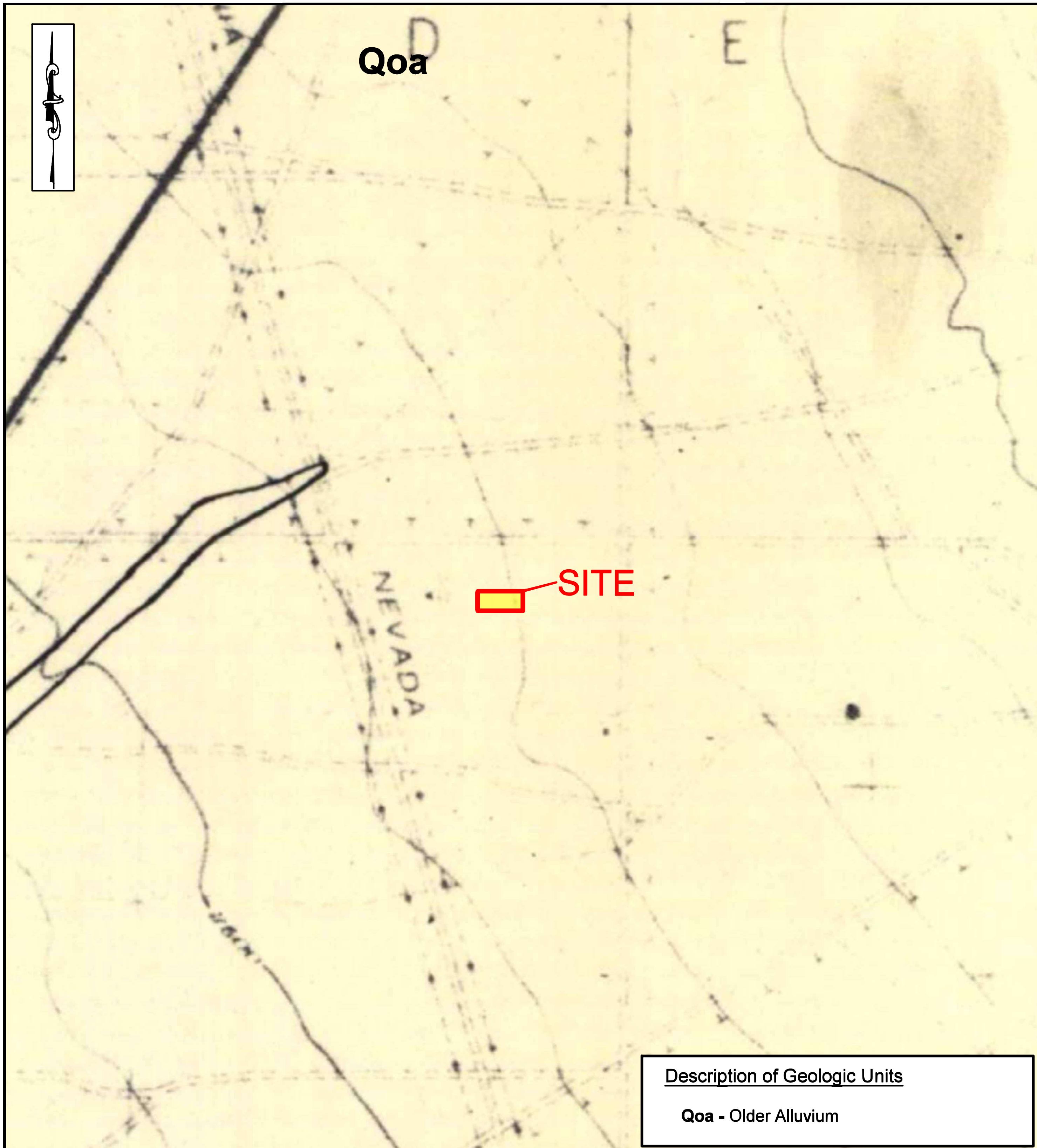
PROJECT NO.: 34002.1

ENCLOSURE: A-2

DATE: May 2024

SCALE: 1" ≈ 50'

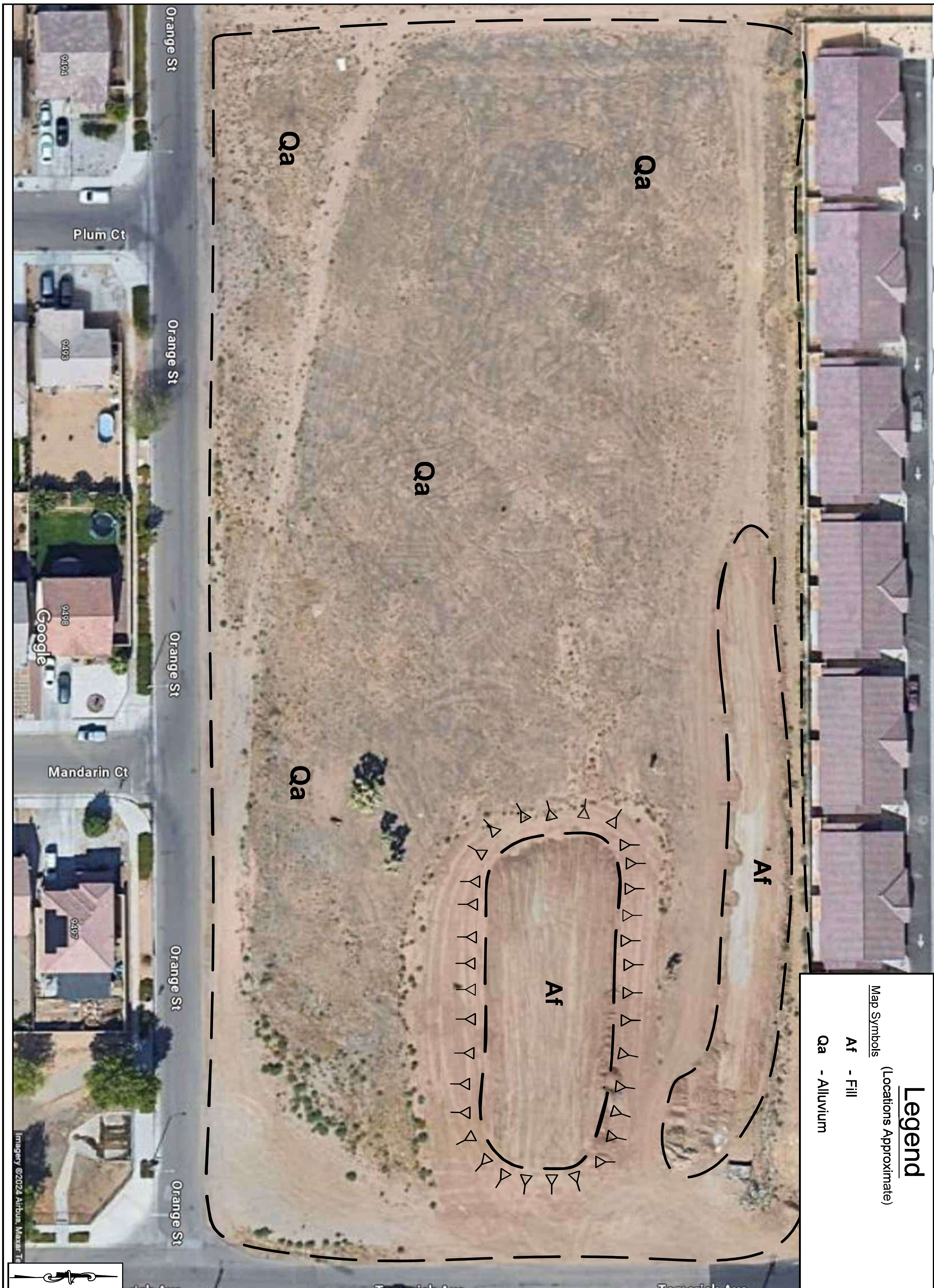
LOR GEOTECHNICAL GROUP, INC.



REGIONAL GEOLOGIC MAP

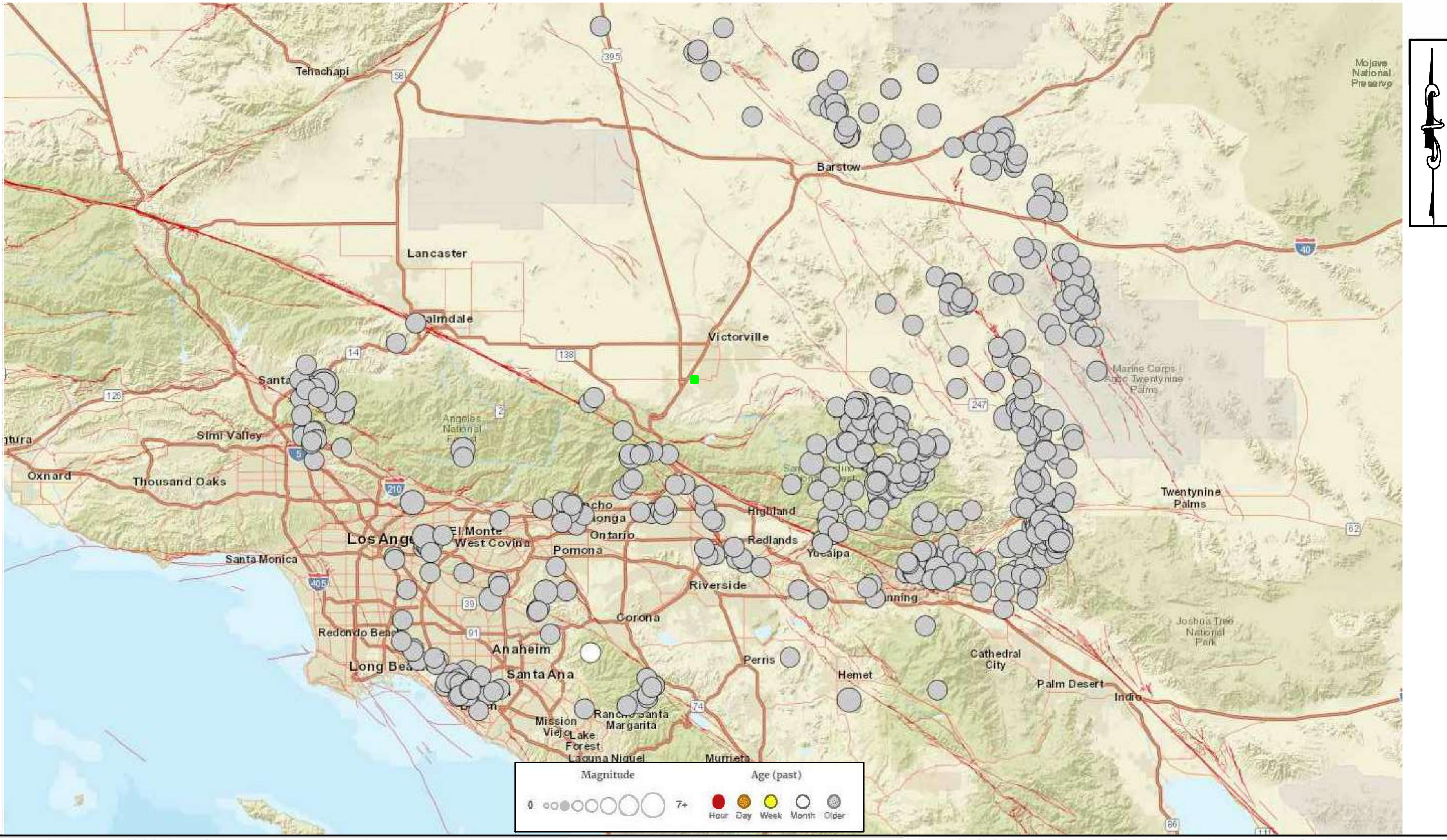
(Dibblee, 1965)

PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2024
		SCALE:	1" ≈ 2,000'



SITE GEOLOGICAL MAP

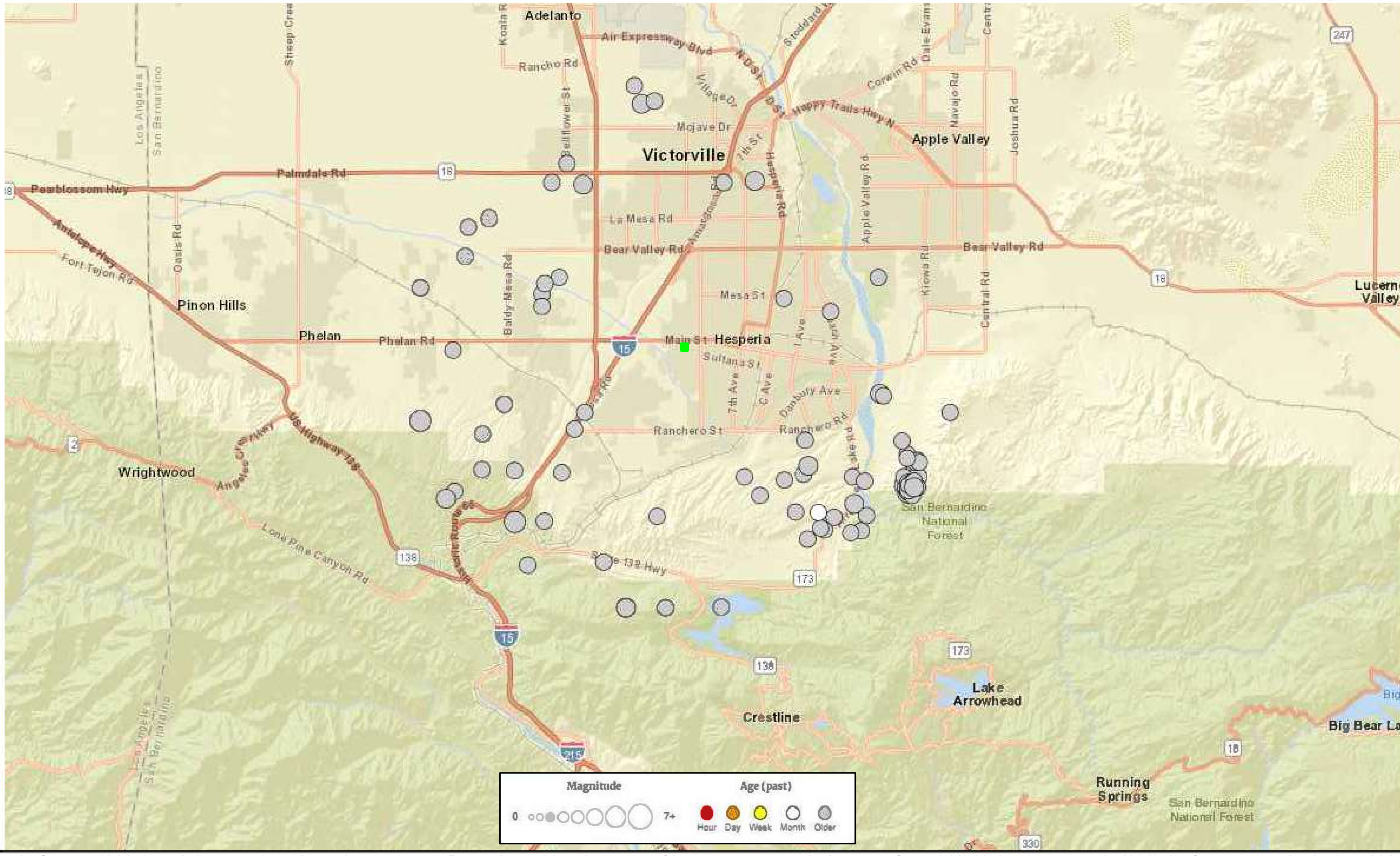
PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2024
		SCALE:	1" ≈ 50'



U.S. Geologic Survey (2024) real-time earthquake epicenter map. Plotted are 505 epicenters of instrument-recorded events from 01/01/32 to present (05/28/24) of local magnitude 4+ within a radius of ~62 miles (100 kilometers) of the site. Location accuracy varies. The site is indicated by the green square (■). The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These events are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 100km Radius

PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2024
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2024) real-time earthquake epicenter map. Plotted are 86 epicenters of instrument-recorded events from 01/01/78 to present (05/28/24) of local magnitude 1+ within a radius of ~9.2 miles (15 kilometers) of the site. Location accuracy varies. The site is indicated by the green square (■). The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These events are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 15km Radius

PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ENCLOSURE:**	A-6
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2024
		SCALE:	1" ≈ 10km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling 6 exploratory borings to depths of approximately 16.5 to 51.5 feet below the existing ground surface using a Mobile B-61 drill rig on May 16, 2024. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

The drilling exploration was conducted using a Mobile B-61 drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by a 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 3.25 inch outside diameter or a Standard Penetration Sampler (SPT) from the ground surface to the total depth explored. The samplers were 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 were recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), borehole diameter (8"), 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.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-6. A Boring Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented as 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:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mr. Mark Maida	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May, 2024

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 No. 200

(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	Tamarisk Apartments Phase 2, Hesperia, California	**PROJECT NO.:**	34002.1
CLIENT:	Mr. Mark Maida	**ENCLOSURE:**	B-ii
		DATE:	May 2024

LOR GEOTECHNICAL GROUP, INC.

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	45	3, 4, 7, 9 10	4.9		124.5			SM	@ 0 feet, ALLUVIUM: SILTY SAND, trace gravel to 1/2" approximately 10% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 25% silty fines, brown, dry, disturbed to upper 6 to 8". @ 2 feet, becomes red brown, slightly coarser grained.
5	29 58		2.5 1.2		112.0 118.3			SW SM	@ 5 feet, WELL GRADED SAND with SILT, trace gravel to 1/2" approximately 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 10% silty fines, light red brown, dry. @ 7 feet, approximately 15% gravel to 3".
10	55		1.8		126.1			SM	@ 10 feet, SILTY SAND, trace gravel to 1/2", approximately 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, light red brown, dry.
15	40		1.6		106.2			SW SM	@ 15 feet, WELL GRADED SAND with SILT, approximately 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 10% silty fines, red brown, dry.
20	35		2.3						@ 20 feet, contains trace gravel to 3/4", ring disturbed.
25	32		2.2					SW	@ 25 feet, SILTY SAND, approximately 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, yellow brown, dry.
30	35		1.9						@ 30 feet, WELL GRADED SAND, approximately 5% gravel to 3/4", 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, white, dry.
35	35		3.8					ML	@ 35 feet, SANDY SILT, approximately 10% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 55% silty fines, trace clay, tan, dry.
40	52		1.6					SW	@ 40 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", 30% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, red brown, dry.
45	55		1.1						
50	38		1.5					SP SM	@ 50 feet, POORLY GRADED SAND with SILT, trace gravel to 1/2" approximately 5% coarse grained sand, 15% medium grained sand, 70% fine grained sand, 10% silty fines, tan, dry. END OF BORING @ 51.5'
55									No fill No groundwater No bedrock

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,418
		DATE DRILLED:	May 16, 2024
		EQUIPMENT:	Mobille B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-1

LOR GEOTECHNICAL GROUP, INC.

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, ALLUVIUM: **SILTY SAND, trace gravel to 1/2" approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines, brown, dry, disturbed to upper 6 to 8".**
	41		5.7		127.8				**@ 2 feet, becomes red brown.**
5	34		3.6		117.6				**@ 5 feet, SILTY SAND, approximately 25% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, red brown, trace thin calcite stringers, damp.**
10	36		7.0		126.2				**@ 10 feet, SILTY SAND, approximately 5" gravel to 3/4", 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 20% silty fines with trace clay, red brown, damp.**
15	71		2.8		107.4			SW	**@ 15 feet, WELL GRADED SAND, approximately 5% gravel to 3", 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, red brown, damp, difficult to drill.**

END OF BORING @ 16.5' due to refusal

No fill
No groundwater
No bedrock

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,421
		DATE DRILLED:	May 16, 2024
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobille B-61
		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		9, 10						SM	@ 0 feet, ALLUVIUM: **SILTY SAND, trace gravel to 1/2", approximately 5% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 25% silt fines, brown, dry, disturbed in upper 6 to 8".**
	55		5.4		125.3				**@ 2 feet, SILTY SAND, trace gravel to 1", approximately 5% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 30% silt fines, red brown, damp.**
5	47		5.1		120.0				**@ 5 feet, approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silt fines with trace clay, red brown, dry.**
10	41		2.6		114.6				**@ 10 feet, slightly coarser grained.**
15	37		2.1		110.4			ML SM	**@ 15 feet, SILTY SAND/SANDY SILT, approximately 5% coarse grained sand, 15% medium grained sand, 30% fine grained sand, 50% silty fines, trace thin calcite stringers, tan, dry.**
20	39		1.3					SW	**@ 20 feet, WELL GRADED SAND, approximately 10% gravel to 1", 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, tan, dry, ring disturbed.**
25	50		1.6		112.1				**@ 25 feet, becomes slightly finer grained.**
									END OF BORING @ 26.5' **No fill** **No groundwater** **No bedrock**

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,421
		DATE DRILLED:	May 16, 2024
		EQUIPMENT:	Mobille B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-3

LOR GEOTECHNICAL GROUP, INC.

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, ALLUVIUM: SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines, brown, dry, disturbed in upper 6 to 8".
	35		6.1		125.5				@ 2 feet, becomes red brown, damp.
5	45		3.6		118.5			SW SM	@ 5 feet, WELL GRADED SAND with SILT, approximately 5% gravel to 1/2", 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 10% silty fines, red brown, dry.
10	39		1.7						@ 10 feet, slight increase in gravel, rings disturbed.
15	47		2.0						@ 15 feet, WELL GRADED SAND with SILT, approximately 20% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 10% silty fines, tan, dry, rings disturbed.
20	52		1.3						@ 20 feet, trace gravel.
25	43		2.5		108.2				@ 20 feet, rings disturbed.
									END OF BORING @ 26.5' No fill No groundwater No bedrock

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,410
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	May 16, 2024
		EQUIPMENT:	Mobille B-61
		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		9, 10						SM	@ 0 feet, FILL: SILTY SAND, approximately 15% gravel to 3/4", 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 20% silty fines, brown, dry.
	18		3.6		117.3				@ 2 feet, SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 40% fine grained sand, 20% silty fines, brown, dry.
5	39		5.3		120.7				@ 5 feet, ALLUVIUM: SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 25% silty fines, red brown, damp, trace calcite stringers.
10	28		1.8		75.4			SW	@ 10 feet, WELL GRADED SAND, approximately 10% gravel to 1/2", 25% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, tan, dry, rings disturbed.
15	29		1.9		107.1				
20	47		2.0		112.1				@ 20 feet, slightly finer grained.
25	58		1.3						@ 25 feet, rings disturbed.

END OF BORING @ 26.5'

Fill to 5'
No groundwater
No bedrock

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,415
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	May 16, 2024
		EQUIPMENT:	Mobille B-61
		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, ALLUVIUM: SILTY SAND, approximately 10% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 20% silty fines, brown, dry, disturbed in upper 6 to 8".
	38		5.4		124.2				@ 2 feet, SILTY SAND, approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines, red brown, damp.
5	45		6.1		129.0				@ 5 feet, trace gravel to 1/2.
10	11		3.1		108.0			SW	@ 10 feet, WELL GRADED SAND, approximately 30% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, red brown, damp.
	13		2.9		108.6				
15	21		4.0		107.3			SM	@ 15 feet, SILTY SAND, approximately 20% coarse grained sand, 20% medium grained sand% 25% fine grained sand, 35% silty fines, red, brown, damp.
20	37		2.7		108.5			SW	@ 20 feet, WELL GRADED SAND, trace gravel to 1/2", approximately 30% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, tan, dry.
25	47		2.0		103.8				
30	39		8.5		109.6			SM	@ 30 feet, approximately 20% coarse grained sand, 20% medium grained sand, 20% fine grained sand, 40% silty fines, tan, damp.

END OF BORING @ 31.5'

No fill
No groundwater
No bedrock

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**ELEVATION:**	3,415
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	May 16, 2024
		EQUIPMENT:	Mobille B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-6

APPENDIX C

Borehole Percolation Testing Program and Infiltration Rate Test Results

APPENDIX C
BOREHOLE PERCOLATION TESTING PROGRAM AND INFILTRATION RATE TEST RESULTS

Four borehole percolation tests were conducted in general accordance with the Deep Percolation Test procedure as outlined in the Technical Guidance Document for Water Quality Management Plans (CDM Smith, 2013). Our tests were conducted at the requested locations and depths as illustrated on Enclosure A-2. Subsequent to drilling, a 3-inch diameter, perforated PVC pipe wrapped in filter fabric was placed within each test hole and 3/4-inch gravel was placed between the outside of the pipe and the hole wall. Test holes were pre-soaked the same day as drilling. Testing took place the next day, May 17, 2024, within 26 hours but not before 15 hours, of the pre-soak. The holes were filled using water from a 200 gallon water tank. Test periods consisted of allowing the water to drop in 10-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 8 readings were recorded. The percolation test data was converted to an infiltration rate using the Porchet Method as outlined by the Technical Guidance Document (CDM Smith, 2013).

Infiltration test results are summarized in the following table:

Test No.	Depth* (ft)	Infiltration Rate** (in/hr)
P-1	10	5.6
P-2	10	1.4
P-3	15	5.8
P-4	15	3.7

* depth measured below existing ground surface
** Porchet Method determined clear water rate

The results of this testing are presented as Enclosures C-1 through C-4.

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	Tamarisk Phase 2	Test Date:	May 17, 2024
Project No.:	34002.1	Test Hole No.:	P-1
Soil Classificaiton:	(SM) Silty sand	Hole Diameter:	8.0 in.
Depth of Test Hole:	10.0 ft.	Date Excavated:	May 16, 2024
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(gal/sf/day)
1	9:41 AM	10:06 AM	25	0.42	0.42	30.00	79.50	120.00	120.00	49.50	65.25	53.0
2	10:07 AM	10:32 AM	25	0.42	0.83	36.00	78.00	120.00	120.00	42.00	63.00	46.5
3	10:33 AM	10:43 AM	10	0.17	1.00	36.00	71.25	120.00	120.00	35.25	66.38	92.8
4	10:44 AM	10:54 AM	10	0.17	1.17	36.00	69.75	120.00	120.00	33.75	67.13	87.9
5	10:55 AM	11:05 AM	10	0.17	1.33	36.00	69.25	120.00	120.00	33.25	67.38	86.3
6	11:06 AM	11:16 AM	10	0.17	1.50	37.00	69.00	120.00	120.00	32.00	67.00	83.5
7	11:17 AM	11:27 AM	10	0.17	1.67	36.00	69.00	120.00	120.00	33.00	67.50	85.5
8	11:28 AM	11:38 AM	10	0.17	1.83	36.00	68.75	120.00	120.00	32.75	67.63	84.7

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	84.00	
H_f	51.25	
ΔH	32.75	
H_{avg}	67.63	
I_t	**5.6**	in/hr (clear water rate)

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	Tamarisk Phase 2	Test Date:	May 17, 2024
Project No.:	34002.1	Test Hole No.:	P-2
Soil Classificaiton:	(SM) Silty sand	Hole Diameter:	8.0 in.
Depth of Test Hole:	10.0 ft.	Date Excavated:	May 16, 2024
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(gal/sf/day)
1	9:44 AM	10:09 AM	25	0.42	0.42	30.00	62.50	120.00	120.00	32.50	73.75	30.9
2	10:10 AM	10:35 AM	25	0.42	0.83	36.00	60.50	120.00	120.00	24.50	71.75	23.9
3	10:36 AM	10:46 AM	10	0.17	1.00	36.00	49.75	120.00	120.00	13.75	77.13	31.3
4	10:47 AM	10:57 AM	10	0.17	1.17	36.00	49.00	120.00	120.00	13.00	77.50	29.4
5	10:58 AM	11:08 AM	10	0.17	1.33	36.00	48.50	120.00	120.00	12.50	77.75	28.2
6	11:09 AM	11:19 AM	10	0.17	1.50	36.00	47.00	120.00	120.00	11.00	78.50	24.6
7	11:20 AM	11:30 AM	10	0.17	1.67	36.00	46.50	120.00	120.00	10.50	78.75	23.4
8	11:31 AM	11:41 AM	10	0.17	1.83	36.00	45.75	120.00	120.00	9.75	79.13	21.6

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	84.00	
H_f	74.25	
ΔH	9.75	
H_{avg}	79.13	
I_t	**1.4**	in/hr (clear water rate)

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	Tamarisk Phase 2	Test Date:	May 17, 2024
Project No.:	34002.1	Test Hole No.:	P-3
Soil Classificaiton:	(SM) Silty sand	Hole Diameter:	8.0 in.
Depth of Test Hole:	15.1 ft.	Date Excavated:	May 16, 2024
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(gal/sf/day)
1	11:44 AM	12:09 PM	25	0.42	0.42	54.00	151.00	181.00	181.00	97.00	78.50	86.8
2	12:11 PM	12:36 PM	25	0.42	0.83	54.00	146.25	181.00	181.00	92.25	80.88	80.1
3	12:38 PM	12:48 PM	10	0.17	1.00	53.00	93.00	181.00	181.00	40.00	108.00	65.5
4	12:50 PM	1:00 PM	10	0.17	1.17	54.00	90.50	181.00	181.00	36.50	108.75	59.3
5	1:02 PM	1:12 PM	10	0.17	1.33	53.00	89.00	181.00	181.00	36.00	110.00	57.9
6	1:14 PM	1:24 PM	10	0.17	1.50	52.00	89.50	181.00	181.00	37.50	110.25	60.1
7	1:26 PM	1:36 PM	10	0.17	1.67	54.00	88.00	181.00	181.00	34.00	110.00	54.6
8	1:38 PM	1:48 PM	10	0.17	1.83	54.00	88.50	181.00	181.00	34.50	109.75	55.6

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	127.00	
H_f	92.50	
ΔH	34.50	
H_{avg}	109.75	
I_t	**3.7**	in/hr (clear water rate)

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	Tamarisk Phase 2	Test Date:	May 17, 2024
Project No.:	34002.1	Test Hole No.:	P-4
Soil Classificaiton:	(SM) Silty sand	Hole Diameter:	8.0 in.
Depth of Test Hole:	15.0 ft.	Date Excavated:	May 16, 2024
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(gal/sf/day)
1	11:47 AM	12:12 PM	25	0.42	0.42	54.00	176.00	180.00	180.00	122.00	65.00	131.1
2	12:14 PM	12:39 PM	25	0.42	0.83	54.00	168.50	180.00	180.00	114.50	68.75	116.5
3	12:41 PM	12:51 PM	10	0.17	1.00	53.00	108.00	180.00	180.00	55.00	99.50	97.5
4	12:53 PM	1:03 PM	10	0.17	1.17	52.00	106.50	180.00	180.00	54.50	100.75	95.5
5	1:05 PM	1:15 PM	10	0.17	1.33	53.00	105.00	180.00	180.00	52.00	101.00	90.9
6	1:17 PM	1:27 PM	10	0.17	1.50	53.00	105.50	180.00	180.00	52.50	100.75	92.0
7	1:29 PM	1:39 PM	10	0.17	1.67	54.00	104.75	180.00	180.00	50.75	100.63	89.0
8	1:41 PM	1:51 PM	10	0.17	1.83	54.00	104.00	180.00	180.00	50.00	101.00	87.4

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	126.00	
H_f	76.00	
ΔH	50.00	
H_{avg}	101.00	
I_t	**5.8**	in/hr (clear water rate)

APPENDIX D

Laboratory Testing Program and Test Results

APPENDIX D
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. The laboratory testing program performed in conjunction with our investigation included in-place moisture content and dry density, laboratory compaction characteristics, direct shear, sieve analysis, sand equivalent, R-value, expansion index, and corrosion screening. 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 the results are shown on the boring logs, Enclosures B-1 through B-6 for convenient correlation with the soil profile.

Laboratory Compaction

A selected soil sample was 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				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-1	0-3	(SM) Silty Sand	136.5	6.5

Direct Shear Test

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 worst case conditions expected in the field.

The results of the shear test on a selected soil sample is presented in the following table:

DIRECT SHEAR TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Apparent Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-1	0-3	(SM) Silty Sand	250	31

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 grain size distribution analyses are presented graphically on Enclosure D-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 D-1.

R-Value Test

Based on the indicator testing above, a soil sample was selected and tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The results of the R-value test are presented on Enclosure D-1.

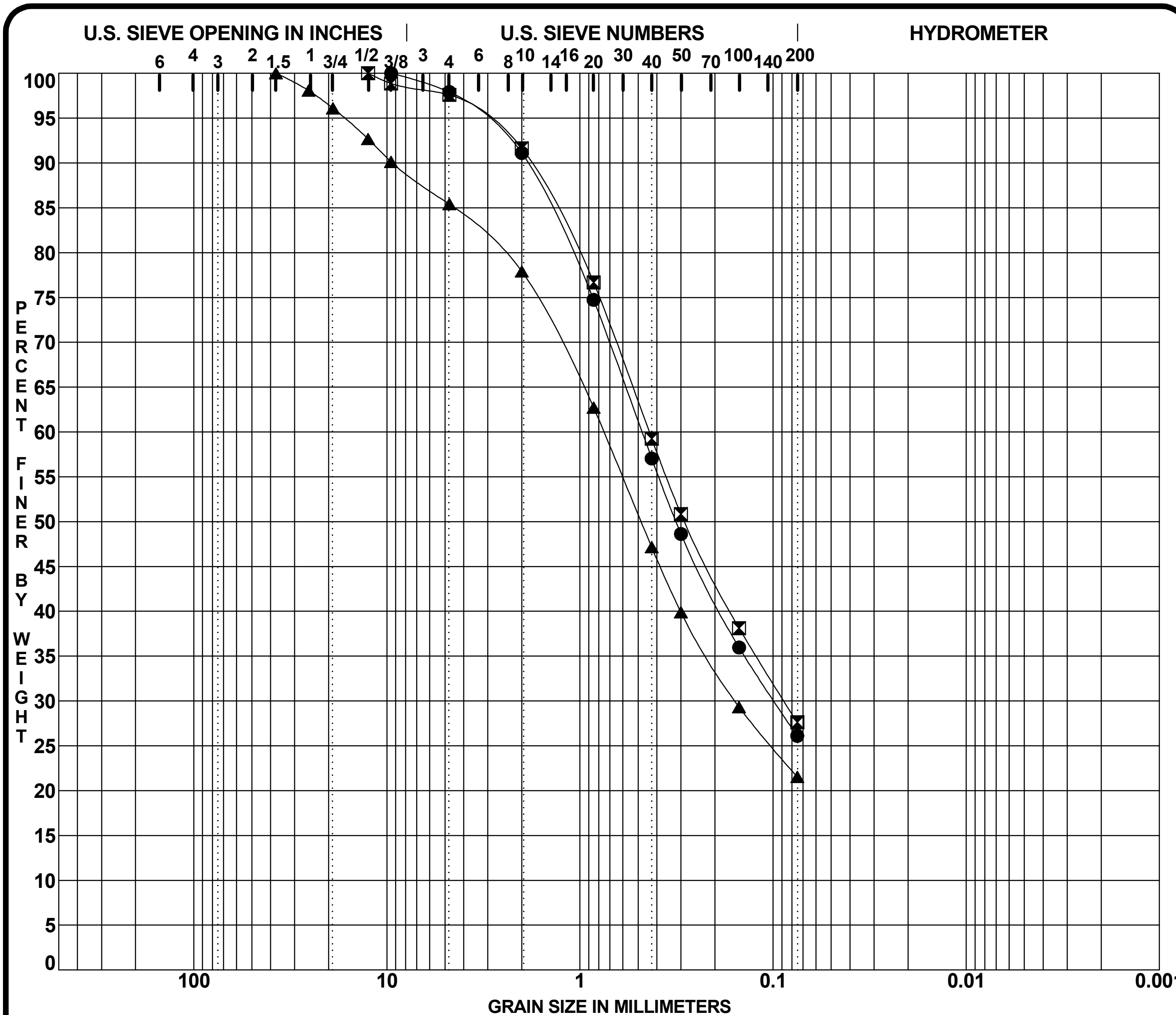
Expansion Index Test

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 result for a select soil sample is presented in the following table

EXPANSION INDEX TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-3	(SM) Silty Sand	0	Very Low
Expansion Index:		0-20 Very low / 21-50 Low	51-90 Medium	91-130 High

Corrosion

Corrosion testing was conducted by our subconsultant, Project X Corrosion Engineering. Test results are enclosed.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-1*	*0-3'*	*(SM) Silty Sand*	*20*	*--*				
⧗	*B-3*	*0-3'*	*(SM) Silty Sand*	*17*	*55*				
▲	*B-5*	*0-3'*	*(SM) Silty Sand*	*22*	*--*				

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*0-3'*	*9.50*	*0.48*	*0.099*		*2.1*	*71.8*	*26.1*	
⧗	*B-3*	*0-3'*	*12.50*	*0.44*	*0.088*		*2.4*	*70.0*	*27.6*	
▲	*B-5*	*0-3'*	*37.50*	*0.75*	*0.157*		*14.6*	*63.9*	*21.5*	

PROJECT:	Tamarisk Apartments Phase 2	**PROJECT NO.:**	34002.1
CLIENT:	Mark Maida	**DATE:**	May 2024

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: D-1

Results Only Soil Testing for Tamarisk Apartments, Hesperia

May 21, 2024

Prepared for:

Andrew Tardie
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Project X Job#: S240520A
Client Job or PO#: 34002.1

Prepared by:
D. Bobrova

Respectfully Submitted,

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REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: LOR Geotechnical
Job Name: Tamarisk Apartments, Hesperia
Client Job Number: 34002.1
Project X Job Number: S240520A
May 21, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
RV - 1 B-1 (SM) Silty Sand	0-3	7.6	0.0008	8.1	0.0008	60,300	11,390	7.6	199	0.5	1.7	3.7	0.06	12.4	12.7	22.8	60.0	2.2	5.9
RV - 3 B-5 (SM) Silty Sand	0-3	17.5	0.0018	8.6	0.0009	80,400	8,040	7.6	195	0.5	0.9	4.8	0.04	17.0	10.1	23.3	71.3	4.0	4.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

Project X
Corrosion Engineering

Lab Request Sheet Chain of Custody
Phone (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number: S240520A LOR 34002.1 TAMARISK 2FULL

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	LOR Geotechnical Group, Inc.	Contact Name:	Andrew Tardie	Phone No:	951-653-1760
Mailing Address:	6121 Quail Valley	Contact Email:	atardie@lorgeo.com		
Accounting Contact:	John Leuer	Invoice Email:	atardie@lorgeo.com		
Client Project No:	34002.1	Project Name:	Tamarisk Apartments, Hesperia		
P.O. #:	--				

	3-5 Day Standard	3 Day Guarantee 50% mark-up	24 Hour RUSH 100% mark-up
(Business Days) Turn Around Time:	●		

For Corrosion Control Recommendations (350g soil sample):
NEED (1) Groundwater depth and
(2) Soil Sample Locations Map

FOR THERMAL RESISTIVITY PROVIDE (1,500g soil sample):
(1) Optimal Moisture %
(2) Dry Density{PCF}
(3) Desired Compaction
Date & Received By:

METHOD ANALYSIS REQUESTED (Please circle)

Group	Analysis	Alternate Method	Alternate Method	Default Method
Full Corrosion Series / Geo Quad	Soil Resistivity	Caltrans CTM643	AASHTO T288	ASTM G187
Full Corrosion Series / Geo Quad	pH	Caltrans CTM643	AASHTO T289	ASTM G51
Full Corrosion Series / Geo Quad	Sulfate	Caltrans CTM417	AASHTO T290	ASTM D4327
Full Corrosion Series / Geo Quad	Chloride	Caltrans CT422	AASHTO T291	ASTM D4327
Full Corrosion Series	Redox Potential		SM 2580B	ASTM G200
Full Corrosion Series	Sulfide			SM 4500-S2
Full Corrosion Series	Ammonia		SM 4500-NH3	ASTM D4327
Full Corrosion Series	Nitrate		SM 4500-NO3	ASTM D4327
Full Corrosion Series	Flouride			SM 2320B
Full Corrosion Series	Phosphate			ASTM D4327
Full Corrosion Series	Lithium			ASTM D4327
Full Corrosion Series	Sodium			ASTM D4327
Full Corrosion Series	Potassium			ASTM D4327
Full Corrosion Series	Magnesium			ASTM D4327
Full Corrosion Series	Calcium			ASTM D4327
	Full Corrosion Series	350g Sample		
Reports	Soil Corrosivity Evaluation Report	*Req: Min. 3 Samples, site map, and groundwater info		
Reports	Water Corrosivity Mini Report			
	Moisture Content			ASTM D2216
	Total Alkalinity			SM 2320B
	Soil Remolding		ASTM D2216	ASTM D698
	Thermal Resistivity		ASTM D5334 8-inch	IEEE 442 6-inch
	(ANC) Alk Neutr Cap		1,500g Sample	
	(SRB) Sulfate Reducing			HACH BART
	Puckorius Index			
	XRF Elemental Analysis	ASTM A751	ASTM E322	ASTM E1621
	Water Hardness			

	#	SAMPLE ID - BORE # - Description	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Soil Remolding	Thermal Resistivity	(ANC) Alk Neutr Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
293	1	RV-1 - B-1 - (SM) Silty Sand	0-3	05/16/2024																●											
294	2	RV-3 - B-5 - (SM) Silty Sand	0-3	05/16/2024																●											
	3																														
	4																														
	5																														
	6																														
	7																														
	8																														
	9																														
	10																														
	11																														
	12																														
	13																														
	14																														
	15																														

APPENDIX E

Seismic Design Spectra

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

ALL values on this page were used for determination of ASCE 7-16 Section 21.3 General Spectrum and are NOT intended to be used for design

Project: Tamarisk Apartments Phase 2
Project Number: 34002.1
Client: Mark Maida
Site Lat/Long: 34.4235, -117.3519
Controlling Seismic Source: North Frontal

REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.3(ii)	F_v	2.5
$0.2^*(S_{D1}/S_{DS})$	T_0	0.194
S_{D1}/S_{DS}	T_S	0.968
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3^*S_{M1}$	S_{D1}	0.9517
Equation 11.4-2 - $F_V^*S_1$ [1]	S_{M1}	1.4275

[1] - F_V as determined by Section 21.3

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	1.474
Design Maps	S_1	0.571
Equation 11.4-1 - $F_A^*S_S$	S_{MS}	1.474
Equation 11.4-3 - $2/3^*S_{MS}$	S_{DS}	0.983
Design Maps	PGA	0.5
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.550
Section 21.5.3	80% of PGA_M	0.440
Design Maps	C_{RS}	0.928
Design Maps	C_{R1}	0.907

REFERENCE	NOTATION	VALUE
$0.2^*(S_{D1}/S_{DS})$	T_0	0.134
S_{D1}/S_{DS}	T_S	0.670
Equation 11.4-4 - $2/3^*S_{M1}$	S_{D1}	0.658
Equation 11.4-2 - $F_V^*S_1$	S_{M1}	0.987

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.928
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.907

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

Period	Cr
0.200	0.928
0.300	0.925
0.400	0.923
0.500	0.920
0.600	0.918
0.680	0.915
1.000	0.907

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

PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

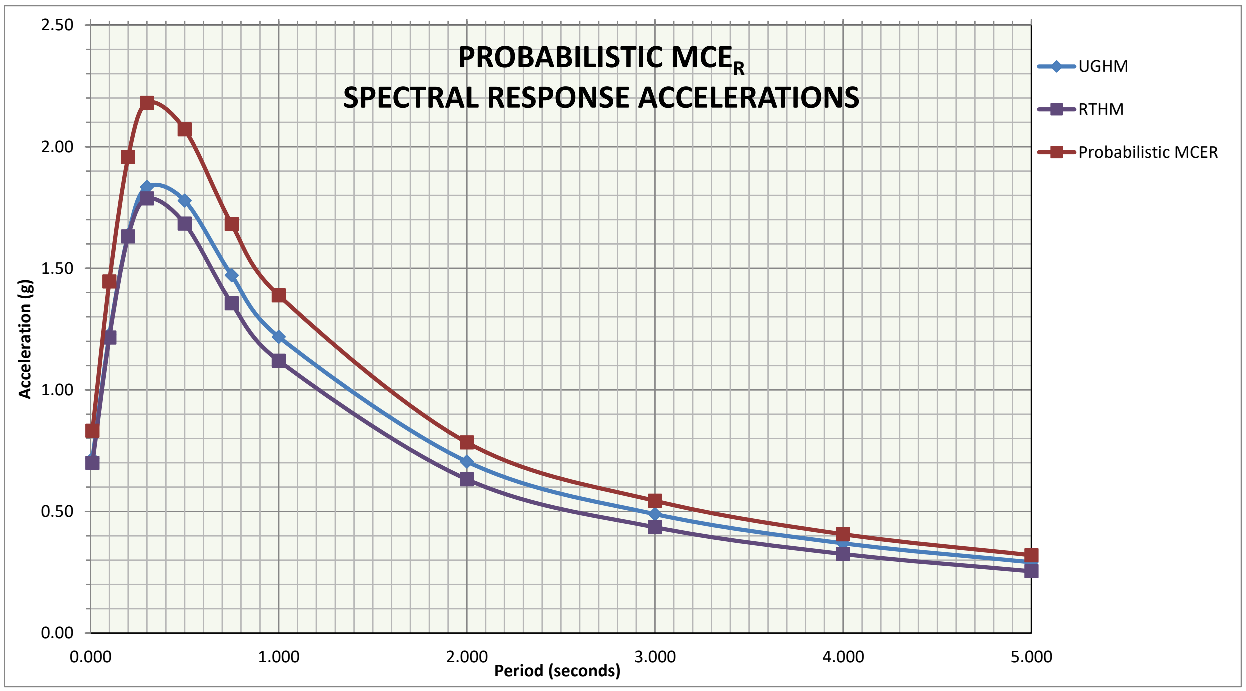
Project No: 34002.1

Period	UGHM	RTGM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.717	0.699	1.19	0.832
0.100	1.226	1.215	1.19	1.446
0.200	1.638	1.631	1.20	1.957
0.300	1.834	1.787	1.22	2.180
0.500	1.778	1.684	1.23	2.071
0.750	1.471	1.356	1.24	1.681
1.000	1.218	1.120	1.24	1.389
2.000	0.705	0.632	1.24	0.784
3.000	0.489	0.435	1.25	0.544
4.000	0.369	0.325	1.25	0.406
5.000	0.291	0.254	1.26	0.320

Probabilistic PGA:	0.717
Is Probabilistic $Sa_{(max)}<1.2F_a$?	NO

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

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



DETERMINISTIC SPECTRUM

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

Controlling Source: North Frontal

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

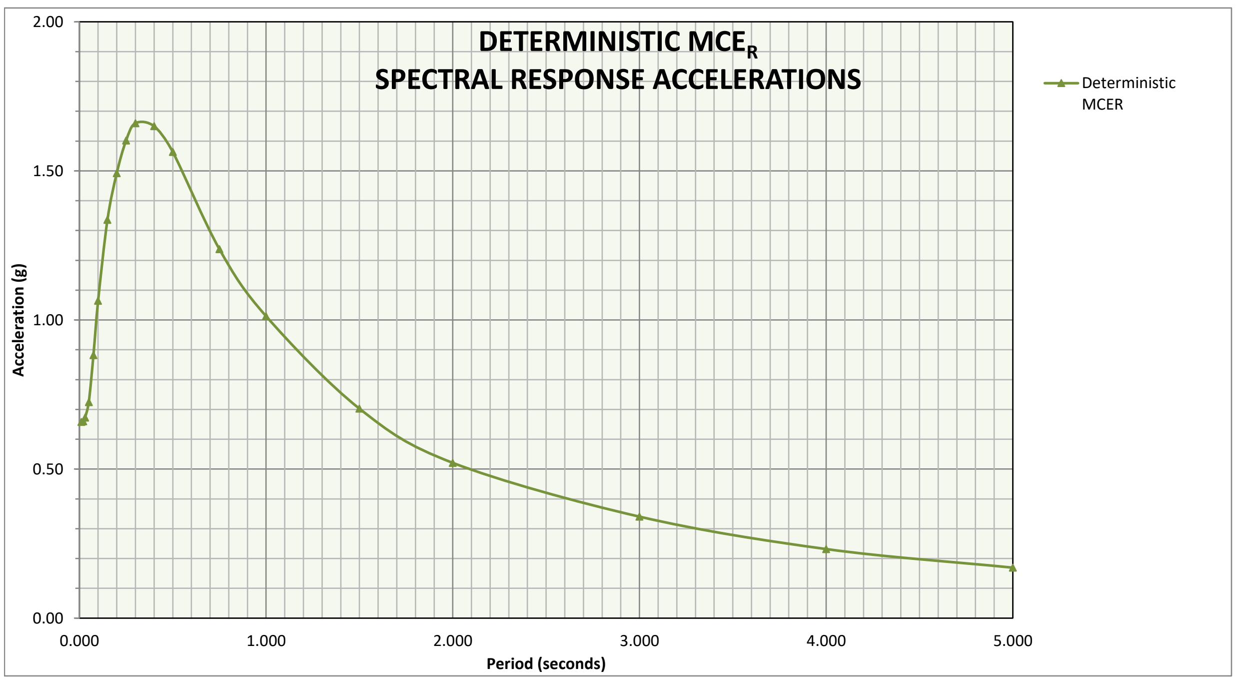
Period	Deterministic PSa Median + 1.σ for 5% Damping	Max Directional Scale Factor[2]	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.553	1.19	0.658	0.658
0.020	0.555	1.19	0.660	0.660
0.030	0.565	1.19	0.673	0.673
0.050	0.608	1.19	0.724	0.724
0.075	0.741	1.19	0.882	0.882
0.100	0.894	1.19	1.064	1.064
0.150	1.113	1.20	1.336	1.336
0.200	1.244	1.20	1.493	1.493
0.250	1.324	1.21	1.602	1.602
0.300	1.361	1.22	1.660	1.660
0.400	1.341	1.23	1.650	1.650
0.500	1.271	1.23	1.564	1.564
0.750	0.998	1.24	1.237	1.237
1.000	0.817	1.24	1.013	1.013
1.500	0.567	1.24	0.703	0.703
2.000	0.420	1.24	0.521	0.521
3.000	0.272	1.25	0.340	0.340
4.000	0.185	1.25	0.232	0.232
5.000	0.134	1.26	0.169	0.169

Project No: 34002.1

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	0.553
Is Deterministic PGA >=F_{PGA}*0.5?	YES

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

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



SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.832	0.658	0.658	0.439
0.100	1.446	1.064	1.064	0.709
0.200	1.957	1.493	1.493	0.995
0.300	2.180	1.660	1.660	1.107
0.500	2.071	1.564	1.564	1.042
0.750	1.681	1.237	1.237	0.825
1.000	1.389	1.013	1.013	0.761
2.000	0.784	0.521	0.521	0.381
3.000	0.544	0.340	0.340	0.254
4.000	0.406	0.232	0.232	0.190
5.000	0.320	0.169	0.169	0.152

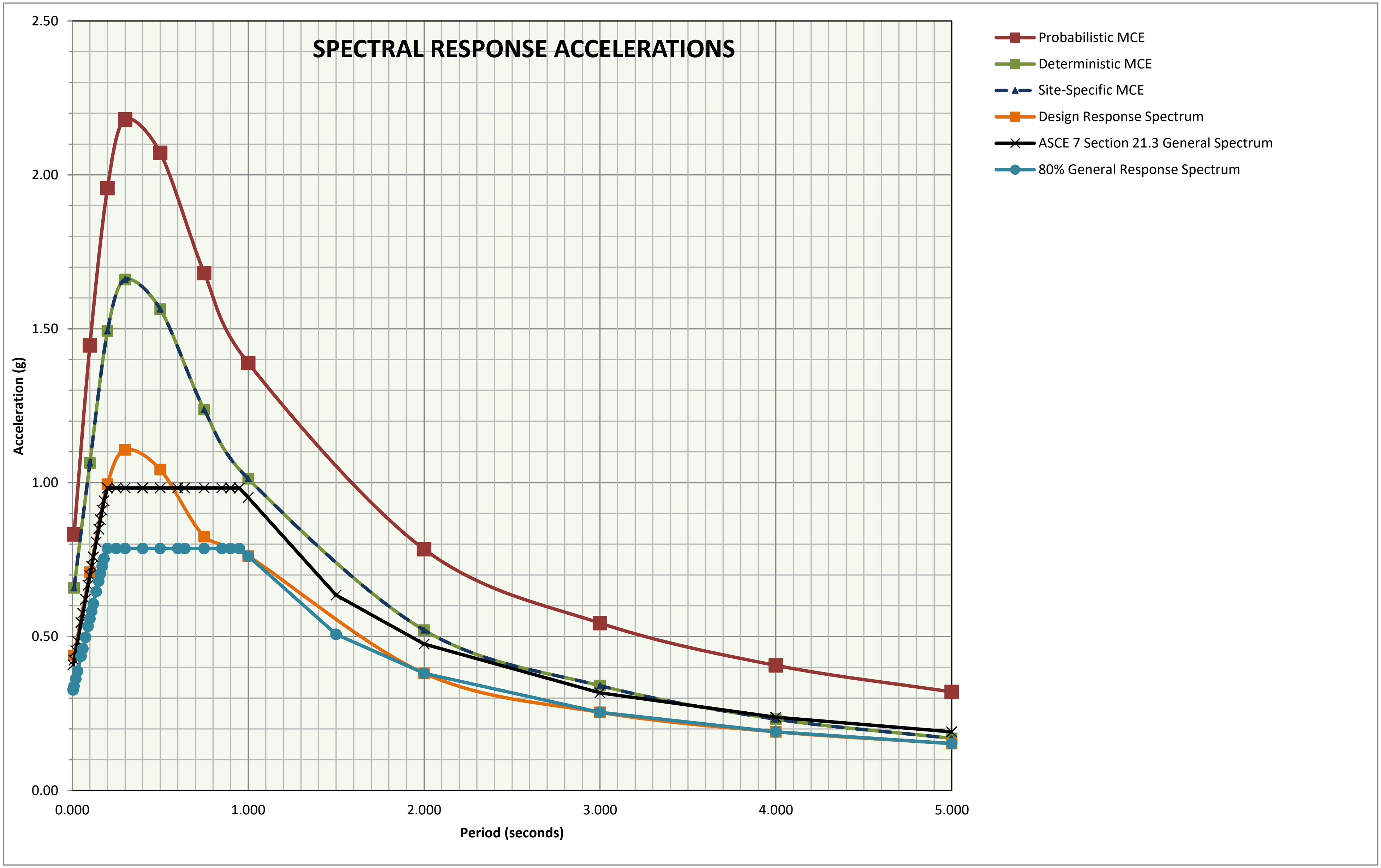
Period	ASCE 7 SECTION 21.3 General Spectrum	80% General Response Spectrum
0.005	0.408	0.327
0.010	0.424	0.339
0.020	0.454	0.363
0.030	0.484	0.388
0.050	0.545	0.436
0.060	0.576	0.461
0.075	0.621	0.497
0.090	0.667	0.534
0.100	0.697	0.558
0.110	0.728	0.582
0.120	0.758	0.607
0.136	0.807	0.646
0.150	0.850	0.680
0.160	0.880	0.704
0.170	0.911	0.728
0.180	0.941	0.753
0.200	0.983	0.786
0.250	0.983	0.786
0.300	0.983	0.786
0.400	0.983	0.786
0.500	0.983	0.786
0.600	0.983	0.786
0.640	0.983	0.786
0.750	0.983	0.786
0.850	0.983	0.786
0.900	0.983	0.786
0.950	0.983	0.786
1.000	0.952	0.761
1.500	0.634	0.508
2.000	0.476	0.381
3.000	0.317	0.254
4.000	0.238	0.190
5.000	0.190	0.152

ASCE 7-16: Section 21.4
Site Specific

	Calculated Value	Design Value
SDS:	0.996	0.996
SD1:	0.761	0.761
SMS:	1.494	1.494
SM1:	1.142	1.142
Site Specific PGAm:	0.553	0.553
Site Class:	D measured	

Seismic Design Category - Short* D
Seismic Design Category - 1s* D
* Risk Categories I, II, or III

Project No: 34002.1



Project No: 34002.1

LOR GEOTECHNICAL GROUP, INC.