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

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION APN 0239-311-01-0000 LYTLE CREEK AREA SAN BERNARDINO COUNTY, CALIFORNIA

PROJECT NO. 13789.1
FEBRUARY 23, 2022

Prepared For:

Land Engineering Consultants, Inc.
P.O. Box 541
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Attention: Mr. Daniel J. Haskins

February 23, 2022

Land Engineering Consultants, Inc.
P.O. Box 541
Calimesa, California 92320

Project No. 13789.1

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Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Commercial/Light Industrial Project, APN 0239-311-01-0000, Lytle Creek Area, San Bernardino County, California.

LOR Geotechnical Group, Inc., is pleased to present this report summarizing our geotechnical investigation for the above referenced project. This report was based upon a scope of services generally outlined in our Proposal Reference No. 5933, dated November 11, 2021, and other written and verbal communications with you.

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.

It is our opinion that the existing fill/topsoil materials and upper portions of the native alluvial soils will not provide uniform and/or adequate support for the proposed improvements. Therefore, we recommend that a compacted fill mat be constructed beneath footings and slabs. All loose soils should be removed from structural areas and areas to receive structural fills. The data developed during this investigation indicates that removals on the order of 2 to 4 feet will be required within currently proposed development areas.

On-site soils were found to have good R-value quality, good infiltration characteristics, and to contain a negligible soluble sulfate content. Near completion and/or at the completion of site grading, foundation soils should be sampled and tested, as necessary to verify their R-value, expansion potential, and soluble sulfate content, as necessary.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During December of 2021 and January and February of 2022, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed construction of two warehouse/light industrial buildings within APN 0239-311-01-0000, in the Lytle Creek area of San Bernardino County, 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 pertinent geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Geologic field reconnaissance 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;
- Geophysical investigation by our subcontractor, Terra Geosciences, using seismic refraction lines to aid in the identification of possible subsurface faulting;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design;
- Performance of two double ring infiltrometer tests within the proposed infiltration facility location; 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, we were provided with a Site Plan prepared by you. This drawing shows the proposed improvements overlain on a topographic map of the existing conditions and a copy was utilized as a base for our Plat, Enclosure A-2, within Appendix A.

PROJECT CONSIDERATIONS

Information furnished to this firm indicates the proposed project will consist of the construction of two, approximately 15,000 square-foot, metal frame buildings, driveways and parking areas, and associated landscaping within the relatively flat, central portion of the subject 16-acre site. Light to moderate foundation loads are anticipated with the proposed structures. A small infiltration basin to serve the site will be built within the southeast portion.

Based upon our review of the current site topographic conditions, it is anticipated that, excluding removals and/or over-excavations, site grading will consist of minor cuts and fills of less than 5 feet to create the proposed planar building pads.

EXISTING SITE CONDITIONS

The subject site consists of an irregular shaped property comprising 16 acres of land located at 3112 Lytle Creek Road and along the west side of this road. The property largely consists of planar land that slopes gently to the south and east. In the northern and southwest portions, outside of the areas of proposed development area, moderate to steep hillsides are present.

The property currently contains a residence in the southwest corner and a warehouse building in the central-western portion. Water from a spring located to the west-northwest of the site provides water for these buildings while an onsite water well, located in the central-eastern portion, has periodically been used for supplying irrigation water for onsite agricultural purposes.

Contiguous property belonging to the owner of the subject site is present to the north and west with national forest lands beyond. Lytle Creek Road is present along much of the eastern side of the property and a residential property is located adjacent to the southern site boundary. Grapevine Canyon crosses from northwest to southeast across the northern half of the site, north of the area of proposed improvements.

AERIAL PHOTOGRAPH ANALYSIS

During our investigation we reviewed a series of aerial photographs on file at the San Bernardino County Flood Control and Transportation Department aerial photography collection. These photographs consisted of large scale regional photographs taken of the site and surrounding area between 1938 and 1972. In addition, we reviewed historic aerial photographs available through Google Earth (2022) and Historic Aerials (2022).

The San Bernardino County Flood Control aerial photographs reviewed consisted of vertical aerial stereographic photograph pairs of varying scales. These photographs were viewed using stereoscopes with magnifications of 2X and 4X for three dimensional enhancement. Due to the relatively large photograph scales involved, the analysis and subsequent interpretation of detail from aerial photographs sometimes require a degree of subjective judgment. The degree of certainty on the interpretation of details depends upon such factors as the scale and the quality of the photograph. A complete list of the aerial photographs reviewed is presented within the References at the end of this report.

The aerial photographs were examined in detail to assess the local and regional geologic and geomorphic characteristics of the site and vicinity. During our review, we also noted major changes that occurred onsite throughout this time span. Prior to the early 1960's, the site remained in a largely natural condition with the exception of disturbances in the form of clearings and crude dirt roads related to the installation of overhead power lines and towers that appear to have been installed across the property in the early 1930's. In addition, disturbance to the land in the far northern portion of the site, north of Grapevine Canyon, has taken place since the 1860's and mainly during the late 1800's. The disturbances here consist of a hydraulically mined hillside (Texas Hill) and the associated outwash that consists of fill soils between this gold mine quarry and Grapevine Canyon. The onsite structures appear to have been built during the early 1960's and fruit trees and a garden area were located in the area between the residence and the water well during much of the late 1900's.

Flooding damage related to the flood event of 1938 is apparent in the earliest photographs. The northern part of the site adjacent to Grapevine Canyon and portions of Lytle Creek Road were eroded away as a result of this flooding. Some years later, the scars from the flooding related erosion were repaired and earthen levees were built along the south side of the main drainage course for Grapevine Canyon, north of the proposed improvement area.

Our examination of reviewed photographs, including both those available through the county and online, did not identify any evidence for faulting across the area of proposed structural improvements. However, evidence for faulting related to the Lytle Creek fault is apparent within the elevated hillside areas to the south, west, and northwest of the site. Features related to faulting, including various photo-lineaments in the area, are identified on our Enclosure A-5, Photo-lineament Map, in Appendix A.

As seen in the aerial images, the development area of the site appears to be underlain by younger alluvial deposits that slope gently to the southeast overall. As illustrated on our Regional Geologic Maps (Enclosure A-3a and A-3b) and our Greater Site Area Geologic Map (Enclosure A-4), igneous and metamorphic bedrock materials form the base of the hillsides along the western and northern sides of the site and these are capped by elevated older alluvial deposits.

FIELD EXPLORATION PROGRAM

During December of 2021, seismic refraction traverse across the general area of proposed improvements was conducted by our subconsultant, Terra Geosciences, to assist in the evaluation of the possible presence of subsurface faulting related to the San Jacinto fault zone. As presented within Appendix D, the findings and conclusions of that investigation indicate that no faults traverse through the area of proposed development.

Physical subsurface field exploration program was conducted by this firm on January 25, 2022, and consisted of excavating 6 exploratory trenches using a JD 410C backhoe. The trenches were excavated to depths of approximately 5 to 15 feet beneath the existing ground surface. In addition, two double-ring infiltration tests were conducted within shallow test pits excavated within the area of the proposed infiltration basin. The approximate locations of our exploratory trenches and infiltration tests are presented on the attached Plat, Enclosure A-2 within Appendix A.

The subsurface conditions encountered in the exploratory trenches were logged by a geologist from this firm. In-place density tests were taken at select intervals in accordance with the ASTM D 1557, the Sand Cone test method. Bulk samples were obtained at select intervals and returned to our geotechnical laboratory in sealed containers for further testing and evaluation. A detailed description of the field exploration program and the trench logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

The site is located within the far eastern portion of the San Gabriel Mountains of the Transverse Ranges geomorphic province. These mountains are underlain by Precambrian to Miocene age igneous and metamorphic bedrock. From late Tertiary through Holocene time, the region has undergone substantial uplift, as evidenced by numerous landslides and elevated older alluvial deposits.

In the Lytle Creek area of the Transverse Ranges province, the San Andreas fault, as well as the San Jacinto fault, act as the boundary between the San Gabriel Mountains to the west and the San Bernardino Mountains to the east. East of the site, but on the northern side of the San Andreas fault and east of the Cajon Pass, are the San Bernardino Mountains portion of the Transverse Ranges province.

The San Gabriel Mountains are situated between the Sierra Madre/Cucamonga fault zone on the south and the San Andreas fault on the north. Portions of the San Jacinto fault become the San Gabriel fault to the west of the site and the Cucamonga fault crosses the mouth of Lytle Creek canyon before merging with the San Jacinto fault to the southeast of the property. Locally, the motions on these various faults has resulted in the San Gabriel Mountains moving to the northwest relative to the San Bernardino Mountains. While this motion is distributed along a very wide shear zone of various other major faults in the region, perhaps as much as one half of the total offset is thought to have occurred along the San Andreas fault.

The San Jacinto and San Andreas faults dominate the local area in terms of seismic potential and fault rupture hazard. While there are other faults in the local area that could cause significant ground shaking, none of these are of near equal significance due to their greater distances and/or lower magnitudes. The regional geology as mapped by the U.S.G.S. (Morton and Miller, 2001) and partial legend is shown on Enclosure A-3a, within Appendix A. In addition, a second Regional Geologic Map, prepared by Dibblee and Minch (2003) is presented as Enclosure A-3b within Appendix A. This map is provided mainly to show the location of an inferred fault that traverses the site as mapped by these authors.

Site Geologic Conditions

The site is underlain by a relatively thin layer of fill/topsoil and near surface soils that have been disturbed through past agricultural use and bioturbation. These are on the order of 1 to 3 feet thick and consist of loose to medium dense silty sand with gravel, cobbles, and boulders.

Beneath the topsoil materials, alluvial deposits ranging from silty sand with gravel in the near surface and poorly graded sands with locally abundant gravels and cobbles below. At depth, numerous cobble to boulder sized rocks were encountered. Rock materials within the alluvium consist mostly of subrounded intrusive igneous rocks.

Although not encountered during our subsurface investigation within the area of proposed improvements, other areas of the site include additional geologic units, including undocumented fill soils, active wash deposits, elevated older alluvial deposits, as well as intrusive and metamorphic bedrock. The general configuration of the geologic conditions across the overall 16-acre site are depicted on our General Site Area Geologic Map, Enclosure A-4.

Groundwater Hydrology

Groundwater was not encountered in our exploratory trenches as advanced to a maximum depth of approximately 15 feet below the existing ground surface. The onsite water well was measured to a depth of 295 feet without water or a bottom encountered. However, the property owner reports that the water table was at a depth of approximately 83 feet when the depth to water in this well was last checked, about 2 to 3 years ago.

According to information available from the California Department of Water Resources (DWR, 2022) and Watermaster Support Services, et al (2021), there are three wells for which they have records within close proximity of the site and each of these is located within the Lytle Creek wash to the northeast and southeast. For each of the last approximately 7 years, the depth to groundwater in these wells has averaged about 75 to 95 feet below the surface. Considering this and the above information, groundwater does not appear to be a factor that could impact site development.

Surface Runoff

Current surface runoff of precipitation waters across the site is generally as sheetflow to the south-southeast.

Mass Movement

The site lies on a relatively flat surface. Mass movement failures such as landslides, rockfalls, or debris flows within the site vicinity are not known to exist and no evidence of mass movement was observed on the site. The westerly of the two proposed buildings is to be located, at its closest point, approximately 40 feet from the toe of a natural hillside that rises approximately 150 feet to the west at fairly steep gradients. However, no evidence for mass movement features was noted during our site reconnaissance or review of aerial photographs and no landslides are shown to be present in this area on regional geologic maps that cover the site area.

Faulting

According to the Official Maps of Alquist-Priolo Earthquake Fault Zones of California (Hart and Bryant, 1997) portions of the subject site lie within a current State of California Earthquake Fault Zone. In addition, a fault mapped as a portion of the local Lytle Creek fault of the San Jacinto fault zone is mapped as projecting toward and terminating just southeast of the southern site boundary. The approximate site boundary is overlain on a copy of Figure 1 that accompanies California Division of Mines and Geology Fault Evaluation Report FER 240 (1994) that was used to develop the current Earthquake Fault Zone maps for this area and is presented as Enclosure A-6 in Appendix A.

Due to the presence of the above mentioned fault that projects toward the proposed development area of the site, we enlisted the geophysical services of Terra Geosciences to conduct a seismic refraction line traverse across the general area of proposed development and in an approximate perpendicular orientation to the projection of this mapped fault. As outlined within the report prepared by Terra Geosciences, in Appendix D, no evidence for the presence of subsurface faults was identified by this study.

During the course of this investigation, the proposed building locations shifted slightly and the current proposed location for the northwesterly of the two buildings now appears to be across the approximate northeast Earthquake Fault Zone boundary. However, the geophysical study conducted covers the area of the proposed building.

Our review of geologic literature pertinent to the site and review of aerial photographs identified one map (Dibblee and Minch, 2003; see Enclosure A-3b) that shows an inferred extension of the Lytle Creek fault as the San Jacinto fault projecting through the western portion of the relatively flat lying potions of the site, along the base of the hillside and through the large cut exposure that identifies Texas Hill in the northwest portion of the property. However, no other authors show this inferred fault on any of their maps and the feature is not recognized by the State of California on the latest Earthquake Fault Zone maps. We did not find any evidence for this feature during our review of aerial imagery or during our site geologic mapping, including observation of the vertical exposures within older alluvial soils within which the hydraulic mining scars of Texas Hill were made and where this inferred fault is projected to extend across.

Reconnaissance mapping of the greater site area revealed that the strongest evidence for faulting is present to the northwest and southwest of the area of proposed development. Some of these features are identified on our Photo-lineament Map, Enclosure A-5, while direct observation of one, if not the strongest, fault related features may be observed within a road cut exposure located approximately 1,000 feet to the west-northwest of the area of proposed development (location noted on Enclosure A-5).

The closest known active earthquake fault with a documented location is the Lytle Creek fault of the San Jacinto fault zone located northwest and southwest of the area of development. Two other strands of the San Jacinto fault zone, each referred to as the San Bernardino Valley Sections, are located at distances of 1.6 kilometers (1 mile) and 3.5 kilometers (2.2 miles) to the northeast. In addition, other relatively close active faults include the Cucamonga fault located approximately 1.9 kilometers (1.2 miles) to the south and the San Andreas fault located approximately 5.9 kilometers (3.7 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. It is believed that the San Jacinto fault zone has an average slip rate of about 12 mm/year and is capable of producing an earthquake magnitude on the order of 6.5 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 Cucmonga fault is capable of producing an earthquake magnitude on the order of 7.0.

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

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 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 USGS. 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 data base contained data from 1932 through February 18, 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-7, within Appendix A, the site lies within a relatively active region associated with the San Jacinto and San Andreas fault zones trending southeast to northwest.

In the second search, the micro seismicity of the area lying within a 10 kilometer (6.2 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 result of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 40± years on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid 1970s are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-8, the San Jacinto fault zone appears to be the source of numerous events.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the San Jacinto and San Andreas faults. 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 loose granular sediments where the depth to groundwater is usually less than 50 feet. As the site is underlain by relatively medium dense alluvial materials and the depth to groundwater is thought to be in excess of 50 feet, 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 an earthquake and affect the site by flooding.

Seismically-Induced Landsliding: Due to the low relief of the site and lack of landsliding noted during this investigation of the ajacent hillside, 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 medium dense alluvial materials, the potential for settlement is considered low. In addition, the earthwork operations recommended to be conducted during the development of the site will mitigate any near surface loose soil conditions.

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2019)

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 previous 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 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 presented in Appendix F.

INFILTRATION TESTING AND TEST RESULTS

A total of 2 double ring infiltration tests were conducted within the area of a proposed basin as illustrated on the enclosed Plat, Enclosure A-2, located in Appendix A. The testing was conducted at the approximate bottom elevations for the proposed system.

A 12-inch diameter steel casing (ring) was installed within the center of each test location, with a 24-inch diameter steel ring centered around it. Each ring was imbedded approximately 4 inches. These rings extended approximately 16 inches above the bottom of each test location. Each test location was tested immediately after the rings were installed by filling both the inside and outside rings and maintaining a water level to a depth of approximately 2 inches above the ground surface. Water was then metered into the test hole to maintain this water level within both rings. The volume of water used in a given time period was recorded at various time intervals to establish the infiltration rate of water within the inner ring.

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

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

Infiltration Test No.	Depth (ft.)*	Clear Water Infiltration Rate** (inches/hour)
DRI-1	2	17.0
DRI-2	5	18.0
* below existing ground surface ** average of final 4 readings rounded to the nearest tenth		

The results of our double ring infiltrometer tests are attached as Enclosures E-1 and E-2, located in Appendix E.

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 improvements to the site are 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 trenches are indicative of the locations explored. 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 in order that we may assess the impact to the recommendations provided.

Foundation Support

Based upon our field investigation and laboratory test data, it is our opinion that the existing fill/topsoil materials and upper portions of the native alluvial soils will not, in their present condition, provide uniform and/or adequate support for the proposed structures. However, the alluvial soils at depths greater than approximately 2 to 4 feet are considered suitable for support of the proposed structures and/or structural fills.

To provide adequate support for the proposed structures, we recommend that a minimum 24-inch thick compacted fill mat be constructed beneath slabs on grade and footings and that this fill mat bear upon medium dense to dense alluvial materials. Conventional foundations, 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 compacted fil mat or when bearing uniformly upon medium dense to dense older alluvial soils.

Soil Expansiveness

As noted by our explorations and laboratory testing, the majority of the site surficial soils consist of silty sands and sands with gravel, cobbles, and local boulders. These materials are considered to have a very low expansion potential. Therefore, conventional design and construction should be applicable for the project.

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

Geologic Mitigations

No special geologic recommendation 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 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).

RECOMMENDATIONS

Geologic Recommendations

It is our recommendation that the bottom of all removal/over-excavation areas be observed by the project geologist in order to determine if any geologic features indicative of the possible presence of faulting are present and to provide any supplemental recommendations that could become warranted. No other special geologic recommendation methods are deemed necessary at this time, other than the geotechnical recommendations provided in the following sections.

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the owner, 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.

It is our recommendation that all existing fill/topsoil 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. Any undocumented fills encountered during grading should be completely removed and cleaned of significant deleterious materials. These may then be reused as compacted fill.

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

Initial Site Preparation

All existing fill/topsoil and loose alluvial materials should be removed from structural areas and areas to receive structural fills. The data developed during this investigation indicates that for the majority of the site, removals on the order of 2 to 4 feet will be required to encounter competent alluvium. Competent alluvium is defined as damp, relatively dense materials with a minimum in-place relative compaction of 85 percent (ASTM D 1557). Removals should extend at a distance equal to the depth of the removals plus proposed fill and at least a minimum of 5 feet. The actual depths of removals should be determined during the grading operation by observation and in-place density testing. Locally, greater removals may be required.

Preparation of Fill Areas

After the removals of the loose, unsuitable portions of the alluvial materials as described above and prior to placing fill, the surfaces of all areas to receive fill should be scarified to a depth of at least 6 inches. The scarified soil should be brought to near optimum moisture content and compacted to a relative compaction of at least 90 percent (ASTM D 1557).

Preparation of Foundation Areas

All footings should rest upon a minimum of 24 inches of properly compacted fill material placed over competent alluvium. Based on the recommended removals discussed above, it is anticipated that this will be accomplished in most areas. However, in areas where the required fill thickness is not accomplished by the removal of any surficial fill and loose alluvial materials and site rough grading, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines. The bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to near optimum moisture content, and compacted to at least 90 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill.

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil. The remedial grading recommended above is anticipated to accomplish the minimum 24 inches of compacted fill. The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Engineered Compacted Fill

The on-site soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in fills.

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

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

Based upon the relative compaction of the near surface 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. Therefore, 1.10 cubic yards to 1.15 cubic yards of in-place materials would be necessary to yield one cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 feet. These values are for estimating purposes only, and are exclusive of losses due to stripping or the removal of subsurface obstructions. These values may vary due to differing conditions within the project boundaries and the limitations of this investigation. Shrinkage should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

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

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

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.

Infiltration

Based upon our field investigation and infiltration test data, a clear water absorption rate of 10 inches per hour appears warranted for design. A factor of safety should be applied as indicated by the Design Handbook for Low Impact Development Best Management Practices (RCFCWCD, 2011). The design infiltration rate should be adjusted using a factor of safety 3.0.

To ensure continued infiltration capability of the infiltration area, a program to maintain the facility should be considered. This program should include periodic removal of accumulated materials, which can slow the infiltration considerably and decrease the water quality. Materials to be removed from the catch basin areas typically consist of litter, dead plant matter, and soil fines (silts and clays). Proper maintenance of the system is critical. A maintenance program should be prepared and properly executed. At a minimum, the program should be as outlined in the Design Handbook for Low Impact Development Best Management Practices (RCFCWCD, 2011).

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

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

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

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

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

For the minimum width and depth, footings may be designed using a maximum net soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads incorporating a factor of safety of 3.0. Soil bearing pressure may be increased 300 psf for every foot of width and 900 psf for every foot of depth up to a maximum soil bearing pressure of 4,000 psf for dead plus live loads. 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.

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 490 pounds per square foot per foot of depth. Base friction may be computed at 0.42 times the normal load. Base friction and passive earth pressure may be combined without reduction.

Footings on very low expansive soils will not required any particular reinforcement from the geotechnical standpoint.

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 slabs-on-grade should bear on a minimum of 12 inches of compacted soil. The final pad surfaces should be rolled or track-walked to provide fairly smooth, dense surfaces upon which to place the concrete.

Since very low expansive soils are anticipated to underlying slab areas, no particular geotechnical and/or structural mitigation measures to control expansive soil problems will be required.

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 and two inches of sand below 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).

Since very low expansive soils are anticipated to underlie flatwork areas, no particular geotechnical and/or structural mitigation measures to counteract expansive soil problems will be required.

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 structures 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 30 pounds per square foot (psf) per foot of depth be used. This assumes level backfill consisting of recompacted, non-square expansive, native soils placed against the structures and with the backcut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

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

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

Sulfate Protection

The results of the soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels are presented on Enclosure C.

Based on the test results it appears that there is a negligible sulfate exposure to concrete elements in contact with on site soils. The CBC, therefore, does not recommend special design criteria for concrete elements in conduct with such materials.

Preliminary Pavement Design

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

AREA	T.I.*	DESIGN R-VALUE	PRELIMINARY SECTION
Car Parking Areas and Access Lanes (ADTT=1)	5.0	50	0.25' AC / 0.35' AB 4.0" PCC / 4.0" AB
Parking and Drive Areas (light vehicular traffic and occasional truck traffic) (ADTT=10)	6.0	50	0.25' AC / 0.35' AB 5.0" PCC / 4.0" AB
Entrance and Service Lanes (ADTT=25)	7.0	50	0.30' AC / 0.45' AB 5.5" PCC / 4.0" AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base PCC-Portland Cement Concrete, MR = 550 psi *Actual Traffic Index should be determined by others			

The above structural section is 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 designed utilizing a Traffic Index of 7.0 with a minimum of 5.5-inch thick concrete, with a 4-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 5.5-inch thick portland cement concrete (PCC) pavement section should have a minimum modulus of rupture (MR) of 550 pounds per square inch (psi).

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. Improvement of the R-value quality of the soils may be provided through mixing with granular soils observed on-site.

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 herein have been incorporated into the design. Additional expansion index and soluble sulfate testing may be required after the site is rough graded.

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

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavation prior to 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.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Land Engineering Consultants, Inc., 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 in order that 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 us as your convenience.

Respectfully submitted,
LOR Geotechnical Group, Inc.

Robert M Markoff, CEG 2073
Engineering Geologist

John P. Leuer, GE 2030
President

RMM:JPL:ss

Distribution: Addressee (2) and via email dan@lecincorporated.com

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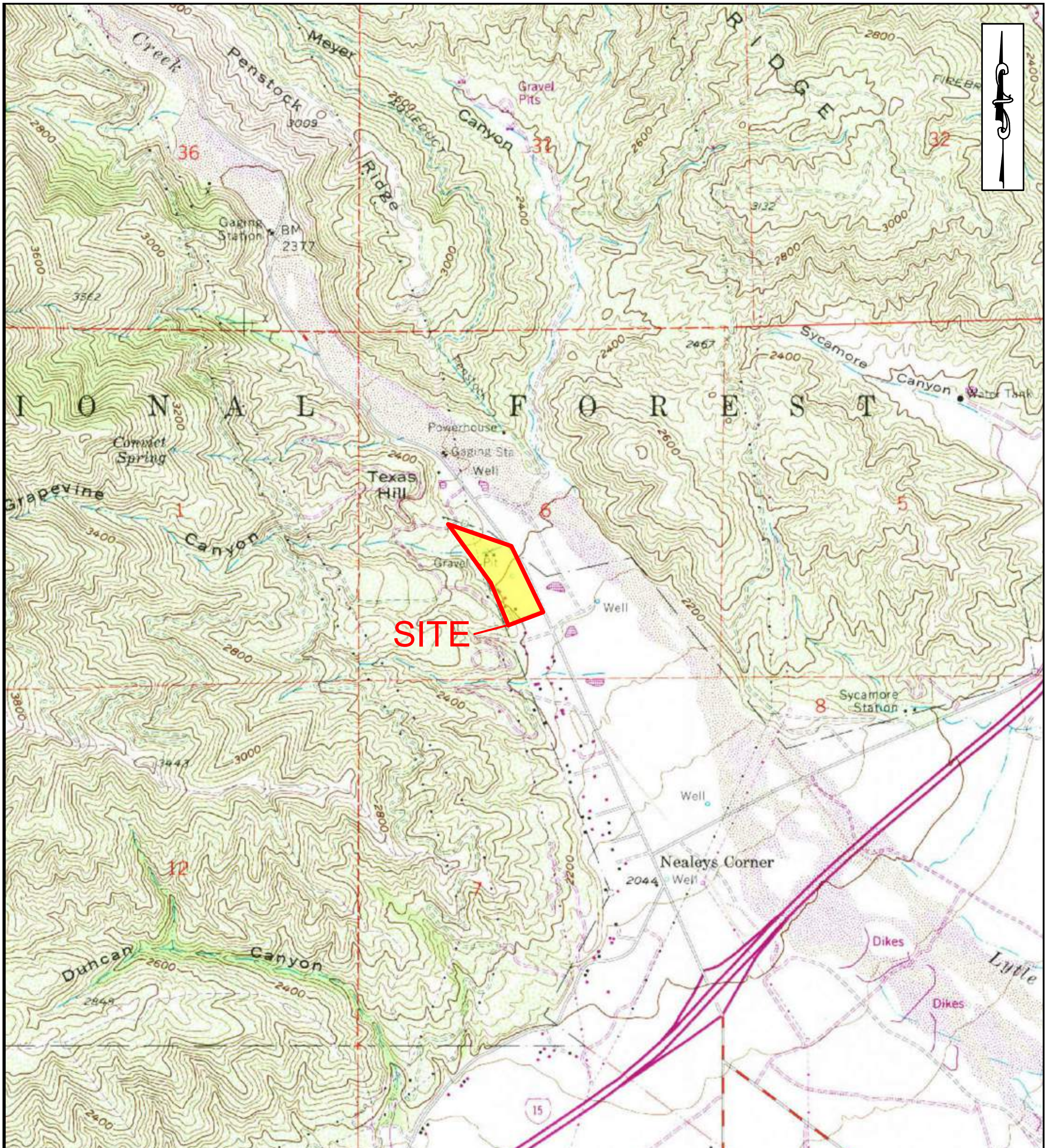
AERIAL PHOTOGRAPHS

SAN BERNARDINO COUNTY FLOOD CONTROL DISTRICT			
DATE	**FLIGHT NO.**	**PHOTO NO(S).**	**SCALE 1"=**
1938	W-73	I-3-7, I-3-8, and I-3-9	1,200'
July 9, 1938	AXL 63	75 and 76	1,600'
November 10, 1955	F-34	7-25 and 7-26	1,200'
May 22, 1962	C-16	26 and 27	1,000'
January 1, 1966	C-144	1-22 through 1-25	500'
April 17, 1967	C-132	25, 26	1,000'
February, 1969	C-295	8 and 9	2,000'
January 24, 1972	C-182	1 through 4	1,000'

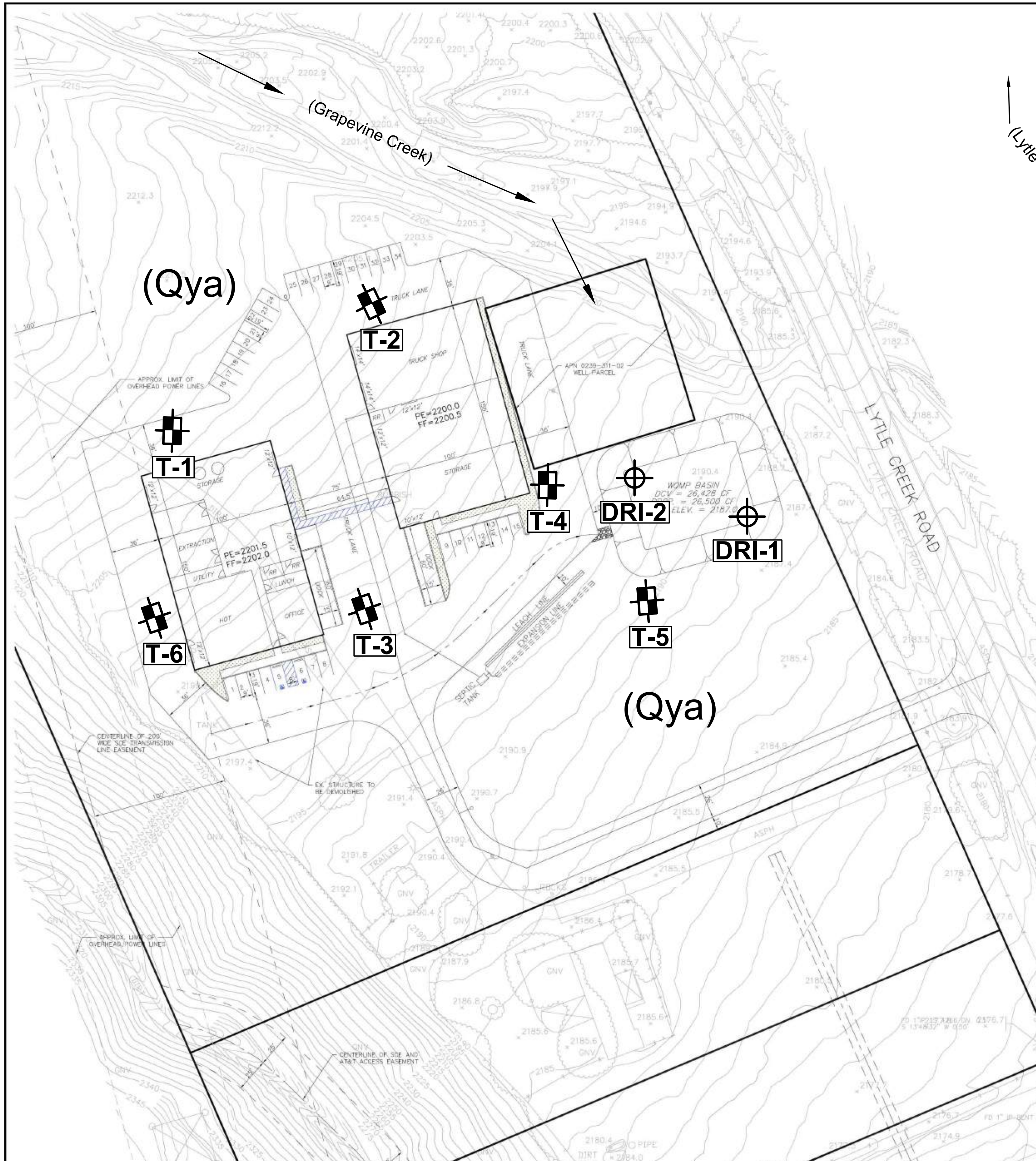
APPENDIX A

Index Map, Plat, Regional Geologic Maps,
Greater Site Area Geologic Map, Photo-Lineament Map,
Earthquake Fault Zone Map, and
Historical Seismicity Maps



INDEX MAP

PROJECT:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'



Legend

(Locations Approximate)

Map Symbols

- **T-6** - Exploratory Trench
- **DRI-2** - Double-Ring Infiltration Test

CONDITIONAL USE PERMIT - SITE PLAN

BEING ALL THAT PORTION OF

LAND ENGINEERING CONSULTANTS, INC. JANUARY, 2022

OWNERS/APPLICANT:
RON SPEARS, APPLICANT
MOUNTAIN AVENUE BEES, INC
5981 LAYTON STREET
ALTA LOMA, CA 91737
PH: (XXX) XXX-XXXX

ENGINEER/MAP PREPARER:
LAND ENGINEERING CONSULTANTS, INC.
P.O. BOX 541, 650 AVENUE K
CALIMESA, CA. 92320
PH: (909) 795-8882
EMAIL: STEVE@LECINCORPORATED.COM

LOCAL AGENCIES SERVICING THIS AREA:

WATER:
WEST VALLEY WATER DISTRICT
855 W BASELINE ROAD
RIALTO, CA 92376
(909) 875-1804

GAS:
PROPANE

SEWER:
SEPTIC

POWER:
SOUTHERN CALIFORNIA EDISON
1351 E. FRANCIS ST
ONTARIO, CA. 91761
(714) 947-2996

TELEPHONE:
FRONTIER
3281 E. GUASTI RD. #700
ONTARIO, CA 91761
PH: (909) 312-4400

CABLE:
FRONTIER
3281 E. GUASTI RD. #700
ONTARIO, CA 91761
PH: (909) 312-4400

LEGAL DESCRIPTION:
REAL PROPERTY IN THE UNINCORPORATED AREA OF THE COUNTY OF SAN BERNARDINO, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

SEE GRANT DEED PER DOC NO. 2020-0170602 OF OFFICIAL RECORDS.

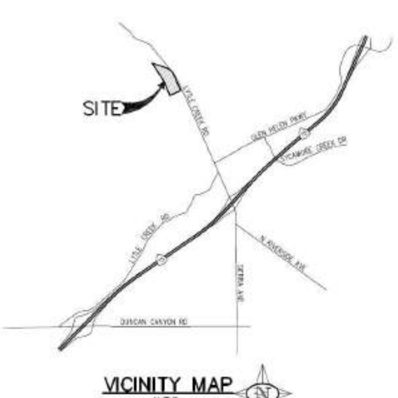
PROJECT SCOPE:
THIS PROJECT PROPOSES TO CONSTRUCT TWO APPROXIMATELY 15,000 SF CONCRETE TILT-UP BUILDINGS.

PARKING ANALYSIS:

USE:	CHAPTER 83.11:	REQUIRED:	PROVIDED:
OFFICE:	1 SPACE PER 250 SF (1,307/250)	5 SPACES	5 SPACES
WAREHOUSE:	1 SPACE PER 1,000 SF (28,683/1,000)	29 SPACES	29 SPACES
LOADING:	1 SPACE PER 5,000 SF (30,000/5,000)	4 SPACES	2 SPACES
		38 SPACES TOTAL	38 SPACES TOTAL
DISABLED:	TOTAL SPACES BETWEEN 26-50	2 SPACES (1 VAN ACCESS)	2 SPACES
ELECTRIC VEHICLE:			

LOT COVERAGE/FLOOR AREA RATIO ANALYSIS:

LANDSCAPING STANDARDS ANALYSIS:



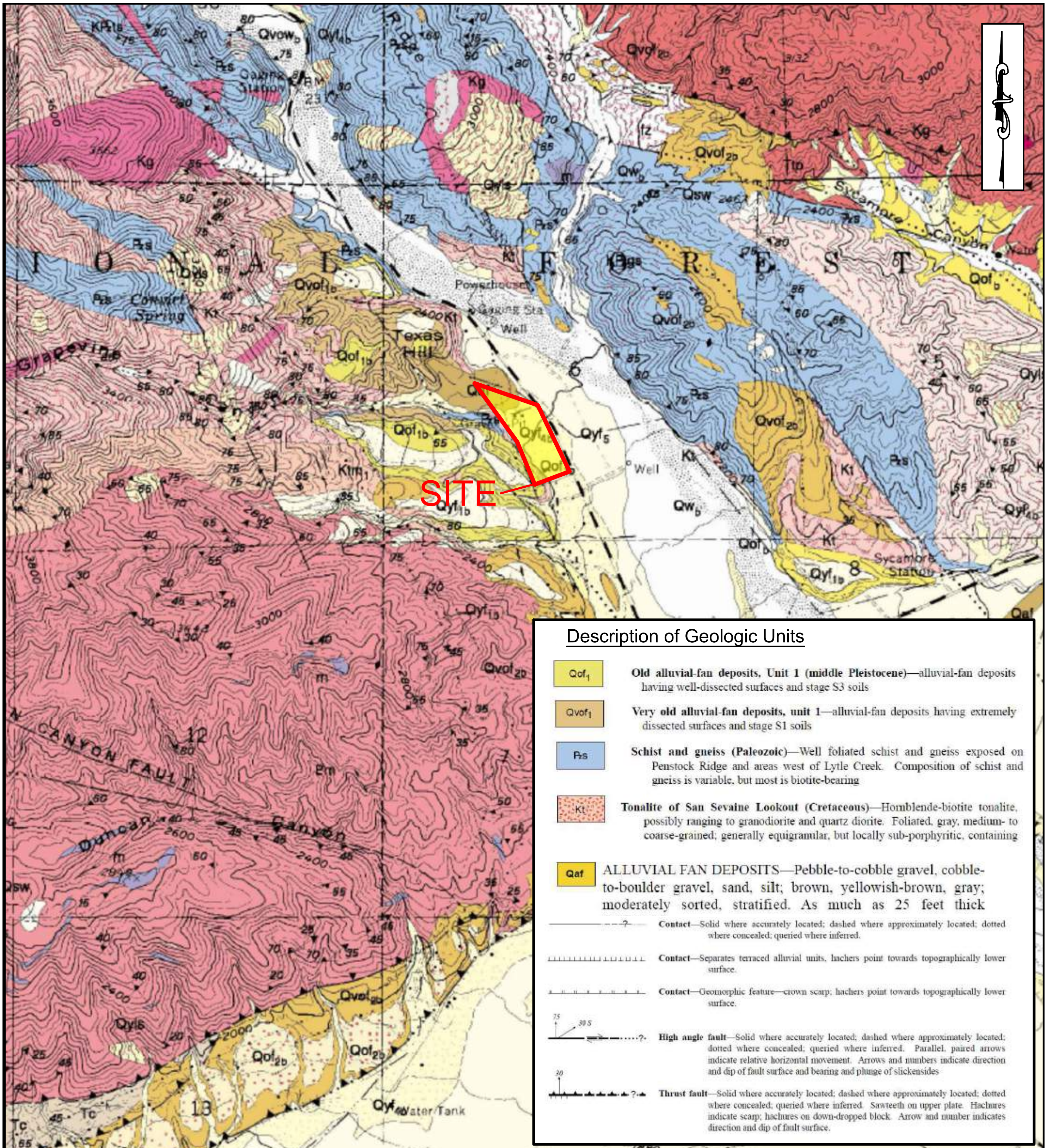
SCALE: 1"=30'

LEC LAND ENGINEERING CONSULTANTS, INC.
P.O. BOX 541, 650 AVENUE K
CALIMESA, CALIFORNIA 92320
TEL: 909-795-8882 FAX: 909-795-8816
STEVEN H. RITCHEY R.C.E. 51129, EXP. 9/30/22

PLAT

PROJECT NO.:	13789.1
ENCLOSURE:	A-2
DATE:	February 2022
SCALE:	1" ≈ 170'
PROJECT:	APN 0239-311-01-0000, Lytle Creek, California
CLIENT:	Land Engineering Consultants, Inc.

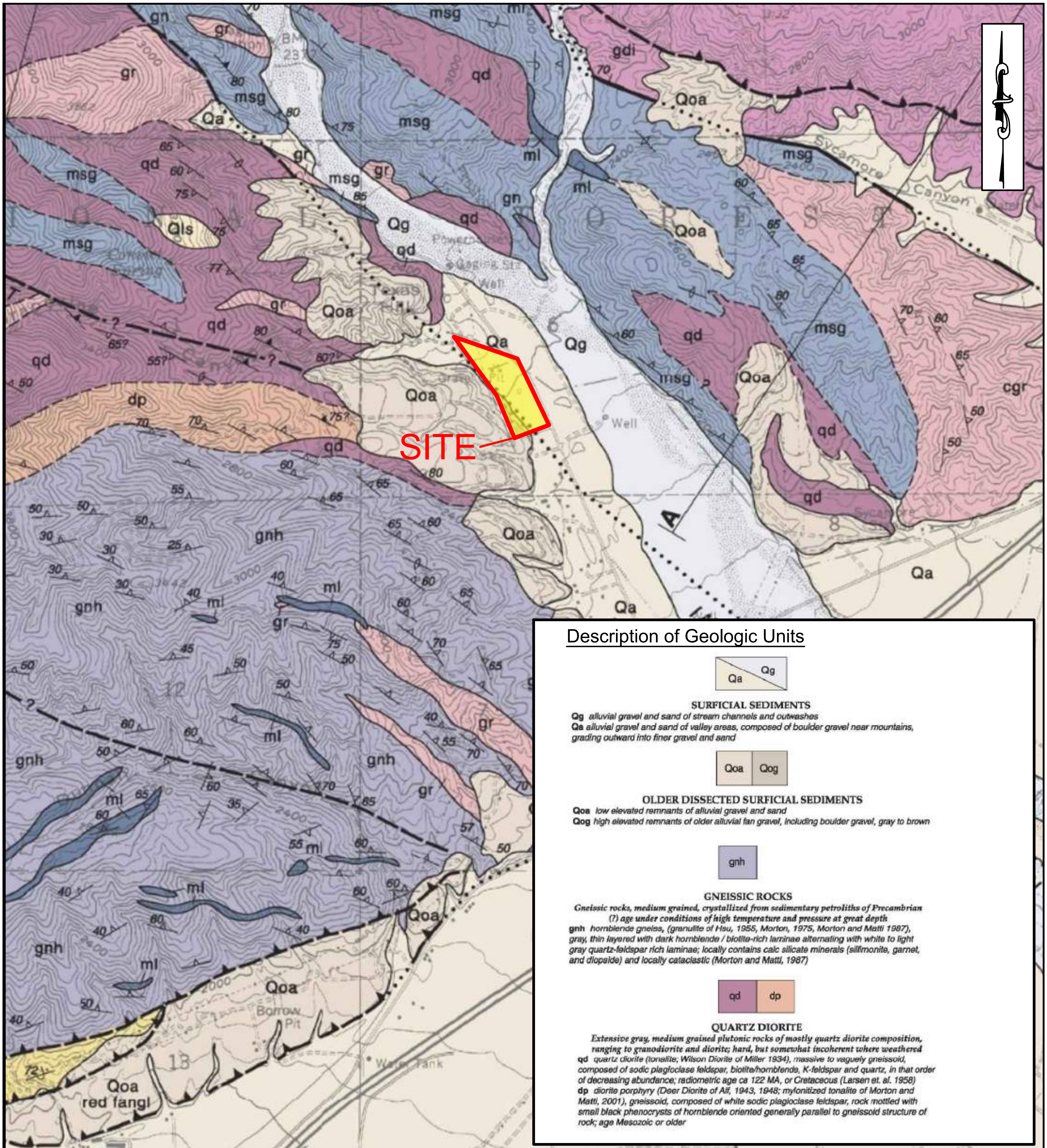
LOR GEOTECHNICAL GROUP, INC.



REGIONAL GEOLOGIC MAP

(Morton and Matti, 2001)

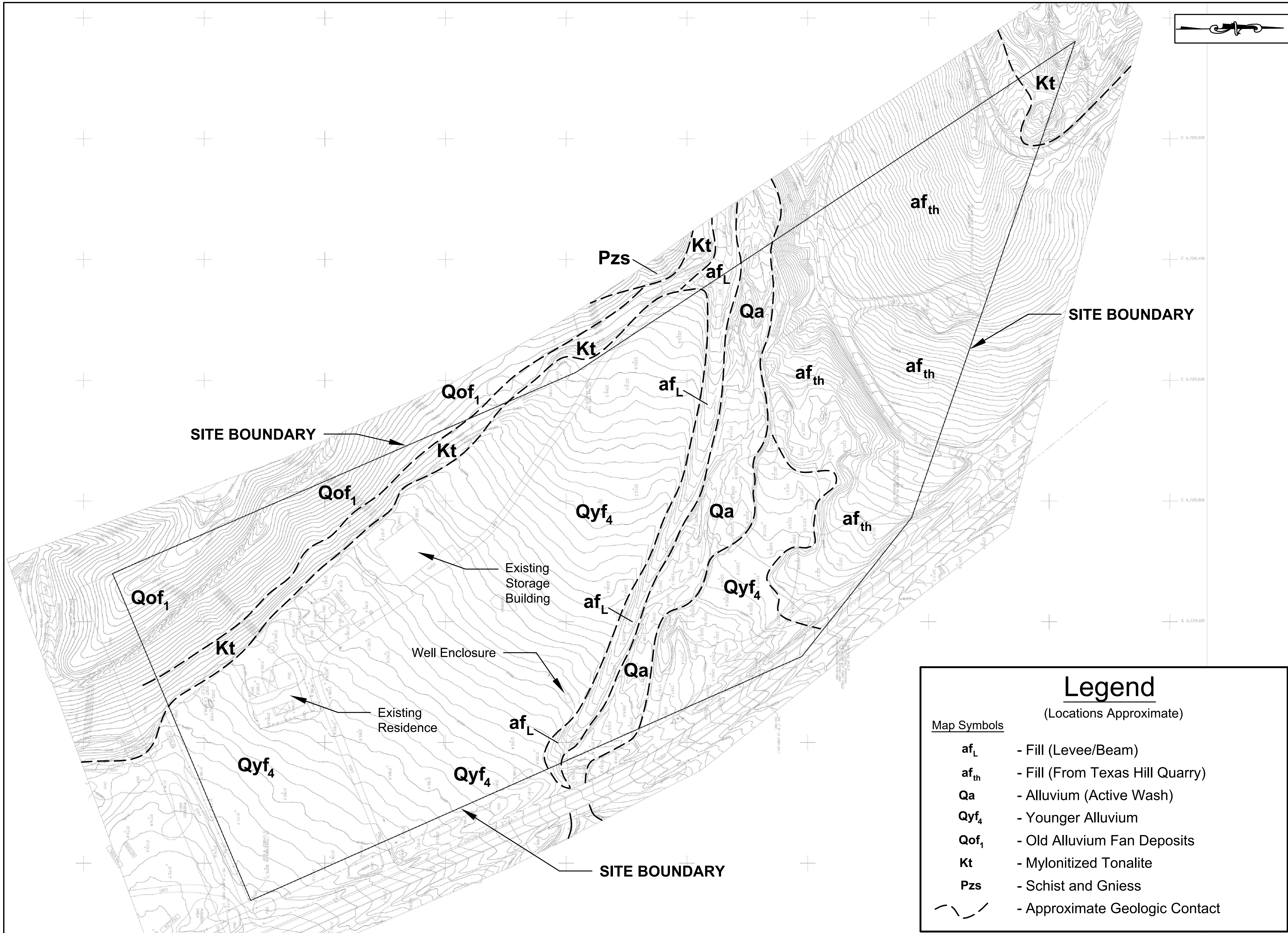
PROJECT:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-3a
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'



REGIONAL GEOLOGIC MAP

(Dibblee and Minch, 2003)

PROJECT:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-3b
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'



GREATER SITE AREA GEOLOGIC MAP

PROJECT: APN 0239-311-01-0000, Lytle Creek, California

CLIENT: Land Engineering Consultants, Inc.

LOR GEOTECHNICAL GROUP, INC.

PROJECT NO.: 13789.1

ENCLOSURE: A-4

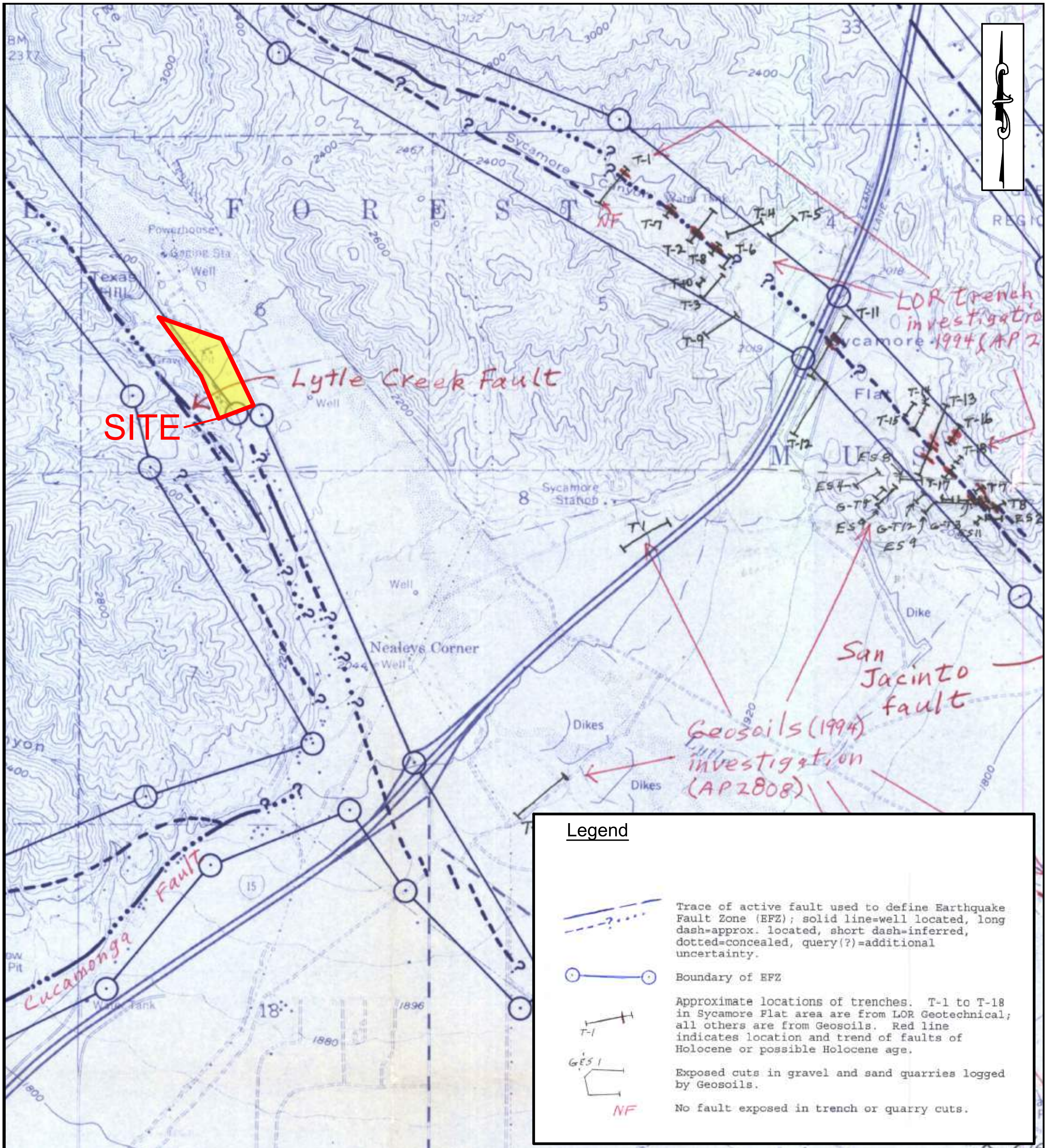
DATE: February 2022

SCALE: 1" ≈ 150'

PHOTO-LINEAMENT MAP

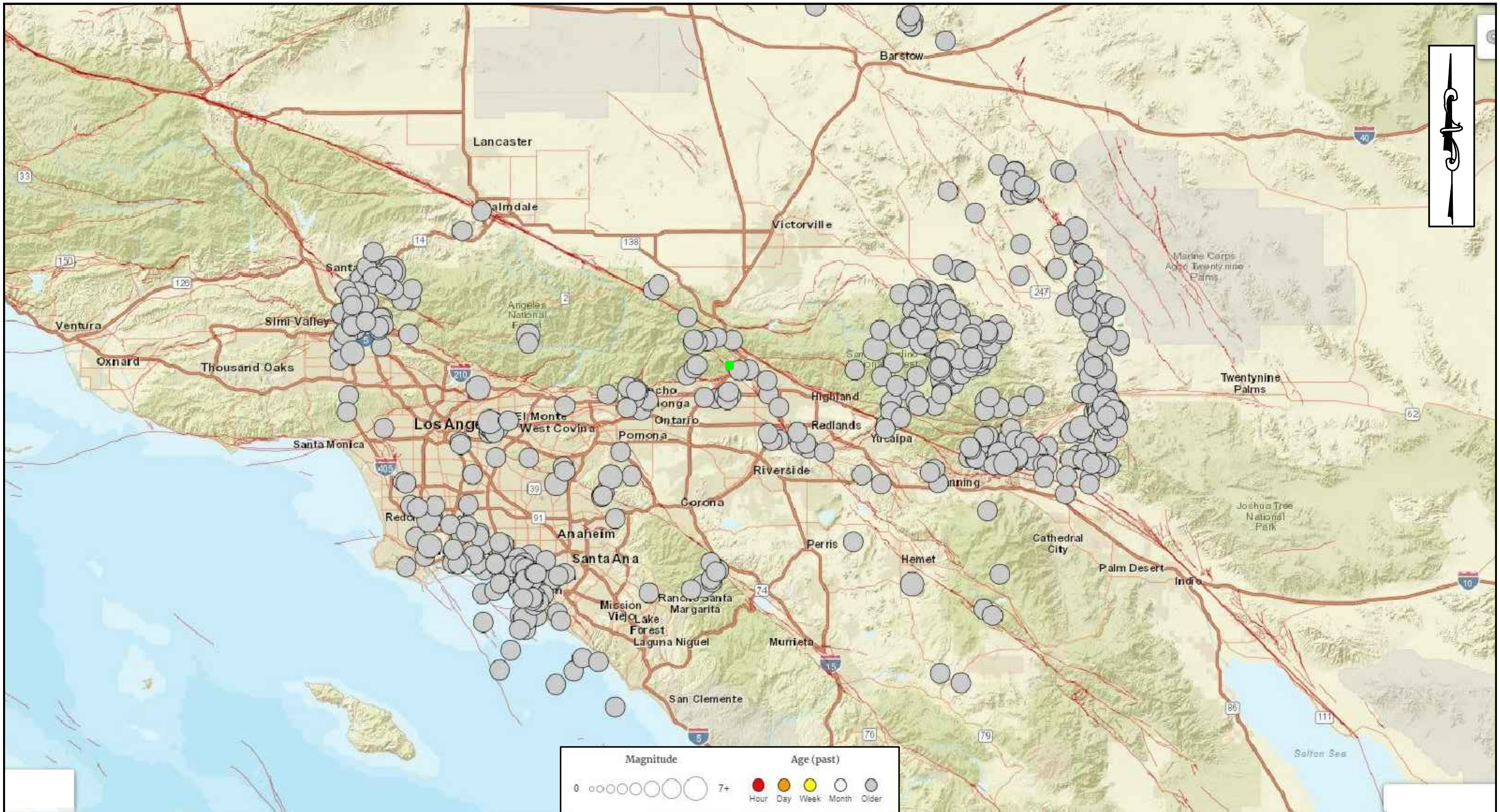
PROJECT:	APN 0239-311-01-0000, Lytle Creek, California
PROJECT NO.:	13789.1
CLIENT:	Land Engineering Consultants, Inc.
ENCLOSURE:	A-5
DATE:	February 2022
SCALE:	1" ≈ 170'

LOR GEOTECHNICAL GROUP, INC.



EARTHQUAKE FAULT ZONE MAP

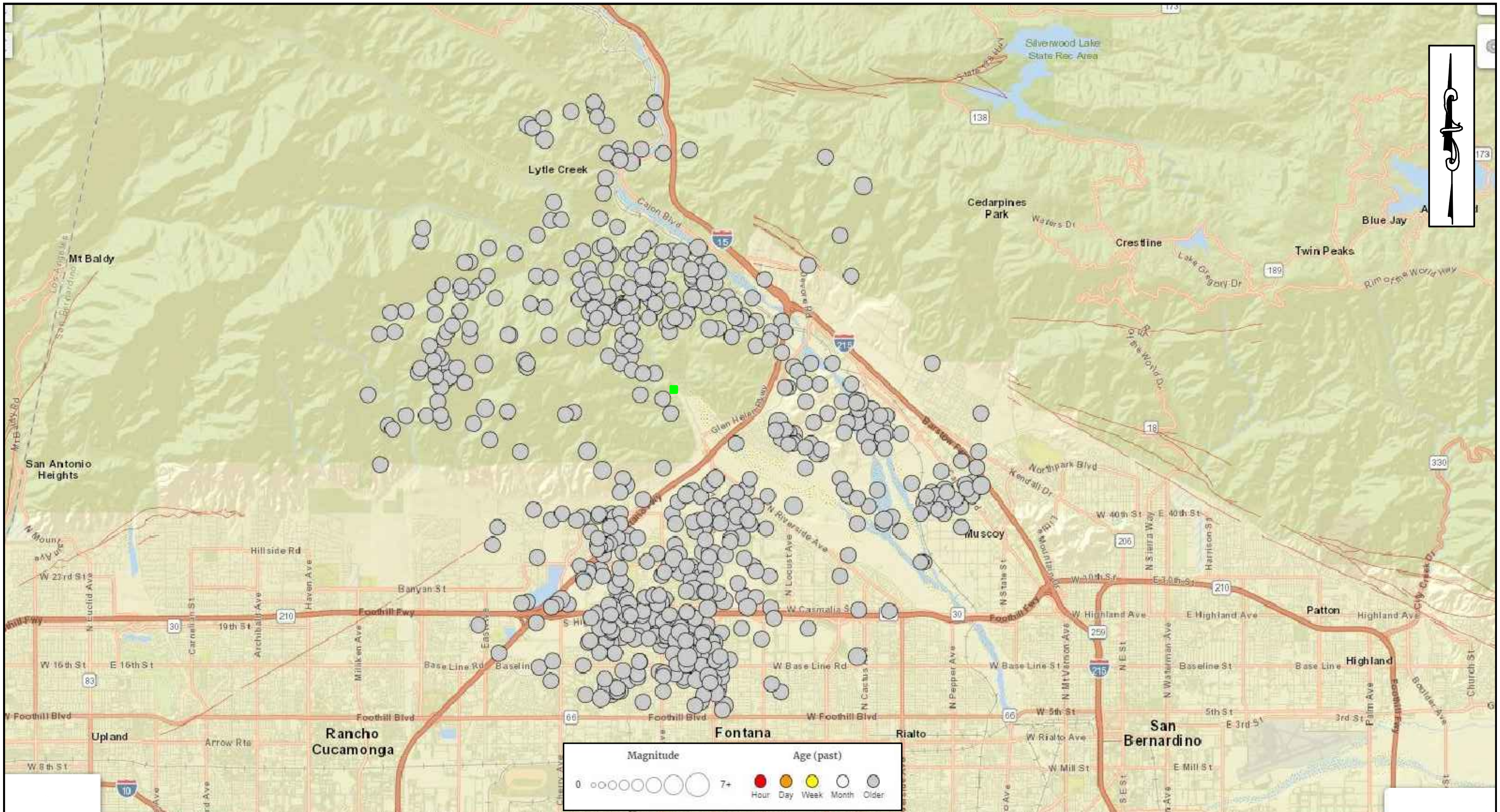
PROJECT:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-6
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 542 epicenters of instrument-recorded events from 01/01/32 to present (02/18/22) 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:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-7
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 641 epicenters of instrument-recorded events from 01/01/78 to present (02/18/22) of local magnitude 2+ 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:	APN 0239-311-01-0000, Lytle Creek, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	A-8
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 10km

APPENDIX B

Field Investigation Program and Trench Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of excavating 6 exploratory trenches to depths of approximately 5 to 15 feet beneath the existing ground surface. The approximate locations of the trenches are shown on our Plat, Enclosure A-2, within Appendix A.

The trenching exploration was conducted using a rubber-tire backhoe equipped with a 36-inch bucket. The soils were continuously logged by our geologist who inspected the site, created detailed logs of the trenches, obtained disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System. In-place density tests were taken at select intervals in accordance with the ASTM D 1557, the Sand Cone Method. Bulk samples were obtained at select intervals and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the trenches are presented on the enclosed Trench Logs, Enclosures B-1 and B-6. A Trench Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented as Enclosure B-ii.

CONSISTENCY OF SOIL

SANDS

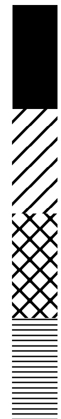
SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

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

TRENCH LOG LEGEND

PROJECT:	APN 0239-311-01-0000, San Bernardino, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022

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 0239-311-01-0000, San Bernardino, California	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022

LOG OF TRENCH T-1

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND with GRAVEL, approximately 20% gravel to 3" diameter, 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 20% silty fines, brown, damp, loose, heavily bioturbated and contains occasional man-made items (wood, concrete).**
								SP	@ 1 foot, ALLUVIUM: **GRAVELLY SAND, approximately 35% gravel to 3" diameter and occasional cobbles, 15% coarse grained sand, 25% medium grained sand, 20% fine grained sand, 5% silty fines, light brown, damp, crudely stratified, medium dense.**
	3, 7, 9 10, 11		82	3.0	117.8				
			86	4.7	122.3				
5									**below 5± feet, decrease in percentage of cobbles and gravels, mostly clean, medium to coarse grained sand.**
10									**below 12± feet, includes occasional boulders to 2' diameter, increase in cobbles, occasional thin lenses of SILTY SAND.**
									END OF TRENCH @ 14' due to severe caving **Fill to 1'** **Moderate caving @ 10', heavy below** **No groundwater** **No bedrock**
15									

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ELEVATION:**	--
LOR GEOTECHNICAL GROUP INC.		**DATE EXCAVATED:**	January 25, 2022
		EQUIPMENT:	JD 410C
		BUCKET WD.: 36	**ENCLOSURE:** B-1

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, FILL/TOPSOIL: SILTY SAND with GRAVEL, approximately 20% gravel to 3" diameter, 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 20% silty fines, brown, damp, loose, heavily bioturbated and contains occasional man-made items (wood, concrete).
								SP	@ 1 foot, ALLUVIUM: GRAVELLY SAND, approximately 10% cobbles, 25% gravel, 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 5% silty fines, light brown, damp, loose. Likely 1938 flood deposits as it overlies at 0.5±' thick darkened layer (2-2.5'), may be just fill.
			85	3.4	121.6				@ 2.5 feet, ALLUVIUM: GRAVELLY SAND, approximately 25% gravel with 5% cobbles, 15% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 5% silty fines, brown, moist, medium dense, moderately stratified.
5			87	3.8	124.5				
									below 4 feet, decrease in gravel and cobbles.
10									
									@ 12± feet, includes minor boulders to 1.5' diameter and local, dark, micaceous SILTY SAND lenses/layers.
15									END OF TRENCH @ 15' Fill to 2.5±' Moderate to heavy caving No groundwater No bedrock

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ELEVATION:**	--
LOR GEOTECHNICAL GROUP INC.		**DATE EXCAVATED:**	January 25, 2022
		EQUIPMENT:	JD 410C
		BUCKET WD.: 36	**ENCLOSURE:** B-2

LOG OF TRENCH T-3

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0	3, 7, 8, 9, 10, 11							SM	@ 0 feet, FILL/TOPSOIL: SILTY SAND with GRAVEL, approximately 20% gravel to 3" diameter, 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 20% silty fines, brown, damp, loose, heavily bioturbated and contains occasional man-made items (wood, concrete).
	3, 7, 9 10, 11		82	6.2	112.6			SP	@ 2 feet, ALLUVIUM: GRAVELLY SAND, approximately 20% gravel to 3" diameter, 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 5% silty fines, brown, damp, medium dense, moderately well stratified, overall decrease in gravel with increase in depth.
			88	7.5	120.1				
5									
									below 6 feet, includes cobbles and small boulders (average 1.5' diameter with one to 3.5' maximum diameter), local SILTY SAND (yellowish-brown and/or dark brown) amongst boulders, very difficult digging.
10									
									END OF TRENCH @ 11' due to severe caving Fill to 2' Heavy caving No groundwater No bedrock
15									

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ELEVATION:**	--
LOR GEOTECHNICAL GROUP INC.		**DATE EXCAVATED:**	January 25, 2022
		EQUIPMENT:	JD 410C
		BUCKET WD.: 36	**ENCLOSURE:** B-3

LOG OF TRENCH T-4

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: SILTY SAND, approximately 15% gravel to 2" diameter, 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 20% silty fines, brown to dark brown, moist, loose to medium dense, includes occasional small roots (to 1/4" diameter), portions of unit appear to be recent flood deposits, sharp lower contact, upper 1.5±' heavily bioturbated.
			84	5.0	114.8			SP	@ 2.5 feet, ALLUVIUM: GRAVELLY SAND, approximately 20% gravel, 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines, brown, damp, stratified, medium dense.
5			88	4.1	120.5				
									below 9± feet, includes occasional small boulders to (1.5' diameter) and cobbles, hard digging.
10									
									END OF TRENCH @ 11.5' due to severe caving Fill to 2.5' Heavy caving No groundwater No bedrock
15									

PROJECT: APN 0239-311-01-0000	**PROJECT NO.:** 13789.1
CLIENT: Land Engineering Consultants, Inc.	**ELEVATION:** --
LOR GEOTECHNICAL GROUP INC.	**DATE EXCAVATED:** January 25, 2022
	EQUIPMENT: JD 410C
	BUCKET WD.: 36 **ENCLOSURE:** B-4

LOG OF TRENCH T-5

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 15% gravel, 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 15% silty fines, brown, damp, loose, heavily bioturbated.**
			84	2.8	115.4			SP	@ 2 feet, ALLUVIUM: **GRAVELLY SAND, approximately 25% gravel with trace of cobbles, 10% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 5% silty fines.**
			87	4.5	118.9				@ 4 to 4.5± feet, thin, moist, finer grained sand with silt layer.
5									
									below 8 feet, includes minor cobbles and boulders to 1.5' diameter.
10									
									@ 12 feet, increase in boulders, includes occasional thin, dark brown, fine to medium grained sand with silt layers/lenses, difficult digging.
15									END OF TRENCH @ 15' Fill to 2' Heavy caving No groundwater No bedrock

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ELEVATION:**	--
LOR GEOTECHNICAL GROUP INC.		**DATE EXCAVATED:**	January 25, 2022
		EQUIPMENT:	JD 410C
		BUCKET WD.: 36	**ENCLOSURE:** B-5

LOG OF TRENCH T-6

DEPTH IN FEET	LABORATORY TESTS		ESTIMATED COMPACTION (%)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND with GRAVEL and man-made debris, approximately 15% gravel with occasional pieces of wood and pieces of concrete to 2' maximum diameter, 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 15% silty fines, brown, damp, loose.**
								SP	@ 3± feet, ALLUVIUM: **GRAVELLY SAND, approximately 25% gravel, 15% coarse grained sand, 30% medium grained sand, 25% fine grained sand, 5% silty fines, brown, damp, medium dense, crudely stratified.**
			90	4.9	123.0				
5									**END OF TRENCH @ 5'** **Fill to 3±'** **Minor caving** **No groundwater** **No bedrock**

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants, Inc.	**ELEVATION:**	--
		DATE EXCAVATED:	January 25, 2022
LOR GEOTECHNICAL GROUP INC.		**EQUIPMENT:**	JD 410C
		BUCKET WD.: 36	**ENCLOSURE:** B-6

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

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

Moisture Density Tests

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

Laboratory Compaction Characteristics

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

LABORATORY COMPACTION				
Trench Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
T-1	2-4	(SP) Gravelly Sand	143.0	5.0
T-3	0-2	(SM) Silty Sand ith Gravel	134.0	7.0
T-3	2-5	(SP) Gravelly Sand	137.0	8.0

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

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

DIRECT SHEAR TESTS				
Trench Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Angle of Internal Friction (degrees)**	**Apparent Cohesion (psf)**
T-1	2-4	(SP) Gravelly Sand	42	0
T-3	0-2	(SM) Silty Sand with Gravel	40	100
T-3	2-5	(SP) Gravelly Sand	45	0

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with the Caltrans Test Number 202 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 in the following table:

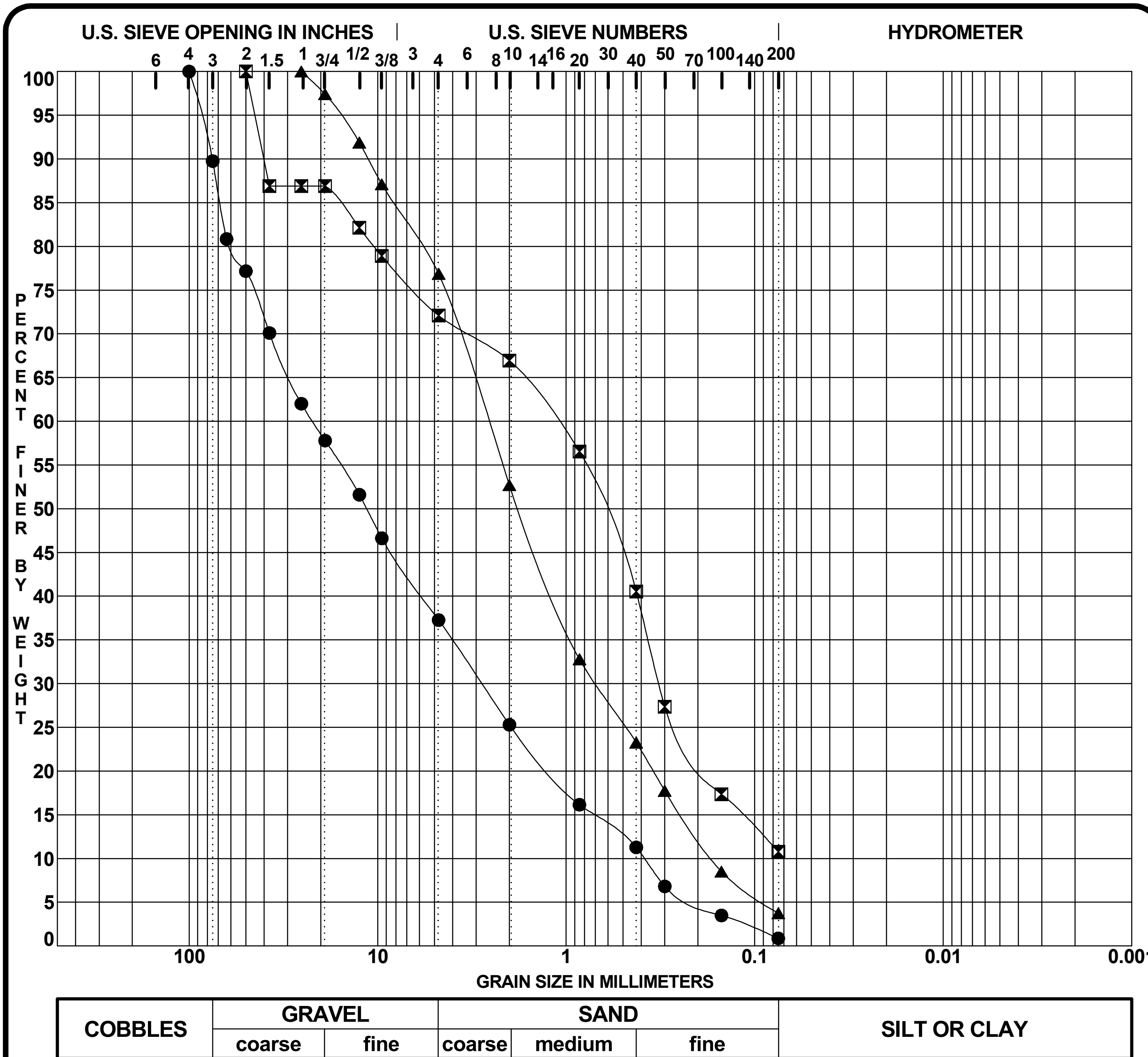
R-Value Test

Soil samples were obtained at the 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 result of the R-value test is presented on Enclosure C-1.

Soluble Sulfate Content Tests

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

SOLUBLE SULFATE CONTENT TESTS			
Trench Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Sulfate Content (percent by weight)**
T-1	2-4	(SP) Gravelly Sand	< 0.005
T-3	0-2	(SM) Silty Sand with Gravel	< 0.005
T-3	2-5	(SP) Gravelly Sand	< 0.005



Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*T-1 @ 2-4'*	*(SP) Gravelly Sand*	*84*	*--*			*0.93*	*57.5*
⊠	*T-3 @ 0-2'*	*(SM) Silty Sand with Gravel*	*50*	*53*			*1.32*	*16.3*
▲	*T-3 @ 2-5'*	*(SP) Gravelly Sand*	*70*	*--*			*1.10*	*15.5*

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt / %Clay
●	*T-1 @ 2-4'*	*100.00*	*22.13*	*2.810*	*0.3849*	*52.5*	*36.4*	*0.9*
⊠	*T-3 @ 0-2'*	*50.00*	*1.13*	*0.322*		*27.9*	*61.3*	*10.8*
▲	*T-3 @ 2-5'*	*25.40*	*2.60*	*0.694*	*0.1679*	*23.1*	*73.1*	*3.7*

PROJECT:	APN 0239-311-01-0000	**PROJECT NO.:**	13789.1
CLIENT:	Land Engineering Consultants	**DATE:**	February 2022

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-1

APPENDIX D

Terra Geosciences Geophysical Investigation Report

TERRA GEOSCIENCES

GEOPHYSICAL SURVEY

PROPOSED COMMERCIAL / LIGHT INDUSTRIAL PROJECT

ASSESSOR'S PARCEL NO. 0239-311-01-0000

LYTLE CREEK, SAN BERNARDINO COUNTY, CALIFORNIA

Project No. 213747-1

December 17, 2021

Prepared for:

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

LOR Geotechnical Group, Inc.
6121 Quail Valley Court
Riverside, CA 92507

Attention: Mr. Robb Markoff

Regarding: Geophysical Survey
Proposed Commercial / Light Industrial Project
Assessor's Parcel No. 0239-311-01-0000
Lytle Creek, San Bernardino County, California
LOR Project No. 13789.1

INTRODUCTION

In accordance with your request, we have completed a geophysical survey using the seismic refraction method across a portion of the subject site, as referenced above. We understand that since the San Jacinto Fault traverses within the vicinity of the site, non-invasive geophysical methods have been deemed appropriate to aid in evaluating the subsurface geological structure, with respect to any faulting potentials that may impact the proposed development. This report will describe in detail the seismic refraction methodology, field procedures used, data processing of the various seismic modeling programs utilized, and the results of this survey, along with the representative seismic models being presented within Appendices A and B for visual and reference purposes. As authorized by you, the following services were performed during this study:

- ***Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site, along with a field reconnaissance.***

- ***Conducting a geophysical survey, using the seismic refraction method, to aid in evaluating the deeper subsurface lithology and geologic structure present beneath the subject site. The field survey and the data analysis were performed by a licensed State of California Professional Geophysicist.***

- ***Preparation of representative seismic models for the seismic traverse displaying the subsurface geologic structure using various computer data analysis programs for both comparative and quality control purposes.***

- ***Preparation of this report, presenting the results of our interpretation of the geophysical data with respect to any possible anomalous structural features at depth.***

Accompanying Map, Illustrations, and Appendices

Plate 1 - Seismic Line Location Map
Plate 2 - Google™ Earth Imagery Map
Appendix A - Layer Velocity Model
Appendix B - Refraction Tomographic Model
Appendix C - References

PROJECT SUMMARY

As requested, we have performed a geophysical survey using the seismic refraction method across a selected portion of the subject property, as referenced above. The purpose of this geophysical study was to provide both a qualitative and quantified geophysical analysis of the subsurface geologic structure and lithologic composition, using the seismic refraction method, in order to discern and any anomalous geologic structures that may be related to faulting beneath the subject site where locally surveyed. Our study involved using various seismic refraction computer modeling programs for both quality control and comparative purposes, which allowed for an unbiased and more thorough analysis. Each of these modeling programs, as described in more detail further in this report, have both strengths and limitations and it was our intention to compare these models to form a more coherent representation of the interpreted subsurface geologic structure.

The location of our seismic traverse was accomplished by use of the Google™ Earth imagery (2021) and Terrain Navigator mapping software (Maptech, 2021), supplemented with GPS (Global Positioning System) coordinates. The traverse location of Seismic Line S-1 was selected in the field based on site topography, physical obstructions, and proposed development coverage. An attempt was made to keep a near-perpendicular orientation to the local fault trend.

From a geologic standpoint, the subject property (where locally surveyed) has been mapped by Morton and Matti (2001) to be surficially mantled by unconsolidated to moderately consolidated Holocene and late Pleistocene age alluvial fan deposits, being directly underlain by progressively older and more consolidated alluvial fan deposits. Based on extrapolation of their geological mapping, these alluvial fan deposits are in turn presumably underlain at depth by cretaceous age granitic rocks (mapped just beyond the beginning of our seismic traverse to the west) comprised of a medium- to coarse-grained hornblende-biotite tonalite.

SUMMARY OF SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method is well suited to identify whether there is a distinct velocity change at depth that could represent a possible subsurface structural differential. The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer.

The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at each of the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

One seismic refraction line was performed (Seismic Line S-1) within the southern portion of the site as directed. This traverse was oriented in a North 59° East direction, being in a near-perpendicular orientation to the local trend of the San Jacinto Fault that traverses through the region. Although the traverse was not surveyed, the location, as presented on Plates 1 and 2, is considered to be fairly accurate, based on the Google™ Earth imagery (2021), physical structures, and GPS coordinates. The survey line consisted of overlapping two individual 230-foot spreads (each with 24, 14-hertz geophones), using 10-foot spacings, with six overlapped geophones in between each spread. This created a combined continuous survey profile of 430 feet in length. Seven shot points were utilized along each spread using forward, reverse, and intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. To produce the necessary seismic wave energy, a 16-pound sledge-hammer was used as the energy source to detect both the direct and refracted waves. Each shot point used multiple hammer impacts to increase the signal to noise ratio, which provided clearer first "P"-Wave arrivals.

The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.10 seconds with no acquisition filters to preserve the raw wave-forms. The data on the paper record and/or display screen were used to analyze the arrival time of the primary seismic "P"-Waves at each geophone station for quality control purposes in the field. Each geophone and seismic shot location were surveyed using a hand level and ruler for relative topographic correction, with "0" representing the lowest elevation point along the line.

Data Reduction

All of the recorded seismic data was subsequently transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004); **Refractor** (Geogiga, 2001-2020); and **Rayfract™** (Intelligent Resources, Inc., 1996-2021). The associated subsurface profile models for each of these computer modeling programs are presented within Appendices A and B for visual and reference purposes.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of three times to improve the accuracy of depths to the top of layer-2.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the Plus-Minus method, and the Generalized Reciprocal Method (GRM). They are described as follows: The Delay-Time method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay-time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The Plus-Minus time analysis method includes a Plus time analysis for depth analysis and a Minus time analysis for velocity determination. The basis of the Plus-Minus time analysis method lies in the traveltime reciprocity, i.e., the traveltime of a seismic wave from source to receiver is equal to the traveltime in the opposite direction if source and receiver are interchanged. It can be used to calculate the depth and velocity variations of an undulating layer boundary for slope angles less than ~10°. The GRM method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

- **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves which generally indicates the relative structure and velocity distribution of the subsurface using first break energy propagation modeling. An initial 1D gradient model is created using the Delta-t-V method which gives a good initial fit between modeled and picked first breaks. This initial model is then refined automatically with a true 2D WET (Wavepath Eikonal Traveltime) tomographic inversion (Schuster and Quintus-Bosz, 1993). WET tomography models multiple signal propagation paths contributing to one first break, whereas conventional ray tracing tomography is limited to the modeling of just one ray per first break.

The combined use of these computer programs provided a more thorough analysis of the subsurface geologic and lithologic structure, and the seismic velocity characteristics, with respect to identifying any anomalous features that may be suggestive of subsurface faulting. Each computer program has a specific purpose based on the objective of the analysis. **Rayfract™** provided tomographic velocity and structural imaging that is very conducive to detecting strong lateral velocity characteristics, while **SIPwin** and **Refractor** are generally based on detecting layered media with some lateral velocity contrast being imaged.

All of the computer programs performed their analysis using exactly the same input data which includes first-arrival "P"-waves and survey line geometry. The resultant travel-time curve (Time-Distance Plot) that was developed from picking of the primary seismic "P"-wave data is presented below on Figure 1 for reference.

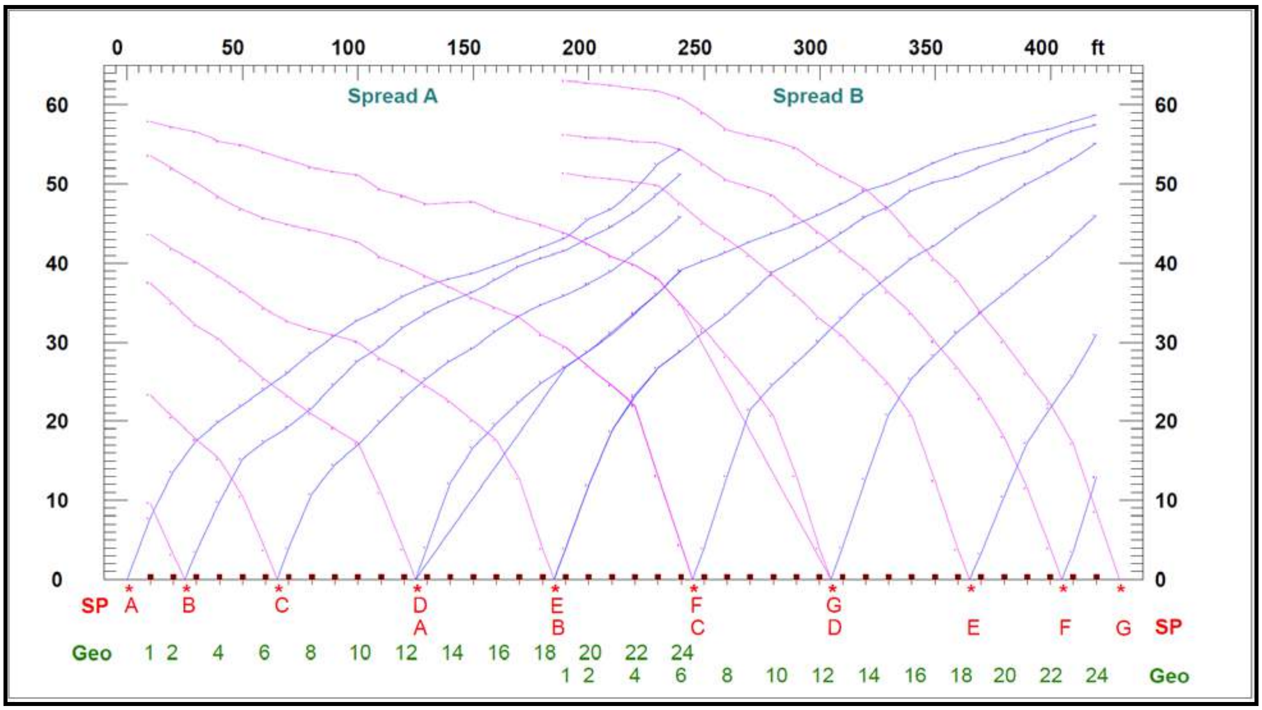


FIGURE 1- Time Distance Plot (S-1).

SUMMARY OF DATA ANALYSIS

As previously discussed, the primary purpose of the seismic refraction survey was to aid in evaluating any possible anomalous geologic structures and/or lithologic variabilities at depth such as offset stratigraphic units (i.e., lateral velocity contrasts) that may be suggestive of subsurface faulting. For this survey we used three different computer processing software programs in order to provide a more thorough analysis of the seismic data of which are described in more detail below, along with the seismic models being presented within Appendices A and B for visual and reference purposes.

♦ **Layer Velocity Model:**

The Layer Velocity Model is a more traditional approach to modeling the subsurface and was analyzed using the computer program **SIPwin** and **Refractor** of which a composite model was produced as presented in Appendix A. Based on the model derived from these programs, two distinct seismic velocity layers were encountered.

The uppermost velocity layer V1 is comprised of loose and unconsolidated younger alluvial type materials that have an average weighted velocity of 1,296 fps, being up to 12± feet in thickness. Directly underlying the surficial materials is the V2 velocity layer that has an average weighted velocity of 3,952 fps, which most likely represents slightly-consolidated relatively older alluvial materials possibly of early Holocene to late Pleistocene age. It should be noted that this velocity range is also representative of weathered granitic bedrock materials also. The deeper V3 velocity layer underlying the subject property at depth, has an average weighted velocity of 7,641 fps, which appears to consist of crystalline granitic bedrock.

♦ **Refraction Tomographic Model:**

The **Rayfract™** tomographic model does not create discrete velocity layers or boundaries but rather produces a "smoothed" tomographic image that displays the velocity gradient within the limits of the seismic wave ray coverage that was sampled. The data appeared to be of good quality which was verified by the Root Mean Square Error (RMS) that is displayed on the tomographic model. The RMS error (misfit between picked and modeled first break times) is automatically calculated during the processing routine, with a value of less than 5.0% being preferred. The resultant model obtained value of 1.7%. Based on the tomographic model presented within Appendix B, generally consistent relatively planar velocity structural contours are depicted with the velocity gradient gradually increasing with depth.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of good quality with little amounts of ambient "noise" being introduced during our survey. Therefore, analysis of the data and picking of the primary "P"-wave arrivals was therefore performed with little difficulty. Based on the results of our comparative seismic analyses of the computer programs **SIPwin**, **Refractor**, and **Rayfract™**, the seismic refraction survey line models appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data.

Based on the layer velocity profiles, it appears that there is a generally thin mantle (up to 12± feet) of loose, unconsolidated younger alluvial materials overlying the site, with the underlying V2 velocity layer most likely consisting of progressively older and denser alluvial deposits. It is possible that this velocity layer could also represent weathered granitic bedrock but based on the subsurface geomorphic expression and the great thickness along the eastern portion of the survey traverse, this layer is most likely older alluvial fan deposits. Additionally, it is also possible that since the average velocity of this layer is representative of both older alluvial deposits and weathered bedrock, there may be some combination of both units in the V2 layer that are not discernable. The lower deposit (V3), underlying the site at depth, is believed to consist of Cretaceous age crystalline granitic bedrock, such as exposed just to the west of the seismic traverse.

The refraction tomographic model revealed overall relatively planar structural velocity contouring with a minor inflection at a distance of around 175 to 200± feet (at a depth of around 25-40± feet), which is also depicted on the layer velocity model along the V2/V3 boundary contact at approximately the same distance/depth interval. This feature on both models appears to represent natural channel scouring and deposition along the bedrock contact at depth and is not suggestive of faulting, as deeper structural velocity contouring is not disrupted on the tomographic model.

In summary, based on the data obtained, there do not appear to be any observable anomalous conditions from a geophysical standpoint that would suggest that subsurface faulting is present within the limits of our survey traverse (i.e., lateral velocity contrasts and/or or other lithologic differentials).

CLOSURE

The field survey was performed by the undersigned on December 11, 2021 using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area. The seismic data was evaluated using various seismic inversion computer programs, including using recently developed tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface structural conditions.

It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. It should be noted that estimates of the layer velocity boundaries are generally considered to be within 10± percent of the total depth of the contact.

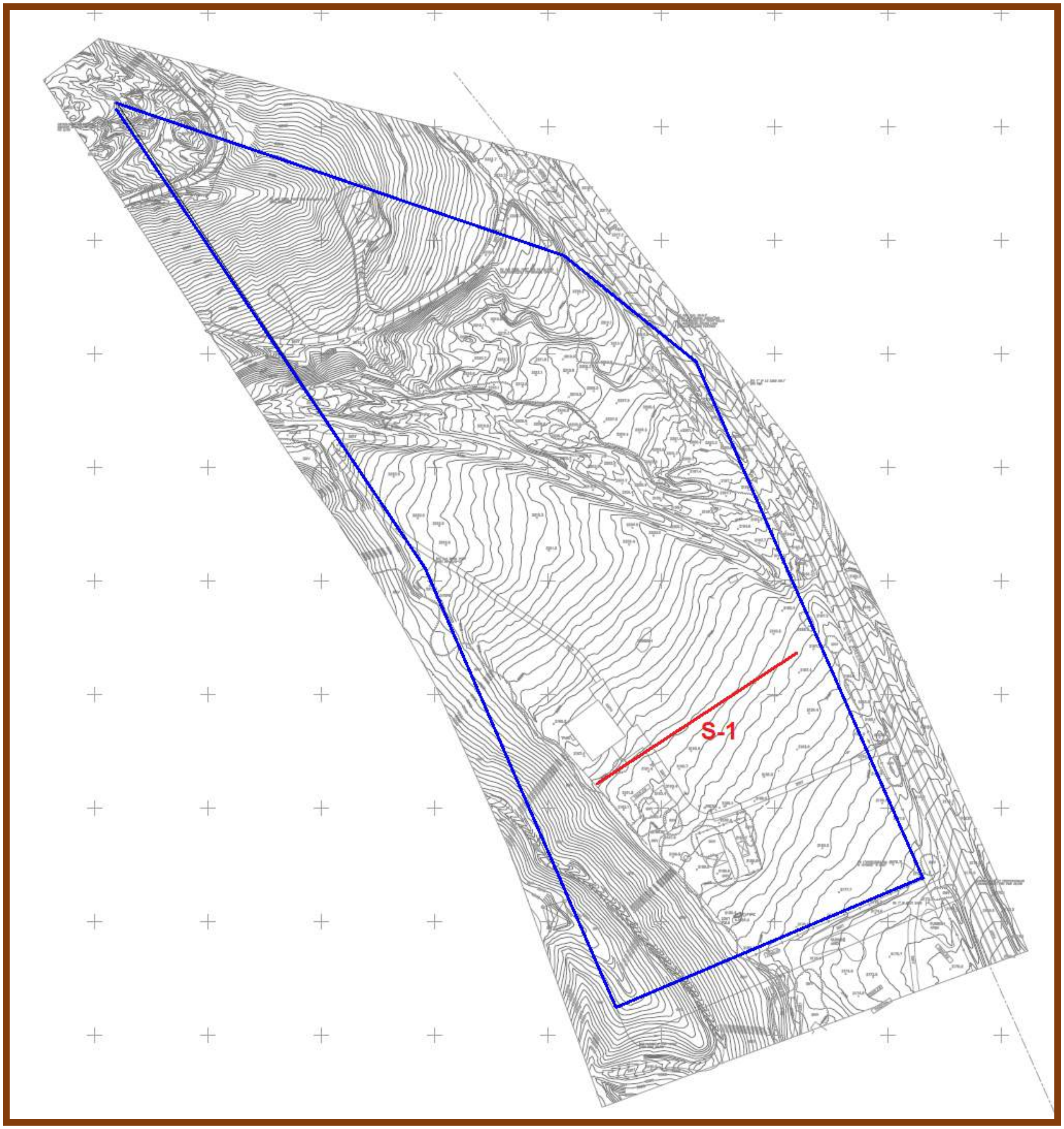
Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and, in the interpretation and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

This opportunity to be of service is sincerely appreciated. If you should have any questions regarding this report or do not understand the limitations of this study or the data that is presented, please do not hesitate to contact our office at your earliest convenience.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Professional Geophysicist
PGP 1002

SEISMIC LINE LOCATION MAP



Topographic base map prepared by TMR Associates, San Bernardino, CA, dated 11/18/21.

GOOGLE™ EARTH IMAGERY MAP

Base Map: Google™ Earth imagery (2021); Seismic Line S-1 indicated by red/yellow line; approximate site boundaries outlined in blue.

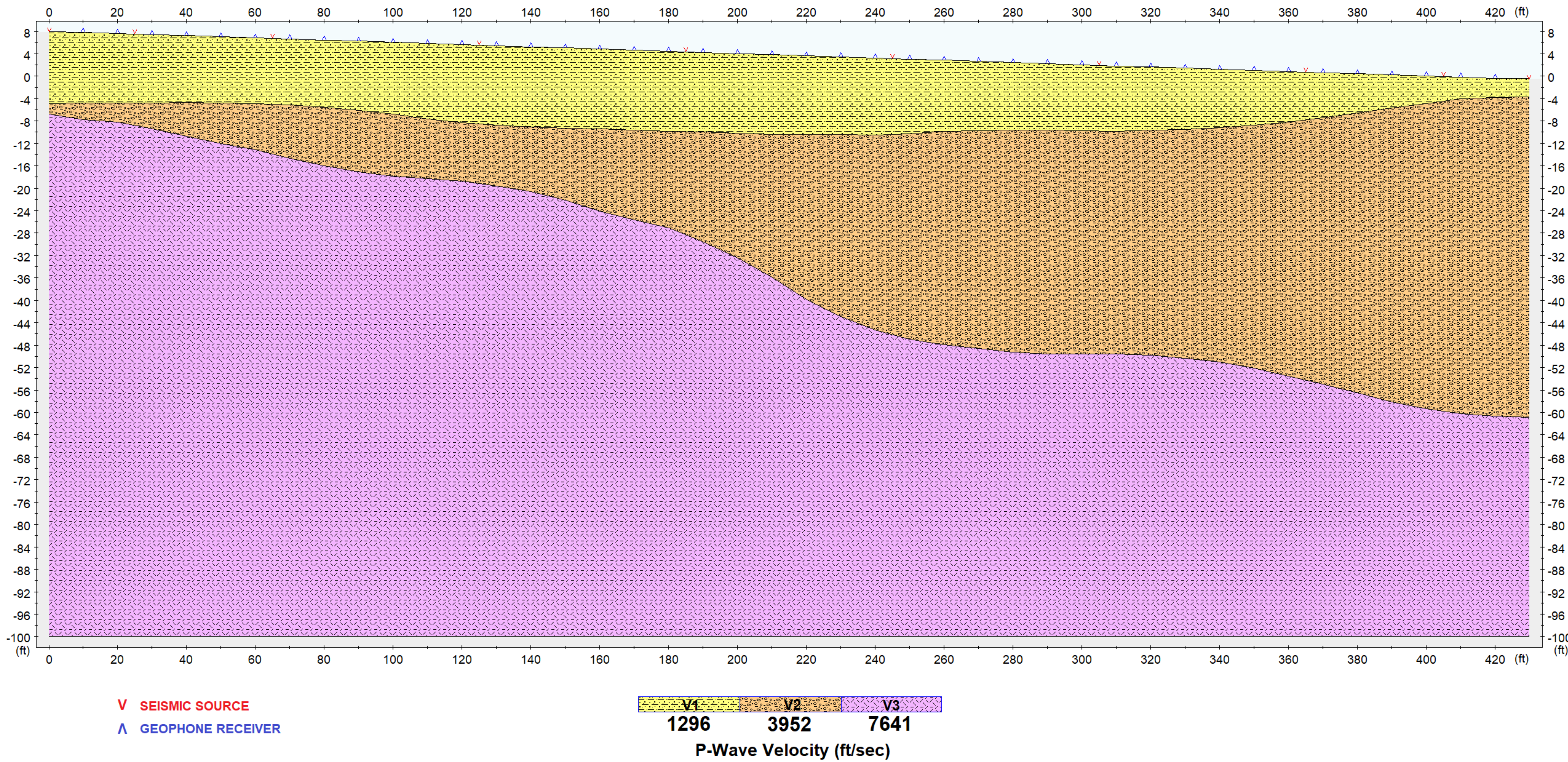
APPENDIX A

LAYER VELOCITY MODEL

SEISMIC LINE S-1

North 59° East >

LAYER VELOCITY MODEL

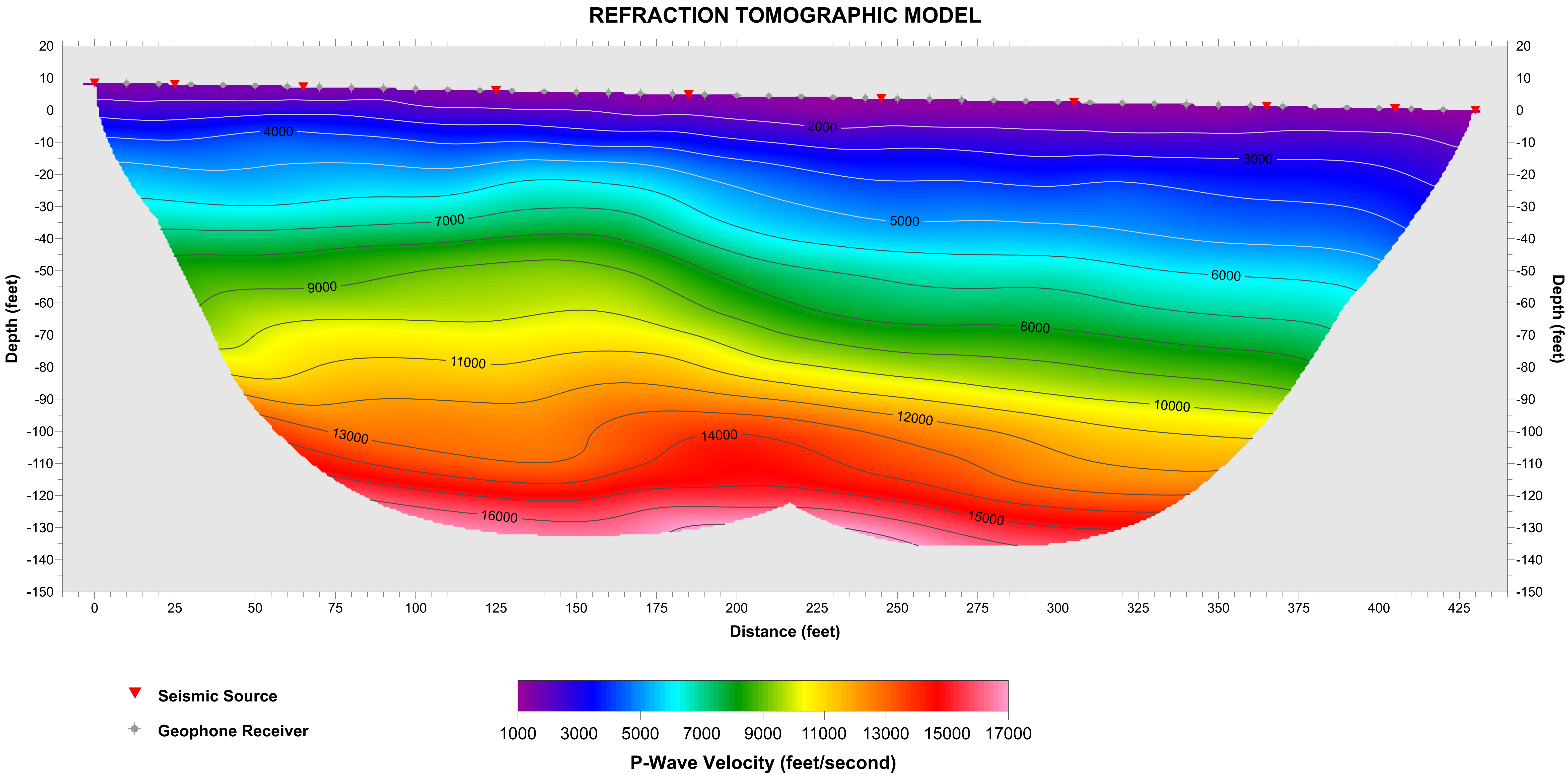


APPENDIX B

REFRACTION TOMOGRAPHIC MODEL

SEISMIC LINE S-1

North 59° East→



SCALE: 1:1 (Horizontal = Vertical)

RMS error 1.7%; Rayfract Version 4.02

APPENDIX C

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REFERENCES

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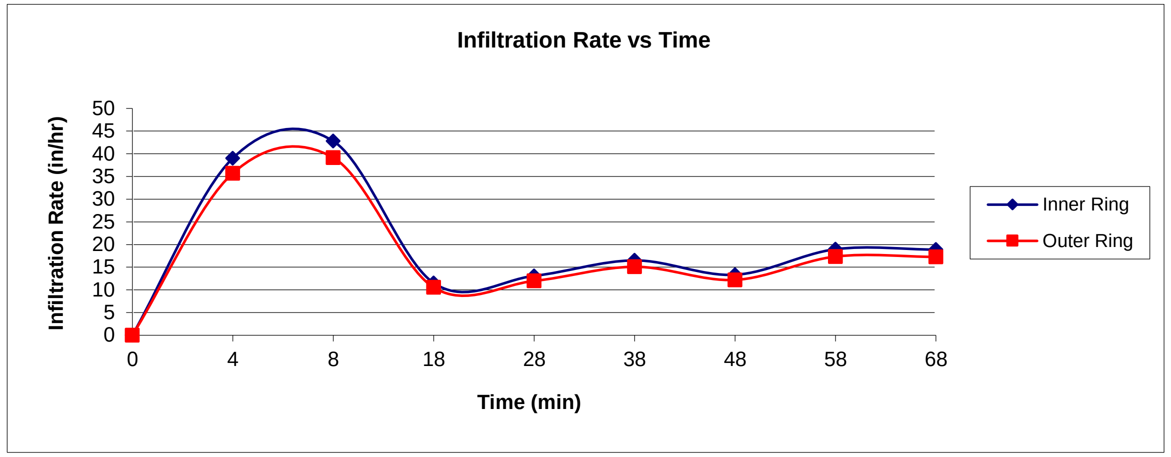
APPENDIX E

Infiltration Test Results

DOUBLE RING INFILTROMETER TEST DATA

Project:	APN 0239-311-01-0000	**Client:**	Land Engineering Consultants,
Project No.:	13789.1	**Test Date:**	January 25, 2022
Soil Classification:	(SP) Poorly graded sand w/ gravel	**Test Hole No.:**	DRI-1
Depth of Test Hole:	2.0 ft.	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	January 25, 2022
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	R.L.	**Depth of Water in Rings:**	3.0 in.
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	6.0 in.
Depth to Water Table:	> 50 ft.		

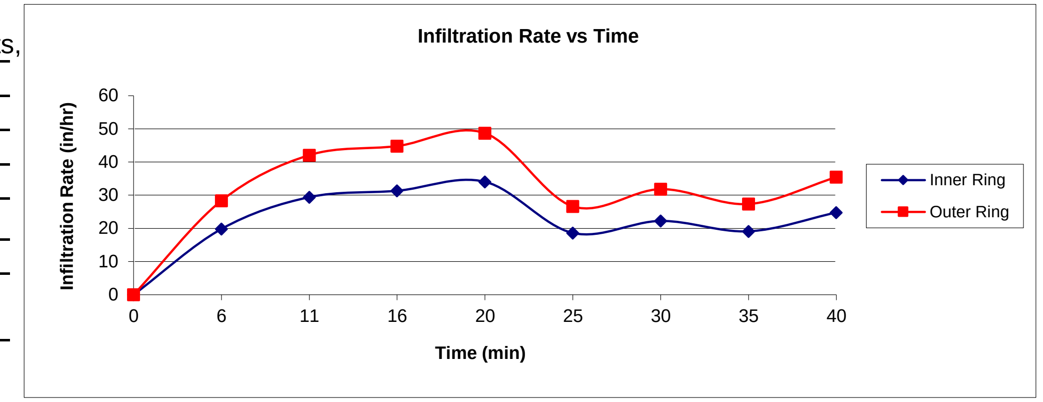


TEST PERIOD

TRIAL NO.	INNER TIME		INNER TIME INTERVAL (minutes)	INNER TOTAL ELASPED TIME (minutes)	ANNULAR SPACE TIME	ANNULAR SPACE TIME INTERVAL (minutes)	ANNULAR SPACE TOTAL ELASPED TIME (minutes)	WATER USED (lbs.) inner	WATER USED (lbs.) annular space	WATER USED (gal) inner	WATER USED (gal) annular space	INFILTRATION RATE (gal/sf.day) inner	INFILTRATION RATE (gal/sf.day) annular space	INFILTRATION RATE (in/hr) inner	INFILTRATION RATE (in/hr) annular space	LIQUID TEMP (°F)	REMARKS
1	S	9:12	4	4	9:12	4	4	10.58	29.09	1.270	3.492	582.5	532.7	39.0	35.7	58	
	E	9:16			9:16											58	
2	S	9:16	4	8	9:16	4	8	11.60	31.90	1.393	3.830	638.6	584.2	42.8	39.1	58	
	E	9:20			9:20											58	
3	S	10:35	10	18	10:35	10	18	7.81	21.47	0.938	2.577	172.0	157.3	11.5	10.5	59	refilled both
	E	10:45			10:45											59	
4	S	10:45	10	28	10:45	10	28	8.87	24.40	1.065	2.929	195.3	178.7	13.1	12.0	59	
	E	10:55			10:55											59	
5	S	10:55	10	38	10:55	10	38	11.18	30.75	1.342	3.691	246.2	225.3	16.5	15.1	59	
	E	11:05			11:05											60	
6	S	11:07	10	48	11:07	10	48	9.04	24.86	1.085	2.984	199.1	182.1	13.3	12.2	60	refilled both
	E	11:17			11:17											60	
7	S	11:17	10	58	11:17	10	58	12.85	35.34	1.543	4.242	283.0	258.9	19.0	17.3	60	
	E	11:27			11:27											60	
8	S	11:27	10	68	11:27	10	68	12.79	35.17	1.535	4.222	281.7	257.6	18.9	17.3	60	
	E	11:37			11:37											60	

DOUBLE RING INFILTROMETER TEST DATA

Project:	APN 0239-311-01-0000	**Client:**	Land Engineering Consultants,
Project No.:	13789.1	**Test Date:**	January 25, 2022
Soil Classification:	(SP) Poorly graded sand w/ gravel	**Test Hole No.:**	DRI-2
Depth of Test Hole:	5.0 ft.	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	January 25, 2022
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	R.L.	**Depth of Water in Rings:**	3.0 in.
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	6.0 in.
Depth to Water Table:	> 50 ft.		



TEST PERIOD

TRIAL NO.	INNER TIME		INNER TIME INTERVAL (minutes)	INNER TOTAL ELASPED TIME (minutes)	ANNULAR SPACE TIME	ANNULAR SPACE TIME INTERVAL (minutes)	ANNULAR SPACE TOTAL ELASPED TIME (minutes)	WATER USED (lbs.) inner	WATER USED (lbs.) annular space	WATER USED (gal) inner	WATER USED (gal) annular space	INFILTRATION RATE (gal/sf.day) inner	INFILTRATION RATE (gal/sf.day) annular space	INFILTRATION RATE (in/hr) inner	INFILTRATION RATE (in/hr) annular space	LIQUID TEMP (°F)	REMARKS
1	S	9:15	6	6	9:15	6	6	8.06	34.66	0.968	4.161	295.8	423.2	19.8	28.4	56	
	E	9:21			9:21											56	
2	S	9:21	5	11	9:21	5	11	9.97	42.87	1.197	5.146	439.1	628.1	29.4	42.1	56	
	E	9:26			9:26											56	
3	S	9:26	5	16	9:26	5	16	10.62	45.66	1.275	5.481	467.7	668.9	31.3	44.8	56	
	E	9:31			9:31											56	
4	S	9:31	4	20	9:31	4	20	9.23	39.69	1.108	4.765	508.1	726.9	34.0	48.7	57	
	E	9:35			9:35											57	
5	S	9:37	5	25	9:37	5	25	6.31	27.13	0.758	3.257	277.9	397.5	18.6	26.6	57	refilled both
	E	9:42			9:42											57	
6	S	9:42	5	30	9:42	5	30	7.54	32.42	0.905	3.892	332.1	475.0	22.2	31.8	57	
	E	9:47			9:47											57	
7	S	9:47	5	35	9:47	5	35	6.48	27.86	0.778	3.345	285.4	408.2	19.1	27.3	57	
	E	9:52			9:52											58	
8	S	9:52	5	40	9:52	5	40	8.40	36.12	1.008	4.336	370.0	529.2	24.8	35.5	58	
	E	9:57			9:57											58	
9	S	10:01	5	45	10:01	5	45	4.27	18.36	0.513	2.204	188.1	269.0	12.6	18.0	58	refilled both
	E	10:06			10:06											58	
10	S	10:06	5	50	10:06	5	50	6.59	28.34	0.791	3.402	290.2	415.2	19.4	27.8	58	
	E	10:11			10:11											58	
11	S	10:11	5	55	10:11	5	55	6.04	25.97	0.725	3.118	266.0	380.5	17.8	25.5	58	
	E	10:16			10:16											59	
12	S	10:16	5	60	10:16	5	60	6.53	28.08	0.784	3.371	287.6	411.4	19.3	27.6	58	
	E	10:21			10:21											59	
13	S	10:21	5	65	10:21	5	65	5.08	21.84	0.610	2.622	223.7	320.0	15.0	21.4	58	
	E	10:26			10:26											59	
14	S	10:26	5	70	10:26	5	70	7.38	31.73	0.886	3.809	325.0	464.9	21.8	31.1	58	
	E	10:31			10:31											59	

APPENDIX F

Seismic Design Spectra

Project: APN 0239-311-01-0000
Project Number: 13789.1
Client: Land Engineering Consultants, Inc.
Site Lat/Long: 34.1980/-117.4466
Controlling Seismic Source: San Jacinto

REFERENCE	NOTATION	VALUE	REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured	Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Site Class D - Table 11.4-1	F_a	1.0	Design Maps	S_s	2.508
Site Class D - 21.3(ii)	F_v	2.5	Design Maps	S_1	0.875
$0.2*(S_{D1}/S_{DS})$	T_0	0.119	Equation 11.4-1 - F_A*S_S	S_{MS}	2.508*
S_{D1}/S_{DS}	T_S	0.593	Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.672*
Fundamental Period (12.8.2)	T	Period	Design Maps	PGA	1.029
Seismic Design Maps or Fig 22-14	T_L	8	Table 11.8-1	F_{PGA}	1.1
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.9917*	Equation 11.8-1 - F_{PGA}*PGA	PGA_M	1.132*
Equation 11.4-2 - F_V*S_1	S_{M1}	1.4875*	Section 21.5.3	80% of PGA_M	0.906
			Design Maps	C_{RS}	0.907
			Design Maps	C_{R1}	0.882

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.907
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.882

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

Period	Cr
0.200	0.907
0.300	0.904
0.400	0.901
0.500	0.898
0.600	0.895
0.680	0.892
1.000	0.882

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

Mapped values from https://seismicmaps.org/

LOR GEOTECHNICAL GROUP, INC.

PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

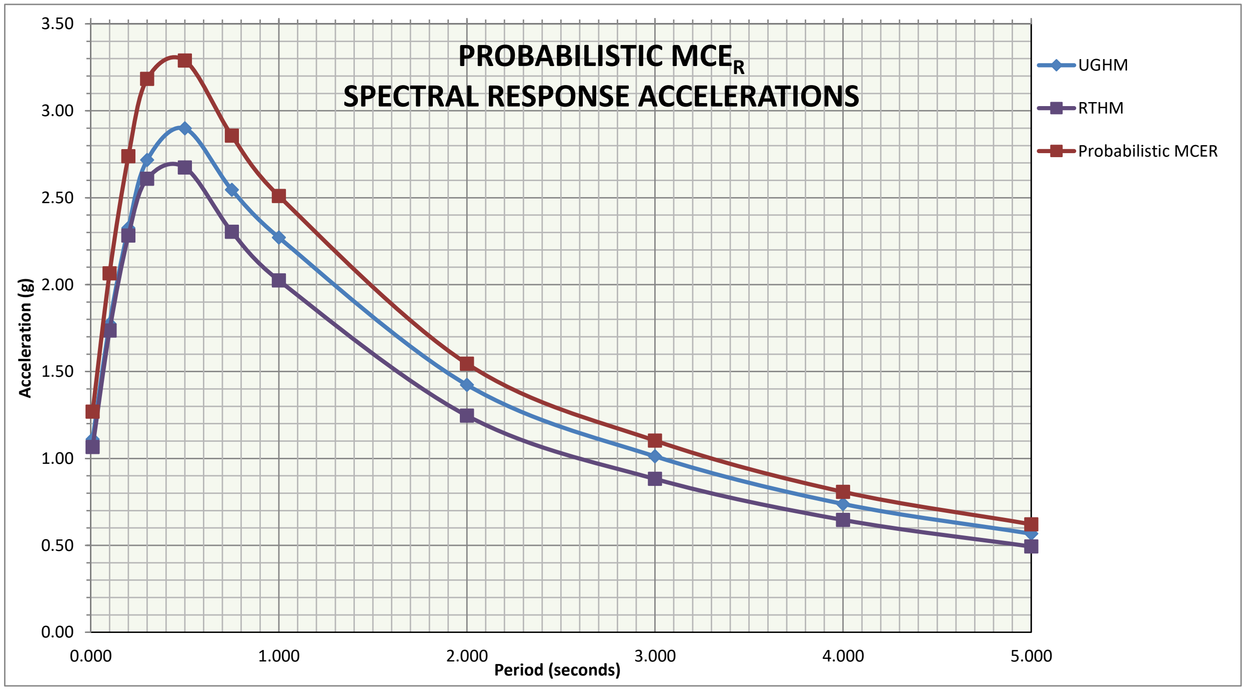
Project No: 13789.1

Period	UGHM	RTHM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	1.110	1.066	1.19	1.269
0.100	1.774	1.735	1.19	2.065
0.200	2.325	2.282	1.20	2.738
0.300	2.717	2.609	1.22	3.183
0.500	2.898	2.674	1.23	3.289
0.750	2.545	2.304	1.24	2.857
1.000	2.271	2.024	1.24	2.510
2.000	1.423	1.246	1.24	1.545
3.000	1.012	0.882	1.25	1.103
4.000	0.738	0.646	1.25	0.808
5.000	0.567	0.493	1.26	0.621

Probabilistic PGA:	1.110
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: San Jacinto

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

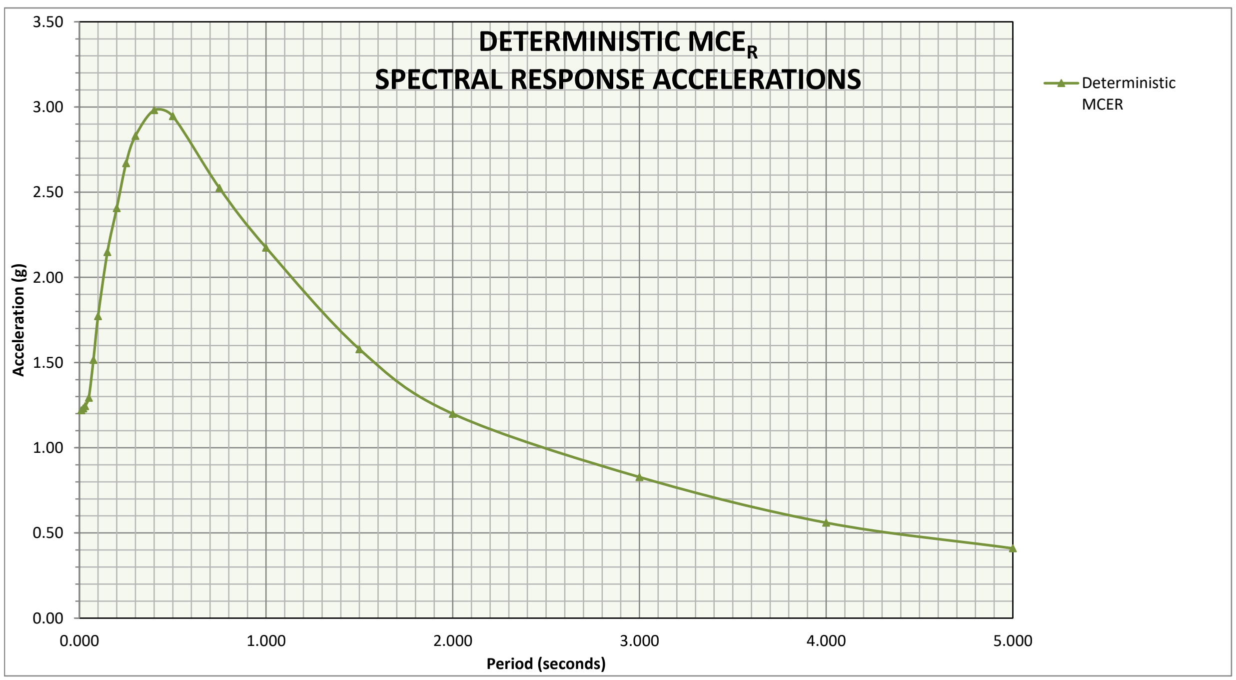
Project No: 13789.1

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	1.025	1.19	1.220	1.220
0.020	1.034	1.19	1.231	1.231
0.030	1.046	1.19	1.244	1.244
0.050	1.087	1.19	1.293	1.293
0.075	1.274	1.19	1.516	1.516
0.100	1.490	1.19	1.773	1.773
0.150	1.791	1.20	2.149	2.149
0.200	2.006	1.20	2.407	2.407
0.250	2.207	1.21	2.671	2.671
0.300	2.320	1.22	2.831	2.831
0.400	2.424	1.23	2.981	2.981
0.500	2.395	1.23	2.946	2.946
0.750	2.036	1.24	2.525	2.525
1.000	1.753	1.24	2.174	2.174
1.500	1.273	1.24	1.578	1.578
2.000	0.967	1.24	1.199	1.199
3.000	0.662	1.25	0.828	0.828
4.000	0.448	1.25	0.560	0.560
5.000	0.325	1.26	0.409	0.409

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	1.025
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)



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SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	1.269	1.220	1.220	0.814
0.100	2.065	1.773	1.773	1.212
0.200	2.738	2.407	2.407	1.604
0.300	3.183	2.831	2.831	1.887
0.500	3.289	2.946	2.946	1.964
0.750	2.857	2.525	2.525	1.683
1.000	2.510	2.174	2.174	1.450
2.000	1.545	1.199	1.199	0.800
3.000	1.103	0.828	0.828	0.552
4.000	0.808	0.560	0.560	0.373
5.000	0.621	0.409	0.409	0.273

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.711	0.569
0.010	0.753	0.603
0.020	0.838	0.670
0.030	0.923	0.738
0.050	1.092	0.873
0.060	1.176	0.941
0.075	1.303	1.042
0.090	1.430	1.144
0.100	1.515	1.212
0.110	1.599	1.279
0.120	1.672	1.338
0.136	1.672	1.338
0.150	1.672	1.338
0.160	1.672	1.338
0.170	1.672	1.338
0.180	1.672	1.338
0.200	1.672	1.338
0.250	1.672	1.338
0.300	1.672	1.338
0.400	1.672	1.338
0.500	1.672	1.338
0.580	1.672	1.338
0.640	1.549	1.240
0.750	1.322	1.058
0.850	1.167	0.933
0.900	1.102	0.881
0.950	1.044	0.835
1.000	0.992	0.793
1.500	0.661	0.529
2.000	0.496	0.397
3.000	0.331	0.264
4.000	0.248	0.198
5.000	0.198	0.159

ASCE 7-16: Section 21.4

Site Specific

	Calculated Value	**Design Value**
SDS:	1.768	1.768
SD1:	1.655	1.655
SMS:	2.651	2.651
SM1:	2.483	2.483
Site Specific PGAm:	1.025	1.025
Site Class:	D measured	

Seismic Design Category - Short* E

Seismic Design Category - 1s* E

* Risk Categories I, II, or III

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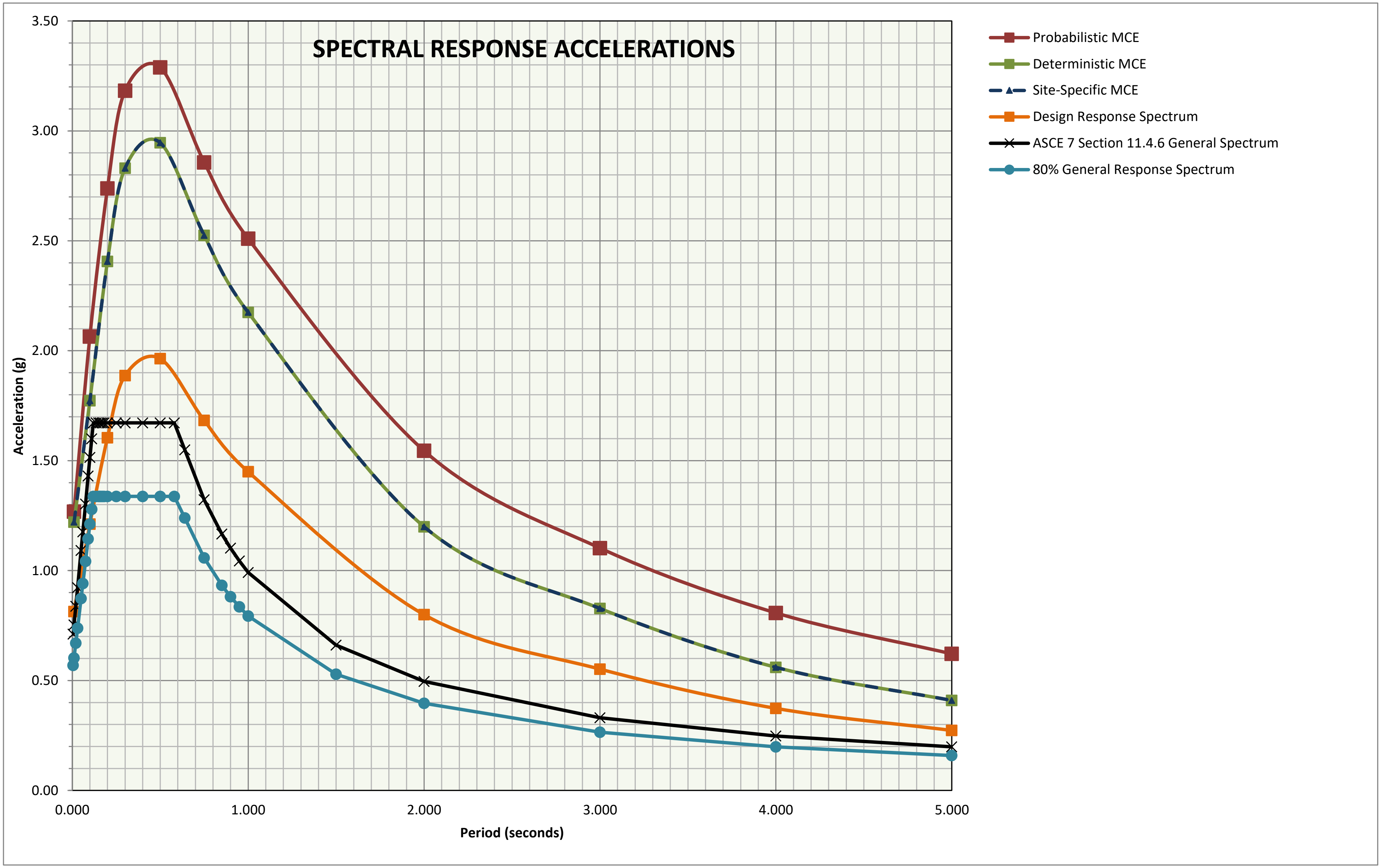

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