APPENDIX F1

CORSICA SITE GEOTECHNICAL INVESTIGATION

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED INDUSTRIAL DEVELOPMENT APNs 330-180-006, -010, -029, AND -046 MENIFEE, CALIFORNIA

PROJECT NO. 23758.1
DECEMBER 30, 2021

Prepared For:

Compass Danbe Real Estate Partners LLC
523 Main Street
El Segundo, California 90245

Attention: Mr. Mark Bachli

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

December 30, 2021

Compass Danbe Real Estate Partners LLC
8151 Auto Drive
Riverside, California 92504

Project No. 23758.1

Attention: Mr. Mark Bachli

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Industrial Development, APNs 330-180-006, -010, -029,and -046, Menifee, California.

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

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. Any undocumented fill material and any loose alluvial materials should be removed from structural areas and areas to receive engineered compacted fills. The data developed during this investigation indicates that removals on the order of approximately 2 to 3 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.

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

Non-conducive infiltration rates were obtained for the soils tested.

LOR Geotechnical Group, Inc.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

INTRODUCTION

During December of 2021, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed industrial development of Assessor's Parcel Numbers (APNs) 330-180-006, -010, -029, and -046, Menifee, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photographs of the site and surrounding regions dated 1966 through 2021;
- Geologic field reconnaissance mapping to verify the aerial distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Percolation testing via the borehole test method;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

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

PROJECT CONSIDERATIONS

To orient our investigation at the site, a Site Plan prepared by CASC Engineering and Consulting, dated December 3, 2021, was furnished for our use. The existing site conditions and proposed building configurations, associated driveways, parking, and landscape areas were indicated on this plan. The Site Plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A. As noted on the site plan, development of the site will include three industrial type structures; a 159,502± square foot building with nineteen (19) dock doors, a 89,156± square foot building with eleven (11) dock doors, and a 35,459± square foot building with four (4) dock doors with the remainder of the property to be used for driveways, parking, and landscape areas. The buildings are anticipated to be of concrete, masonry, or similar type construction and light to moderate foundation loads are anticipated with these structures.

Infiltration is proposed via underground chamber type systems. Depths and locations were provided by CASC Engineering and Consulting.

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

AERIAL PHOTO ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photograph images of varying scales. We reviewed imagery available from Google Earth Pro (2021) computer software and from online Historic Aerials (2021).

To summarize briefly, the site was vacant land utilized for dry land farming since 1966, the earliest photograph available, until the 2002 photograph. The 2002 photograph shows the western approximate one-third of APN 330-180-010 was fenced and contained a small structure, perhaps a residence (mobile home) in the eastern portion. In the 2003 photograph, the residence was no longer present on APN 330-180-010, however, a large slab was present in the western portion and cars and other small items were present. The existing residence on APN 330-180-029 was also present in the 2003 photograph. By 2005, the fencing, cars, and other items were no longer present on APN 330-180-010. The 2009 photograph shows two additional outbuildings, one just south of the existing residence and one along the northern property line are present on APN 330-180-029. Numerous shade structures and animal pens are present along the western boundary of APN 330-180-029 in the 2018 photograph. No evidence for the presence of faults traversing the site area or mass movement features was noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The approximate 14-acre site is located within the northwestern portion of the city of Menifee, California. It consists of mostly vacant land. The exception is the previously noted large concrete slab along the far western portion of APN 330-180-010 and the residence, outbuildings, shade structures, and animal pens on APN 330-180-029. A water well is also present on APN 330-180-029. Details regarding the depth of the well and the depth to water are not known. The property is partially situated along the south and west side of Corsica Lane, an unimproved roadway, along the east side of Goetz Road, a partially improved roadway, and partially along the west side of Wheat Street, an unimproved roadway. Concrete K-rails are present along the south side of APN 330-180-046. Very sparse weeds cover the undeveloped portions of the site. Several large trees are present within the developed residential portion of the site (APN 330-180-029). The undeveloped areas of the site were recently disced. Topographically, the site is planar with a gentle fall to the north-northwest.

Power lines and vacant land bound the site on the south. A tract of single family homes is present to the west of Goetz Road. Large lot residential properties and vacant land lie north, northeast, and east of the site.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on December 6 and 7, 2021. The work consisted of advancing a total of 12 exploratory borings using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. The approximate locations of our exploratory borings are presented on Enclosure A-2, within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by a geologist from this firm. The borings were drilled to maximum depths of 15.25 to 30.42 feet below the existing ground surface. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet, and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the boring logs is presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

As shown on Enclosure A-1, within Appendix A, the site is located within the United States Geological Survey Romoland 7.5 minute quadrangle topographic map. This region lies along the north-central portion of the Perris block of the northern Peninsular Ranges geologic province of southern California. While the Perris block is considered to be a relatively stable structural block, it is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the east, and the Cucamonga fault zone on the north. The Perris block is underlain by rocks of the Peninsular Ranges batholith, a very large mass of crystalline igneous rocks of Cretaceous age and with no known floor, and by prebatholithic metasedimentary and metavolcanic rocks of older ages.

The Perris block has a series of erosional surfaces, marked by low topographic relief and capped with unconsolidated alluvial sediments stripped from the surrounding highlands. This area was mapped by the California Division of Mines and Geology as being underlain by deposits of old alluvial fan deposits (Morton, 2003).

The interior of the Perris Plain is considered to be relatively stable with few known active faults. However, this plain is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the northeast, the San Andreas fault zone on the north, and the Agua-Tibia fault zone on the south. As the subject site is located near the western margin of the Perris Plain, the Elsinore fault is the closest known active fault in relation to the site. At its closest approach, the Elsinore fault is located approximately 12.6 kilometers (7.8 miles) southwest from the site. A complete listing of the distances to known active faults in relation to the various planning areas is given in the Faulting section of this report.

The site is shown within the regional geologic setting as mapped by the U.S.G.S. on the enclosed Regional Geologic Map, Enclosure A-3, within Appendix A.

Site Geologic Conditions

Fill/Topsoil: Fill/topsoil materials were encountered within our exploratory borings located within the currently undeveloped portion of the site to depths of approximately 2 feet. Minor clean sand fill (arena sand) was encountered within one of our two borings placed within APN 330-180-029. The fill/topsoil materials are believed to be associated with current and past weed abatement (discing) practices at the site. As encountered, the fill/topsoil materials were comprised of lean clay with sand, silty sand with clay, and clayey sand which were predominantly red-brown, dry, and in a loose state. Locally, deeper fills are anticipated to be present and primarily associated with the existing development in APN 330-180-029. Expansion index testing indicates that these materials will have a low expansion potential when used as compacted fill.

Older Alluvium: Older alluvial materials were encountered underlying the fill materials described above and at the surface within 5 of our exploratory borings. The older alluvial soils encountered were a maximum of approximately 8 feet in thickness and rest upon bedrock materials. These units were noted to mainly consist of lean clay with sand with minor units of silty sand with clay and clayey sand. The older alluvial materials were in a relatively medium dense to very stiff/very dense state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Expansion index testing indicates that these materials will have a low expansion potential when used as compacted fill.

Bedrock: Bedrock materials were encountered within all of our exploratory borings at depths of approximately 2 to 8 feet. Igneous bedrock was encountered within our exploratory borings placed within the eastern approximate two-thirds of the site (boring B-1 through B-7 and B-10). Metamorphic bedrock was encountered within our exploratory borings placed within the western approximate one-third of the site (borings B-8, B–9, B-11, and B-12).

The igneous bedrock was gabro in composition which was typically coarse grained, severely to moderately weathered upon first encounter becoming less weathered with depth, dry to damp, and in a hard to very hard state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Refusal was experienced within one boring (B-8) at approximately 18 feet.

The metamporphic bedrock was phyllite in composition which was typically fine grained, severely to highly weathered upon first encounter becoming less weathered with depth, damp, and in a hard to very hard state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Refusal was experienced within one boring (B-12) at approximately 16 feet.

A detailed description of the subsurface soil conditions as encountered within our exploratory borings, is presented on the Boring Logs within Appendix B. Excluding the surficial layer of fill/topsoil, the natural earth materials encountered during this investigation are shown on Enclosure A-2.

Groundwater Hydrology

Groundwater was not encountered within any of our exploratory borings as advanced to a maximum depth of approximately 30.42 feet below the existing ground surface nor was any groundwater seepage observed during our site reconnaissance.

In order to estimate the approximate depth to groundwater in the site area, a search was conducted for local groundwater (well) level measurements within the Cooperative Well Measuring Program, Spring 2021 (Watermaster Support Services et al., 2021). This database contains depth to groundwater measurements dating back to 1993. We also conducted a search of the water well database information provided in the California Department of Water Resources (CDWR) Water Library Data website (CDWR, 2021).

The only database with nearby well records was the CDWR database. One well, State Well No. 05S03W17A001S, located approximately 1 kilometer (0.62 miles) to the northeast was identified. Data for this well was limited to one reading in 1995. A measuring point elevation of 1,424± feet above mean sea level was reported. The depth provided was 22 feet (elevation of 1,402± feet above mean sea level).

As noted on Enclosure A-2, the lowest elevation of the site is 1,456 feet above mean sea level. Based on the information above, groundwater in the region appears to be at depths on the order of 50 feet below the ground surface. Groundwater may seep into the bedrock beneath the site region along fractures and joints within the bedrock, the presence of hard bedrock beneath the site generally precludes the development of groundwater conditions or a groundwater table in these areas. Any groundwater that might be encountered during site development would likely be the result of infiltration of surface waters/irrigation waters traveling downward into the bedrock along these joints and fractures.

Mass Movement

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

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2003) nor does the site lie within a County of Riverside fault zone (CRTLMA, 2021). No evidence of faulting projecting into or crossing the site was noted during our aerial photograph review or our review of published geologic maps.

As previously mentioned, the closest known active earthquake fault with a documented location is the Elsinore fault located approximately 12.6 kilometers (7.8 miles) to the southwest. In addition, other relatively close active faults include the San Jacinto fault located approximately 18.4 kilometers (11.4 miles) to the northeast, and the San Andreas fault located approximately 42.3 kilometers (26.3 miles) to the northeast.

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

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

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

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 closer distance and larger 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. (2021). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from January 1, 1932 through December 15, 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 associated with the San Jacinto fault to the northeast.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 miles) 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 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 time period for the events on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid to late1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the Elsinore fault zone to the southwest and the San Jacinto fault zone to the northeast 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 in the region around the subject site. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seismic-induced settlement, seiches and tsunamis, earthquake induced flooding, landsliding, and rockfalls.

Liquefaction: The site lies within an area mapped by the County of Riverside has having a very low potential for liquefaction (CRTLMA, 2021). The potential for liquefaction generally occurs during strong ground shaking within granular loose sediments where the groundwater is usually less than 50 feet below the ground surface. As found during this investigation, the site is underlain by relatively shallow igneous and metamorphic bedrock in the upper 50 feet, therefore, the possibility of liquefaction at the site is considered nil.

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

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

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

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

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

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 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 C very dense soil and soft rock.

CBC Earthquake Design Summary

Earthquake design criteria have been formulated in accordance with the 2019 CBC and ASCE 7-16 for the site based on the results of our investigation to determine the Site Class and an assumed Risk Category II. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region. In addition, the building official should confirm the Risk Category utilized in our design (Risk Category II). Our design values are provided below:

CBC 2019 SEISMIC DESIGN SUMMARY* Site Location (USGS WGS84) 33.73758, -117.22143, Risk Category II	
Site Class Definition Chapter 20 ASCE 7-16	C
S_s Mapped Spectral Response Acceleration at 0.2s Period	1.433
S_1 Mapped Spectral Response Acceleration at 1s Period	0.527
S_{MS} Adjusted Spectral Response Acceleration at 0.2s Period	1.719
S_{M1} Adjusted Spectral Response Acceleration at 1s Period	0.776
S_{DS} Design Spectral Response Acceleration at 0.2s Period	1.146
S_{D1} Design Spectral Response Acceleration at 1s Period	0.518
F_a Short Period Site Coefficient at 0.2s Period	1.2
F_v Long Period Site Coefficient at 1s Period	1.473
PGA_M	0.608
Seismic Design Category	D
*Values obtained from OSHPD Seismic Design Maps tool	

PERCOLATION TESTING AND TEST RESULTS

Four borehole percolation tests were conducted in general accordance with the Shallow Percolation Test procedure as outlined in the Design Handbook for Low Impact Development Best Management Practices (CRFCWCD, 2011). The requested locations of our test are illustrated on Enclosure A-2. Test borings were drilled to depths of approximately 7, 8, 9, and 10 feet, as requested, below the existing ground surface on December 6, 2021. Subsequent to drilling, a 3-inch diameter, perforated PVC pipe wrapped in filter fabric was placed within each test hole and 3/4-inch gravel was placed between the outside of the pipe and the hole wall. Test holes were pre-soaked the same day as drilling. Testing took place the next day, December 7, 2021, within 26 hours but not before 15 hours, of the pre-soak. The holes were filled with a variable height column of water using water from a 200 gallon water tank. Test periods consisted of allowing the water to drop in 30-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 12 readings were recorded.

Infiltration test results are summarized in the following table:

Test No.	Depth*	Infiltration Rate** (in/hr)
P-1	10	1.24
P-2	9	0.27
P-3	8	0.54
P-4	7	0.08

* depth measured below existing ground surface
** Porchet Method determined rate with an effective diameter due to loss in volume of water due to gravel packing.

The results of this testing are presented as Enclosures D-1 through D-4 in Appendix D. The test results indicate variable infiltration characteristics for the materials tested.

CONCLUSIONS

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

It should be noted that the subsurface conditions encountered in our exploratory borings are indicative of the locations explored and the subsurface conditions may vary. If conditions are encountered during the construction of the project that differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Rippability of Bedrock Units

The rippability of the bedrock units at the subject site was estimated based on the relative ease, or lack of, excavation during our boring exploration. The bedrock units which underlie the site are anticipated to be rippable by conventional earthmoving equipment down to the depths explored. Excavations deeper than this may require specialized methods, such as

D8R or larger dozer using a multi or single shank ripper. It is also anticipated that some larger non-rippable rock "floaters" may be encountered. These may require special handling. Excavations in these materials may require specialized methods.

If a more precise estimation of the rippability of the bedrock units is required, a seismic refraction investigation should be conducted at the site. Such a study should involve the measuring of the seismic velocities of the underlying bedrock units, as they increase with depth, then comparing these to estimates of velocities verses ease of excavation charts.

In summary, the most important consideration for the proposed grading should include selecting an experienced, well-qualified contractor. The success to excavating the bedrock materials at the site will require the contractor to have knowledge of the appropriate ripper-equipment selection (i.e., down pressure available at the tip, tractor flywheel horsepower, tractor gross weight, etc.), ripping techniques (i.e., single- or multi-shank teeth, pass spacing, tandem pushing, etc.). It should also be noted that while in some areas where deeper cuts may be possible with standardized earthmoving equipment, specialized methods may increase the speed of the excavations at the site.

Foundation Support

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. The construction of this compacted fill mat will allow for the removal of the existing fill material which was loose and any current subsurface improvements, such as utilities, foundations, etc., that may be present locally.

Conventional foundation systems utilizing either individual spread footings and/or continuous wall footings will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat.

Soil Expansiveness

Our expansion index testing of a representative sample of the on-site soils indicates a low expansion potential. For low expansive soils, specialized foundation design and construction procedures to resist expansive soil activity are necessary and provided in the following sections of this report.

Careful evaluation of onsite 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.

Infiltration

The results of our field investigation and percolation test data indicates the site soils at the depths tested are not conducive to infiltration. Based on the results of this investigation, infiltration is also not anticipated to occur at other depths due to the amount of silty/clayey fines and dense to very dense nature of the soils and hard to very hard nature of the bedrock.

Geologic Mitigations

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

Seismicity

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

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

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

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

RECOMMENDATIONS

Geologic Recommendations

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

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An onsite, pre-job meeting with the developer, the contractor, the jurisdictional agency, and the geotechnical engineer should occur prior to all grading related operations.

Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

Any undocumented fill encountered during grading should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. It is our recommendation that any existing fills under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

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

The existing fill/topsoil material, as well as any loose older alluvial soils and any loose bedrock, if encountered, 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 3 feet deep will be required from proposed development areas in order to encounter competent older alluvium or competent bedrock upon which engineered compacted fill can be placed. The given removal depths are preliminary. Deeper fills may be present, primarily in areas of past and current improvements. Removals should expose older alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557) or relatively unweathered, hard, bedrock. The actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

Preparation of Fill Areas

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

Engineered Compacted Fill

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

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

Preparation of Foundation Areas

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

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

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations 5 feet deep and greater should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements. Short-term excavations of 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based upon the findings from our exploratory borings, it appears that Type C soils are the predominant type of soil on the project and all short-term excavations should be based on this type of soil.

Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

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

Slope Construction

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

Slope Protection

Since the site soil materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after 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.

Soil Expansiveness

The upper materials encountered during this investigation were tested and found to have a low expansion potential. Therefore, specialized foundation design and construction procedures to specifically resist expansive soil activity are anticipated at this time and are provided within.

Additional evaluation of on-site and any imported soils for their expansion potential should be conducted following completion of the grading operation.

Foundation Design

Due to low expansive soil conditions, we recommend that all structures be supported on reinforced, stiffened mat foundations resting over 24 inches of engineered compacted fill placed over competent native earth materials.

The design of the structural slab foundation should be performed in conformance to the Wire Reinforcement Institute (WRI) method or the Post-Tensioning Institute (PTI) method. For the application of the WRI method, a minimum effective plasticity index of 27 is recommended for foundation design. The slab thickness should be a minimum of 5 inches and should have a reinforcement of at least Asfy equal to 3,300 pounds. This could consist of #3 reinforcing bars of 60-grade steel placed at a maximum spacing of 18 inches on center, each way or equivalent. Prior to placing concrete slabs, the upper 12 inches of the subgrade soil should be pre-saturated to 2 to 4 percent over optimum moisture content.

These reinforcement, depth, and spacing recommendations should be considered minimum. The actual requirements for slab-on-grade foundations design and construction should be provided by a structural engineer experienced in these matters.

These conditions should be verified during the site grading by additional evaluation of on-site and any imported soils for their expansion potential and plasticity characteristics.

If slab-on-grade foundations per the PTI method are proposed, the following geotechnical parameters should be used for design:

• Edge Moisture Variation Distance, em:	
Center Lift Loading Conditions:	9.0 ft
Edge Lift Loading Conditions:	7.8 ft
• Differential Swell, ym:	
Center Lift	4.0 in
Edge Lift	8.0 in
• Subgrade Soil Friction Coefficient, μ:	0.30

The above design parameters are based upon the data collected during our site investigation and are in accordance with Design of Post-Tensioned Slabs-on-Ground, third edition, published by the Post-Tensioning Institute (2008).

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure may be increased by 200 psf for each additional foot of width, and by 500 psf for each additional foot of depth, up to a maximum of 4,000 psf.

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 over turning should not exceed the increased allowable pressure. Buildings should be setback from slopes in accordance with the California Building Code.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 230 pounds per square foot per foot of depth. Base friction may be computed at 0.23 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Settlement

Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Maximum settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlements between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Building Area Slab-on-Grade

To provide adequate support, concrete floor slabs-on-grade should bear on a minimum of 24 inches of engineered fill compacted soil placed and maintained at 2 to 4 percent above optimum moisture content. The final pad surfaces should be rolled to provide smooth, dense surfaces. Concrete slabs-on-grade should be a minimum of 5 inches in thickness with No. 3 bars spaced 12 inches on center each way.

The actual requirements for slab-on-grade design and construction details should be provided by a structural engineer experienced in these matters. These conditions should be verified during the site grading by additional evaluation of on-site and any imported soils for their expansion potential and plasticity characteristics.

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

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

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork

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

To resist expansive soil forces, flatwork supported by low expansive soils should be reinforced with a minimum of # 3 rebar at 18 inches each way. Flatwork areas should be pre-saturated to 2 to 4 percent over optimum moisture content to a minimum depth of 12 inches prior to placing concrete.

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

Wall Pressures

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

For design of retaining walls unrestrained against movement at the top, we recommend an active pressure of 56 pounds per square foot (psf) per foot of depth be used.

This assumes level backfill consisting of compacted, non-expansive, 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. Non-expansive import soils may be required. 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 0.53 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

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

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

Preliminary Pavement Design

Testing and design for preliminary onsite pavement was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing, and upon an assumed Traffic Index generally used for similar projects, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject on-site pavement improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
On site vehicular parking with occasional truck traffic (ADTT=10)	6.0	10	0.25’ AC / 1.05' AB or 5" JPCP / 6" AB
Light to moderate truck traffic (ADTT=25)	7.0	10	0.30'AC / 1.25'AB or 6" JPCP / 6" AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base JPCP - Jointed Plain Concrete Pavement with MR ≥ 600 psi			

The above structural sections are predicated upon 90 percent relative compaction (ASTM D 1557) of all utility trench backfills and 95 percent relative compaction (ASTM D 1557) of the upper 12 inches of pavement subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 Aggregate Base.

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 5 inch thick concrete, with a 6 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 concrete pavement sections should have a minimum modulus of rupture (MR) of 600 pounds per square inch (psi). Transverse joints should be sawcut in the pavement at approximately 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other.

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

Infiltration

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

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by the project geotechnical consultant prior to construction to confirm that the intent of the recommendations presented in this report have been incorporated into the design.

Additional R-value, expansion, and soluble sulfate content testing may be needed after/during site rough grading.

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

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavations prior to the processing and preparation of the bottom areas for fill placement.

3. Scarifying and recompacting prior to fill placement.

4. 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

5. Foundation excavations.

6. Subgrade preparation for pavements and slabs-on-grade. This includes pre-saturation testing of slab-on-grade and flatwork areas to verify moisture content.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Compass Danbe Real Estate Partners, LLC and their design consultants for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

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

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

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

TIME LIMITATIONS

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

CLOSURE

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

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

Robert M. Markoff, CEG
Engineering Geologist

John P. Leuer, GE 2030
President

REGISTERED PROFESSIONAL ENGINEER
JOHN P. LEUER
NO. 2030
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ROBERT M. MARKOFF
No. 2073
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

AAT:RMM:JPL:ss

Distribution:	Addressee (2) and PDF via email mbachli@danbe.com
cc:	Vicky Valenzuela via email vicky@cdrepartners.com

REFERENCES

American Society of Civil Engineers, 2016, Minimum Design Load for Buildings and Other Structures, ASCE 7-16.

California Building Standards Commission and International Conference of Building Officials, 2019, California Building Code, 2019 Edition.

California Department of Water Resources, 2021, Online Water Data Library (WDL), https://wdl.water.ca.gov/waterdatalibrary/Map.aspx, accessed August 2021.

County of Riverside, Flood Control and Water Conservation District (CRFCWCD), 2011, Design Handbook for Low Impact Development Best Management Practices, dated September 2011.

County of Riverside, Transportation and Land Management Agency (CRTLMA), 2021, Geographic Information System, http://www3.tlma.co.riverside.ca.us, accessed August 2021.

Google Earth, 2021, Imagery from various years, www.google.com/earth.

Hart, E.W. and W.A. Bryant, 2010, Fault-Rupture Hazard Zones in California, California Dept. of Conservation Division of Mines and Geology Special Publication 42.

Historic Aerials (Nationwide Environmental Title Research, LLC), 2021, Imagery from Various Years, https://www.historicaerials.com/, accessed August 2021.

Larson, R., and Slosson, J., 1992, The Role of Seismic Hazard Evaluation in Engineering Reports, in Engineering Geology Practice in Southern California, AEG Special Publication Number 4, pp 191-194.

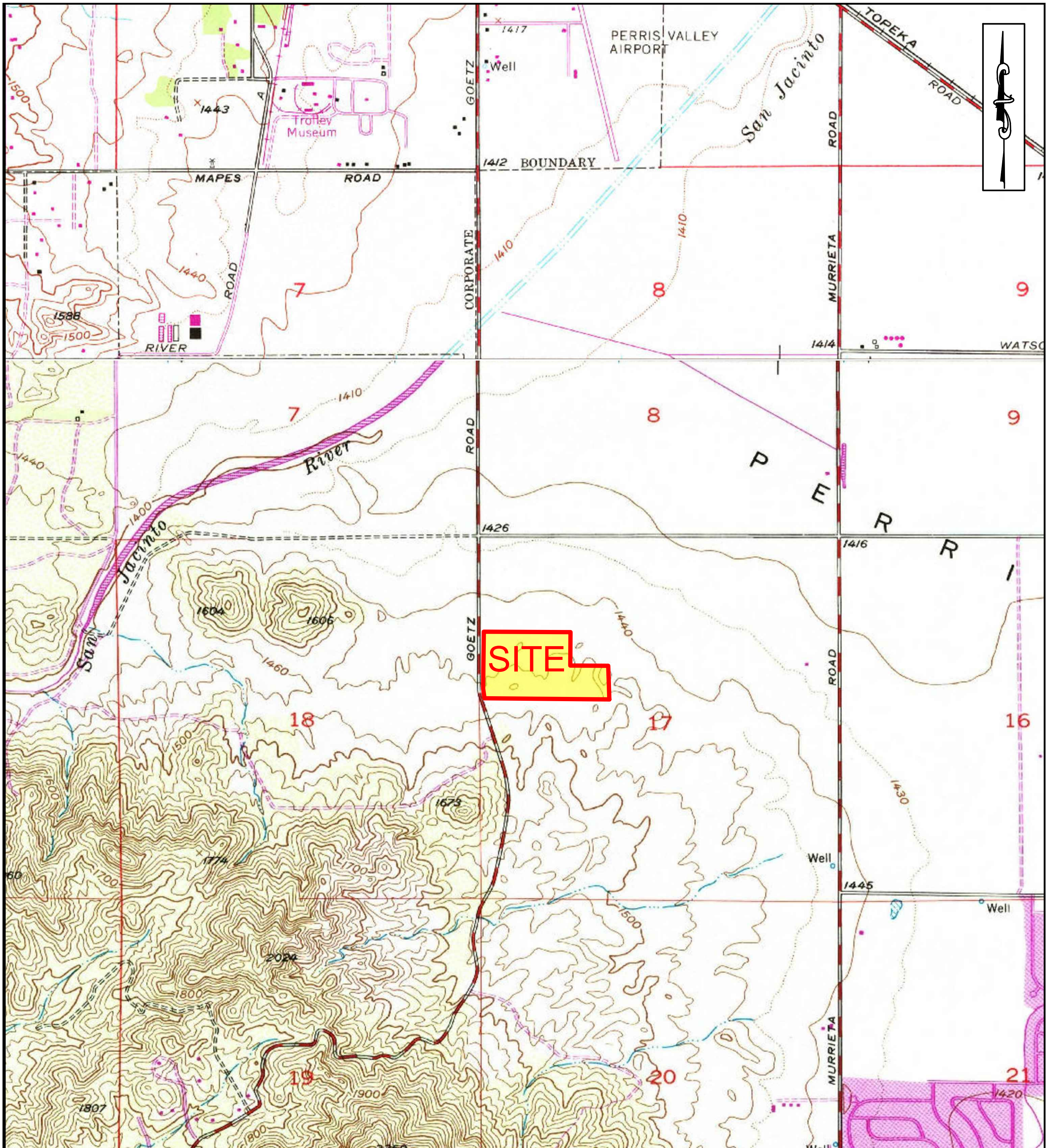
Morton, D.M., 2003, Preliminary Geologic Map of the Romoland 7.5' Quadrangle, Riverside County, California, U.S.G.S. Open File Report 03-102.

USGS, 2021, https://earthquake.usgs.gov/earthquakes/map/.

Watermaster Support Services, Western Municipal Water District, and San Bernardino Valley Water Conservation District, 2021, Cooperative Well Measuring Program, Spring 2021, Covering the Upper Santa Ana River Watershed, San Jacinto Watershed, and Santa Margarita Watershed, July 1, 2021.

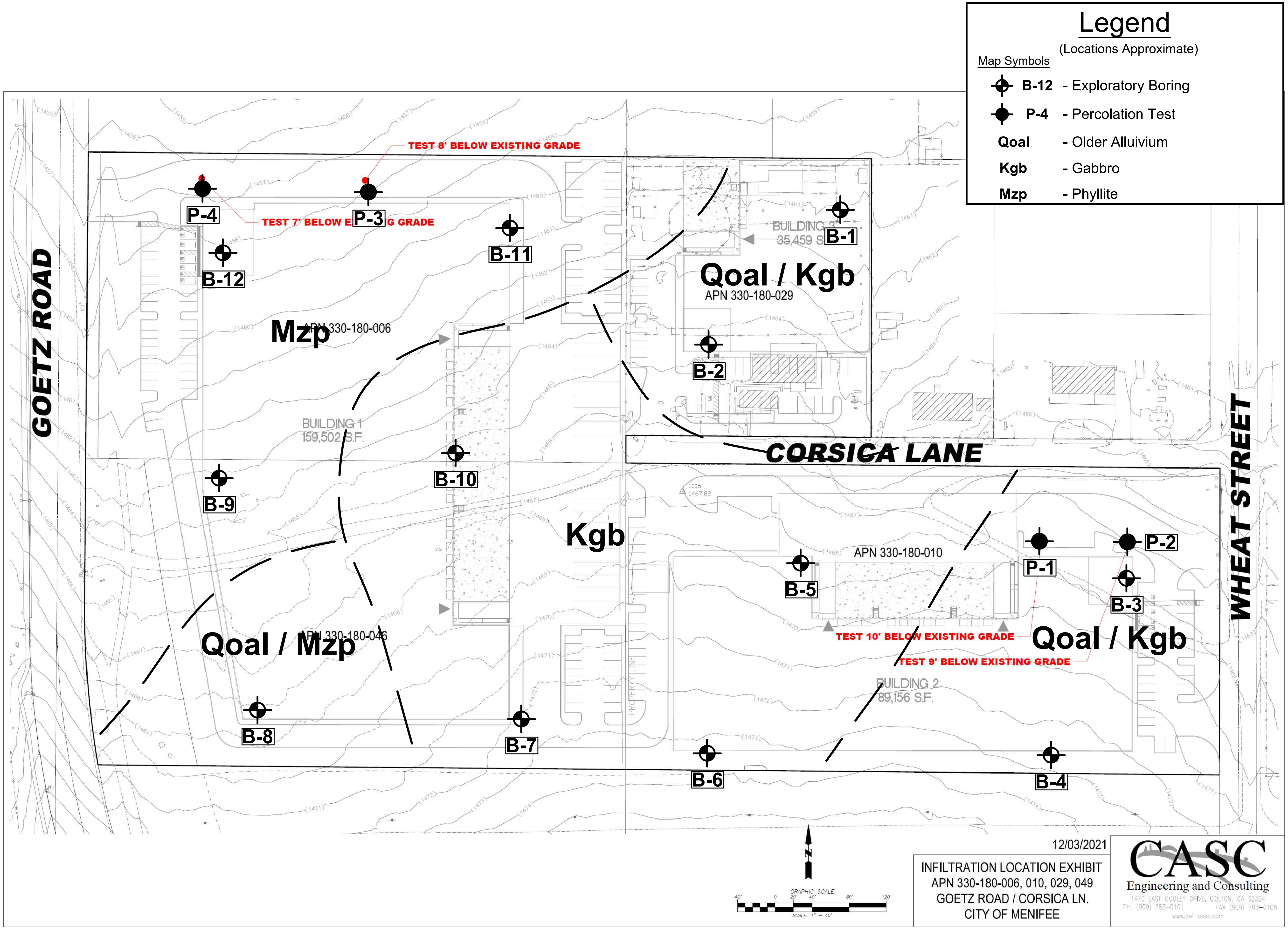
APPENDIX A

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



INDEX MAP

PROJECT:	APNs 330-180-006, –010, –029, and –046, Menifee, California	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC.	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 2021
		SCALE:	1" ≈ 2,000'



SITE PLAN

PROJECT NO.: 23758.1

ENCLOSURE: A-2

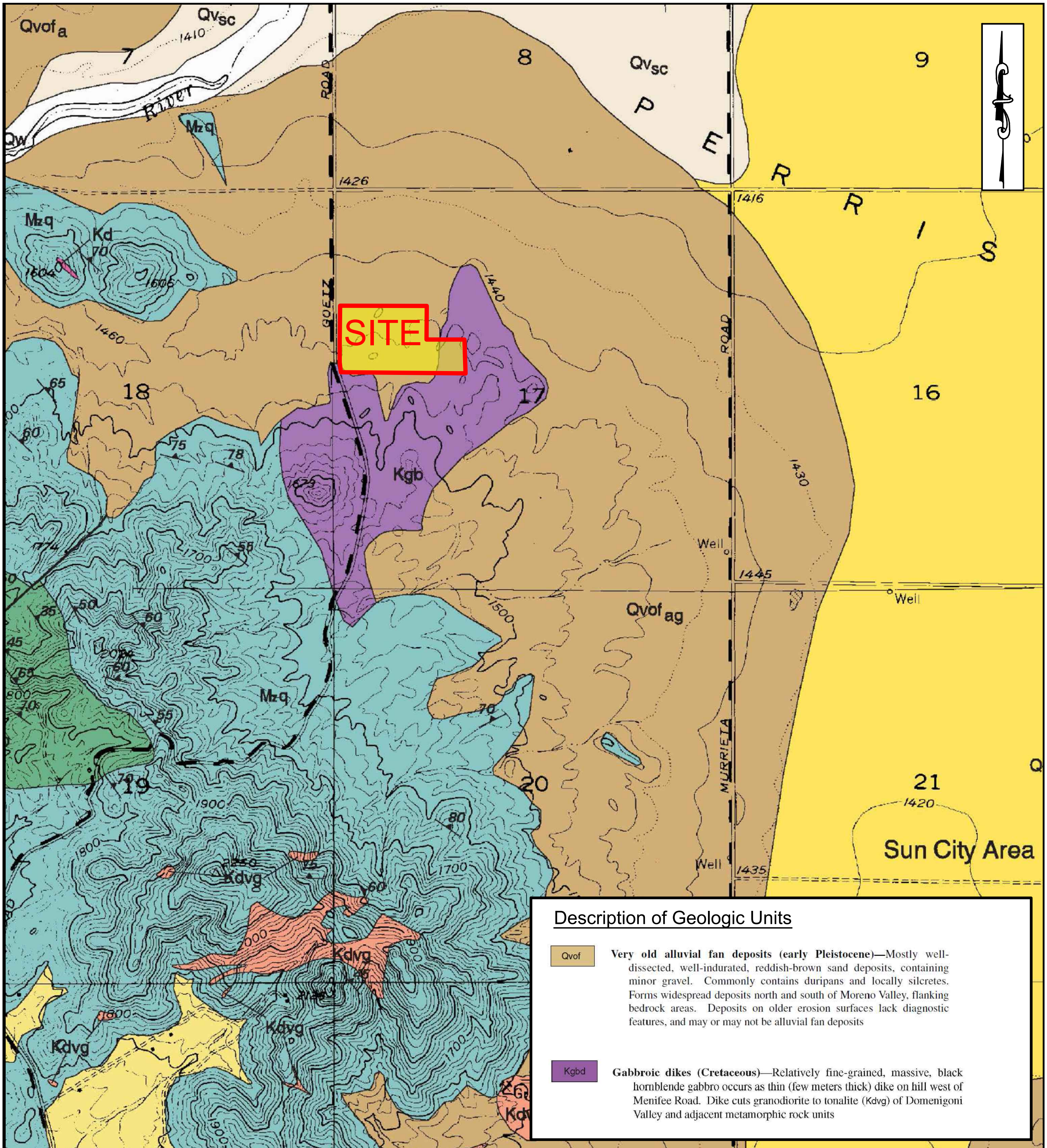
DATE: December 2021

SCALE: 1" = 100'

PROJECT: APNs 330-180-006, -010, -029, and -046, Menifee, California

CLIENT: Compass Danbe Real Estate Partners, LLC.

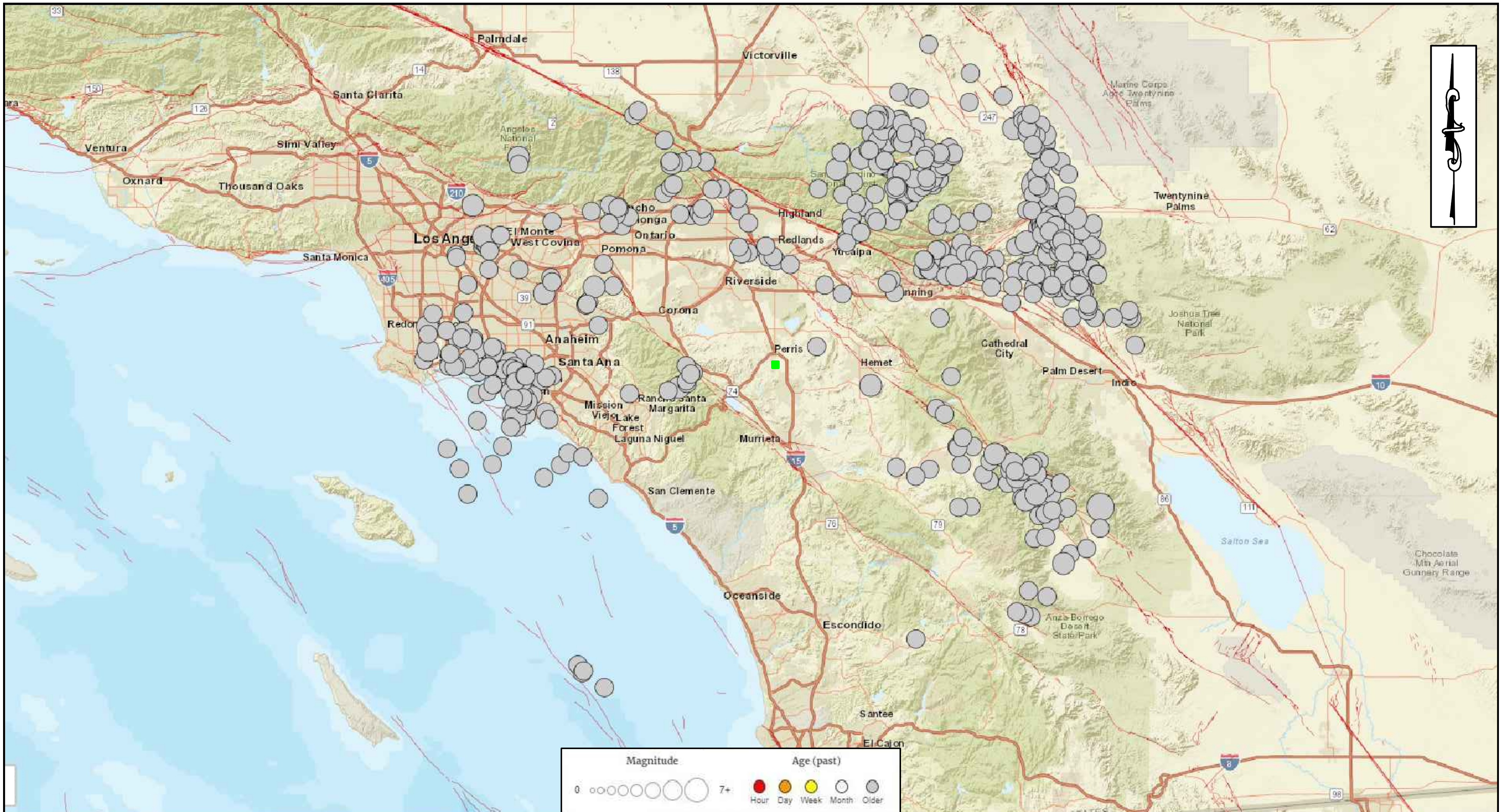
LOR GEOTECHNICAL GROUP, INC.



REGIONAL GEOLOGIC MAP

(Morton 2003)

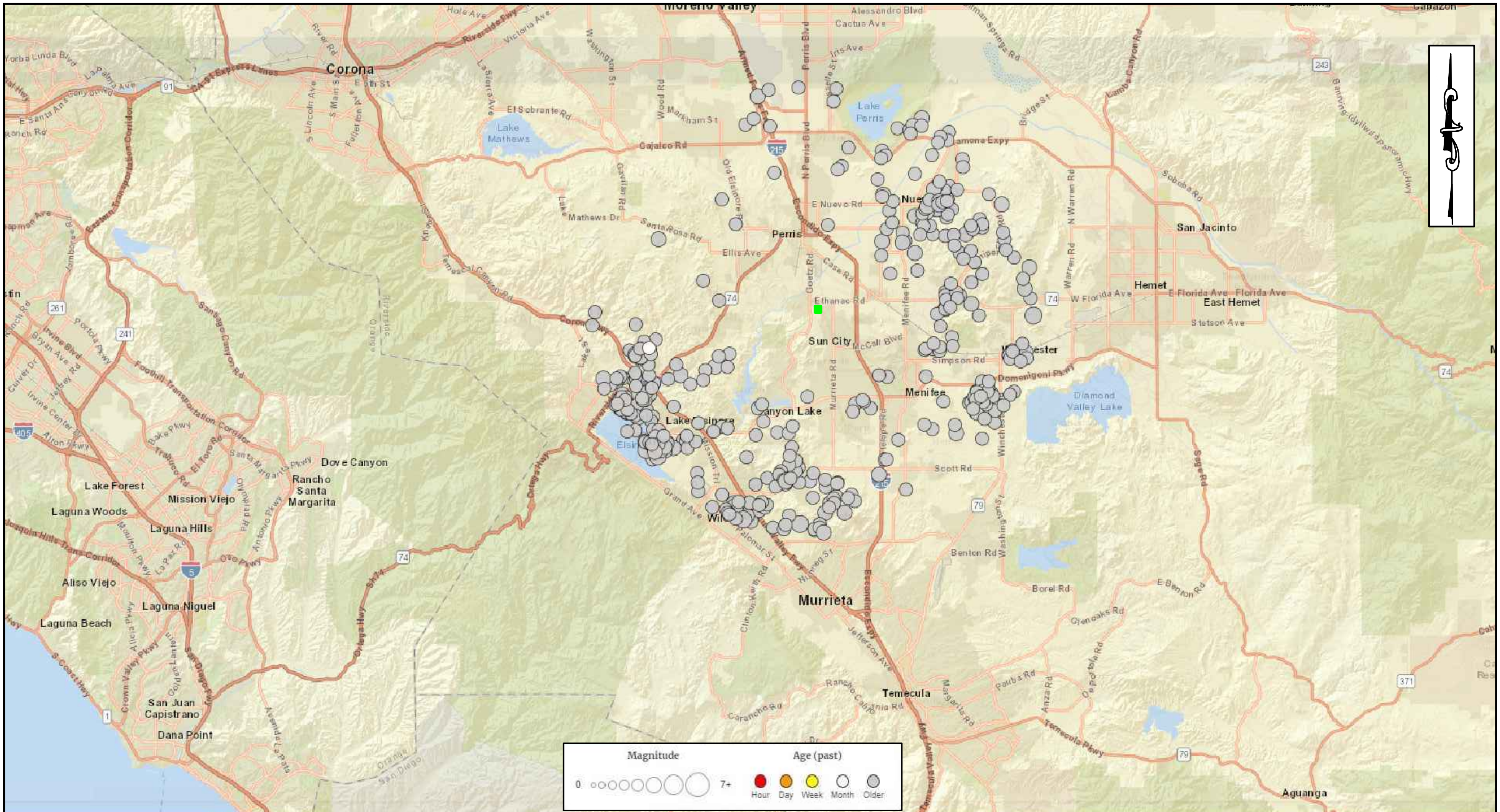
PROJECT:	APNs 330-180-006, –010, –029, and –046, Menifee, California	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC.	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 2021
		SCALE:	1" ≈ 4,000'



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 531 epicenters of instrument-recorded events from 01/01/32 to present (12/15/21) of local magnitude greater than M4.0 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 events are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 100km Radius

PROJECT:	APNs 330-180-006, –010, –029, and –046, Menifee, California	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC.	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 2021
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 439 epicenters of instrument-recorded events from 01/01/78 to present (12/15/21) of local magnitude greater than M1.0 within a radius of ~9.2 miles (15 kilometers) of the site. Location accuracy varies. The site is indicated by the green square. The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These events are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 15km Radius

PROJECT:	APNs 330-180-006, –010, –029, and –046, Menifee, California	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC.	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 2021
		SCALE:	1" ≈ 10km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling 12 exploratory borings to depths between approximately 15.25 to 30.42 feet below the existing ground surface using a Mobile B-61 drill rig on December 6 and7, 2021. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

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

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

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

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-12. A Boring Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented as Enclosure B-ii.

CONSISTENCY OF SOIL

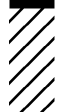
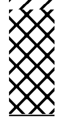
SANDS

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

COHESIVE SOILS

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

SAMPLE KEY

Symbol	Description
	INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE
	INDICATES BULK SAMPLE
	INDICATES SAND CONE OR NUCLEAR DENSITY TEST
	INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

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

BORING LOG LEGEND

PROJECT:	Proposed Industrial Developement	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 2021

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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:	Proposed Industrial Developement	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	December 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		1, 3, 4, 7, 8, 9, 10						SP	@ 0 feet, FILL: **POORLY GRADED SAND (arena sand).**
								CL	**@ 0.16 feet,** OLDER ALLUVIUM: **LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 20% fine grained sand, 65% clayey fines of low plasticity, red-brown, moist.**
	24		20.6		107.2				
5	56		8.5		112.9			SM	**@ 5 feet, SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 45% fine grained sand, 20% silty fines, yellow-brown, damp.**
									@ 7 feet, IGNEOUS BEDROCK: **GABBRO, moderately weathered, fine grained, dry.**
10	46 for 6"		1.5		122.9				
15	46 for 6"								**@ 15 feet, no recovery, somewhat difficult drilling.**
20	51 for 4"								**@ 20 feet, no recovery.**
25									

END OF BORNG @ 20.25'

Fill to 0.16'
No groundwater
Bedrock @ 7'

PROJECT: Proposed Industrial Development		**PROJECT NO.:**	23758.1
CLIENT: Compass Danbe Real Estate Partners, LLC		**ELEVATION:**	1461
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	December 6, 2021
		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								CL	@ 0 feet, OLDER ALLUVIUM: **LEAN CLAY with SAND, trace gravel to 1/2", approximately 5% coarse grained sand, 10% meidum grained sand, 25% fine grained sand, 60% clayey fines of low plasticty, red-brown, moist.**
	17		19.9		107.0				
5	61 for 11"		15.0		115.4				@ 5 feet, IGNEOUS BEDROCK: **GABBRO, highly weathered, coarse grained, damp.**
	61 for 11"		10.0		118.7				
10	115 for 9"		5.9						**@ 10 feet, slightly less weathered.**
15	111 for 8"		11.2						
									END OF BORING @ 15.67' **No fill** **No groundwater** **Bedrock @ 5'**
20									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1464
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	December 6, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-2

LOG OF BORING B-3

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% gravel to 3/4", 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 45% silty fines, red-brown, dry, loose.**
	20		3.3		105.7				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND with GRAVEL, approximately 15% gravel to 3", 10% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 25% silty fines, red-brown, dry.**
5	21		9.1		91.3			CL	@ 5 feet, **LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% clayeye fines of low plasticity, red-brown, damp.**
									@ 8 feet, IGNEOUS BEDROCK: **slightly weathered, coarse grained.**
10	63		10.7		109.0				
15	82		6.2						@ 15 feet, slightly weathered, difficult to drill.
20	134 for 9"		8.7						@ 20 feet, becomes fine grained.
25	73 for 5"		3.3						@ 25 feet, medium to coarse grained.
30	77 for 5"		5.3						**END OF BORING @ 30.42'** **Fill to 2'** **No groundwater** **Bedrock @ 8'**
35									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1468
		DATE DRILLED:	December 6, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-3

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-4

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SC	@ 0 feet, FILL/TOPSOIL: **CLAYEY SAND, approximately 5% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 30% clayey fines of low plasticity, red-brown, dry, loose.**
	10		4.7		98.2			SM	@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 60% fine grained sand, 35% silty fines with trace clay, red-brown, dry, some pinhole porosity.**
5	32		7.1		98.4				**@ 5 feet, trace cobbles, no visible porosity, damp.**
	33		9.5		108.1				
									@ 7.5 feet, IGNEOUS BEDROCK: **GABBRO, highly weathered, coarse grained, damp.**
10	84 for 11"		7.8		112.5				
15	78 for 9"		5.0						
20									

END OF BORING @ 15.75'

Fill to 2'
No groundwater
Bedrock @ 7.5'

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1473
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	December 6, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-4

LOG OF BORING B-5

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10						SC	@ 0 feet, FILL/TOPSOIL: **CLAYEY SAND, approximately 5% gravel to 3/4", 5% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 40% clayey fines of low plasticity, red-brown, dry, loose.**
	46		12.2		122.9				@ 2 feet, IGNEOUS BEDROCK: **GABBRO, severly weathered, coarse grained, red-brown, damp.**
5	57		9.0		138.4				**@ 5 feet, becomes moderately weathered.**
10	70 for 9"		3.9						
15	46 for 3"		7.9						**END OF BORING @ 15.25'** **Fill to 2'** **No groundwater** **Bedrock @ 2'**
20									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1468
		DATE DRILLED:	December 6, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-5

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-6

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								CL	@ 0 feet, FILL/TOPSOIL: **LEAN CLAY with SAND, trace gravel to 1/2", approximately 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% clayey fines of low plasticity, red-brown, dry, loose.**
	41		7.8		130.0				@ 2 feet, IGNEOUS BEDROCK: **GABBRO, severly weathered, coarse grained, damp.**
5	70 for 11"		15.4						@ 5 feet, becomes moderately weathered, rings disturbed.
	93 for 10"		7.9						
10	65 for 6"		4.8						@ 10 feet, much less weathered.
15	65 for 6"		3.6						
									END OF BORING @ 15.5' Fill to 2' No groundwater Bedrock @ 2'
20									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1473
		DATE DRILLED:	December 6, 2021
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-6

LOG OF BORING B-7

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	U.S.C.S.	DESCRIPTION
0		3, 7, 9, 10				SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 30% silty fines with clay, red-brown, dry, loose.**
	58		9.5		121.1		@ 2 feet, IGNEOUS BEDROCK: **GABBRO, severly weathered, coarse grained, red-brown, damp.**
5	60		5.4		130.1		
10	100		4.8				@ 10 feet, becomes moderately weathered.
15	108 for 11"		6.5				@ 15 feet, becomes slightly weathered, fine grained.
20	117 for 11"		3.5				
25							

END OF BORING @ 20.92'

Fill to 2'
No groundwater
Bedrock @ 2'

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1472
		DATE DRILLED:	December 7, 2021
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-7

LOG OF BORING B-8

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								CL	@ 0 feet, FILL/TOPSOIL: **LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% clayey fines of low plasticity, red-brown, dry, loose.**
	26		10.5		115.5				@ 2 feet, OLDER ALLUVIUM: **LEAN CLAY with SAND, approximately 5% coarse grained sand, 15% medium grained sand, 25% fine grained sand, 55% clayey fines of low plasticity, red-brown, damp, some pinhole porosity.**
									@ 4 feet, METAMORPHIC BEDROCK: **PHYLLITE, highly weathered, fine to medium grained, red-brown, damp.**
5	70		10.9		114.4				
10	95		4.8						@ 10 feet, slightly to moderately weathered.
15	107 for 8"		2.7						
									@ 17 feet, difficult to drill.
									END OF BORING @ 18' due to refusal **Fill to 2'** **No groundwater** **Bedrock @ 4'**
20									
25									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1468
		DATE DRILLED:	December 7, 2021
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-8

LOG OF BORING B-9

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10, 11						CL	@ 0 feet, <u>FILL/TOPSOIL:</u> **LEAN CLAY with SAND, trace gravel to 1/2", trace coarse grained sand, 10% medium grained sand, 30% fine grained sand, 60% clayey fines of low plasticity, red-brown, loose.**
	35		13.3		106.1				@ 2 feet, <u>METAMORPHIC BEDROCK:</u> **PHYLLITE, severely weathered, fine to medium grained with clay, red-brown, damp.**
5	64		10.8		120.7				@ 5 feet, less weathered, fine grained, gray.
10	78		8.6						
15	96		5.9						
20	73 for 6"		5.9						**END OF BORING @ 20.5'** **Fill to 2'** **No groundwater** **Bedrock @ 2'**
25									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1463
		DATE DRILLED:	December 7, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-9

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-10

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								CL	@ 0 feet, FILL/TOPSOIL: **LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 25% fine grained sand, 60% clayey fines of low plasticity, red-brown, dry, loose.**
	36		15.6		108.6				@ 2 feet, IGNEOUS BEDROCK: **GABBRO, severly weathered, contains clay, red-brown, damp.**
5	54		9.9		117.0				**@ 5 feet, becomes moderately to highly weathered.**
10	116 for 10"		12.6						
15	65 for 5"		6.8						**@ 15 feet, less weathered.**
20	115 for 9"		12.1						**END OF BORING @ 20.75'** **Fill to 2'** **No groundwater** **Bedrock @ 2'**
25									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1465
		DATE DRILLED:	December 7, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-10

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-11

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								CL	**@ 0 feet,** FILL/TOPSOIL: **LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 25% fine grained sand, 60% clayey fines of low plasticity, red-brown, dry, loose.**
	42		8.0		121.8				**@ 2 feet,** METAMORPHIC BEDROCK: **PHYLLITE, highly weathered, fine grained, gray.**
5	50		9.9		114.2				**@ 5 feet, remains highly weathered.**
10	117 for 11"		4.8						
15	107		5.3						**@ 15 feet, less weathered.**
20	115 for 11"		7.5						**END OF BORING @ 20.92'** **Fill to 2'** **No groundwater** **Bedrock @ 2'**
25									

PROJECT: Proposed Industrial Development	**PROJECT NO.:** 23758.1
CLIENT: Compass Danbe Real Estate Partners, LLC	**ELEVATION:** 1460
	DATE DRILLED: December 7, 2021
LOR GEOTECHNICAL GROUP, INC.	**EQUIPMENT:** Mobile B-61
	HOLE DIA.: 8" **ENCLOSURE:** B-11

LOG OF BORING B-12

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10, 11						SC	@ 0 feet, FILL/TOPSOIL: **CLAYEY SAND, approximately 10% gravel to 3/4", 5% coarse grained sand, 10% medium grained sand, 35% fine grained sand, 40% clayey fines of low plasticity, red-brown, dry, loose.**
	61 for 8"		10.2		104.6				@ 2 feet, METAMORPHIC BEDROCK: **PHYLLITE, highly weathered, fine grained, tan, dramp.**
5	66 for 8"		3.1		119.2				**@ 5 feet, less weathered, gray, dry.**
10	65 for 5"		2.8						**@ 10 feet, much less weathered, hard, somewhat difficult to drill.**
15	65 for 2"								**@ 15 feet, no recovery.**
									END OF BORING @ 16' due to refusal **Fill to 2'** **No groundwater** **Bedrock @ 2'**
20									

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**ELEVATION:**	1458
		DATE DRILLED:	December 7, 2021
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-12

LOR GEOTECHNICAL GROUP, INC.

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction characteristics, direct shear, sieve analysis, sand equivalent, R-value, expansion index, Atterberg limits, 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 2921 and ASTM D 2216, respectively, and the results are shown on the boring logs, Enclosures B-1 through B-12 for convenient correlation with the soil profile.

Laboratory Compaction

A selected soil sample was tested in the laboratory to determine compaction characteristics using the ASTM D 1557 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-1	0-3	(CL) Lean Clay with Sand	127.5	10.0
B-7	0-3	(SM) Silty Sand	126.5	11.5

Direct Shear Test

Shear tests are performed in general accordance with ASTM D 3080 with a direct shear machine at a constant rate-of-strain (0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in remolded condition (90 percent relative compaction per ASTM D 1557) and soaked, to represent the worse case conditions expected in the field.

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

DIRECT SHEAR TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Apparent Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-1	0-3	(CL) Lean Clay with Sand	400	23
B-7	0-3	(SM) Silty Sand	500	31

Sieve Analysis

A quantitative determination of the grain size distribution was performed for selected samples in accordance with the ASTM D 422 laboratory test procedure. The determination is performed by passing the soil through a series of sieves, and recording the weights of retained particles on each screen. The results of the grain size distribution analyses are presented graphically on Enclosures C-1 and C-2.

Sand Equivalent

The sand equivalent of selected soils were evaluated using the California Sand Equivalent Test Method, Caltrans Number 217. The results of the sand equivalent tests are presented with the grain size distribution analyses on Enclosures C-1 and C-2.

R-Value Test

A 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 results of the R-value test is presented on Enclosure C-1.

Expansion Index Test

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

EXPANSION INDEX TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-3	(CL) Lean Clay with Sand	44	Low
Expansion Index:	0-20 Very low	21-50 Low	51-90 Medium	91-130 High

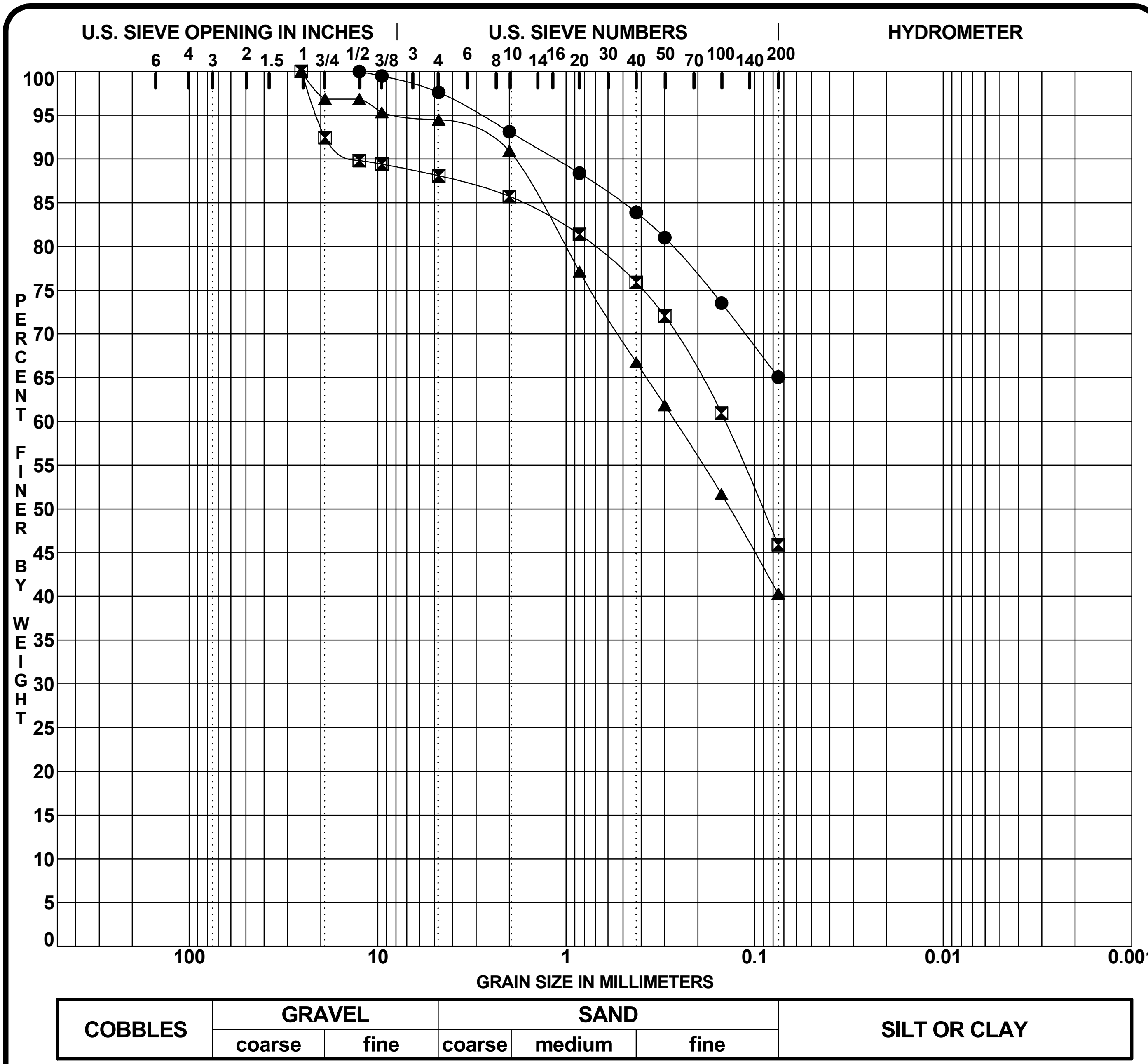
Atterberg Limits

Selected samples of the fine-grained soil units encountered at the site are tested for their Atterberg limits in accordance with ASTM D 4318. The results of these tests are presented on Enclosure C-3.

Soluble Sulfate Content Test

The soluble sulfate content of a selected subgrade soil was evaluated. The concentration of soluble sulfates in the soil 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 sample. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test result is presented in the following table:

SOLUBLE SULFATE CONTENT TEST			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Sulfate Content (% by weight)**
B-3	0-3	(SM) Silty Sand	< 0.005
B-9	0-3	(CL) Lean Clay with Sand	< 0.005
B-12	0-3	(SC) Clayey Sand	< 0.005



	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-01*	*@ 0-3'*	*(CL) Lean Clay with Sand*	*6*	*13*	*10*	*27*		
⊠	*B-03*	*@ 0-3'*	*(SM) Silty Sand*	*11*	*--*				
▲	*B-05*	*@ 0-3'*	*(SC) Clayey Sand*	*13*	*--*				

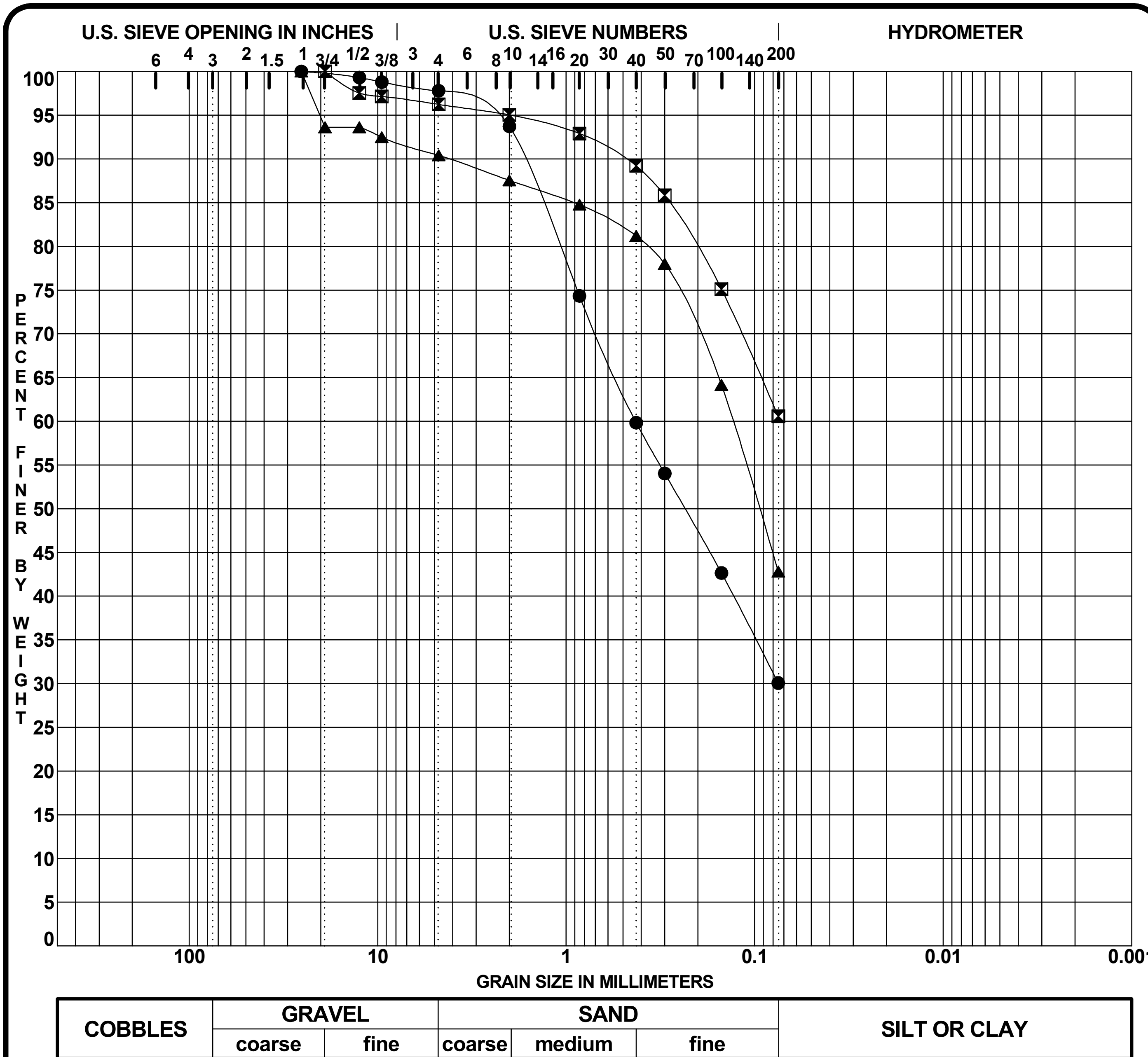
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-01*	*@ 0-3'*	*12.50*				*2.4*	*32.6*	*65.1*	
⊠	*B-03*	*@ 0-3'*	*25.40*	*0.14*			*11.9*	*42.2*	*45.9*	
▲	*B-05*	*@ 0-3'*	*25.40*	*0.26*			*5.5*	*54.2*	*40.3*	

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**DATE:**	December 2021

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-1



Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-07 @ 0-3'*	*(SM) Silty Sand*	*19*	*--*				
⊠	*B-09 @ 0-3'*	*(CL) Lean Clay with Sand*	*7*	*--*				
▲	*B-12 @ 0-3'*	*(SC) Clayey Sand*	*12*	*--*				

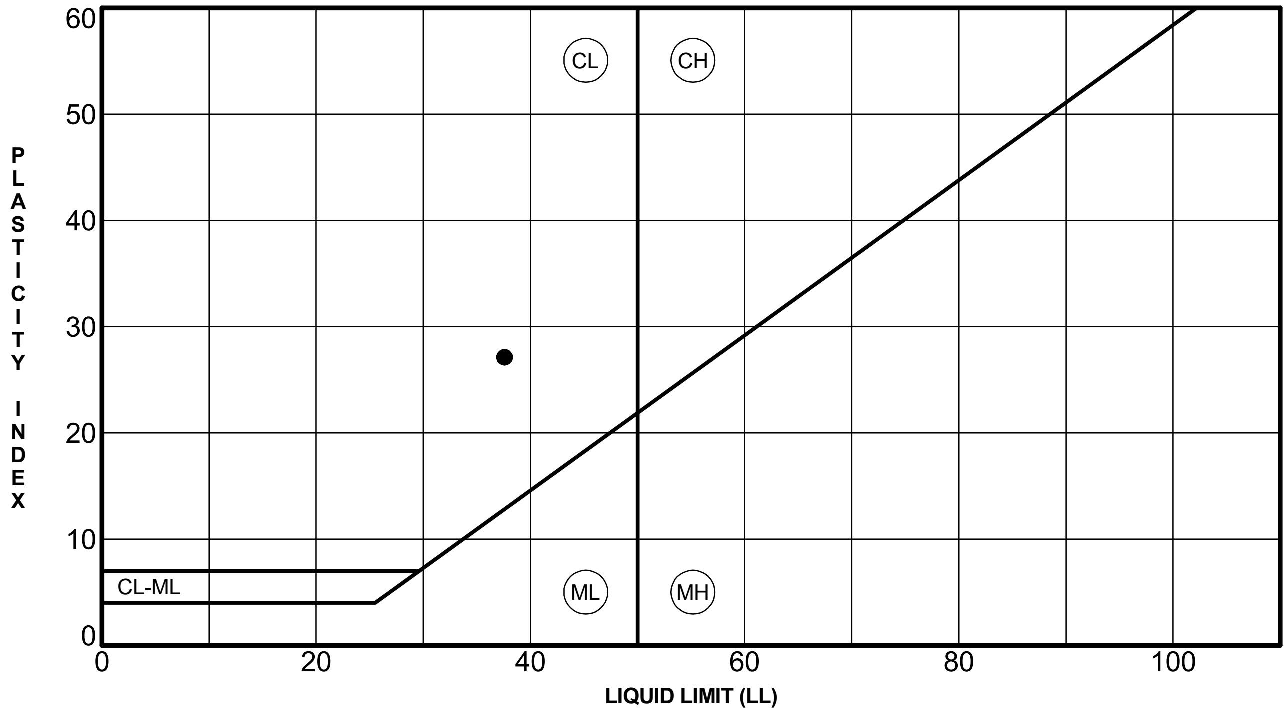
Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-07 @ 0-3'*	*25.40*	*0.43*			*2.2*	*67.7*	*30.1*	
⊠	*B-09 @ 0-3'*	*19.00*				*3.8*	*35.7*	*60.6*	
▲	*B-12 @ 0-3'*	*25.40*	*0.13*			*9.6*	*47.6*	*42.8*	

PROJECT: Proposed Industrial Development
PROJECT NO.: 23758.1
CLIENT: Compass Danbe Real Estate Partners, LLC
DATE: December 2021

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-2



	Specimen Identification	LL	PL	PI	Fines	Classification
●	B-01 @ 0-3'	38	10	27	65.1	(CL) Lean Clay with Sand

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23758.1
CLIENT:	Compass Danbe Real Estate Partners, LLC	**DATE:**	December 2021

ATTERBERG LIMITS RESULTS

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-3

APPENDIX D

Infiltration Test Results

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN's 330-180-006, -10, 029, and -046, Menifee	Test Date:	December 7, 2021
Project No.:	23758.1	Test Hole No.:	P-1
Soil Classificaiton:	Bedrock	Effective Hole Dia*:	4.8 in.
Depth of Test Hole:	10.0 ft.	Date Excavated:	December 6, 2021
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	8:17 AM	8:47 AM	30	0.50	0.50	50.00	91.00	120.00	120.00	41.00	49.50	0.7
2	8:47 AM	9:17 AM	30	0.50	1.00	48.00	84.00	120.00	120.00	36.00	54.00	0.8
3	9:17 AM	9:47 AM	30	0.50	1.50	48.00	82.50	120.00	120.00	34.50	54.75	0.9
4	9:47 AM	10:17 AM	30	0.50	2.00	49.00	81.00	120.00	120.00	32.00	55.00	0.9
5	10:17 AM	10:47 AM	30	0.50	2.50	48.00	79.00	120.00	120.00	31.00	56.50	1.0
6	10:47 AM	11:17 AM	30	0.50	3.00	48.00	78.50	120.00	120.00	30.50	56.75	1.0
7	11:17 AM	11:47 AM	30	0.50	3.50	48.00	78.00	120.00	120.00	30.00	57.00	1.0
8	11:47 AM	12:17 PM	30	0.50	4.00	48.00	78.00	120.00	120.00	30.00	57.00	1.0
9	12:17 PM	12:47 PM	30	0.50	4.50	48.00	78.50	120.00	120.00	30.50	56.75	1.0
10	12:47 PM	1:17 PM	30	0.50	5.00	48.00	78.00	120.00	120.00	30.00	57.00	1.0
11	1:17 PM	1:47 PM	30	0.50	5.50	48.00	78.00	120.00	120.00	30.00	57.00	1.0
12	1:47 PM	2:17 PM	30	0.50	6.00	48.00	78.00	120.00	120.00	30.00	57.00	1.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	72.00	
H_f	42.00	
ΔH	30.00	
H_{avg}	57.00	
I_t	**1.24**	in/hr (clear water rate)

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

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN's 330-180-006, -10, 029, and -046, Menifee	Test Date:	December 7, 2021
Project No.:	23758.1	Test Hole No.:	P-2
Soil Classificaiton:	Bedrock	Effective Hole Dia*:	4.8 in.
Depth of Test Hole:	9.0 ft.	Date Excavated:	December 6, 2021
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	8:19 AM	8:49 AM	30	0.50	0.50	48.00	55.00	108.00	108.00	7.00	56.50	4.3
2	8:49 AM	9:19 AM	30	0.50	1.00	48.00	55.00	108.00	108.00	7.00	56.50	4.3
3	9:19 AM	9:49 AM	30	0.50	1.50	48.00	52.00	108.00	108.00	4.00	58.00	7.5
4	9:49 AM	10:19 AM	30	0.50	2.00	48.00	54.00	108.00	108.00	6.00	57.00	5.0
5	10:19 AM	10:49 AM	30	0.50	2.50	48.00	55.00	108.00	108.00	7.00	56.50	4.3
6	10:49 AM	11:19 AM	30	0.50	3.00	48.00	54.00	108.00	108.00	6.00	57.00	5.0
7	11:19 AM	11:49 AM	30	0.50	3.50	48.00	55.00	108.00	108.00	7.00	56.50	4.3
8	11:49 AM	12:19 PM	30	0.50	4.00	49.00	54.00	108.00	108.00	5.00	56.50	6.0
9	12:19 PM	12:49 PM	30	0.50	4.50	48.00	54.50	108.00	108.00	6.50	56.75	4.6
10	12:49 PM	1:19 PM	30	0.50	5.00	48.00	55.00	108.00	108.00	7.00	56.50	4.3
11	1:19 PM	1:49 PM	30	0.50	5.50	48.00	54.00	108.00	108.00	6.00	57.00	5.0
12	1:49 PM	2:19 PM	30	0.50	6.00	48.00	54.50	108.00	108.00	6.50	56.75	4.6

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	60.00	
H_f	53.50	
ΔH	6.50	
H_{avg}	56.75	
I_t	**0.27**	in/hr (clear water rate)

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

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN's 330-180-006, -10, 029, and -046, Menifee	Test Date:	December 6, 2021
Project No.:	23758.1	Test Hole No.:	P-3
Soil Classificaiton:	Bedrock	Effective Hole Dia*:	4.8 in.
Depth of Test Hole:	8.0 ft.	Date Excavated:	December 6, 2021
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	8:25 AM	8:55 AM	30	0.50	0.50	46.00	59.00	96.00	96.00	13.00	43.50	2.3
2	8:55 AM	9:25 AM	30	0.50	1.00	47.00	59.00	96.00	96.00	12.00	43.00	2.5
3	9:25 AM	9:55 AM	30	0.50	1.50	46.00	57.50	96.00	96.00	11.50	44.25	2.6
4	9:55 AM	10:25 AM	30	0.50	2.00	44.00	56.00	96.00	96.00	12.00	46.00	2.5
5	10:25 AM	10:55 AM	30	0.50	2.50	46.00	58.00	96.00	96.00	12.00	44.00	2.5
6	10:55 AM	11:25 AM	30	0.50	3.00	47.00	58.00	96.00	96.00	11.00	43.50	2.7
7	11:25 AM	11:55 AM	30	0.50	3.50	48.00	58.00	96.00	96.00	10.00	43.00	3.0
8	11:55 AM	12:25 PM	30	0.50	4.00	48.00	58.00	96.00	96.00	10.00	43.00	3.0
9	12:25 PM	12:55 PM	30	0.50	4.50	47.00	57.00	96.00	96.00	10.00	44.00	3.0
10	12:55 PM	1:25 PM	30	0.50	5.00	48.00	58.00	96.00	96.00	10.00	43.00	3.0
11	1:25 PM	1:55 PM	30	0.50	5.50	46.00	56.00	96.00	96.00	10.00	45.00	3.0
12	1:55 PM	2:25 PM	30	0.50	6.00	48.00	58.00	96.00	96.00	10.00	43.00	3.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	48.00	
H_f	38.00	
ΔH	10.00	
H_{avg}	43.00	
I_t	**0.54**	in/hr (clear water rate)

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

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN's 330-180-006, -10, 029, and -046, Menifee	Test Date:	December 6, 2021
Project No.:	23758.1	Test Hole No.:	P-4
Soil Classificaiton:	Bedrock	Effective Hole Dia*:	4.8 in.
Depth of Test Hole:	7.0 ft.	Date Excavated:	December 6, 2021
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	8:57 AM	9:27 AM	30	0.50	0.50	37.00	38.00	84.00	84.00	1.00	46.50	30.0
2	9:27 AM	9:57 AM	30	0.50	1.00	38.00	40.00	84.00	84.00	2.00	45.00	15.0
3	9:57 AM	10:27 AM	30	0.50	1.50	40.00	42.00	84.00	84.00	2.00	43.00	15.0
4	10:27 AM	10:57 AM	30	0.50	2.00	42.00	44.00	84.00	84.00	2.00	41.00	15.0
5	10:57 AM	11:27 AM	30	0.50	2.50	36.00	38.00	84.00	84.00	2.00	47.00	15.0
6	11:27 AM	11:57 AM	30	0.50	3.00	38.00	40.00	84.00	84.00	2.00	45.00	15.0
7	11:57 AM	12:27 PM	30	0.50	3.50	40.00	41.50	84.00	84.00	1.50	43.25	20.0
8	12:27 PM	12:57 PM	30	0.50	4.00	41.50	43.00	84.00	84.00	1.50	41.75	20.0
9	12:57 PM	1:27 PM	30	0.50	4.50	36.00	37.50	84.00	84.00	1.50	47.25	20.0
10	1:27 PM	1:57 PM	30	0.50	5.00	37.50	39.00	84.00	84.00	1.50	45.75	20.0
11	1:57 PM	2:27 PM	30	0.50	5.50	39.00	40.50	84.00	84.00	1.50	44.25	20.0
12	2:27 PM	2:57 PM	30	0.50	6.00	40.50	42.00	84.00	84.00	1.50	42.75	20.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	43.50	
H_f	42.00	
ΔH	1.50	
H_{avg}	42.75	
I_t	**0.08**	in/hr (clear water rate)

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

APPENDIX F2

EVANS SITE GEOTECHNICAL INVESTIGATION

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

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED INDUSTRIAL DEVELOPMENT APN 331-060-018 MENIFEE, CALIFORNIA

PROJECT NO. 23798.1
MAY 12, 2022

Prepared For:

Compass Danbe Real Estate Partners II, LLC
999 N. Pacific Coast Highway
El Segundo, California 90245

Attention: Mr. Mark Bachli

May 12, 2022

Compass Danbe Real Estate Partners II, LLC
999 N. Pacific Coast Highway
El Segundo, California 90245

Project No. 23798.1

Attention: Mr. Mark Bachli

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Industrial Development, APN 331-060-018, Menifee, California.

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

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. Any undocumented fill material and any loose older alluvial materials should be removed from structural areas and areas to receive engineered compacted fills. The data developed during this investigation indicates that removals on the order of approximately 3 to 5 feet will be required from currently planned development areas. The given removal depths are preliminary and the actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

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

Non-conducive infiltration rates were obtained for the soils tested.

LOR Geotechnical Group, Inc.

TABLE OF CONTENTS

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

INTRODUCTION

During April and May of 2022, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed industrial development of Assessor's Parcel Number (APN) 331-060-018, Menifee, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photographs of the site and surrounding regions dated 1966 through 2021;
- Geologic field reconnaissance mapping to verify the aerial distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Percolation testing via the borehole test method;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

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

PROJECT CONSIDERATIONS

To orient our investigation at the site, an Alta Survey prepared by J.D. Cole and Associates, Inc., dated January 30, 2022, was furnished for our use. The existing site conditions and proposed building configurations, associated driveways, parking, and landscape areas were indicated on this plan. Also provided was a Site Plan prepared by Herdman Architecture + Design, dated April 19, 2022. These plans were utilized as base maps for our field investigation and are presented as Enclosures A-2a and A-2b, respectively, within Appendix A. As noted on the Site Plan, development of the site will include an industrial type structure comprising 131,918± square foot building with the remainder of the property to be used for driveways, parking, and landscape areas. The building is anticipated to be of concrete, masonry, or similar type construction and light to moderate foundation loads are anticipated with these structures.

Infiltration is proposed via underground chamber type systems. Depths and locations were provided by CASC Engineering and Consulting.

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

AERIAL PHOTO ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photograph images of varying scales. We reviewed imagery available from Google Earth Pro (2022) computer software and from online Historic Aerials (2022).

To summarize briefly, the site was vacant land utilized for farming since 1966, the earliest photograph available, to current day. No evidence for the presence of faults traversing the site area or mass movement features was noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The approximate 6.8-acre site is located at the southeast corner of Ethanac Road and Evans Road in the city of Menifee, California. It consists of vacant land partially used for farming within the southern one-sixth in conjunction with the adjacent property to the south. Some manure was present spread out over the northern five-sixths of the site. Topographically, the site is planar with a gentle fall to the west-northwest.

Ethanac Road, a fully improved roadway, lies north of the site with vacant land beyond. Evans Road, a dirt roadway lies west of the site with a horse ranch beyond. An unlined earthen channel bounds the site on the east followed by vacant land. Vacant farm land lies south of the site.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on April 21, 2022. The work consisted of advancing a total of 6 exploratory borings and 2 percolation test holes using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. The approximate locations of our exploratory borings are presented on Enclosure A-2, within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by a geologist from this firm. The borings were drilled to maximum depths of 11.5 to 51.5 feet below the existing ground surface. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet, and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the boring logs is presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

As shown on Enclosure A-1, within Appendix A, the site is located within the United States Geological Survey Romoland 7.5 minute quadrangle topographic map. This region lies along the north-central portion of the Perris block of the northern Peninsular Ranges geologic province of southern California. While the Perris block is considered to be a relatively stable structural block, it is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the east, and the Cucamonga fault zone on the north. The Perris block is underlain by rocks of the Peninsular Ranges batholith, a very large mass of crystalline igneous rocks of Cretaceous age and with no known floor, and by prebatholithic metasedimentary and metavolcanic rocks of older ages.

The Perris block has a series of erosional surfaces, marked by low topographic relief and capped with unconsolidated alluvial sediments stripped from the surrounding highlands. This area was mapped by the California Division of Mines and Geology as being underlain by deposits of old alluvial fan deposits (Morton, 2003).

The interior of the Perris Plain is considered to be relatively stable with few known active faults. However, this plain is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the northeast, the San Andreas fault zone on the north, and the Agua-Tibia fault zone on the south. As the subject site is located near the western margin of the Perris Plain, the Elsinore fault is the closest known active fault in relation to the site. At its closest approach, the Elsinore fault is located approximately 14.6 kilometers (9.1 miles) southwest from the site. A complete listing of the distances to known active faults in relation to the various planning areas is given in the Faulting section of this report.

The site is shown within the regional geologic setting as mapped by the U.S.G.S. on the enclosed Regional Geologic Map, Enclosure A-3, within Appendix A.

Site Geologic Conditions

Fill/Topsoil: Fill/topsoil materials were encountered within our exploratory borings to depths of approximately 2 feet. The fill/topsoil materials are believed to be associated with current and past agricultural practices (discing) at the site. As encountered, the fill/topsoil materials were comprised of silty sand which were predominantly brown, dry, and in a loose state. Some manure was noted at the surface.

Older Alluvium: Older alluvial materials were encountered underlying the fill materials described above within all of our exploratory borings. The older alluvial soils were encountered to the maximum depth explored of approximately 51.5 feet. These units were noted to mainly consist of clayey sand, sandy silt, silty sand, lean clay with sand and minor units of poorly graded sand and well graded sand. The older alluvial materials were in a relatively dense/hard to very dense/very hard state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Swell testing indicates that the fine grained, lean clay with sand materials will have a very low expansion potential in their natural state.

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

Groundwater Hydrology

Groundwater was not encountered within any of our exploratory borings as advanced to a maximum depth of approximately 51.5 feet below the existing ground surface nor was any groundwater seepage observed during our site reconnaissance.

In order to estimate the approximate depth to groundwater in the site area, a search was conducted for local groundwater (well) level measurements within the Cooperative Well Measuring Program, Spring 2021 (Watermaster Support Services et al., 2021). This database contains depth to groundwater measurements dating back to 1993. We also conducted a search of the water well database information provided in the California Department of Water Resources (CDWR) Water Library Data website (CDWR, 2022).

The closest well is an Eastern Municipal Water District (EMWD) well, State Well No. 05S03W16K001S (Well "EMWD14429"), located approximately 0.4 mile to the south-southwest of the subject property. Groundwater level data for this well was limited to one reading in October 2021. A measuring point elevation of 1,425± feet above mean sea level was reported. The depth to groundwater from the measuring point was approximately 62 feet. Two other wells are located within approximately 0.6 mile of the subject property. One of these wells, State Well No. 05S03W17A001S, located west-southwest of the subject property, has groundwater level data limited to one reading in 1995. A measuring point elevation of 1,424± feet above mean sea level was reported. The depth to groundwater provided was 22 feet. The second of these wells, Well "EMWD12765", located south-southwest of the subject property, has groundwater level data ranging from October 2011 to October 2012. A measuring point elevation of 1,428± feet above mean sea level was reported. The depth to groundwater from the measuring point has ranged from approximately 66 to 70 feet.

As noted on Enclosure A-2, the lowest elevation of the site is 1,418 feet above mean sea level. Based on the information above, groundwater in the region appears to be at depths on the order of 50 to 60 feet below the ground surface.

Mass Movement

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

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2003) nor does the site lie within a County of Riverside fault zone (CRTLMA, 2021). No evidence of faulting projecting into or crossing the site was noted during our aerial photograph review or our review of published geologic maps.

As previously mentioned, the closest known active earthquake fault with a documented location is the Elsinore fault located approximately 14.6 kilometers (9.1 miles) to the southwest. In addition, other relatively close active faults include the San Jacinto fault located approximately 16.5 kilometers (10.2 miles) to the northeast, and the San Andreas fault located approximately 39.5 kilometers (25.5 miles) to the northeast.

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

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

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

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 closer distance and larger 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. (2021). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from January 1, 1932 through May 9, 2022.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As depicted on Enclosure A-4, within Appendix A, the site lies within a relatively active region associated with the San Jacinto fault to the northeast.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 miles) 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 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 time period for the events on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid to late1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the Elsinore fault zone to the southwest and the San Jacinto fault zone to the northeast 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 in the region around the subject site. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seismic-induced settlement, seiches and tsunamis, earthquake induced flooding, landsliding, and rockfalls.

Liquefaction: The site lies within an area mapped by the County of Riverside has having a low potential for liquefaction (CRTLMA, 2022). The potential for liquefaction generally occurs during strong ground shaking within granular loose sediments where the groundwater is usually less than 50 feet below the ground surface. As found during this investigation, the site is underlain by dense/hard to very dense /very hard older alluvial soils in the upper 50 feet, therefore, the possibility of liquefaction at the site is considered nil.

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

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

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

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

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

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 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 current investigation, mapping by others, and our experience in the site region indicates that the materials beneath the site are considered Site Class D stiff soil.

CBC Earthquake Design Summary

Earthquake design criteria have been formulated in accordance with the 2019 CBC and ASCE 7-16 for the site based on the results of our investigation to determine the Site Class and an assumed Risk Category II. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region. In addition, the building official should confirm the Risk Category utilized in our design (Risk Category II). Our design values are provided in Appendix D.

PERCOLATION TESTING AND INFILTRATION RATE RESULTS

Two borehole percolation tests were conducted in general accordance with the Shallow Percolation Test procedure as outlined in the Design Handbook for Low Impact Development Best Management Practices (CRFCWCD, 2011). The requested locations of our test are illustrated on Enclosure A-2. Test borings were drilled to depths of approximately 8 and 10 feet below the existing ground surface as requested, on April 21, 2022. Subsequent to drilling, a 3-inch diameter, perforated PVC pipe wrapped in filter fabric was placed within each test hole and 3/4-inch gravel was placed between the outside of the pipe and the hole wall. Test holes were pre-soaked the same day as drilling. Testing took place the next day, April 22, 2022, within 26 hours but not before 15 hours, of the pre-soak. The holes were filled with a variable height column of water using water from a 200 gallon water tank. Test periods consisted of allowing the water to drop in 30-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 12 readings were recorded. The percolation test data was then converted to an infiltration rate using the Porchet Method.

Infiltration rate results are summarized in the following table:

Test No.	Depth*	Clear Water Infiltration Rate** (in/hr)
P-1	8	0.02
P-2	10	0.09

* Depth measured below existing ground surface.
** Porchet Method determined Infiltration rate with an effective diameter due to loss in volume of water due to gravel packing.

The results of this testing are presented as Enclosures E-1 and E-2 in Appendix E. The Infiltration rates indicate non-conducive infiltration characteristics for the materials tested.

CONCLUSIONS

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

It should be noted that the subsurface conditions encountered in our exploratory borings are indicative of the locations explored and the subsurface conditions may vary. If conditions are encountered during the construction of the project that differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Foundation Support

To provide adequate support for the proposed structure we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. The construction of this compacted fill mat will allow for the removal of the existing fill material which was loose and any current subsurface improvements, such as utilities, foundations, etc., that may be present locally.

Conventional foundation systems utilizing either individual spread footings and/or continuous wall footings will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat.

Soil Expansiveness

Our expansion index testing of a representative sample of the on-site soils indicates a very low expansion potential. For very low expansive soils, specialized foundation design and construction procedures to resist expansive soil activity are not considered necessary.

Careful evaluation of onsite 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.

Infiltration

The results of our field investigation and percolation test data indicates the site soils at the depths tested are not conducive to infiltration. Based on the results of this investigation, acceptable infiltration is also not anticipated to occur at other depths due to the amount of silty/clayey fines and dense to very dense nature of the 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).

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

RECOMMENDATIONS

Geologic Recommendations

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

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An onsite, pre-job meeting with the developer, the contractor, the jurisdictional agency, and the geotechnical engineer should occur prior to all grading related operations.

Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

Any undocumented fill encountered during grading should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. It is our recommendation that any existing fills under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

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

The existing fill/topsoil material, as well as any loose older alluvial soils, if encountered, should be removed from all proposed structural and/or fill areas. The data developed during this investigation indicates that removals on the order of 3 to 5 feet deep will be required from proposed development areas in order to encounter competent older alluvium upon which engineered compacted fill can be placed. The given removal depths are preliminary. Deeper fills may be present, primarily in areas of current improvements. Removals should expose older alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557). The actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

Preparation of Fill Areas

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

Engineered Compacted Fill

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

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

Preparation of Foundation Areas

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

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

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations 5 feet deep and greater should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements. Short-term excavations of 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based upon the findings from our exploratory borings, it appears that Type C soils are the predominant type of soil on the project and all short-term excavations should be based on this type of soil.

Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1). Short-term excavation construction and maintenance are the responsibility of the contractor and should be a consideration of his methods of operation and the actual soil conditions encountered.

Slope Construction

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

Slope Protection

Since the site soil materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after 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.

Soil Expansiveness

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

Additional evaluation of on-site and any imported soils for their expansion potential should be conducted following completion of the grading operation.

Foundation Design

If the site is prepared as recommended, the proposed structure may be safely founded on conventional shallow foundations, either individual spread footings or continuous wall footings, bearing on a minimum of 24 inches of engineered compacted fill placed over competent older alluvial materials. 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 soil bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads. Footings at least 15 inches wide, placed at least 18 inches below the lowest adjacent final grade, may be designed for a maximum soil bearing pressure of 2,400 psf for dead plus live loads.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure. The buildings should be setback from slopes as indicted within the California Building Code (2019).

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 270 pounds per square foot per foot of depth. Base friction may be computed at 0.28 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Settlement

Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Maximum settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlements between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Building Area Slab-on-Grade

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

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

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

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork

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

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

Wall Pressures

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

For design of retaining walls unrestrained against movement at the top, we recommend an active pressure of 46 pounds per square foot (psf) per foot of depth be used.

This assumes level backfill consisting of compacted, non-expansive, 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. Non-expansive import soils may be required. 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 0.45 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

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

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

Preliminary Pavement Design

Testing and design for preliminary onsite pavement was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing, and upon an assumed Traffic Index generally used for similar projects, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject on-site pavement improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
On site vehicular parking with occasional truck traffic (ADTT=10)	6.0	25	0.25’ AC / 0.80' AB or 5" JPCP / 4" AB
Light to moderate truck traffic (ADTT=25)	7.0	25	0.30'AC / 0.95'AB or 6" JPCP / 4" AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base JPCP - Jointed Plain Concrete Pavement with MR ≥ 550 psi			

The above structural sections are predicated upon 90 percent relative compaction (ASTM D 1557) of all utility trench backfills and 95 percent relative compaction (ASTM D 1557) of the upper 12 inches of pavement subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 Aggregate Base.

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 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 concrete pavement sections should have a minimum modulus of rupture (MR) of 500 pounds per square inch (psi). Transverse joints should be sawcut in the pavement at approximately 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other.

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

Infiltration

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

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by the project geotechnical consultant prior to construction to confirm that the intent of the recommendations presented in this report have been incorporated into the design.

Additional R-value, expansion, and soluble sulfate content testing may be needed after/during site rough grading.

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

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavations prior to the processing and preparation of the bottom areas for fill placement.

3. Scarifying and recompacting prior to fill placement.

4. 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

5. Foundation excavations.

6. Subgrade preparation for pavements and slabs-on-grade. This includes pre-saturation testing of slab-on-grade and flatwork areas to verify moisture content.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Compass Danbe Real Estate Partners II, LLC and their design consultants for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

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

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

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

TIME LIMITATIONS

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

CLOSURE

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

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

Robert M. Markoff, CEG
Engineering Geologist

John P. Leuer, GE 2030
President

REGISTERED PROFESSIONAL ENGINEER
JOHN P. LEUER
NO. 2030
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ROBERT M. MARKOFF
No. 2073
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

AAT:RMM:JPL:ss

Distribution: Addressee (2) and PDF via email mbachli@danbe.com
cc: Vicky Valenzuela via email vicky@cdrepartners.com

REFERENCES

American Society of Civil Engineers, 2016, Minimum Design Load for Buildings and Other Structures, ASCE 7-16.

California Building Standards Commission and International Conference of Building Officials, 2019, California Building Code, 2019 Edition.

California Department of Water Resources, 2022, Online Water Data Library (WDL), https://wdl.water.ca.gov/waterdatalibrary/Map.aspx, accessed May 2022.

County of Riverside, Flood Control and Water Conservation District (CRFCWCD), 2011, Design Handbook for Low Impact Development Best Management Practices, dated September 2011.

County of Riverside, Transportation and Land Management Agency (CRTLMA), 2022, Geographic Information System, http://www3.tlma.co.riverside.ca.us, accessed April 2022.

Google Earth, 2022, Imagery from various years, www.google.com/earth.

Hart, E.W. and W.A. Bryant, 2010, Fault-Rupture Hazard Zones in California, California Dept. of Conservation Division of Mines and Geology Special Publication 42.

Historic Aerials (Nationwide Environmental Title Research, LLC), 2022, Imagery from Various Years, https://www.historicaerials.com/, accessed May 2022.

Larson, R., and Slosson, J., 1992, The Role of Seismic Hazard Evaluation in Engineering Reports, in Engineering Geology Practice in Southern California, AEG Special Publication Number 4, pp 191-194.

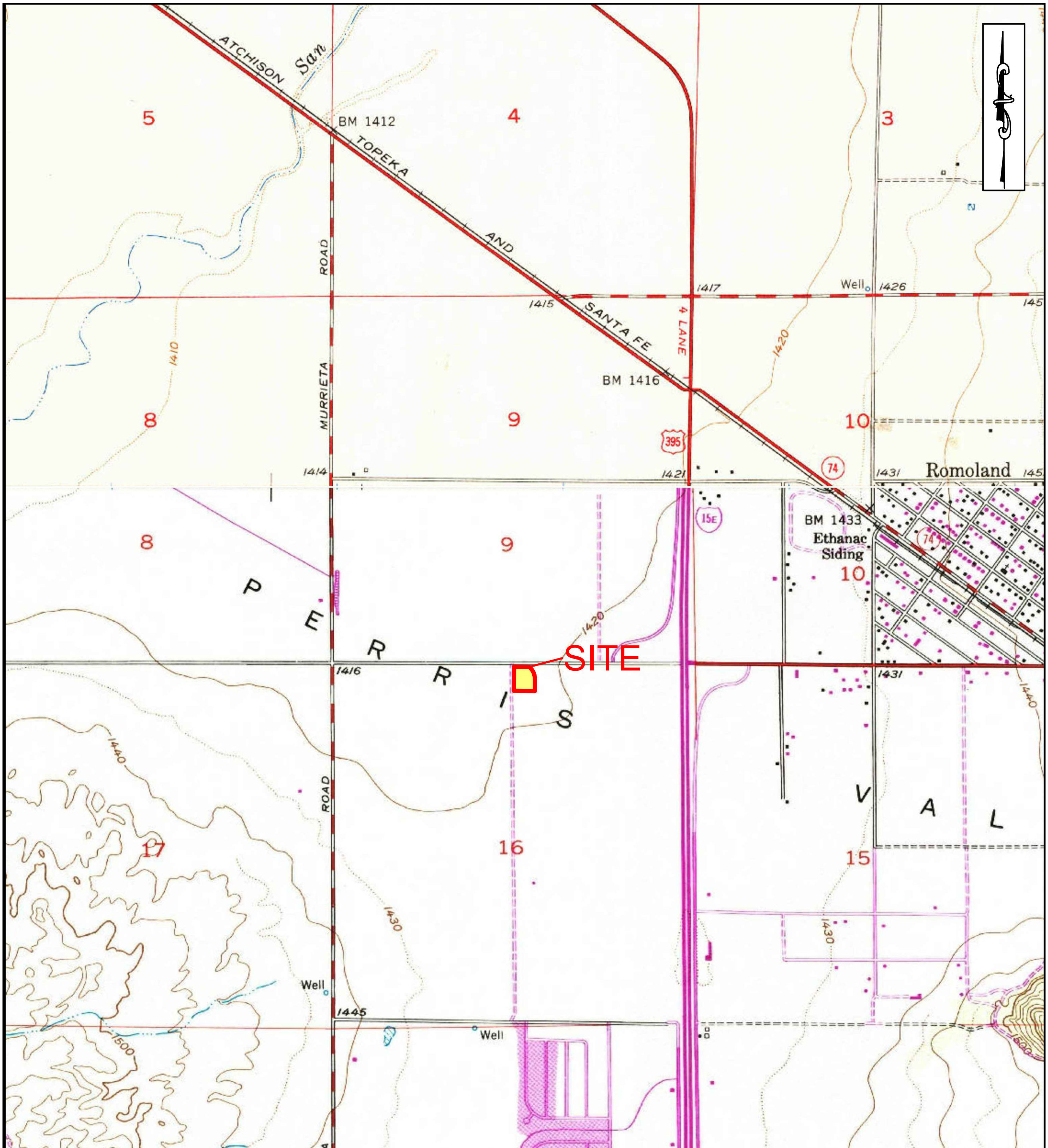
Morton, D.M., 2003, Preliminary Geologic Map of the Romoland 7.5' Quadrangle, Riverside County, California, U.S.G.S. Open File Report 03-102.

USGS, 2021, https://earthquake.usgs.gov/earthquakes/map/, accessed February 2022.

Watermaster Support Services, Western Municipal Water District, and San Bernardino Valley Water Conservation District, 2021, Cooperative Well Measuring Program, Spring 2021, Covering the Upper Santa Ana River Watershed, San Jacinto Watershed, and Santa Margarita Watershed, July 1, 2021.

APPENDIX A

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



INDEX MAP

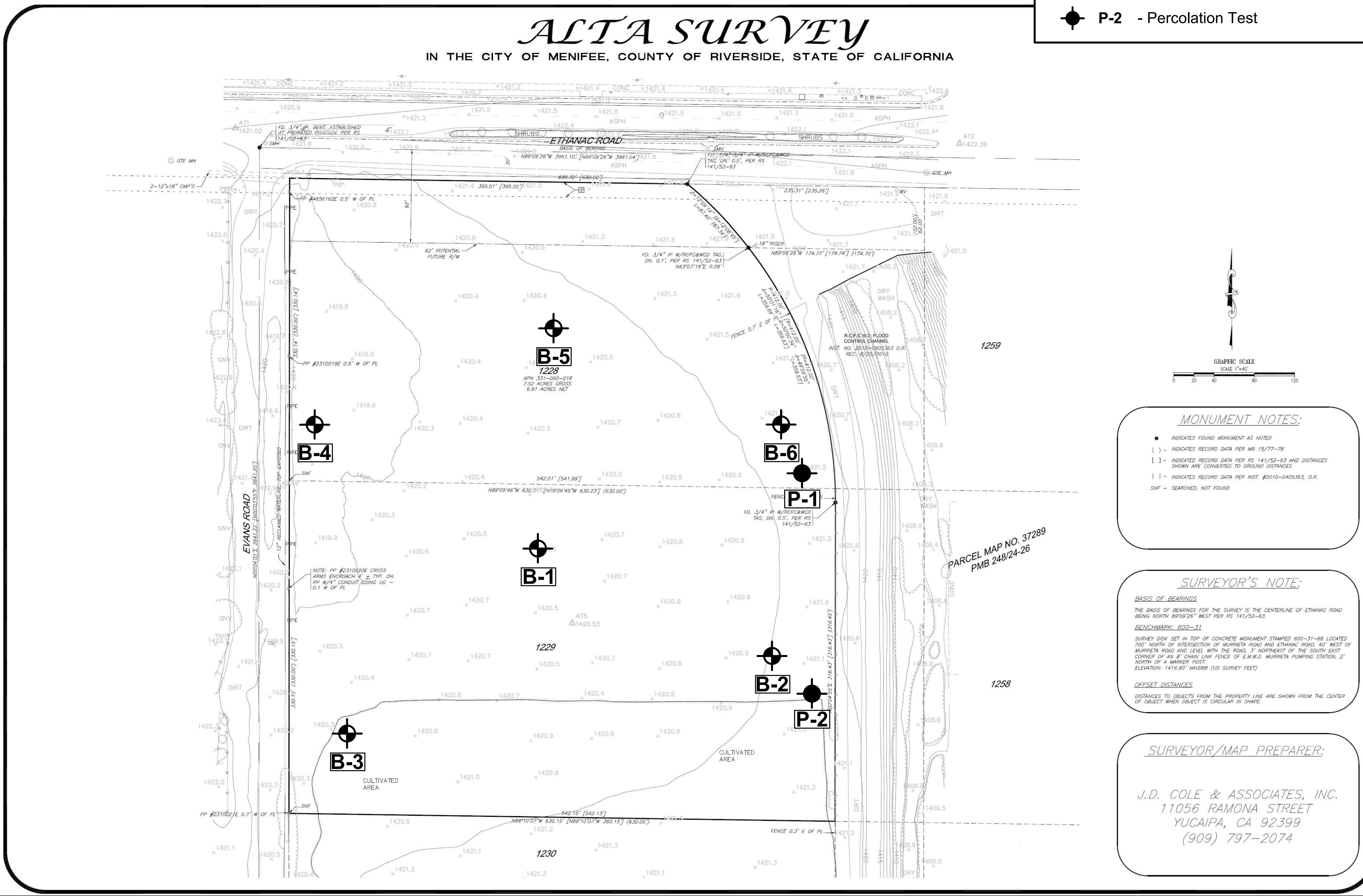
PROJECT:	APN 331-060-018, Menifee, California	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2022
		SCALE:	1" ≈ 2,000'

Legend

(Locations Approximate)

Map Symbols

- **B-6** - Exploratory Boring
- **P-2** - Percolation Test



ALTA SURVEY

PROJECT NO.: 23798.1

ENCLOSURE: A-2a

DATE: May 2022

SCALE: 1" ≈ 100'

PROJECT: APN 331-060-018, Menifee, California

CLIENT: Compass Danbe Real Estate Partners II, LLC

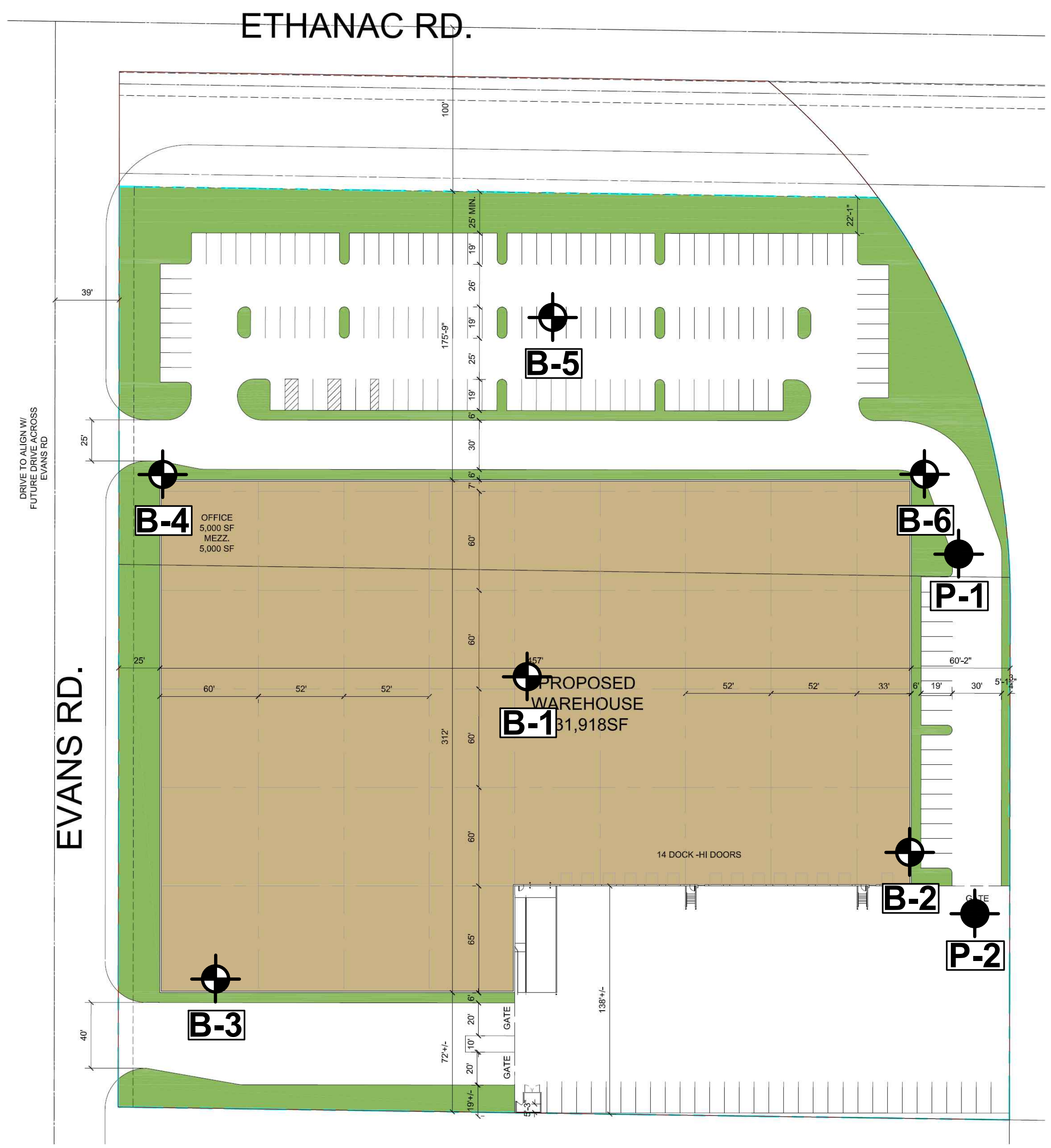
LOR GEOTECHNICAL GROUP, INC.

Legend

(Locations Approximate)

Map Symbols

- **B-6** - Exploratory Boring
- **P-2** - Percolation Test



A CONCEPTUAL SITE PLAN
SCALE: 1" = 30'

PROJECT INFORMATION - SCHEME 8				04.19.2022
GROSS SITE AREA	297,289 SF		6.82 AC	
TOTAL BUILDING AREA			131,918 SF	
NET FAR			44.4%	
MAX FAR			100%	
ZONING			EDC-NG	
STREET SETBACK			25'	
INTERIOR SIDE/REAR			0'	
MAX BUILDING HEIGHT			70'	
CONSTRUCTION TYPE			III-B	
LANDSCAPE REQUIRED		10%	29,729 SF	
LANDSCAPE PROVIDED		15.2%	45,248 SF	
BUILDING AREA			**131,918 SF**	
FOOTPRINT			126,918 SF	
WAREHOUSE			121,918 SF	
OFFICE			5,000 SF	
MEZZANINE			5,000 SF	
OFFICE			5,000 SF	
PARKING REQUIRED			162	
WAREHOUSE @ 1/1,000			122	
OFFICE @ 1/250			40	
PARKING PROVIDED			177	
STANDARD			156	
VAN ADA			1	
STND ADA			3	
EV VAN ADA			1	
EV STND ADA			1	
EVC			9	
CLEAN AIR			6	
TRAILER				

CDRE- EVAN RD.
MENIFEE, CA

APN - 331-060-018

PROJECT
SCHEME 8
SITE PLAN

HERDMAN
ARCHITECTURE + DESIGN
16201 Scientific Way
Irvine, CA 92618
www.Herdman-ad.com
714.389.2800
info@Herdman-ad.com

A21-2396
04.19.2022

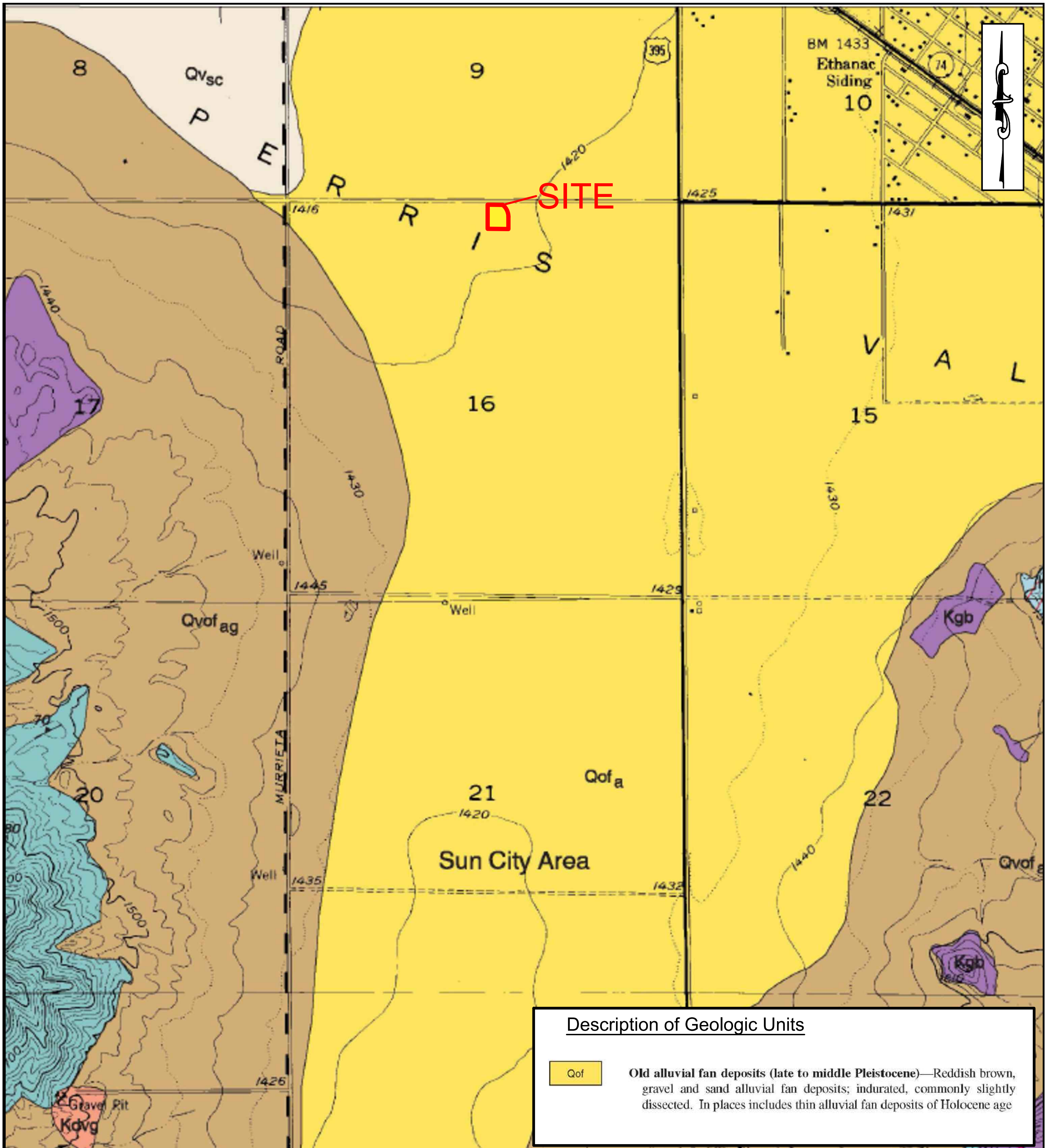
NORTH

CONCEPTUAL SITE PLAN

SITE PLAN

PROJECT: APN 331-060-018, Menifee, California
CLIENT: Compass Danbe Real Estate Partners II, LLC
PROJECT NO.: 23798.1
ENCLOSURE: A-2b
DATE: May 2022
SCALE: 1" ≈ 100'

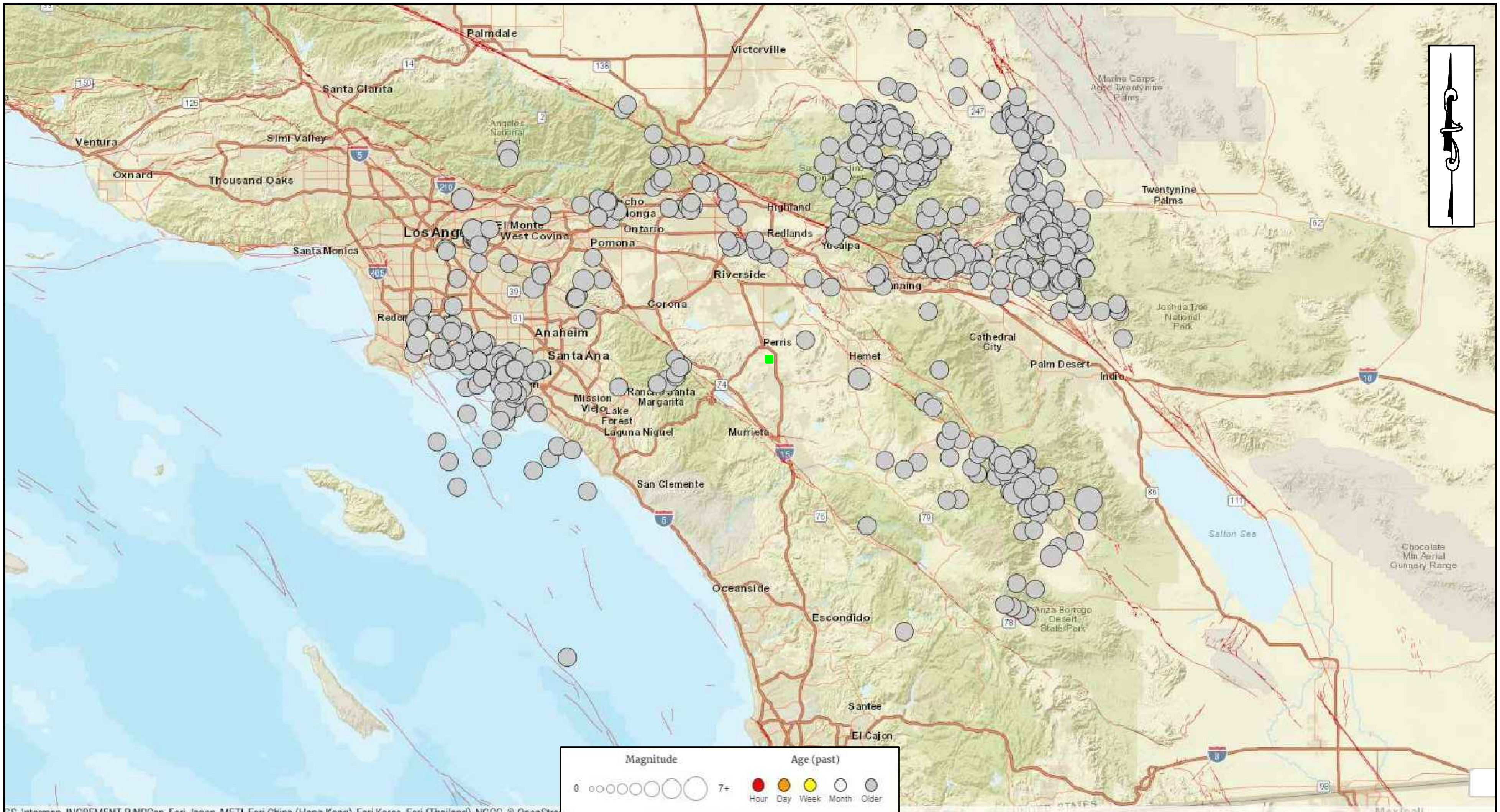
LOR GEOTECHNICAL GROUP, INC.



REGIONAL GEOLOGIC MAP

(Morton, 2003)

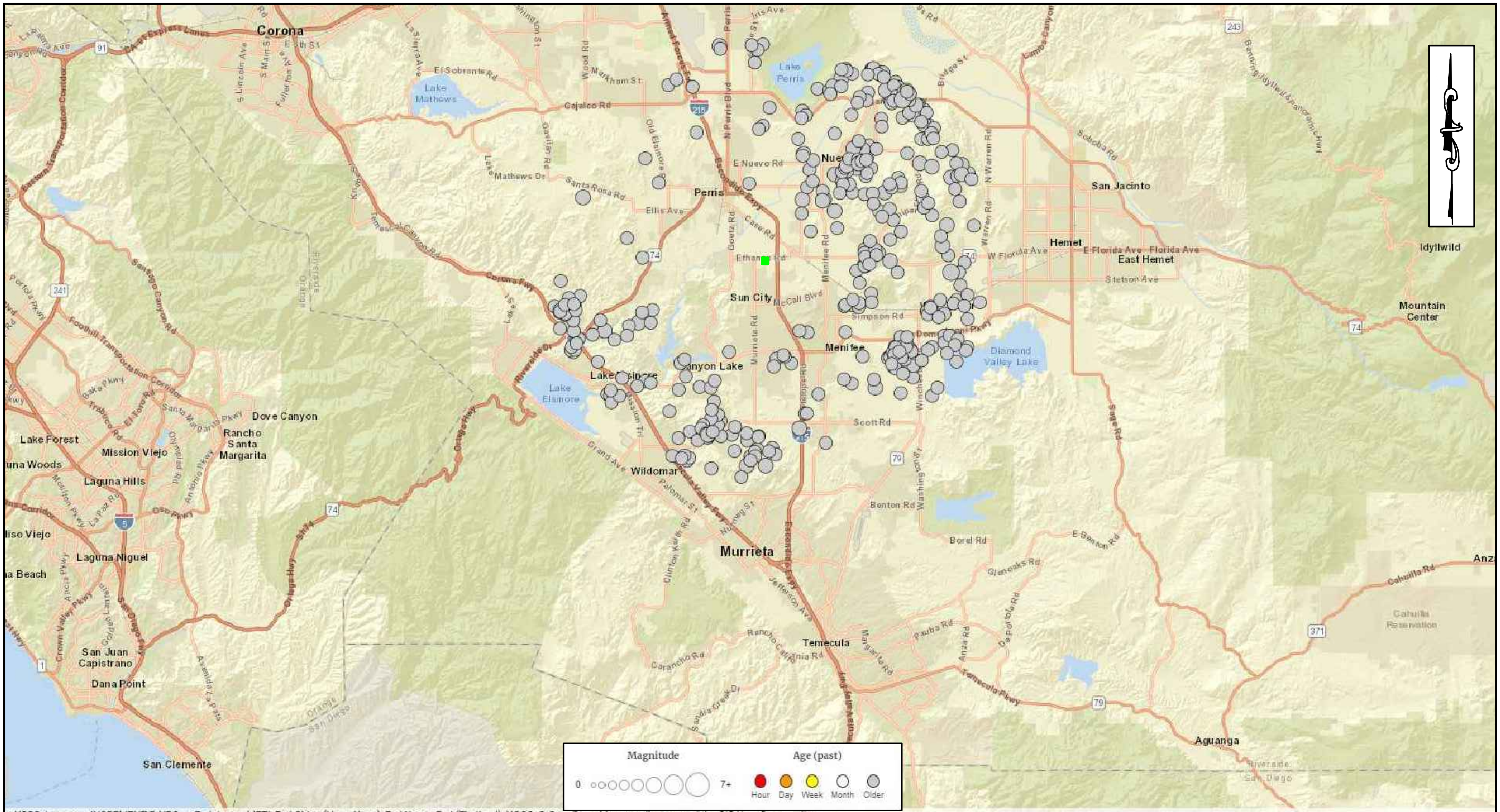
PROJECT:	APN 331-060-018, Menifee, California	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	April 2022
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2022) real-time earthquake epicenter map. Plotted are 530 epicenters of instrument-recorded events from 01/01/32 to present (05/09/22) 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 331-060-018, Menifee, California	**PROJECT NO.:**	23789.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2022
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2022) real-time earthquake epicenter map. Plotted are 427 epicenters of instrument-recorded events from 01/01/78 to present (05/09/22) of local magnitude 1+ within a radius of ~9.2 miles (15 kilometers) of the site. Location accuracy varies. The site is indicated by the green square. The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These events are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 15km Radius

PROJECT:	APN 331-060-018, Menifee, California	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2022
		SCALE:	1" ≈ 10km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling 6 exploratory borings to depths between approximately 11.5 to 51.5 feet below the existing ground surface using a Mobile B-61 drill rig on April 21, 2022. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

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

Relatively undisturbed samples of the subsoils were obtained at a maximum interval of 5 feet. The samples were recovered by using a California split barrel sampler of 2.50 inch inside diameter and 3.25 inch outside diameter or a Standard Penetration Sampler (SPT) from the ground surface to the total depth explored. The samplers were driven by a 140 pound automatic trip hammer dropped from a height of 30 inches. The number of hammer blows required to drive the sampler into the ground the final 12 inches were recorded and further converted to an equivalent SPT N-value. Factors such as efficiency of the automatic trip hammer used during this investigation (80%), borehole diameter (8"), and rod length at the test depth were considered for further computing of equivalent SPT N-values corrected for field procedures (N60) which are included in the boring logs, Enclosures B-1 through B-6.

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

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

CONSISTENCY OF SOIL

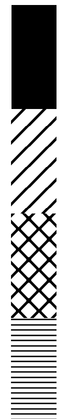
SANDS

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

COHESIVE SOILS

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

SAMPLE KEY



Symbol	Description
	INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE
	INDICATES BULK SAMPLE
	INDICATES SAND CONE OR NUCLEAR DENSITY TEST
	INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

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

BORING LOG LEGEND

PROJECT:	Proposed Industrial Developement	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2022

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

PARTICLE SIZE LIMITS

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

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

(U.S. STANDARD SIEVE SIZE)

SOIL CLASSIFICATION CHART

PROJECT:	Proposed Industrial Developement	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	May 2022

LOG OF BORING B-1

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		3, 4, 7, 9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 40% fine grained sand, 35% silty fines, brown, dry, loose, some manure on surface.**
	64 for 11"		7.9		128.4			SC	@ 2 feet, OLDER ALLUVIUM: **CLAYEY SAND, approximately 10% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 40% clayey fines of low plasticity, red brown, damp, some secondary calcite.**
5	41		18.3		108.2				
10	61		17.9		110.1			SM	**@ 10 feet, SILTY SAND, approximately 5% medium grained sand, 75% fine grained sand, 20% silty fines, yellow brown, moist, micaceous.**
15	47		20.3		93.0			ML	**@ 15 feet, SANDY SILT, approximately 30% fine grained sand, 70% silty fines, brown, damp.**
20	68 for 11"		13.8		118.6			CL	**@ 20 feet, LEAN CLAY with SAND, approximately 20% fine grained sand, 80% clayey fines of low plasticity, red brown, damp.**
25	76		22.9		103.1			ML	**@ 25 feet, SANDY SILT, approximately 20% fine grained sand, 80% silty fines with trace clay, yellow brown, moist.**
30	67		4.5					SP	**@ 30 feet, POORLY GRADED SAND, approximately 5% coarse grained sand, 35% medium grained sand, 55% fine grained sand, 5% silty fines, tan, dry.**
35	118		3.7						
40	77		3.5						
45	81		4.6					SW	**@ 45 feet, WELL GRADED SAND, approximately 30% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, white, dry.**
50	36		16.5					SM	**@ 50 feet, SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, red brown, moist.**
								CL	**@ 51 feet, LEAN CLAY with SAND, approximately 20% fine grained sand, 80% clayey fines of low plasticity, red brown, moist.**
55									**END OF BORING @ 51.5'**
60									**Fill to 2'** **No groundwater** **No bedrock**

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1,421
		DATE DRILLED:	April 21, 2022
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)	U.S.C.S.	DESCRIPTION
0						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 45% fine grained sand, 35% silty fines, dry, loose, manure on surface.**
	27		10.4		121.1	SC	@ 2 feet, OLDER ALLUVIUM: **CLAYEY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% clayey fines of low plasticity, red brown, damp.**
5	64		11.5		118.4		
10	33		28.2		93.2		@ 10 feet, becomes slightly coarser grained, moist.
15	61		20.6		103.8	ML	@ 15 feet, SANDY SILT, approximately 40% fine grained sand, 60% silty fines, red brown, damp.
20	43		32.2		87.9	CL	@ 20 feet, LEAN CLAY with SAND, approximately 20% fine grained sand, 80% clayey fines of low plasticity, red brown, moist.
25							

END OF BORING @ 21.5'

Fill to 2'
No groundwater
No bedrock

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1,421
		DATE DRILLED:	April 21, 2022
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-2

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-3

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	U.S.C.S.	DESCRIPTION
0		9, 10, 11				SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 35% silty fines, brown, dry, loose.**
	41		5.7		121.6		@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 30% silty fines, red brown, damp.**
5	29		8.3		120.2		**@ 5 feet, becomes slightly coarser grained.**
10	62		10.9		126.5	CL	**@ 10 feet, LEAN CLAY with SAND, approximately 5% medium grained sand, 20% fine grained sand, 75% clayey fines of low plasticity, red brown, damp.**
15	47		28.7		92.8		**@ 15 feet, becomes moist.**
20	33		34.3		84.8		**@ 20 feet, LEAN CLAY with SAND, approximately 15% fine grained sand, 85% clayey fines of low plasticity, gray, moist.**
25	82 for 11"		15.7		114.6	SC	**@ 25 feet, CLAYEY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 35% clayey fines of low plasticity, red brown, damp.**
30							

END OF BORING @ 26.42'

Fill to 2'
No groundwater
No bedrock

PROJECT: Proposed Industrial Development		**PROJECT NO.:**	23798.1
CLIENT: Compass Danbe Real Estate Partners II, LLC		**ELEVATION:**	1,420
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	April 21, 2022
		EQUIPMENT:	Mobile B-61
HOLE DIA.:	8"	**ENCLOSURE:**	B-3

LOG OF BORING B-4

TEST DATA								
DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	U.S.C.S.	DESCRIPTION
0							SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 45% fine grained sand, 30% silty fines, brown, dry, loose, some manure on surface.**
	76 for 11"		7.8		116.1		SC	@ 2 feet, OLDER ALLUVIUM: **CLAYEY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 30% clayey fines of low plasticity, red brown, damp.**
5	71		10.4		118.6		CL	@ 5 feet, **LEAN CLAY with SAND, approximately 15% fine grained sand, 85% clayey fines of low plasticity, red brown, damp.**
10	54		13.9		120.6			
15	63		8.0		120.0		SC	@ 15 feet, **CLAYEY SAND, approximately 20% coarse grained sand, 20% medium grained sand, 20% fine grained sand, 40% clayey fines of low plasticity, red brown, damp.**
20	50		20.1		109.1		ML	@ 20 feet, **SANDY SILT, approximately 5% medium grained sand, 30% fine grained sand, 65% silty fines with trace clay, red brown, moist.**
25								

END OF BORING @ 21.5'

Fill to 2'
No groundwater
No bedrock

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1,420
		DATE DRILLED:	April 21, 2022
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-4

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-5

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		8, 9, 10, 11						SM	**@ 0 feet,** FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% silty fines, brown, dry, loose, some manure on surface.**
	35		5.9		124.8			SC	**@ 2 feet,** OLDER ALLUVIUM: **CLAYEY SAND, approximately 5% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 45% clayey fines of low plasticity, red brown, damp, some thin calcite stringers.**
5	52		12.2		120.9			ML	**@ 5 feet, SANDY SILT, approximately 20% medium grained sand, 25% fine grained sand, 55% silty fines with trace clay, red brown, damp.**
10	36		12.6		122.7				
									END OF BORING @ 11.5' **Fill to 2'** **No groundwater** **No bedrock**
15									

PROJECT: Proposed Industrial Development		**PROJECT NO.:**	23798.1
CLIENT: Compass Danbe Real Estate Partners II, LLC		**ELEVATION:**	1,420
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	April 21, 2022
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-5

LOG OF BORING B-6

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 35% silty fines, brown, dry, loose, some manure on surface.**
	43		6.6		127.5				@ 2 feet, OLDER ALLUVIUM: **CLAYEY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 35% clayey fines of low plasticity, red brown, dry.**
5	60		12.3		113.2				@ 5 feet, becomes slightly coarser grained, moist.
10	65		7.2		122.3			ML SM	@ 10 feet, SANDY SILT/SILTY SAND, approximately 20% medium grained sand, 30% fine grained sand, 50% silty fines, yellow brown, damp.
15	68		16.7		109.8			SM	@ 15 feet, SILTY SAND, approximately 80% fine grained sand, 20% silty fines, yellow brown, moist, some secondary calcite.
20	38		31.4		91.7			CL	@ 20 feet, LEAN CLAY with SAND, approximately 20% fine grained sand, 80% clayey fines of low plasticity, red brown, moist, some secondary calcite.
25									

END OF BORING @ 21.5'

Fill to 2'
No groundwater
No bedrock

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1,421
		DATE DRILLED:	April 21, 2022
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-6

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction characteristics, direct shear, sieve analysis, sand equivalent, R-value, expansion index, swell, 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 2921 and ASTM D 2216, respectively, and the results are shown on the boring logs, Enclosures B-1 through B-6 for convenient correlation with the soil profile.

Laboratory Compaction

A selected soil sample was tested in the laboratory to determine compaction characteristics using the ASTM D 1557 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-1	0-3	(SC) Clayey Sand	127.0	10.0

Direct Shear Test

Shear tests are performed in general accordance with ASTM D 3080 with a direct shear machine at a constant rate-of-strain (0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in remolded condition (90 percent relative compaction per ASTM D 1557) and soaked, to represent the worse case conditions expected in the field.

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

DIRECT SHEAR TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Apparent Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-1	0-3	(SC) Clayey Sand	250	28

Sieve Analysis

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

Sand Equivalent

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

R-Value Test

A 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 results of the R-value test is presented on Enclosure C-1.

Expansion Index Test

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

EXPANSION INDEX TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-3	(SC) Clayey Sand	15	Very Low
Expansion Index:	0-20 Very low	21-50 Low	51-90 Medium	91-130 High

One-Dimensional Swell Test

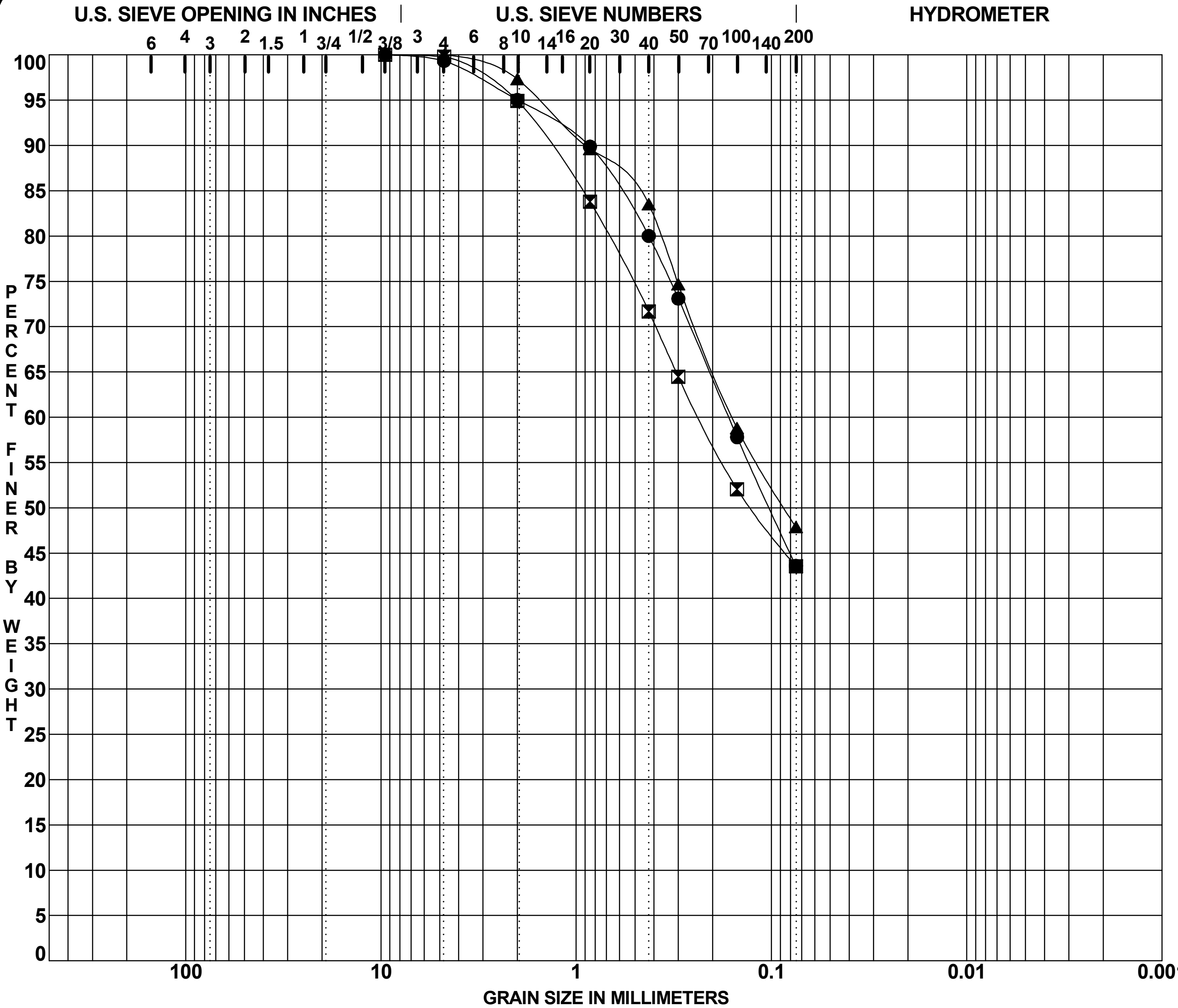
The apparatus used for the one-dimensional swell or settlement potential (odometer) is designed to test a one-inch high portion of the undisturbed soil sample as contained in a sample ring. Porous stones and filler paper are placed in contact with the top and bottom of the specimen to permit the addition or release of water. A load of 500 psf was applied to the test specimen to initiate its insitu condition, and the resulting axial deformations are recorded. The results of the one-dimensional swell or settlement potential is presented in the following table:

ONE-DIMENSIONAL SWELL TEST			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**% Swell**
B-4	5-6	(CL) Lean Clay with Sand	1.6

Soluble Sulfate Content Test

The soluble sulfate content of a selected subgrade soil was evaluated. The concentration of soluble sulfates in the soil 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 sample. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test result is presented in the following table:

SOLUBLE SULFATE CONTENT TEST			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Sulfate Content (% by weight)**
B-1	0-3	(SC) Clayey Sand	< 0.005
B-3	0-3	(SM) Silty Sand	0.008
B-5	0-3	(SC) Clayey Sand	0.015



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-1*	*@ 0-3'*	*(SC) Clayey Sand*	*15*	*--*				
⧗	*B-3*	*@ 0-3'*	*(SM) Silty Sand*	*15*	*--*				
▲	*B-5*	*@ 0-3'*	*(SC) Clayey Sand*	*13*	*26*				

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*@ 0-3'*	*9.50*	*0.17*			*0.7*	*55.7*	*43.6*	
⧗	*B-3*	*@ 0-3'*	*9.50*	*0.23*			*0.2*	*56.3*	*43.5*	
▲	*B-5*	*@ 0-3'*	*4.75*	*0.16*			*0.0*	*52.1*	*47.9*	

PROJECT:	Proposed Industrial Development	**PROJECT NO.:**	23798.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**DATE:**	May 2022

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-1

APPENDIX D

Seismic Design Spectra

Project: APN 331-060-018
Project Number: 23798.1
Client: Compass Danbe Real Estate Partners II, LLC
Site Lat/Long: 33.74198 / -117.19665
Controlling Seismic Source: Elsinore / San Jacinto

REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.3(ii)	F_v	2.5
$0.2*(S_{D1}/S_{DS})$	T_0	0.132
S_{D1}/S_{DS}	T_S	0.658
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.6196*
Equation 11.4-2 - F_V*S_1	S_{M1}	0.9294*

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.8
Design Maps	S_s	1.412
Design Maps	S_1	0.523
Equation 11.4-1 - F_A*S_S	S_{MS}	1.412*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	0.941*
Design Maps	PGA	0.5
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.55*
Section 21.5.3	80% of PGA_M	0.440
Design Maps	C_{RS}	0.937
Design Maps	C_{R1}	0.92

RISK COEFFICIENT

REFERENCE	NOTATION	VALUE
Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.937
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	0.92

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

Period	Cr
0.200	0.937
0.300	0.935
0.400	0.933
0.500	0.931
0.600	0.929
0.680	0.927
1.000	0.92

* **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: 23798.1

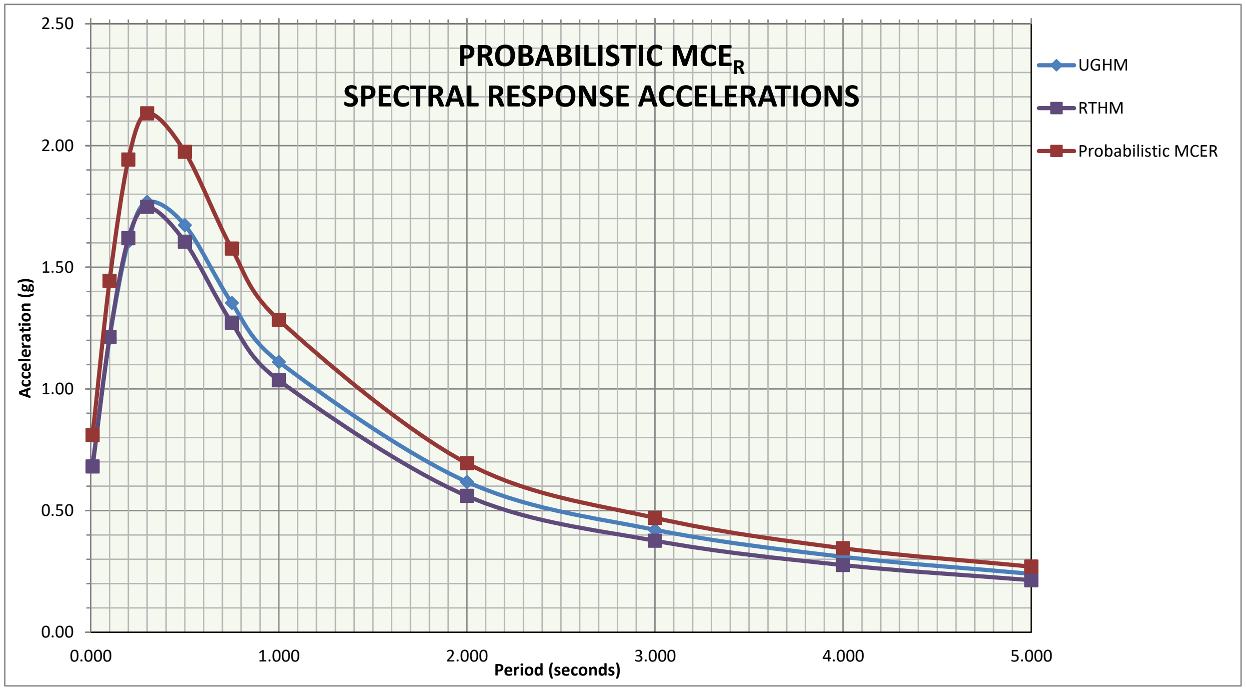
Period	UGHM	RTHM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.690	0.681	1.19	0.810
0.100	1.207	1.213	1.19	1.443
0.200	1.605	1.619	1.20	1.943
0.300	1.768	1.748	1.22	2.133
0.500	1.673	1.605	1.23	1.974
0.750	1.353	1.271	1.24	1.576
1.000	1.111	1.035	1.24	1.283
2.000	0.618	0.560	1.24	0.694
3.000	0.421	0.376	1.25	0.470
4.000	0.310	0.276	1.25	0.345
5.000	0.240	0.214	1.26	0.270

Probabilistic PGA: 0.690

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: Elsinore / San Jacinto

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

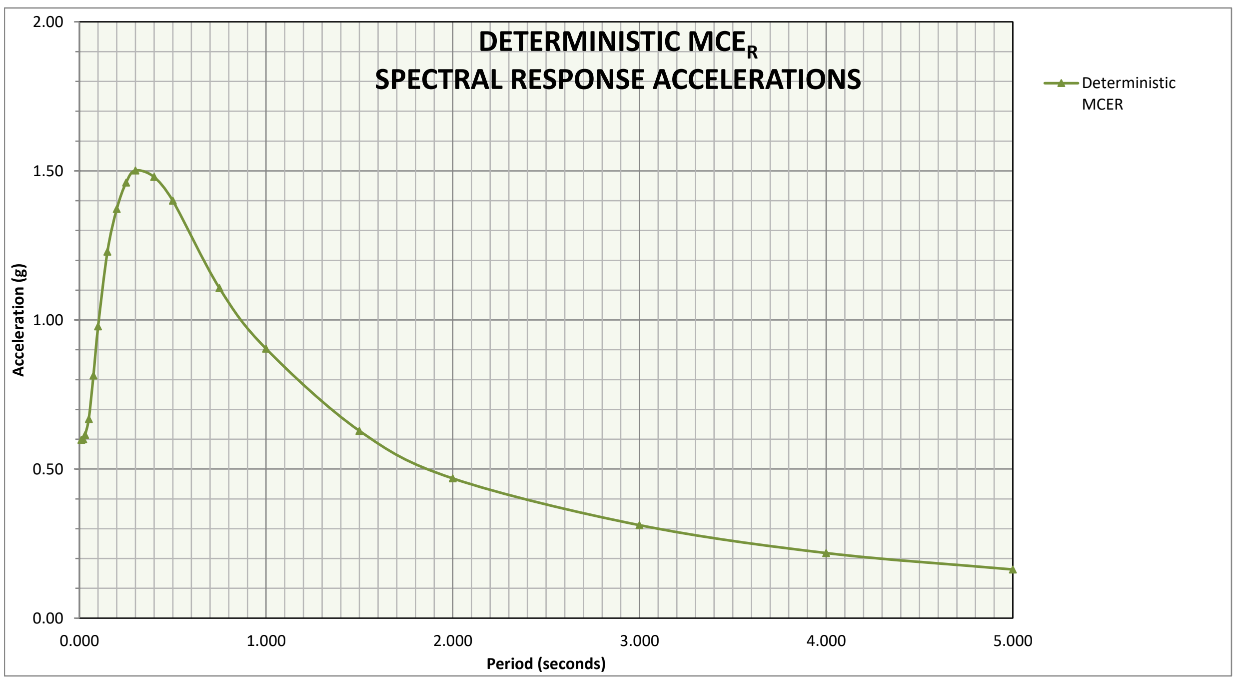
Project No: 23798.1

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	0.502	1.19	0.598	0.598
0.020	0.504	1.19	0.600	0.600
0.030	0.516	1.19	0.615	0.615
0.050	0.561	1.19	0.668	0.668
0.075	0.684	1.19	0.814	0.814
0.100	0.822	1.19	0.978	0.978
0.150	1.024	1.20	1.229	1.229
0.200	1.143	1.20	1.372	1.372
0.250	1.207	1.21	1.461	1.461
0.300	1.231	1.22	1.501	1.501
0.400	1.203	1.23	1.480	1.480
0.500	1.138	1.23	1.400	1.400
0.750	0.893	1.24	1.107	1.107
1.000	0.729	1.24	0.903	0.903
1.500	0.507	1.24	0.628	0.628
2.000	0.378	1.24	0.469	0.469
3.000	0.249	1.25	0.312	0.312
4.000	0.175	1.25	0.218	0.218
5.000	0.129	1.26	0.163	0.163

Is Deterministic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	~~0.502~~
Is Deterministic PGA >=F_{PGA}*0.5?	NO
Deterministic PGA:	0.550

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

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



SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.810	0.598	0.598	0.399
0.100	1.443	0.978	0.978	0.652
0.200	1.943	1.372	1.372	0.915
0.300	2.133	1.501	1.501	1.001
0.500	1.974	1.400	1.400	0.933
0.750	1.576	1.107	1.107	0.738
1.000	1.283	0.903	0.903	0.602
2.000	0.694	0.469	0.469	0.312
3.000	0.470	0.312	0.312	0.208
4.000	0.345	0.218	0.218	0.146
5.000	0.270	0.163	0.163	0.109

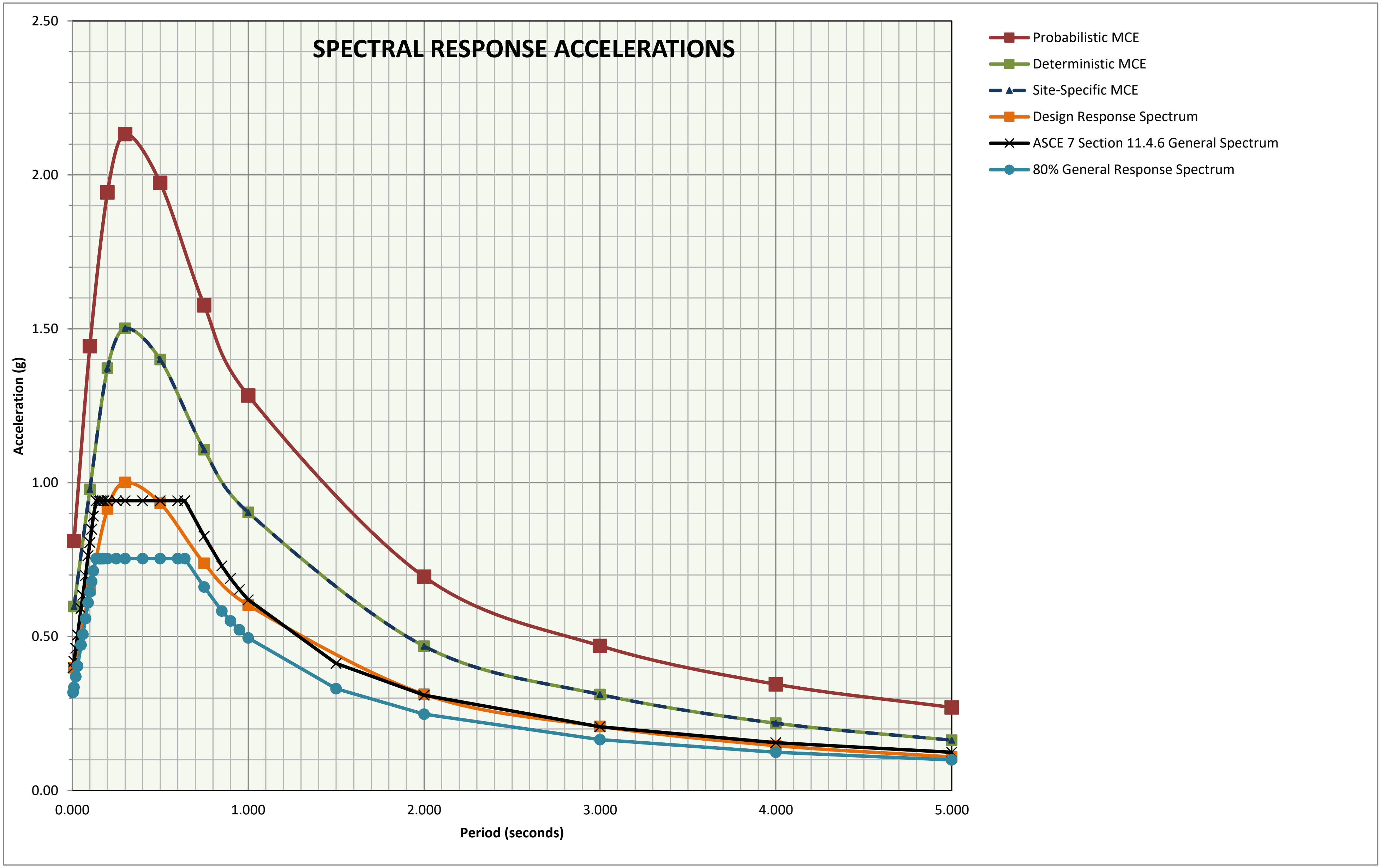
ASCE 7-16: Section 21.4
Site Specific

	Calculated Value	Design Value
SDS:	0.901	0.901
SD1:	0.625	0.625
SMS:	1.351	1.351
SM1:	0.937	0.937
Site Specific PGAm:	0.550	0.550
Site Class:	D measured	

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

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.398	0.318
0.010	0.419	0.336
0.020	0.462	0.370
0.030	0.505	0.404
0.050	0.591	0.473
0.060	0.634	0.507
0.075	0.698	0.559
0.090	0.763	0.610
0.100	0.806	0.644
0.110	0.848	0.679
0.120	0.891	0.713
0.136	0.941	0.753
0.150	0.941	0.753
0.160	0.941	0.753
0.170	0.941	0.753
0.180	0.941	0.753
0.200	0.941	0.753
0.250	0.941	0.753
0.300	0.941	0.753
0.400	0.941	0.753
0.500	0.941	0.753
0.600	0.941	0.753
0.640	0.941	0.753
0.750	0.826	0.661
0.850	0.729	0.583
0.900	0.688	0.551
0.950	0.652	0.522
1.000	0.620	0.496
1.500	0.413	0.330
2.000	0.310	0.248
3.000	0.207	0.165
4.000	0.155	0.124
5.000	0.124	0.099

Project No: 23798.1



Project No: 23798.1

LOR GEOTECHNICAL GROUP, INC.

APPENDIX E

Infiltration Test Results

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN 331-060-018	Test Date:	April 22, 2022
Project No.:	23798.1	Test Hole No.:	P-1
Soil Classificaiton:	(SM) Silty sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	8.0 ft.	Date Excavated:	April 21, 2022
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	9:53 AM	10:23 AM	30	0.50	0.50	42.00	42.25	96.00	96.00	0.25	53.88	120.0
2	10:23 AM	10:53 AM	30	0.50	1.00	42.25	42.50	96.00	96.00	0.25	53.63	120.0
3	10:53 AM	11:23 AM	30	0.50	1.50	42.50	43.00	96.00	96.00	0.50	53.25	60.0
4	11:23 AM	11:53 AM	30	0.50	2.00	43.00	43.50	96.00	96.00	0.50	52.75	60.0
5	11:53 AM	12:23 PM	30	0.50	2.50	43.50	44.00	96.00	96.00	0.50	52.25	60.0
6	12:23 PM	12:53 PM	30	0.50	3.00	44.00	44.50	96.00	96.00	0.50	51.75	60.0
7	12:53 PM	1:23 PM	30	0.50	3.50	44.50	45.00	96.00	96.00	0.50	51.25	60.0
8	1:23 PM	1:53 PM	30	0.50	4.00	45.00	45.50	96.00	96.00	0.50	50.75	60.0
9	1:53 PM	2:23 PM	30	0.50	4.50	45.50	46.00	96.00	96.00	0.50	50.25	60.0
10	2:23 PM	2:53 PM	30	0.50	5.00	46.00	46.50	96.00	96.00	0.50	49.75	60.0
11	2:53 PM	3:23 PM	30	0.50	5.50	46.50	47.00	96.00	96.00	0.50	49.25	60.0
12	3:23 PM	3:53 PM	30	0.50	6.00	47.00	47.50	96.00	96.00	0.50	48.75	60.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	49.00	
H_f	48.50	
ΔH	0.50	
H_{avg}	48.75	
I_t	**0.02**	in/hr (clear water rate)

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

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	APN 331-060-018	Test Date:	April 21, 2002
Project No.:	23798.1	Test Hole No.:	P-2
Soil Classificaiton:	(SC) Clayey sand	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	10.0 ft.	Date Excavated:	April 21, 2022
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	9:58 AM	10:28 AM	30	0.50	0.50	46.50	50.00	120.00	120.00	3.50	71.75	8.6
2	10:28 AM	10:58 AM	30	0.50	1.00	50.00	53.00	120.00	120.00	3.00	68.50	10.0
3	10:58 AM	11:28 AM	30	0.50	1.50	48.00	51.00	120.00	120.00	3.00	70.50	10.0
4	11:28 AM	11:58 AM	30	0.50	2.00	51.00	54.00	120.00	120.00	3.00	67.50	10.0
5	11:58 AM	12:28 PM	30	0.50	2.50	48.00	50.50	120.00	120.00	2.50	70.75	12.0
6	12:28 PM	12:58 PM	30	0.50	3.00	50.50	53.00	120.00	120.00	2.50	68.25	12.0
7	12:58 PM	1:28 PM	30	0.50	3.50	48.00	51.00	120.00	120.00	3.00	70.50	10.0
8	1:28 PM	1:58 PM	30	0.50	4.00	51.00	53.50	120.00	120.00	2.50	67.75	12.0
9	1:58 PM	2:28 PM	30	0.50	4.50	48.00	51.00	120.00	120.00	3.00	70.50	10.0
10	2:28 PM	2:58 PM	30	0.50	5.00	51.00	53.00	120.00	120.00	2.00	68.00	15.0
11	2:58 PM	3:28 PM	30	0.50	5.50	48.00	50.50	120.00	120.00	2.50	70.75	12.0
12	3:28 PM	3:58 PM	30	0.50	6.00	50.50	53.00	120.00	120.00	2.50	68.25	12.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	69.50	
H_f	67.00	
ΔH	2.50	
H_{avg}	68.25	
I_t	**0.09**	in/hr (clear water rate)

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

APPENDIX F3

WHEAT STREET SITE GEOTECHNICAL INVESTIGATION

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

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED INDUSTRIAL DEVELOPMENT APN 330-180-012 26201 WHEAT STREET MENIFEE, CALIFORNIA

PROJECT NO. 23796.1
FEBRUARY 14, 2022

Prepared For:

Compass Danbe Real Estate Partners II, LLC
523 Main Street
El Segundo, California 90245

Attention: Mr. Mark Bachli

6121 Quail Valley Court ▲ Riverside, CA 92507 ▲ (951) 653-1760 ▲ (951) 653-1741 (Fax) ▲ www.lorgeo.com

February 14, 2022

Compass Danbe Real Estate Partners II, LLC
8151 Auto Drive
Riverside, California 92504

Project No. 23796.1

Attention: Mr. Mark Bachli

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Industrial Development, APN 330-180-012, 26201 Wheat Street, Menifee, California.

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

To provide adequate support for the proposed structures, we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. Any undocumented fill material and any loose older alluvial materials should be removed from structural areas and areas to receive engineered compacted fills. The data developed during this investigation indicates that removals on the order of approximately 5 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.

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

Non-conducive infiltration rates were obtained for the soils tested.

LOR Geotechnical Group, Inc.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

INTRODUCTION

During January and February of 2022, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed industrial development of Assessor's Parcel Number (APN) 330-180-012, 26201 Wheat Street, Menifee, California. The purpose of this investigation was to provide a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed development. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photographs of the site and surrounding regions dated 1966 through 2021;
- Geologic field reconnaissance mapping to verify the aerial distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Percolation testing via the borehole test method;
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings, and providing conclusions and recommendations for site development.

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

PROJECT CONSIDERATIONS

To orient our investigation at the site, an Alta Survey prepared by J.D. Cole and Associates, Inc., undated, was furnished for our use. The existing site conditions and proposed building configurations, associated driveways, parking, and landscape areas were indicated on this plan. This plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A. As noted on the Site Plan, development of the site will include an industrial type structure comprising 82,272± square foot building with ten to eleven (10 to 11) dock doors, with the remainder of the property to be used for driveways, parking, and landscape areas. The building is anticipated to be of concrete, masonry, or similar type construction and light to moderate foundation loads are anticipated with these structures.

Infiltration is proposed via underground chamber type systems. Depths and locations were provided by CASC Engineering and Consulting.

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

AERIAL PHOTO ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photograph images of varying scales. We reviewed imagery available from Google Earth Pro (2022) computer software and from online Historic Aerials (2022).

To summarize briefly, the site was vacant land utilized for dry land farming since 1966, the earliest photograph available, until the 1997 photograph. The 2002 photograph shows the site developed with a residence and outbuildings, very similar to that seen today. No evidence for the presence of faults traversing the site area or mass movement features was noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The approximate 5-acre site is located within the northwestern portion of the city of Menifee, California. It consists of a residence and three outbuildings within the center and the remainder vacant land. A water well is present near the residence. Details regarding the depth of the well and the depth to water are not known. Several large trees are present near the structures and along the western site boundary. The property is situated along the west side of Wheat Street, an unimproved roadway. Topographically, the site is planar with a gentle fall to the north-northwest.

Large lot residential properties lie south and east of the site. North and east of the site, the properties are vacant.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on January 20, 2022. The work consisted of advancing a total of 5 exploratory borings using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. The approximate locations of our exploratory borings are presented on Enclosure A-2, within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by a geologist from this firm. The borings were drilled to maximum depths of 15.5 to 41 feet below the existing ground surface. Relatively undisturbed and bulk samples were obtained at a maximum depth interval of 5 feet, and returned to our geotechnical laboratory in sealed containers for further testing and evaluation.

A detailed description of the subsurface field exploration program and the boring logs is presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

As shown on Enclosure A-1, within Appendix A, the site is located within the United States Geological Survey Romoland 7.5 minute quadrangle topographic map. This region lies along the north-central portion of the Perris block of the northern Peninsular Ranges geologic province of southern California. While the Perris block is considered to be a relatively stable structural block, it is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the east, and the Cucamonga fault zone on the north. The Perris block is underlain by rocks of the Peninsular Ranges batholith, a very large mass of crystalline igneous rocks of Cretaceous age and with no known floor, and by prebatholithic metasedimentary and metavolcanic rocks of older ages.

The Perris block has a series of erosional surfaces, marked by low topographic relief and capped with unconsolidated alluvial sediments stripped from the surrounding highlands. This area was mapped by the California Division of Mines and Geology as being underlain by deposits of old alluvial fan deposits (Morton, 2003).

The interior of the Perris Plain is considered to be relatively stable with few known active faults. However, this plain is bounded by active faults. These include the Elsinore fault zone on the west, the San Jacinto fault zone on the northeast, the San Andreas fault zone on the north, and the Agua-Tibia fault zone on the south. As the subject site is located near the western margin of the Perris Plain, the Elsinore fault is the closest known active fault in relation to the site. At its closest approach, the Elsinore fault is located approximately 13.0 kilometers (8.1 miles) southwest from the site. A complete listing of the distances to known active faults in relation to the various planning areas is given in the Faulting section of this report.

The site is shown within the regional geologic setting as mapped by the U.S.G.S. on the enclosed Regional Geologic Map, Enclosure A-3, within Appendix A.

Site Geologic Conditions

Fill/Topsoil: Fill/topsoil materials were encountered within our exploratory borings to depths of approximately 1 foot. The fill/topsoil materials are believed to be associated with current and past weed abatement (discing) practices at the site. As encountered, the fill/topsoil materials were comprised of silty sand which were predominantly brown to red-brown, dry, and in a loose state. Locally, deeper fills are anticipated to be present and primarily associated with the existing improvements.

Older Alluvium: Older alluvial materials were encountered underlying the fill materials described above within all of our exploratory borings. The older alluvial soils encountered were a maximum of approximately 9 feet in thickness and rest upon bedrock materials. These units were noted to mainly consist of sandy silt, silty sand, and minor units of lean clay with sand. The older alluvial materials were in a relatively medium /medium dense to very stiff/very dense state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Consolidation testing of a relatively low density, low blow count sample indicates normal consolidation characteristics. Expansion index testing indicates that these materials will have a very low to nearly low expansion potential when used as compacted fill.

Bedrock: Igneous bedrock materials were encountered within all of our exploratory borings at depths of approximately 6.5 to 10 feet. The igneous bedrock was typically coarse grained, highly to moderately weathered upon first encounter becoming less weathered with depth, dry to damp, and in a hard to very hard state based on our equivalent Standard Penetration Test (SPT) data and in-place density testing.

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

Groundwater Hydrology

Groundwater was not encountered within any of our exploratory borings as advanced to a maximum depth of approximately 41 feet below the existing ground surface nor was any groundwater seepage observed during our site reconnaissance.

In order to estimate the approximate depth to groundwater in the site area, a search was conducted for local groundwater (well) level measurements within the Cooperative Well Measuring Program, Spring 2021 (Watermaster Support Services et al., 2021). This database contains depth to groundwater measurements dating back to 1993. We also conducted a search of the water well database information provided in the California Department of Water Resources (CDWR) Water Library Data website (CDWR, 2021).

The only database with nearby well records was the CDWR database. One well, State Well No. 05S03W17A001S, located approximately 1 kilometer (0.62 miles) to the northeast was identified. Data for this well was limited to one reading in 1995. A measuring point elevation of 1,424± feet above mean sea level was reported. The depth provided was 22 feet (elevation of 1,402± feet above mean sea level).

As noted on Enclosure A-2, the lowest elevation of the site is 1,440 feet above mean sea level. Based on the information above, groundwater in the region appears to be at depths on the order of 40 feet below the ground surface. Groundwater may seep into the bedrock beneath the site region along fractures and joints within the bedrock, the presence of hard bedrock beneath the site generally precludes the development of groundwater conditions or a groundwater table in these areas. Any groundwater that might be encountered during site development would likely be the result of infiltration of surface waters/irrigation waters traveling downward into the bedrock along these joints and fractures.

Mass Movement

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

Faulting

No active or potentially active faults are known to exist at the subject site. In addition, the subject site does not lie within a current State of California Earthquake Fault Zone (Hart and Bryant, 2003) nor does the site lie within a County of Riverside fault zone (CRTLMA, 2021). No evidence of faulting projecting into or crossing the site was noted during our aerial photograph review or our review of published geologic maps.

As previously mentioned, the closest known active earthquake fault with a documented location is the Elsinore fault located approximately 13.0 kilometers (8.1 miles) to the southwest. In addition, other relatively close active faults include the San Jacinto fault located approximately 18.7 kilometers (11.6 miles) to the northeast, and the San Andreas fault located approximately 40.6 kilometers (25.2 miles) to the northeast.

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

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

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

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 closer distance and larger 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. (2021). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from January 1, 1932 through February 8, 2022.

In our first search, the general seismicity of the region was analyzed by selecting an epicenter map listing all events of magnitude 4.0 and greater, recorded since 1932, within a 100 kilometer (62 mile) radius of the site, in accordance with guidelines of the California Division of Mines and Geology. This map illustrates the regional seismic history of moderate to large events. As depicted on Enclosure A-4, within Appendix A, the site lies within a relatively active region associated with the San Jacinto fault to the northeast.

In the second search, the micro seismicity of the area lying within a 15 kilometer (9.3 miles) 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 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 time period for the events on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid to late1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the Elsinore fault zone to the southwest and the San Jacinto fault zone to the northeast 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 in the region around the subject site. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

Other secondary seismic hazards generally associated with severe ground shaking during an earthquake include liquefaction, seismic-induced settlement, seiches and tsunamis, earthquake induced flooding, landsliding, and rockfalls.

Liquefaction: The site lies within an area mapped by the County of Riverside has having a very low potential for liquefaction (CRTLMA, 2021). The potential for liquefaction generally occurs during strong ground shaking within granular loose sediments where the groundwater is usually less than 50 feet below the ground surface. As found during this investigation, the site is underlain by relatively shallow igneous bedrock in the upper 50 feet, therefore, the possibility of liquefaction at the site is considered nil.

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

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

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

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

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

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 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 current investigation, mapping by others, and our experience in the site region indicates that the materials beneath the site are considered Site Class D stiff soil.

CBC Earthquake Design Summary

Earthquake design criteria have been formulated in accordance with the 2019 CBC and ASCE 7-16 for the site based on the results of our investigation to determine the Site Class and an assumed Risk Category II. However, these values should be reviewed and the final design should be performed by a qualified structural engineer familiar with the region. In addition, the building official should confirm the Risk Category utilized in our design (Risk Category II). Our design values are provided in Appendix D.

PERCOLATION TESTING AND TEST RESULTS

Two borehole percolation tests were conducted in general accordance with the Shallow Percolation Test procedure as outlined in the Design Handbook for Low Impact Development Best Management Practices (CRFCWCD, 2011). The requested locations of our test are illustrated on Enclosure A-2. Test borings were drilled to depths of approximately 8 feet below the existing ground surface as requested, on January 20, 2022.

Subsequent to drilling, a 3-inch diameter, perforated PVC pipe wrapped in filter fabric was placed within each test hole and 3/4-inch gravel was placed between the outside of the pipe and the hole wall. Test holes were pre-soaked the same day as drilling. Testing took place the next day, January 21, 2022, within 26 hours but not before 15 hours, of the pre-soak. The holes were filled with a variable height column of water using water from a 200 gallon water tank. Test periods consisted of allowing the water to drop in 30-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 12 readings were recorded.

Infiltration test results are summarized in the following table:

Test No.	Depth*	Clear Water Infiltration Rate** (in/hr)
P-1	8	0.05
P-2	8	0.21

* depth measured below existing ground surface
** Porchet Method determined rate with an effective diameter due to loss in volume of water due to gravel packing.

The results of this testing are presented as Enclosures E-1 and E-2 in Appendix E. The test results indicate variable infiltration characteristics for the materials tested.

CONCLUSIONS

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

It should be noted that the subsurface conditions encountered in our exploratory borings are indicative of the locations explored and the subsurface conditions may vary. If conditions are encountered during the construction of the project that differ significantly from those presented in this report, this firm should be notified immediately so we may assess the impact to the recommendations provided.

Rippability of Bedrock Units

The rippability of the bedrock units at the subject site was estimated based on the relative ease, or lack of, excavation during our boring exploration. The bedrock units which underlie the site are anticipated to be rippable by conventional earthmoving equipment down to the depths explored. Excavations deeper than this may require specialized methods, such as D8R or larger dozer using a multi or single shank ripper. It is also anticipated that some larger non-rippable rock "floaters" may be encountered. These may require special handling. Excavations in these materials may require specialized methods.

If a more precise estimation of the rippability of the bedrock units is required, a seismic refraction investigation should be conducted at the site. Such a study should involve the measuring of the seismic velocities of the underlying bedrock units, as they increase with depth, then comparing these to estimates of velocities verses ease of excavation charts.

In summary, the most important consideration for the proposed grading should include selecting an experienced, well-qualified contractor. The success to excavating the bedrock materials at the site will require the contractor to have knowledge of the appropriate ripper-equipment selection (i.e., down pressure available at the tip, tractor flywheel horsepower, tractor gross weight, etc.), ripping techniques (i.e., single- or multi-shank teeth, pass spacing, tandem pushing, etc.). It should also be noted that while in some areas where deeper cuts may be possible with standardized earthmoving equipment, specialized methods may increase the speed of the excavations at the site.

Foundation Support

To provide adequate support for the proposed structure we recommend that a compacted fill mat be constructed beneath footings and slabs. The compacted fill mat will provide a dense, high-strength soil layer to uniformly distribute the anticipated foundation loads over the underlying soils. The construction of this compacted fill mat will allow for the removal of the existing fill material which was loose and any current subsurface improvements, such as utilities, foundations, etc., that may be present locally.

Conventional foundation systems utilizing either individual spread footings and/or continuous wall footings will provide adequate support for the anticipated downward and lateral loads when utilized in conjunction with the recommended fill mat.

Soil Expansiveness

Our expansion index testing of a representative sample of the on-site soils indicates a very low to near low expansion potential. For low expansive soils, specialized foundation design and construction procedures to resist expansive soil activity are necessary and provided in the following sections of this report.

Careful evaluation of onsite 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.

Infiltration

The results of our field investigation and percolation test data indicates the site soils at the depths tested are not conducive to infiltration. Based on the results of this investigation, infiltration is also not anticipated to occur at other depths due to the amount of silty/clayey fines and dense to very dense nature of the soils and hard to very hard nature of the bedrock.

Geologic Mitigations

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

Seismicity

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

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

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

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

RECOMMENDATIONS

Geologic Recommendations

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

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An onsite, pre-job meeting with the developer, the contractor, the jurisdictional agency, and the geotechnical engineer should occur prior to all grading related operations.

Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed in accordance with the following recommendations as well as applicable portions of the California Building Code, and/or applicable local ordinances.

All areas to be graded should be stripped of significant vegetation and other deleterious materials.

Any undocumented fill encountered during grading should be completely removed, cleaned of significant deleterious materials, and may be reused as compacted fill. It is our recommendation that any existing fills under any proposed flatwork and paved areas be removed and replaced with engineered compacted fill. If this is not done, premature structural distress (settlement) of the flatwork and pavement may occur.

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

The existing fill/topsoil material, as well as any loose older alluvial soils and any loose bedrock, if encountered, should be removed from all proposed structural and/or fill areas. The data developed during this investigation indicates that removals on the order of 5 feet deep will be required from proposed development areas in order to encounter competent older alluvium upon which engineered compacted fill can be placed. The given removal depths are preliminary. Deeper fills may be present, primarily in areas of current improvements. Removals should expose older alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557). The actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

Preparation of Fill Areas

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

Engineered Compacted Fill

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

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

Preparation of Foundation Areas

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

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

Short-Term Excavations

Following the California Occupational and Safety Health Act (CAL-OSHA) requirements, excavations 5 feet deep and greater should be sloped or shored. All excavations and shoring should conform to CAL-OSHA requirements. Short-term excavations of 5 feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547. Based upon the findings from our exploratory borings, it appears that Type C soils are the predominant type of soil on the project and all short-term excavations should be based on this type of soil.

Deviation from the standard short-term slopes are permitted using option 4, Design by a Registered Professional Engineer (Section 1541.1).

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

Slope Construction

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

Slope Protection

Since the site soil materials are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after 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.

Soil Expansiveness

The upper materials encountered during this investigation were tested and found to have a very low to near low expansion potential. Therefore, specialized foundation design and construction procedures to specifically resist expansive soil activity are anticipated at this time and are provided within.

Additional evaluation of on-site and any imported soils for their expansion potential should be conducted following completion of the grading operation.

Foundation Design

Due to near low expansive soil conditions, we recommend that all structures be supported on reinforced, stiffened mat foundations resting over 24 inches of engineered compacted fill placed over competent native earth materials.

The design of the structural slab foundation should be performed in conformance to the Wire Reinforcement Institute (WRI) method or the Post-Tensioning Institute (PTI) method. For the application of the WRI method, a minimum effective plasticity index of 15 is recommended for foundation design. The slab thickness should be a minimum of 5 inches and should have a reinforcement of at least Asfy equal to 3,300 pounds. This could consist of #3 reinforcing bars of 60-grade steel placed at a maximum spacing of 18 inches on center, each way or equivalent. Prior to placing concrete slabs, the upper 12 inches of the subgrade soil should be pre-saturated to 2 to 4 percent over optimum moisture content.

These reinforcement, depth, and spacing recommendations should be considered minimum. The actual requirements for slab-on-grade foundations design and construction should be provided by a structural engineer experienced in these matters.

These conditions should be verified during the site grading by additional evaluation of on-site and any imported soils for their expansion potential and plasticity characteristics.

If slab-on-grade foundations per the PTI method are proposed, the following geotechnical parameters should be used for design:

- Edge Moisture Variation Distance, em:
  - Center Lift Loading Conditions: 9.0 ft
  - Edge Lift Loading Conditions: 8.2 ft
- Differential Swell, ym:
  - Center Lift 3.0 in
  - Edge Lift 6.0 in
- Subgrade Soil Friction Coefficient, μ: 0.30

The above design parameters are based upon the data collected during our site investigation and are in accordance with Design of Post-Tensioned Slabs-on-Ground, third edition, published by the Post-Tensioning Institute (2008).

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure may be increased by 200 psf for each additional foot of width, and by 500 psf for each additional foot of depth, up to a maximum of 4,000 psf.

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 over turning should not exceed the increased allowable pressure. Buildings should be setback from slopes in accordance with the California Building Code.

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 280 pounds per square foot per foot of depth. Base friction may be computed at 0.28 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values are for dead load plus live load and may be increased by one-third for wind or seismic loading.

Settlement

Total settlement of individual foundations will vary depending on the width of the foundation and the actual load supported. Maximum settlement of shallow foundations designed and constructed in accordance with the preceding recommendations are estimated to be on the order of 0.5 inch. Differential settlements between adjacent footings should be about one-half of the total settlement. Settlement of all foundations is expected to occur rapidly, primarily as a result of elastic compression of supporting soils as the loads are applied, and should be essentially completed shortly after initial application of the loads.

Building Area Slab-on-Grade

To provide adequate support, concrete floor slabs-on-grade should bear on a minimum of 24 inches of engineered fill compacted soil placed and maintained at 2 to 4 percent above optimum moisture content. The final pad surfaces should be rolled to provide smooth, dense surfaces. Concrete slabs-on-grade should be a minimum of 5 inches in thickness with No. 3 bars spaced 12 inches on center each way.

The actual requirements for slab-on-grade design and construction details should be provided by a structural engineer experienced in these matters. These conditions should be verified during the site grading by additional evaluation of on-site and any imported soils for their expansion potential and plasticity characteristics.

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

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

The slabs should be protected from rapid and excessive moisture loss which could result in slab curling. Careful attention should be given to slab curing procedures, as the site area is subject to large temperature extremes, humidity, and strong winds.

Exterior Flatwork

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

To resist expansive soil forces, flatwork supported by low expansive soils should be reinforced with a minimum of # 3 rebar at 18 inches each way. Flatwork areas should be pre-saturated to 2 to 4 percent over optimum moisture content to a minimum depth of 12 inches prior to placing concrete.

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

Wall Pressures

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

For design of retaining walls unrestrained against movement at the top, we recommend an active pressure of 51 pounds per square foot (psf) per foot of depth be used.

This assumes level backfill consisting of compacted, non-expansive, 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. Non-expansive import soils may be required. 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 0.47 times the surcharge load. Any isolated or line loads from adjacent foundations or vehicular loading will impose additional wall loads and should be considered individually.

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

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

Preliminary Pavement Design

Testing and design for preliminary onsite pavement was conducted in accordance with the California Highway Design Manual. Based upon our preliminary sampling and testing, and upon an assumed Traffic Index generally used for similar projects, it appears that the structural sections tabulated below should provide satisfactory pavements for the subject on-site pavement improvements:

AREA	T.I.	DESIGN R-VALUE	PRELIMINARY SECTION
On site vehicular parking with occasional truck traffic (ADTT=10)	6.0	10	0.25’ AC / 1.05' AB or 5" JPCP / 6" AB
Light to moderate truck traffic (ADTT=25)	7.0	10	0.30'AC / 1.25'AB or 6" JPCP / 6" AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base JPCP - Jointed Plain Concrete Pavement with MR $\geq$ 600 psi			

The above structural sections are predicated upon 90 percent relative compaction (ASTM D 1557) of all utility trench backfills and 95 percent relative compaction (ASTM D 1557) of the upper 12 inches of pavement subgrade soils and of any aggregate base utilized. In addition, the aggregate base should meet Caltrans specifications for Class 2 Aggregate Base.

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 5 inch thick concrete, with a 6 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 concrete pavement sections should have a minimum modulus of rupture (MR) of 600 pounds per square inch (psi). Transverse joints should be sawcut in the pavement at approximately 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other.

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

Infiltration

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

Construction Monitoring

Post investigative services are an important and necessary continuation of this investigation. Project plans and specifications should be reviewed by the project geotechnical consultant prior to construction to confirm that the intent of the recommendations presented in this report have been incorporated into the design.

Additional R-value, expansion, and soluble sulfate content testing may be needed after/during site rough grading.

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

1. Site preparation-stripping and removals.

2. Excavations, including approval of the bottom of excavations prior to the processing and preparation of the bottom areas for fill placement.

3. Scarifying and recompacting prior to fill placement.

4. 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

5. Foundation excavations.

6. Subgrade preparation for pavements and slabs-on-grade. This includes pre-saturation testing of slab-on-grade and flatwork areas to verify moisture content.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Compass Danbe Real Estate Partners II, LLC and their design consultants for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

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

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

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

TIME LIMITATIONS

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

CLOSURE

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

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

Robert M. Markoff, CEG
Engineering Geologist

John P. Leuer, GE 2030
President

REGISTERED PROFESSIONAL ENGINEER
JOHN P. LEUER
NO. 2030
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
ROBERT M. MARKOFF
No. 2073
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

AAT:RMM:JPL:ss

Distribution:	Addressee (2) and PDF via email mbachli@danbe.com
cc:	Vicky Valenzuela via email vicky@cdrepartners.com

REFERENCES

American Society of Civil Engineers, 2016, Minimum Design Load for Buildings and Other Structures, ASCE 7-16.

California Building Standards Commission and International Conference of Building Officials, 2019, California Building Code, 2019 Edition.

California Department of Water Resources, 2022, Online Water Data Library (WDL), https://wdl.water.ca.gov/waterdatalibrary/Map.aspx, accessed February 2022.

County of Riverside, Flood Control and Water Conservation District (CRFCWCD), 2011, Design Handbook for Low Impact Development Best Management Practices, dated September 2011.

County of Riverside, Transportation and Land Management Agency (CRTLMA), 2022, Geographic Information System, http://www3.tlma.co.riverside.ca.us, accessed February 2022.

Google Earth, 2021, Imagery from various years, www.google.com/earth.

Hart, E.W. and W.A. Bryant, 2010, Fault-Rupture Hazard Zones in California, California Dept. of Conservation Division of Mines and Geology Special Publication 42.

Historic Aerials (Nationwide Environmental Title Research, LLC), 2022, Imagery from Various Years, https://www.historicaerials.com/, accessed February 2022.

Larson, R., and Slosson, J., 1992, The Role of Seismic Hazard Evaluation in Engineering Reports, in Engineering Geology Practice in Southern California, AEG Special Publication Number 4, pp 191-194.

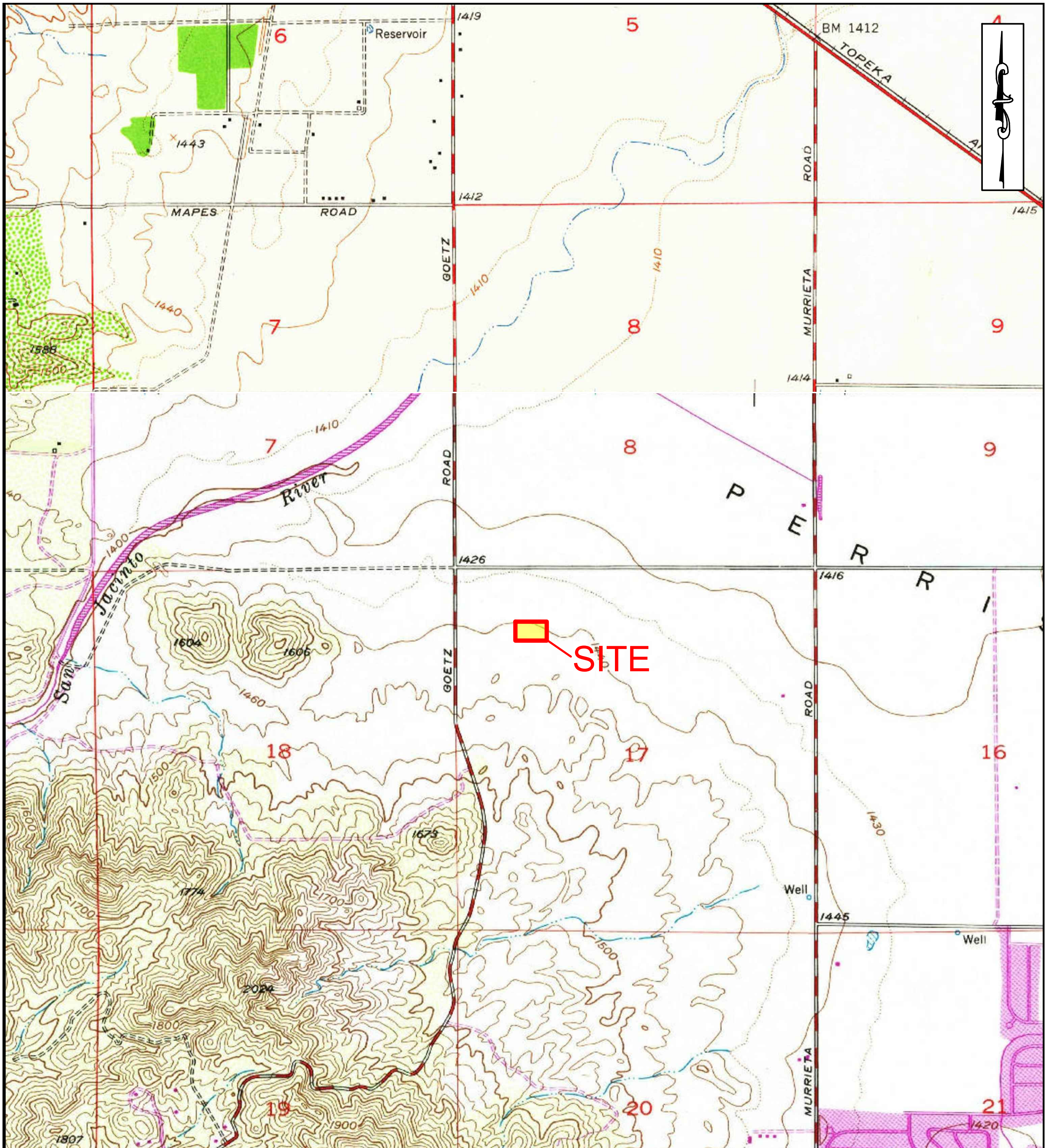
Morton, D.M., 2003, Preliminary Geologic Map of the Romoland 7.5' Quadrangle, Riverside County, California, U.S.G.S. Open File Report 03-102.

USGS, 2021, https://earthquake.usgs.gov/earthquakes/map/, accessed February 2022.

Watermaster Support Services, Western Municipal Water District, and San Bernardino Valley Water Conservation District, 2021, Cooperative Well Measuring Program, Spring 2021, Covering the Upper Santa Ana River Watershed, San Jacinto Watershed, and Santa Margarita Watershed, July 1, 2021.

APPENDIX A

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



INDEX MAP

PROJECT:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'

ALTA SURVEY

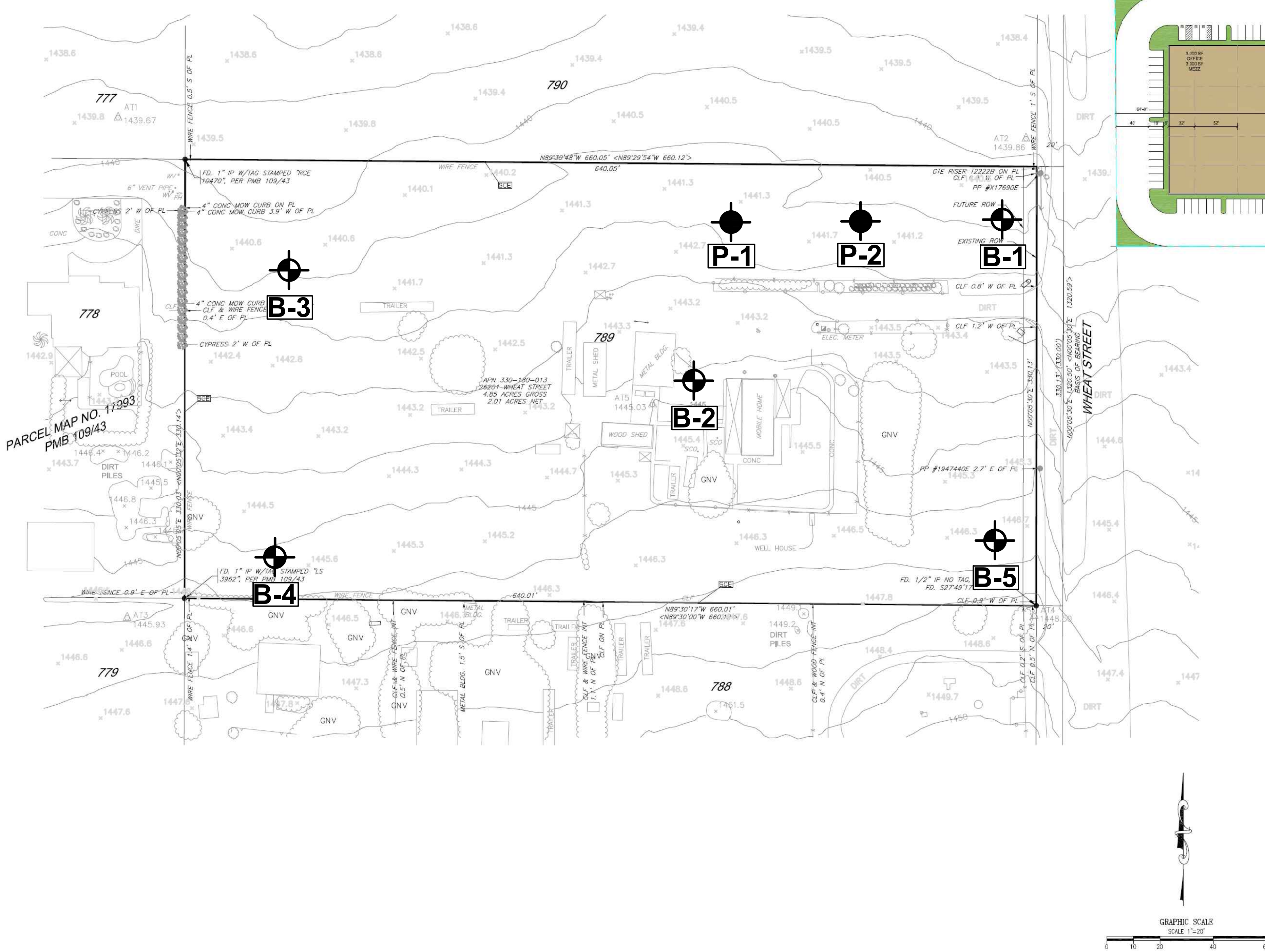
IN THE CITY OF MENIFEE, COUNTY OF RIVERSIDE, STATE OF CALIFORNIA

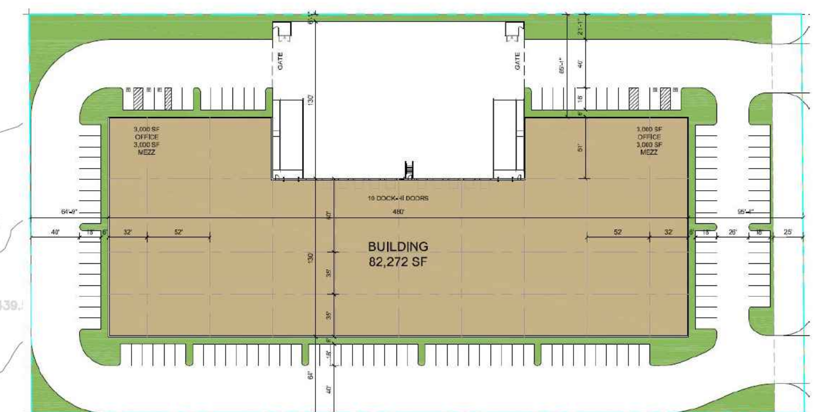
Legend

(Locations Approximate)

Map Symbols

- **B-5** - Exploratory Boring
- **P-2** - Percolation Test





MONUMENT NOTES:

● INDICATES FOUND MONUMENT AS NOTED

INDICATED FOUND 1" CALDOT MON. PER RS 161/65-97

( ) - INDICATES RECORD DATA PER MB 14/91

[ ] - INDICATED RECORD DATA PER RS 111/77-78

< > - INDICATES RECORD DATA PER PMB 109/43

<< >>- INDICATES RECORD DATA PER PMB 81/23

{ } - INDICATED RECORD DATA PER PMB 75/32

SCE - INDICATES SCE EASEMENT ALONG PROPERTY LINE WITH UNDISCLOSED WIDTH PER BK. 765, PAGE 185, O.R.

SURVEYOR'S NOTE:

BASIS OF BEARINGS

THE BASIS OF BEARINGS FOR THE SURVEY IS THE CENTERLINE OF WHEAT ROAD BEING NORTH 00°05'30" WEST PER PMB 109/43.

OFFSET DISTANCES

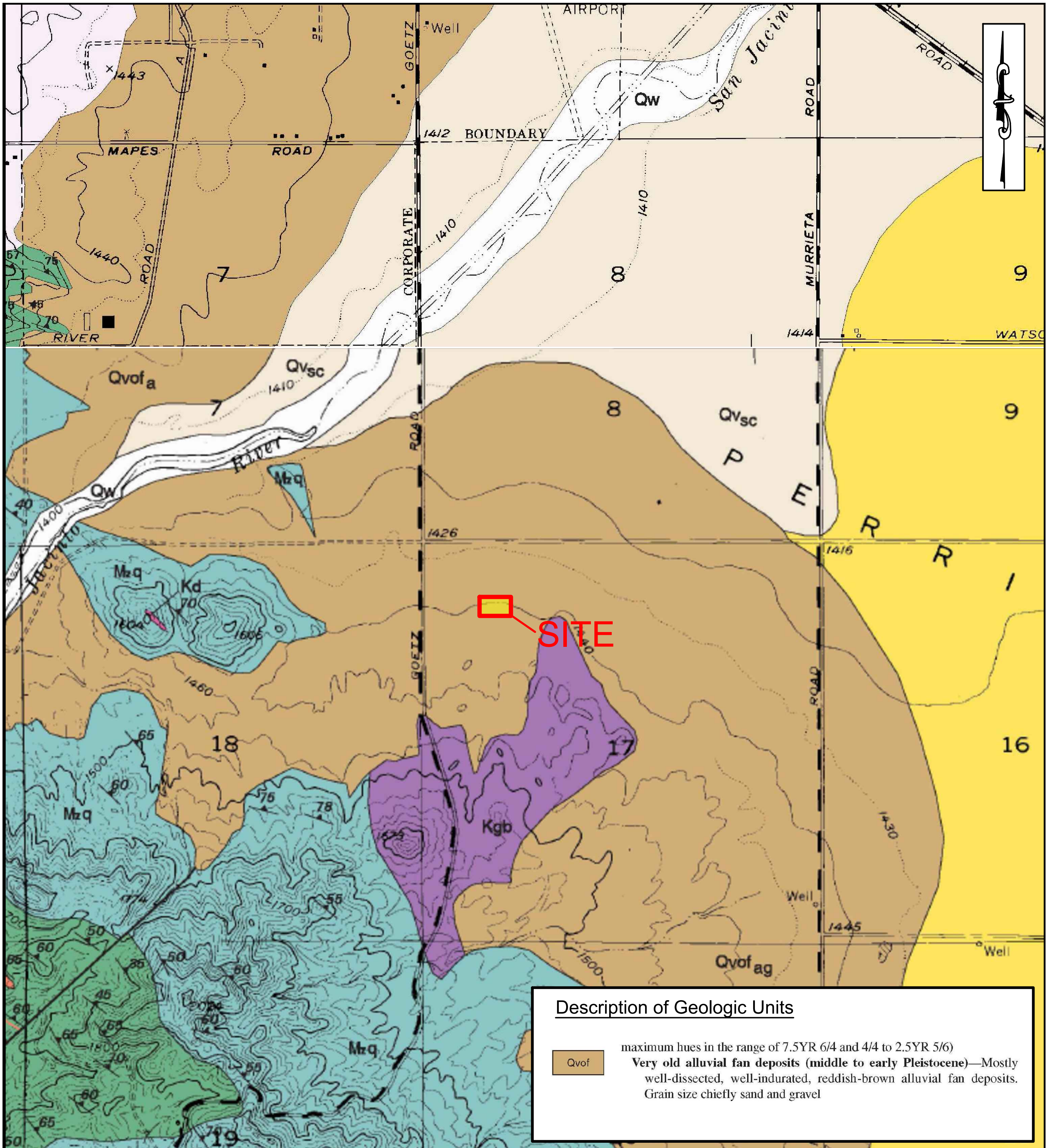
DISTANCES TO OBJECTS FROM THE PROPERTY LINE ARE SHOWN FROM THE CENTER OF OBJECT WHEN OBJECT IS CIRCULAR IN SHAPE.

SURVEYOR/MAP PREPARER:

J.D. COLE & ASSOCIATES, INC.
11056 RAMONA STREET
YUCAIPA, CA 92399
(909) 797-2074

SITE PLAN

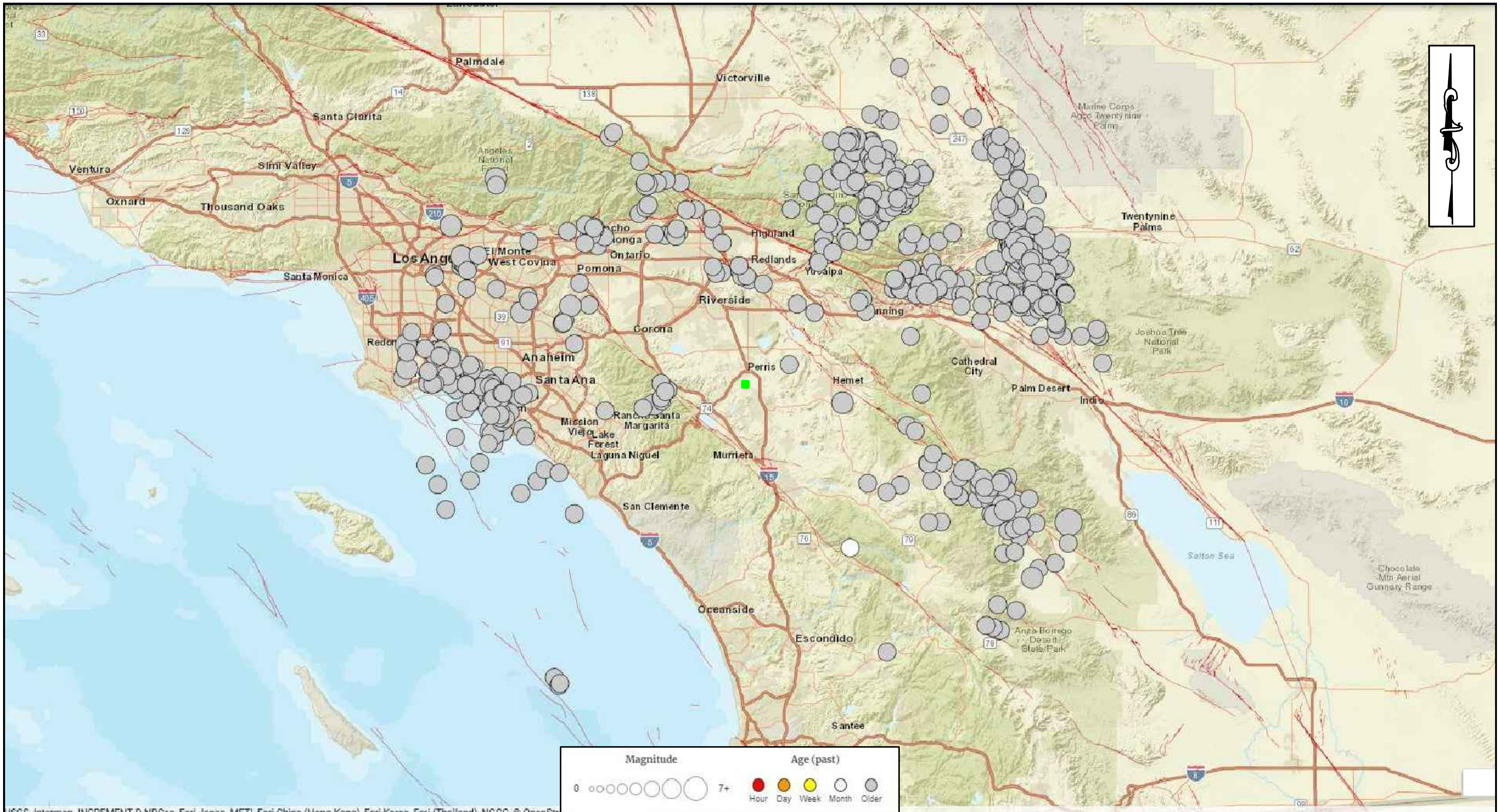
PROJECT:	Proposed Industrial Development, Menifee, California	PROJECT NO.:	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	ENCLOSURE:	A-2
LOR GEOTECHNICAL GROUP, INC.		DATE:	February 2022
		SCALE:	1" ≈ 50'



REGIONAL GEOLOGIC MAP

(Morton, 2003)

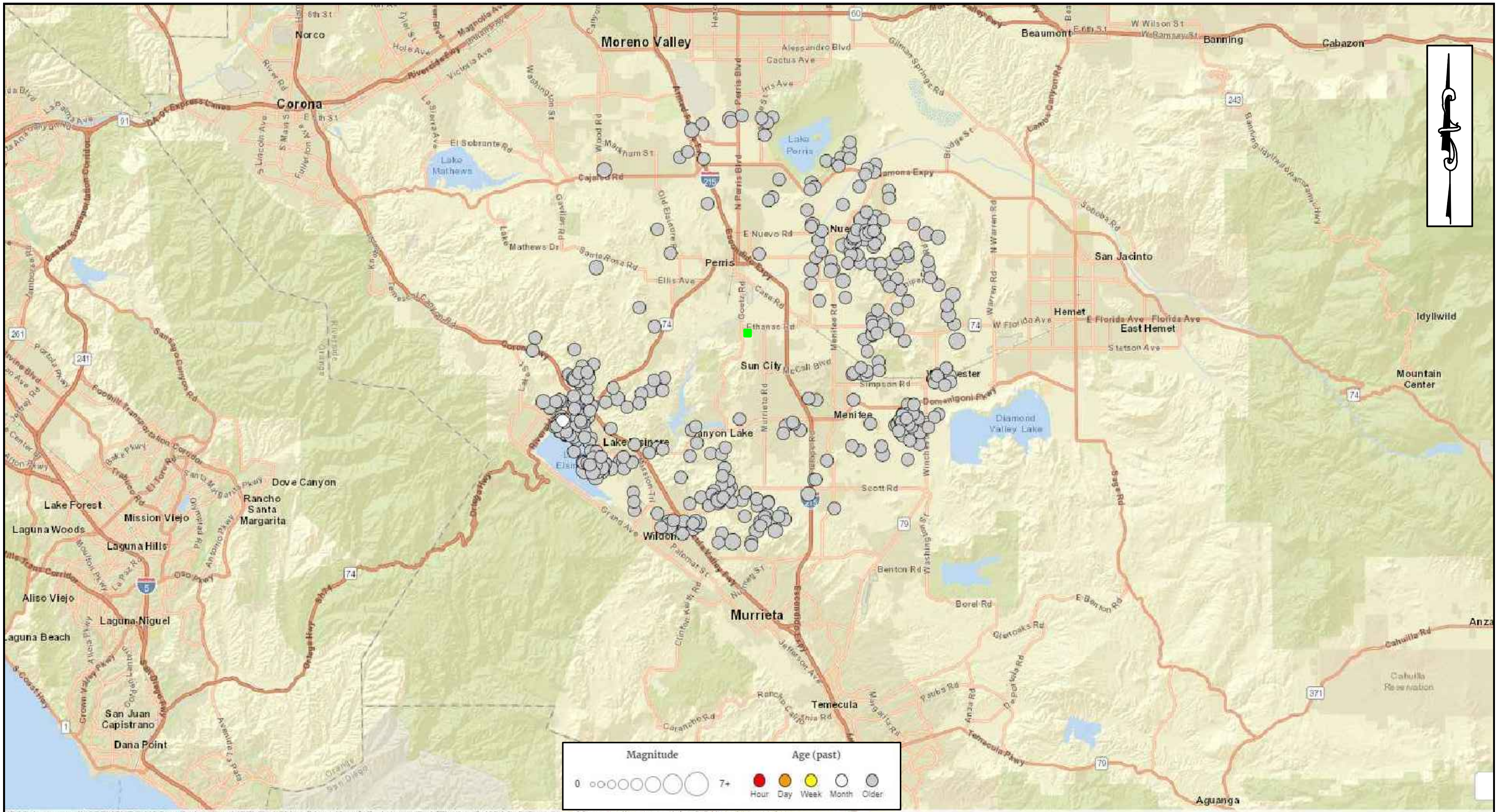
PROJECT:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 531 epicenters of instrument-recorded events from 01/01/32 to present (02/08/22) 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:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 431 epicenters of instrument-recorded events from 01/01/78 to present (02/08/22) 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:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022
		SCALE:	1" ≈ 15km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

Our subsurface exploration of the site consisted of drilling 5 exploratory borings to depths between approximately 16.5 to 41feet below the existing ground surface using a Mobile B-61 drill rig on January 20, 2022. The approximate locations of the borings are shown on Enclosure A-2 within Appendix A.

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

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

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

All samples obtained were taken to our geotechnical laboratory for storage and testing. Detailed logs of the borings are presented on the enclosed Boring Logs, Enclosures B-1 through B-5. A Boring Log Legend is presented on Enclosure B-i. A Soil Classification Chart is presented as Enclosure B-ii.

CONSISTENCY OF SOIL

SANDS

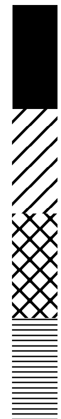
SPT BLOWS	CONSISTENCY
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Over 50	Very Dense

COHESIVE SOILS

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

SAMPLE KEY

Symbol **Description**



INDICATES CALIFORNIA SPLIT SPOON SOIL SAMPLE

INDICATES BULK SAMPLE

INDICATES SAND CONE OR NUCLEAR DENSITY TEST

INDICATES STANDARD PENETRATION TEST (SPT) SOIL SAMPLE

TYPES OF LABORATORY TESTS

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

HAND AUGER BORING LOG LEGEND

PROJECT:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
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:	Proposed Industrial Development, Menifee, California	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	February 2022

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		4, 8, 9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 15% medium grained sand, 40% fine grained sand, 45% silty fines with clay, red brown, damp, loose.**
	5		10.8		108.2			ML	**@ 1 foot,** OLDER ALLUVIUM: **SANDY SILT, trace gravel to 1/2", trace coarse grained sand, approximately 10% medium grained sand, 35% fine grained sand, 55% silty fines with clay, red brown, dramp.**
5	38		13.5		104.5			CL	**@ 5 feet, LEAN CLAY with SAND, approximately 10% medium grained sand, 30% fine grained sand, 60% clayey fines of low plasticity, red brown, damp to moist, trace thin calcite stringers.**
	64		20.5		106.3			SM	**from 6 to 7 feet, some gravel, rig chatter.** **@ 7 feet, SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 40% silty fines, red brown, moist, micaceous.**
10	70 for 11"		14.7		108.9				**@ 10 feet,** GRANITIC BEDROCK: **highly weathered, coarse to medium grained, red brown.**
15	77 for 11"		21.3		105.9				**@ 15 feet, becomes slightly less weathered, yellow brown.**
20	83 for 10"		10.5		122.4				**@ 20 feet, red brown.**
									END OF BORING @ 20.83' **Fill to 1'** **No groundwater** **Bedrock @ 10'**
25									

PROJECT:	Proposed Industrial Developemnt	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1442
		DATE DRILLED:	January 20, 2022
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-1

LOG OF BORING B-2

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		3, 4, 7, 9, 10, 11						GW	@ 0 feet, 3/4" gravel.
								SM	@ 0.8 feet, FILL/TOPSOIL: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 45% fine grained sand, 35% silty fines, red brown, damp.
	27		12.7		119.1				@ 1 foot, OLDER ALLUVIUM: SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% silty fines with clay, red brown, damp, trace pinhole porosity.
5	26		16.1		103.1				
	78		12.0		105.6				@ 6.5 feet, GRANITIC BEDROCK: moderately weathered, coarse to medium grained, tan, damp.
10	68 for 11"		11.8		106.9				
15	116 for 11"		13.6						
									END OF BORING @ 16.42' Fill to 1' No groundwater Bedrock @ 6.5'
20									

PROJECT:	Proposed Industrial Developemnt	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1444
		DATE DRILLED:	January 20, 2022
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, FILL/TOPSOIL: **SILTY SAND, trace gravel to 1", approximately 5% coarse grained sand, 15% medium grained sand, 45% fine grained sand, 35% silty fines with trace clay, brown, dry.**
	16		4.2		98.5			ML	**@ 1 foot,** OLDER ALLUVIUM: **SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 20% fine grained sand, 65% silty fines with clay, red brown, dry, some pinhole porosity.**
5	25		7.7		110.1				**@ 5 feet, trace gravel to 1/2", slightly coarser grained.**
	64 for 10'		16.5		97.9			SM	**@ 7 feet, SILTY SAND, approximately 15% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines, red brown, moist.**
10	46 for 5"		14.5		105.6				**@ 10 feet,** GRANITIC BEDROCK: **slightly to moderately weathered, coarse to medium grained, red brown.**
15	65 for 6"		15.7						
20	44		18.2						**@ 20 feet, slightly coarser grained, damp.**
25	73		23.4						
30	117		20.1						**@ 30 feet, difficult drilling to end of boring.**
35	172 for 10"		25.2						
40	124		28.1						**END OF BORING @ 41' due to very slow progress.** **Fill to 1'** **No groundwater** **Bedrock @ 10'**
45									

PROJECT:	Proposed Industrial Developemnt	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**ELEVATION:**	1441
		DATE DRILLED:	January 20, 2022
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-3

LOR GEOTECHNICAL GROUP, INC.

LOG OF BORING B-4

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, trace gravel to 1", approximately 10% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 35% silty fines, brown, dry.**
	21		6.6		103.1			ML	**@ 1 foot,** OLDER ALLUVIUM: **SANDY SILT, approximately 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% silty fiines with clay, red brown, damp.** **@ 2 feet, trace pinhole porosity, dry.**
5	31		11.7		110.2			CL	**@ 5 feet, LEAN CLAY with SAND, approximately 5% coarse grained sand, 10% medium grained sand, 20% fine grained sand, 65% clayey fines of low plasticity, red brown, damp.**
10	61		5.9		116.8				**@ 10 feet,** GRANTIIC BEDROCK: **highly weathered, friable, coarse to medium grained, yellow brown, dry.**
15	46 for 6"		15.2						**@ 15 feet, less weathered, finer grained, damp, rings disturbed.**
20									

END OF BORING @ 15.5'

Fill to 1'
No goundwater
Bedrock @ 10'

PROJECT: Proposed Industrial Developemnt	**PROJECT NO.:** 23796.1
CLIENT: Compass Danbe Real Estate Partners II, LLC	**ELEVATION:** 1445
LOR GEOTECHNICAL GROUP, INC.	**DATE DRILLED:** January 20, 2022
	EQUIPMENT: Mobile B-61
	HOLE DIA.: 8" **ENCLOSURE:** B-4

LOG OF BORING B-5

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	**@ 0 feet,** FILL/TOPSOIL: **SILTY SAND, trace gravel to 1", approximately 5% coarse grained sand, 10% medium grained sand, 60% fine grained sand, 25% silty fines, brown, dry.**
								ML	**@ 1 foot,** OLDER ALLUVIUM: **SANDY SIILT, apprximately 5% coarse grained sand, 10% medium grained sand, 25% fine grained sand, 60% silty fines with clay, red brown, dry, some pinhole porosity.**
	13		6.4		104.8				
5	10	2	8.5		106.8				**@ 5 feet, increase in clay content, damp, remains porous.**
	68		5.2		129.3				**@ 7 feet,** GRANTITIC BEDROCK: **moderately weathered, friable, coarse to medium grained, yellow brown, dry.**
10	68		5.2						**@ 10 feet, rings disturbed.**
15	46 for 6"		6.0						**@ 15 feet, rings disturbed.**
									END OF BORING @ 15.5' **Fill to 1'** **No groundwater** **Bedrock @ 7'**
20									

PROJECT: Proposed Industrial Developemnt		**PROJECT NO.:**	23796.1
CLIENT: Compass Danbe Real Estate Partners II, LLC		**ELEVATION:**	1447
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	January 20, 2022
		EQUIPMENT:	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-5

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

Selected soil samples obtained from the borings were tested in our geotechnical laboratory to evaluate the physical properties of the soils affecting foundation design and construction procedures. The laboratory testing program performed in conjunction with our investigation included moisture content, dry density, laboratory compaction characteristics, direct shear, sieve analysis, sand equivalent, R-value, expansion index, Atterberg limits, 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 2921 and ASTM D 2216, respectively, and the results are shown on the boring logs, Enclosures B-1 through B-5 for convenient correlation with the soil profile.

Laboratory Compaction

A selected soil sample was tested in the laboratory to determine compaction characteristics using the ASTM D 1557 compaction test method. The results are presented in the following table:

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-2	0-3	(SM) Silty Sand	138.0	7.5

Direct Shear Test

Shear tests are performed in general accordance with ASTM D 3080 with a direct shear machine at a constant rate-of-strain (0.04 inches/minute). The machine is designed to test a sample partially extruded from a sample ring in single shear. Samples are tested at varying normal loads in order to evaluate the shear strength parameters, angle of internal friction and cohesion. Samples are tested in remolded condition (90 percent relative compaction per ASTM D 1557) and soaked, to represent the worse case conditions expected in the field.

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

DIRECT SHEAR TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Apparent Cohesion (psf)**	**Angle of Internal Friction (degrees)**
B-2	0-3	(SM) Silty Sand	150	27

Sieve Analysis

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

Sand Equivalent

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

R-Value Test

A 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 results of the R-value test is presented on Enclosure C-1.

Expansion Index Test

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

EXPANSION INDEX TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-3	(ML) Silty Sand	18	Very Low
B-2	0-3	(SM) Silty Sand	3	Very Low
Expansion Index:	0-20 Very low	21-50 Low	51-90 Medium	91-130 High

Consolidation Test

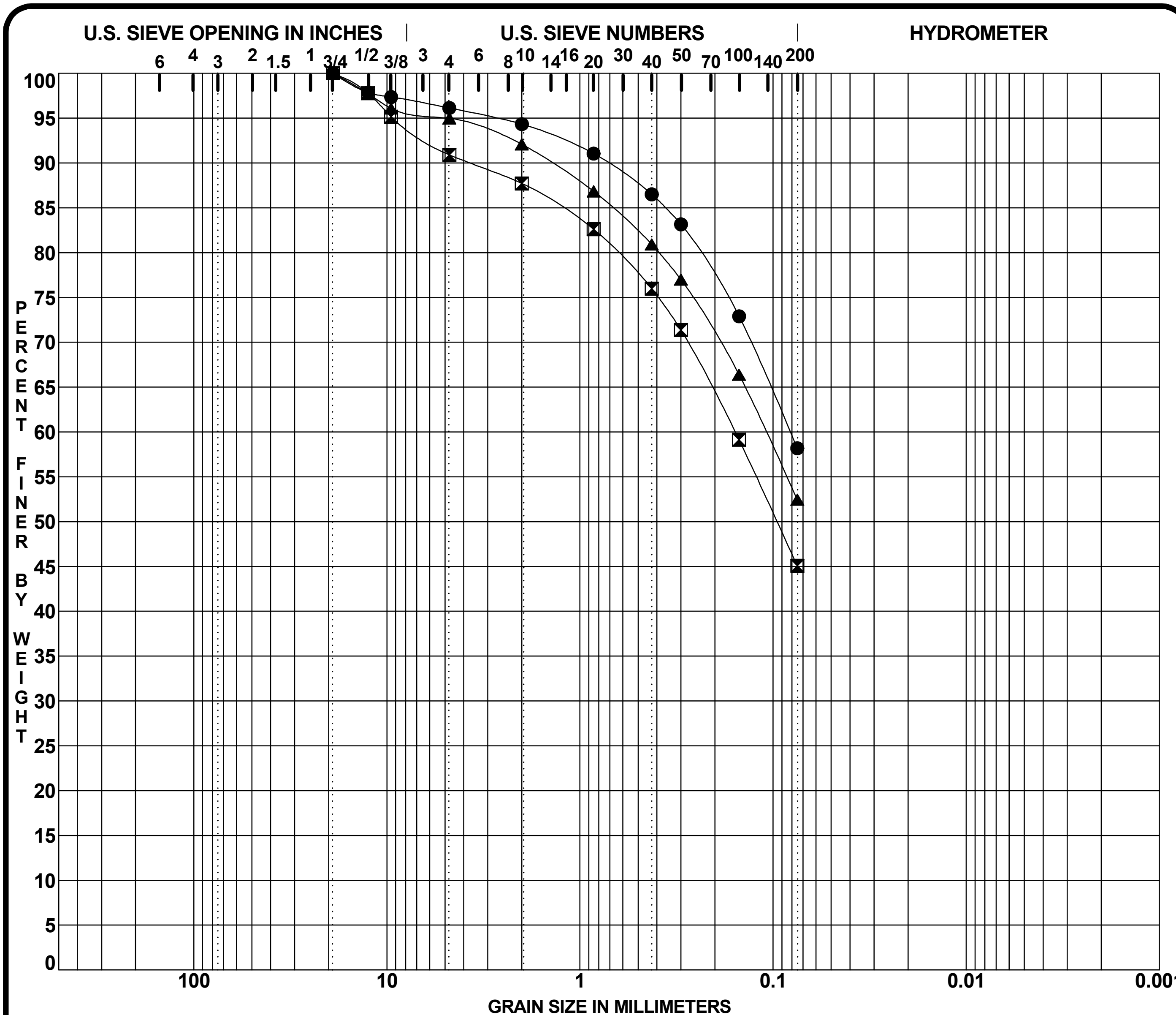
The apparatus used for the consolidation tests (odometer) is designed to test a one-inch high portion of the undisturbed soil sample as contained in a sample ring. Porous stones and filler paper are placed in contact with the top and bottom of the specimen to permit the addition or release of water. Loads are applied to the test specimen in specified increments, and the resulting axial deformations are recorded. The results are plotted as log of axial pressure versus consolidation or compression, expressed as strain or sample height.

Samples are tested at field and greater-than field moisture contents. The results are shown on Enclosure C-2.

Soluble Sulfate Content Test

The soluble sulfate content of a selected subgrade soil was evaluated. The concentration of soluble sulfates in the soil 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 sample. The measured optical density is correlated with readings on precipitates of known sulfate concentrations. The test result is presented in the following table:

SOLUBLE SULFATE CONTENT TEST			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Sulfate Content (% by weight)**
B-1	0-3	(ML) Sandy Silt	< 0.005
B-2	0-3	(SM) Silty Sand	< 0.005
B-4	0-3	(ML) Sandy 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*	*@ 0-3'*	*(ML) Sandy Silt*	*8*	*12*				
⊠	*B-2*	*@ 0-3'*	*(SM) Silty Sand*	*11*	*--*				
▲	*B-4*	*@ 0-3'*	*(ML) Sandy Siilt*	*9*	*--*				

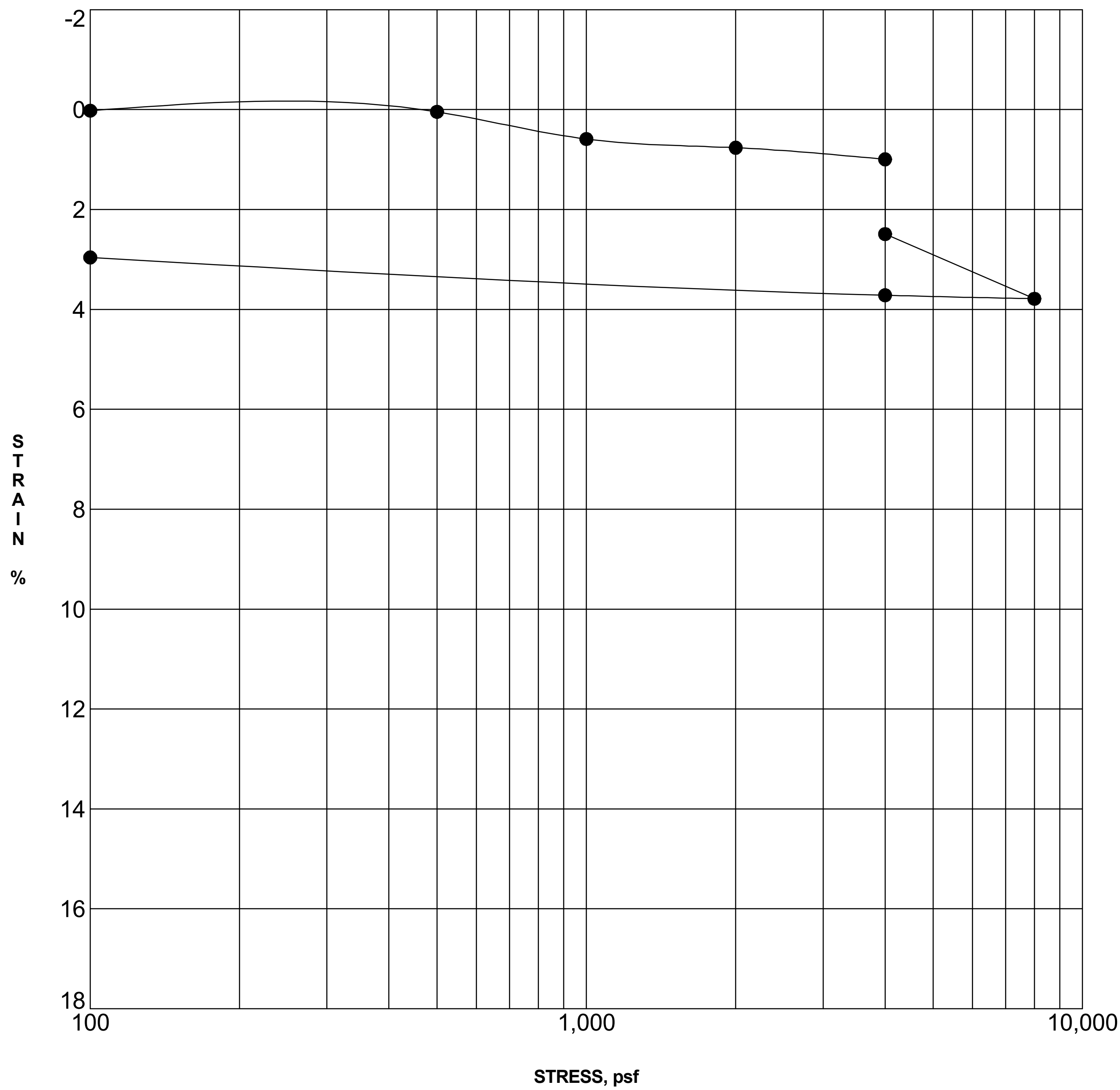
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*@ 0-3'*	*19.00*	*0.08*			*3.9*	*38.0*	*58.2*	
⊠	*B-2*	*@ 0-3'*	*19.00*	*0.16*			*9.1*	*45.8*	*45.1*	
▲	*B-4*	*@ 0-3'*	*19.00*	*0.11*			*5.0*	*42.5*	*52.4*	

PROJECT:	Proposed Industrial Developemnt	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**DATE:**	February 2022

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-1



	Specimen Identification		Classification	DD	MC%
●	B-5	@ 5'	*(ML) Sandy Silt*	109	9

PROJECT:	Proposed Industrial Developemnt	**PROJECT NO.:**	23796.1
CLIENT:	Compass Danbe Real Estate Partners II, LLC	**DATE:**	February 2022

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-2

APPENDIX D

Seismic Design Spectra

Project: APN 330-180-012, 26201 Wheat Street, Menifee
Project Number: 23796.1
Client: Compass Danbe Real Estate Partners II, LLC
Site Lat/Long: 33.74081/-117.22049
Controlling Seismic Source: Elsinore

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	0.130
S_{D1}/S_{DS}	T_S	0.652
Fundamental Period (12.8.2)	T	Period
Seismic Design Maps or Fig 22-14	T_L	8
Equation 11.4-4 - $2/3*S_{M1}$	S_{D1}	0.6229*
Equation 11.4-2 - F_V*S_1	S_{M1}	0.9344*

REFERENCE	NOTATION	VALUE
Fv (Table 11.4-2)[Used for General Spectrum]	F_v	1.8
Design Maps	S_s	1.432
Design Maps	S_1	0.527
Equation 11.4-1 - F_A*S_S	S_{MS}	1.432*
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	0.955*
Design Maps	PGA	0.5
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.55*
Section 21.5.3	80% of PGA_M	0.440
Design Maps	C_{RS}	0.936
Design Maps	C_{R1}	9.21

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=C_{RS}	C_{RS}	0.936
Cr - At Periods >=1.0, Cr=C_{R1}	C_{R1}	9.21

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

Period	Cr
0.200	0.936
0.300	1.970
0.400	3.005
0.500	4.039
0.600	5.073
0.680	5.900
1.000	9.21

* **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: 23796.1

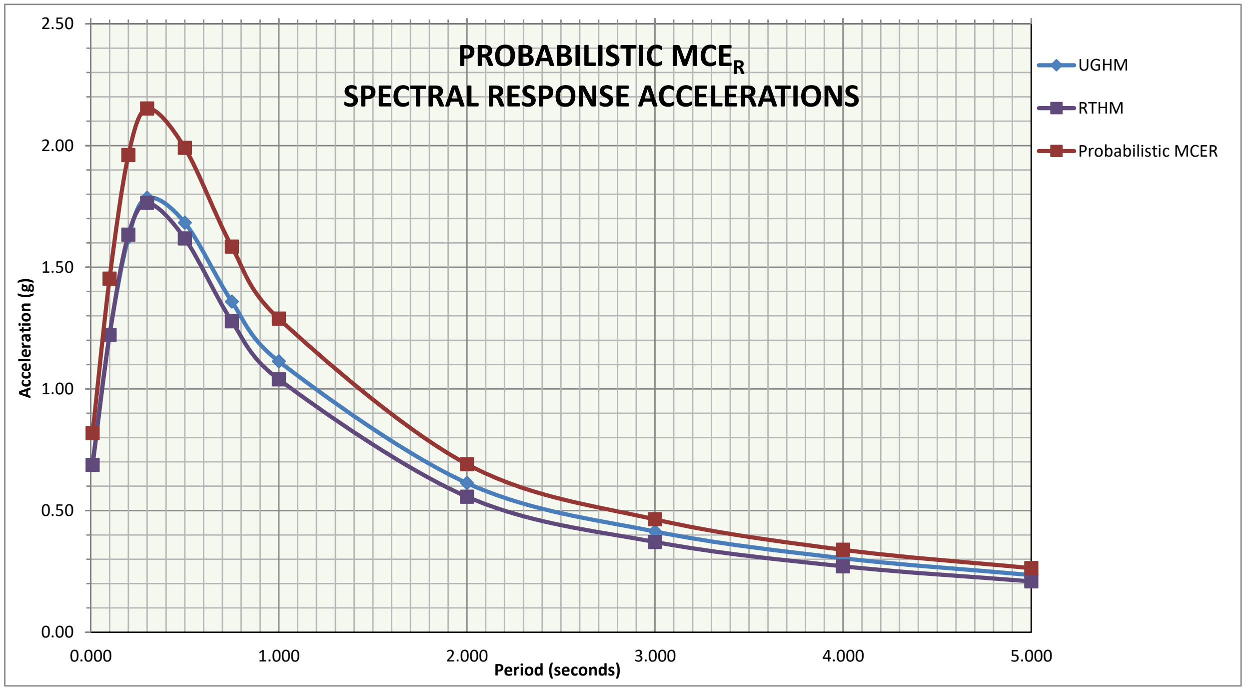
Period	UGHM	RTHM	Max Directional Scale Factor[2]	Probabilistic MCE
0.010	0.696	0.688	1.19	0.819
0.100	1.221	1.221	1.19	1.453
0.200	1.621	1.634	1.20	1.961
0.300	1.785	1.764	1.22	2.152
0.500	1.683	1.618	1.23	1.990
0.750	1.359	1.278	1.24	1.585
1.000	1.113	1.039	1.24	1.288
2.000	0.613	0.557	1.24	0.691
3.000	0.414	0.371	1.25	0.464
4.000	0.304	0.271	1.25	0.339
5.000	0.235	0.209	1.26	0.263

Probabilistic PGA: 0.696

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)



LOR GEOTECHNICAL GROUP, INC.

DETERMINISTIC SPECTRUM

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

Controlling Source: Elsinore

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

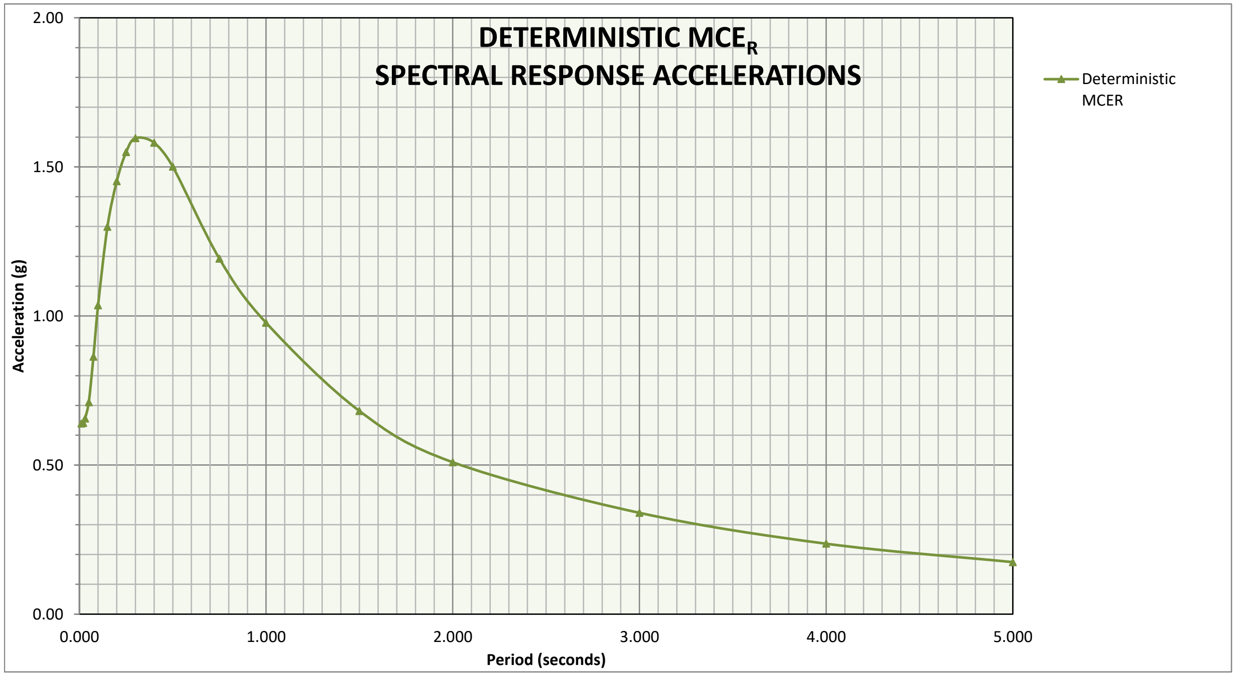
Project No: 23796.1

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	0.537	1.19	0.639	0.639
0.020	0.539	1.19	0.641	0.641
0.030	0.551	1.19	0.656	0.656
0.050	0.597	1.19	0.710	0.710
0.075	0.725	1.19	0.863	0.863
0.100	0.870	1.19	1.036	1.036
0.150	1.082	1.20	1.299	1.299
0.200	1.210	1.20	1.452	1.452
0.250	1.280	1.21	1.549	1.549
0.300	1.309	1.22	1.596	1.596
0.400	1.286	1.23	1.581	1.581
0.500	1.220	1.23	1.501	1.501
0.750	0.961	1.24	1.192	1.192
1.000	0.788	1.24	0.977	0.977
1.500	0.549	1.24	0.681	0.681
2.000	0.411	1.24	0.509	0.509
3.000	0.272	1.25	0.340	0.340
4.000	0.189	1.25	0.236	0.236
5.000	0.138	1.26	0.174	0.174

Is Determinstic $Sa_{(max)}$<1.5*Fa?	NO
Section 21.2.2 Scaling Factor:	N/A
Deterministic PGA:	~~0.537~~
Is Deterministic PGA >=F_{PGA}*0.5?	NO
Deterministic PGA:	0.550

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

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



SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	0.819	0.639	0.639	0.426
0.100	1.453	1.036	1.036	0.690
0.200	1.961	1.452	1.452	0.968
0.300	2.152	1.596	1.596	1.064
0.500	1.990	1.501	1.501	1.001
0.750	1.585	1.192	1.192	0.795
1.000	1.288	0.977	0.977	0.651
2.000	0.691	0.509	0.509	0.340
3.000	0.464	0.340	0.340	0.226
4.000	0.339	0.236	0.236	0.157
5.000	0.263	0.174	0.174	0.116

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.404	0.323
0.010	0.426	0.341
0.020	0.470	0.376
0.030	0.514	0.411
0.050	0.601	0.481
0.060	0.645	0.516
0.075	0.711	0.569
0.090	0.777	0.622
0.100	0.821	0.657
0.110	0.865	0.692
0.120	0.909	0.727
0.136	0.955	0.764
0.150	0.955	0.764
0.160	0.955	0.764
0.170	0.955	0.764
0.180	0.955	0.764
0.200	0.955	0.764
0.250	0.955	0.764
0.300	0.955	0.764
0.400	0.955	0.764
0.500	0.955	0.764
0.600	0.955	0.764
0.640	0.955	0.764
0.750	0.831	0.664
0.850	0.733	0.586
0.900	0.692	0.554
0.950	0.656	0.525
1.000	0.623	0.498
1.500	0.415	0.332
2.000	0.311	0.249
3.000	0.208	0.166
4.000	0.156	0.125
5.000	0.125	0.100

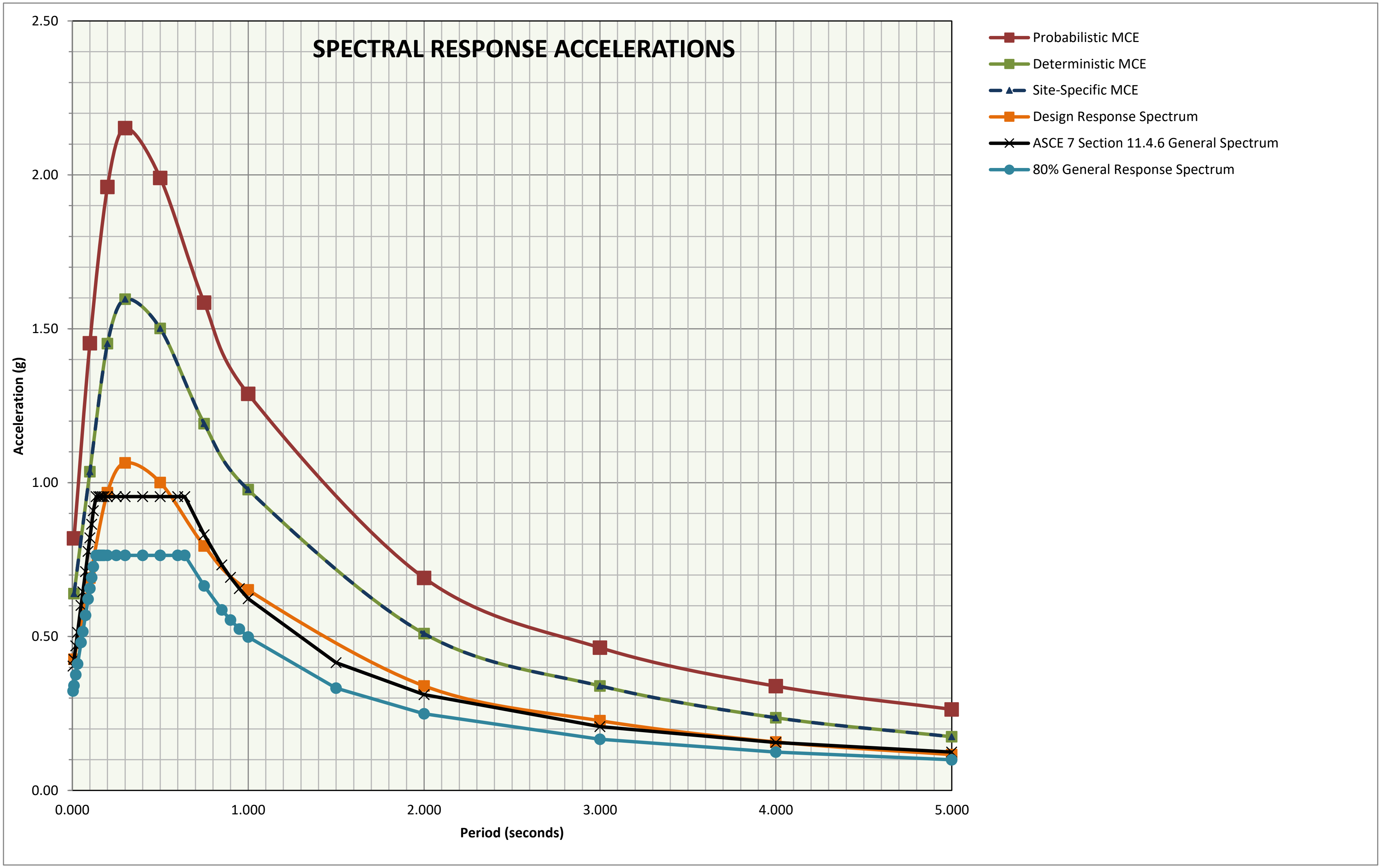
ASCE 7-16: Section 21.4
Site Specific

	Calculated Value	Design Value
SDS:	0.958	0.958
SD1:	0.679	0.679
SMS:	1.437	1.437
SM1:	1.019	1.019
Site Specific PGAm:	0.550	0.550
Site Class:	D measured	

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

* Risk Categories I, II, or III

Project No: 23796.1



Project No: 23796.1

LOR GEOTECHNICAL GROUP, INC.

APPENDIX E

Infiltration Test Results

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	26201 Wheat Street	Test Date:	January 21, 2022
Project No.:	23796.1	Test Hole No.:	P-1
Soil Classificaiton:	(ML) Sandy silt	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	7.8 ft.	Date Excavated:	January 20, 2022
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	9:01 AM	9:31 AM	30	0.50	0.50	43.00	44.00	94.00	94.00	1.00	50.50	30.0
2	9:31 AM	10:01 AM	30	0.50	1.00	44.00	45.00	94.00	94.00	1.00	49.50	30.0
3	10:01 AM	10:31 AM	30	0.50	1.50	45.00	46.00	94.00	94.00	1.00	48.50	30.0
4	10:31 AM	11:01 AM	30	0.50	2.00	46.00	47.00	94.00	94.00	1.00	47.50	30.0
5	11:01 AM	11:31 AM	30	0.50	2.50	47.00	48.00	94.00	94.00	1.00	46.50	30.0
6	11:31 AM	12:01 PM	30	0.50	3.00	48.00	49.00	94.00	94.00	1.00	45.50	30.0
7	12:01 PM	12:31 PM	30	0.50	3.50	49.00	50.00	94.00	94.00	1.00	44.50	30.0
8	12:31 PM	1:01 PM	30	0.50	4.00	50.00	51.00	94.00	94.00	1.00	43.50	30.0
9	1:01 PM	1:31 PM	30	0.50	4.50	51.00	52.00	94.00	94.00	1.00	42.50	30.0
10	1:31 PM	2:01 PM	30	0.50	5.00	48.00	49.00	94.00	94.00	1.00	45.50	30.0
11	2:01 PM	2:31 PM	30	0.50	5.50	49.00	50.00	94.00	94.00	1.00	44.50	30.0
12	2:31 PM	3:01 PM	30	0.50	6.00	50.00	51.00	94.00	94.00	1.00	43.50	30.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	44.00	
H_f	43.00	
ΔH	1.00	
H_{avg}	43.50	
I_t	**0.05**	in/hr (clear water rate)

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

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	26201 Wheat Street	Test Date:	January 21, 2022
Project No.:	23796.1	Test Hole No.:	P-2
Soil Classificaiton:	(ML) Sandy silt	Effective Hole Dia.*:	4.8 in.
Depth of Test Hole:	7.9 ft.	Date Excavated:	January 20, 2022
Tested By:	A.L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	(min/in)
1	9:03 AM	9:33 AM	30	0.50	0.50	48.00	53.00	95.00	95.00	5.00	44.50	6.0
2	9:33 AM	10:03 AM	30	0.50	1.00	48.00	53.00	95.00	95.00	5.00	44.50	6.0
3	10:03 AM	10:33 AM	30	0.50	1.50	48.00	52.50	95.00	95.00	4.50	44.75	6.7
4	10:33 AM	11:03 AM	30	0.50	2.00	48.00	52.50	95.00	95.00	4.50	44.75	6.7
5	11:03 AM	11:33 AM	30	0.50	2.50	48.00	53.00	95.00	95.00	5.00	44.50	6.0
6	11:33 AM	12:03 PM	30	0.50	3.00	48.00	52.50	95.00	95.00	4.50	44.75	6.7
7	12:03 PM	12:33 PM	30	0.50	3.50	48.00	52.50	95.00	95.00	4.50	44.75	6.7
8	12:33 PM	1:03 PM	30	0.50	4.00	48.00	52.00	95.00	95.00	4.00	45.00	7.5
9	1:03 PM	1:33 PM	30	0.50	4.50	48.00	52.00	95.00	95.00	4.00	45.00	7.5
10	1:33 PM	2:03 PM	30	0.50	5.00	48.00	52.00	95.00	95.00	4.00	45.00	7.5
11	2:03 PM	2:33 PM	30	0.50	5.50	48.00	52.00	95.00	95.00	4.00	45.00	7.5
12	2:33 PM	3:03 PM	30	0.50	6.00	48.00	52.00	95.00	95.00	4.00	45.00	7.5

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	47.00	
H_f	43.00	
ΔH	4.00	
H_{avg}	45.00	
I_t	**0.21**	in/hr (clear water rate)

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