APPENDIX E1

PRELIMINARY GEOTECHNICAL INVESTIGATION APN'S: 1167-161-03 & -04

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

PRELIMINARY GEOTECHNICAL INVESTIGATION
APN'S: 1167-161-03 & -04
CITY OF GRAND TERRACE
SAN BERNARDINO COUNTY, CALIFORNIA

PROJECT NO. 33318A.1
OCTOBER 20, 2022

Prepared For:

Lewis Management Corporation
1156 N. Mountain Avenue
Upland, California 91786

Attention: Ms. Waen Messner

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

October 20, 2022

Lewis Management Corporation
1156 N. Mountain Avenue
Upland, California 91786

Project No. 33318A.1

Attention: Ms. Waen Messner

Subject: Preliminary Geotechnical Investigation, APN's: 1167-161-03 & -04, Grand Terrace, San Bernardino County, California.

LOR Geotechnical Group, Inc., is pleased to present this report summarizing our geotechnical investigation for the proposed residential development within the subject site. This report was based upon a scope of services generally outlined in our Proposal dated June 22, 2016, and other written and verbal communications with you. Recent updating of our earlier draft report was based on our Work Authorization Agreement dated October 4, 2022.

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

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to all undocumented fill material and any loose alluvial materials should be removed from areas to receive engineered compacted fill. The data developed during this investigation indicates that average removal depths of approximately 3 to 6 feet below existing grades will be required within the majority of the site. However, deeper removals ranging from approximately 10 to 15 feet will be required within the drainage course that traverses the southern portion of the property.

Soils with low expansion index and a negligible sulfate content soils were encountered on the site. In addition, our test results for a representative sample indicate a low to moderate R-value for pavement design.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During January and February of 2017, a preliminary geotechnical investigation was performed by LOR Geotechnical Group, Inc., for proposed residential development of approximately 3 acres of property located on the north side of De Berry Street and west of Michigan Avenue in the City of Grand Terrace, California. Although a draft report was prepared by this firm and delivered to you for review, a final report was not issued. This report represents an updated version of our earlier draft report. Updating has mainly been limited to the site description in terms of site conditions, and updating of seismic design criteria following 2019 CBC guidelines. In addition, we have included responses to items presented for evaluation in the geology and soils portion of CEQA guidelines (2022).

The site consists of APN's 1167-161-03 and -04. The purpose of our 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;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

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

To orient our investigation at the site, a parcel map was furnished for our use. An image from Google Earth has been utilized as a base map for our Geotechnical Map, Enclosure A-2, in Appendix A.

PROJECT CONSIDERATIONS

Information furnished to this firm indicates the project will consist of the construction of an unspecified number of residential structures and associated improvements within the 3 acre site. Although specific information pertaining to the types of residences that will be constructed was unavailable at this time, the residential structures are anticipated to be one to two stories and of wood or metal frame construction with plaster veneer exterior. Light to moderate foundation loads are anticipated with such structures.

Grading plans were also not available at this time. It is likely that grading will entail filling within the natural drainage that traverses the southern portion of the site. This may involve cutting and removal of material from the upper, relatively flat areas outside of this drainage, the importation of fill soils, or a combination of these options.

EXISTING SITE CONDITIONS

The site is located on the north side of De Berry Street, west of Michigan Avenue, in the City of Grand Terrace, San Bernardino County, California. A vacant single-family residence, located at 21992 De Berry Street, was situated within the southeast corner of the otherwise vacant property during the time of our field investigation work, but has since been removed (2019). The majority of the site is relatively flat, however, a drainage course traverses the southern portion of the site from east to west. This drainage is approximately 15 feet lower than the surrounding areas. At the time of our investigation, the site was covered by a moderate growth of annual grasses and weeds. Minor amounts of asphalt and concrete and a few trees are present around the residence. The site is bounded by commercial developments to the north, a pump house on the north half of the eastern site boundary with residential properties further south and east, by a construction company storage yard on the west, and by De Berry Street and residential properties to the south. A more detailed description of the existing site conditions and the history of site use is presented within our Environmental Site Assessment report (LOR, 2017).

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on January 6 and 25, 2017 and consisted of the excavation of 2 exploratory trenches using backhoe equipment and advancing 4 exploratory borings using a track mounted CME-55 drill rig equipped with 8-inch diameter hollow stem augers. The trenches were excavated to depths ranging from approximately 11 to 15 feet below the existing ground surface. In-place density tests were taken in accordance with ASTM D 2922, the Nuclear Gauge Method. Bulk samples of the encountered materials were obtained and returned to the laboratory in sealed containers for further testing and evaluation.

The borings were drilled to depths ranging from approximately 16.5 to 51.5 feet below the existing ground surface. Relatively undisturbed in-place and bulk samples of the materials encountered were obtained and returned to our geotechnical laboratory for further testing and evaluation. Samples of the encountered materials were obtained and returned to our geotechnical laboratory in sealed containers for further testing and evaluation. The approximate locations of our exploratory borings and trenches are presented on the attached Geotechnical Map, Enclosure A-2, Appendix A.

Logs of the subsurface conditions encountered in the exploratory trenches and borings were maintained by a geologist from this firm. A detailed description of the field exploration program and trench and boring logs is presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

The site is located within the northeastern portion of the Riverside area which in turn lies within the northern end of the Perris Valley. The property is situated between the La Loma Hills to the west and the Box Springs Mountains to the east and southeast. This area is located on the Perris block within the northern Peninsular Ranges geologic province of southern California. While the Perris block is considered to be a relatively stable structural block, it is bounded by active faults. These include the Elsinore fault zone on the southwest, the San Jacinto fault zone on the northeast, and the Cucamonga fault zone on the north. The Perris block is underlain predominately by a very large mass of crystalline igneous rocks of Cretaceous age and older metasedimentary and metavolcanic rocks.

The Perris block has a series of erosional surfaces, marked by low topographic relief and capped with unconsolidated alluvial sediments stripped from the surrounding highlands, such as the La Loma Hills and Box Springs Mountains. This region including and around the site was mapped by the California Division of Mines and Geology as being underlain by deposits of relatively well-indurated, reddish-brown, older alluvium (Morton and Matti, 2001, and Dibblee, 2003).

The nearest known active fault zone is the San Jacinto fault zone located approximately 3 kilometers (1.9 miles) to the northeast. Other major faults within the region include the San Andreas fault zone located approximately 16 kilometers (10 miles) to the northeast, and the Cucamonga fault zone located approximately 19.2 kilometers (12 miles) to the northwest.

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

Site Geologic Conditions

The subject site is underlain by surficial topsoil and localized fill soils followed by thick older alluvial materials. Within the drainage that traverses the southern portion of the site, younger alluvial soils are present. The earth materials encountered during our site investigation are described in detail on the Trench and Boring Logs within Appendix B and are generally described as follows:

Artificial Fill: The artificial fill soils encountered at the site appear to be mainly present around the area of the existing and former residences in the southern portion of the site. These are associated with grading that likely took place during site development, including the installation of underground utilities. Along the top of the bank for the western half of the south side of the onsite drainage course, it appears that fill was placed to steepen the bank and extend the rear yard. In addition, our review of aerial photographs suggests that a small drainage course traversed the far northwest corner of the site and there are currently no drainage pathways in that area. Although, only minimally excavated within during our investigation, the fill soils appear to consist of locally derived silty sand soils.

Topsoil: Mantling the surface of the vast majority of the site are topsoil materials that consist of silty sand. These soils average approximately 1.5 to 2 feet in thickness and are relatively loose. At the time of our site investigation, these soils were moist as a result of recent heavy rains.

Younger Alluvium: Relatively young alluvial soils are present within the low drainage course that traverses the southern portion of the site from east to west. The younger alluvial materials consist mainly of loose to medium dense silty sand with well graded sand soils at depth. At the location of our exploratory trench, these sediments were found to exceed 15 feet in thickness with the looser, silty sand materials present in the upper 8 to 11 feet and the denser, sandier soils present below these depths.

Older Alluvium: Older alluvium is present at shallow depth across the site and underlies the fill soils and younger alluvium. In general, the older alluvium consists of stiff sandy silt in the near surface and generally becomes sandier with an increase in depth. The dense silty sand and the sandy silt layers are typically massive and porous in the near surface. However, the porosity decreases below the first couple of feet and a blocky soil structure is evident in the deeper, denser materials.

Groundwater Hydrology

Groundwater was not encountered in any of our exploratory trenches or borings, nor was any groundwater seepage observed during our site reconnaissance. Regional studies by Carson and Matti (1985) indicate that the depth to groundwater at the site is on the order of 100 to 120 feet beneath the site. Recent groundwater data for a well located approximately one mile to the southwest of the site, just southeast of the intersection of Main Street and Taylor Street, indicates that the depth to groundwater at that location ranged from 157 to 177 feet during the time period from October 2011 to March of 2016. According to the Western Municipal Water District and the San Bernardino Valley Water Conservation District Cooperative Well Measuring Program, the depth to groundwater was approximately 160 to 170 feet in nearby wells during the fall of 2014. The anticipated groundwater flow direction below the site is anticipated to be to the southwest following the regional surface topography.

Surface Runoff

Current surface runoff of precipitation waters across the site is largely from the northeast to the southwest with the far southern portion flowing to the north. Runoff occurs as sheetflow into the onsite drainage course and then offsite to the west.

Mass Movement

The majority of the site consists of relatively flat surfaces with gently sloping areas in between. Locally, along the drainage course in the southwestern portion of the property, the slopes approach 2:1 (horizontal to vertical). However, considering the site geologic conditions and the overall gently sloping nature of the property, the potential for mass movement failures such as landslides or debris flows is considered very low. In addition, no loose, un-rooted rocks that could fall or topple and roll were noted to be present above at or above the site and the potential for rockfalls occurring at the site is also considered to be nil.

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, 1995). No evidence of faulting was noted during our field reconnaissance nor during our review of aerial photographs covering the property and immediate surrounding region. The closest known active fault is the San Jacinto fault zone, located approximately 3 kilometers (1.9 miles) to the northeast.

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

Other faults in the region include the San Andreas fault located approximately 16 kilometers (10 miles) to the northeast, the Cucamonga fault located approximately 19.2 kilometers (12 miles) to the northwest, and the Elsinore fault approximately 32 kilometers (20 miles) to the southwest.

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 or greater.

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

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

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

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 September 10, 2022.

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 of southern California.

In the second search, the micro seismicity of the area lying within a 10 kilometer (6.1 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 2.0 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, numerous faults have occurred in connection with the San Jacinto fault zone.

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, seiches and tsunamis, earthquake induced flooding, landsliding and rockfalls, and seismic-induced settlement.

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within granular, loose sediments where the depth to groundwater is usually less than 50 feet. As the site is underlain at depth by dense, older alluvium; the upper, loose alluvial soils are anticipated to be replaced with compacted fill; and the depth to groundwater is on the order of 100 feet or more, the possibility of liquefaction at the site is considered to be very low to 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 an earthquake and affect the site by flooding.

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

Rockfalls: The flat lying nature of the property and surrounding area and the absence of nearby rock outcrops precludes the potential for rockfalls occurring at 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 (stiff), older alluvial materials, the potential for settlement is considered low. In addition, the remedial earthwork operations to be conducted for the development of the site will mitigate any surficial loose soil conditions.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code)

Design requirements for structures can be found within Chapter 16 of the 2019 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. Bedrock is assigned one of three of these six site classes and these are: A, B, or C. Soil is assigned as C, D, E, or F. Per ASCE 7-16, Site Class A and Site Class B shall be measured on-site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Site Class A and Site Class B shall not be used if more than 10 feet of soil is between the rock surface and bottom of the spread footing or mat foundation. Site Class C can be used for very dense soil and soft rock with Ñ values greater than 50 blows per foot. Site Class D can be used for stiff soil with Ñ values ranging from 15 to 50 blows per foot. Site Class E is for soft clay soils with Ñ values less than 15 blows per foot. Our current investigation, mapping by others, and our experience in the site region indicates that the materials beneath the site are considered Site Class C very dense soil/soft rock.

CBC Earthquake Design Summary

Earthquake design criteria have been formulated in accordance with the 2019 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 within Appendix D.

CONCLUSIONS

General

This investigation provides a broad overview of the geotechnical and geologic factors which are expected to influence future site planning and development. On the basis of our field investigation and testing program, it is the opinion of LOR Geotechnical Group, Inc. that the proposed development is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into design and implemented during grading and construction.

The subsurface conditions encountered in our exploratory trenches and borings are indicative of the locations explored. The subsurface conditions presented here are not to be construed as being present the same everywhere on the site. If conditions are encountered during the construction of the project which differ significantly from those presented in this report, this firm should be notified immediately in order that we may assess the impact to the recommendations provided.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the younger alluvial soils and portions of the older alluvial soils, will not, in their present conditions, provide uniform and/or adequate support for the proposed structures. However, the removal and recompaction of these soils will create an acceptable solution.

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. This compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. In addition, the construction of this compacted fill mat will allow for the removal of the existing unsuitable alluvial materials within the building pad areas.

Soil Expansiveness

As noted by our explorations and testing, the majority of the site surficial soils consist of silty sands and sandy silts with a very low to low expansion potential. Although the site grading will likely involve relatively significant mixing and blending of the site materials and a reduction of the overall expansion potential of the fill soils, sandy silt soils of low expansion index will still remain beneath the fill in most areas and mitigation measures for expansive soils will be necessary. These measures are described in the Foundation Design, Building Area Slab-on-Grade, and Exterior Flatwork sections of this report.

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

Geologic Mitigations

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

Seismicity

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

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

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

CEQA Considerations

In response to an item presented within a recent review of our 2017 draft report by Lilburn (2022), we have completed the CEQA Appendix G Checklist questions for Geology and Soils. Our rankings of the anticipated impacts that the proposed project will have on considerations related to geology and soils are presented below:

VII. GEOLOGY AND SOILS. Would the project

a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:

i)Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map, issued by the State Geologist for the area or based on other substantial evidence of a known fault? Refer to Division of Mines and Geology Special Publication 42. - *Response - No Impact*

ii) Strong seismic ground shaking?
Response - Less Than Significant with Mitigation Incorporated

iii) Seismic-related ground failure, including liquefaction?
Response - No Impact

iv) Landslides?
Response - No Impact

b) Result in substantial soil erosion or the loss of topsoil?
Response - Less Than Significant with Mitigation Incorporated

c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?
Response - Less Than Significant Impact

d) Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property?
Response - No Impact

e) Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?
Response - No Impact

f) Directly or indirectly destroy a unique paleontological resource or site or unique geologic feature?
Response - No Impact

RECOMMENDATIONS

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor, and 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. These materials should not be incorporated within engineered compacted fill. It is our recommendation that any existing undocumented fills encountered be removed and replaced with engineered compacted fill. This pertains to all grading areas including proposed flatwork and/or paved areas. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

Cavities created by removal of subsurface obstructions should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended in the following Engineered Compacted Fill section of this report.

Initial Site Preparation

All undocumented fill material and any loose alluvial materials should be removed from structural areas and areas to receive engineered compacted fill. The data developed during this investigation indicates that removals on the order of approximately 3 to 6 feet will be required from areas underlain by older alluvium. Deeper removals will be required in the drainage area that transverses the southern half of the site and contains younger alluvium, with removals of 10 to 15 anticipated. The actual depths of removals should be verified during the grading operation by observation and/or in-place density testing. Removals should expose older alluvial materials with a relative in-situ compaction of at least 83 percent and/or an in-situ saturation of at least 85 percent.

Preparation of Fill Areas

Prior to placing fill, the surfaces of all areas to receive fill should be scarified to a depth of 6 to12 inches. The scarified soil should be brought to near optimum moisture content and recompacted to a relative compaction of at least 90 percent (ASTM D 1557).

Preparation of Building Pad Areas

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

No structure should be placed across any areas where the ratio of 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. For example, if one edge of the building pad of a cut-to-fill transition lot requires 10 feet of fill, then the cut portion of the lot should be over-excavated to a minimum of 3 feet below the footing elevations.

Engineered Compacted Fill

All fill materials should be free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in building area fills (within two feet of the bottom of the footings), the upper one foot of road subgrade, or within trench backfill. Materials greater than 12 inches in diameter should be placed in approved disposal areas, typically 10 feet or more below proposed finish grade elevations.

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

Fill should be spread in maximum 8-inch thick, uniform, loose lifts with each lift brought to near optimum moisture content and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557. The upper 12 inches of areas to be paved should be compacted to at least 95 percent (ASTM D 1557).

Based upon the relative compaction of the younger alluvial soils determined during this investigation and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage factor of approximately 10 to 15 percent for the younger alluvium. The older alluvial soils are denser and removal and replacement of these soils should result in a compaction shrinkage factor of approximately 5 to 10 percent. Shrinkage factors should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

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

Short-Term Excavations

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

Short-term excavation 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based on our exploratory trenches and borings and our observations, it appears that the alluvial soils can be classified as Type C soils. Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

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.

Where fills are to be placed against existing slopes steeper than five horizontal to one vertical, the fill should be properly keyed and benched into competent native materials. The key, constructed across the toe of the slope, should be a minimum of 12 to 15-feet wide, a minimum of two feet deep at the toe, and sloped back at two percent. Benches should be constructed at approximately two to four feet vertical intervals. Typical keying and benching operations are presented on Enclosure D-1, within Appendix D.

Slope Protection

Since the native 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 completion. 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.

Foundation Design

If the site is prepared as recommended, the proposed residential structures may be safely founded on conventional shallow foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 24 inches of engineered compacted fill placed over competent native materials. All foundations should have a minimum width of 12 inches and, because the site soils are of low expansion potential, should be established a minimum of 18 inches below lowest adjacent grade.
Footings at least 12 to 15 inches wide and placed at least 18 inches below the lowest adjacent final grade could be designed for a maximum soil bearing pressure of 2,100 psf for dead plus live loads.

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

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 300 pounds per square foot per foot of depth. Base friction may be computed at 0.30 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Footings on low expansive soils should be reinforced with a minimum of two # 4 rebars, one near the top and one near the bottom of the footings.

The preceding recommendations to counteract expansive soil activity should be considered minimum and should be revised upon the completion of the site grading. More stringent parameters for design of foundations on expansive soils can be specified by a structural engineer experienced in these matters.

<u>Post-Tension Design Parameters</u>

For low expansive soils, we recommend that the planned buildings be supported on post-tensioned slab foundations resting on a minimum of 2.0 feet of engineered compacted fill placed over competent native materials.

- Allowable Soil Bearing Pressure, q_{allow}: 1,800 psf

- Edge Moisture Variation Distance, e_m:
  Center Lift Loading Conditions: 9.0 ft
  Edge Lift Loading Conditions: 6.0 ft

- Differential Swell, y_m:
  Center Lift 0.23 in
  Edge Lift 0.53 in

- Subgrade Soil Friction Coefficient, μ: 0.30

The above design parameters were determined in accordance with Design of Post-Tensioned Slabs-on-Ground, third edition, published by the Post-Tensioning Institute.

It should also be noted that the post-tension design parameters presented above are preliminary. It is understood that during the site rough grading some mixing and blending of the site soils will occur. Therefore, further testing and verification will be necessary to confirm that these conditions are indeed present at the conclusion of the site rough grading and that the post-tension design parameters presented above remain accurate.

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

Concrete floor slabs should bear on a minimum of 24 inches of engineered compacted fill placed over competent native materials. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete. Slab areas should be properly pre-soaked prior to pouring concrete. Slab areas should be pre-soaked to approximately 4 percent above the optimum moisture content to a minimum depth of 18 inches. Unless more stringent parameters are given by the structural engineer with expansive soil design experience, the slab thickness should be a minimum of 4 inches. Minimum slab reinforcement should consist of #3 rebars placed at a maximum spacing of 18 inches on center, each way.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete. The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork

To provide adequate support, exterior flatwork improvements should rest on a minimum of 12 inches of soil compacted to at least 90 percent (ASTM D 1557). Flatwork areas should be pre-soaked prior to pouring concrete to a minimum depth of 18 inches and to approximately 4 percent above the optimum moisture content. All sidewalks, patio slabs, and driveways with a minimum dimension greater than 5 feet should be reinforced with #3 rebars placed at a maximum spacing of 18 inches on center, each way. Reinforcement for curbing should be one continuous #4 rebar at top and bottom. In addition, it is recommended that sidewalks, patio slabs, curbs, etc., have a thickness of at least 4 inches, with saw cuts every 10 feet or less. Driveways should be at least 5-inch thick, with saw cuts every 15 feet or less.

Flatwork surface should be sloped a minimum of 1 percent away from buildings and slopes, to approved drainage structures.

Again, the above recommendations to counteract low expansive soil activity should be considered minimum as determined by our preliminary findings and should be revised, as necessary, as based upon the results of additional testing conducted during, or near the completion of site grading.

Wall Pressures

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

For design of retaining walls unrestrained against movement at the top, we recommend an equivalent fluid density of 45 pounds per cubic foot (pcf) be used. This assumes level backfill consisting of recompacted, non-expansive, native soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

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

As noted before, low expansive soils are present at the site. Since these materials have very low permeability, uncertain behavior, and exert higher lateral earth pressures on earth retaining structures than more granular soils, the onsite soils should not be used as wall backfills.

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

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, properly drained, non-expansive backfill with no additional surcharge loadings. If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters.

Preliminary Pavement Design

Testing and design for preliminary on-site pavements was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing, R-values for subgrade soils will range from approximately 10 to 30. Traffic Indices generally used for these kinds of developments, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
Typical Residential Collections	5.0	10	0.25' AC/0.75' AB
	6.0	10	0.30' AC/0.95' AB
	7.0	10	0.35' AC/1.15' AB
Typical Residential Collections	5.0	30	0.25' AC/0.50' AB
	6.0	30	0.30' AC/0.60' AB
	7.0	30	0.35' AC/0.75' AB
AC - Asphalt Concrete			
AB - Class 2 Aggregate Base			

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 0.5 foot thick concrete, with a 0.50 foot thick aggregate base. Concrete pads are also recommended in areas adjacent to trash storage areas where heavier loads will occur due to operation of trucks lifting trash dumpsters.

The recommended 0.5 feet thick portland cement concrete (PCC) pavement section should have a minimum modulus of rupture (MR) of 550 pounds per square inch (psi).

The portland cement concrete pavement section may be placed directly over the native subgrade prepared as described above and pre-soaked as indicated in this report. In addition, the concrete section should be reinforced as indicated within this report. Transverse joints should be sawcut in the pavement at 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 designs were 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. The actual design traffic index's for various roads should be supplied by the local controlling agency responsible for the roadways.

Sulfate Protection

The results of the soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented on Enclosure C. Based on the test results, it appears that there is a negligible to moderate sulfate exposure to concrete elements in contact with the on site soils per the 2016 CBC. This should be verified by additional sampling and testing when the actual finish and near finish surface soils are obtained.

Grading Plan Review/Supplemental Geotechnical Investigation

At the present time, no grading plans showing anticipated developed site conditions are available. Once these become available, they should be reviewed by this office in order to address potential site specific geotechnical and/or geologic concerns that could require mitigation. At that time, it may also prove beneficial to conduct limited supplemental geotechnical investigation within selected areas of the site in order to focus on a particular concern, such as expansion potential of the local onsite materials or the anticipated depths of removal across areas underlain by alluvial sediments.

Construction Monitoring

As mentioned above, post investigation services are an important and necessary continuation of geotechnical work associated with planning and development of this project. Once project plans and specifications have been reviewed by this firm, construction monitoring, including testing for on-site pavement design, should be performed during and after the site rough grading operations. During and/or near the completion of site grading, additional expansion index testing should be conducted to characterize selected areas and to develop lot specific recommendations for foundation design as related to the expansion potential of the graded site soils.

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

1. Site preparation-stripping and removals.
2. Excavations, including approval of the bottom of excavation prior to filling.
3. Scarifying and recompacting prior to fill placement.
4. Subgrade preparation for pavements and slabs-on-grade.
5. Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.
6. Foundation excavations, including footings.

TIME LIMITATIONS

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

LIMITATIONS

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

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations, and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

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

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

CLOSURE

It has been a pleasure to assist you with this project. We look forward to being of further assistance to you as construction begins.

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

John P. Leuer, GE 2030
President

Robert M Markoff, CEG 2073
Engineering Geologist

RMM:JPL/ss

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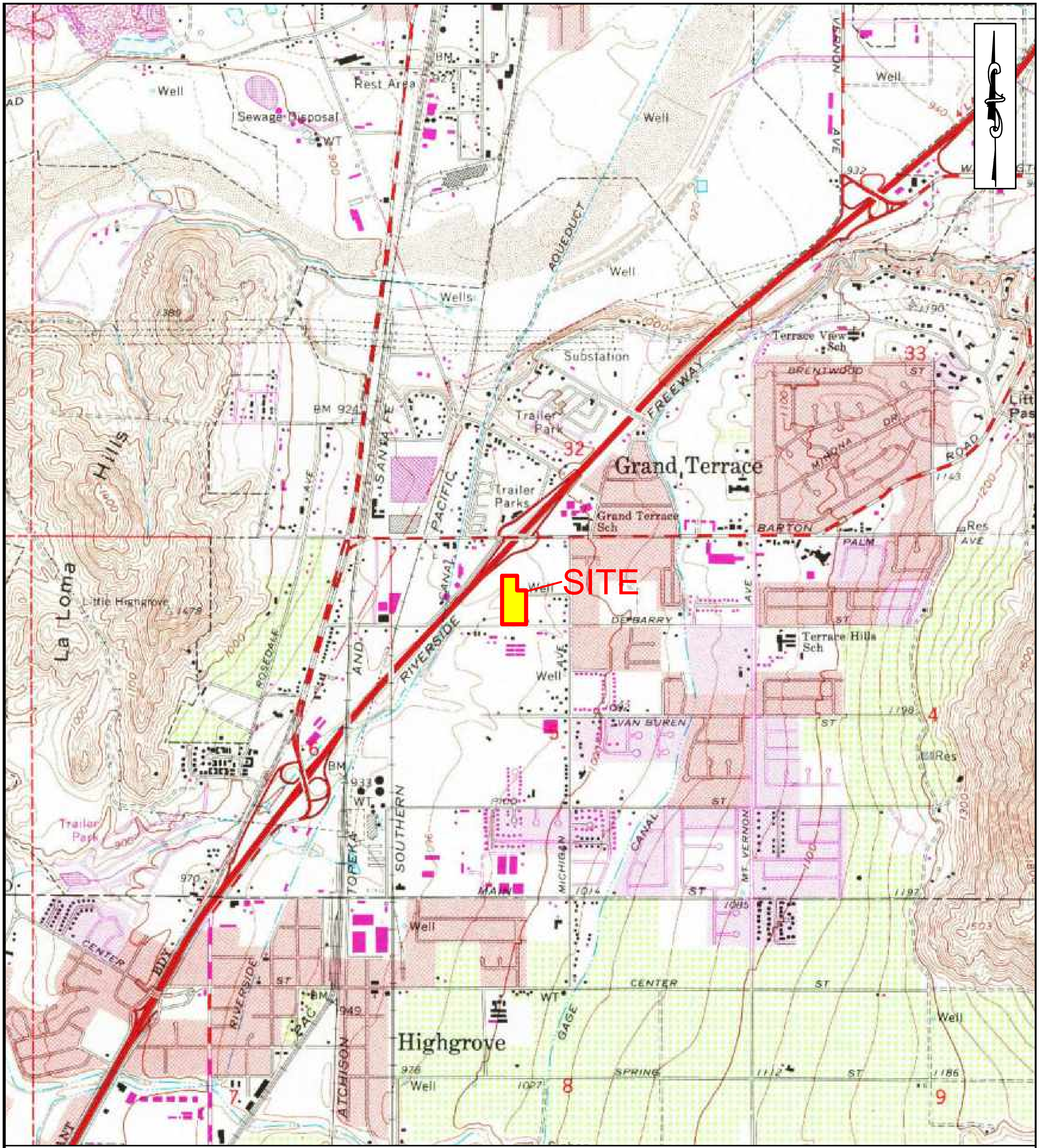
AERIAL PHOTOGRAPHS
(SAN BERNARDINO COUNTY FLOOD CONTROL DISTRICT)

DATE	FLIGHT NO.	PHOTO NO(S).	SCALE
1938	W-83	K-2-20 & -21	1" = 1,000'
11/18/1955	F-34	2-101 & 2-102	1" = 2,000'
2/1/1969	C-293	100 & 101	1" = 2,000'
2/1969	C-295	96	1" = 2,000'
10/30/1972	C-194	72 & 73	1" = 2,000'
1/21/1978	C-279	51 & 52	1" = 2,000'
2/25/1986	C-480	53 & 54	1" = 2,000'
7/1/1991	C-487	69 & 70	1" = 2,000'
4/20/1996	C-258	77 & 78	1" = 2,000'
6/15/2001	C-541	88, 89, & 90	1" = 2,000'
1/19/2005	C-553	9-44 & 9-45	1" = 1,000'

APPENDIX A

Index Map, Geotechnical Map,
Regional Geologic Map,
and
Historical Seismicity Maps



INDEX MAP

PROJECT: APNS: 1167-161-03 & 04, Grand Terrace, California

PROJECT NO.: 33318A.1

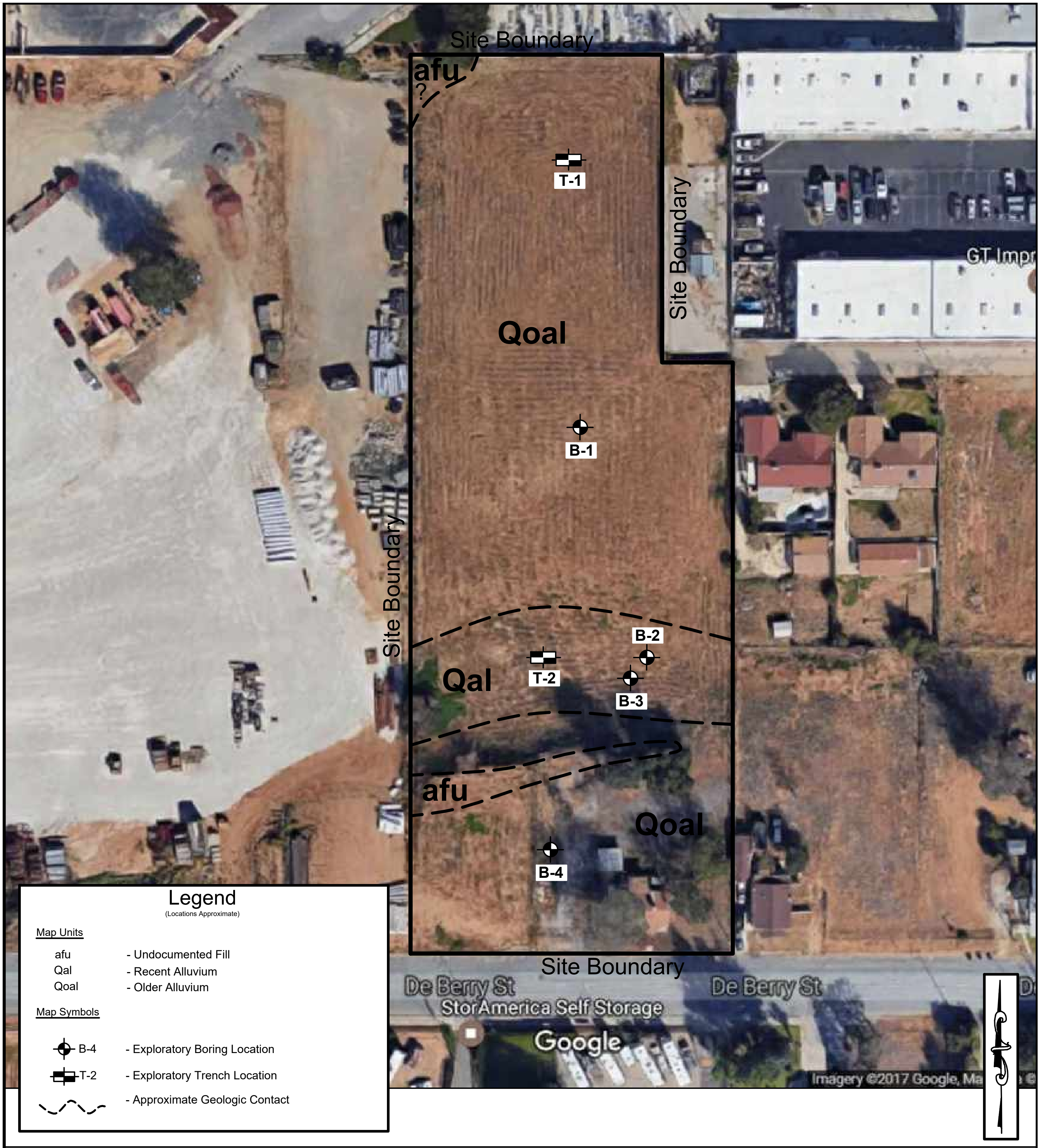
CLIENT: Lewis Management Corp.

ENCLOSURE: A-1

LOR GEOTECHNICAL GROUP, INC.

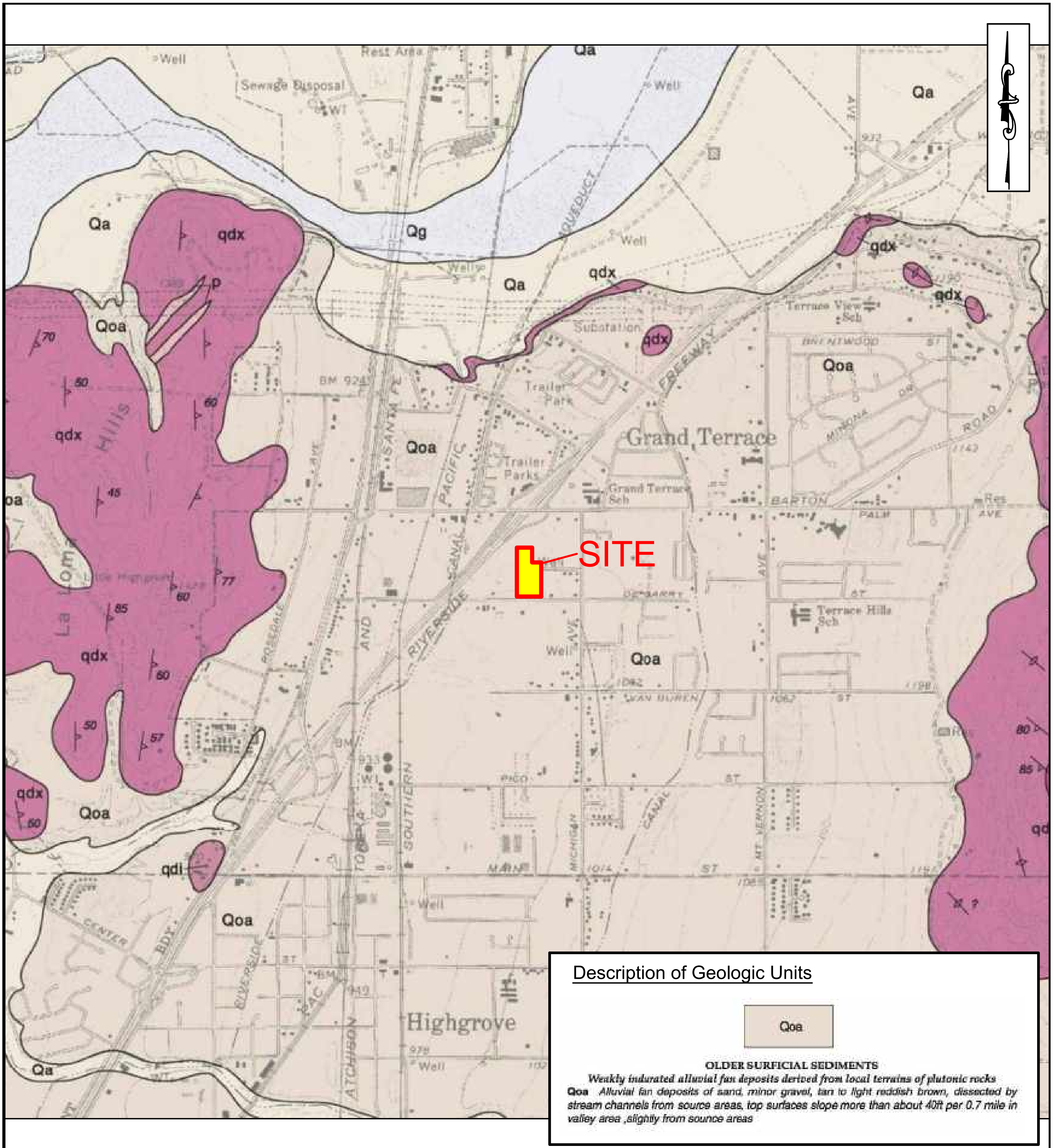
DATE: October 2022

SCALE: 1" ≈ 2,000'



GEOTECHNICAL MAP

PROJECT:	APNS: 1167-161-03 & 04, Grand Terrace, California	**PROJECT NO.:**	33318A.1
CLIENT:	Lewis Management Corp.	**ENCLOSURE:**	A-2
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2022
		SCALE:	1" ≈ 100'



Description of Geologic Units

Qoa

OLDER SURFICIAL SEDIMENTS

Weakly indurated alluvial fan deposits derived from local terrains of plutonic rocks

Qoa *Alluvial fan deposits of sand, minor gravel, tan to light reddish brown, dissected by stream channels from source areas, top surfaces slope more than about 40ft per 0.7 mile in valley area ,slightly from source areas*

REGIONAL GEOLOGIC MAP

(Dibblee, 2003)

PROJECT: APNS: 1167-161-03 & 04, Grand Terrace, California

PROJECT NO.: 33318A.1

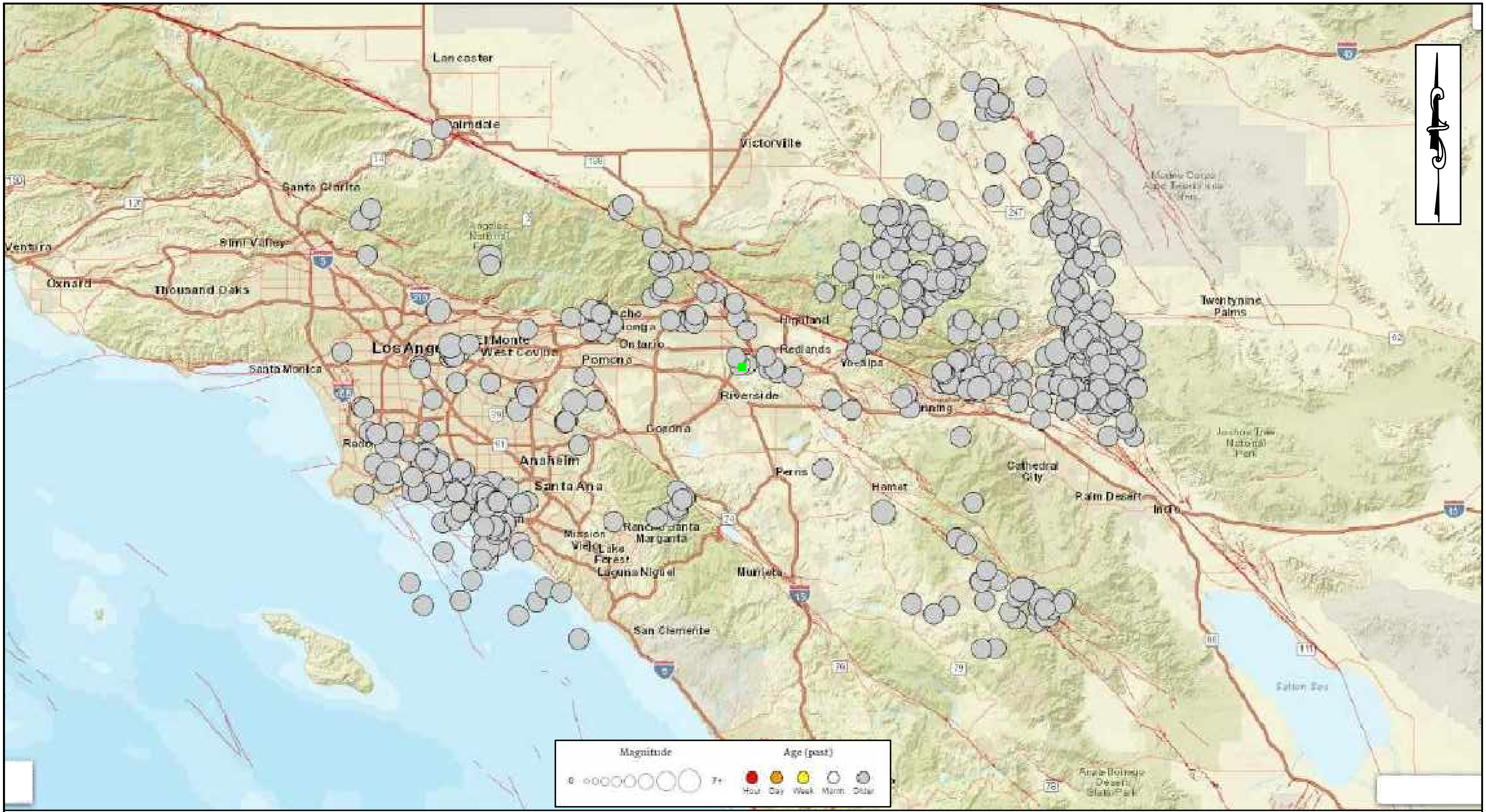
CLIENT: Lewis Management Corp.

ENCLOSURE: A-3

LOR GEOTECHNICAL GROUP, INC.

DATE: October 2022

SCALE: 1" ≈ 2,000'



U.S. Geologic Survey (2022) real-time earthquake epicenter map. Plotted are 524 epicenters of instrument-recorded events from 01/01/32 to present (10/10/22) 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: 1167-161-03 & 04, Grand Terrace, California

PROJECT NO.: 33318A.1

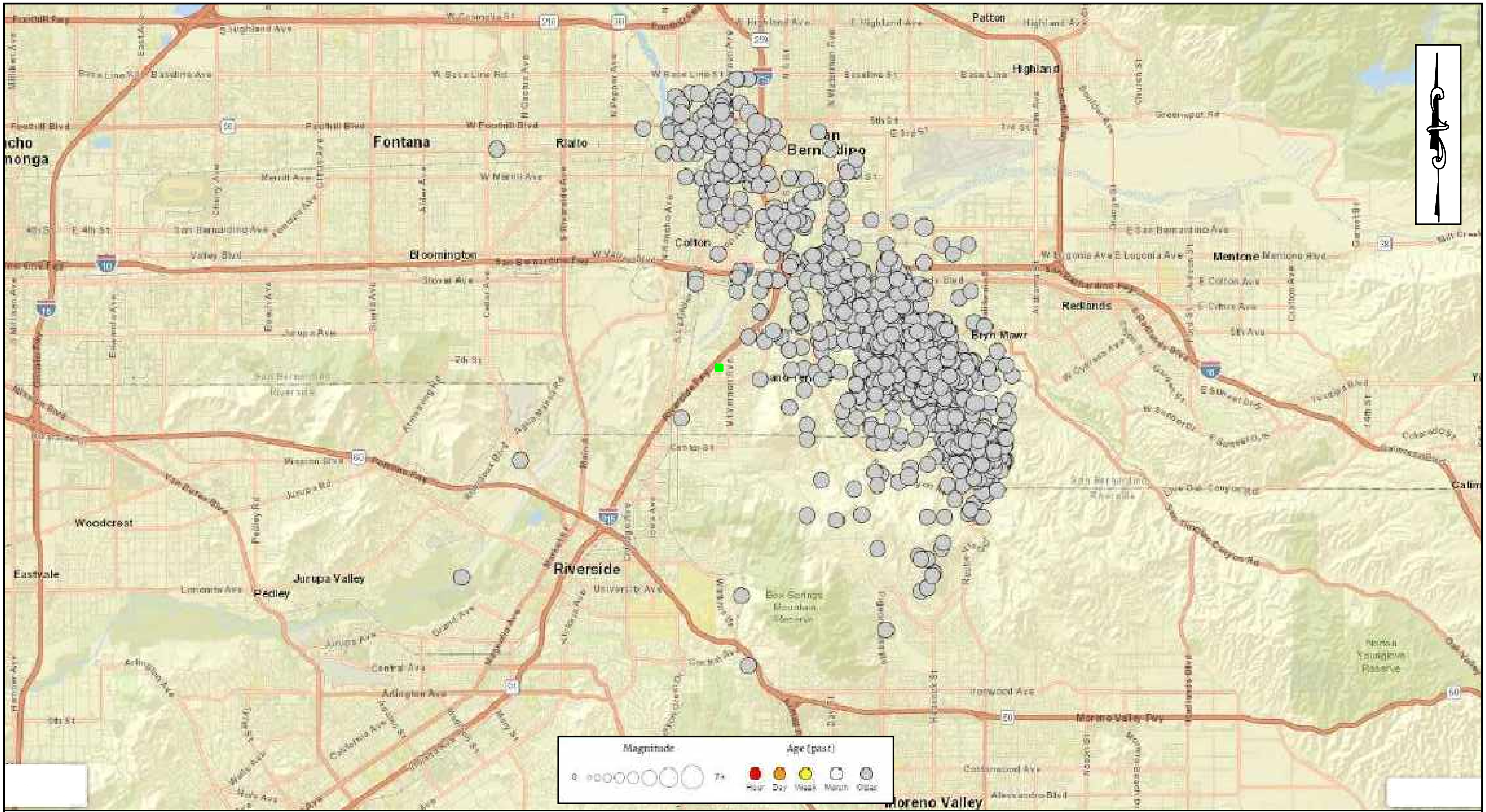
CLIENT: Lewis Management Corp.

ENCLOSURE: A-4

DATE: October 2022

SCALE: 1" ≈ 40km

LOR GEOTECHNICAL GROUP, INC.



U.S. Geologic Survey (2022) real-time earthquake epicenter map. Plotted are 864 epicenters of instrument-recorded events from 01/01/78 to present (10/10/22) of local magnitude 2+ within a radius of ~6.2 miles (10 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 - 10km Radius

PROJECT: APNS: 1167-161-03 & 04, Grand Terrace, California

PROJECT NO.: 33318A.1

CLIENT: Lewis Management Corp.

ENCLOSURE: A-5

LOR **GEOTECHNICAL GROUP, INC.**

DATE: October 2022

SCALE: 1" ≈ 10km

APPENDIX B

Field Investigation Program and Boring and Trench Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on January 6 and 25, 2016 and consisted of excavating two exploratory trenches to depths between 11 and 15 feet below the existing ground surface and advancing two exploratory borings to depths of 16.5 and 51.5 feet below the existing ground surface. The approximate locations of our trenches and borings are shown on Enclosure A-2, within Appendix A.

The trenching exploration was conducted using a John Deere 410G backhoe with a 24-inch bucket. The soils encountered were continuously logged by an engineering geologist from this firm who visually observed the site, maintained detailed logs of the trenches, obtained disturbed soil samples for laboratory evaluation and testing, and classified the soils encountered by visual examination in accordance with the Unified Soil Classification System.

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

The drilling exploration was conducted using a track mounted CME-55 drill rig equipped with 8-inch diameter hollow stem augers. As with the exploratory trenches, the soils encountered within the borings were continuously logged by a geologist from this firm who 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 typical maximum interval of 5 feet. The relatively undisturbed samples were recovered by using a California split barrel sampler of 2.50-inch inside diameter and 3.25-inch outside. The sampler was driven by a 140-pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-values which are included in the boring logs.

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 laboratory for storage and testing. Detailed logs of the trenches and boring are presented on the enclosed Trench and Boring Logs, Enclosures B-1 through B-6. A Boring/Trench Log Legend and Soil Classification Chart are presented on Enclosures B-i and B-ii, respectively.

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: APN'S: 1167-161-03 & -04, GRAND TERRACE, CALIFORNIA

PROJECT NO.: 33318A.1

CLIENT: LEWIS LAND DEVELOPERS, LLC

ENCLOSURE: B-i

DATE: JANUARY 2017

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 ON 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: APN'S: 1167-161-03 & -04, GRAND TERRACE, CALIFORNIA

PROJECT NO.: 33318A.1

CLIENT: LEWIS LAND DEVELOPERS. LLC

ENCLOSURE: B-ii

LOR Geotechnical Group, Inc.

DATE: JANUARY 2017

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								SM	@ 0 feet, TOPSOIL: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% silty fines, brown, moist, loose.
								ML	@ 2 feet, OLDER ALLUVIUM:SANDY SILT, approximately 10% coarse to medium grained sand,, 20% fine grained sand, 70% silty fines, brown, moist, slightly to moderately porous, stiff.
5	69 for 10"		7.8		127.5	■			
									@ 7 feet, becomes lighter in color, slower drilling.
10	67 for 9"		11.7		100.6	■			
15	84 for 10"		8.2		119.5	■			
								SM	@ 17 feet, SILTY SAND, approximately 10% medium grained sand, 50% fine grained sand, 40% silty fines, brown, damp to moist, medium dense to dense.
20	56		7.0		107.1	■			@ 20 feet, becomes sandier.
25	51 for 6"		7.6		105.4	■			@ 25 feet, again silty.
30	97 for 11"		5.6		109.7	■			@ 30 feet, fine grained SILTY SAND, approximately 5% medium grained sand, 65% fine grained sand, 35% silty fines.
35	109		10.5			▥			
40	94		10.8			▥			
45	89		13.6			▥			
								SP	@ 48 feet, POORLY GRADED SAND, approximately 5% coarse grained sand, 10% medium grained sand, 80% fine grained sand, 5% silty fines, light brown, damp, dense.
50	107		1.9			▥			END OF BORING No fill No groundwater No bedrock
55									

PROJECT:	APNs: 1167-161-03 & -04	PROJECT NUMBER:	33318A.1
CLIENT:	Lewis Land Developers, LLC	ELEVATION:	
		DATE DRILLED:	January 18, 2017
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Track Rig
		HOLE DIA.: 8"	ENCLOSURE: B-1

LOG OF BORING B-2

DEPTH IN FEET	TEST DATA								
	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0							▨	SM	**@ 0 feet, ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% siilty fines, brown, damp, loose.**
									@ 4 feet, SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 15% fine grained sand, 70% silty fines, brown, damp, porous, loose.
5	19		5.1		99.9	▮			
10	80 for 11"		11.4		122.4	▮			**@ 10 feet, OLDER ALLUVIUM: SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 20% fine grained sand, 65% silty fines, brown, damp, stiff to very stiff.**
15	71		11.3		110.9	▮			
									END OF BORING **No fill** **No groundwater** **No bedrock**
20									

PROJECT:	**APNs: 1167-161-03 & -04**	PROJECT NUMBER:	**33318A.1**
CLIENT:	**Lewis Land Developers, LLC**	ELEVATION:	
		DATE DRILLED:	**January 18, 2017**
		EQUIPMENT:	**Track Rig**
		HOLE DIA.: **8"**	ENCLOSURE: **B-2**

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-3

	TEST DATA								
DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								**SM**	**@ 0 feet, ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% siilty fines, brown, damp, loose.**
								ML	**@ 4 feet, SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 15% fine grained sand, 70% silty fines, brown, damp, porous, loose.**
5									
									@ 9 feet, OLDER ALLUVIUM: SANDY SILT, approximately 5% coarse grained sand, 20% medium grained sand, 20% fine grained sand, 55% silty fines, brown, damp, stiff.
10	**59**		**8.1**		**125.4**	▮			
								SM	**@ 14 feet, SILTY SAND, approximately 5% medium grained sand, 60% fine grained sand, 35% silty fines, brown, damp, medium dense to dense.**
15	**50**		**7.8**		**118.0**	▮			
									END OF BORING **No fill** **No groudwater** **No bedrock**
20									

PROJECT:	**APNs: 1167-161-03 & -04**	PROJECT NUMBER:	**33318A.1**
CLIENT:	**Lewis Land Developers, LLC**	ELEVATION:	
		DATE DRILLED:	**January 18, 2017**
		EQUIPMENT:	**Track Rig**
		HOLE DIA.: **8"**	ENCLOSURE: **B-3**

LOR GEOTECHNICAL GROUP INC.

LOG OF BORING B-4

DEPTH IN FEET	TEST DATA SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL: **SILTY SAND, fine to coarse grained, damp, loose to medium dense, contains traces of asphalt.**
									@ 3.5 feet, TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% silt and clay, brown, moist, loose to medium dense.**
5	**23**		**11.8**		**120.6**	▮		ML	@ 5 feet, OLDER ALLUVIUM: **SANDY SILT, approximately 10% medium grained sand, 15% fine grained sand, 75% silty fines, brown, damp to moist, non-porous, stiff to very stiff.**
10	**78**		**10.9**		**124.3**	▮			
15	**78 for 11"**		**9.7**		**124.5**	▮			
									END OF BORING **No fill** **No groundwater** **No bedrock**
20									

PROJECT:	**APNs: 1167-161-03 & -04**	PROJECT NUMBER:	**33318A.1**
CLIENT:	**Lewis Land Developers, LLC**	ELEVATION:	
		DATE DRILLED:	**January 18, 2017**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Track Rig**
		HOLE DIA.: **8"**	ENCLOSURE: **B-4**

LOG OF TRENCH T-1

DEPTH IN FEET	TEST DATA LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, <u>TOPSOIL:</u> **SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 40% fine grained sand, 40% silty fines, brown, moist, loose.**
			73	10.0	93.0	▨		ML	@ 2 feet, <u>OLDER ALLUVIUM:</u> **SANDY SILT, approximately 10% medium grained sand, 20% fine grained sand, 70% silty fines, reddish-brown, damp, porous in the upper 2 to 3 feet.**
			82	7.4	104.1	▨			
5									**@ 5 feet, becomes darker in color, non-porous, very stiff, contains minor calcium carbonate.**
									@ 8 feet, hard digging.
10									
									@ 11 feet, pratical refusal. **END OF TRENCH** **No fill** **No caving** **No groundwater** **No bedrock**
15									

PROJECT:	**APNs: 1167-161-03 & -04**	PROJECT NUMBER:	**33318A.1**
CLIENT:	**Lewis Land Developers, LLC**	ELEVATION:	
LOR GEOTECHNICAL GROUP INC.		DATE EXCAVATED:	**January 6, 2017**
		EQUIPMENT:	**John Deere 410G**
		BUCKET W.: **24"**	ENCLOSURE: **B-5**

LOG OF TRENCH T-2

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, <u>ALLUVIUM:</u> **SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% silty fines, brown, damp, loose to medium dense, porous.**
			71	9.5	93.3	▨			
5			73	5.8	95.8	▨			
									@ 8 feet, finer grained sand, less porous.
10									
								SW	**@ 11 feet, WELL GRADED SAND, approximately 20% coarse grained sand, 40% medium grained sand, 35% fine grained sand, 5% silty fines, brown, damp, easy digging, non-cemented**
				5.4		▨			
15									**END OF TRENCH** **No fill** **Slight caving** **No groundwater** **No bedrock**

PROJECT:	**APNs: 1167-161-03 & -04**	PROJECT NUMBER:	**33318A.1**
CLIENT:	**Lewis Land Developers, LLC**	ELEVATION:	
		DATE EXCAVATED:	**January 6, 2017**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**John Deere 410G**
		BUCKET W.: **24"**	ENCLOSURE: **B-6**

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings and trenches were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. Laboratory testing included moisture content, dry density, laboratory compaction, direct shear, sieve analysis, expansion potential, R-Value, and soluble sulfate content. Descriptions of the laboratory tests are presented in the following paragraphs.

Moisture-Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, in accordance with ASTM D 2937 and 2922, and ASTM D 2216, respectively, and the results are shown on the Trench and Boring Logs, Enclosures B-1 through B-6 for convenient correlation with the soil profile.

Laboratory Compaction

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

LABORATORY COMPACTION

Trench/Boring Number	Sample Depth (feet)	Soil Description (U.S.C.S.)	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
T-1	0 - 2	(SM) Silty Sand	130.5	9.5
T-2	2 - 4	(ML) Sandy Silt	127.5	10.5

Direct Shear Tests

Shear tests are performed with a direct shear machine in general accordance with ASTM D 3080 at a constant rate-of-strain (usually 0.05 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 per ASTM D 1557) and soaked, according to conditions expected in the field.

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

DIRECT SHEAR TESTS

Trench/Boring Number	Sample Depth (feet)	Soil Description (U.S.C.S.)	Apparent Cohesion (psf)	Angle of Internal Friction (degrees)
T-1	2 - 4	(ML) Sandy Silt	450	24

Expansion Index Tests

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

EXPANSION INDEX TESTS

Trench/ Boring Number	Sample Depth (feet)	Soil Description	Expansion Index (EI)	Expansion Potential
T-1	2 - 4	(ML) Sandy Silt	29	Low

Expansion Index:	0-20	21-50	51-90	91-130
	Very low	Low	Medium	High

Atterberg Limits

A selected sample of the fine-grained soil units encountered at the site are tested for their Atterberg limits in accordance with ASTM D 4318. The results of these tests are presented on Enclosure C-2.

Sieve Analysis

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

Sand Equivalent

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

R-Value Test

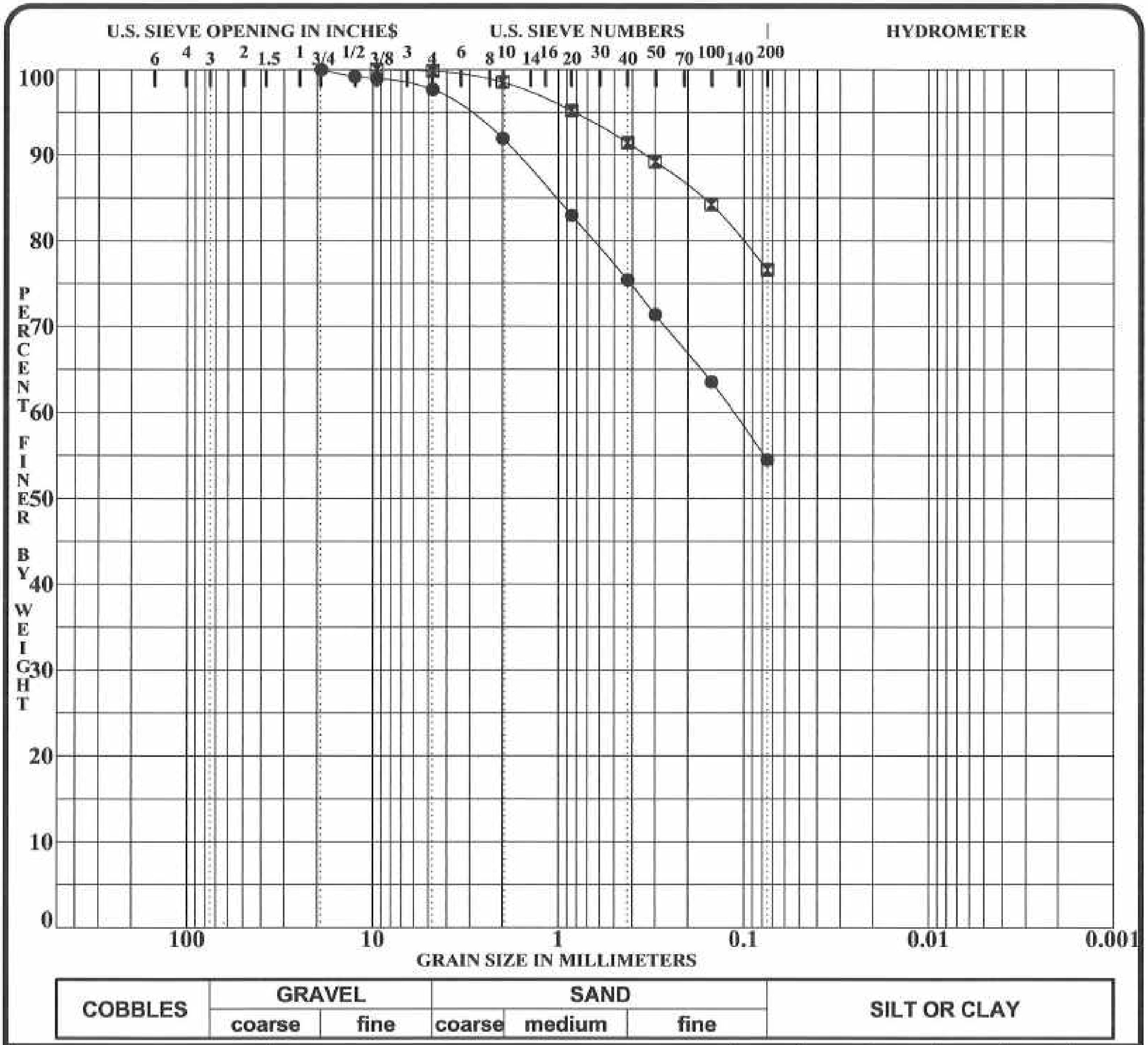
Soil samples were obtained at probable pavement subgrade level and sieve analysis and sand equivalent tests were conducted. Based on these indicator tests, a selected soil sample was tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The results of the sieve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

Soluble Sulfate Content Tests

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

SOLUBLE SULFATE CONTENT TESTS

Trench Number	Sample Depth (feet)	Soil Description	Sulfate Content (percent by weight)
T-1	0 - 2	(SM) Silty Sand	< 0.005
T-1	2 - 4	(ML) Sandy Silt	< 0.005



	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*T-1*	*@ 0 - 2 ft*	*(ML) Sandy Silt*	*8*	*--*				
☒	*T-1*	*@ 2 - 4 ft*	*(ML) Sandy Silt*	*3*	*--*	*12*	*10*		

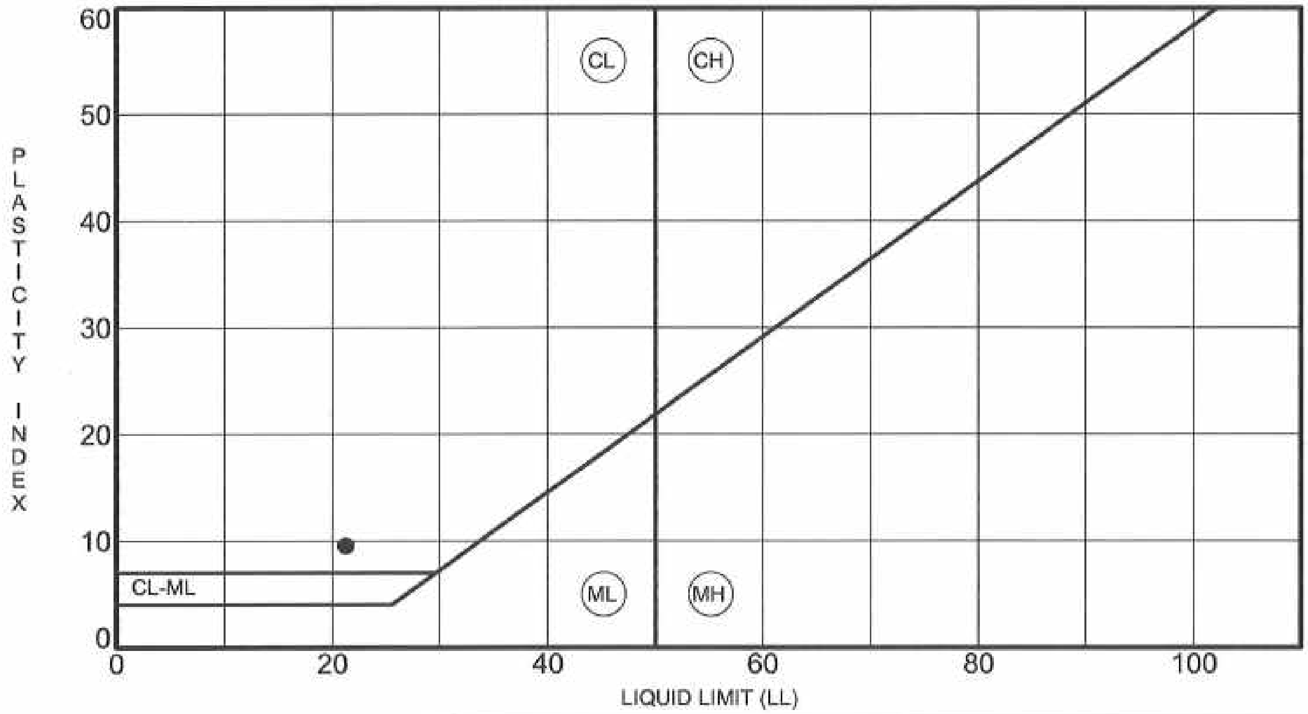
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*T-1*	*@ 0 - 2 ft*	*19.00*	*0.11*			*2.3*	*43.2*	*54.5*	
☒	*T-1*	*@ 2 - 4 ft*	*9.50*				*0.2*	*23.2*	*76.6*	

PROJECT *APNs: 1167-161-03 & -04*

PROJECT NO. *33318A.1*
DATE *2/6/17*

GRADATION CURVES
LOR Geotechnical Group, Inc.

ENCLOSURE C-1



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**T-1**	**@ 2 - 4 ft**	**21**	**12**	**10**	**76.6**	**(ML) Sandy Silt**

Project: **APNs: 1167-161-03 & -04 - Grand Terrace** Project No.: **33318A.1**

Date: **2/6/17**

ATTERBERG LIMITS' RESULTS
LOR Geotechnical Group, Inc.

ENCLOSURE: C - 2

APPENDIX D

Seismic Design Spectra

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

Project: APNs 1167-161-03 and -04, Grand Terrace
Project Number: 33318A.1
Client: Lewis Management Corporation
Site Lat/Long: 34.0313/-117.3250
Controlling Seismic Source: San Jacinto

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	0.197
S_{D1}/S_{DS}	T_S	0.984
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - 2/3*S_{M1}	S_{D1}	1.2383
Equation 11.4-2 - F_V*S_1 [1]	S_{M1}	1.8575

[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	1.888
Design Maps	S_1	0.743
Equation 11.4-1 - F_A*S_S	S_{MS}	1.888*
Equation 11.4-3 - 2/3*S_{MS}	S_{DS}	1.259*
Design Maps	PGA	0.797
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.877*
Section 21.5.3	80% of PGA_M	0.701
Design Maps	C_{RS}	0.920
Design Maps	C_{R1}	0.895

REFERENCE	NOTATION	VALUE
0.2*(S_{D1}/S_{DS})	T_0	0.134*
S_{D1}/S_{DS}	T_S	0.669*
Equation 11.4-4 - 2/3*S_{M1}	S_{D1}	0.8421*
Equation 11.4-2 - F_V*S_1	S_{M1}	1.2631*

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.920
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.895

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

Period	Cr
0.200	0.920
0.300	0.917
0.400	0.914
0.500	0.911
0.600	0.908
0.680	0.905
1.000	0.895

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

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

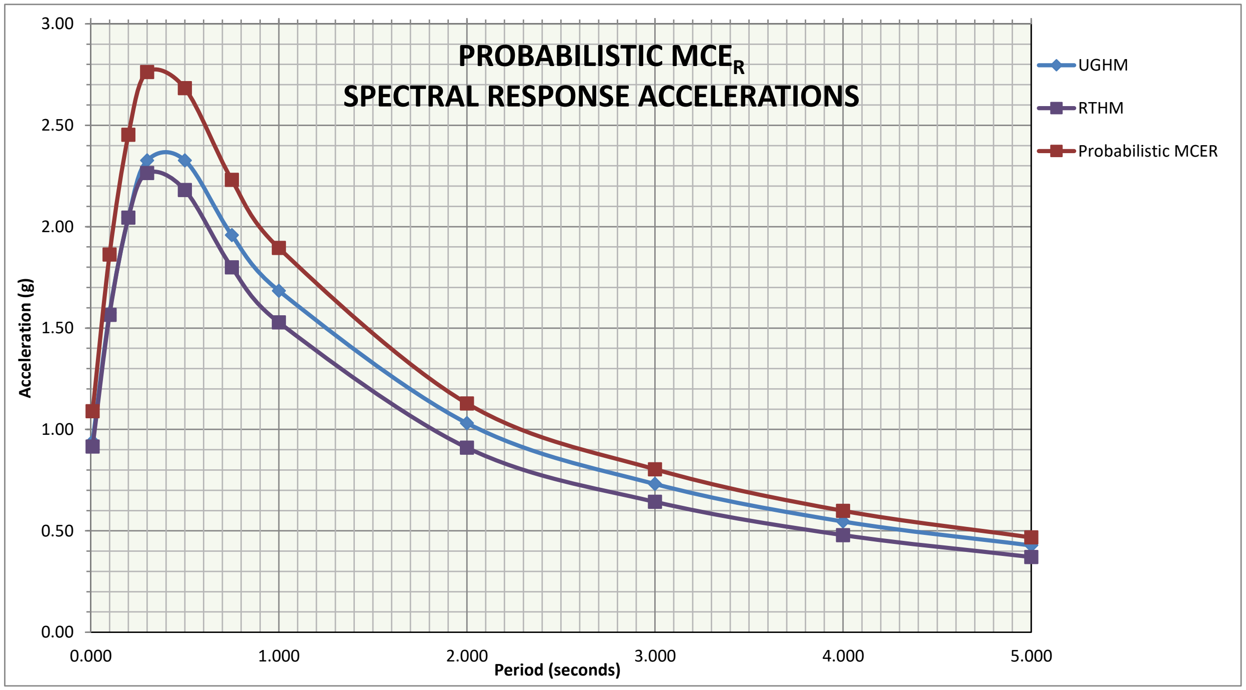
PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

Project No: 33318A.1

Period	UGHM	RTGM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.941	0.916	1.19	1.090
0.100	1.575	1.565	1.19	1.862
0.200	2.044	2.044	1.20	2.453
0.300	2.326	2.264	1.22	2.762
0.500	2.327	2.181	1.23	2.683
0.750	1.958	1.799	1.24	2.231
1.000	1.684	1.528	1.24	1.895
2.000	1.031	0.910	1.24	1.128
3.000	0.731	0.643	1.25	0.804
4.000	0.546	0.479	1.25	0.599
5.000	0.428	0.371	1.26	0.467

Probabilistic PGA: 0.941

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



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

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

DETERMINISTIC SPECTRUM

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

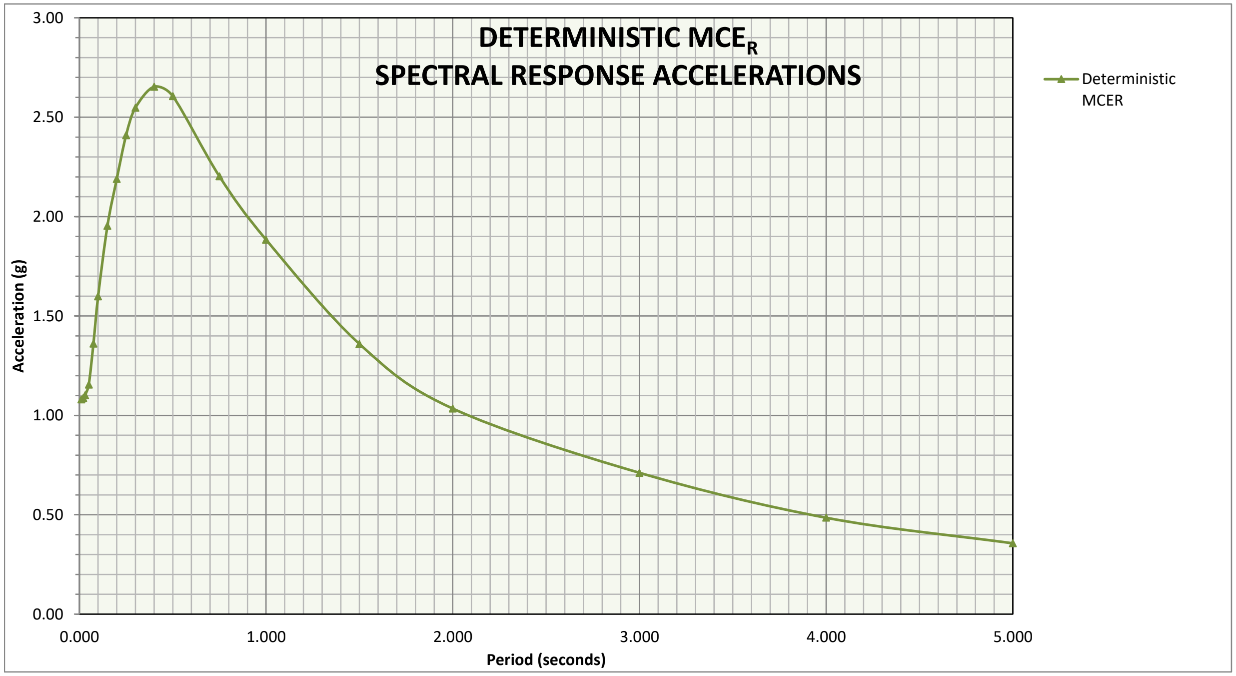
Controlling Source: San Jacinto

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

Project No: 33318A.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	0.907	1.19	1.080	1.080
0.020	0.914	1.19	1.087	1.087
0.030	0.926	1.19	1.102	1.102
0.050	0.971	1.19	1.155	1.155
0.075	1.144	1.19	1.361	1.361
0.100	1.343	1.19	1.598	1.598
0.150	1.628	1.20	1.953	1.953
0.200	1.824	1.20	2.189	2.189
0.250	1.991	1.21	2.409	2.409
0.300	2.088	1.22	2.547	2.547
0.400	2.157	1.23	2.653	2.653
0.500	2.119	1.23	2.606	2.606
0.750	1.776	1.24	2.202	2.202
1.000	1.519	1.24	1.884	1.884
1.500	1.096	1.24	1.359	1.359
2.000	0.834	1.24	1.034	1.034
3.000	0.569	1.25	0.711	0.711
4.000	0.388	1.25	0.485	0.485
5.000	0.283	1.26	0.356	0.356

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	0.907
Is Deterministic PGA >=F_{PGA}*0.5?	YES



LOR GEOTECHNICAL GROUP, INC.

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

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

SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	1.090	1.080	1.080	0.720
0.100	1.862	1.598	1.598	1.065
0.200	2.453	2.189	2.189	1.459
0.300	2.762	2.547	2.547	1.698
0.500	2.683	2.606	2.606	1.737
0.750	2.231	2.202	2.202	1.468
1.000	1.895	1.884	1.884	1.256
2.000	1.128	1.034	1.034	0.689
3.000	0.804	0.711	0.711	0.474
4.000	0.599	0.485	0.485	0.323
5.000	0.467	0.356	0.356	0.238

ASCE 7-16: Section 21.4

Site Specific

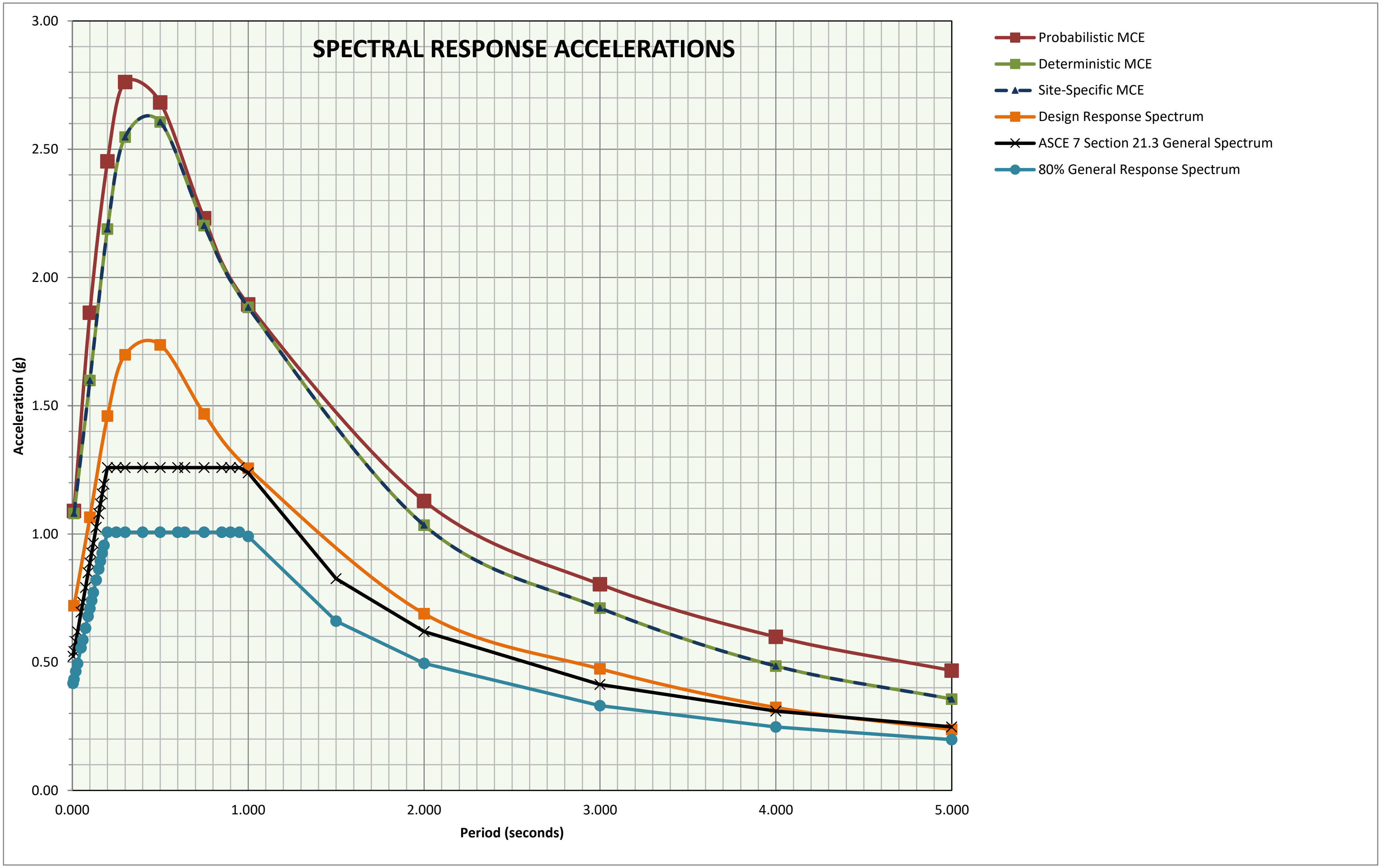
	Calculated Value	Design Value
SDS:	1.564	1.564
SD1:	1.422	1.422
SMS:	2.345	2.345
SM1:	2.133	2.133
Site Specific PGAm:	0.907	0.907
Site Class:	D measured	

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

* Risk Categories I, II, or III

Period	ASCE 7 SECTION 21.3 General Spectrum	80% General Response Spectrum
0.005	0.523	0.418
0.010	0.542	0.433
0.020	0.580	0.464
0.030	0.619	0.495
0.050	0.695	0.556
0.060	0.734	0.587
0.075	0.791	0.633
0.090	0.849	0.679
0.100	0.887	0.710
0.110	0.926	0.741
0.120	0.964	0.771
0.136	1.025	0.820
0.150	1.079	0.863
0.160	1.118	0.894
0.170	1.156	0.925
0.180	1.194	0.955
0.200	1.259	1.007
0.250	1.259	1.007
0.300	1.259	1.007
0.400	1.259	1.007
0.500	1.259	1.007
0.600	1.259	1.007
0.640	1.259	1.007
0.750	1.259	1.007
0.850	1.259	1.007
0.900	1.259	1.007
0.950	1.259	1.007
1.000	1.238	0.991
1.500	0.826	0.660
2.000	0.619	0.495
3.000	0.413	0.330
4.000	0.310	0.248
5.000	0.248	0.198

Project No: 33318A.1



Project No: 33318A.1

LOR GEOTECHNICAL GROUP, INC.