PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED INDUSTRIAL PROJECT APN's 026-202-109 and -113 SAN BERNARDINO COUNTY, CALIFORNIA

PROJECT NO. 23720.1
MAY 17, 2021

Prepared For:

Cajon Blvd Industrial Park, LLC
1212 S. Mountain View Avenue
San Bernardino, California 92408

Attention: Mr. Parviz Razavian

May 17, 2021

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Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Industrial Project, APN's 026-202-109 and -113, San Bernardino County, California.

LOR Geotechnical Group, Inc., is pleased to present this report of our geotechnical investigation for the subject project. In summary, it is our opinion that the proposed development is feasible from a geotechnical perspective, provided the recommendations presented in the attached report are incorporated into design and construction. However, the contents of this summary should not be solely relied upon.

To provide adequate support for the proposed structure, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. Any undocumented fill material and all loose alluvial materials should be removed from structural areas and areas to receive engineered compacted fill. The data developed during this investigation indicates that removals on the order of 2 to 4 feet will be required from currently planned development areas. The given removal depths are preliminary and the actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

The results of our field investigation and test data indicates the site soils tested are conducive to infiltration. Very low expansion potential, good R-value quality, and negligible soluble sulfate content generally characterize the onsite materials tested.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During April and May of 2021, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for proposed industrial development of APN 026-202-109 and -113 in the Devore area of San Bernardino County, California. The purpose of this investigation was to conduct a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed improvements. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photographs of the site and surrounding region dated 1938 through 2020;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from the reviewed documents, literature, and reports;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Infiltration testing via the borehole test method;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings and providing conclusions and recommendations for site development.

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

To orient our investigation at the site, you provided us with a conceptual site plan prepared by MM Architect Services, Inc., dated December 4, 2020, that showed the proposed development and location of proposed infiltration. As indicated on that map, a truck terminal and truck parking facility, to be developed in two phases, are currently proposed. Associated landscaping and access improvements are also proposed. The preliminary site plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A.

PROJECT CONSIDERATIONS

The proposed structure is anticipated to be of concrete tilt-up type construction. Moderate foundation loads are anticipated with such a structure. Given the relatively flat topographic conditions at the site, it is anticipated that cuts and fills during site grading will be minimal. No cut or fill slopes are shown on the site plan provided.

EXISTING SITE CONDITIONS

The subject site consists of 9.6 acres of currently vacant land located along the east side of Cajon Boulevard, south of Kendall Drive, and on the west side of the Atchison, Topeka and Santa Fe railroad tracks in the Devore area of San Bernardino County, California. The property slopes gently to the south-southeast and contains a moderate growth of annual grasses and weeds plus several scattered sycamore trees.

A storm pipe that apparently extends from Cajon Boulevard, south to within the southwest portion of the site, outlets onto the property with the runoff draining to the south a short distance within an earthen channel and then as surface flow to the south. No man-made structures were noted to be present and there is only minor trash and debris present within the site. This includes a pile of railroad track and miscellaneous discarded debris in the southwest portion.

To the south of the site is an industrial facility and to the west-northwest is an auto dismantling yard. Beyond the railroad tracks to the east is a vacant field and Kendall Drive.

AERIAL PHOTOGRAPH ANALYSIS

During our investigation we reviewed a series of aerial photographs available through Google Earth (2021) and Historic Aerials (2021). The aerial photographs ranged in dates from 1938 through 2020 and 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 minor changes that occurred onsite throughout this time span.

The site appears to have remained in a fairly natural condition since at least 1938 and the adjacent roads, railroad tracks, and onsite drainage outlet were present prior to this time. Since 1938 changes to the site appear to have been largely limited to clearing of brush and weeds on several occasions. Our review of historic aerial photographs did not identify evidence for onsite or nearby faulting as no photo-lineaments were noted to project through or within close proximity to the site and no evidence for onsite or adjacent site mass movements, such as landslides, was noted on the photographs reviewed.

FIELD EXPLORATION PROGRAM

Our subsurface field exploration program was conducted on April 23, 2021 and consisted of the drilling of 4 exploratory borings with a truck-mounted Mobile B-61 drill rig equipped with 8-inch diameter hollow stem augers. The borings were drilled to refusal depths of approximately 17 to 39 feet below the existing ground surface. The approximate locations of our exploratory borings are presented on the attached Site Plan, Enclosure A-2 within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by a geologist from this firm. Relatively undisturbed and bulk samples were obtained from our exploratory borings and returned to our geotechnical laboratory in sealed containers for further testing and evaluation. A detailed description of the field exploration program and the boring logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

The site is located on a broad, coalescing alluvial fan that emanates from the San Gabriel and San Bernardino Mountains to the north. These sediments fill a deep structural depression known as the upper Santa Ana River Valley. According to Fife and others (1976), the alluvial deposits beneath the site are approximately 300 feet thick and rest on a basement of granitic bedrock.

The upper Santa Ana River Valley is bordered by the San Gabriel Mountains and the active Cucamonga fault to the northwest, the Puente Hills and the potentially active Chino fault to the west. To the south are the Jurupa Mountains and other resistant granitic and metamorphic hills. The eastern boundary of the valley is the San Bernardino Mountains and the active San Andreas fault, located a short distance to the northeast.

According to a study conducted by the United States Geological Survey (Morton and Matti, 2001), the region of the site is underlain by units of younger alluvium that were derived mainly from the mountains to the north and northeast. The dominant drainage in the area is the Cajon Creek wash located approximately one-quarter mile to the southwest of the site. This drainage has resulted in the deposition of these relatively unconsolidated alluvial units over the local area of the valley floor. Cajon Creek joins with Lytle Creek about 2.5 miles to the south, in the area between Muscoy and the city of Rialto.

Active earthquake faults in the region include the San Jacinto fault located approximately 1.2 kilometers (0.75 miles) to the southwest, the San Andreas fault located approximately 1.8 kilometers (1.1 miles) to the northeast, and the Cucamonga fault located approximately 7.2 kilometers (4.5 miles) to the southwest.

The geologic conditions of the site and immediate surrounding region as mapped by the U.S.G.S. (Morton and Matti, 2001) is shown on Enclosure A-3, within Appendix A.

Site Geologic Conditions

The site lies just northeast of Cajon Wash and near the base of the nearby San Bernardino Mountains. The alluvial soils that underlie the site are described below.

Alluvium: As encountered within our exploratory borings, these soils predominantly consist of poorly graded sand with silt and/or gravel with local, surficial silty sand soils. In general, these soil materials are massive to crudely stratified, loose to medium dense in the near surface, becoming more dense with an increase in rock content with increasing depth. Borings were terminated due to refusal on rocks of unknown dimensions.

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

Groundwater Hydrology

Groundwater was not encountered within our exploratory borings advanced to a maximum depth of approximately 39 feet below the existing ground surface.

Records for nearby wells which were readily available from the State of California Department of Water Resources online database (CDWR, 2021) were reviewed as a part of this investigation. This database indicates that one groundwater well, located approximately 0.3 miles to the northwest, had depths to groundwater that ranged from 115 feet to 258 feet from the time period extending from June of 2011 to March of 2021, with the most recent recorded depth of approximately 150 feet below the ground surface. Historically, the depth to groundwater appears to have been more shallow in the past. According to information published by Carson and Matti (1985), the depth to groundwater in the vicinity of the site ranged from between 50 and 75 feet during the time period from 1973 to 1975.

Surface Runoff

Current surface runoff of precipitation waters across the site is generally as sheet flow to the southeast.

Mass Movement

The site lies on a relatively flat surface. The occurrence of mass movement failures such as landslides, rockfalls, or debris flows within such areas is generally not considered common and no evidence of mass movement was observed on the site.

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2007), nor within a county of San Bernardino Earthquake Fault Zone (County of San Bernardino, 2021).

As previously mentioned, the closest known active fault is the San Jacinto fault, which is located approximately 1.2 kilometers (0.75 mile) to the southwest. In addition, other relatively close active faults include the San Andreas fault located 1.8 kilometers (1.1 miles) to the northeast, and the Cucamonga fault located 7.2 kilometers (4.5 miles) to the southwest.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region.

This fault has been active in recent times with several large magnitude events. It is believed that the San Jacinto fault is capable of producing an earthquake magnitude on the order of 6.5 or greater.

The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific plate and the North American plate. While estimates vary, the San Andreas fault is generally thought to have an average slip rate on the order of 24 mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

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

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

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search website of the U.S.G.S. (2020). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from 1932 through May 11, 2021.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As depicted on Enclosure A-4, within Appendix A, the site lies within a relatively active region with the San Jacinto and the San Andreas faults 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 1.0 and greater since 1978. In addition, only the “A” events, or most accurate events were selected. Caltech indicates the accuracy of the “A” events to be approximately 1 kilometer. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 40± years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the subject site lies within an area underlain by very numerous small events in the general area.

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 faults described within. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

<u>Secondary Seismic Hazards</u>

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.

<u>Liquefaction</u>: 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 groundwater is thought to be in excess of 50 feet beneath the site and the site is underlain by relatively dense alluvial deposits, the possibility of liquefaction within these units is considered nil.

<u>Seiches/Tsunamis</u>: The potential for the site to be affected by a seiche or tsunami (earthquake generated wave) is considered nil due to the absence of any large bodies of water near the site.

<u>Flooding (Water Storage Facility Failure):</u> There are no large water storage facilities located on or upstream near the site which could possibly rupture during an earthquake and affect the site by flooding.

<u>Seismically-Induced Landsliding</u>: Our research and review of aerial photographs identified no evidence for the presence of landslides within the site or within the vicinity of the site.

Therefore, the potential for seismically-induced landsliding to impact the site is considered to be low.

Rockfalls: No large, exposed, loose or unrooted boulders that could affect the integrity of the site are present upon or above the site.

Seismically-Induced Settlement: Settlement generally occurs within areas of loose, granular soils with relatively low density. Since the site is underlain by 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, and thus the design requirements, shall be the responsibility of the structural engineer and the building official. For structures at the site to be designed in accordance with the provisions of Chapter 16, the subject site specific criteria is provided below:

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. 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 $\bar{N}$ values greater than 50 blows per foot. Site Class D can be used for stiff soil with $\bar{N}$ values ranging from 15 to 50 blows per foot. Site Class E is for soft clay soils with $\bar{N}$ values less than 15 blows per foot. Our Standard Penetration Test (SPT) data indicate that the materials beneath the site are considered Site Class D-Stiff Soils.

CBC Earthquake Design Summary

As determined in the previous section, earthquake design criteria have been formulated for the site. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region. Our design values are provided in Appendix E.

INFILTRATION TESTING AND TEST RESULTS

Infiltration Testing

Two borehole infiltration tests were conducted at the general locations requested and illustrated on Enclosure A-2. Test borings were drilled to depths of approximately 5 feet below the existing ground surface. Subsequent to drilling, a 3-inch diameter, perforated PVC pipe wrapped in filter fabric was placed within each test hole and 3/4-inch gravel was placed between the outside of the pipe and the hole wall. Testing took place on April 23, 2021. The holes were filled using water from a 350 gallon storage tank. Test periods consisted of allowing the water to drop in 1 to 2 minute intervals. After each reading, the hole was refilled to a depth of approximately 3 feet. Testing was terminated after a total of 12 readings were recorded.

Infiltration test results are summarized in the following table:

Test No.	Depth*	Infiltration Rate** (in/hr)
I-1	5.0	176
I-2	5.3	169
* depth measured below existing ground surface ** Porchet Method determined rate		

The results of this testing are presented as Enclosures D-1 and D-2 in Appendix D. The test results indicate very good infiltration characteristics for the soils tested at a depth of approximately 5 feet.

CONCLUSIONS

General

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

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

If conditions are encountered during the construction of the project which differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Foundation Support

Based upon the field investigation and test data, it is our opinion that the existing upper alluvial soils will not, in their present condition, provide uniform and/or adequate support for the proposed improvements. Left as is, this condition could cause unacceptable differential and/or overall settlements upon application of the anticipated foundation loads.

To provide adequate support for the proposed structural improvements, we recommend that a compacted fill mat be constructed beneath footings and slabs. This compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. Conventional foundation systems, using either individual spread footings and/or continuous wall footings, will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat.

Soil Expansiveness

Our borings placed across the site encountered typically very low expansive, relatively granular soils. For very low expansive soils, no specialized construction procedures to resist expansive soil activity are necessary.

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

Sulfate Protection

The results of the soluble sulfate tests conducted on selected subgrade soils expected to be encountered at foundation levels indicate that there is a negligible sulfate exposure to concrete elements in contact with the on site soils per the 2019 CBC. Therefore, no specific recommendations are given for concrete elements to be in contact with the onsite soils.

Geologic Mitigations

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

Seismicity

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

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

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

RECOMMENDATIONS

Geologic Recommendations

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

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An 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 any existing fills under any proposed flatwork and/or 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 undocumented fill soils and/or 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

Any undocumented fill material and all loose alluvial soils should be removed from all proposed structural and/or fill areas. The data developed during this investigation indicates that removals on the order of 2 to 4 feet will be required from proposed development areas in order to encounter competent alluvium upon which engineered compacted fill can be placed. The given removal depths are preliminary. The actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing. Removals should expose alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557).

Preparation of Fill Areas

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

Engineered Compacted Fill

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

If required, import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 6 inches in maximum dimension. Sources for import fill should be approved by the geotechnical engineer prior to their use. Fill should be spread in maximum 8-inch uniform, loose lifts, each lift brought to near optimum moisture content, and compacted to a relative compaction of at least 90 percent in accordance with ASTM D 1557.

Preparation of Foundation Areas

All foundations for structures, including retaining walls or free-standing walls, should rest upon at least 24 inches of properly compacted fill material placed over competent alluvium. In areas where the required fill thickness is not accomplished by the recommended removals or by site rough grading, the foundation areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines. The bottom of all excavations should be scarified to a depth of 12 inches, brought to near optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557) prior to the placement of compacted fill.

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

Foundation Design

If the site is prepared as recommended, the proposed structures, including retaining walls or free-standing walls, may be safely founded on conventional shallow foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 24 inches of engineered compacted fill. All foundations should have a minimum width of 12 inches and should be established a minimum of 12 inches below lowest adjacent grade.

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 2,000 psf. This bearing pressure may be increased by 500 psf for each additional foot of width and by 500 psf for each additional foot of depth, up to a maximum of 4,000 psf.

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

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 310 pounds per square foot per (psf) foot of depth. Base friction may be computed at 0.31 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Wall Pressures

The design of footings for retaining walls should be performed in accordance with the recommendations described earlier under Preparation of Foundation Areas and

Foundation Design. For design of retaining wall footings, the resultant of the applied loads should act in the middle one-third of the footing, and the maximum edge pressure should not exceed the basic allowable value without increase.

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

Retaining structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.38 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

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

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

Slab-On-Grade Design

Concrete floor slabs should bear on a minimum of 24 inches of engineered fill compacted to at least 90 percent (ASTM D 1557). The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane will reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

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.

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.

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 borings, 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.

Where fills are to be placed against existing slopes steeper than five horizontal to one vertical, the existing slopes should be properly keyed and benched into competent native materials. The key, constructed across the toe of the slope, should be a minimum of 12 to 15 feet wide, a minimum of 2 feet deep at the toe, and sloped back to 2 percent. Benches should be constructed at approximately 2 to 4 foot vertical intervals.

Slope Protection

Since the site soils 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, the watering system should be monitored to assure proper operation and to prevent over watering.

Exterior Flatwork

To provide adequate support, exterior flatwork improvements should rest on a minimum of 12 inches of soil compacted to at least 90 percent (ASTM D 1557).

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

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
Parking and Drive Areas (light vehicular traffic and occasional truck traffic)	6.0	50	0.25' AC / 0.35' AB
Industrial Collector Secondary Major - Off-site	8.0	50	0.40' AC / 0.45' AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base *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 a minimum of 6-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 6-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.

Infiltration

Based upon our field investigation and test data, design of an infiltration system at the site may utilize an adjusted clear water rate of 10 inches per hour. A factor of safety of 3 should be applied to this application rate as indicated by the San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP), (2013).

To ensure continued infiltration capability of the infiltration area, a program to maintain the facility should be considered. This program should include periodic removal of accumulated materials, which can slow the infiltration and decrease the water quality. Materials to be removed 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 San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP), (2013). The program should also incorporate the recommendations presented below 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.

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, R-value, and soluble sulfate testing may be required during site rough grading.

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

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavation prior to processing and/or filling.

3. Processing and compaction of removal and/or over-excavation of bottom soils 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 Cajon Blvd. Industrial Park, LLC, and their design constituents, 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 as indicated by this report, please contact this office immediately in order that we might evaluate these conditions.

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

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Engineering Geologist

John P. Leuer, GE 2030
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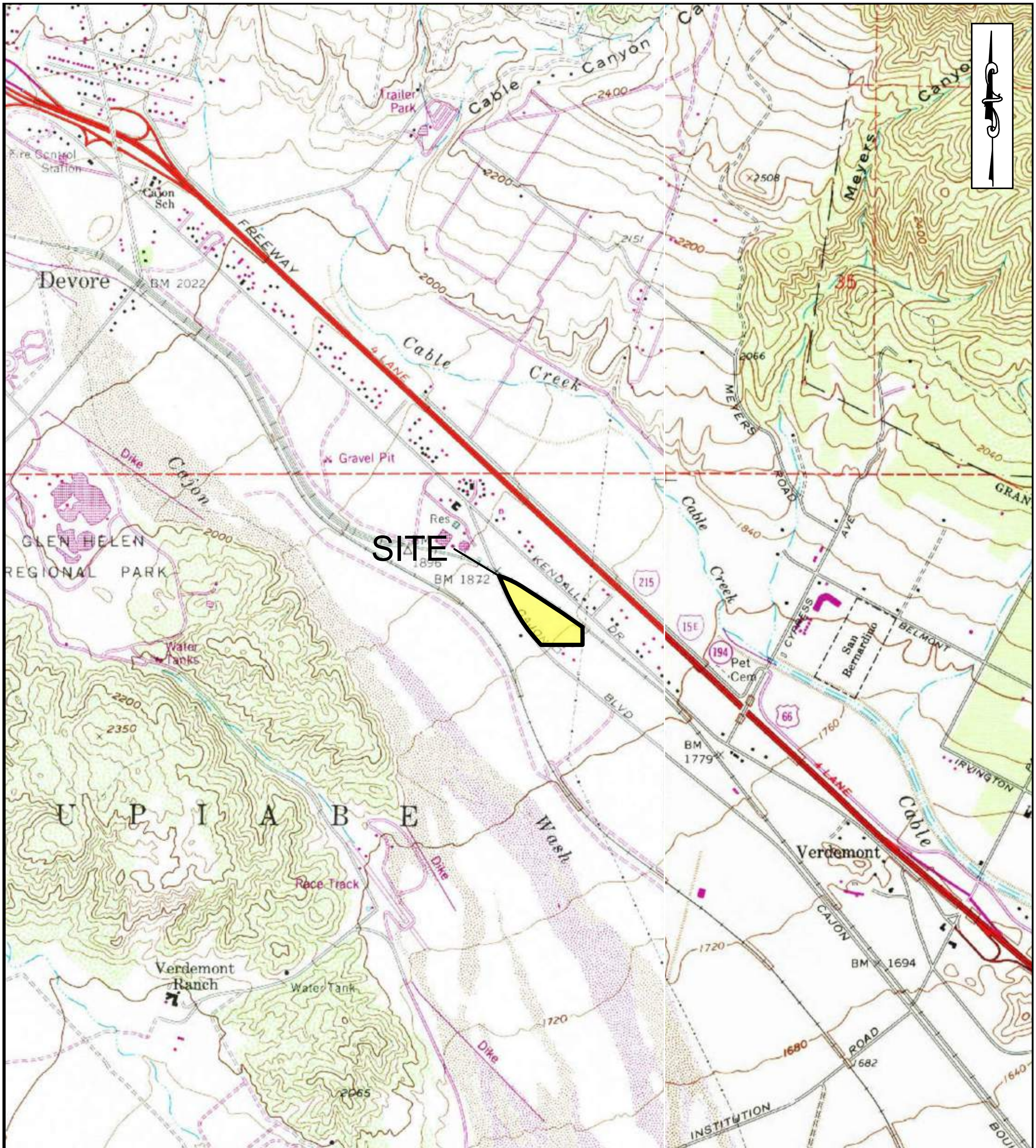
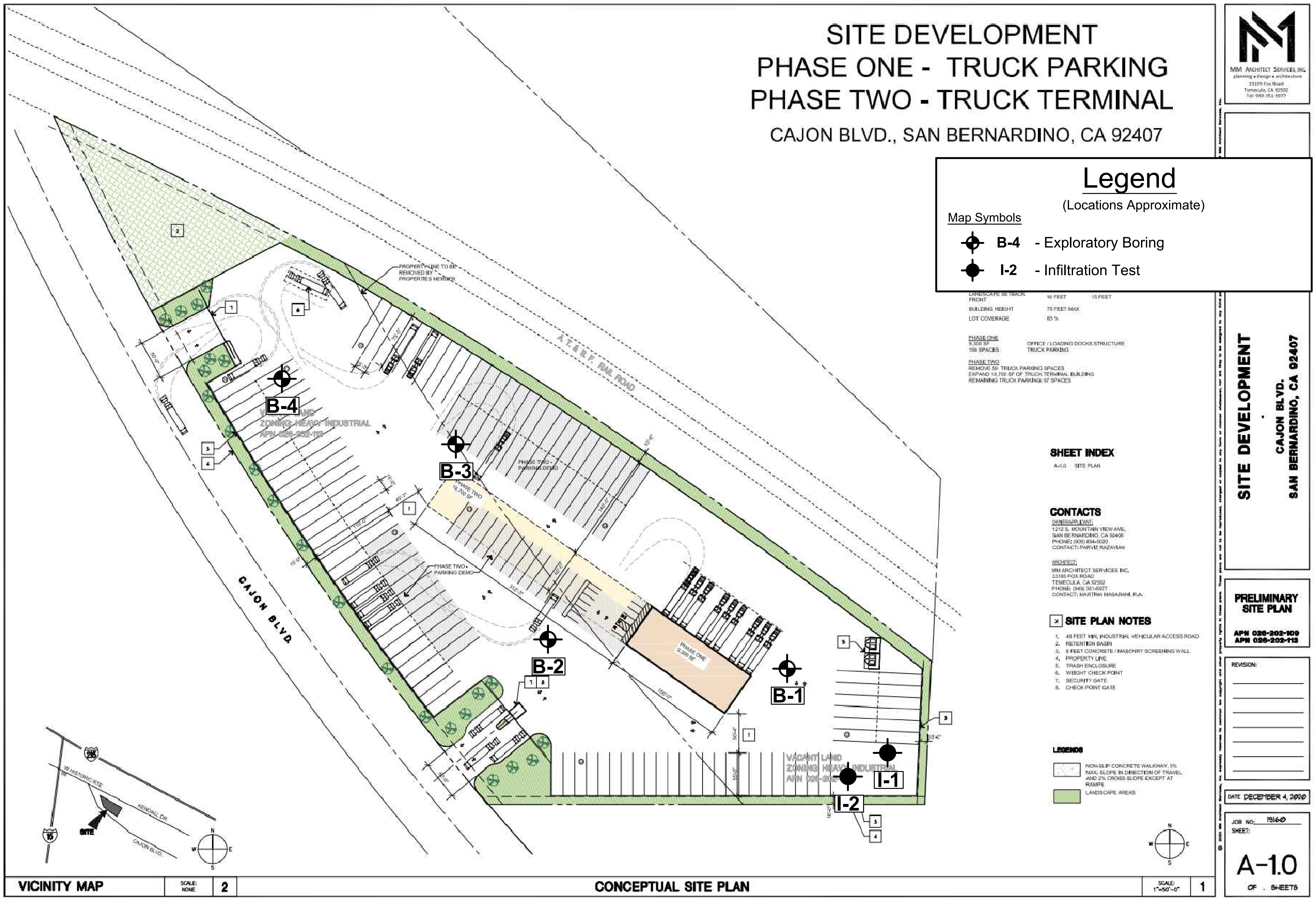
APPENDIX A

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



INDEX MAP

PROJECT:	APN 026-202-109 & 113	**PROJECT NO:**	23720.1
CLIENT:	CAJON BLVD. INDUSTRIAL PARK, LLC	**ENCLOSURE:**	A-1
LOR Geotechnical Group, Inc.		**DATE:**	MAY 2021
		SCALE:	1" ≈ 2,000'



SITE PLAN

PROJECT NO: 23720.1

ENCLOSURE: A-2

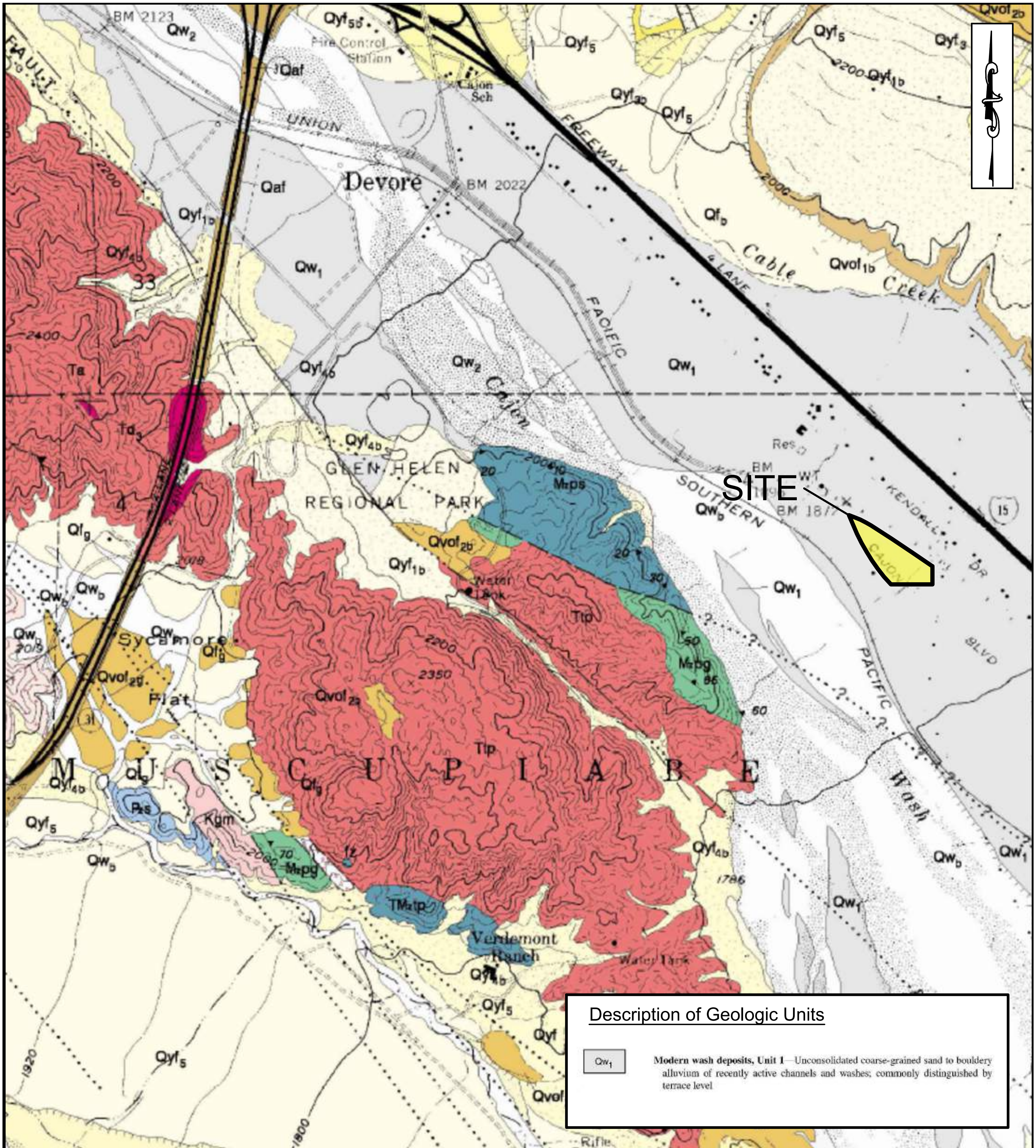
DATE: MAY 2021

SCALE: 1" ≈ 110'

PROJECT: APN 026-202-109 & 113

CLIENT: CAJON BLVD. INDUSTRIAL PARK, LLC

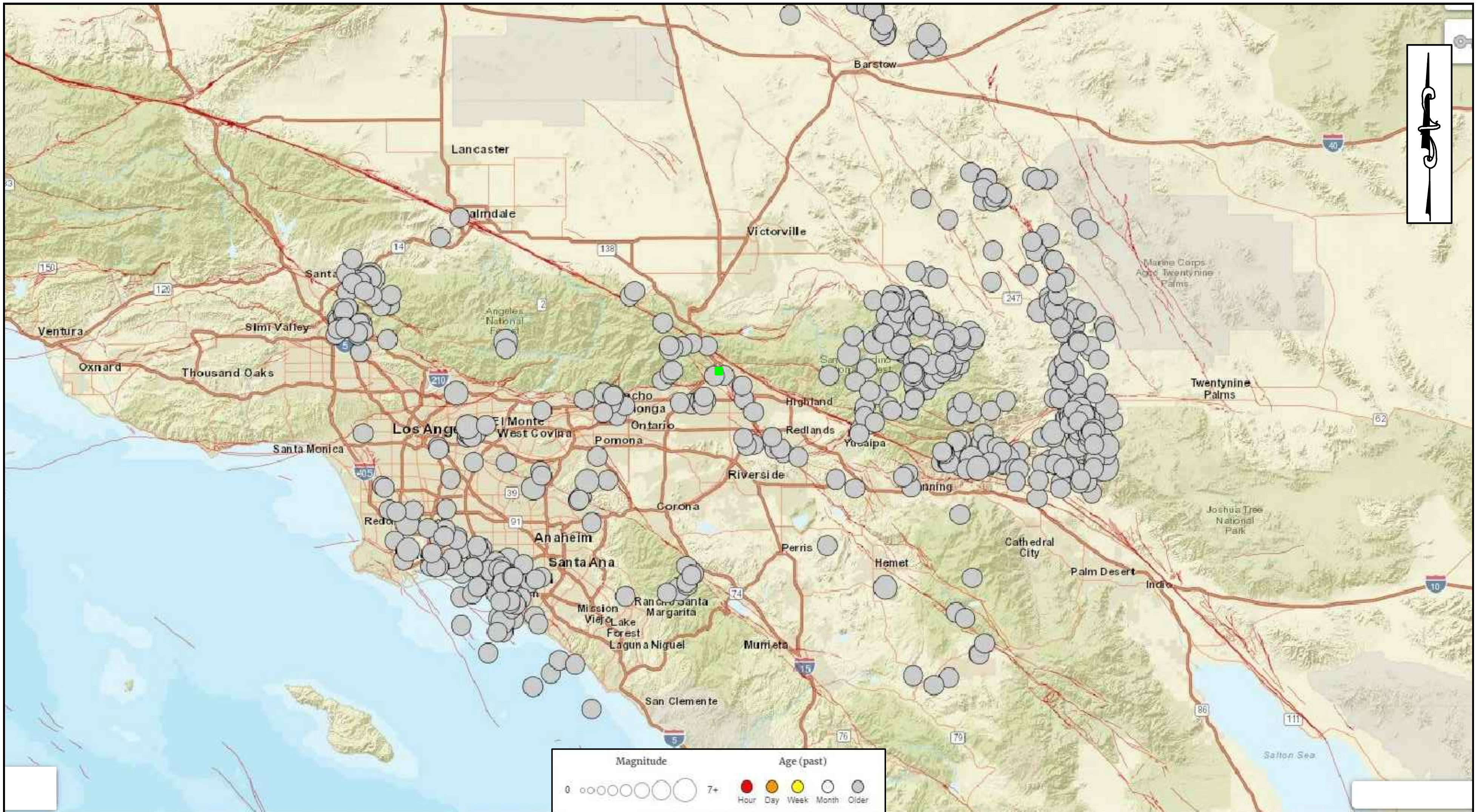
LOR Geotechnical Group, Inc.



REGIONAL GEOLOGIC MAP

(Morton and Matti, 2001)

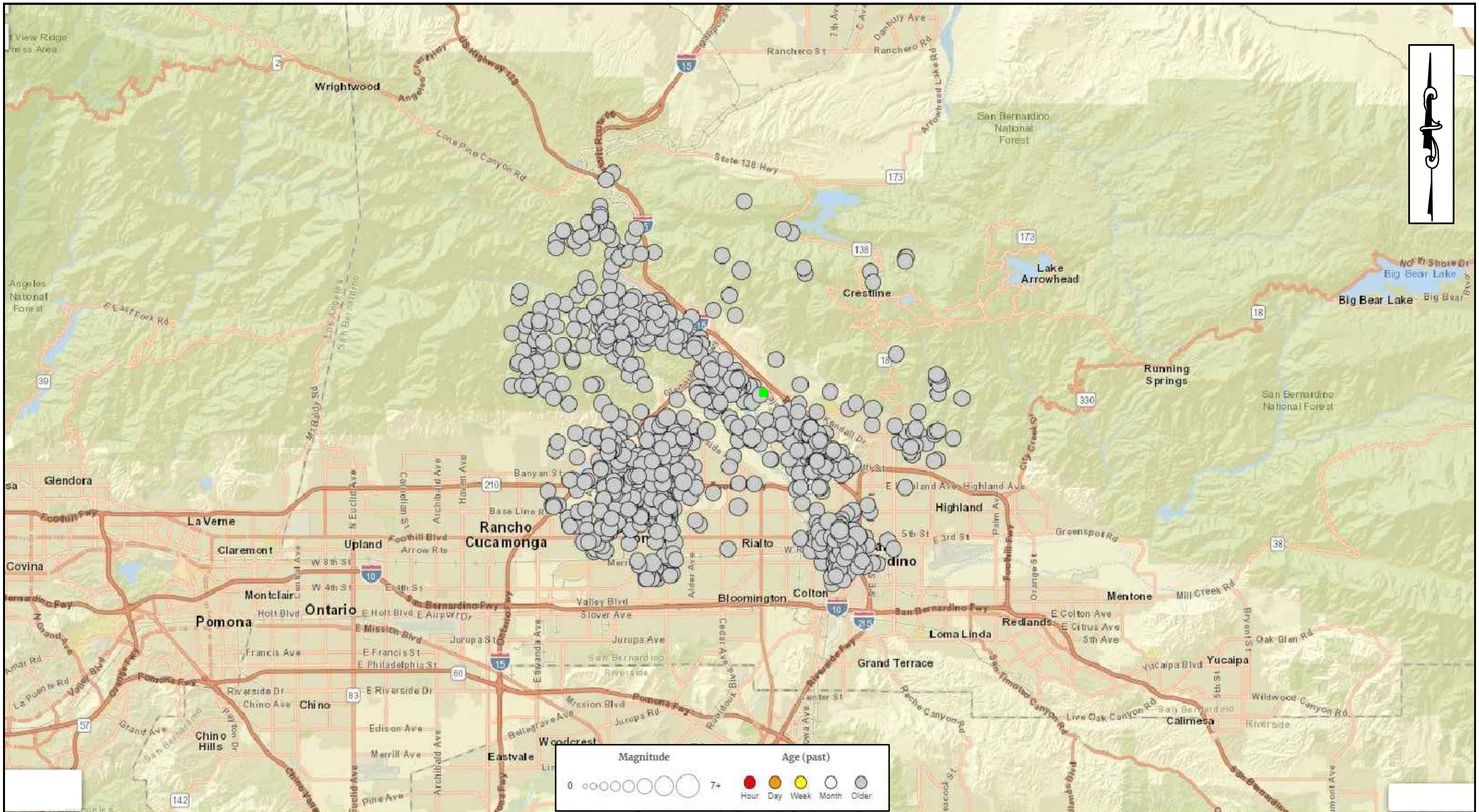
PROJECT:	APN 026-202-109 & 113	**PROJECT NO:**	23720.1
CLIENT:	CAJON BLVD. INDUSTRIAL PARK, LLC	**ENCLOSURE:**	A-3
LOR Geotechnical Group, Inc.		**DATE:**	MAY 2021
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 547 epicenters of instrument-recorded events from 1932 to present (05/11/21) of local magnitude of M4.0 to M10.0 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 evens 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 026-202-109 & 113	**PROJECT NO:**	23720.1
CLIENT:	CAJON BLVD. INDUSTRIAL PARK, LLC	**ENCLOSURE:**	A-4
LOR Geotechnical Group, Inc.		**DATE:**	MAY 2021
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 960 epicenters of instrument-recorded events from 1978 to present (05/11/21) of local magnitude of M2.0 to M10.0 within a radius of ~6.3 miles (10 kilometers) of the site. Location accuracy varies. The site is indicated by the green square. The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These evens are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 10km Radius

PROJECT:	APN 026-202-109 & 113	**PROJECT NO:**	23720.1
CLIENT:	CAJON BLVD. INDUSTRIAL PARK, LLC	**ENCLOSURE:**	A-5
LOR Geotechnical Group, Inc.		**DATE:**	MAY 2021
		SCALE:	1" ≈ 7km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on April 23, 2021 and consisted of the excavation and logging of 4 exploratory borings to refusal depths from approximately 17 feet and 39 feet below the existing ground surface. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

The drilling exploration was conducted using a Mobile B-61 drill rig equipped with 8-inch diameter hollow stem augers. The soils were continuously logged by our geologist who inspected the site, created detailed logs of the borings, obtained undisturbed, as well as disturbed, soil samples for evaluation and testing, and classified the soils by visual examination in accordance with the Unified Soil Classification System.

Relatively undisturbed samples of the subsoils were obtained at a typical interval of 5 feet. The samples were recovered by using a California split barrel sampler of 2.50 inch inside diameter and 3.25 inch outside diameter or a Standard Penetration Sampler (SPT) from the ground surface to the total depth explored. The samplers were driven by a 140 pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), borehole diameter (8"), and rod length at the test depth were considered for further computing of equivalent SPT N-values corrected for field procedures (N60) which are included in the boring logs, Enclosures B-1 through B-4.

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

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-4. A Boring Log Legend and Soil Classification Chart are presented on Enclosures B-I and B-ii, respectively.

CONSISTENCY OF SOIL

SANDS

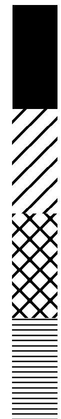
SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

COHESIVE SOILS

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

SAMPLE KEY

Symbol **Description**



INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE

INDICATES BULK SAMPLE

INDICATES SAND CONE OR NUCLEAR DENSITY TEST

INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

1	Atterberg Limits
2	Consolidation
3	Direct Shear (undisturbed or remolded)
4	Expansion Index
5	Hydrometer
6	Organic Content
7	Proctor (4", 6", or Cal216)
8	R-value
9	Sand Equivalent
10	Sieve Analysis
11	Soluble Sulfate Content
12	Swell
13	Wash 200 Sieve

BORING LOG LEGEND

PROJECT:	APN's 026-202-109- & -113, SAN BERNARDINO, CALIFORNIA	PROJECT NO.:	23720.1
CLIENT:	CAJON BLVD., INDUSTRIAL PARK, LLC	ENCLOSURE:	B-i
LOR Geotechnical Group, Inc.		DATE:	MAY 2021

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

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

12" 3" 3/4" No. 4 No. 10 No. 40 200
(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT	**APN's 026-202-109- & -113, SAN BERNARDINO, CALIFORNIA**	**PROJECT NO.**	**23720.1**
CLIENT:	**CAJON BLVD., INDUSTRIAL PARK, LLC**	**ENCLOSURE:**	**B-ii**
LOR Geotechnical Group, Inc.		**DATE:**	**MAY 2021**

LOG OF BORING B-1

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SP SM	**@ 0 feet, <u>ALLUVIUM</u>: POORLY GRADED SAND with SILT, approximately 5% gravel, 5% coarse grained sand, 50% medium grained sand, 35% fine grained sand, and 5% silty fines, light brown, damp, loose.**
	23	3, 7, 10, 11	2.6		113.2				**@ 1 foot, loose to 1.5±', becomes medium dense with increase in depth.**
5	23		2.6						**@ 5 feet, POORLY GRADED SAND with SILT and GRAVEL, increase in gravel content (approximately 20% gravel to 2.5" diameter), sample disturbed.**
	35		2.9		113.6				**@ 7 feet, slightly coarser grained overall**
10	36		2.9		120.6				
15	77		2.3		122.7				**@ 16 feet, abundant gravel/cobbles.**
20	70		3.2		121.4				
25	77		2.8		112.9				**@ 25 feet, trace to minor amounts of large gravels and cobbles.**
30	72		3.5		112.5				
35	54 for 3"								**@ 35 feet, no recovery of sample.** **@ 36 feet, abundant gravel and cobbles.**
									END OF BORING @ 39' due to refusal on rocks **No fill** **No groundwater** **No bedrock**
40									
45									

PROJECT:	**APN's 026-202-109 & -113**	PROJECT NUMBER:	**23720.1**
CLIENT:	**Cajon Blvd., Industrial Park**	ELEVATION:	**--**
		DATE DRILLED:	**April 23, 2021**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Mobile B-61**
		HOLE DIA.: **8"**	ENCLOSURE: **B-1**

LOG OF BORING B-2

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SP SM	@ 0 feet, ALLUVIUM: **POORLY GRADED SAND with SILT, approximately 5% gravel, 5% coarse grained sand, 50% medium grained sand, 35% fine grained sand, and 5% silty fines, light brown, damp, loose.**
	16		2.0		105.9				**@ 1 foot, loose to 1.5±' and contains minor silt, becomes medium dense with increase in depth.**
5	56 for 10"		1.6						**@ 5 feet, increase in gravel/cobbles, sample disturbed.**
	27		2.1		115.7				
10	52		2.8						**@ 10± feet, increase in gravel, sample disturbed.**
15	62		2.9		116.2				**@ 15 feet, coarser grained, increase in coarse grained sand and fine gravel.**
20	62		2.3		121.0				
25	76		2.7		112.7				**@ 25 feet, very dense, increase in rock content.**
30	54 for 4"		1.7		108.3				**@ 30 feet, cobbles, no recovery of sample.**
									END OF BORING @ 30.5' due to refusal on rocks
									No fill
									No groundwater
									No bedrock
35									

PROJECT:	**APN's 026-202-109 & -113**	PROJECT NUMBER:	**23720.1**
CLIENT:	**Cajon Blvd., Industrial Park**	ELEVATION:	**--**
		DATE DRILLED:	**April 23, 2021**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Mobile B-61**
		HOLE DIA.: **8"**	ENCLOSURE: **B-2**

LOG OF BORING B-3

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, ALLUVIUM: SILTY SAND, approximately 5% gravel, 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, and 20% silty fines, light brown, damp, loose.
	15		1.7		108.3			SP	@ 1.5 feet, POORLY GRADED SAND with GRAVEL, approximately 15% fine gravel, 15% coarse grained sand, 45% medium grained sand, 20% fine grained sand, and 5% silty fines, brown, damp, medium dense.
		3, 7, 10, 11							
5	20		2.4		114.9				
	43		3.0		115.3				@ 7 feet, dense
10	40		3.5		117.7				@ 10 feet, much coarser grained overall.
15	46 for 4"								@ 15 feet, cobbles, no recovery of sample.
									@ 16 feet, increase in rock content.
									END OF BORING @ 17' due to refusal on rocks No fill No groundwater No bedrock
20									

TEST DATA

PROJECT:	APN's 026-202-109 & -113	PROJECT NUMBER:	23720.1
CLIENT:	Cajon Blvd., Industrial Park	ELEVATION:	--
		DATE DRILLED:	April 23, 2021
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-3

LOG OF BORING B-4

DEPTH IN FEET	TEST DATA: SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		8, 11						SM	@ 0 feet, ALLUVIUM: SILTY SAND, approximately 5% gravel, 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, and 20% silty fines, light brown, damp, loose.
	10		1.0					SP	@ 2 feet, POORLY GRADED SAND, approximately 10% fine gravel, 15% coarse grained sand, 45% medium grained sand, 25% fine grained sand, and 5% silty fines, brown, damp, medium dense, sample disturbed.
5	17		1.3						@ 5 feet, sample disturbed.
	31		1.9		109.6				@ 7 feet, increase in gravel.
10	29		2.3		118.2				@ 10 feet, coarser grained overall.
									@ 12 feet, abundant gravel/cobbles, hard drilling.
15	57		2.2		123.5				
20	82 for 11"		1.4		117.7				@ 20 feet, abundant gravel/cobbles.
									END OF BORING @ 23' due to refusal on rocks No fill No groundwater No bedrock
25									

PROJECT:	APN's 026-202-109 & -113	PROJECT NUMBER:	23720.1
CLIENT:	Cajon Blvd., Industrial Park	ELEVATION:	--
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	April 23, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	ENCLOSURE: B-4

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

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

Moisture Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, in accordance with ASTM D 2922 and ASTM D 2216, respectively, and the results are shown on the Boring Logs, Enclosures B-1 through B-4 for convenient correlation with the soil profile.

Laboratory Compaction

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

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-1	1-4	(SP-SM) Poorly Graded Sand w/ silt	118.0	10.0
B-3	4-7	(SP) Poorly Graded Sand w/ gravel	125.5	9.5

Direct Shear Tests

Shear tests are performed with a direct shear machine in general accordance with ASTM D 3080 at a constant rate-of-strain (usually 0.04 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 represent the worst case conditions expected in the field.

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

DIRECT SHEAR TESTS				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Angle of Internal Friction (degrees)**	**Apparent Cohesion (psf)**
B-1	1-4	(SP-SM) Poorly Graded Sand w/ silt	33	500
B-3	4-7	(SP) Poorly Graded Sand w/ gravel	45	100

Sieve Analysis

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

R-Value Test

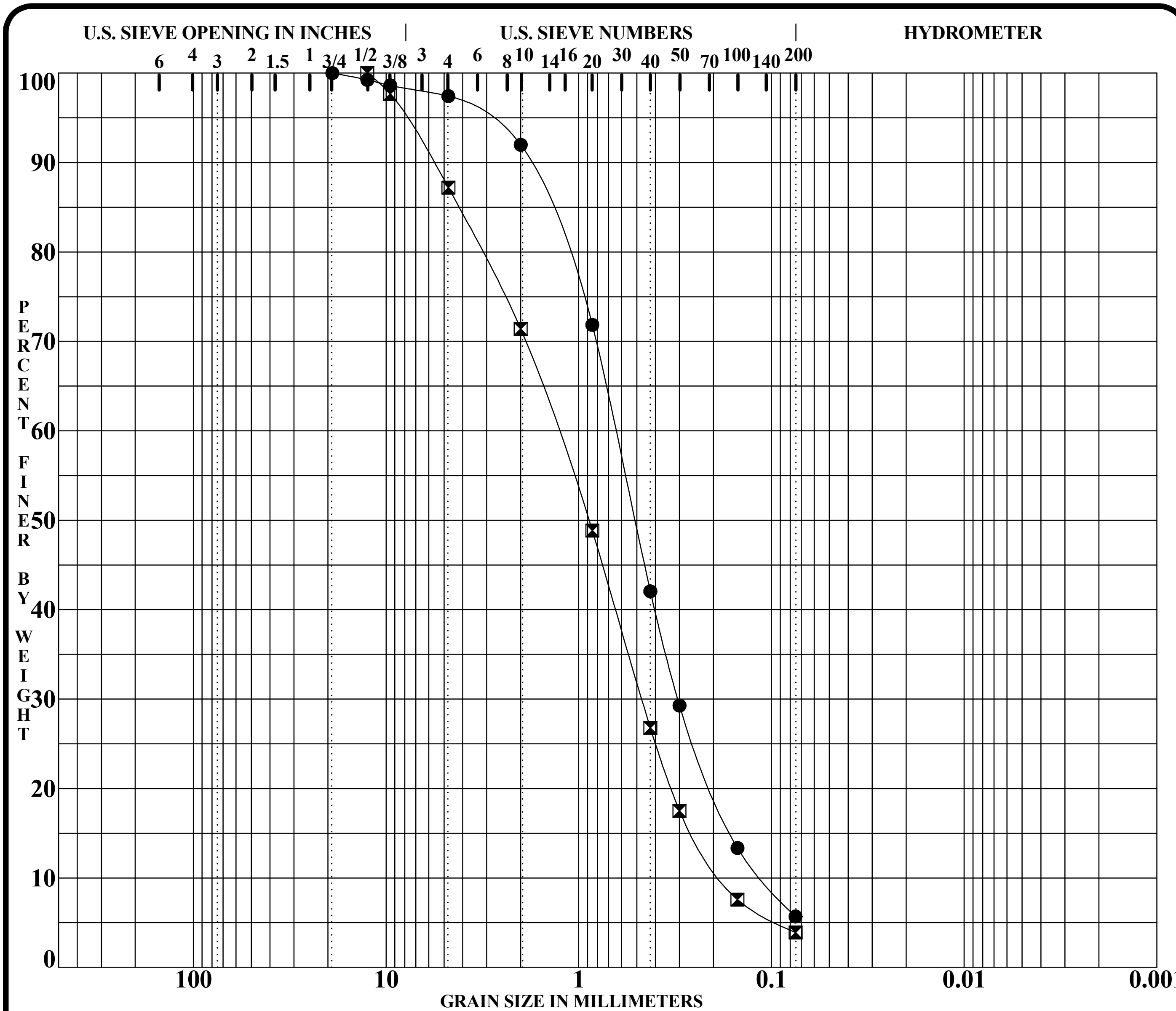
A representative soil sample was obtained at probable pavement subgrade level and was tested to determine its R-value using the California R-Value Test Method, Caltrans Number 301. The result of the R-value test is presented on the following table:

R-VALUE TEST			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**R-Value**
B-4	0-3	(SP-SM) Poorly Graded Sand with silt	75

Soluble Sulfate Content Tests

The soluble sulfate content of selected subgrade soils was evaluated and 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			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Sulfate Content (percent by weight)**
B-1	1-4	(SP-SM) Poorly Graded Sand w/ silt	< 0.005
B-3	4-7	(SP) Poorly Graded Sand w/ gravel	< 0.005
B-4	0-3	(SP-SM) Poorly Graded Sand with silt	< 0.005



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-1*	*@ 1-4 ft*	*(SP-SM) Poorly Graded Sand w/ Silt*	--	--			*1.31*	*5.8*
⊠	*B-3*	*@ 4-7 ft*	*(SP) Poorly Graded Sand*	--	--			*0.96*	*7.3*

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*@ 1-4 ft*	*19.00*	*0.65*	*0.306*	*0.1109*	*2.6*	*91.7*	*5.7*	
⊠	*B-3*	*@ 4-7 ft*	*12.50*	*1.30*	*0.470*	*0.1777*	*12.8*	*83.3*	*3.9*	

PROJECT *APN's 026-202-109 & -113*

PROJECT NO. *23720.1*

DATE *5/13/21*

GRADATION CURVES
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ENCLOSURE C-1

APPENDIX D

Infiltration Test Results

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN 0262-021-09, -13	Test Date:	April 23, 2021
Project No.:	23720.1	Test Hole No.:	P-1
Soil Classificaiton:	(SW) Well graded sand	Test Hole Diameter:	8.0 in.
Depth of Test Hole:	5.0 ft.	Date Excavated:	April 23, 2021
Tested By:	Andrew L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	in/hr
1	9:40 AM	9:42 AM	2	0.03	0.03	36.00	60.00	60.00	60.00	24.00	12.00	720.0
2	9:43 AM	9:45 AM	2	0.03	0.07	36.00	60.00	60.00	60.00	24.00	12.00	720.0
3	9:46 AM	9:47 AM	1	0.02	0.08	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
4	9:48 AM	9:49 AM	1	0.02	0.10	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
5	9:50 AM	9:51 AM	1	0.02	0.12	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
6	9:52 AM	9:53 AM	1	0.02	0.13	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
7	9:54 AM	9:55 AM	1	0.02	0.15	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
8	9:56 AM	9:57 AM	1	0.02	0.17	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
9	9:58 AM	9:59 AM	1	0.02	0.18	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
10	10:00 AM	10:01 AM	1	0.02	0.20	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
11	10:02 AM	10:03 AM	1	0.02	0.22	36.00	58.00	60.00	60.00	22.00	13.00	1320.0
12	10:04 AM	10:05 AM	1	0.02	0.23	36.00	58.00	60.00	60.00	22.00	13.00	1320.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	24.00	
H_f	2.00	
ΔH	22.00	
H_{avg}	13.00	
I_t	176.00	in/hr (clear water rate)

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN 0262-021-09, -13	Test Date:	April 23, 2021
Project No.:	23720.1	Test Hole No.:	P-2
Soil Classificaiton:	(SW) Well graded sand	Test Hole Diameter:	8.0 in.
Depth of Test Hole:	5.3 ft.	Date Excavated:	April 23, 2021
Tested By:	Andrew L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	in/hr
1	10:10 AM	10:11 AM	1	0.02	0.02	36.00	63.00	63.00	63.00	27.00	13.50	1620.0
2	10:12 AM	10:13 AM	1	0.02	0.03	36.00	62.00	63.00	63.00	26.00	14.00	1560.0
3	10:14 AM	10:15 AM	1	0.02	0.05	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
4	10:16 AM	10:17 AM	1	0.02	0.07	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
5	10:18 AM	10:19 AM	1	0.02	0.08	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
6	10:20 AM	10:21 AM	1	0.02	0.10	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
7	10:22 AM	10:23 AM	1	0.02	0.12	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
8	10:24 AM	10:25 AM	1	0.02	0.13	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
9	10:26 AM	10:27 AM	1	0.02	0.15	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
10	10:28 AM	10:29 AM	1	0.02	0.17	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
11	10:30 AM	10:31 AM	1	0.02	0.18	36.00	60.00	63.00	63.00	24.00	15.00	1440.0
12	10:32 AM	10:33 AM	1	0.02	0.20	36.00	60.00	63.00	63.00	24.00	15.00	1440.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	27.00	
H_f	3.00	
ΔH	24.00	
H_{avg}	15.00	
I_t	169.41	in/hr (clear water rate)

APPENDIX E

Seismic Design Spectra

Project: APN's 026-202-109 & -113
Project Number: 23720.1
Client: Cajon Blvd. Industrial Park, LLC
Site Lat/Long: 34.20274/-117.38191
Controlling Seismic Source: San Jacinto / San Andreas

REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.3(ii)	F_v	2.5
$0.2*(S_{D1}/S_{DS})$	T_0	0.138
S_{D1}/S_{DS}	T_S	0.692
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	1.0993*
Equation 11.4-2 - F_V*S_1	S_{M1}	1.649*

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	2.382
Design Maps	S_1	0.970
Equation 11.4-1 - F_A*S_S	S_{MS}	2.382*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.588*
Design Maps	PGA	1.008
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	1.109*
Section 21.5.3	80% of PGA_M	0.887
Design Maps	C_{RS}	0.897
Design Maps	C_{R1}	0.881

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.897
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.881

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

Period	Cr
0.200	0.897
0.300	0.895
0.400	0.893
0.500	0.891
0.600	0.889
0.680	0.887
1.000	0.881

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

Mapped values from https://seismicmaps.org/

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PROBABILISTIC SPECTRA[1]
2% in 50 year Exceedence

Project No: 23720.1

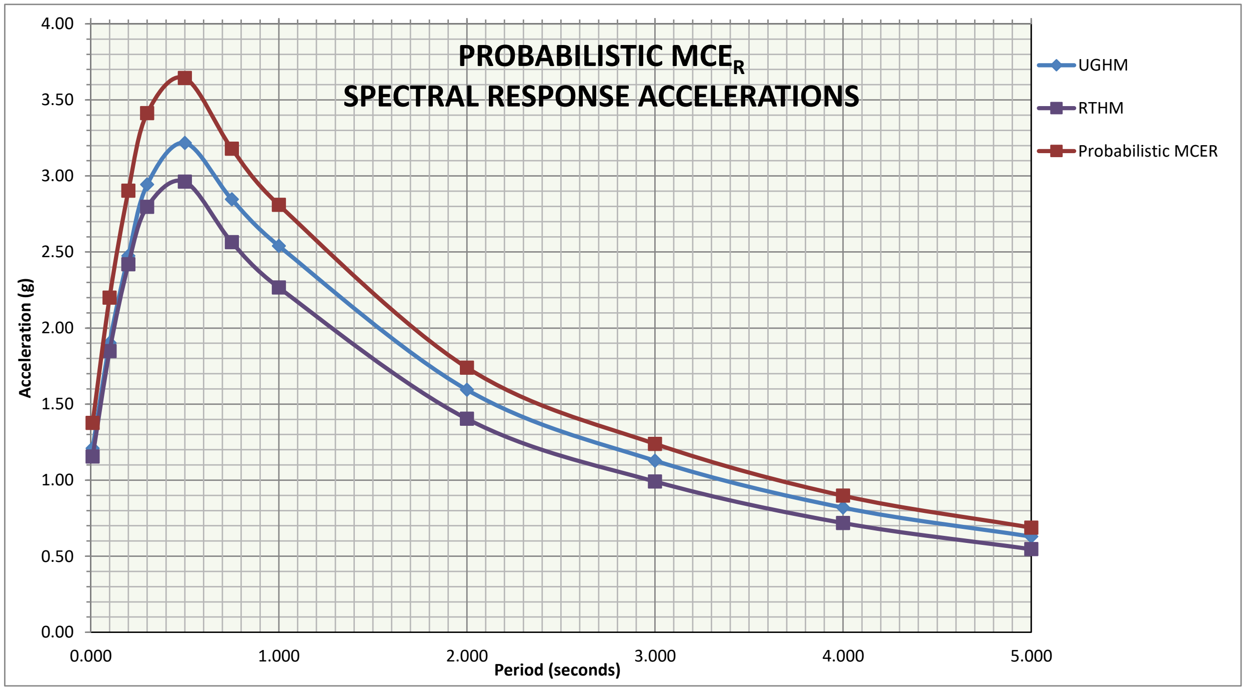
Period	UGHM	RTHM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	1.209	1.156	1.19	1.376
0.100	1.902	1.848	1.19	2.199
0.200	2.475	2.419	1.20	2.903
0.300	2.943	2.797	1.22	3.412
0.500	3.217	2.962	1.23	3.643
0.750	2.846	2.564	1.24	3.179
1.000	2.540	2.266	1.24	2.810
2.000	1.594	1.403	1.24	1.740
3.000	1.128	0.991	1.25	1.239
4.000	0.819	0.718	1.25	0.898
5.000	0.629	0.546	1.26	0.688

Probabilistic PGA: 1.209

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 / San Andreas

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

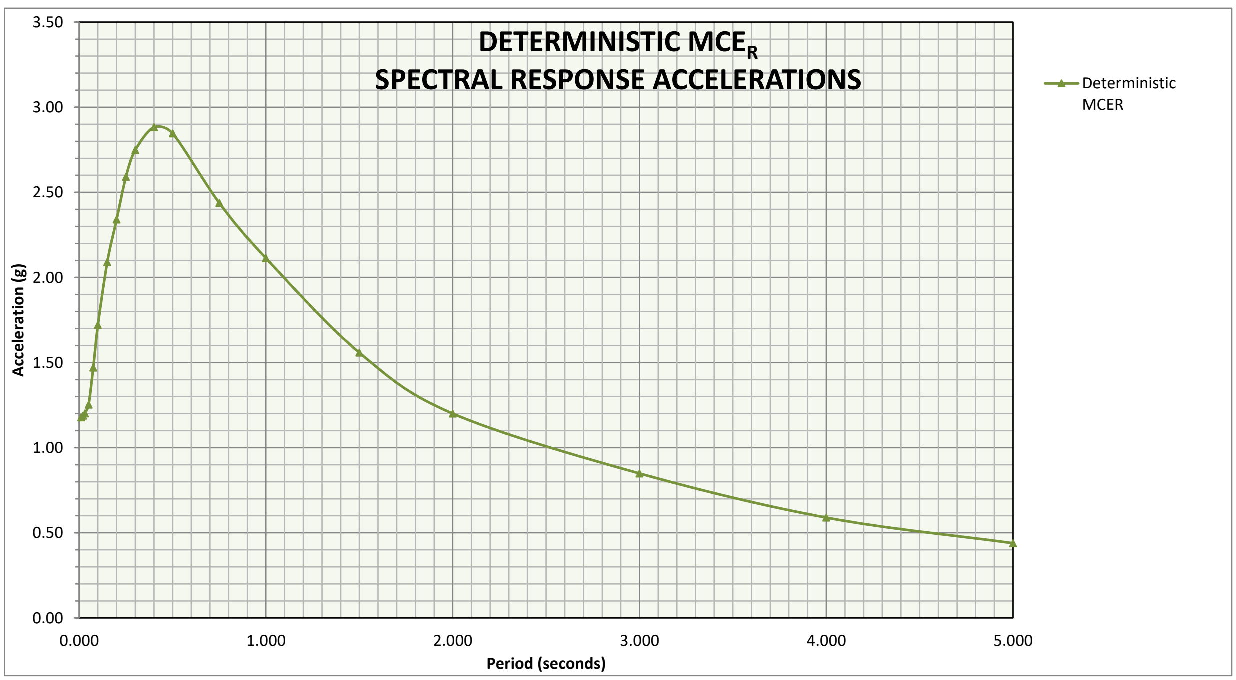
Project No: 23720.1

Period	Deterministic PSa Median + 1.σ for 5% Damping	Max Directional Scale Factor[2]	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.990	1.19	1.178	1.178
0.020	0.998	1.19	1.188	1.188
0.030	1.010	1.19	1.202	1.202
0.050	1.053	1.19	1.253	1.253
0.075	1.236	1.19	1.471	1.471
0.100	1.446	1.19	1.721	1.721
0.150	1.742	1.20	2.090	2.090
0.200	1.950	1.20	2.340	2.340
0.250	2.141	1.21	2.591	2.591
0.300	2.253	1.22	2.748	2.748
0.400	2.343	1.23	2.882	2.882
0.500	2.314	1.23	2.846	2.846
0.750	1.966	1.24	2.437	2.437
1.000	1.704	1.24	2.113	2.113
1.500	1.256	1.24	1.558	1.558
2.000	0.968	1.24	1.200	1.200
3.000	0.679	1.25	0.849	0.849
4.000	0.472	1.25	0.589	0.589
5.000	0.348	1.26	0.439	0.439

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	0.990
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	1.376	1.178	1.178	0.786
0.100	2.199	1.721	1.721	1.147
0.200	2.903	2.340	2.340	1.560
0.300	3.412	2.748	2.748	1.832
0.500	3.643	2.846	2.846	1.897
0.750	3.179	2.437	2.437	1.625
1.000	2.810	2.113	2.113	1.409
2.000	1.740	1.200	1.200	0.800
3.000	1.239	0.849	0.849	0.566
4.000	0.898	0.589	0.589	0.393
5.000	0.688	0.439	0.439	0.293

ASCE 7-16: Section 21.4
Site Specific

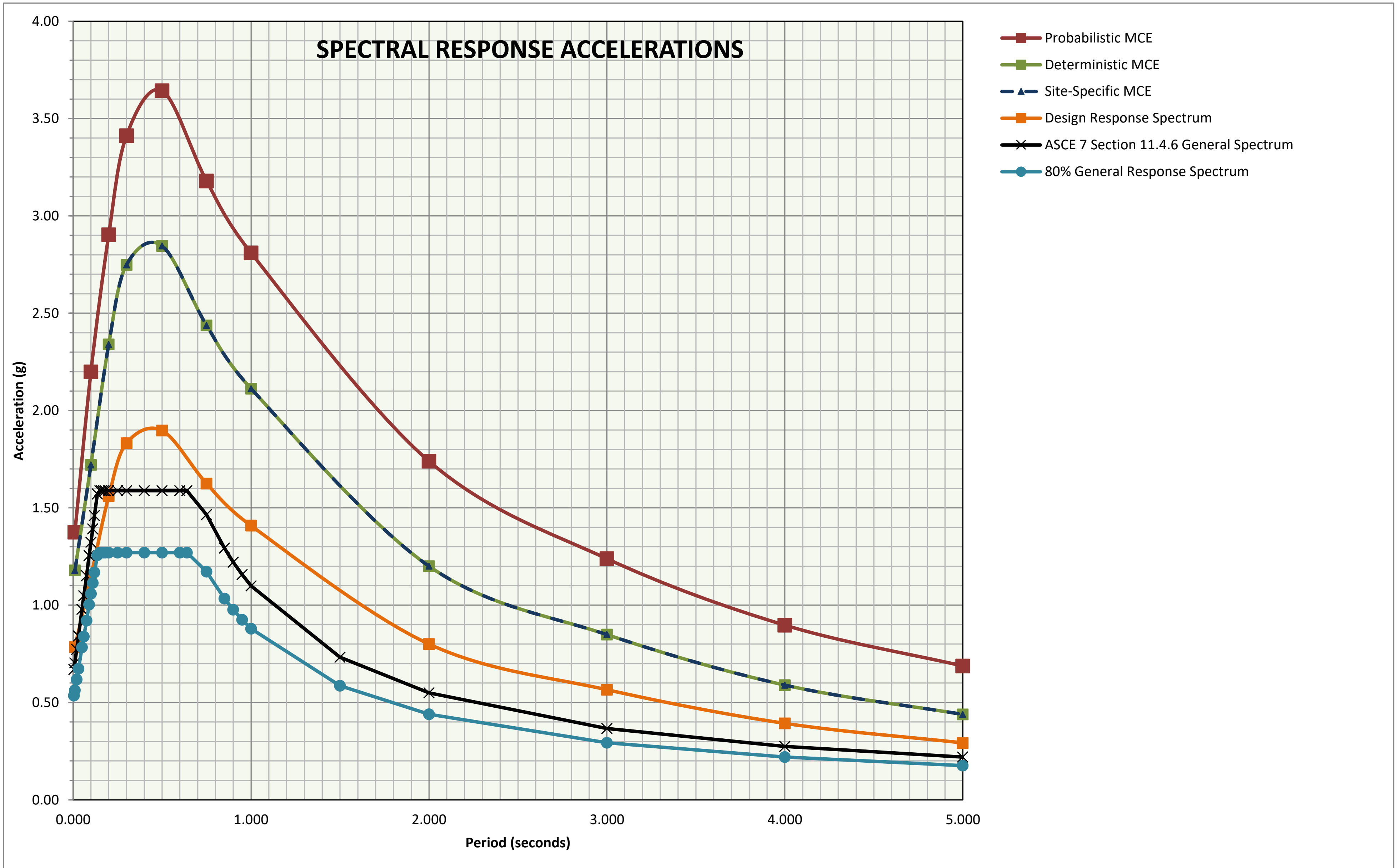
	Calculated Value	Design Value
SDS:	1.707	1.707
SD1:	1.698	1.698
SMS:	2.561	2.561
SM1:	2.546	2.546
Site Specific PGAm:	0.990	0.990
Site Class:	D measured	

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

* Risk Categories I, II, or III

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.670	0.536
0.010	0.704	0.563
0.020	0.773	0.618
0.030	0.842	0.673
0.050	0.979	0.783
0.060	1.048	0.838
0.075	1.151	0.921
0.090	1.255	1.004
0.100	1.323	1.059
0.110	1.392	1.114
0.120	1.461	1.169
0.136	1.571	1.257
0.150	1.588	1.270
0.160	1.588	1.270
0.170	1.588	1.270
0.180	1.588	1.270
0.200	1.588	1.270
0.250	1.588	1.270
0.300	1.588	1.270
0.400	1.588	1.270
0.500	1.588	1.270
0.600	1.588	1.270
0.640	1.588	1.270
0.750	1.466	1.173
0.850	1.293	1.035
0.900	1.221	0.977
0.950	1.157	0.926
1.000	1.099	0.879
1.500	0.733	0.586
2.000	0.550	0.440
3.000	0.366	0.293
4.000	0.275	0.220
5.000	0.220	0.176

Project No: 23720.1



Project No: 23720.1