PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED CRYSTAL COVE MULTI-FAMILY RESIDENTIAL DEVELOPMENT, APN 484-030-028, MORENO VALLEY, CALIFORNIA

PROJECT NO. 33767.1
OCTOBER 20, 2021

Prepared For:

Empire CM, Inc.
151 Kalmus Drive, Suite A202
Costa Mesa, California 92626

Attention: Mr. Josh Gause

October 20, 2021

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Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Crystal Cove Multi-Family Residential Development, APN 484-030-028, Moreno Valley, 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 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, 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.

Poor infiltration rates were obtained for the soils tested.

LOR Geotechnical Group, Inc.

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INTRODUCTION

During September and October of 2021, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc., for the proposed Crystal Cove Multi-Family Residential development of Assessor Parcel Number (APN) 484-030-028, Moreno Valley, 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 2020;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from documents, literature, and reports reviewed;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Percolation testing via the borehole test method;
- 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 was furnished for our use. The proposed building configuration and 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 nine, 2-and 3-story apartment buildings, a clubhouse, swimming pool, and driveways, parking, and landscape areas. The buildings are anticipated to be of wood frame and stucco or similar type construction and light to moderate foundation loads are anticipated with these structures.

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 has remained vacant land since 1966, the earliest photograph available. From 1966 to 1997, the site appeared to be dry land farmed in conjunction with the adjacent properties to the south and southwest. No evidence for the presence of faults traversing the site area or mass movement features was noted during our review of the photographs covering the site and nearby vicinity.

EXISTING SITE CONDITIONS

The subject site is approximately 9 acres of vacant land, roughly rectangular in shape, located at the southwest corner of the intersection of Alessandro Boulevard and Lasselle Street in Moreno Valley. The site is situated at elevations ranging from approximately 1,567 to 1,582 feet above mean sea level. The topography of the subject site is relatively planar in general with a gentle fall to the south-southwest. Along most of the west boundary, excluding the north end, is an approximately 6-foot high wall constructed of pilasters and wrought iron fencing. Natural vegetation onsite includes grasses and tumbleweeds up to approximately 4 feet high. Large swaths along the periphery of the subject site have been disced. A rectangular area within the central and east central portions of the subject site has not been disced for weed control. Bare areas of soil are present along/near the roadways for Copper Cove Lane and Lasselle Street. There are relatively minor amounts of trash and debris onsite. In relatively close proximity to the southwest corner of the subject site are a few, relatively small piles of apparent dumped soils that include concrete and asphalt concrete debris.

Alessandro Boulevard, an asphalt-paved roadway, borders the site to the north, with vacant land beyond. Lasselle Street, an asphalt-paved roadway, borders the site on the east with vacant land beyond. Copper Cove Lane, an asphalt-paved roadway, borders the site on the south, with a tract of residential properties beyond. A church property is located adjacent to the site on the west.

SUBSURFACE FIELD INVESTIGATION

Our subsurface field exploration program was conducted on September 27, 2021. The work consisted of advancing a total of 6 exploratory borings using a truck-mounted drill rig equipped with 8-inch diameter hollow stem augers. 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 15.33 to 30.33 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, consolidation, expansion index, 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

The subject site is situated within the northeastern portion of the Peninsular Ranges Geomorphic Province of Southern California. This province incorporates several northwest-trending mountain ranges, such as the Santa Ana and San Jacinto Mountains, which extend from the Transverse Ranges Geomorphic Province, northeast of Los Angeles, into the Baja California Peninsula. Lying in between these small ranges are a series of valleys and basins, such as the Perris Plain. The Perris Plain is composed of rocks of the Peninsular Ranges batholith, a very large mass composed primarily of batholithic crystalline igneous rocks, with lesser amounts of metasedimentary and metavolcanic rocks which predate the intrusion of the batholith. The batholithic rocks actually consist of numerous separate plutonic intrusions which range in composition from gabbro to granite, with tonalite the predominate lithology. While the floor of the Perris Plain is relatively flat, it is dotted with small remnant hills composed of rocks highly resistant to erosion, as observed offsite to the east across Lasselle Street. Erosion of the hills has resulted in the covering of a thin to thick veneer of various ages of alluvial fan materials across the flanks of the hills and out into the adjoining valley floor.

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 Cucamonga fault zone on the north, and the Agua-Tibia fault zone on the south. As the subject site is located near the northeastern margin of Perris Plain, the San Jacinto fault is the closest known active fault in relation to the site. At its closest approach, the San Jacinto fault is located approximately 6.6 kilometers (4.1 miles) to the northeast of the subject site.

The geology of the subject site and immediate surrounding vicinity have been largely mapped as underlain by very old alluvial fan deposits of early Pleistocene age which are mostly well-dissected, well-indurated, and reddish brown sand deposits containing minor gravel, commonly containing duripans and locally silcretes (Morton and Matti, 2001). Offsite and across Lasselle Street to the east, a bedrock unit of tonalite (undifferentiated) of Cretaceous age has also been mapped.

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

Site Geologic Conditions

Fill: Fill materials were encountered within all of our exploratory borings to depths of approximately 2 feet. These materials are believed to be associated with past site use for dry land farming and current and past weed abatement (discing) practices at the site. As encountered, the fill materials were comprised of silty sand which was predominantly brown, dry, and in a loose state. Expansion index testing of these materials indicates a very low expansion potential.

Older Alluvium: Older alluvial materials were encountered underlying the fill materials described above within all of our exploratory borings to the maximum depths explored. These units were noted to mainly consist of silty sand with minor units of sandy silt, clayey sand, and lean clay with sand at depths greater than 7 feet. These materials were typically red brown in color, contained pinhole porosity and calcite stringers, and damp. The older alluvial materials were in a relatively loose to medium dense upon first encounter, becoming dense to very dense quickly with depth based on our equivalent Standard Penetration Test (SPT) data and in-place density testing. Consolidation testing of these materials showed that the upper portions of the older alluvial units to a depth of approximately 5 feet have a moderate to moderately severe potential for collapse. Consolidation testing of the older alluvial units beneath this depth indicate a negligible potential for collapse. Hydro-collapsible soils are primarily defined as unsaturated materials in a low density state that is maintained by apparent cohesion due to clays or accumulated soluble salts at their intergranular contacts. These soils are relatively strong at their natural water contents but experience a significant decrease in volume (settlement) due to softening of the binder upon the introduction of water.

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 our exploratory borings advanced to a maximum depth of approximately 30 feet below the existing ground surface nor was any groundwater seepage observed during our site reconnaissance.

Local water level measurements were researched at the California Department of Water Resources (DWR) online Water Data Library, California State Water Resources Control Board (SWRCB) online GeoTracker database, and Spring 2021 Cooperative Well Measuring Program (California DWR, California SWRCB, and Watermaster Support

Services et al., 2021). The closest well found in the DWR online Water Data Library is Well EMWD25695, located approximately 2.1 kilometers (1.3 miles) to the southeast of the subject site. Groundwater levels measured in this wells have ranged from approximately 40 to 67 feet bgs (1,440 to 1,467 feet above mean sea level [amsl]) within the time period from November 2011 to March 2021. The closest wells in the SWRCB GeoTracker database are associated with a Leaking Underground Storage Tank (LUST) case at a gasoline station (TOSCO/76 Station #6962), addressed 25020 Alessandro Boulevard, located approximately 1.3 kilometers (0.8 mile) west-northwest of the subject site. Twelve groundwater monitoring wells at this LUST site have measurements over the time period from 2000 to 2010 ranging from approximately 25 to 36 feet bgs, with corresponding elevations ranging from approximately 1,530 to 1,540 feet amsl. The closest groundwater wells for which the Cooperative Well Measuring Program has water level data are the monitoring wells associated with the TOSCO/76 Station #6962 LUST site approximately 1.3 kilometers (0.8 mile) west-northwest of the subject site. Based on the lowest subject site ground surface elevation of 1,567 feet amsl, the groundwater associated with the EMWD well to the southeast would imply groundwater at approximately 100 feet bgs at the subject site. However, the closer LUST site groundwater monitoring wells, with shallowest groundwater elevation at approximately 1,540 feet amsl, would imply groundwater at approximately 27 feet bgs at the subject site. However, based on the relatively shallow igneous bedrock (tonalite) encountered at the subject site at depths ranging from approximately 7.5 to 23 feet bgs and the lack of bedrock encountered at the LUST site maximum exploration depth up to 65 feet bgs, and the distance between the subject site and the LUST site, it is difficult to accurately estimate where groundwater may be present beneath the subject site. If groundwater is present beneath the subject site, it would be anticipated as infilling of cracks and fissures within the igneous bedrock at depth. Our exploration boring, B-6, advanced to approximately 30.33 feet bgs from a ground surface elevation of approximately 1,568 feet amsl, suggests no groundwater is present beneath the subject site down to an elevation of approximately 1,538 feet amsl.

The local groundwater flow direction is estimated towards the San Jacinto River, generally in a southerly direction, coincident with the fall in local ground surface topography.

<u>Mass Movement</u>

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 San Jacinto fault located approximately 6.6 kilometers (4.1 miles) to the northeast. In addition, other relatively close active faults include the San Andreas fault located approximately 22.0 kilometers (13.7 miles) to the northeast, and the Elsinore fault located approximately 29.0 kilometers (18.0 miles) to the southwest.

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

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.

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 October 8, 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 10 kilometer (6.2 miles) radius of the site was examined by selecting an epicenter map listing events on the order of 1.0 and greater since 1978. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the time period for the events on the detail map is to enhance the accuracy of the map. Events recorded prior to the mid to late1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, 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. Since groundwater does not lie within 50 feet beneath the site, as found during this investigation, and the site is underlain by relatively dense to very dense older alluvial materials and hard igneous bedrock, 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 to very dense 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 investigation, mapping by others, and our experience in the site region indicates that the materials beneath the site are considered Site Class C 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) 33.91638, -117.21011, Risk Category II	
Site Class Definition Chapter 20 ASCE 7	C
$\mathbf{S_s}$ Mapped Spectral Response Acceleration at 0.2s Period	1.714
$\mathbf{S_1}$ Mapped Spectral Response Acceleration at 1s Period	0.669
$\mathbf{S_{MS}}$ Adjusted Spectral Response Acceleration at 0.2s Period	2.0557
$\mathbf{S_{M1}}$ Adjusted Spectral Response Acceleration at 1s Period	0.937
$\mathbf{S_{DS}}$ Design Spectral Response Acceleration at 0.2s Period	1.371
$\mathbf{S_{D1}}$ Design Spectral Response Acceleration at 1s Period	0.625
$\mathbf{F_a}$ Short Period Site Coefficient at 0.2s Period	1.2
$\mathbf{F_v}$ Long Period Site Coefficient at 1s Period	1.4
PGA_M	0.87
Seismic Design Category	D
*Values obtained from OSHPD Seismic Design Maps tool	

INFILTRATION TESTING AND TEST RESULTS

Infiltration Testing

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 general locations of our tests are illustrated on Enclosure A-2 and were conducted at the requested locations. Test borings were drilled to depths of approximately 5 feet below the existing ground surface on September 27, 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, September 28, 2021, within 26 hours but not before 15 hours, of the pre-soak. The holes were filled using water from a 200 gallon water tank. Test periods consisted of allowing the water to drop in 30-minute intervals. After each reading, the hole was refilled. Testing was terminated after a total of 12 readings were recorded.

Infiltration test results are summarized in the following table:

Test No.	Depth*	Infiltration Rate** (in/hr)
P-1	4.8	2.6
P-2	4.9	1.0

* depth measured below existing ground surface
** Porchet Method determined rate

The results of this testing are presented as Enclosures D-1 and D-2 in Appendix D. The test results indicate poor infiltration characteristics for the soils 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, no specialized construction procedures to resist expansive soil activity are 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 test data indicates the site soils have a poor infiltration rate. Recommendations for design and maintenance of the proposed system are presented within the **RECOMMENDATIONS** section of this report.

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, which are anticipated in areas of the site which were previously developed, 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 material and 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 5 feet deep, exclusive of the end dump stockpiles, 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 are anticipated to be present locally, primarily in areas of previous improvements. Removals should expose alluvial materials with an in-situ relative compaction of at least 85 percent (ASTM D 1557). The actual depths of the removals should be determined during the grading operation by observation and/or in-place density testing.

Preparation of Fill Areas

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

Engineered Compacted Fill

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

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

Preparation of Foundation Areas

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

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 on our exploratory borings, it appears that Type C soils are the predominant type of soil in the upper 7 feet 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 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 and/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 2,100 psf for dead plus live loads.

The above values are net pressures; therefore, the weight of the foundations and the backfill over the foundations may be neglected when computing dead loads. The values apply to the maximum edge pressure for foundations subjected to eccentric loads or overturning. The recommended pressures apply for the total of dead plus frequently applied live loads, and incorporate a factor of safety of at least 3.0. The allowable bearing pressures may be increased by one-third for temporary wind or seismic loading. The resultant of the combined vertical and lateral seismic loads should act within the middle one-third of the footing width. The maximum calculated edge pressure under the toe of foundations subjected to eccentric loads or overturning should not exceed the increased allowable pressure. 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 350 pounds per square foot per foot of depth. Base friction may be computed at 0.35 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 0.50 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 60 pounds per square foot (psf) per foot of depth be used.

This assumes level backfill consisting of compacted, non-expansive, on-site soils placed against the structures and within the back cut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

Retaining structures subject to uniform surcharge loads within a horizontal distance behind the structures equal to the structural height should be designed to resist additional lateral loads equal to 0.50 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 minor truck traffic (ADTT=1)	5.0	15	0.25’ AC / 0.65' AB or 4.5" JPCP / 4" AB
Occasional truck traffic (ADTT=10)	6.0	15	0.25'AC / 0.95'AB or 5" JPCP / 4" AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base JPCP - Jointed Plain Concrete Pavement with MR $\geq$ 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 550 pounds per square inch (psi). Transverse joints should be sawcut in the pavement at approximately 12 to 15-foot intervals within 4 to 6 hours of concrete placement, or preferably sooner. Sawcut depth should be equal to approximately one quarter of slab thickness. Construction joints should be constructed such that adjacent sections butt directly against each other and are keyed into each other. Parallel pavement sections should also be keyed into each other.

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

Infiltration

Based upon our field investigation and infiltration test data, a clear water absorption rate of approximately 1 to 2.6 inches per hour was obtained. It is our opinion that a design clear water rate of 1 inch per hour is appropriate for the planned infiltration in the area and depth tested.

A factor of safety should be applied as indicated by the Design Handbook for Low Impact Development Best Management Practices (RCFCWCD, 2011). The design infiltration rate should be adjusted using a minimum factor of safety 3.0.

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

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

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

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

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

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. Foundation excavations.

5. Subgrade preparation for pavements and slabs-on-grade.

6. Placement of engineered compacted fill and backfill, including approval of fill materials and the performance of sufficient density tests to evaluate the degree of compaction being achieved.

LIMITATIONS

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

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. If conditions are encountered during the construction of the project, which differ significantly from those presented in this report, this firm should be notified immediately 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 (4) and PDF via email jgause@empirecminc.com

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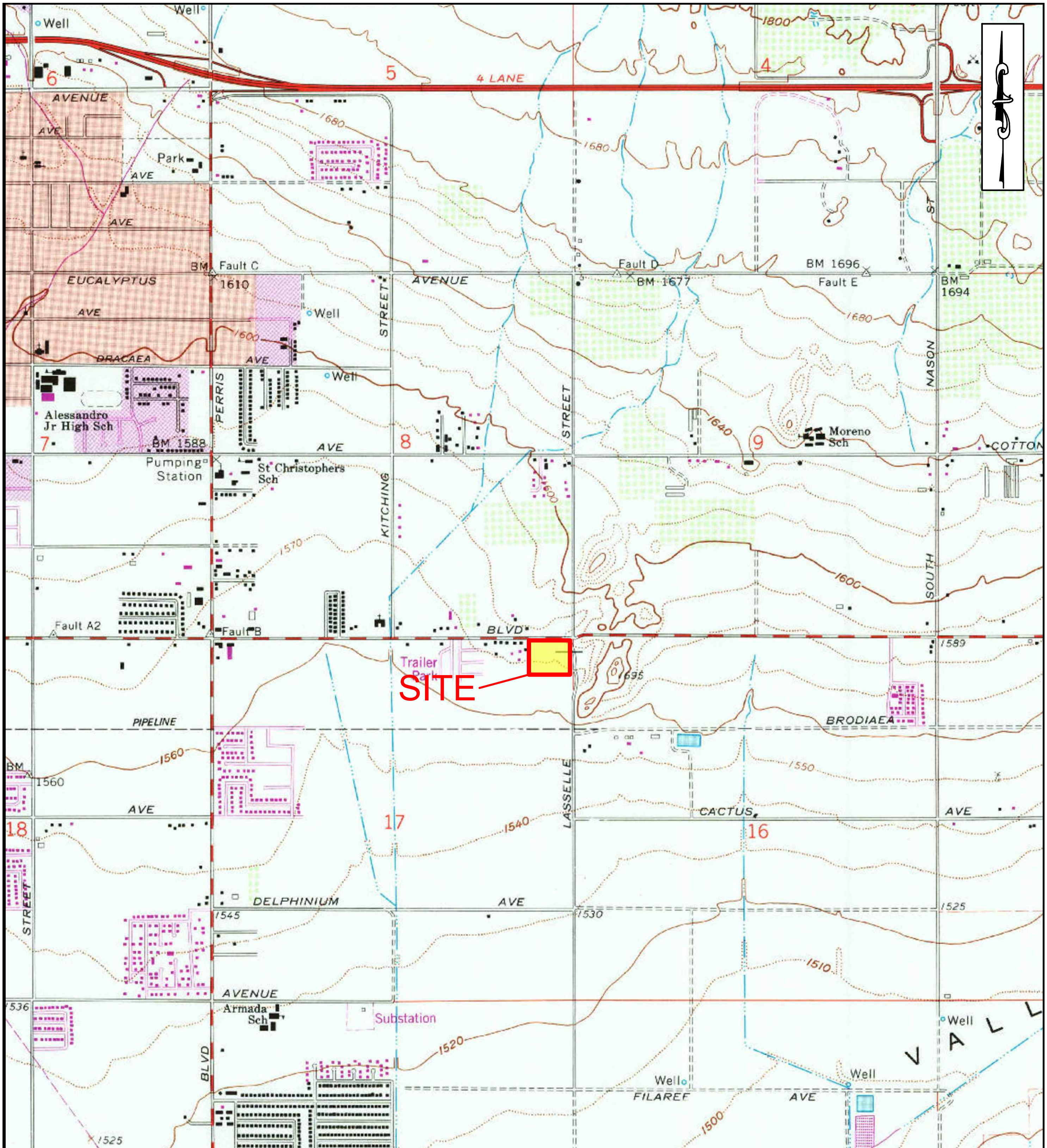
APPENDIX A

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



INDEX MAP

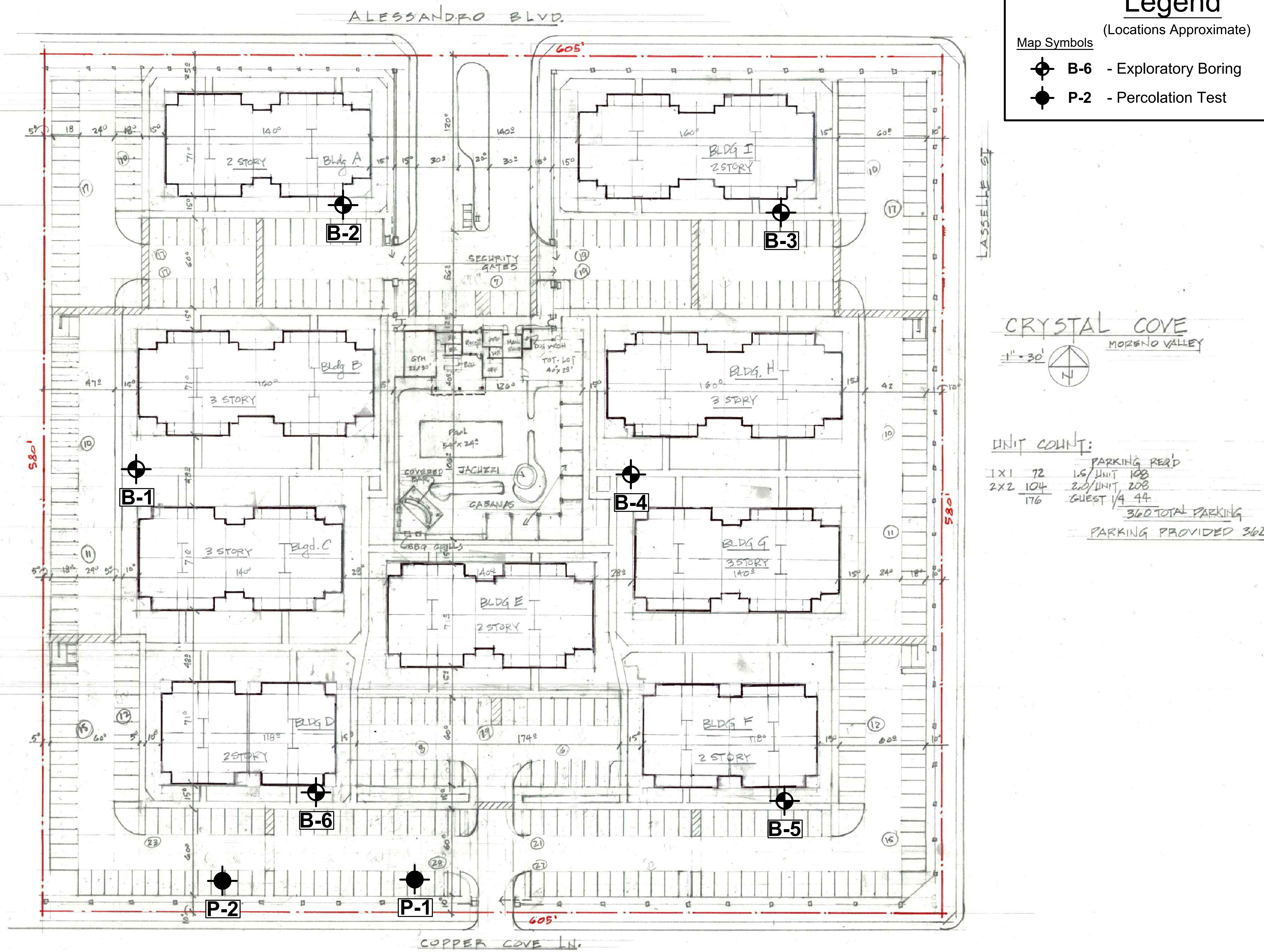
PROJECT:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	A-1
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2021
		SCALE:	1" ≈ 2,000'

Legend

(Locations Approximate)

Map Symbols

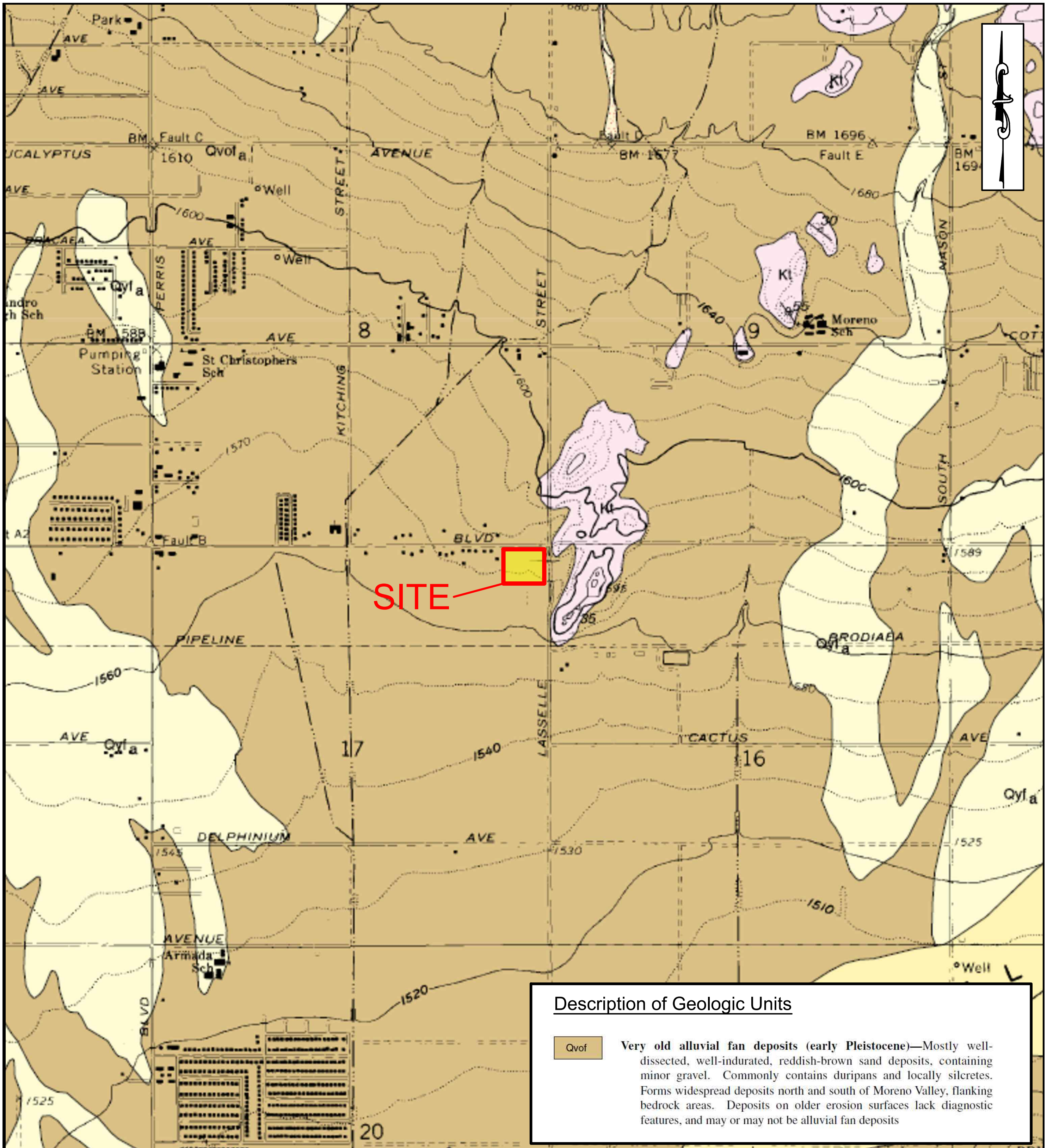
- **B-6** - Exploratory Boring
- **P-2** - Percolation Test



SITE PLAN

PROJECT NO:	33767.1
ENCLOSURE:	A-2
DATE:	October 2021
SCALE:	1" ≈ 18'
PROJECT:	Crystal Cove, Moreno Valley, California
CLIENT:	Empire CM, Inc.

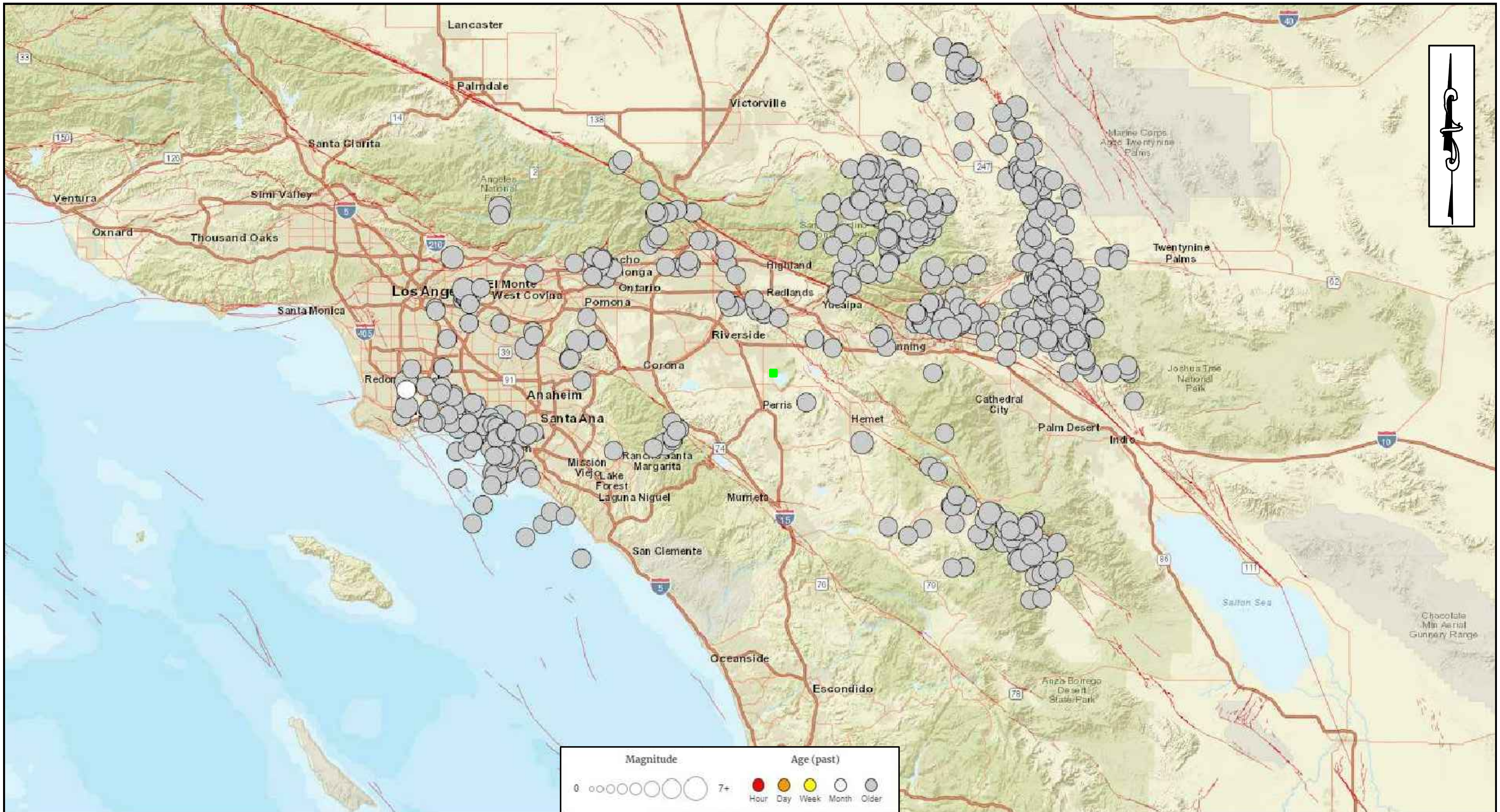
LOR GEOTECHNICAL GROUP, INC.



REGIONAL GEOLOGIC MAP

(Morton & Matti, 2001)

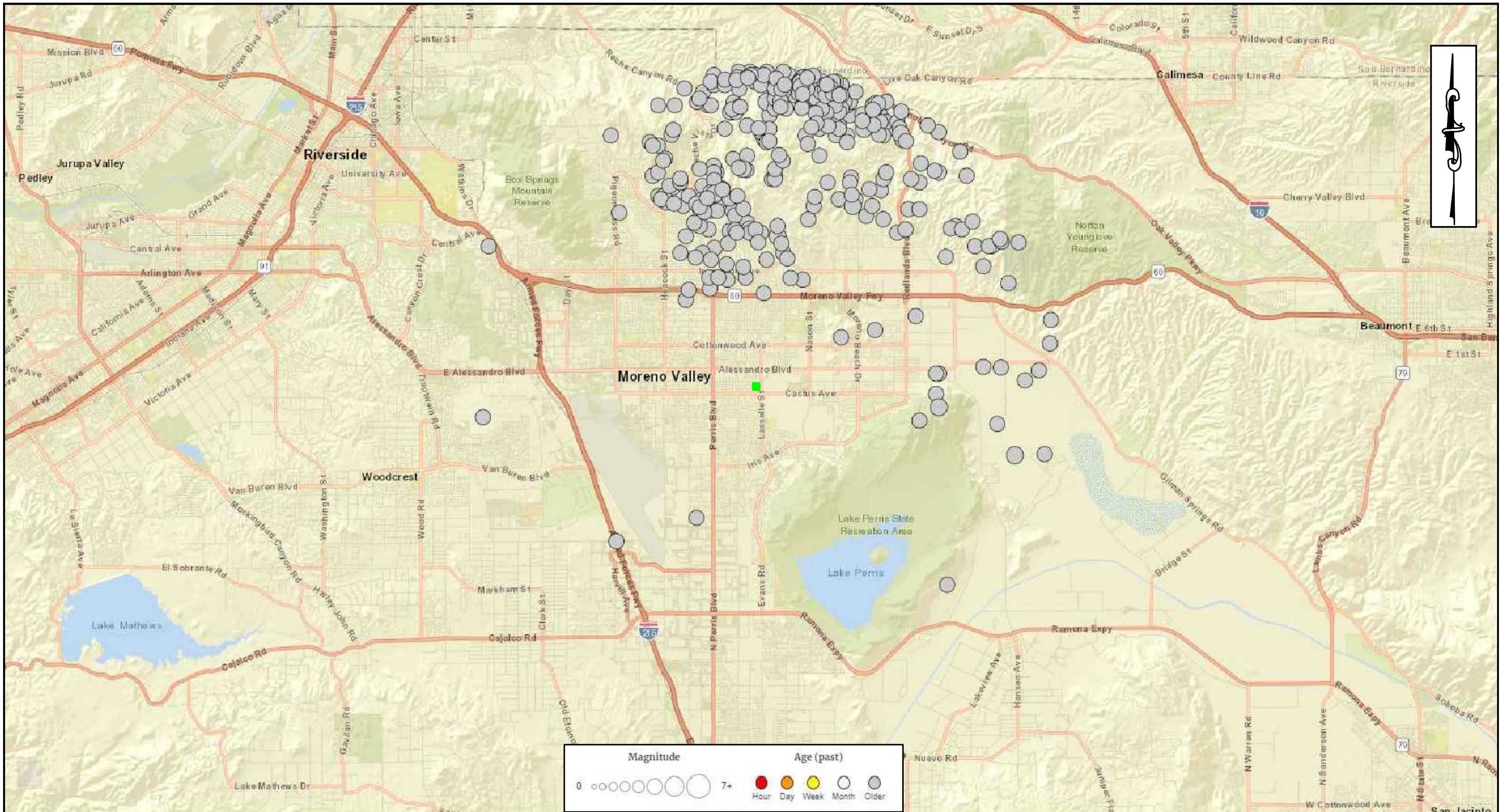
PROJECT:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	A-3
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2021
		SCALE:	1" ≈ 2,000'



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 541 epicenters of instrument-recorded events from 1932 to present (10/08/21) 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 evens are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 100km Radius

PROJECT:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	A-4
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2021
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2021) real-time earthquake epicenter map. Plotted are 333 epicenters of instrument-recorded events from 1932 to present (10/08/21) of local magnitude greater than M1.0 within a radius of ~6.2 miles (10 kilometers) of the site. Location accuracy varies. The site is indicated by the green square. The selected magnitude corresponds to a threshold intensity value where very light damage potential begins. These evens are also generally widely felt by persons. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 10km Radius

PROJECT:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	A-5
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2021
		SCALE:	1" ≈ 7km

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 15.33 and 30.33 feet below the existing ground surface using a Mobile B-61 drill rig on September 27, 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-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:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	B-i
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 2021

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:	Crystal Cove, Moreno Valley, California	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ENCLOSURE:**	B-ii
LOR GEOTECHNICAL GROUP, INC.		**DATE:**	October 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		3, 4, 7 9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	14	2	4.0		111.1				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 5% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 30% silty fines with trace clay, red-brown, damp.**
5	30		4.3		113.2				**@ 5 feet, becomes slightly coarser grained, decrease in porosity, some thin calcite stringers.**
	26		7.9		117.7				**@ 7 feet, remains slightly porous, increase in moisture.**
10	26		7.7		125.0			ML	**@ 10 feet, SANDY SILT, approximately 5% course grained sand, 15% medium grained sand, 20% fine grained sand, 60% fines with trace clay, red-brown, damp, trace pinhole porosity.**
	83 for 9"		4.5		120.0				@ 12 feet, BEDROCK: **TONALITE, course grained, speckled white-tan, damp, friable.**
15	46 for 5"		2.6		118.1				**@ 15 feet, becomes less weathered, remains friable.**
20	73 for 4"		2.4						**@ 20 feet, becomes dry, remains somewhat friable.**
									END OF BORING @ 20.33'
									Fill to 2' No groundwater Bedrock @ 12'
25									

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1572
		DATE DRILLED:	September 27, 2021
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-1

LOG OF BORING B-2

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY *(PCF)*	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	10		5.5		112.5				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines with trace clay, red-brown, damp, some pinhole porosity.**
5	18		9.0		115.5				**@ 5 feet, trace pinhole porosity.**
	23		7.9		116.3			SC	**@ 7 feet, CLAYEY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 30% clayey fines of low plasticity, red-brown, damp, trace pinhole porosity, trace thin calcite stringers.**
10	33		8.3		117.3				**@ 10 feet, becomes gray-brown, trace pinhole porosity, trace thin calcite stringers.**
15	65 for 11"		7.4		122.2			SM	**@ 15 feet, SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 20% silty fines, red-brown, damp.**
									@ 17 feet, BEDROCK: **becomes TONALITE, coarse grained, speckled white-tan, dry, friable.**
20	73 for 4"		1.9						**@ 20 feet, less friable.**
25	73 for 3"		2.3						**END OF BORING @ 25.25'** **Fill to 2'** **No groundwater** **Bedrock @ 17'**
30									

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1574
		DATE DRILLED:	September 27, 2021
LOR GEOTECHNICAL GROUP, INC.		**EQUIPMENT:**	Mobile B-61
		HOLE DIA.: 8"	**ENCLOSURE:** B-2

LOG OF BORING B-3

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0		9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	20		4.0		112.7				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 25% silty fines with trace clay, red-brown, damp, trace pinhole porosity.**
5	43		5.8		117.4				**@ 5 feet, SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 25% silty fines, red-brown, damp, micaceous.**
10	46 for 4"		2.8						@ 10 feet, BEDROCK: **TONALITE, coarse grained, speckled white-tan, dry, friable, rings disturbed.**
15	65 for 6"		3.0						**@ 15 feet, becomes slightly less weathered, remains friable.**
20	73 for 6"		4.2						**@ 20 feet, less weathered, slightly friable.**
25									

END OF BORING @ 20.5'

Fill to 2'
No groundwater
Bedrock @ 10'

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1576
		DATE DRILLED:	September 27, 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								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	21		3.4		113.3				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 30% silty fines with trace clay, red-brown, damp, some pinhole porosity, trace root hairs.**
5	31		5.1		113.7				**@ 5 feet, slightly coarser grained, porosity slightly larger than pinhole, some secondary calcite.**
10	66		7.3		119.1				**@ 10 feet, SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 20% silty fines, red-brown, damp, micaceous.**
									@ 12 feet, BEDROCK: **TONALITE, coarse grained, speckled white-tan, dry, friable.**
15	65 for 6"		3.5						
20	73 for 4"		2.6						**END OF BORING @ 20.33'** **Fill to 2'** **No groundwater** **Bedrock @ 12'**
25									

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1572
LOR GEOTECHNICAL GROUP, INC.		**DATE DRILLED:**	September 27, 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								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	7	2	5.2		114.0				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 25% silty fines with trace clay, red-brown, damp, some pinhole porosity.**
5	16	2	6.1		117.2				**@ 5 feet, SILTY SAND, approximately 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 15% silty fines, red-brown, damp, micaceous.**
	55		6.0		124.0				
									@ 7.5 feet, BEDROCK: **TONALITE, coarse grained, speckled white-tan, damp, friable.**
10	65 for 6"		3.8						
15	65 for 4"		2.7						
									END OF BORING @ 15.33' **Fill to 2'** **No groundwater** **Bedrock @ 7.5'**
20									

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1569
		DATE DRILLED:	September 27, 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		9, 10, 11						SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 35% fine grained sand, 20% silty fines, light brown, dry, loose, rodent burrows.**
	10	2	4.6		107.9				@ 2 feet, OLDER ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 20% medium grained sand, 35% fine grained sand, 30% silty fines with trace clay, red-brown, damp, some pinhole porosity.**
5	14		3.6		111.3				**@ 5 feet, pinhole porosity remains, trace root hairs.**
	20		7.3		118.6				**@ 7 feet, pinhole porosity and trace root hairs.**
10	37		5.2		122.2				**@ 10 feet, no visible pinholes, no root hairs, micaceous.**
	63 for 10"		11.8					CL	**@ 12 feet, LEAN CLAY with SAND, 5% coarse grained sand, 10% medium grained sand, 30% fine grained sand, 55% clayey fines of low plasticity, red-brown, damp.**
15	33		11.3						
20	71		12.1					SC	**@ 20 feet, CLAYEY SAND, approximately 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 25% clayey fines of low plasticity, red-brown, damp.**
									@ 23 feet, BEDROCK: **TONALITE, coarse grained, speckled white-black, dry, friable.**
25	73 for 3"		2.6						
30	77 for 4"		3.6						**END OF BORING @30.33'** **Fill to 2'** **No groundwater** **Bedrock @ 23'**
35									

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**ELEVATION:**	1568
		DATE DRILLED:	September 27, 2021
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, consolidation, expansion index, 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	(SM) Silty Sand	136.5	7.0

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	(SM) Silty Sand	450	25

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.

Consolidation Tests

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 Enclosures C-2 through C-5.

Expansion Index Test

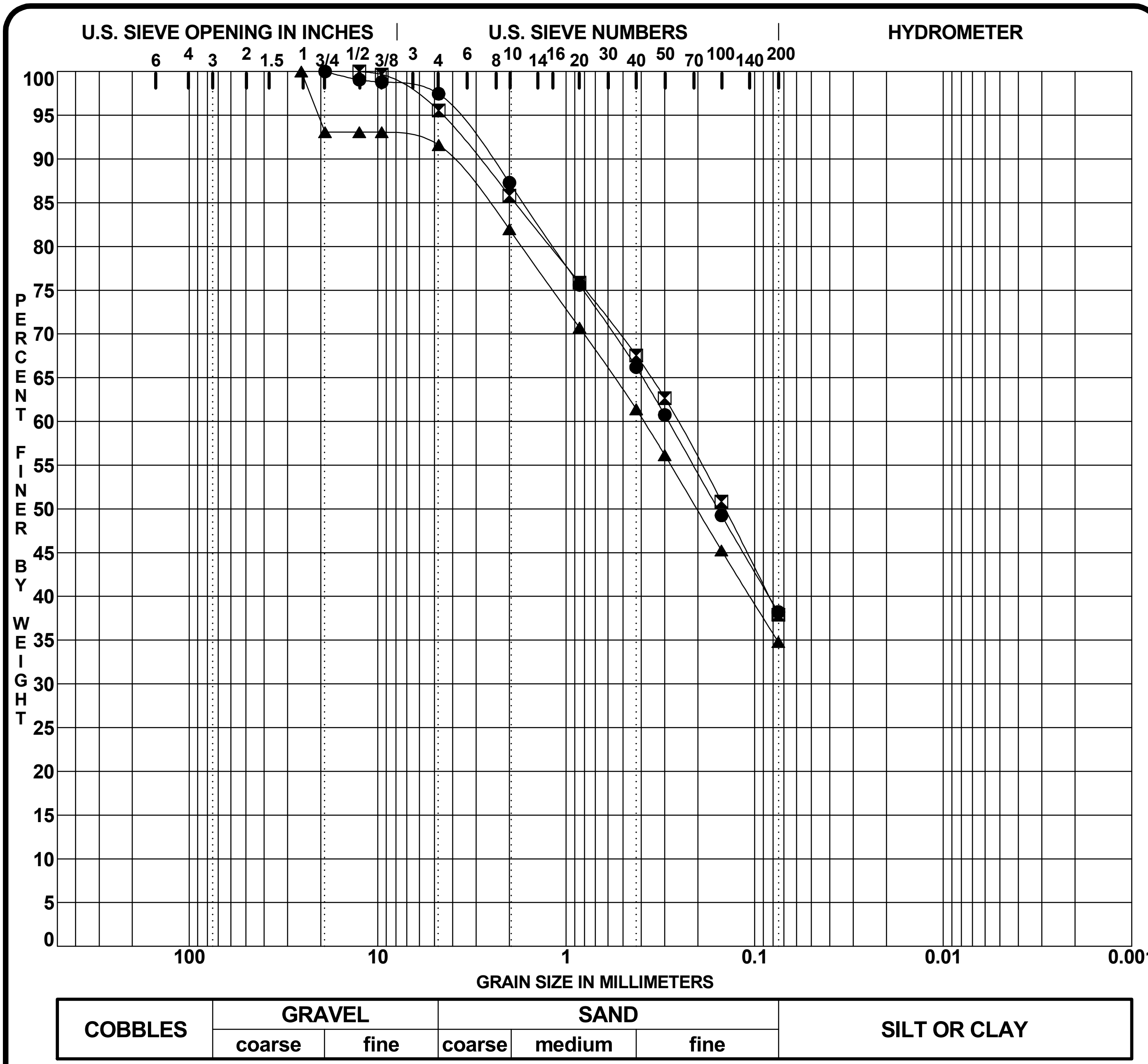
Remolded samples are tested to determine their expansion potential in accordance with the Expansion Index (EI) test. The test is performed in accordance with the Uniform Building Code Standard 18-2. The test result for a select soil sample is presented in the following table:

EXPANSION INDEX TEST				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-3	(SM) Silty Sand	11	Very Low
Expansion Index:	0-20 Very low	21-50 Low	51-90 Medium	91-130 High

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	(SM) Silty Sand	< 0.005
B-3	0-3	(SM) Silty Sand	< 0.005
B-6	0-3	(SM) Silty Sand	< 0.005



	Specimen Identification		Soil Classification	SE	RV	PL	PI	Cc	Cu
●	*B-1*	*@ 0-3'*	*(SM) Silty Sand*	*15*	*17*				
⧗	*B-3*	*@ 0-3'*	*(SM) Silty Sand*	*17*	*--*				
▲	*B-6*	*@ 0-3'*	*(SM) Silty Sand*	*18*	*--*				

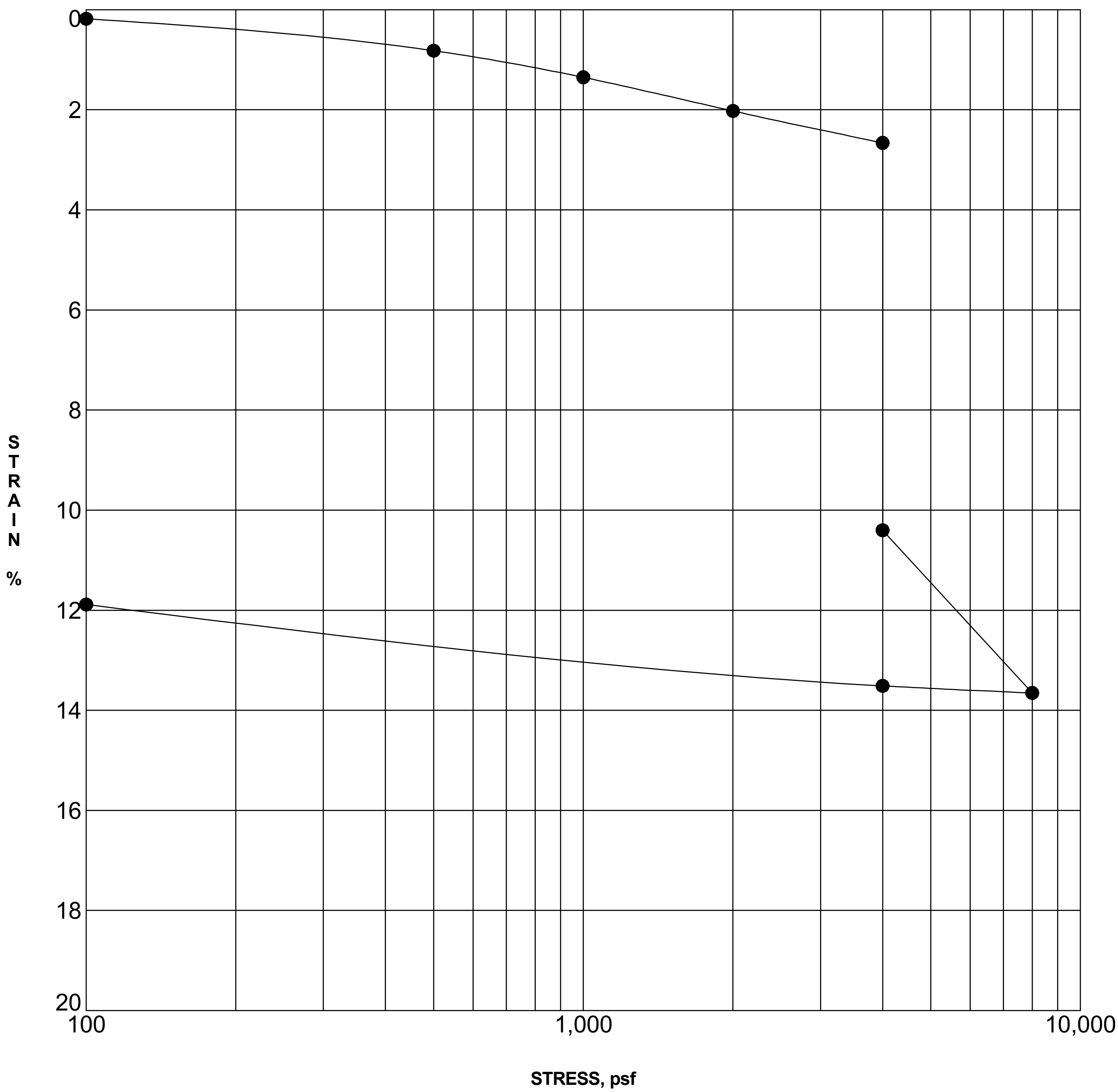
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*@ 0-3'*	*19.00*	*0.29*			*2.5*	*59.2*	*38.2*	
⧗	*B-3*	*@ 0-3'*	*12.50*	*0.26*			*4.5*	*57.6*	*37.9*	
▲	*B-6*	*@ 0-3'*	*25.40*	*0.39*			*8.4*	*56.8*	*34.8*	

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**DATE:**	October 2021

GRADATION CURVES

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-1



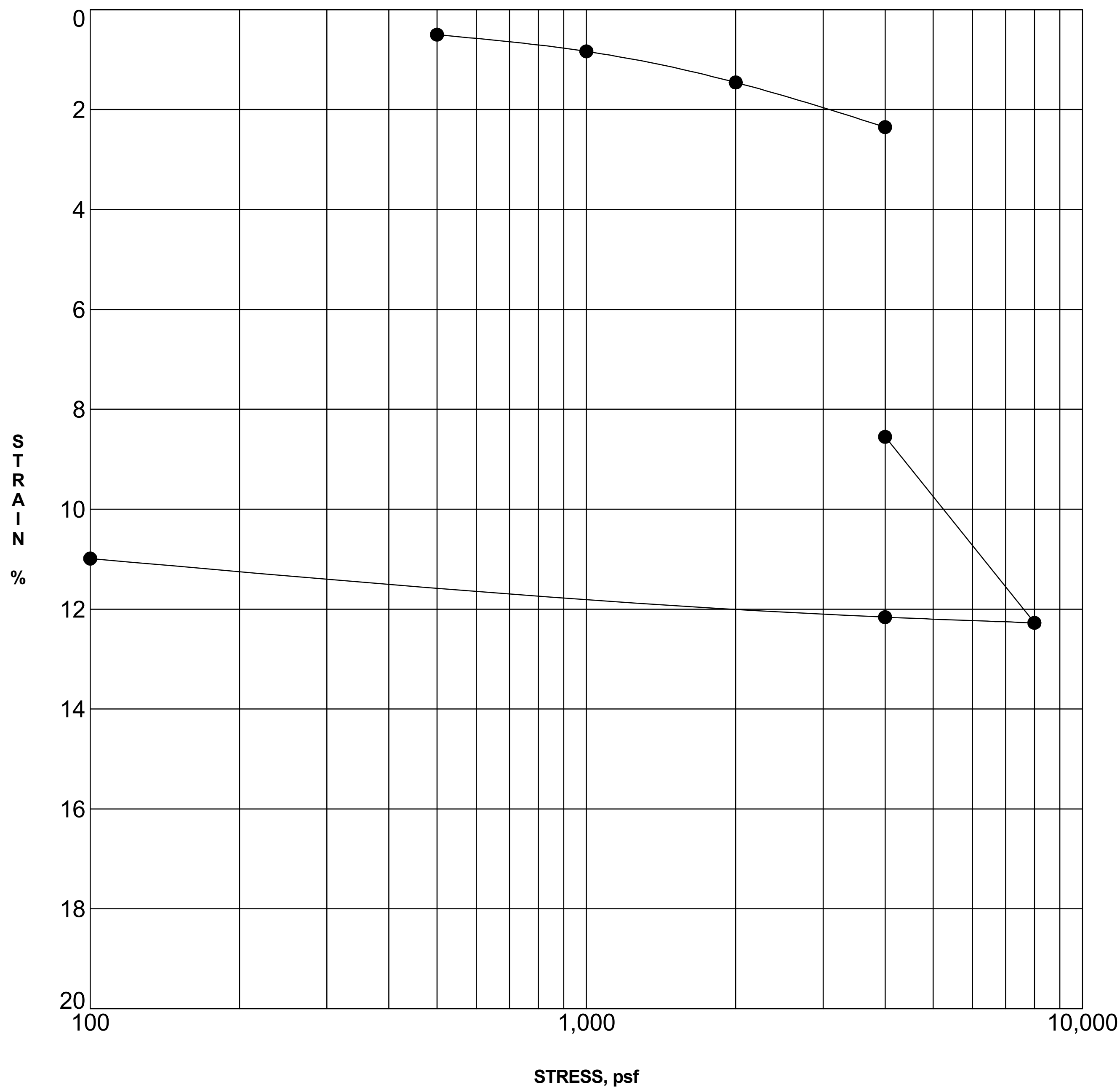
	Specimen Identification		Classification	DD	MC%
●	B-1	@ 2'	*(SM) Silty Sand*	106	4

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**DATE:**	October 2021

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-2



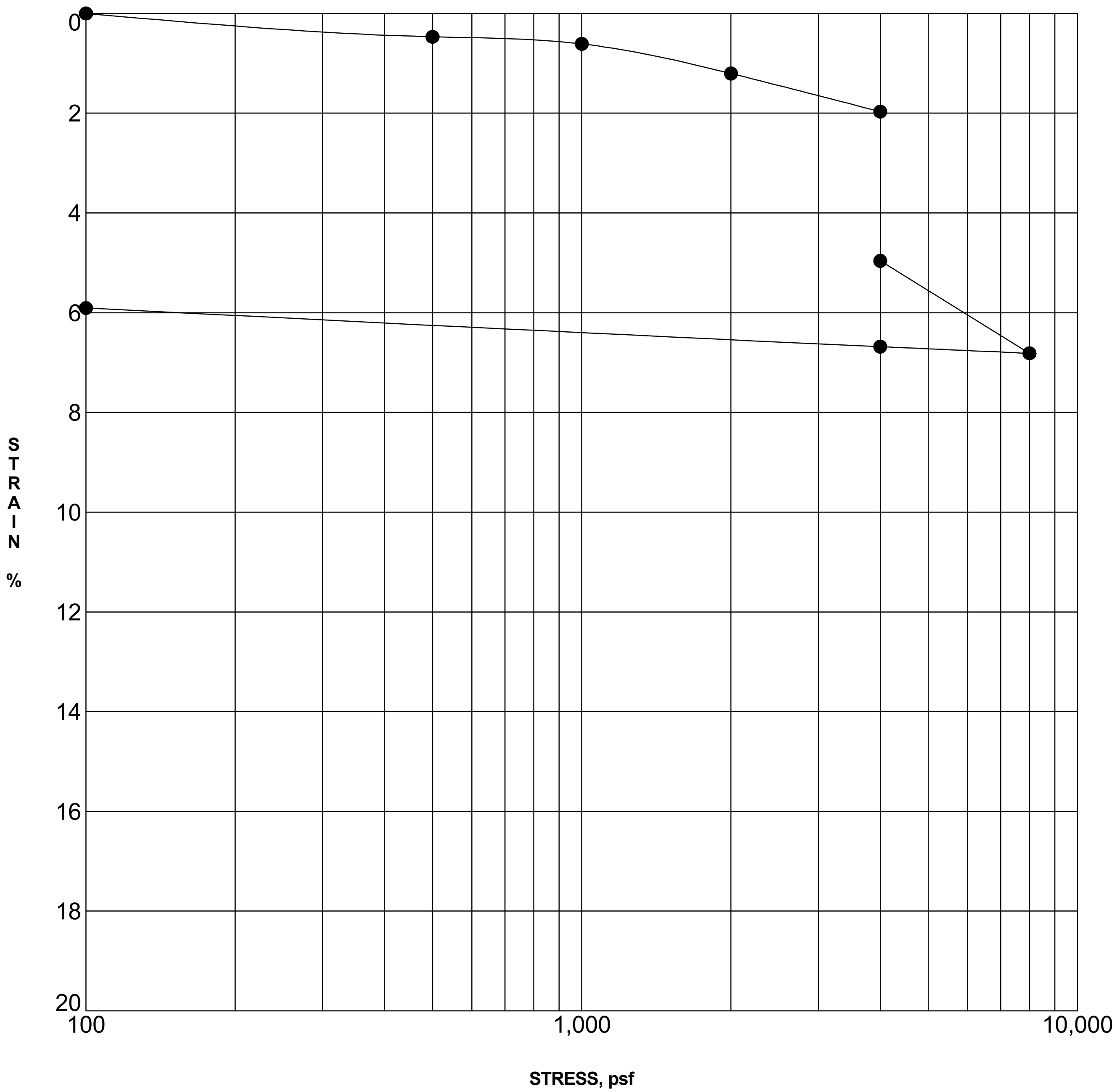
	Specimen Identification		Classification	DD	MC%
●	B-5	@ 2'	*(SM) Silty Sand*	111	5

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**DATE:**	October 2021

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-3



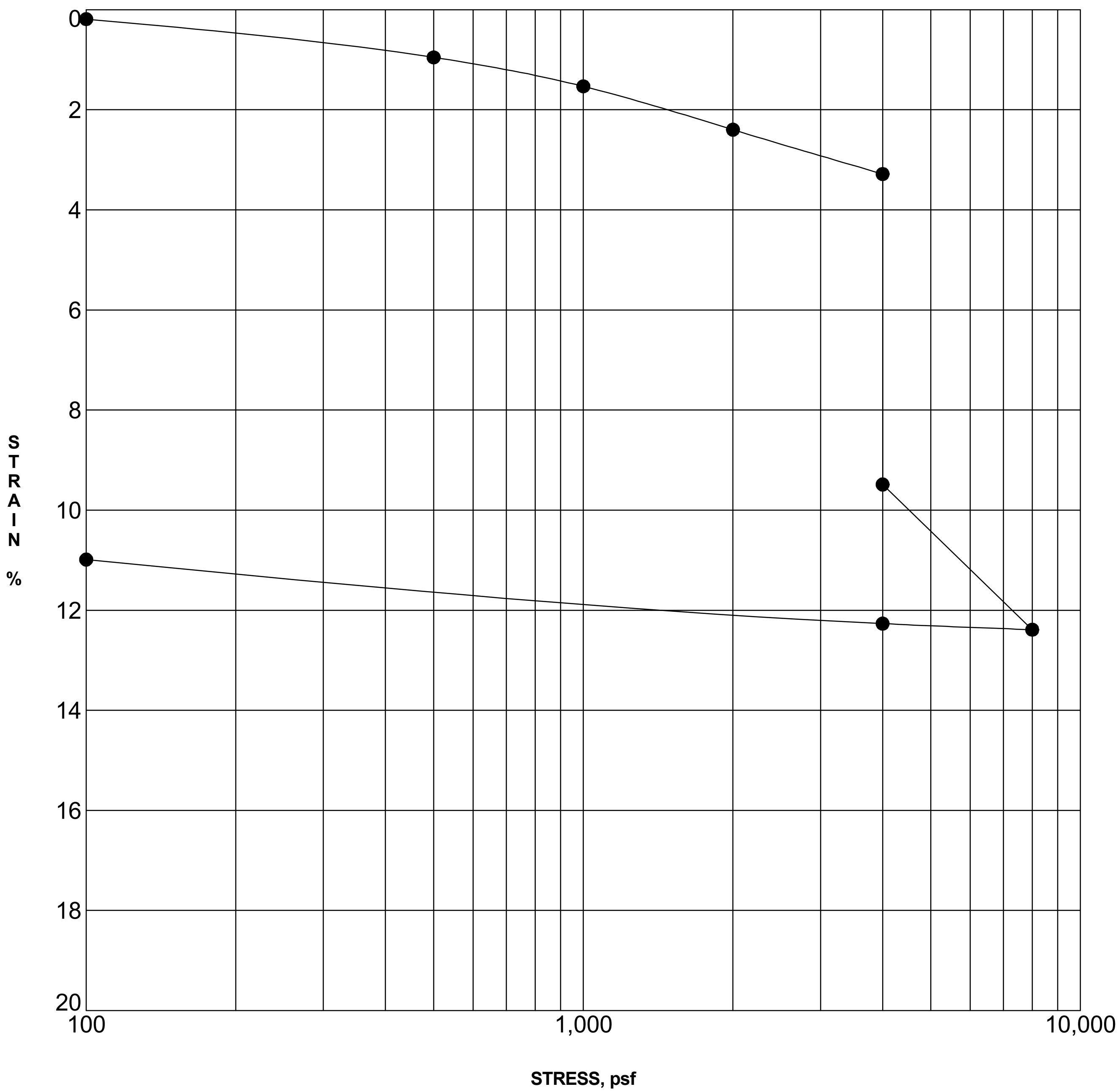
	Specimen Identification		Classification	DD	MC%
●	B-5	@ 5'	*(SM) Silty Sand*	110	6

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**DATE:**	October 2021

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-4



	Specimen Identification		Classification	DD	MC%
●	B-6	@ 2'	*(SM) Silty Sand*	108	5

PROJECT:	Crystal Cove	**PROJECT NO.:**	33767.1
CLIENT:	Empire CM, Inc.	**DATE:**	October 2021

CONSOLIDATION TEST

LOR GEOTECHNICAL GROUP, INC.

ENCLOSURE: C-5

APPENDIX D

Percolation Test Results

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project:	Crystal Cove, Moreno Valley	Test Date:	September 28, 2021
Project No.:	33767.1	Test Hole No.:	P-1
Soil Classificaiton:	(SM) Silty sand	Test Hole Diameter:	8.0 in.
Depth of Test Hole:	4.8 ft.	Date Excavated:	September 27, 2021
Tested By:	Andrew L.		

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	in/hr
1	9:07 AM	9:37 AM	30	0.50	0.50	25.00	44.00	57.60	57.00	19.00	22.80	38.0
2	9:37 AM	10:07 AM	30	0.50	1.00	26.00	44.00	57.00	57.00	18.00	22.00	36.0
3	10:07 AM	10:37 AM	30	0.50	1.50	26.00	42.00	57.00	57.00	16.00	23.00	32.0
4	10:37 AM	11:07 AM	30	0.50	2.00	26.00	42.00	57.00	57.00	16.00	23.00	32.0
5	11:07 AM	11:37 AM	30	0.50	2.50	25.00	42.00	57.00	57.00	17.00	23.50	34.0
6	11:37 AM	12:07 PM	30	0.50	3.00	26.00	42.00	57.00	57.00	16.00	23.00	32.0
7	12:07 PM	12:37 PM	30	0.50	3.50	27.00	43.00	57.00	57.00	16.00	22.00	32.0
8	12:37 PM	1:07 PM	30	0.50	4.00	26.00	42.50	57.00	57.00	16.50	22.75	33.0
9	1:07 PM	1:37 PM	30	0.50	4.50	25.00	42.00	57.00	57.00	17.00	23.50	34.0
10	1:37 PM	2:07 PM	30	0.50	5.00	27.00	43.00	57.00	57.00	16.00	22.00	32.0
11	2:07 PM	2:37 PM	30	0.50	5.50	26.00	42.00	57.00	57.00	16.00	23.00	32.0
12	2:37 PM	3:07 PM	30	0.50	6.00	26.00	42.00	57.00	57.00	16.00	23.00	32.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	31.00	
H_f	15.00	
ΔH	16.00	
H_{avg}	23.00	
I_t	**2.56**	in/hr (clear water rate)

BOREHOLE METHOD PERCOLATION TEST RESULTS

Project: Crystal Cove, Moreno Valley
Project No.: 33767.1
Soil Classificaiton: (SM) Silty sand
Depth of Test Hole: 4.9 ft.
Tested By: Andrew L.

Test Date: September 28, 2021
Test Hole No.: P-2
Test Hole Diameter: 8.0 in.
Date Excavated: September 27, 2021

READING	TIME START	TIME STOP	TIME INTERVAL		TOTAL TIME	INITIAL WATER LEVEL	FINAL WATER LEVEL	INITIAL HOLE DEPTH	FINAL HOLE DEPTH	CHANGE IN WATER LEVEL	AVERAGE WETTED DEPTH	PERCOLATION RATE
			min	hr.	hr.	in.	in.	in.	in.	in.	in.	in/hr
1	9:08 AM	9:38 AM	30	0.50	0.50	20.00	25.00	58.80	59.00	5.00	36.40	10.0
2	9:38 AM	10:08 AM	30	0.50	1.00	25.00	33.00	59.00	59.00	8.00	30.00	16.0
3	10:08 AM	10:38 AM	30	0.50	1.50	25.00	32.50	59.00	59.00	7.50	30.25	15.0
4	10:38 AM	11:08 AM	30	0.50	2.00	25.00	32.00	59.00	59.00	7.00	30.50	14.0
5	11:08 AM	11:38 AM	30	0.50	2.50	24.00	31.00	59.00	59.00	7.00	31.50	14.0
6	11:38 AM	12:08 PM	30	0.50	3.00	24.00	29.00	59.00	59.00	5.00	32.50	10.0
7	12:08 PM	12:38 PM	30	0.50	3.50	26.00	33.00	59.00	59.00	7.00	29.50	14.0
8	12:38 PM	1:08 PM	30	0.50	4.00	26.00	33.50	59.00	59.00	7.50	29.25	15.0
9	1:08 PM	1:38 PM	30	0.50	4.50	26.00	34.00	59.00	59.00	8.00	29.00	16.0
10	1:38 PM	2:08 PM	30	0.50	5.00	27.00	35.00	59.00	59.00	8.00	28.00	16.0
11	2:08 PM	2:38 PM	30	0.50	5.50	25.00	33.00	59.00	59.00	8.00	30.00	16.0
12	2:38 PM	3:08 PM	30	0.50	6.00	26.00	34.00	59.00	59.00	8.00	29.00	16.0

PERCOLATION RATE CONVERSION (Porchet Method):

H_O	33.00	
H_f	25.00	
ΔH	8.00	
H_{avg}	29.00	
I_t	**1.03**	in/hr (clear water rate)