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

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT APNs 0410-242-03 AND -04 HESPERIA, CALIFORNIA

PROJECT NO. 33979.1
JANUARY 18, 2024

Prepared For:

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January 18, 2024

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Project No. 33979.1

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Multi-Family Residential Development, APNs 0410-242-03 and -04, 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 and structural improvements, 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, compressible alluvial materials and any 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 of approximately 2 feet will be required within 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 moderate R-value quality content generally characterize the upper onsite materials tested. Near completion and/or at the completion of site grading, additional testing of foundation and subgrade soils should be conducted, as necessary, to verify their expansion potential, soluble sulfate content, and R-value quality.

The results of our field investigation and percolation test data indicate the site earth materials at the depths and locations tested are not conducive to acceptable infiltration. Therefore, water quality storm water systems should not incorporate on-site infiltration when determining storm water treatment capacity.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During January of 2024, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed multi-family residential development within Assessor's Parcel Numbers (APNs) 0410-242-03 and -04 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 1952 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;
- Percolation testing via the borehole test method to determine Infiltration characteristics;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

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

PROJECT CONSIDERATIONS

To orient our investigation at the site, a Site Plan prepared by Steeno Design Studio, Inc., revised dated November 2023, was furnished for our use. The current site conditions, proposed building configurations and associated driveway, parking, and landscape areas were indicated on this plan. The Site Plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A.

As noted on the site plan, development of the site will include eight, two-story apartment buildings, three, single-story apartment buildings, a recreation building, garages, a swimming pool, and associated parking and landscape areas. In addition, infiltration of on-site storm waters is proposed. The buildings are anticipated to be of wood frame and stucco or similar type construction and light to moderate foundation loads are anticipated with these types of structures.

Grading plans have not yet been developed. However, based on the current topography of the site and adjacent areas, very minor cuts and fills are anticipated to create level surfaces for the proposed improvements.

AERIAL PHOTO ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photograph images of varying scales. We reviewed imagery available from Google Earth Pro (2024) computer software and from online Historic Aerials (2024).

To summarize briefly, the existing small residence and detached garage within the northwest portion of the northern parcel of the site were present in the 1952 photograph. By 1959, a small shed was present in the western portion of the southern parcel. The existing small residence and detached garage were present in the 1959 photograph. The site has remained essentially the same since that time and very similar to that seen today. 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 property consists of 4.7± acres of roughly rectangular shaped vacant land, located along the east-southeast side of ‘C’ Avenue approximately 200 feet north-northwest of Lime Street in the city of Hesperia. The topography of the site consists of a very gentle gradient to the east. The site is currently being used as two single family residential lots with a total of two single family residences, two detached garages, several small sheds, and other miscellaneous stored items such as trailers and cars. The majority of the existing improvements are limited to the western half of the site. The eastern half of the site is vacant. Large desert brush is present scattered throughout the site.

Vacant land is present adjacent to the site on the east. ‘C’ Avenue, a paved roadway, is present along the east-northeast of the site with multi-family residential beyond. Large lot residential properties, similar to the site, lie adjacent the site on the north-northeast and south-southwest.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on January 4, 2024. The work consisted of advancing a total of 6 exploratory borings using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. In addition, four borehole percolation tests were conducted in general accordance with the Shallow 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.

The subsurface conditions encountered in the exploratory borings were logged by a licensed geologist from this firm. The borings were drilled to depths ranging from approximately 20.5 and 51.5 feet below the existing ground surface. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet, and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

Percolation test borings were drilled to the requested depths of approximately 5 feet below the existing ground surface at the requested locations and tested on January 4, 2024.

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 and sandy clay. These fans lie on a very thick sequence of terrestrial sedimentary rocks, which in turn overlie crystalline bedrock (Dibblee, 1960 and 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 active fault to the subject site noted in the documents reviewed during our study is the North Frontal fault located approximately 7.7 kilometers (4.8 miles) southeast of the site. A complete listing of the distances to known active faults in relation to the site is given in the Faulting section of this report.

The site and the regional geologic setting are shown on Enclosure A-3 within Appendix A.

Site Geologic Conditions

As observed and encountered during this investigation, the subject site generally contains a relatively thin veneer of fill soils locally, overlying alluvial materials. These units are described in further detail in the following sections:

Fill: Although fill materials were not encountered within any of our exploratory borings, minor amounts of fill soils were noted locally. These materials were generally on the order of less than one foot in thickness and consisted of locally derived silty sand soils. The fill materials are considered to be non-engineered fill.

Alluvium: Alluvial materials were encountered within all of our exploratory borings to the maximum depths explored. These units were noted to mainly consist of silty sand to well graded sand. These materials were typically red-brown to tan in color. The alluvial materials were in a medium dense to 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.

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 within any of our exploratory borings as advanced to a maximum depth of approximately 51.5 feet below the existing ground surface.

Local groundwater level measurements were researched at the California Department of Water Resources (CDWR) online Water Data Library (CDWR, 2024). The closest groundwater well found in this search was State Well Number 04N04W28H001S located approximately 0.25 kilometers (0.15 miles) to the east of the site. This well has groundwater measurements available from 2012 back to 1998 and ranged from approximately 420 to 451 feet below the existing ground surface elevation of approximately 3,238 feet above mean sea level.

Based on this information and findings from our borings, the depth to groundwater beneath the subject property is greater than 400 feet.

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, 2003) nor does the site lie within a County of San Bernardino fault zone (San Bernardino County, 2023).

As previously mentioned, the closest known active fault is the North Frontal fault, located approximately 7.7 kilometers (4.8 miles) to the southeast. In addition, other relatively close active faults include the Cleghorn fault located approximately 9.6 kilometers (6.0 miles) to the south, the San Andreas fault located approximately 20.6 kilometers (12.8 miles) to the southwest, and the Helendale fault located approximately 25.8 kilometers (16.1 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 24mm/yr and capable of generating large magnitude events on the order of 7.5.

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

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. (2022). 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 January 16, 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-4, 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.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.2 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-5, a few events are present in the area associated with the North Frontal Fault.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring in the region around the subject site. 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) 34.4065, -117.2987, Risk Category II	
Site Class Definition Chapter 20 ASCE 7	D
S_s Mapped Spectral Response Acceleration at 0.2s Period	1.422
S_1 Mapped Spectral Response Acceleration at 1s Period	0.557
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period	1.870
S_{M1} Adjusted Spectral Response Acceleration at 1s Period	1.432
S_{DS} Design Spectral Response Acceleration at 0.2s Period	1.247
S_{D1} Design Spectral Response Acceleration at 1s Period	0.955
F_a Short Period Site Coefficient at 0.2s Period	1.0
F_v Long Period Site Coefficient at 1s Period	1.7
PGA_M Site Modified Peak Ground Acceleration	0.709

CBC 2022/ASCE 7-16 SEISMIC DESIGN SUMMARY* Site Location (USGS WGS84) 34.4065, -117.2987, Risk Category II	
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 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.

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 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 to 8.6, neutral to slightly alkaline. 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 indicates the soils are 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, at the approximate locations and depth of proposed retention basins have poor infiltration rates ranging from approximately 0.07 to 0.21 inches per hour.

Geologic Mitigations

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

Seismicity

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

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

No secondary seismic hazards are anticipated to impact the proposed development.

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. Any undocumented fill encountered during grading should be completely removed, cleaned of significant deleterious materials and may then be reused as compacted fill. Undocumented fill is anticipated locally, primarily in the currently developed areas of the site. 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.

Cavities created by the removal of any subsurface obstructions that could be encountered, such as foundations, utilities, and septic systems associated with the current on-site development, should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

Initial Site Preparation

The existing loose alluvial soils and any 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 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. In addition, the removal depth should also identify features associated with buried obstructions associated with the past/current land use, requiring deeper removals. 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.

It should be noted that no structure should be placed across any areas where the maximum depth of fill to minimum depth of fill is greater than a 3 to 1 ratio as measured from the bottom of the footing.

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 1,500 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 2,200 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	25	0.25' AC / 0.50' AB or 4.5" PCC / 4.0" AB
On site vehicular parking with occasional truck traffic (ADTT=10)	6.0	25	0.25' AC / 0.80' AB or 5.0" PCC / 4.0" AB
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.

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 4.5 inch thick concrete, with a 4.0 inch thick aggregate base. Concrete pads are also recommended in areas adjacent to trash storage areas where heavier loads will occur due to operation of trucks lifting trash dumpsters.

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.

Infiltration

The results of our field investigation and percolation test data indicate that the site earth materials at the depth and locations tested are not conducive to acceptable infiltration. Therefore, water quality storm water systems should not incorporate on-site infiltration when determining storm water treatment capacity.

Corrosion Protection

Based on the test results, this soil is classified as 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 Mr. Andrew Taylor, and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance.

The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. If conditions are encountered during the construction of the project, which differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

If parties other than LOR Geotechnical Group, Inc., provide construction monitoring services, they must be notified that they will be required to assume responsibility for the geotechnical phase of the project being completed by concurring with the recommendations provided in this report or by providing alternative recommendations.

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

TIME LIMITATIONS

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

CLOSURE

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

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie, CEG 2794
Vice President

CERTIFIED ENGINEERING GEOLOGIST
ANDREW A. TARDIE
No. 2794
STATE OF CALIFORNIA

John P. Leuer, GE 2030
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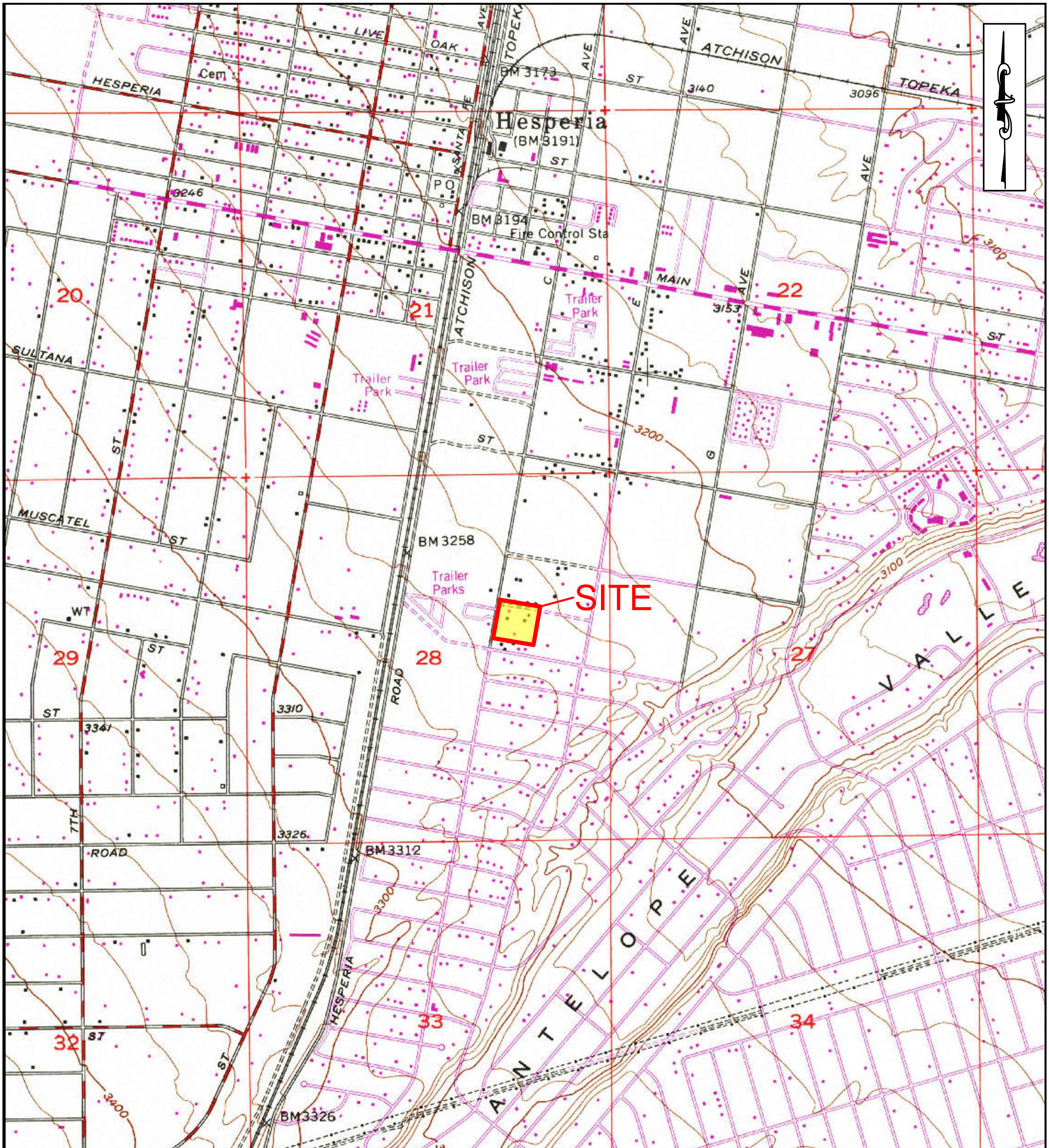
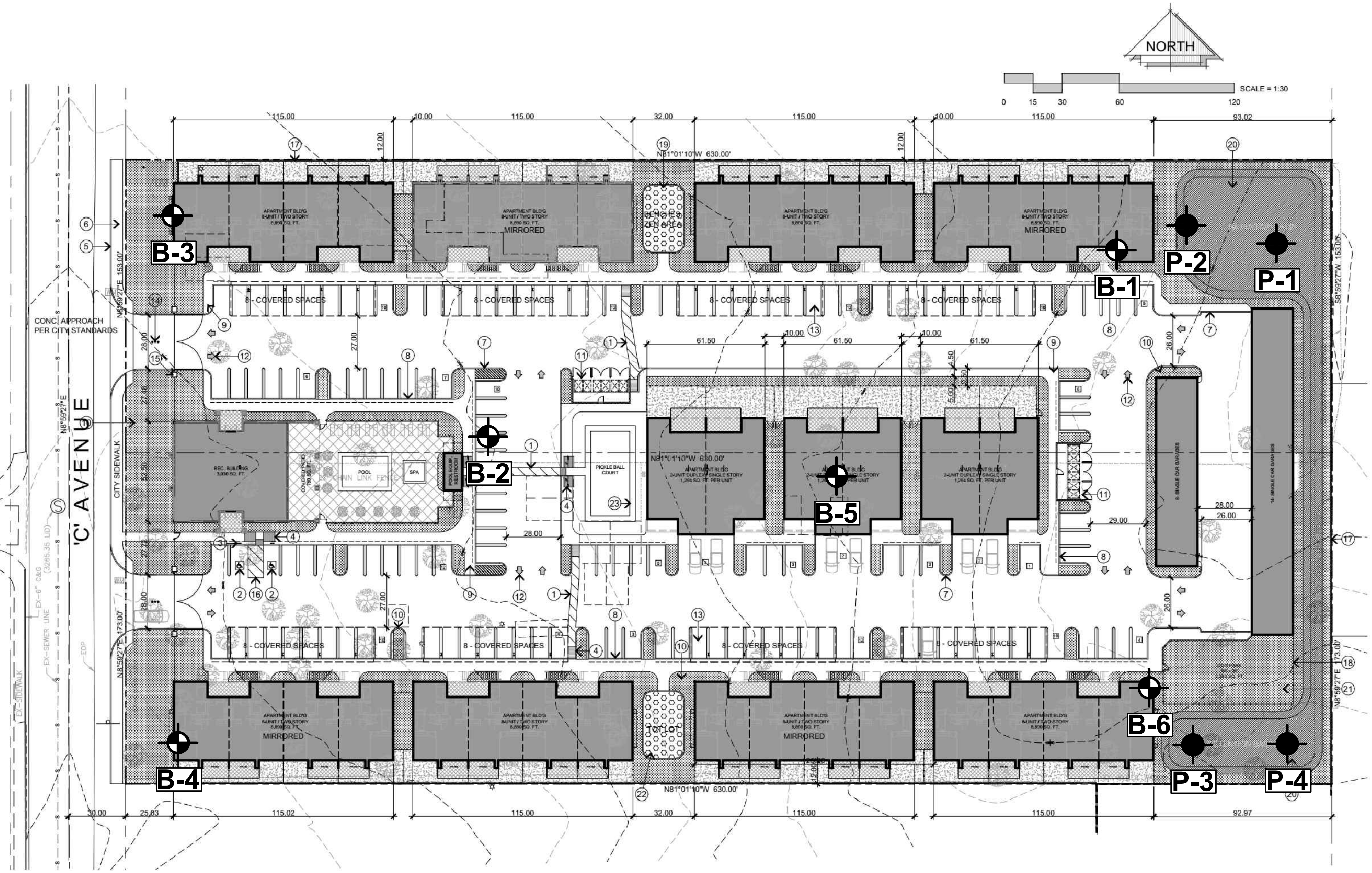
APPENDIX A

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



INDEX MAP

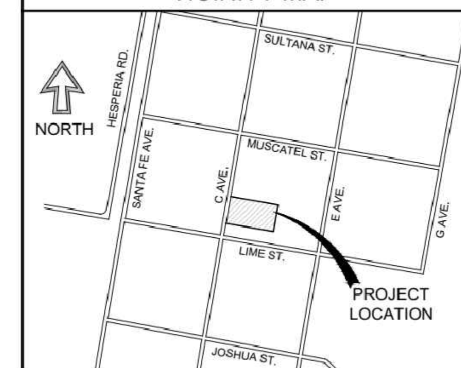
PROJECT:	APNs 0410-242-03 and -04, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	January 2024
		SCALE:	1" ≈ 2,000'



SITE PLAN

SCALE: 1:30

VICINITY MAP



HESPERIA UTILITIES

WATER (PROPOSED): HESPERIA WATER DISTRICT 9393 SANTA FE AVE. HESPERIA, CA 92345 (760) 947-1400	ELECTRIC (PROPOSED): SOUTHERN CALIFORNIA EDISON CO. 12353 HESPERIA RD. VICTORVILLE, CA 92392 (760) 951-3237
TELEPHONE (PROPOSED): VERIZON 15168 LA PAZ ROAD VICTORVILLE, CA 92392 (800) 772-5153	DISPOSAL: ADVANCED DISPOSAL INC. 17105 MESA RD. HESPERIA, CA 92345
GAS (PROPOSED): SOUTHWEST GAS CO. 13471 MARIPOSA ROAD VICTORVILLE CA. 92392 (760) 951-4055	SEWER HESPERIA WATER DISTRICT 9393 SANTA FE AVE. HESPERIA, CA 92345 (760) 947-1400

BUILDING DATA

OCTOPLEX BUILDINGS WITH 8 UNITS		= 8
DUPLEX BUILDINGS WITH 2 UNITS		= 3
GARAGE BUILDINGS WITH (8) NON-LIVABLE UNITS		= 2
RECREATIONAL BUILDING		= 1
TOTAL # OF BUILDINGS		= 14
TOTAL # OF LIVABLE UNITS	(8 BLDG. X 8) =	64
	(3 BLDG. X 2) =	6
TOTAL PROP. UNITS		70
BUILDING AREA (PER TWO STORY BUILDING):		
LOWER LEVEL SQ. FT. (4 UNITS PER LEVEL)		
LIVABLE SPACE (PER UNIT)		1,114.5 SF.
OUTDOOR SPACE (PER UNIT)		227.5 SF.
TOTAL		5,368 SF.
UPPER LEVEL SQ. FT. (4 UNITS PER LEVEL)		
LIVABLE SPACE (PER UNIT)		1,114.5 SF.
OUTDOOR SPACE (PER UNIT)		204 SF.
TOTAL		5,274 SF.
TOTAL LIVABLE AREA (PER TWO STORY BUILDING)		8,916 SF.
TOTAL BUILDING AREA (PER TWO STORY BUILDING)		10,642 SF.
BUILDING AREA (PER SINGLE STORY DUPLEX):		
LIVABLE SPACE (PER UNIT)		1,294 SF.
OUTDOOR SPACE (PER UNIT)		323.5 SF.
GARAGE SPACE (PER UNIT)		322 SF.
TOTAL LIVABLE AREA (PER SINGLE STORY DUPLEX)		2,588 SF.
TOTAL BUILDING AREA (PER SINGLE STORY DUPLEX)		3,879 SF.
BUILDING AREA (PER ACCESSORY STORAGE BUILDING):		
TOTAL GARAGE SPACE PER BUILDING		4,557 SF.
BUILDING AREA (PER RECREATION BUILDING):		
RECREATIONAL NON-LIVABLE SPACE		3,030 SF.
OUTDOOR SPACE		780 SF.
TOTAL RECREATIONAL BUILDING AREA		3,810 SF.
PROJECT DENSITY:		
LAND AREA		4.70 ACRES
TWO STORY UNITS		64 UNITS
SINGLE STORY DUPLEX UNITS		6 UNITS
TOTAL UNITS		70 UNITS
UNITS PER ACRE		14.89 UNITS PER ACRE

KEYED NOTES

1. 48" WIDE ACCESSIBILITY ACCESS W/ BLUE STRIPES
2. ALL ACCESSIBLE PARKING SPACES ARE ALSO VAN ACCESSIBLE, TYP. ACCESSIBILITY STALL EMBLEM PAINTED AS SHOWN
3. WARNING SIGNAGE REGARDING UNAUTHORIZED USE OF DISABLED PARKING SPACES
4. A.D.A RAMP NOT TO EXCEED 5% SLOPE IN DIRECTION OF RUN AND 2% MAX CROSS SLOPE TRUNCATED DOMES, 36" DEPT x WIDTH OF RAMP
5. PROPOSED CONC. CURB & GUTTER PER CITY/COUNTY STANDARDS
6. PROPOSED X' WD. CONC. SIDEWALK PER CITY/COUNTY STANDARDS
7. 6" WIDE CONCRETE CURB.
8. 6" WIDE CONCRETE CURB. W/ 2' PARKING NOSE OVER
9. 4" NOMINAL THICK CONCRETE WALKWAYS
10. LANDSCAPED AREA SEE LANDSCAPE PLANS
11. 6'-0" HIGH TRASH ENCLOSURE W/ 6' X 24' LOADING ZONE IN FRONT.PER CITY STANDARDS & 8" THK. REINFORCED CONCRETE PAD W/ 2% MAXIMUM SLOPE AWAY FROM T/E
12. TRAFFIC FLOW DIRECTIONAL ARROWS PAINTED ON PAVING AS SHOWN ON PLAN
13. COVERED PARKING (CARPORT)
14. GATE CONTROL STATION
15. 20'-0" WIDE DOUBLE WROUGHT IRON SWING ENTRY GATE
16. 8'-0" WIDE ACCESSIBILITY ACCESS W/ BLUE STRIPES
17. 6' HI. MASONRY WALL
18. 6' HI. WROUGHT IRON FENCING
19. BENCHES ZEN AREA
20. RETENTION BASIN
21. DOG PARK
22. TOT LOT
23. PICKLE BALL COURT

SHEET INDEX

A-0	SITE PLAN

PROJECT DATA

ZONE:	MDR
OCCUPANCY'S:	B
CONSTRUCTION TYPE:	V - B
STORIES:	2
A.P.N.	0410-242-03 & 04

APPLICANT / OWNER:
ANDREW TAYLOR
8561 'C' AVE.
HESPERIA, CA. 92345

ARCHITECT / REPRESENTATIVE:
STEENO DESIGN STUDIO, INC.
ARCHITECT: TOM STEENO
11774 HESPERIA RD. SUITE B-1
PH: 760.244.5001 FX: 760.244.1948

SCOPE OF WORK

TO OBTAIN PERMITS FOR A PROPOSED 8 - (2) STORY MULTI-TENANT DWELLING. 8 UNITS PER BUILDING (2 BED-2 BATH), 3 - (1) STORY MULTI-TENANT DUPLEX. (3 BED-2 BATH) & 2 - (1) STORY ACCESSORY STORAGE BUILDINGS (3 RV UNITS - 5 STANDARD UNITS)

SITE DATA

LEGAL DESCRIPTION:
03 - HESPERIA N 1/2 S 1/2 LOT B BLK 116 2.50 AC02 - TR NO 3213 LOT 8 EX STATE HGWY
04 - HESPERIA S 1/2 N 1/2 LOT B BLK 116 EX N 20 FT 2.3

SITE AREA

AREA
NET LAND AREA (4.70 ACRE)

PROPOSED LAND AREA & COVERAGE

AREA
TOTAL BUILDING FOOTPRINT
A/C PAVING
CONCRETE HARDSCAPE & CURBING
LANDSCAPED AREA
TOTAL NET LAND AREA COVERAGE

PARKING DATA

REQUIRED PARKING:

2 BEDROOM UNITS	64 X 2.25 SPACES PER UNIT = 144 TOTAL
3 BEDROOM UNITS (DUPLEX)	6 X 2.25 SPACES PER UNIT = 14 TOTAL

Legend

(Locations Approximate)

Map Symbols

- B-6 - Exploratory Boring
- P-4 - Percolation Test

SITE PLAN

PROJECT: APNs 0410-242-03 and -04, Hesperia, California

CLIENT: Mr. Andrew Taylor

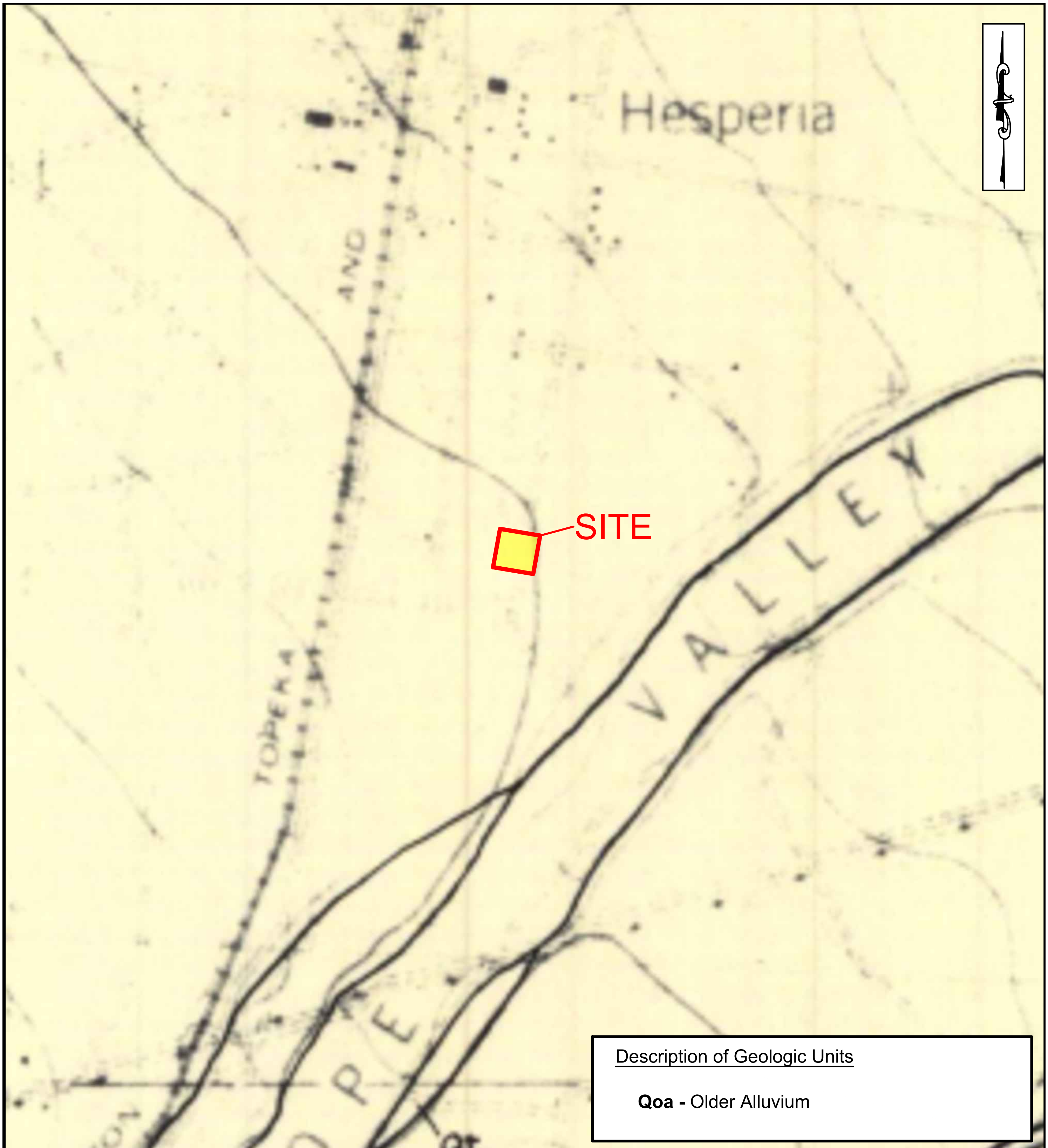
LOR GEOTECHNICAL GROUP, INC.

PROJECT NO.: 33979.1

ENCLOSURE: A-2

DATE: January 2024

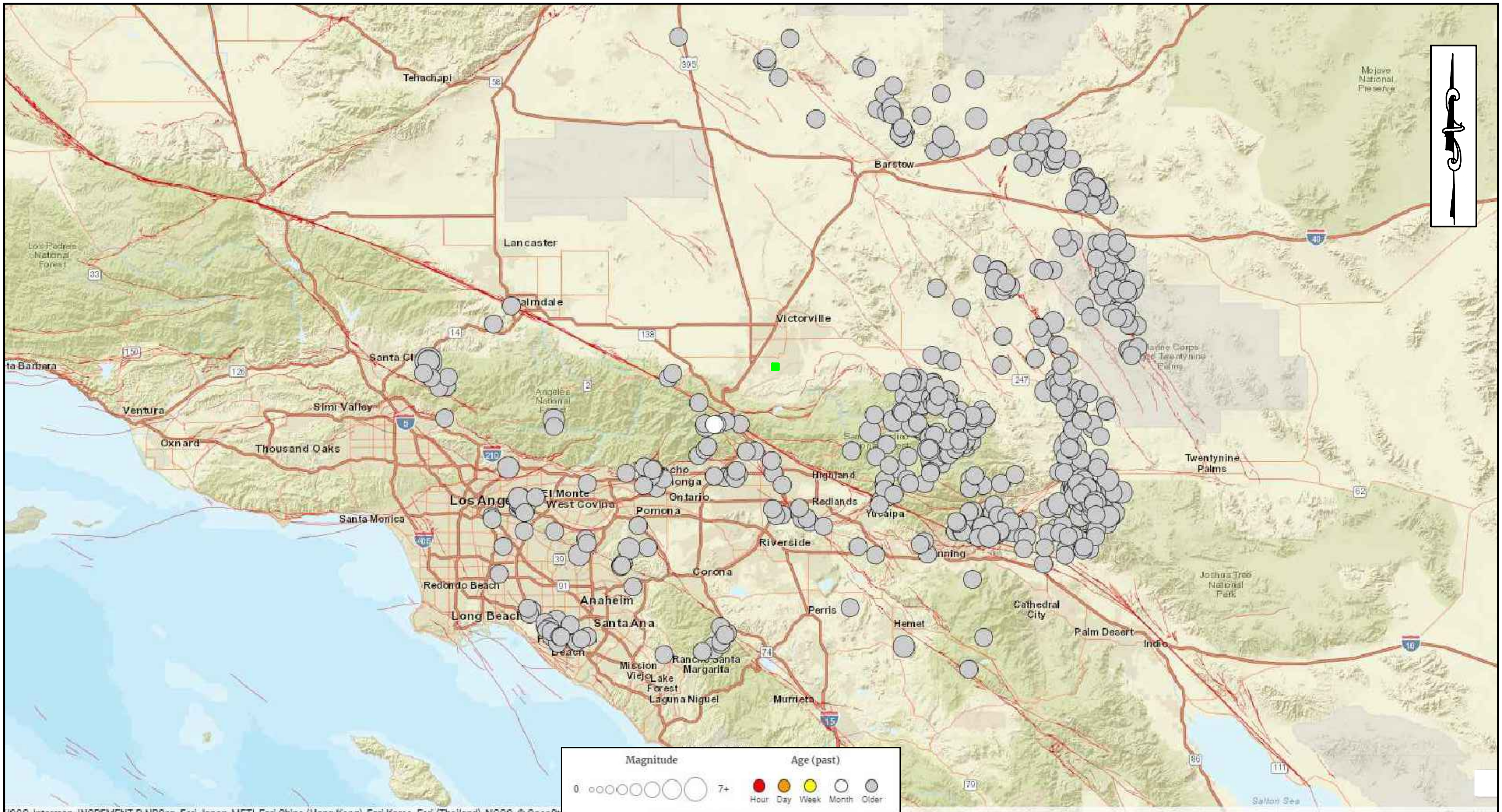
SCALE: 1" ≈ 65'



REGIONAL GEOLOGIC MAP

(Dibblee, 1965)

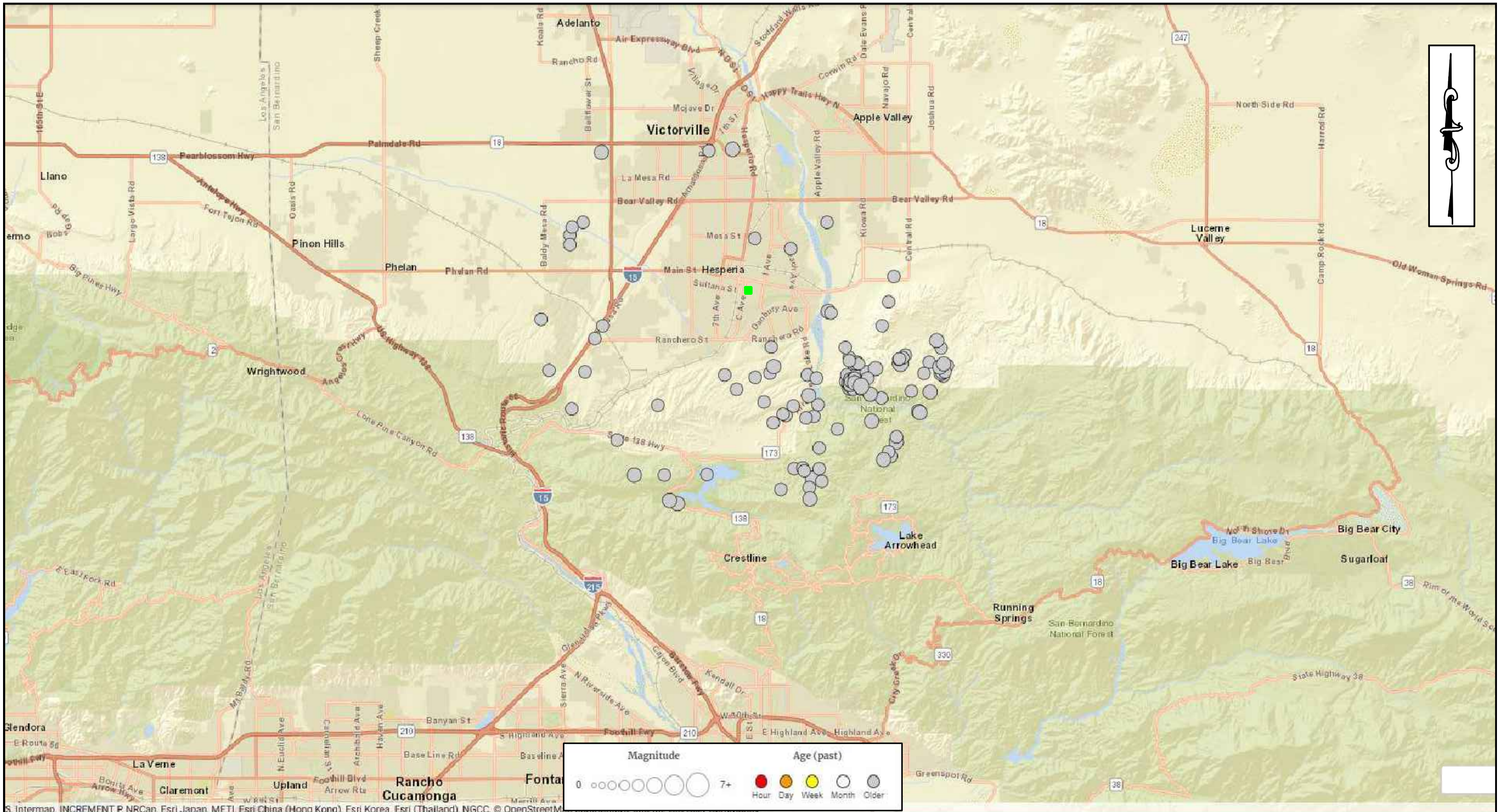
PROJECT:	APNs 0410-242-03 and -04, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	January 2024
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2024) real-time earthquake epicenter map. Plotted are 532 epicenters of instrument-recorded events from 01/01/32 to present (01/16/24) 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:	APNs 0410-242-03 and -04, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	January 2024
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2024) real-time earthquake epicenter map. Plotted are 119 epicenters of instrument-recorded events from 01/01/78 to present (01/16/24) 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:	APNs 0410-242-03 and -04, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	January 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 ranging from approximately 20.5 and 51.5 feet below the existing ground surface using a Mobile B-61 drill rig on January 4, 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:	Proposed Residential Development, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	B-i
		DATE:	January 2024

LOR GEOTECHNICAL GROUP, INC.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO.200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" No. 4 No. 10 No. 40 200

(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	Proposed Residential Development, Hesperia, California	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	January 2024

LOG OF BORING B-1

DEPTH IN FEET	TEST DATA: SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		3, 4, 7, 8, 9, 10						SM	@ 0 feet, ALLUVIUM: **SILTY SAND, approximately 10% gravel to 3/4", 10% coarse grained sand, 35% medium grained sand, 20% fine grained sand, 25% silty fines, light red brown, damp.**
									@ 0.5 feet, becomes dry.
	57 for 11"		**3.9**		**121.6**				**@ 2 feet, contains trace gravel and clay, slightly coarser grained sand portion.**
5	**60**		**6.3**		**115.4**				**@ 5 feet, SILTY SAND, approximately 15% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 15% silty fines, red brown, damp.**
10	**68**		**3.9**		**113.9**			SW SM	**@ 10 feet, WELL GRADED SAND with GRAVEL and SILT, approximately 15% gravel to 3/4", 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 10% silty fines, light red brown, dry.**
15	**63**		**4.2**		**119.8**			SW	**@ 15 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, tan, dry, weakly cemented.**
20	**63**		**3.7**		**104.6**				
									END OF BORING @ 21.5' **No fill** **No groundwater** **No bedrock**

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,256
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	January 4, 2024
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-1

LOG OF BORING B-2

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	U.S.C.S.	DESCRIPTION
0	40 for 5"					SM	@ 0 feet, ALLUVIUM: **SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 30% silty fines, light red brown, damp.** **0.5 feet, becomes dry.** **@ 2 feet, SILTY SAND with GRAVEL, approximately 15% gravel to 1", 20% coarse grained sand, 20% medium grained sand, 25% fine grained sand, 20% silty fines, light red brown, dry, no recovery.**
5	64		4.9				**@ 5 feet, rings disturbed.**
10	39		3.1		111.4		
15	46 for 5"		1.7				**@ 15 feet cobble in tip of sampler, rings disturbed.**
20	62		2.7		110.2	SW	**@ 20 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", approximately 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines.**
25	50		2.7				
30	55		2.6				
35	61		2.9				
40	55		3.1				
45	83		2.8				
50	54		3.0				**END OF BORING @ 51.5'** **No fill** **No groundwater** **No bedrock**
55							

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,262
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	January 4, 2024
		EQUIPMENT:	Mobile 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, <u>ALLUVIUM:</u> **SILTY SAND, approximately 5% gravel to 1/2", 5% coarse grained sand, 35% medium grained sand, 30% fine grained sand, 25% silty fines, light red brown, damp.**
									@ 0.5 feet, becomes dry.
	71		6.8		125.4				**@ 2 feet, SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 30% silty fines, light red brown, dry.**
5	62		5.3		119.0				**@ 5 feet, SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines, red brown, dry.**
10	46 for 4"		6.9		106.1				
15	46 for 4"								**@ 15 feet, some cobbles ?, no recovery, rig chatter.**
20	51 for 6"								**@ 20 feet, no recovery.**
									END OF BORING @ 20.5 **No fill** **No groundwater** **No bedrock**

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,263
		DATE DRILLED:	January 4, 2024
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-3

LOG OF BORING B-4

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 20% silty fines, light red brown, damp.**
									@ 0.5 feet, becomes dry.
	65		3.5		122.9				**@ 2 feet, trace gravel to 1/2".**
5	54		5.9		113.5				**@ 5 feet, SILTY SAND, approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines, light red brown, dry.**
10	38		3.9		113.0				**@ 10 feet, SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light red brown, dry.**
15	47		3.8		104.8				
20	49		2.1		110.1			SW	**@ 25 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, tan, dry.**
25	62		1.8		105.2				
									END OF BORING @ 26.5' **No fill** **No groundwater** **No bedrock**
30									

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,265
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	January 4, 2024
		EQUIPMENT:	Mobile 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)	U.S.C.S.	DESCRIPTION
0		9, 10				SM	@ 0 feet, ALLUVIUM: **SILTY SAND, approximately 10 gravel to 1/2", 10% coarse grained sand, 35% medium grained sand, 25% fine grained sand, 20% silty fines, brown, damp.**
							@ 0.5 feet, becomes dry.
	72 for 11"		6.1		126.3		**@ 2 feet, SILTY SAND, trace gravel to 1/2", approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% silty fines with trace clay, red brown, dry.**
5	56		6.2		119.6		**@ 5 feet, slightly finer grained sand, no clay.**
10	68		3.6		115.9	SW	**@ 10 feet, WELL GRADED SAND, approximately 5% gravel to 1", 25% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 5% silty fines, tan, dry.**
15	56		2.5		110.9		
20	55		1.4				**@ 20 feet, rings disturbed.**
							END OF BORING @ 21.5'
							No fill **No groundwater** **No bedrock**

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,258
		DATE DRILLED:	January 4, 2024
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile 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, <u>ALLUVIUM:</u> **SILTY SAND, trace gravel to 1/2", approximately 20% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 20% silty fines, light red brown, damp.**
	23		**3.4**		**114.4**				**@ 0.5 feet, becomes dry.**
5	**28**		**2.9**		**116.7**				**@ 5 feet, becomes slightly coarser grained.**
10	**36**		**3.2**		**111.7**				**@ 10 feet, SILTY SAND, approximately 10% gravel to 1/2", 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 15% silty fines, light red brown, dry.**
15	**62**		**4.6**		**113.6**				
20	**91 for 11"**		**3.2**		**110.9**				
									END OF BORING @ 21.5' **No fill** **No groundwater** **No bedrock**

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**ELEVATION:**	3,256
		DATE DRILLED:	January 4, 2024
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile 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 Shallow 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, January 5, 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 30-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 12 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	5.0	0.07
P-2	5.0	0.21
P-3	5.0	0.14
P-4	5.0	0.18

* 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:	APNs 0410-242-03 and -04, Hesperia, California	Test Date:	January 5, 2024
Project No.:	33979.1	Test Hole No.:	P-1
Soil Classificaiton:	(SM) Silty sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	5.0 ft.	Date Excavated:	January 4, 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.	(min/in)
1	10:10 AM	10:40 AM	30	0.50	0.50	22.00	24.00	60.00	60.00	2.00	37.00	15.0
2	10:41 AM	11:11 AM	30	0.50	1.00	24.00	26.00	60.00	60.00	2.00	35.00	15.0
3	11:12 AM	11:42 AM	30	0.50	1.50	26.00	28.00	60.00	60.00	2.00	33.00	15.0
4	11:43 AM	12:13 PM	30	0.50	2.00	28.00	29.75	60.00	60.00	1.75	31.13	17.1
5	12:14 PM	12:44 PM	30	0.50	2.50	24.00	26.00	60.00	60.00	2.00	35.00	15.0
6	12:45 PM	1:15 PM	30	0.50	3.00	26.00	27.75	60.00	60.00	1.75	33.13	17.1
7	1:16 PM	1:46 PM	30	0.50	3.50	27.75	29.50	60.00	60.00	1.75	31.38	17.1
8	1:47 PM	2:17 PM	30	0.50	4.00	24.00	25.75	60.00	60.00	1.75	35.13	17.1
9	2:18 PM	2:48 PM	30	0.50	4.50	25.75	27.75	60.00	60.00	2.00	33.25	15.0
10	2:49 PM	3:19 PM	30	0.50	5.00	27.75	29.50	60.00	60.00	1.75	31.38	17.1
11	3:20 PM	3:50 PM	30	0.50	5.50	24.00	25.75	60.00	60.00	1.75	35.13	17.1
12	3:51 PM	4:21 PM	30	0.50	6.00	25.75	27.50	60.00	60.00	1.75	33.38	17.1

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	34.25	
H_f	32.50	
ΔH*	1.05	
H_{avg}	33.38	
I_t	**0.07**	in/hr (clear water rate)

* adjusted due to the loss in volume of water because of gravel packing

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APNs 0410-242-03 and -04, Hesperia, California	Test Date:	January 5, 2024
Project No.:	33979.1	Test Hole No.:	P-2
Soil Classificaiton:	(SM) Silty sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	5.0 ft.	Date Excavated:	January 4, 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.	(min/in)
1	10:13 AM	10:43 AM	30	0.50	0.50	24.00	30.00	60.00	60.00	6.00	33.00	5.0
2	10:43 AM	11:13 AM	30	0.50	1.00	21.00	27.00	60.00	60.00	6.00	36.00	5.0
3	11:13 AM	11:43 AM	30	0.50	1.50	24.00	29.50	60.00	60.00	5.50	33.25	5.5
4	11:43 AM	12:13 PM	30	0.50	2.00	24.00	29.50	60.00	60.00	5.50	33.25	5.5
5	12:13 PM	12:43 PM	30	0.50	2.50	24.00	29.25	60.00	60.00	5.25	33.38	5.7
6	12:43 PM	1:13 PM	30	0.50	3.00	24.00	29.00	60.00	60.00	5.00	33.50	6.0
7	1:13 PM	1:43 PM	30	0.50	3.50	24.00	29.00	60.00	60.00	5.00	33.50	6.0
8	1:43 PM	2:13 PM	30	0.50	4.00	24.00	29.00	60.00	60.00	5.00	33.50	6.0
9	2:13 PM	2:43 PM	30	0.50	4.50	24.00	29.00	60.00	60.00	5.00	33.50	6.0
10	2:43 PM	3:13 PM	30	0.50	5.00	24.00	29.00	60.00	60.00	5.00	33.50	6.0
11	3:13 PM	3:43 PM	30	0.50	5.50	24.00	29.00	60.00	60.00	5.00	33.50	6.0
12	3:43 PM	4:13 PM	30	0.50	6.00	24.00	29.00	60.00	60.00	5.00	33.50	6.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	36.00	
H_f	31.00	
ΔH*	3.00	
H_{avg}	33.50	
I_t	**0.21**	in/hr (clear water rate)

* adjusted due to the loss in volume of water because of gravel packing

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APNs 0410-242-03 and -04, Hesperia, California	Test Date:	January 4, 2024
Project No.:	33979.1	Test Hole No.:	P-3
Soil Classificaiton:	(SM) Silty sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	5.0 ft.	Date Excavated:	January 5, 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.	(min/in)
1	10:22 AM	10:52 AM	30	0.50	0.50	30.00	34.25	60.00	60.00	4.25	27.88	7.1
2	10:53 AM	11:23 AM	30	0.50	1.00	30.00	33.50	60.00	60.00	3.50	28.25	8.6
3	11:24 AM	11:54 AM	30	0.50	1.50	24.00	27.75	60.00	60.00	3.75	34.13	8.0
4	11:55 AM	12:25 PM	30	0.50	2.00	27.75	30.25	60.00	60.00	2.50	31.00	12.0
5	12:26 PM	12:56 PM	30	0.50	2.50	24.00	27.50	60.00	60.00	3.50	34.25	8.6
6	12:57 PM	1:27 PM	30	0.50	3.00	27.50	30.50	60.00	60.00	3.00	31.00	10.0
7	1:28 PM	1:58 PM	30	0.50	3.50	24.00	27.75	60.00	60.00	3.75	34.13	8.0
8	1:59 PM	2:29 PM	30	0.50	4.00	27.75	30.75	60.00	60.00	3.00	30.75	10.0
9	2:30 PM	3:00 PM	30	0.50	4.50	24.00	27.50	60.00	60.00	3.50	34.25	8.6
10	3:01 PM	3:31 PM	30	0.50	5.00	27.50	30.75	60.00	60.00	3.25	30.88	9.2
11	3:32 PM	4:02 PM	30	0.50	5.50	24.00	27.50	60.00	60.00	3.50	34.25	8.6
12	4:03 PM	4:33 PM	30	0.50	6.00	27.25	30.50	60.00	60.00	3.25	31.13	9.2

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	32.75	
H_f	29.50	
ΔH*	1.95	
H_{avg}	31.13	
I_t	**0.14**	in/hr (clear water rate)

* adjusted due to the loss in volume of water because of gravel packing

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APNs 0410-242-03 and -04, Hesperia, California	Test Date:	January 4, 2024
Project No.:	33979.1	Test Hole No.:	P-4
Soil Classificaiton:	(SM) Silty sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	5.0 ft.	Date Excavated:	January 5, 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.	(min/in)
1	10:26 AM	10:56 AM	30	0.50	0.50	26.00	30.50	60.00	60.00	4.50	31.75	6.7
2	10:57 AM	11:27 AM	30	0.50	1.00	24.00	29.00	60.00	60.00	5.00	33.50	6.0
3	11:28 AM	11:58 AM	30	0.50	1.50	24.00	28.25	60.00	60.00	4.25	33.88	7.1
4	11:59 AM	12:29 PM	30	0.50	2.00	28.25	32.25	60.00	60.00	4.00	29.75	7.5
5	12:30 PM	1:00 PM	30	0.50	2.50	24.00	28.50	60.00	60.00	4.50	33.75	6.7
6	1:01 PM	1:31 PM	30	0.50	3.00	28.50	32.25	60.00	60.00	3.75	29.63	8.0
7	1:32 PM	2:02 PM	30	0.50	3.50	24.00	28.25	60.00	60.00	4.25	33.88	7.1
8	2:03 PM	2:33 PM	30	0.50	4.00	28.25	32.00	60.00	60.00	3.75	29.88	8.0
9	2:34 PM	3:04 PM	30	0.50	4.50	24.00	28.00	60.00	60.00	4.00	34.00	7.5
10	3:05 PM	3:35 PM	30	0.50	5.00	28.00	31.75	60.00	60.00	3.75	30.13	8.0
11	3:36 PM	4:06 PM	30	0.50	5.50	24.00	28.00	60.00	60.00	4.00	34.00	7.5
12	4:07 PM	4:37 PM	30	0.50	6.00	28.00	32.00	60.00	60.00	4.00	30.00	7.5

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	32.00	
H_f	28.00	
ΔH*	2.40	
H_{avg}	30.00	
I_t	**0.18**	in/hr (clear water rate)

* adjusted due to the loss in volume of water because of gravel packing

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. 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	132.0	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	0	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.

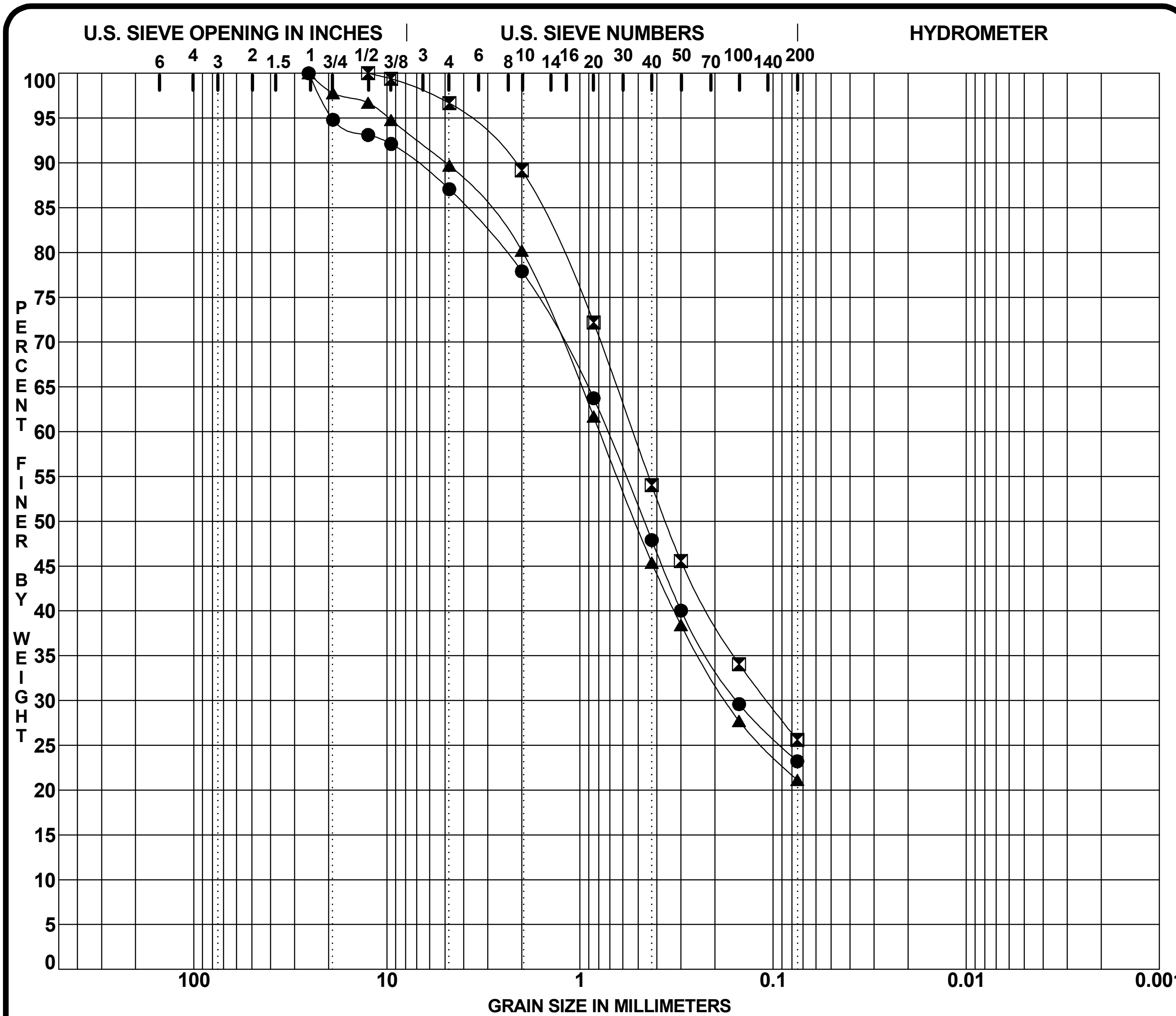
Expansion Index Test

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	4	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	17	27				
⊠ B-3	@ 0-3'	(SM) Silty Sand	19	--				
▲ B-5	@ 0-3'	(SM) Silty Sand	20	--				

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-1	@ 0-3'	25.40	0.72	0.154		12.9	63.8	23.2	
⊠ B-3	@ 0-3'	12.50	0.53	0.108		3.3	71.1	25.6	
▲ B-5	@ 0-3'	25.40	0.79	0.174		10.3	68.6	21.1	

PROJECT:	Proposed Residential Development	**PROJECT NO.:**	33979.1
CLIENT:	Mr. Andrew Taylor	**DATE:**	January 2024

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: D-1

Results Only Soil Testing for Taylor Apartments

January 11, 2024

Prepared for:

Andrew Tardie
LOR Geotechnical
6121 Quail Valley Ct.
Riverside, 92507 CA
atardie@lorgeo.com

Project X Job#: S240110C
Client Job or PO#: 33979.1

Respectfully Submitted,

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

Soil Analysis Lab Results

Client: LOR Geotechnical
Job Name: Taylor Apartments
Client Job Number: 33979.1
Project X Job Number: S240110C
January 11, 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%)	(Ohm-cm)	(Ohm-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	7.8	0.0008	7.2	0.0007	48,910	10,050	7.7	116	2.0	6.6	0.2	0.0	13.1	9.7	14.3	92.9	6.0	14.3
RV-2 - B-3 (SM) Silty Sand	0-3	18.1	0.0018	30.0	0.0030	73,700	16,750	8.5	134	1.1	14.9	1.6	0.0	44.6	13.2	11.6	75.1	1.5	9.6
RV-3 - B-5 (SM) Silty Sand	0-3	8.6	0.0009	6.4	0.0006	47,570	5,695	7.6	196	1.0	26.6	0.2	0.0	12.0	5.6	11.7	74.1	0.5	0.2

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 which 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.

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 S240110C LOR 33979.1 Taylor Apartments 3 Full

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:	33979.1	**Project Name:**	Taylor Apartments		

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)

	#	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
					Geo Quad				Full Corrosion Series												Reports										
					Caltrans CTM643	Caltrans CTM643	Caltrans CTM417	Caltrans C1422												350g Sample	*Req: Min. 3 Samples, site map, and groundwater info					ASTM D5334 8-inch	1,500g Sample			ASTM A751	
					AASHTO 12888	AASHTO T 289	AASHTO T 290	AASHTO T 291	SM 2580B		SM 4500-NH3	SM 4500-NO3													ASTM D2216					ASTM E322	
		Default Method			ASTM G187	ASTM G 51	ASTM D4327	ASTM D4327	ASTM G200	SM 4500-S2	ASTM D4327	ASTM D4327	SM 2320B	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327				ASTM D2216	SM 2320B	ASTM D698	IEEE 442 6-inch		HACH BART		ASTM E1621	
57	1	RV-1 - B-1 - (SM) Silty Sand	0-3	01/04/2024																●											
58	2	RV-2 - B-3 - (SM) Silty Sand	0-3	01/04/2024																●											
59	3	RV-3 - B-5 - (SM) Silty Sand	0-3	01/04/2024																●											
	4																														
	5																														
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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: APNs 0410-242-03 and -04, Hesperia
Project Number: 33979.1
Client: Mr. Andrew Taylor
Site Lat/Long: 34.4065/-117.2987
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	0.196
S_{D1}/S_{DS}	T_S	0.979
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}	0.9283
Equation 11.4-2 - F_V*S_1 [1]	S_{M1}	1.3925

[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	1.422
Design Maps	S_1	0.557
Equation 11.4-1 - F_A*S_S	S_{MS}	1.422
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	0.948
Design Maps	PGA	0.564
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.620
Section 21.5.3	80% of PGA_M	0.496
Design Maps	C_{RS}	0.925
Design Maps	C_{R1}	0.904

REFERENCE	NOTATION	VALUE
$0.2*(S_{D1}/S_{DS})$	T_0	0.137
S_{D1}/S_{DS}	T_S	0.683
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.647
Equation 11.4-2 - F_V*S_1	S_{M1}	0.971

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.925
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.904

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

Period	Cr
0.200	0.925
0.300	0.922
0.400	0.920
0.500	0.917
0.600	0.915
0.680	0.912
1.000	0.904

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

LOR GEOTECHNICAL GROUP, INC.

PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

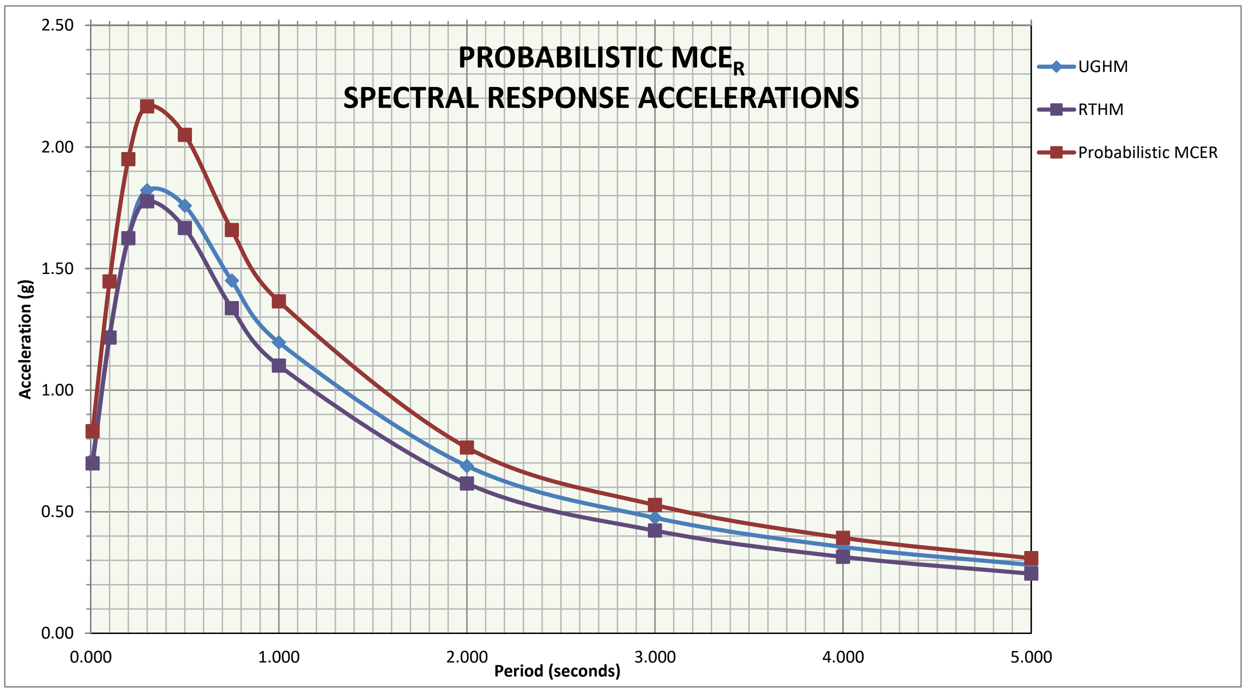
Project No: 33979.1

Period	UGHM	RTGM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.716	0.698	1.19	0.831
0.100	1.224	1.216	1.19	1.447
0.200	1.630	1.625	1.20	1.950
0.300	1.822	1.776	1.22	2.167
0.500	1.758	1.666	1.23	2.049
0.750	1.450	1.337	1.24	1.658
1.000	1.196	1.101	1.24	1.365
2.000	0.688	0.616	1.24	0.764
3.000	0.475	0.422	1.25	0.528
4.000	0.355	0.314	1.25	0.393
5.000	0.281	0.245	1.26	0.309

Probabilistic PGA: 0.716
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

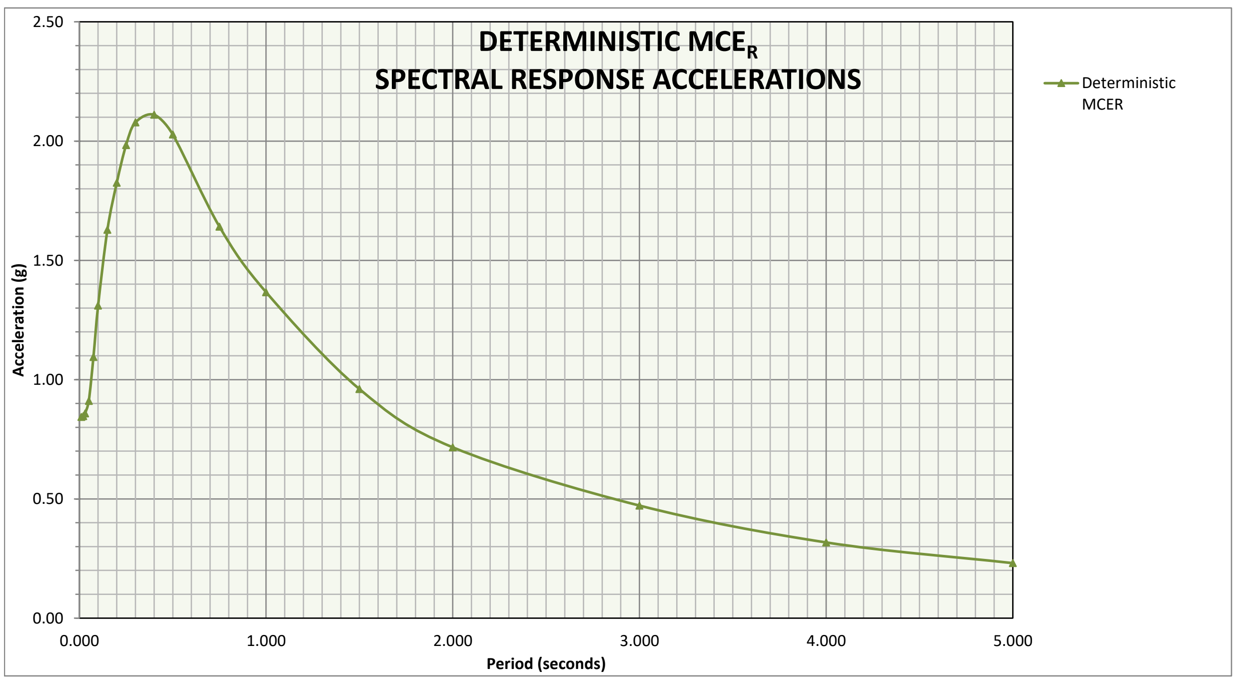
Project No: 33979.1

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	0.709	1.19	0.843	0.843
0.020	0.711	1.19	0.847	0.847
0.030	0.721	1.19	0.858	0.858
0.050	0.765	1.19	0.910	0.910
0.075	0.920	1.19	1.095	1.095
0.100	1.101	1.19	1.310	1.310
0.150	1.357	1.20	1.628	1.628
0.200	1.521	1.20	1.825	1.825
0.250	1.639	1.21	1.984	1.984
0.300	1.703	1.22	2.078	2.078
0.400	1.716	1.23	2.110	2.110
0.500	1.649	1.23	2.028	2.028
0.750	1.324	1.24	1.641	1.641
1.000	1.102	1.24	1.366	1.366
1.500	0.774	1.24	0.960	0.960
2.000	0.577	1.24	0.716	0.716
3.000	0.377	1.25	0.472	0.472
4.000	0.254	1.25	0.317	0.317
5.000	0.183	1.26	0.231	0.231

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	0.709
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)



LOR GEOTECHNICAL GROUP, INC.

SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.831	0.843	0.831	0.554
0.100	1.447	1.310	1.310	0.873
0.200	1.950	1.825	1.825	1.217
0.300	2.167	2.078	2.078	1.385
0.500	2.049	2.028	2.028	1.352
0.750	1.658	1.641	1.641	1.094
1.000	1.365	1.366	1.365	0.910
2.000	0.764	0.716	0.716	0.477
3.000	0.528	0.472	0.472	0.315
4.000	0.393	0.317	0.317	0.212
5.000	0.309	0.231	0.231	0.154

Period	ASCE 7 SECTION 21.3 General Spectrum	80% General Response Spectrum
0.005	0.394	0.315
0.010	0.408	0.327
0.020	0.437	0.350
0.030	0.466	0.373
0.050	0.524	0.420
0.060	0.553	0.443
0.075	0.597	0.478
0.090	0.641	0.512
0.100	0.670	0.536
0.110	0.699	0.559
0.120	0.728	0.582
0.136	0.774	0.619
0.150	0.815	0.652
0.160	0.844	0.675
0.170	0.873	0.698
0.180	0.902	0.722
0.200	0.948	0.758
0.250	0.948	0.758
0.300	0.948	0.758
0.400	0.948	0.758
0.500	0.948	0.758
0.600	0.948	0.758
0.640	0.948	0.758
0.750	0.948	0.758
0.850	0.948	0.758
0.900	0.948	0.758
0.970	0.948	0.758
1.000	0.928	0.743
1.500	0.619	0.495
2.000	0.464	0.371
3.000	0.309	0.248
4.000	0.232	0.186
5.000	0.186	0.149

ASCE 7-16: Section 21.4

Site Specific

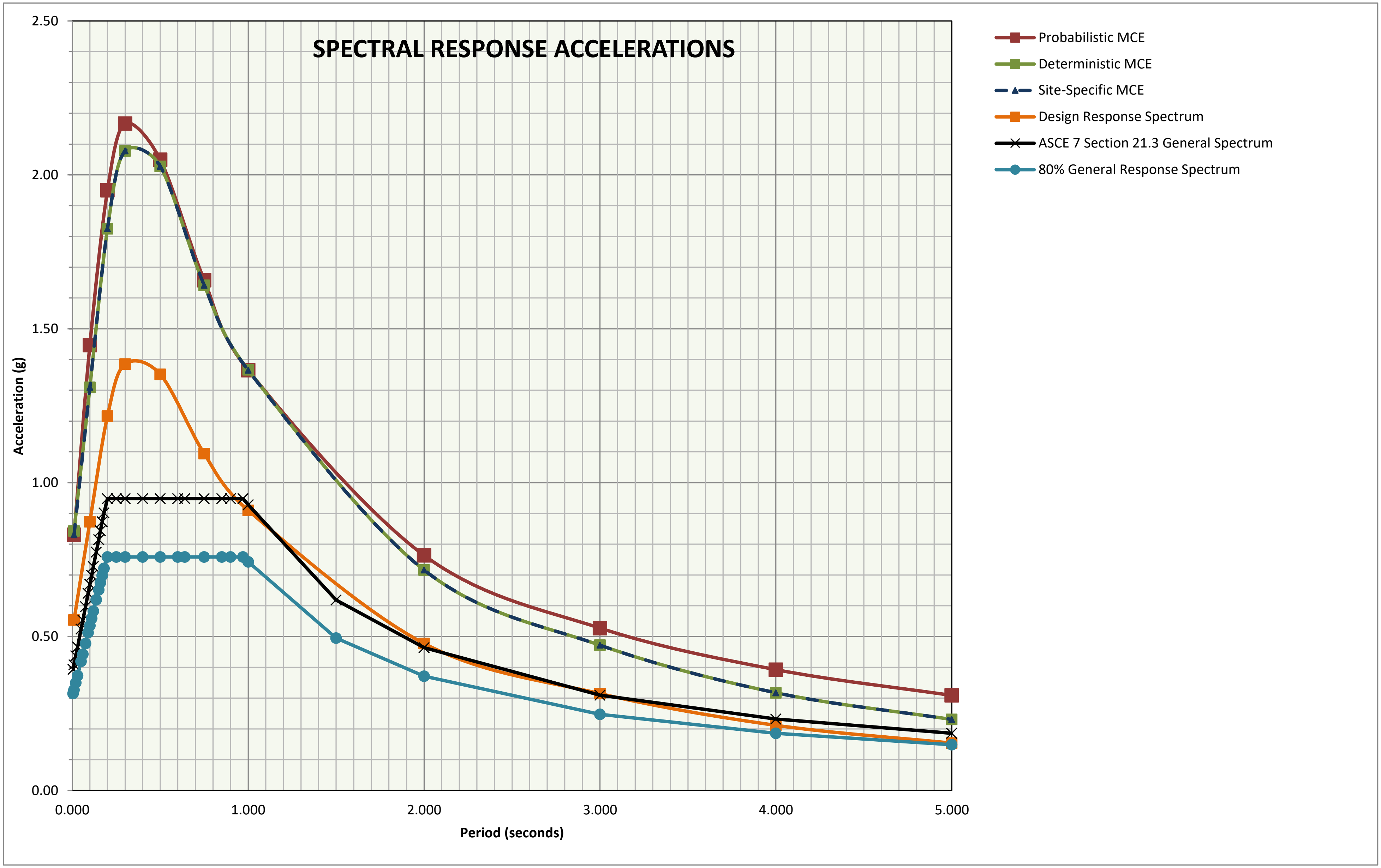
	Calculated Value	**Design Value**
SDS:	1.247	1.247
SD1:	0.955	0.955
SMS:	1.870	1.870
SM1:	1.432	1.432
Site Specific PGAm:	0.709	0.709
Site Class:	D measured	

Seismic Design Category - Short* D

Seismic Design Category - 1s* D

* Risk Categories I, II, or III

Project No: 33979.1



Project No: 33979.1