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

PRELIMINARY GEOTECHNICAL AND INFILTRATION FEASIBILITY INVESTIGATION PROPOSED RETAIL DEVELOPMENT APN's 404-190-001& -003 NORTHWEST CORNER OF BEAUMONT AVENUE AND OAK VALLEY PARKWAY BEAUMONT, CALIFORNIA

PROJECT NO. 13627.1
APRIL 7, 2020

Prepared For:

Santiago Holdings, LLC
c/o Thatcher Engineering & Associates
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Attention: Ms. Kristin Tissot

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Attention: Ms. Kristin Tissot

Subject: Preliminary Geotechnical and Infiltration Feasibility Investigation, Proposed Retail Development, APN's 404-190-001 and -003, Northwest Corner of Beaumont Avenue and Oak Valley Parkway, Beaumont, California.

LOR Geotechnical Group, Inc., is pleased to present this report summarizing our geotechnical investigation for the above referenced 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.

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. All fill/topsoil material and any loose alluvial materials should be removed from structural areas and areas to receive engineered compacted fill. The data developed during this investigation indicates that removals on the order of 3 to 5 feet from existing grades will be required to encounter competent native materials within the majority of the proposed development portion of the site and that removals on the order of 10 to 14 feet from existing grades will be required to encounter competent native materials within areas of the previous fault trenching that had been conducted by others. The given removal depths are preliminary. The actual depths of the removals should be determined during the grading operations by observation and/or in-place density testing.

Very low expansive soils and poor R-value quality soils were encountered on the site. A negligible sulfate content was found for the soils tested. Near completion and/or at the completion of site grading, additional foundation and subgrade soils should be tested to further evaluate their expansion potential, soluble sulfate content, and R-value quality.

Variable 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

Page No.

INTRODUCTION

During March and April of 2020, a Preliminary Geotechnical and Infiltration Feasibility Investigation was performed by LOR Geotechnical Group, Inc. for proposed retail development of APN's 404-190-001 and -003 in the City of Beaumont, California. The purpose of this investigation was to conduct a technical evaluation of the geologic setting of the site and to provide geotechnical design recommendations for the proposed improvements. The scope of our services included:

- Review of available geotechnical literature, reports, maps, and agency information pertinent to the study area;
- Interpretation of aerial photographs of the site and surrounding region dated 1966 through 2018;
- Geologic field reconnaissance mapping to verify the areal distribution of earth units and significance of surficial features as compiled from the reviewed documents, literature, and reports;
- A subsurface field investigation to determine the physical soil conditions pertinent to the proposed development;
- Infiltration testing via the double ring infiltrometer method.
- Laboratory testing of selected soil samples obtained during the field investigation;
- Development of geotechnical recommendations for site grading and foundation design; and
- Preparation of this report summarizing our findings and providing conclusions and recommendations for site development.

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

To orient our investigation at the site, you provided us with a Site Plan, prepared by Thatcher Engineering & Associates, Inc., dated February 18, 2020, that showed the proposed development. As noted on that map, the site will be developed with seven retail structures, including a gas station, and the associated improvements. The Site Plan was utilized as a base map for our field investigation and is presented as Enclosure A-2, within Appendix A.

PROJECT CONSIDERATIONS

The proposed structures are anticipated to be one-story in height and are anticipated to be of wood or steel frame construction with an exterior plaster veneer or of concrete construction. Light to moderate foundation loads are anticipated with such structures. Cuts up to 6± feet and fills up to 10± feet are proposed for the project (Thatcher, 2020).

EXISTING SITE CONDITIONS

The subject site consists of approximately 10± acres of vacant land with an irregular outline, located along the west side of Beaumont Avenue and the north side of Oak Valley Parkway. The topography of the proposed development area of the site is relatively planar and consists of a gentle slope to the west. However, within the southwest portion of the site, a small topographic high is present. Within the northern and western portion of the property, where no current development is proposed, the area consists of a natural drainage (Marshall Creek) with near vertical slopes up to approximately 20 feet in height, locally. This active, incised channel area contains a light growth of shrubs. The remainder of the site contains a light growth of weeds with large trees along the eastern and southeastern boundaries. A small concrete slab was present in the southeast corner of the site. Cross fencing was present in the northeast portion of the site.

The site is bound on the north and west by vacant land, similar to the site. A shopping center lies east of the site, across Beaumont Avenue, a partially improved roadway. Across Oak Valley Parkway, a partially improved roadway to the south, are large lot single family residences.

AERIAL PHOTOGRAPH ANALYSIS

The aerial photographs reviewed consisted of vertical aerial photographs of varying scales. We reviewed imagery available from Google Earth (2020) and from Historic Aerials (2020).

In summary, the site contained two residences with small outbuildings in the northeast and southeast corners in 1966, the earliest photograph available. The structures in the southeast corner were gone by the time of the 1972 photograph and the structures in the northeast corner were gone by the time of the 2003 photograph.

PREVIOUS GEOLOGIC REPORT

A previous geologic investigation was conducted for the site by Earth Systems Southwest in 2016. At that time, the proposed development of the site was a shopping center, similar to that which is currently proposed. Their report was conducted to address potential fault-rupture hazard at the site. In brief summary, their work consisted of reviewing previous geologic work conducted for the site and excavating 7 trenches to depths of approximately 10 to 12 feet. Trenches ranged in length from approximately 58 to 284 feet for a total of approximately 920 feet. The trenches were T-shaped and approximately 19 feet wide at the top, with 7.5 foot wide benches on either side of the deepest 4 foot wide portion of the trench. The report also states that Salem performed a 630 foot long fault trench in 2007. The depth of this trench was approximately 10 to 14 feet. Based on their investigation, Earth Systems Southwest recommended structural setbacks for habitable structures of approximately 25 to 50 feet beyond each side of the two mapped active fault zones, mapping of the fault zone locations during mass grading to confirm the fault zones presented, and removal and replacement of the backfill for both Salem's and Earth Systems Southwest's fault trenches. The location of Earth Systems Southwest's fault trenches and subsequent structural setback zones were surveyed by Tuttle Engineering. Salem's fault trench location was reportedly recorded based on hand held gps data.

FIELD EXPLORATION PROGRAM

Our subsurface field exploration program was conducted on March 12th and March 24th, 2020 and consisted of drilling 9 exploratory borings with a truck-mounted Mobile B-61 drill rig and a track mounted drill rig, both equipped with 8-inch diameter hollow stem augers. The borings were drilled to depths of approximately 16.5 to 51.5 feet below the existing ground surface. The approximate locations of our exploratory borings are presented on the attached Site Plan, Enclosure A-2 within Appendix A.

The subsurface conditions encountered in the exploratory borings were logged by a geologist from this firm. Relatively undisturbed and bulk samples were obtained 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 field exploration program and the boring logs are presented in Appendix B.

LABORATORY TESTING PROGRAM

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

GEOLOGIC CONDITIONS

Regional Geologic Setting

The subject site is located within the City of Beaumont, which in turn is situated along the junction of two major geomorphic provinces of southern California, or at the end of the Peninsular Ranges geomorphic province where it meets the Transverse Ranges geomorphic province. The Peninsular Ranges include a series of small northwestern trending mountains, separated by wide flat valleys, that extend from the Los Angeles region southeastward into Baja California. The northern margin of this province butts up against a series of mountain ranges that lie in a transverse direction to this normal northwestern trend, or extend east and west. These mountains include the Santa Monica Mountains, the San Gabriel Mountains, and the San Bernardino Mountains. In the Beaumont locality these two major provinces are termed the Peninsular Ranges Block to the south and the San Bernardino Mountains Block to the north and are separated by a series of complex faults known collectively as the San Andreas fault zone. In this tectonically complex area the Peninsular Ranges Block is generally sliding to the northwest, while partially being thrust underneath the San Bernardino Mountains Block. Therefore, the resulting faults end up with a complex mix of strike slip and thrust faults. The Banning and San Andreas faults, located to the north and northeast of the site, as well as the San Jacinto fault to the southwest, are all strike slip faults with right lateral offsets. However, the San Gorgonio Pass fault zone, which lies to the east of the site, is a result of compressional thrust faulting. In addition, differential movement along these faults has even led to the formation of the Beaumont Plains fault zone which is a series of tension, or pull apart, faults which traverse the site (Earth Systems, 2016). Therefore, the topography of the land in this region has been drastically altered by differing tectonic forces. The subject site itself is located along the central portion of a relatively broad fan-like surface known as the Beaumont Plain which extends from Calimesa southeast to Banning. This plain, along with the Badlands to the west, is the result of erosion of the mountains onto a valley like setting, then uplifting and offsetting of the region by interactions along and between the San Andreas and San Jacinto faults, then continued offsetting of the region along the San Andreas fault, and finally incising by drainages flowing from the northeast to the southwest.

In the vicinity of the subject site, along the southern edge of the Beaumont valley, the basement rocks in this region are considered to be igneous and metamorphic crystalline rocks, none of which are exposed at the subject site. These rocks have been covered with a relatively thick sequence of sedimentary rocks of the San Timoteo Formation, that outcrop along the flanks of the San Gorgonio Mountains to the south, and older alluvial sediments deposited from streams flowing to the north and northwest from the mountains to the south.

The geology of the site and immediate surrounding region as mapped by Dibblee (2003) is shown on Enclosure A-3, within Appendix A.

Site Geologic Conditions

Fill/Topsoil: As encountered within our exploratory borings placed at the site, fill/topsoil materials to a depth of 1 foot are present. These materials mainly consist of silty sand. The fill/topsoil materials are a result of past and current weed abatement practices (discing). Deeper fill materials are present, mainly associated with the previous fault trenching by others, and in the general areas of the previously noted residences and outbuildings.

Older Alluvium: Underlying the fill/topsoil materials within the currently proposed development portion of the site, older alluvial materials were encountered underlying the fill/topsoil materials noted above within our exploratory borings. These units primarily consist of silty sand with lessor units of well graded sand, sandy silt, poorly graded sand with silt, well graded sand with silt, and lean clay with sand to clayey sand. The older alluvial materials were in a relatively medium dense state 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 indicated normal consolidation characteristics.

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

Groundwater Hydrology

Groundwater was not encountered within our exploratory borings advanced to a maximum depth of approximately 50 feet below the existing ground surface.

Records for nearby wells which were readily available from the State of California Department of Water Resources online database (CDWR, 2020) were reviewed as a part of this investigation. This database indicates that the nearest state water wells are numbers 02S01W33R001S and -002S which are located approximately 0.6 kilometers (0.3 miles) to the west. These wells lie at elevations of approximately 2,602 and 2,622 feet above mean sea level (m.s.l.), respectively. Only one recorded groundwater measurement was available for well number -001S. The record indicates that groundwater in this well lied at a depth of 363 feet in October of 1982. Records for well number -002S were available from April of 2005 to October of 2010. Groundwater is noted to be getting sightly deeper over that time from a depth of approximately 384 feet in 2005 to a depth of approximately 414 feet in 2010.

As illustrated on Enclosure A-2, the lowest elevation at the site is approximately 2,614 feet above m.s.l. in the northwest portion of the currently proposed development area. Based on the information above, groundwater is anticipated to be at a depth of approximately 375 feet.

Surface Runoff

Current surface runoff of precipitation waters across the site is generally as sheet flow to the on-site drainage (Marshall Creek) which in turn flows to the southwest.

Mass Movement

The majority of 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. However, the over steepened slopes adjacent to the existing Marshal Creek in the northeast portion of the site were noted to exhibit some local surficial mass movement in the form of wedge failures resulting from undercutting of the steep banks.

Faulting

Based on the study by Earth Systems Southwest, known active faults exist at the site (2016). Recommendations were provided by Earth Systems Southwest which included set back zones for habitable structures from the two active faults identified.

As previously noted, the subject site lies near the middle of a large wedge shaped area in between the San Jacinto fault, located approximately 10 kilometers (6.2 miles) to the southwest, and the San Andreas fault which lies approximately 11.5 kilometers (7 miles) to the northeast. Both of these faults are considered to be major active faults which move in a right lateral strike-slip fashion with relative movement of the fault such that the southwest side moves northwest and the northeast side moves southeast during earthquakes. The San Andreas fault is considered to be the major tectonic feature of California, separating the Pacific plate and the North American plate. While estimates vary, the San Andreas fault is generally thought to have an average slip range on the order of 24 mm/yr and capable of generating large magnitude events on the order of 7.5 or greater.

The San Jacinto fault zone is a sub-parallel branch of the San Andreas fault zone, extending from the northwestern San Bernardino area, southward into the El Centro region. It is believed that the San Jacinto fault zone has an average slip rate of about 12 mm/year and is capable of producing an earthquake magnitude on the order of 6.5 or greater.

Lying in between these two faults are numerous smaller faults with varying types of motion. Perhaps the largest of these, based on length and estimated amounts of past displacement in the region around the site, is the Banning fault. Based on mapping conducted by the U.S.G.S., the Banning fault bifurcates off of the San Andreas fault just north of Indio, then extends through the Banning-Beaumont pass area and into the Calimesa area. Some authors refer to this portion of this fault across the Beaumont region as the San Gorgonio Pass fault system (Matti et al, 1992). These faults generally extend along the base of the hills north of Beaumont, approximately 3 to 5 kilometers (2 to 3 miles) north of the site.

Differential movement along the faults discussed above has resulted in the extension of the Beaumont Plain. This has created a series of northwest trending en-echelon fault scarps, referred to as normal faults, in which the center block drops downward along an irregular fault line. These have collectively been named the Beaumont Plain fault zone, which may have some thrusting motion along the northern portions. The motions and activities of these faults are poorly understood. However, the study by Earth Systems

Southwest, (2016) indicates that two of these faults are active and that they traverse the site.

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

Historical Seismicity

In order to obtain a general perspective of the historical seismicity of the site and surrounding region a search was conducted for seismic events at and around the area within various radii. This search was conducted utilizing the historical seismic search website of the U.S.G.S. (2020). This website conducts a search of a user selected cataloged seismic events database, within a specified radius and selected magnitudes, and then plots the events onto a map. At the time of our search, the database contained data from January 1, 1932 through April 2, 2020.

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

In the second search, the micro seismicity of the area lying within a 10 kilometer (6.2 mile) radius of the site was examined by selecting an epicenter map listing events on the order of 2.0 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 km. The results of this search is a map that presents the seismic history around the area of the site with much greater detail, not permitted on the larger map. The reason for limiting the events to the last 40± years on the detail map is to enhance the accuracy of the map. Events recorded prior the mid 1970's are generally considered to be less accurate due to advancements in technology. As depicted on this map, Enclosure A-5, the subject site lies within an area underlain by very numerous small events in the general area. No specific trend of the events is noted. In contrast, these events are scattered across the entire region with the closest event, perhaps, occurring under the site. This would be the anticipated result of numerous earthquake faults with dipping fault planes. This factor allows the earthquake focus to spread out across a wide area.

In summary, the historical seismicity of the site entails numerous small to medium magnitude earthquake events occurring around the subject site, predominately associated with the presence of the faults described within. Any future developments at the subject site should anticipate that moderate to large seismic events could occur very near the site.

Secondary Seismic Hazards

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

Liquefaction: The potential for liquefaction generally occurs during strong ground shaking within loose granular sediments where the depth to groundwater is usually less than 50 feet. As groundwater is thought to be in excess of 50 feet beneath the site and the site is underlain by relatively dense to very dense older alluvium, the possibility of liquefaction within these units 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 the 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 upstream near the site which could possibly rupture during an earthquake and affect the site by flooding.

Seismically-Induced Landsliding: As previously mentioned, evidence for past surficial mass movement of the incised bank of the on-site Marshall Creek was noted to be present locally. However, due to the relatively low relief of the site and adjacent surrounding region, the potential for deep seated landslides to occur at the site is considered nil.

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

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

SOILS AND SEISMIC DESIGN CRITERIA (California Building Code 2019)

Design requirements for structures can be found within Chapter 16 of the 2019 California Building Code (CBC) based on building type, use and/or occupancy. The classification of use and occupancy of all proposed structures at the site, and thus the design requirements, shall be the responsibility of the structural engineer and the building official. For structures at the site to be designed in accordance with the provisions of Chapter 16, the subject site specific criteria is provided below:

Site Classification

Chapter 20 of the ASCE 7-16 defines six possible site classes for earth materials that underlie any given site. Bedrock is assigned one of three of these six site classes and these are: A, B, or C. Per ASCE 7-16, Site Class A and Site Class B shall be measured on-site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Site Class A and Site Class B shall not be used if more than 10 feet of soil is between the rock surface and bottom of the spread footing or mat foundation. Site Class C can be used for very dense soil and soft rock with values greater than 50 blows per foot. Site Class D can be used for stiff soil with values ranging from 15 to 50 blows per foot. Site Class E is for soft clay soils with values less than 15 blows per foot. Our Standard Penetration Test (SPT) data indicate that the materials beneath the site are considered Site Class D soils.

CBC Earthquake Design Summary

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

INFILTRATION TESTING AND TEST RESULTS

Eight double ring infiltration tests were conducted at the general locations and depths requested. The locations are illustrated on Enclosure A-2. Test pits were excavated to depths ranging from approximately 6 to 14 feet below the existing ground surface and a 12-inch diameter casing was installed within the center of the test locations with a 24-inch diameter casing centered around it. Each 20-inch tall casing was imbedded to a depth of approximately 3.5-inches. The test locations were tested immediately after the casings were installed by filling both the inside and outside casings and maintaining a water level to depths ranging from approximately 3 to 5.5 inches.

The testing procedure was as follows:

Both the inside and outside areas of the casings were filled with water to a level of approximately 3 to 5.5 inches above the ground surface. Water was then metered to maintain this water level within both rings. The volume of water use in a given time period was recorded at various time intervals to establish the infiltration rate of the water within the inner ring.

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

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

Test No.	Depth (ft)*	Clear Water Infiltration Rate** in/hr
DRI-1	6	3.1
DRI-2	10	1.5
DRI-3	12	0.9
DRI-4	12	0.4
DRI-5	12	0.3
DRI-6	11	0.3
DRI-7	11	8.9
DRI-8	14	4.0
* depth measured below existing ground surface ** final reading		

The results of our infiltration testing are attached as Enclosures D-1 through D-8.

CONCLUSIONS

General

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

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

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

Foundation Support

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

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

Soil Expansiveness

Our laboratory testing found that the soils tested have a very low expansion potential. Therefore, conventional design and construction should be applicable for the project.

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

Sulfate Protection

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

Geologic Mitigations

Marshall Creek is presented as a partially incised natural drainage along the north and western portion of the site. This has resulted in near vertical slopes up to approximately 20 feet high. The taller portion of these over steepened slopes were noted to exhibit some failure as soil fall. While the active portion of the drainage is not currently directed towards these, during heavy rainfall and/or flooding events, continued erosion may occur.

Seismicity

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

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

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

RECOMMENDATIONS

Geologic Recommendations

Provisions should be made to protect the area of the site adjacent to the existing southern side of the on-site Marshall Creek. Such measures may include: channelization of the creek and/or slope protection to mitigate any future erosion. Based on our review of the Site Plan provided, currently proposed structures appear to set back a significant distance from the existing over steepened slope. Should the orientation of the structures change, this firm should be provided plans so that we may access any potential impacts.

The Restricted Use Zone, for habitable structures previously established by Earth Systems Southwest (2016) should be incorporated into the project.

During site rough grading, all removals and over-excavation bottom areas should be observed by the project engineering geologist in order to evaluate the geologic conditions exposed. Although not anticipated, widening of, or additions to, the established Restricted Use Zones is a possibility should evidence for active faulting near the limits or outside of the zone be encountered.

General Site Grading

It is imperative that no clearing and/or grading operations be performed without the presence of a qualified geotechnical engineer. An on-site, pre-job meeting with the owner, the developer, the contractor, and geotechnical engineer should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

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

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

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

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

Initial Site Preparation

Any and all existing uncontrolled fill/topsoil, fault trench backfill, and loose/soft native alluvial soils should be removed from structural areas and areas to receive structural fills. The data developed during this investigation indicates that within the majority of the site, removals on the order of 3 to 5 feet from existing grades will be required to encounter competent native materials. However, deeper removals on the order of 10 to 14 feet will be required within the areas of fault trenches excavated by others (Earth Systems Southwest, 2016). Removals should extend horizontally at a distance equal to the depth of the removals plus proposed fill and at least a minimum of 5 feet. The actual depths of removals should be determined during the grading operation by observation and/or by in-place density testing.

As previously discussed, all removal and over-excavation bottom areas should be observed by the project engineering geologist prior to processing and/or fill placement.

Preparation of Fill Areas

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

Preparation of Shallow Foundation Areas

All footings should rest upon a minimum of 24 inches of properly compacted fill material placed over competent natural alluvial soils. In areas where the required fill thickness is not accomplished by the removal of unsuitable soils, the footing areas should be further subexcavated to a depth of at least 24 inches below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing lines. The bottom of this excavation should then be scarified to a depth of at least 6 inches, brought to between 2 to 4 percent optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557) prior to refilling the excavation to grade as properly compacted fill. Fill areas should not be constructed so as to place structures across any area where the maximum depth of fill to minimum depth of fill is greater than a 3:1 ratio.

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

No structure should be placed across any areas where the ratio of the maximum depth of fill to minimum depth of fill is greater than a 3 to 1 ratio as measured from the bottom of the footing. For example, if one edge of the building pad of a cut-to-fill transition lot requires 10 feet of fill, then the cut portion of the lot should be over-excavated to a minimum of 3 feet below the footing elevations.

Engineered Compacted Fill

Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in fills.

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

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

Based upon the relative compaction of the near surface soils determined during this investigation and the relative compaction anticipated for compacted fill soil, we estimate a compaction shrinkage factor of approximately 10 to 15 percent. Therefore, 1.10 to 1.15 cubic yards of in-place materials would be necessary to yield one cubic yard of properly compacted fill material. Subsidence is anticipated to be 0.10 feet. These values are for estimating purposes only, and are exclusive of losses due to stripping or the removal of subsurface obstructions.

These values may vary due to differing conditions within the project boundaries and the limitations of this investigation. Shrinkage should be monitored during construction. If percentages vary, provisions should be made to revise final grades or adjust quantities of borrow or export.

Short-Term Excavations

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

Short-term excavations 5-feet deep and greater shall conform to Title 8 of the California Code of Regulations, Construction Safety Orders, Section 1504 and 1539 through 1547.

Based on our exploratory borings, it appears that Type C soil is the predominant type of soil on the project and all short-term excavations should be based on this type of soil. Deviation from the standard short-term slopes are permitted using Option 4, Design by a Registered Professional Engineer (Section 1541.1).

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

Slope Construction

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

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

Slope Protection

Since the site soils are susceptible to erosion by running water, measures should be provided to prevent surface water from flowing over slope faces. Slopes at the project should be planted with a deep rooted ground cover as soon as possible after completion. The use of succulent ground covers such as iceplant or sedum is not recommended. If watering is necessary to sustain plant growth on slopes, the watering system should be monitored to assure proper operation and to prevent over watering.

Shallow Foundation Design

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

For the minimum width and depth, spread foundations may be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure may be increased by 100 psf for each additional foot of width, and by 400 psf for each additional foot of depth, up to a maximum of 6,000 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 200 pounds per square foot per foot of depth. Base friction may be computed at 0.25 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

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

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

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

For design of retaining walls unrestrained against movement at the top, we recommend an active pressure of 51 pounds per square foot (psf) be used. This assumes level backfill consisting of recompacted, non-expansive, soils placed against the structures and with the backcut slope extending upward from the base of the stem at 35 degrees from the vertical or flatter.

To avoid overstressing or excessive tilting during placement of backfill behind walls, heavy compaction equipment should not be allowed within the zone delineated by a 45 degree line extending from the base of the wall to the fill surface.

The backfill directly behind the walls should be compacted using light equipment such as hand operated vibrating plates and rollers. No material larger than 3-inches in diameter should be placed in direct contact with the wall.

Wall pressures should be verified prior to construction, when the actual backfill materials and conditions have been determined. Recommended pressures are applicable only to level, non-expansive, properly drained backfill (with no additional surcharge loadings).

If inclined backfills are proposed, this firm should be contacted to develop appropriate active earth pressure parameters. Toe bearing pressure for non-structural walls on soils, not prepared as described earlier under Preparation of Foundation Areas, should not exceed California Building Code values.

Sulfate Protection

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

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

Preliminary Pavement Design

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

AREA	T.I.*	DESIGN R-VALUE	PRELIMINARY SECTION
Parking and Drive Areas (light vehicular traffic and occasional truck traffic)	6.0	10	0.25' AC/1.05' AB
Industrial Collector Secondary Major - Off-site	8.0	10	0.40' AC/1.35'AB
AC - Asphalt Concrete AB - Class 2 Aggregate Base *Actual Traffic Index should be determined by others			

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

In areas of the pavement which will receive high abrasion loads due to start-ups and stops, or where trucks will move on a tight turning radius, consideration should be given to installing concrete pads. Such pads should be a minimum of 0.5-foot thick concrete, with a 0.35-foot thick aggregate base. Concrete pads are also recommended in areas adjacent to trash storage areas where heavier loads will occur due to operation of trucks lifting trash dumpsters. The recommended 0.5 feet thick portland cement concrete (PCC) pavement section should have a minimum modulus of rupture (MR) of 550 pounds per square inch (psi).

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 0.3 to 8.9 inches per hour was obtained. It is our opinion that the design clear water rate is the clear water rate obtained during this investigation at each of the planned infiltration in the areas and depths 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 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 herein have been incorporated into the design. Additional expansion index, R-value, and soluble sulfate testing may be required during site rough grading.

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

1. Site preparation-stripping and removals.

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

3. Mapping of the fault zone locations during mass grading by the project engineering geologist to confirm the fault zones presented by Earth Systems Southwest (2016).

4. Processing and compaction of removal and/or over-excavation of bottom soils prior to fill placement.

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.

7. Foundation excavations.

LIMITATIONS

This report contains geotechnical conclusions and recommendations developed solely for use by Santiago Holdings, LLC c/o Thatcher Engineering & Associates, Inc., and their design consultants, for the purposes described earlier. It may not contain sufficient information for other uses or the purposes of other parties. The contents should not be extrapolated to other areas or used for other facilities without consulting LOR Geotechnical Group, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance.

The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. If conditions are encountered during the construction of the project which differ significantly from those presented in this report, this firm should be notified immediately in order that we may assess the impact to the recommendations provided.

Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed and tested by the project geotechnical consultant.

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

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

TIME LIMITATIONS

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

CLOSURE

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

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

Respectfully submitted,
LOR Geotechnical Group, Inc.

Andrew A. Tardie
Staff Geologist

Robert M. Markoff, CEG
Engineering Geologist

John P. Leuer, GE 2030
President

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

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NO. 2073
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STATE OF CALIFORNIA

AAT:RMM:JPL:ss

Distribution: Addressee (4) and PDF via email kristint@thatchereingineering.com

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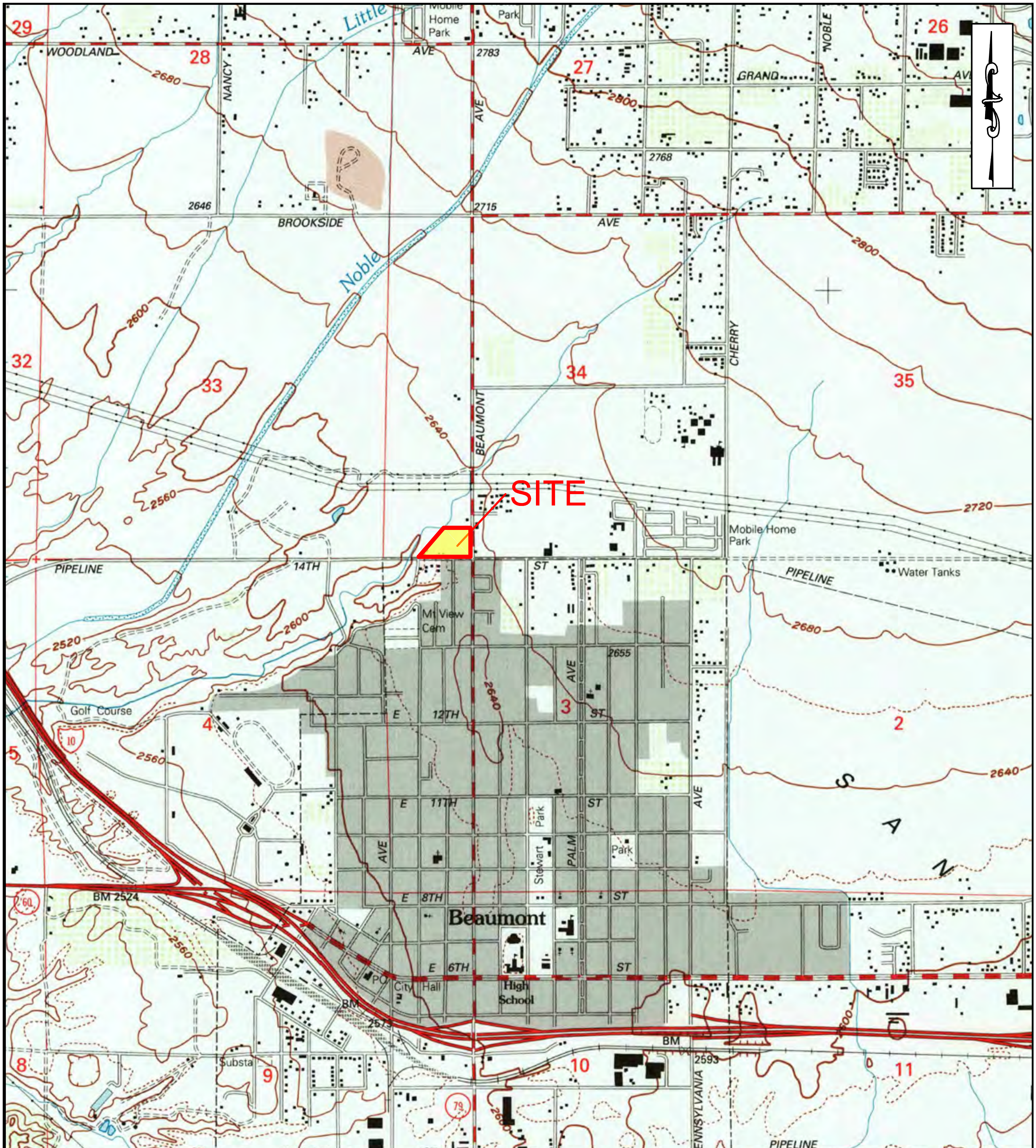
APPENDIX A

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



INDEX MAP

PROJECT:	APN'S 404-190-001 & -003, BEAUMONT, CALIFORNIA	**PROJECT NO:**	13627.1
CLIENT:	SANTIAGO HOLDINGS, LLC, C/O THATCHER ENGINEERING & ASSOCIATES, INC.	**ENCLOSURE:**	A-1
LOR Geotechnical Group, Inc.		**DATE:**	APRIL 2020
		SCALE:	1" ≈ 2000'

SITE PLAN

PROPOSED RETAIL & RESTAURANT
APN 404-190-001 & 003
NORTHWEST CORNER OF BEAUMONT AVENUE & OAK VALLEY PARKWAY
CITY OF BEAUMONT

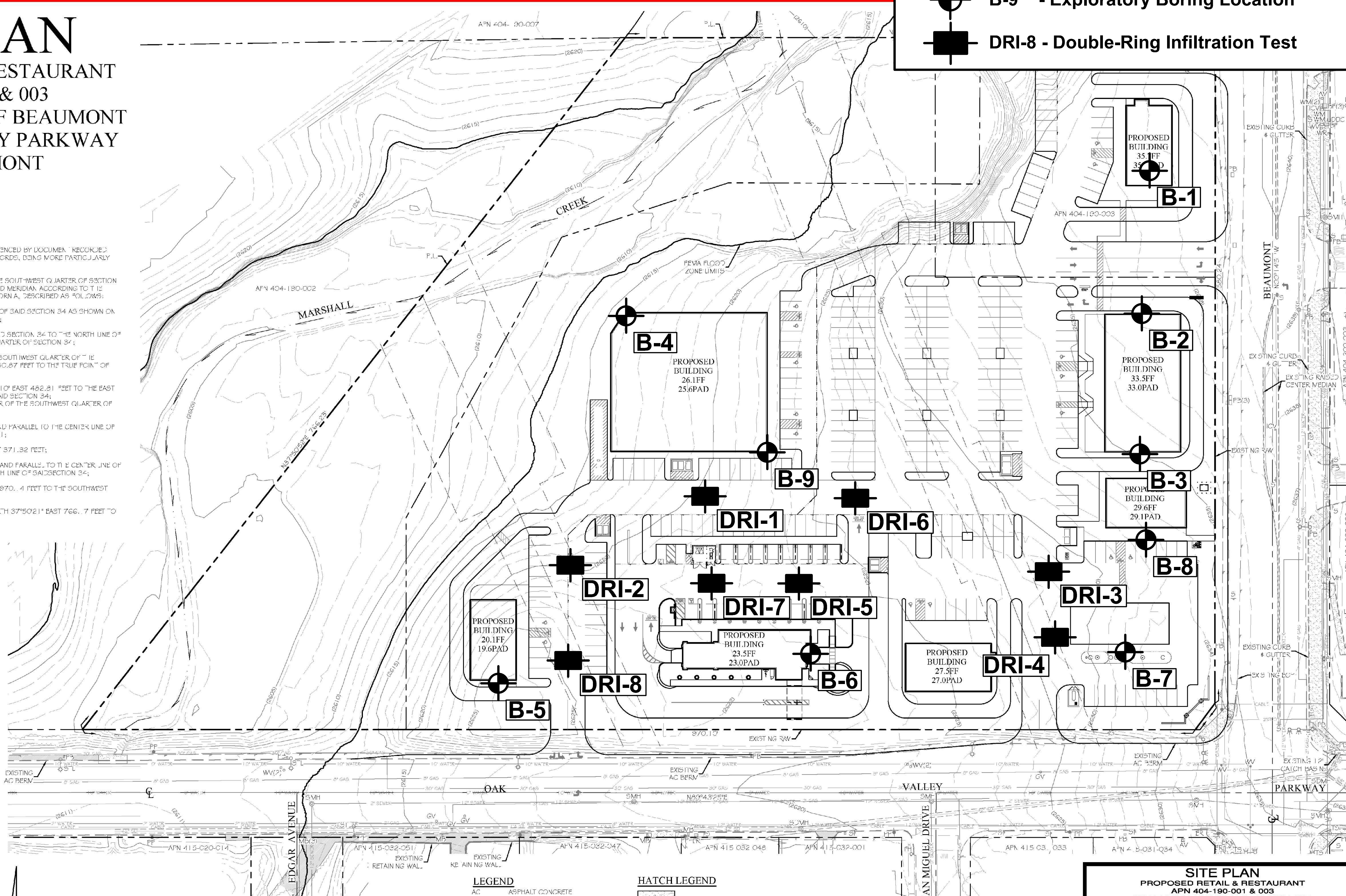
Legend

(Locations Approximate)

Map Symbols

B-9 - Exploratory Boring Location

DRI-8 - Double-Ring Infiltration Test



LEGAL DESCRIPTION

PARCEL C AS SHOWN ON LOT LINE ADJUSTMENT NO. 07-LLA-02 AS EVIDENCED BY DOCUMENT RECORDED OCTOBER 29, 2007 AS INSTRUMENT NO. 07-663184 OF OFFICIAL RECORDS, BEING MORE PARTICULARLY DESCRIBED AS FOLLOWS:

THAT PORTION OF THE SOUTH HALF OF THE SOUTHWEST QURTER OF THE SOUTHWEST QUARTER OF SECTION 34, IN TOWNSHIP 2 SOUTH, RANGE 1 WEST, SAN BERNARDINO BASE AND MERIDIAN ACCORDING TO THE OFFICIAL PLAT THEREOF, IN THE COUNTY OF RIVERSIDE, STATE OF CALIFORNIA, DESCRIBED AS FOLLOWS:

BEGINNING AT THE SOUTHWEST CORNER OF THE SOUTHWEST QUARTER OF SAID SECTION 34 AS SHOWN ON PARCEL MAP NO. 26229, PM 173/21, RECORDS OF RIVERSIDE COUNTY;

THENCE NORTH 00°49'05" EAST 657 FEET ALONG THE WEST LINE OF SAID SECTION 34 TO THE NORTH LINE OF THE NORTH HALF OF THE SOUTHWEST QUARTER OF THE SOUTHWEST QUARTER OF SECTION 34;

THENCE EASTERLY ALONG THE NORTH LINE OF THE SOUTH HALF OF THE SOUTHWEST QUARTER OF THE SOUTHWEST QUARTER OF SAID SECTION 34, NORTH 89°42'10" EAST 840.87 FEET TO THE TRUE POINT OF BEGINNING.

THENCE CONTINUING EASTERLY ALONG SAID NORTH LINE, NORTH 89°42'10" EAST 482.81 FEET TO THE EAST LINE OF THE SOUTHWEST QUARTER OF THE SOUTHWEST QUARTER OF SAID SECTION 34;
THENCE SOUTHERLY ALONG THE EAST LINE OF THE SOUTHWEST QUARTER OF THE SOUTHWEST QUARTER OF SAID SECTION 34, SOUTH 00°46'11" WEST 208.71 FEET;

THENCE NORTH 89°42'10" EAST 35.34 FEET TO A LINE 50 FEET WEST AND PARALLEL TO THE CENTER LINE OF BEAUMONT AVENUE AS SHOWN ON PARCEL MAP NO. 26229, PM 173/21;

THENCE SOUTHERLY ALONG SAID PARALLEL LINE, SOUTH 00°14'51" EAST 371.32 FEET;

THENCE SOUTH 36°12'03" WEST 28.49 FEET TO A LINE 55 FEET NORTH AND PARALLEL TO THE CENTER LINE OF FOURTEENTH STREET, FOURTEENTH STREET CENTERLINE BEING THE SOUTH LINE OF SAIDSECTION 34;

THENCE WESTERLY ALONG SAID PARALLEL LINE, SOUTH 89°43'07" WEST 970.[illegible]4 FEET TO THE SOUTHWEST CORNER OF PARCEL 1 OF PARCEL MAP NO. 26229, PM 173/2[illegible];

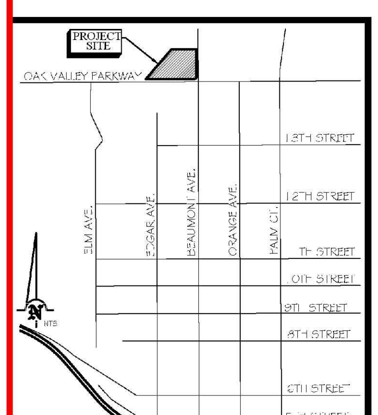
THENCE NORTHEASTERLY ALONG THE WEST LINE OF SAID PARCEL [illegible], NORTH 37°50'21" EAST 766.[illegible]7 FEET TO THE TRUE POINT OF BEGINNING.

APN: 404-190-001 AND 404-[illegible]90-003

EXISTING EASEMENTS

UTILITIES:

ELECTRIC: SOUTHERN CALIFORNIA EDISON COMPANY 287 TENNESSEE STREET REDLANDS, CA 92373 (909) 307 6788	GAS: SOUTHERN CALIFORNIA GAS COMPANY 1981 WEST LUGONIA AVENUE REDLANDS, CA 92373 (909) 335-7836
TELEPHONE: VERIZON 9 S. 4TH STREET REDLANDS, CA 92373 (909) 748-6640	CABLE: CHARTER COMMUNICATIONS [illegible]500 AUTO CENTER DRIVE ONTARIO, CA 91761 (909)634-3224
WATER: BEAUMONT-CHERRY VALLEY WATER DISTRICT 560 MAGNOLIA AVENUE BEAUMONT, CA 92223 (951) 845-9581	SEWER: CITY OF BEAUMONT 550 E. 6TH STREET BEAUMONT, CA 92223 (951)769-8518



VICINITY MAP

LEGEND

AC	ASPHALT CONCRETE
AV	AIR VALVE
ER	ELECTRIC RISER
EX	EXISTING
GV	GAS VALVE
FH	FIRE HYDRANT
ICV	IRRIGATION CONTROL VALVE
LT	LIGHT
PB	PULL BOX
PCC	PORTLAND CEMENT CONCRETE
P.L.	PROPERTY LINE
R/W	RIGHT-OF-WAY
S	SIGN
SDMH	STORM DRAIN MANHOLE
SMH	SEWER MANHOLE
STL	STREET LIGHT
TS	TRAFFIC SIGNAL
TYP	TYPICAL
WV	WATER VALVE
WM	WATER METER
WR	WATER RISER

HATCH LEGEND

EXISTING PCC

EXISTING AC SIDEWALK

SOURCE OF SURVEY

TOPOGRAPHIC SURVEY
DATED NOVEMBER 20[illegible]7
AS CONDUCTED BY
INLAND AERIAL SURVEYS, INC.
7117 ARLINGTON AVE., SUITE A
RIVERSIDE, CA 92503
PHONE: (951) 687-4252

SOIL ENGINEER

REPORT DATED NOVEMBER 17, 20[illegible]7
PROJECT NO. 17059-BMP
AS CONDUCTED BY
SOILS SOUTHWEST, INC.
897 VIA LATA, SUITE N
COLTON, CA 92324
PHONE: (909) 370-0474
FAX: (909) 370-3[illegible]56

PROPERTY OWNER/APPLICANT:

SANTIAGO HOLDINGS, LLC
C/O CAMDEN HOLDINGS, LLC
ATTN: MR. ARI MILLER
9454 WILSHIRE BLVD., 6TH FLOOR
BEVERLY HILLS, CA 90212
(310) 553-1031

SITE PLAN

PROPOSED RETAIL & RESTAURANT
APN 404-190-001 & 003
NORTHWEST CORNER OF BEAUMONT AVENUE & OAK VALLEY PARKWAY
CITY OF BEAUMONT

• land planning
• civil engineering
• landscape architecture
phone 909.748.7777
fax 909.748.7776
est. 1989
thatcher engineering & associates, inc.
1461 ford street, suite 105, redlands, ca 92373

REGISTERED PROFESSIONAL ENGINEER
PATRICK C. FLANAGAN, JR.
No. 86046
Exp. 9/30/20
CIVIL
STATE OF CALIFORNIA

Patrick C. Flanagan, Jr. R.C.E. 86046 Exp. Sep 30, 2020

Job Number:	Date Prepared:	Drawn By:	Reference Number:
17160	2/18/20	R.	171601SP

17 601SP

SITE PLAN

PROJECT NO: 13627.1

ENCLOSURE: A-2

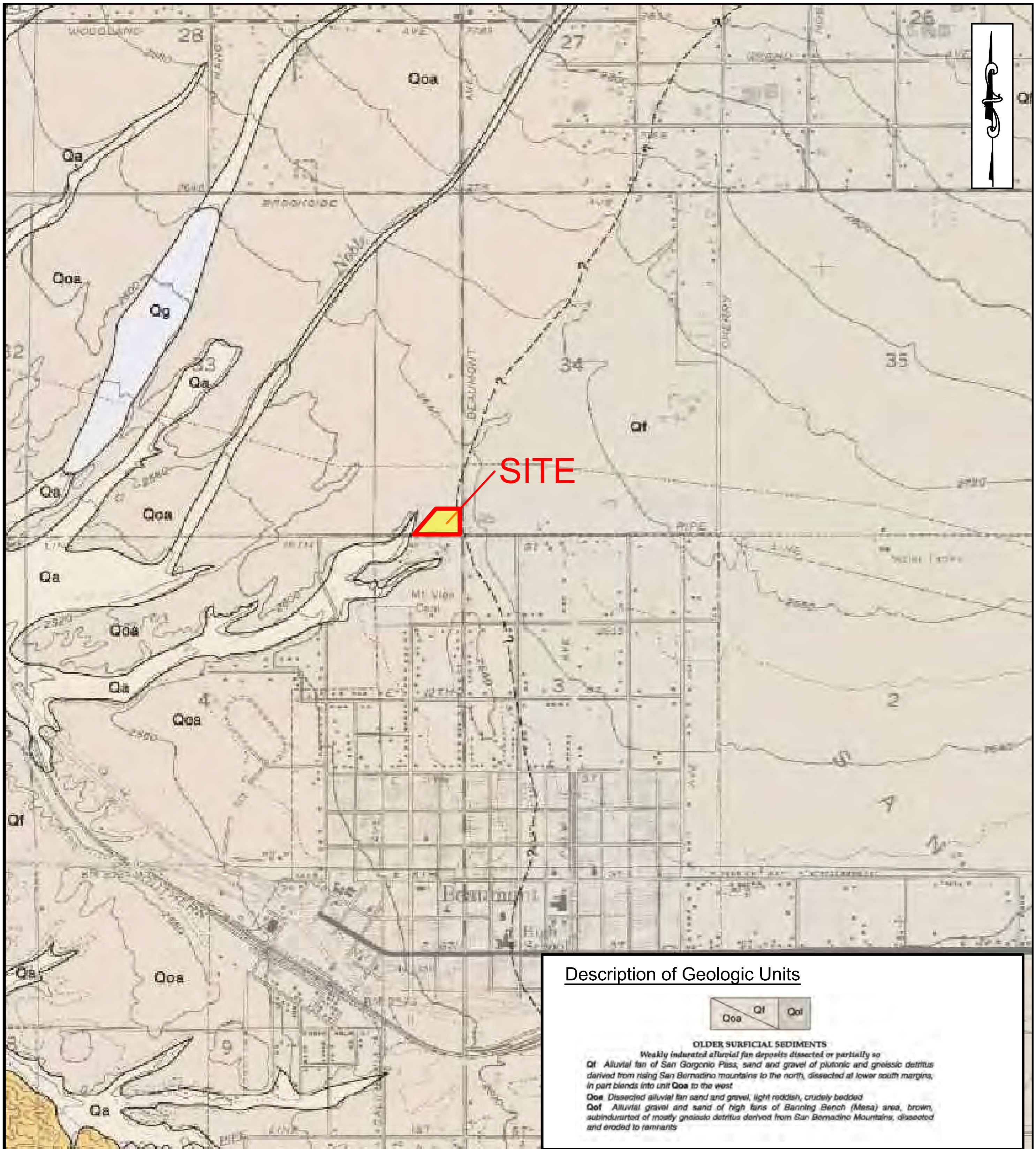
DATE: APRIL 2020

SCALE: 1" ≈ 100'

PROJECT: APN'S 404-190-001 & -003, BEAUMONT, CALIFORNIA

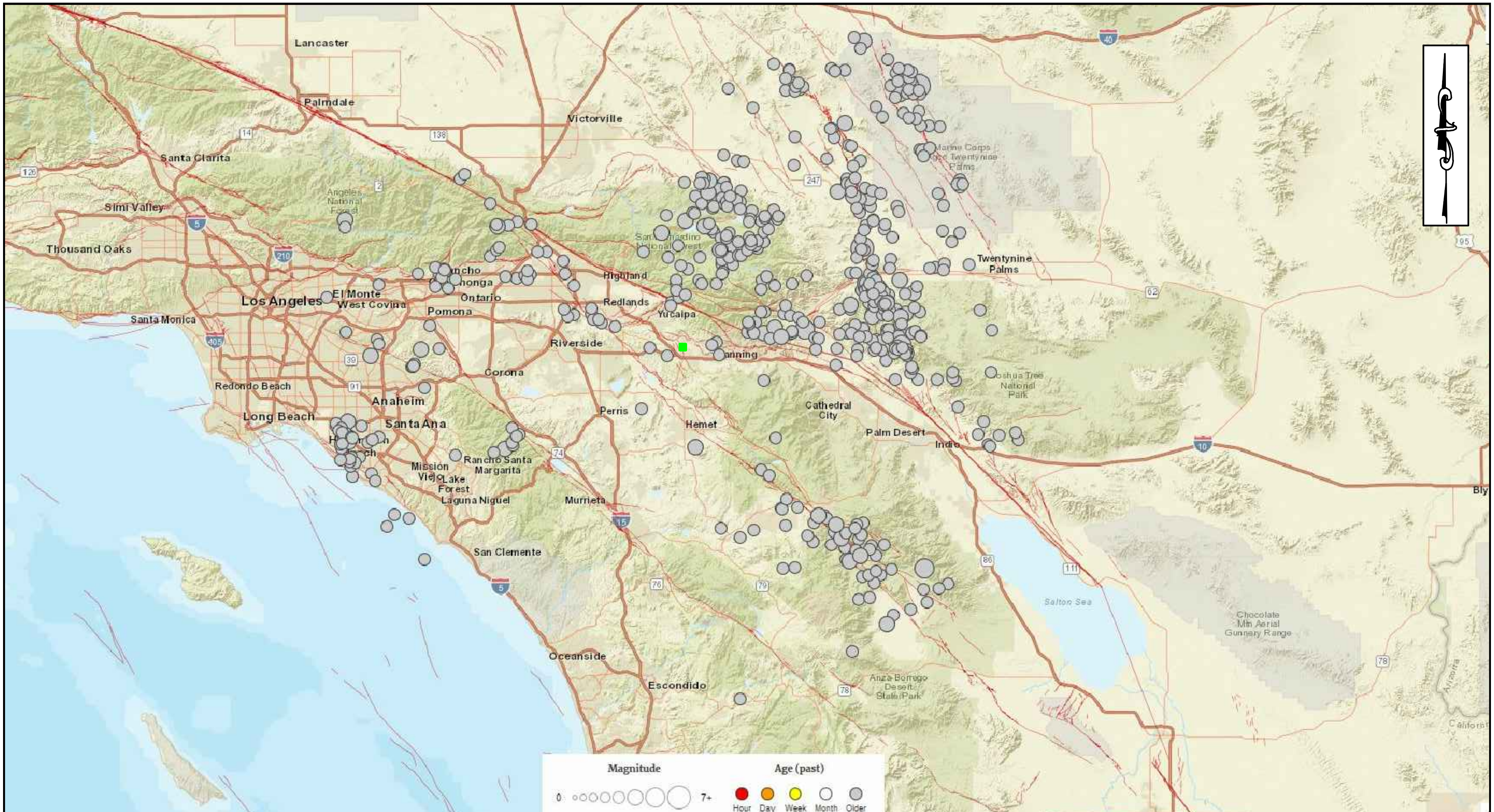
CLIENT: SANTIAGO HOLDINGS, LLC, C/O THATCHER ENGINEERING & ASSOCIATES, INC.

LOR Geotechnical Group, Inc.



REGIONAL GEOLOGIC MAP

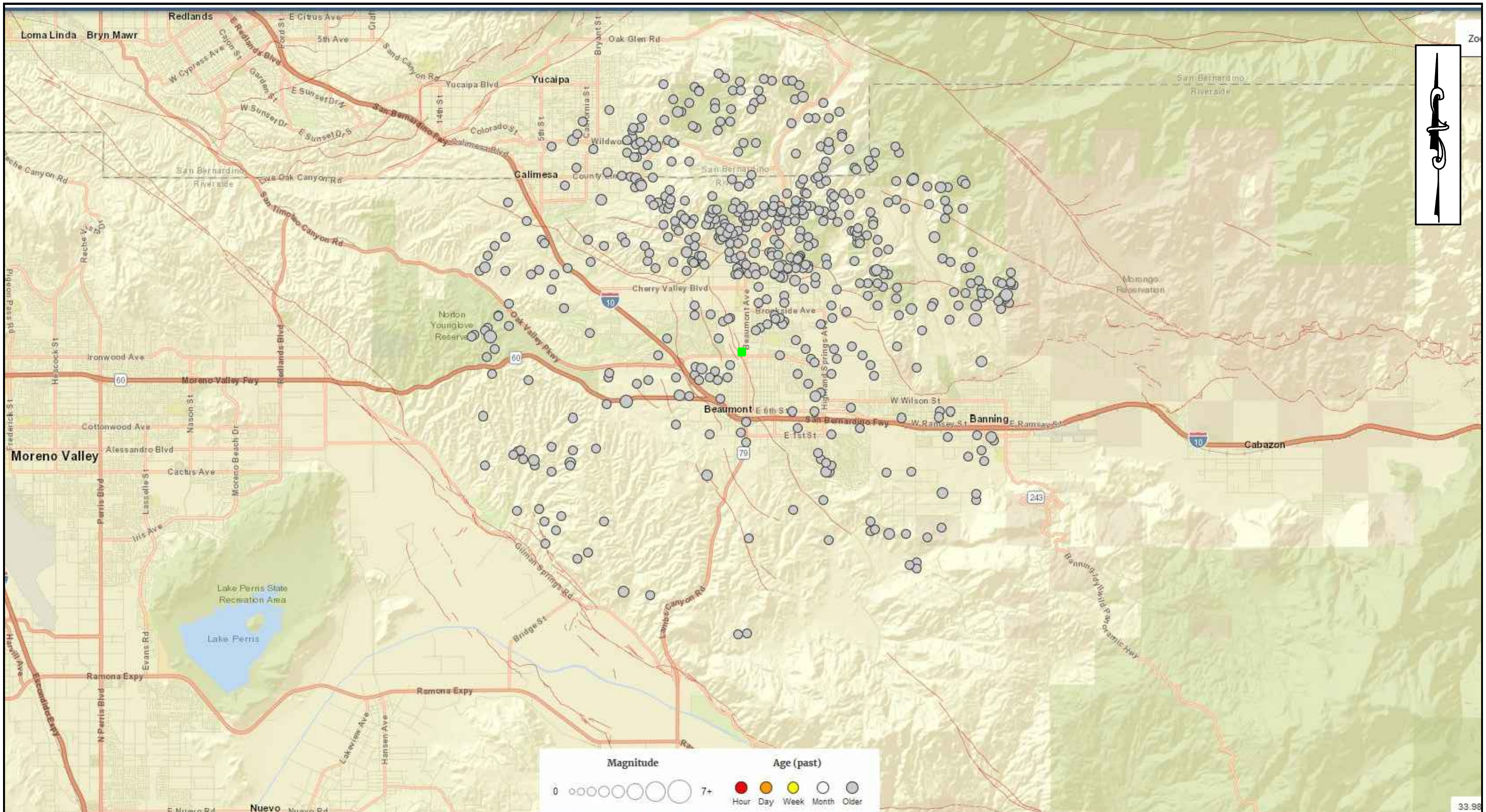
PROJECT:	APN'S 404-190-001 & -003, BEAUMONT, CALIFORNIA	**PROJECT NO:**	13627.1
CLIENT:	SANTIAGO HOLDINGS, LLC, C/O THATCHER ENGINEERING & ASSOCIATES, INC.	**ENCLOSURE:**	A-3
LOR Geotechnical Group, Inc.		**DATE:**	FEBRUARY 2020
		SCALE:	1" ≈ 2000'



U.S. Geologic Survey (2020) real-time earthquake epicenter map. Plotted are 544 epicenters of instrument-recored events from 1932 to present (04/02/20) 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:	APN'S 404-190-001 & -003, BEAUMONT, CALIFORNIA	**PROJECT NO:**	13627.1
CLIENT:	SANTIAGO HOLDINGS, LLC, C/O THATCHER ENGINEERING & ASSOCIATES, INC.	**ENCLOSURE:**	A-4
LOR Geotechnical Group, Inc.		**DATE:**	APRIL 2020
		SCALE:	1" ≈ 40km



U.S. Geologic Survey (2020) real-time earthquake epicenter map. Plotted are 551 epicenters of instrument-recored events from 1978 to present (04/02/20) of local magnitude greater than M2.0 within a radius of ~6.2 miles (10 kilometers) of the site. Location accuracy varies. The site is indicated by the green square. Red lines mark the surface traces of known Quaternary-age faults.

HISTORICAL SEISMICITY MAP - 10km Radius

PROJECT:	APN'S 404-190-001 & -003, BEAUMONT, CALIFORNIA	**PROJECT NO:**	13627.1
CLIENT:	SANTIAGO HOLDINGS, LLC, C/O THATCHER ENGINEERING & ASSOCIATES, INC.	**ENCLOSURE:**	A-5
LOR Geotechnical Group, Inc.		**DATE:**	APRIL 2020
		SCALE:	1" ≈ 4km

APPENDIX B

Field Investigation Program and Boring Logs

APPENDIX B
FIELD INVESTIGATION

Subsurface Exploration

The site was investigated on March 12th and March 24th 2020 and consisted of advancing 9 exploratory borings to depths from approximately 16.5 feet and 51.5 feet below the existing ground surface. The approximate locations of the borings are shown on Enclosures A-2 through A-4, within Appendix A.

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

Relatively undisturbed samples of the subsoils were obtained at a 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-9.

The undisturbed soil samples were retained in brass sample rings of 2.42 inches in diameter and 1.00 inch in height, and placed in sealed containers. Disturbed soil samples were obtained at selected levels within the borings and placed in sealed containers for transport to the 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-9. A Boring Log Legend and Soil Classification Chart are presented on Enclosures B-i and B-ii, respectively.

CONSISTENCY OF SOIL

SANDS

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

COHESIVE SOILS

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

SAMPLE KEY

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

TYPES OF LABORATORY TESTS

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

BORING LOG LEGEND

PROJECT:	**PROPOSED RETAIL DEVELOPMENT, BEAUMONT, CALIFORNIA**	**PROJECT NO.:**	**13627.1**
CLIENT:	**SANTIAGO HOLDINGS, LLC. C/O THATCHER ENGINEERING & ASSOCIATES**	**ENCLOSURE:**	**B-i**
LOR Geotechnical Group, Inc.		**DATE:**	**APRIL 2020**

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 RETAIL DEVELOPMENT, BEAUMONT, CALIFORNIA	PROJECT NO.	13627.1
CLIENT:	SANTIAGO HOLDINGS, LLC. C/O THATCHER ENGINEERING & ASSOCIATES	ENCLOSURE:	B-ii
LOR Geotechnical Group, Inc.		DATE:	APRIL 2020

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	61	3, 4, 7, 9, 10, 11	7.4		122.7			SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, trace gravel to 1/2", approximately 15% coarse gained sand, 20% medium grained sand, 30% fine grained sand, 35% silty fines with trace clay, dark brown, wet (due to recent rain).**
								ML	**@ 1 foot,** OLDER ALLUVIUM: **SANDY SILT, trace gravel to 3/4", approximately 5% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 60% silty fines with trace clay, light red brown, damp.**
5	42		6.3		113.7			SM	**@ 5 feet, SILTY SAND, approximately 10% coarse grained sand, 20% medium grained sand, 30% fine grained sand, 40% silty fines, light red brown, damp, some pinhole porosity.**
10	73		2.5		114.5			SW SM	**@ 10 feet, WELL GRADED SAND with GRAVEL and SILT, approximately 15% gravel to 1", 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 10% silty fines, red brown, dry.**
15	33		13.5		92.2			SP SM	**@ 15 feet, POORLY GRADED SAND with SILT, approximately 5% coarse grained sand, 15% medium grained sand, 70% fine grained sand, 10% silty fines, yellow brown, damp.**
20	73		2.2		120.0			SW	**@ 20 feet, WELL GRADED SAND with GRAVEL, approximately 20% gravel to 1", 25% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 5% silty fines, red brown, dry.**
25	92		2.0		110.8				
									END OF BORING @ 26.33' **Fill/topsoil to 1'** **No groundwater** **No bedrock**
30									

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2637**
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	**March 12, 2020**
		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	43		8.8		120.3			SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, trace gravel to 3/4", approximately 15% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 30% silty fines with trace clay, brown, wet (due to recent rain).** @ 1 foot, OLDER ALLUVIUM: **SILTY SAND, approximately 15% coarse grained sand, 20% medium grained sand, 25% fine grained sand, 40% silty fines with trace clay, red brown, damp, some pinhole porosity.**
5	34		9.4		119.4			SC CL	**@ 5 feet, SANDY CLAY/LEAN CLAY with SAND, approximately 15% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 50% clayey fines of low plasticity, red brown, damp.**
10	57		3.5		122.5			SM	**@ 10 feet, SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, yellow brown, dry.**
15	34		3.1		106.8				**@ 15 feet, SILTY SAND, approximately 15% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 20% silty fines, yellow brown, dry.**
20	56		5.5		97.0				**@ 20 feet, SILTY SAND, approximately 5% coarse grained sand, 10% medium grained sand, 70% fine grained sand, 15% silty fines, yellow brown, dry.**
									END OF BORING @ 21.5' **Fill/topsoil to 1'** **No groundwater** **No bedrock**
25									

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2637**
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	**March 12, 2020**
		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								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 15% coarse grained sand, 20% medium grained sand, 25% fine grained sand, 35% silty fines with trace clay, brown, wet (due to recent rain).**
	22		6.8		119.8				@ 1 foot, OLDER ALLUVIUM: **SILTY SAND, approximately 5% gravel to 1/2", 15% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 45% silty fines with trace clay, red brown, dry to damp.**
5	29		10.8		120.3				**@ 5 feet, SILTY SAND, approximately 20% coarse grained sand, 25% medium grained sand, 30% fine grained sand, 25% silty fines with trace clay, red brown, damp, some pinhole porosity.**
10	26		4.7		113.0				**@ 10 feet, SILTY SAND, trace gravel to 1/2", approximately 15% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 15% silty fines, light red brown, damp.**
15	33		2.3		112.3				**@ 15 feet, becomes yellow brown, dry.**
20	68		5.2		112.2				
25	59		4.4		112.3				**@ 25 feet, SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 30% silty fines, yellow brown, dry.**
30	98		4.8		116.9			ML SM	**@ 30 feet, SILTY SAND/SANDY SILT, trace gravel to 1/2", approximately 10% coarse grained sand, 15% medium grained sand, 25% fine grained sand, 50% silty fines, yellow brown, dry.**
35									

END OF BORING @ 30.42'

Fill/topsoil to 1'
No groundwater
No bedrock

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2635**
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	**March 12, 2020**
		EQUIPMENT:	**Mobile B-61**
		HOLE DIA.: **8"**	ENCLOSURE: **B-3**

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	15	7, 9, 10, 11	3.9		109.0			SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, trace gravel to1/2", approximately 5% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 30% silty fines, dark brown, moist, loose.** **@ 1 foot,** OLDER ALLUVIUM: **SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 35% fine grained sand, 45% silty fines, light yellow brown, dry, some roothairs, trace pinhole porosity.**
5	31		9.5		116.4				**@ 5 feet, SILTY SAND, approximately 10% coarse grained sand, 20% medium grained sand, 40% fine grained sand, 30% silty fines with trace clay, light red brown, damp, some pinhole porosity.**
10	48		9.1		113.0				**@ 10 feet, SILTY SAND, approximately 20% medium grained sand, 60% fine grained sand, 20% silty fines, light red brown, damp.**
15	48		6.7		133.0				**@ 15 feet, trace gravel to 1/2".**
20	68		3.3		112.6			SW	**@ 20 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, tan, dry.**
25	76		2.7						
30	54		3.5						**@ 30 feet, becomes slightly finer grained.**
35	46		7.7					SM	**@ 35 feet, SILTY SAND, approximately 5% gravel to 1/2", 15% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 20% silty fines, red brown, damp.**
40	46		7.6					SP SM	**@ 40 feet, POORLY GRADED SAND with SILT, approximately 90% fine grained sand, 10% silty fines, light red brown, dry to damp.**
45	35		16.4						**@ 45 feet, increase in moisture, moist.**
50	58		6.2						**@ 50 feet, decrease in moisture, damp.**
55									

END OF BORING @ 51.5'

Fill/topsoil to 1'
No groundwater
No bedrock

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2616**
		DATE DRILLED:	**March 24, 2020**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Track Rig**
		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 5% coarse grained sand, 20% medium grained sand, 45% fine grained sand, 30% silty fines, dark brown, moist, loose.
	13		11.6		116.3				@ 1 foot, OLDER ALLUVIUM: SILTY SAND, approximately 5% gravel to 1/2", 15% coarse grained sand, 20% medium grained sand, 20% fine grained sand, 40% silty fines with trace clay, red brown, damp.
5	10		6.6		107.6				@ 5 feet, SILTY SAND, approximately 15% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 25% silty fines, light yellow brown, damp.
	10		6.7		100.1				@ 7 feet, becomes dry, light red brown.
10	24		4.6		119.3				@ 10 feet, SILTY SAND, approximately 5% gravel to 1/2", 20% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 20% silty fines, yellow brown, dry.
15	80		4.6		123.4				@ 15 feet, SILTY SAND with GRAVEL, approximately 15% gravel to 1", 20% coarse grained sand, 25% medium grained sand, 25% fine grained sand, 15% silty fines, yellow brown, dry.
20	59		4.3		112.6				@ 20 feet, SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 60% fine grained sand, 20% silty fines, red brown, dry.
25	66		5.6		138.0				
									END OF BORING @ 26.5' Fill/topsoil to 1' No groundwater No bedrock
30									

PROJECT:	Proposed Retail Development	PROJECT NUMBER:	13627.1
CLIENT:	Thatcher Engineering	ELEVATION:	2626
LOR GEOTECHNICAL GROUP INC.		DATE DRILLED:	March 24, 2020
		EQUIPMENT:	Track Rig
		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, 25% medium grained sand, 35% fine grained sand, 35% silty fines, dark brown, moist, loose.** **@ 1 foot,** OLDER ALLUVIUM: **SILTY SAND, trace coarse grained sand, approximately 15% medium grained sand, 50% fine grained sand, 35% silty fines with trace clay, red brown, damp, some pinhole porosity.**
	22		**8.7**		**111.8**				
5	**27**		**13.6**		**113.4**				**@ 5 feet, trace pinhole porosity.**
10	**21**		**11.0**		**105.7**				**@ 10 feet, SILTY SAND, approximately 5% coarse grained sand, 15% medium grained sand, 55% fine grained sand, 25% silty fines, yellow brown, damp.**
15	**29**		**20.2**		**99.3**			ML	**@ 15 feet, SANDY SILT, approximatley 20% fine grained sand, 80% silty fines, yellow brown, damp.**
20	**60**		**3.7**		**111.6**			SW	**@ 20 feet, WELL GRADED SAND, approximately 5% gravel to 1/2", 25% coarse grained sand, 30% medium grained sand, 35% fine grained sand, 5% silty fines, yellow brown, dry.**
									END OF BORING @ 21.5'
									Fill/topsoil to 1' **No groundwater** **No bedrock**
25									

PROJECT: **Proposed Retail Development**	PROJECT NUMBER: **13627.1**
CLIENT: **Thatcher Engineering**	ELEVATION: **2623**
LOR GEOTECHNICAL GROUP INC.	DATE DRILLED: **March 24, 2020**
	EQUIPMENT: **Track Rig**
	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		8, 9, 10, 11				SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 35% silty fines, dark brown, moist, loose.**
	32		10.4		114.0	ML	@ 1 foot, OLDER ALLUVIUM: **SANDY SILT, trace fine gravel, approximately 5% coarse grained sand, 15% medium grained sand, 20% fine grained sand, 60% silty fines with trace clay, red brown, damp, trace pinhole porosity.**
5	50		11.3		120.3		
10	34		6.3		107.5	SM	**@ 10 feet, SILTY SAND, approximately 10% coarse grained sand, 35% medium grained sand, 40% fine grained sand, 15% silty fines, light yellow brown, damp.**
15	37		5.4		109.4		**@ 15 feet, SILTY SAND, approximately 35% medium grained sand, 50% fine grained sand, 15% silty fines, yellow brown, dry.**
20	69		2.1		113.7	SW	**@ 20 feet, WELL GRADED SAND, approximately 10% gravel to 1/2", 25% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 5% silty fines, yellow brown, dry.**
25	85		2.0		115.8		
							END OF BORING @ 26.42' **Fill/topsoil to 1'** **No groundwater** **No bedrock**
30							

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2632**
		DATE DRILLED:	**March 24, 2020**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Track Rig**
		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								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 20% medium grained sand, 45% fine grained sand, 30% silty fines, dark brown, moist, loose.** @ 1 foot, OLDER ALLUVIUM: **SILTY SAND, approximately 10% coarse grained sand, 30% medium grained sand, 30% fine grained sand, 30% silty fines with trace clay, red brown, damp, trace pinhole porosity.**
	29		9.5		117.6				
5	23		12.9		117.7				
10	38		11.8		117.1				
15	43		3.7		114.6				**@ 15 feet, SILTY SAND, approximately 25% medium grained sand, 60% fine grained sand, 15% silty fines, yellow brown, dry.**
									END OF BORING @ 16.5' **Fill/topsoil to 1'** **No groundwater** **No bedrock**
20									

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2632**
		DATE DRILLED:	**March 24, 2020**
LOR GEOTECHNICAL GROUP INC.		EQUIPMENT:	**Track Rig**
		HOLE DIA.: **8"**	ENCLOSURE: **B-8**

LOG OF BORING B-9

TEST DATA

DEPTH IN FEET	SPT BLOW COUNTS	LABORATORY TESTS	MOISTURE CONTENT (%)		DRY DENSITY (PCF)	SAMPLE TYPE	LITHOLOGY	U.S.C.S.	DESCRIPTION
0								SM	@ 0 feet, FILL/TOPSOIL: **SILTY SAND, approximately 5% coarse grained sand, 25% medium grained sand, 40% fine grained sand, 30% silty fines, dark brown, moist, loose.**
									@ 1 foot, OLDER ALLUVIUM: **SILTY SAND, approximately 10% coarse grained sand, 25% medium grained sand, 35% fine grained sand, 30% silty fines with trace clay, red brown, damp, some roothairs, trace pinhole porosity.**
	29		8.9		108.2				
5	19		9.3		116.6				@ 5 feet, contains trace gravel to 1/2", no visible porosity, yellow brown.
10	12	2	10.6		99.9				@ 10 feet, SILTY SAND, approximately 70% fine grained sand, 30% silty fines, red brown, damp.
15	32		10.9		112.2				
20	36		13.0		111.2				
									END OF BORING @ 21.5' **Fill/topsoil to 1'** **No groundwater** **No bedrock**
25									

PROJECT:	**Proposed Retail Development**	PROJECT NUMBER:	**13627.1**
CLIENT:	**Thatcher Engineering**	ELEVATION:	**2623**
		DATE DRILLED:	**March 24, 2020**
		EQUIPMENT:	**Track Rig**
		HOLE DIA.: **8"**	ENCLOSURE: **B-9**

LOR GEOTECHNICAL GROUP INC.

APPENDIX C

Laboratory Testing Program and Test Results

APPENDIX C
LABORATORY TESTING

General

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

Moisture Density Tests

The moisture content and dry density information provides an indirect measure of soil consistency for each stratum, and can also provide a correlation between soils on this site. The dry unit weight and field moisture content were determined for selected undisturbed samples, in accordance with ASTM D 2922 and ASTM D 2216, respectively, and the results are shown on the Boring Logs, Enclosures B-1 through B-9 for convenient correlation with the soil profile.

Laboratory Compaction

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

LABORATORY COMPACTION				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.G.S.)**	**Maximum Dry Density (pcf)**	**Optimum Moisture Content (percent)**
B-1	0-3	(ML) Sandy Silt	127.5	10.0
B-4	0-3	(SM) Silty Sand	130.0	9.5

Direct Shear Tests

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

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

DIRECT SHEAR TESTS				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.G.S.)**	**Angle of Internal Friction (degrees)**	**Apparent Cohesion (psf)**
B-1	0-3	(ML) Sandy Silt	25	350

Sieve Analysis

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

Sand Equivalent

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

R-Value Test

Soil samples were obtained at probable pavement subgrade level and 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 Tests

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

EXPANSION INDEX TESTS				
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.C.S.)**	**Expansion Index (EI)**	**Expansion Potential**
B-1	0-4	(ML) Sandy Silt	12	Very Low

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

Consolidation Tests

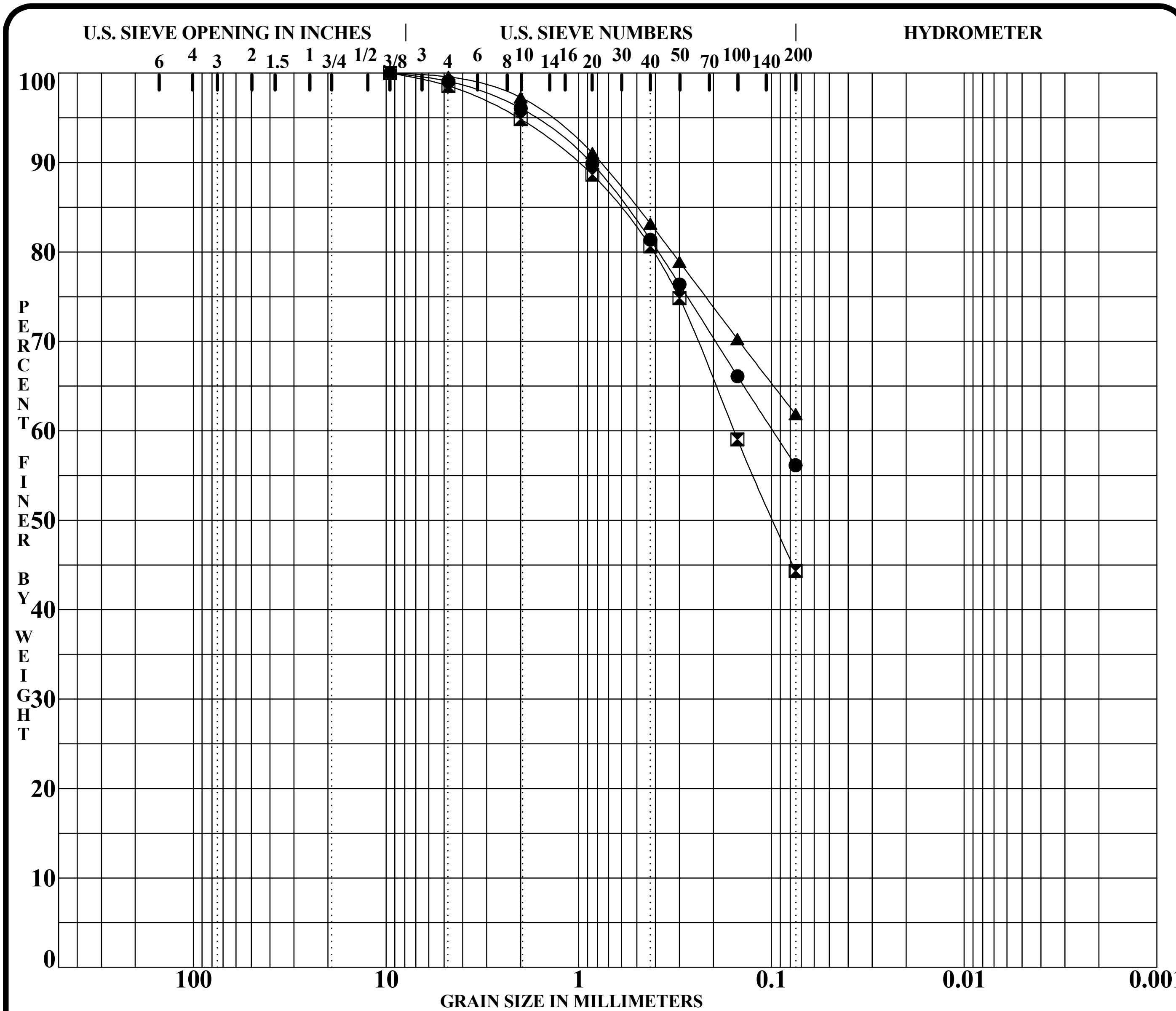
The apparatus used for the consolidation tests (odometer) is designed to test a one-inch high portion of the undisturbed soil sample as contained in a sample ring. Porous stones and filler paper are placed in contact with the top and bottom of the specimen to permit the addition or release of water. 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 Enclosure C-2.

Soluble Sulfate Content Tests

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

SOLUBLE SULFATE CONTENT TESTS			
Boring Number	**Sample Depth (feet)**	**Soil Description (U.S.G.S.)**	**Sulfate Content (percent by weight)**
B-1	0-3	(ML) Sandy Silt	< 0.005
B-4	0-3	(SM) Silty Sand	< 0.005
B-7	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 ft*	*(ML) Sandy Silt*	*10*	*--*				
⧗	*B-4*	*@ 0-3 ft*	*(SM) Silty Sand*	*14*	*--*				
▲	*B-7*	*@ 0-3 ft*	*(ML) Sandy Silt*	*8*	*12*				

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	*B-1*	*@ 0-3 ft*	*9.50*	*0.10*			*0.9*	*42.9*	*56.1*	
⧗	*B-4*	*@ 0-3 ft*	*9.50*	*0.16*			*1.4*	*54.2*	*44.3*	
▲	*B-7*	*@ 0-3 ft*	*9.50*				*0.4*	*37.7*	*61.9*	

PROJECT *Proposed Retail Development*

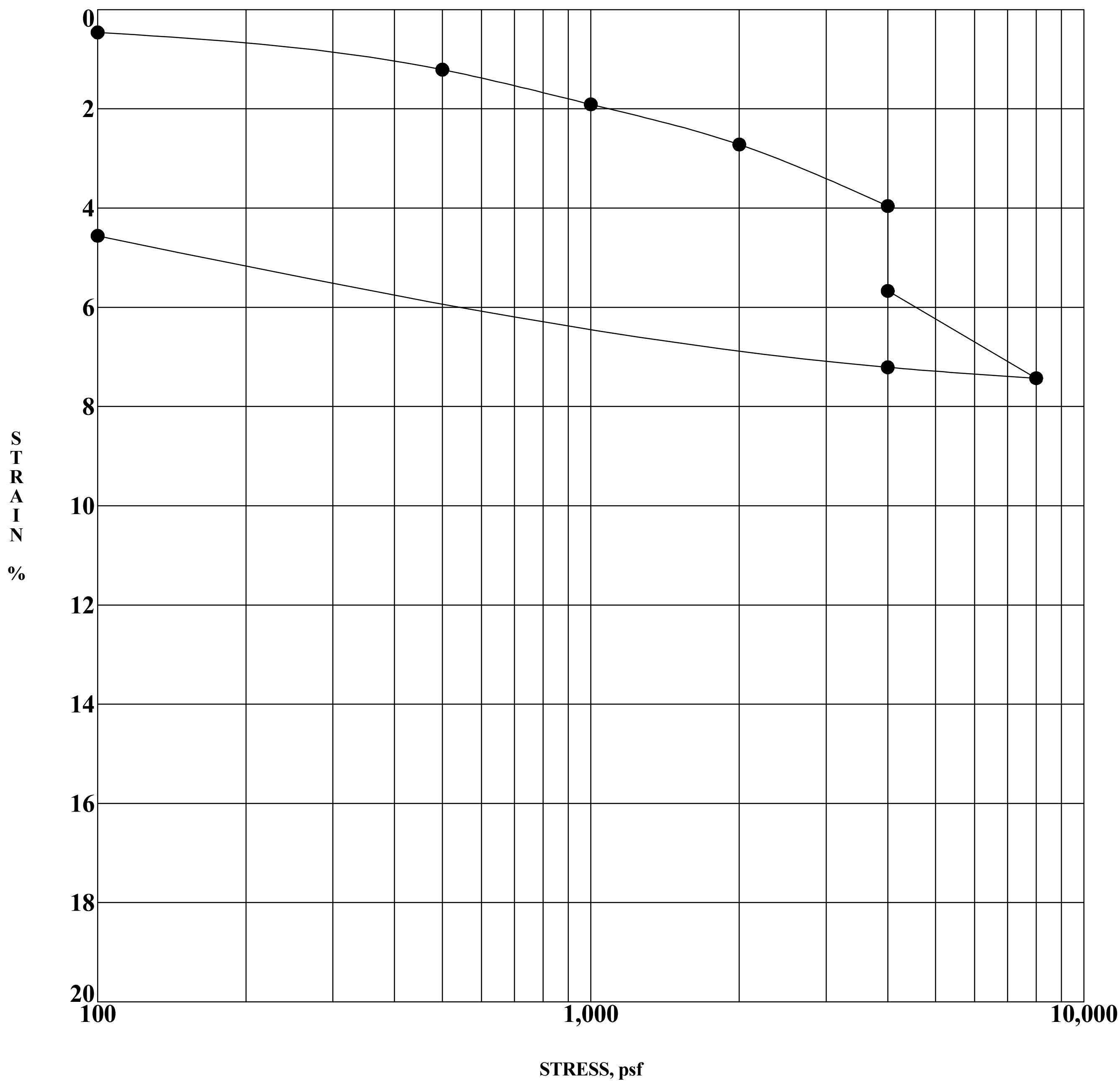
PROJECT NO. *13627.1*

DATE *4/3/20*

GRADATION CURVES

LOR Geotechnical Group, Inc.

ENCLOSURE C-1



	Specimen I.D.		Classification	DD	MC%
●	B-9	@ 10 ft	**(SM) Silty Sand**	*100*	*11*

PROJECT *Proposed Retail Development* **PROJECT NO.** *13627.1*

DATE *4/3/20*

CONSOLIDATION TEST

LOR Geotechnical Group, Inc.

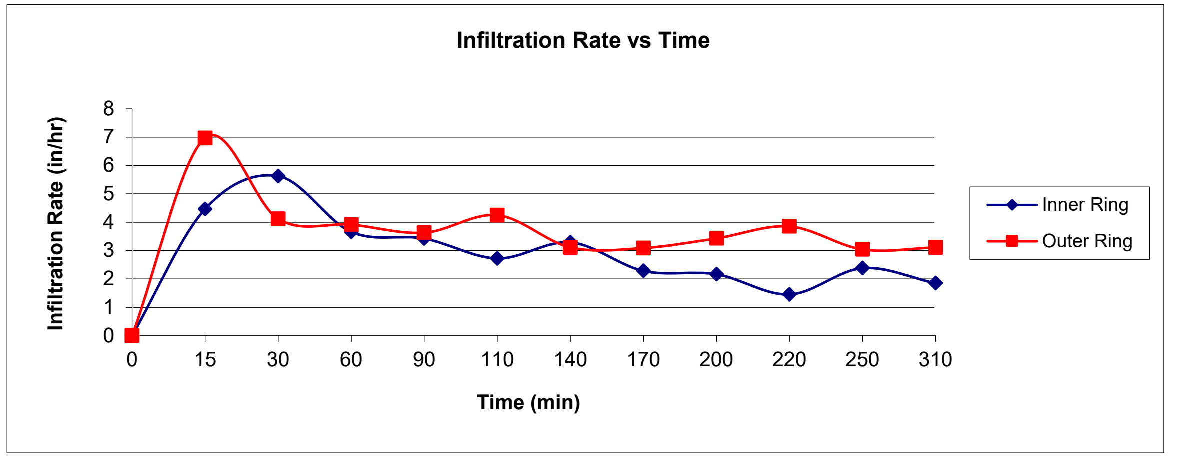
ENCLOSURE C-2

APPENDIX D

Infiltration Test Results

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 9, 2020
Soil Classification:	(SM) Silty Sand	**Test Hole No.:**	DRI-1
Depth of Test Hole:	6 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	A.L. & C.P.	**Depth of Water in Rings:**	5.5 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	2 in
Depth to Water Table:	300+ ft		

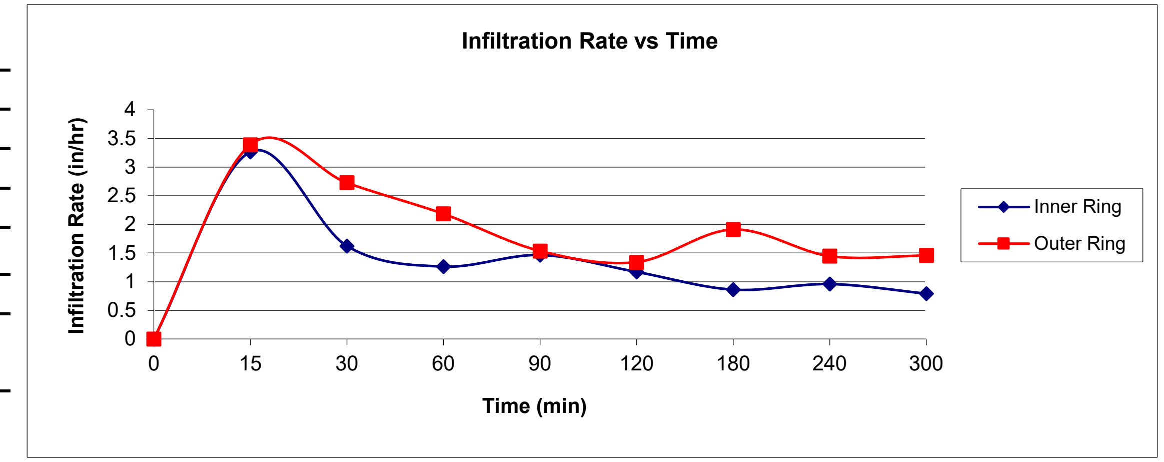


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:08	15	15	9:08	15	15	4.54	21.30	0.545	2.557	66.7	104.0	4.5	7.0	58	outer ring leak
	E	9:23			9:23											58	
2	S	9:23	15	30	9:23	15	30	5.72	12.58	0.687	1.510	84.0	61.4	5.6	4.1	58	outer emptied
	E	9:38			9:38											58	
3	S	9:45	30	60	9:45	30	60	7.44	23.93	0.893	2.873	54.6	58.4	3.7	3.9	59	outer refill
	E	10:15			10:15											59	
4	S	10:19	30	90	10:19	30	90	6.96	22.18	0.836	2.663	51.1	54.2	3.4	3.6	59	outer refill
	E	10:49			10:49											59	
5	S	10:49	20	110	10:49	20	110	3.69	17.31	0.443	2.078	40.6	63.4	2.7	4.2	59	outer emptied
	E	11:09			11:09											59	
6	S	11:13	30	140	11:13	30	140	6.69	19.01	0.803	2.282	49.1	46.4	3.3	3.1	60	outer refill
	E	11:43			11:43											60	
7	S	11:45	30	170	11:45	30	170	4.64	18.88	0.557	2.267	34.1	46.1	2.3	3.1	60	inner refill (outer emptied)
	E	12:15			12:15											60	
8	S	12:18	30	200	12:18	30	200	4.40	20.96	0.528	2.516	32.3	51.2	2.2	3.4	60	outer refill
	E	12:48			12:48											60	
9	S	12:48	20	220	12:48	20	220	1.97	15.69	0.236	1.884	21.7	57.5	1.5	3.9	60	outer emptied
	E	13:08			13:08											60	
10	S	13:13	30	250	13:13	30	250	4.84	18.57	0.581	2.229	35.5	45.3	2.4	3.0	60	outer refill
	E	13:43			13:43											60	
11	S	13:52	60	310	13:52	60	310	7.51	38.03	0.902	4.565	27.6	46.4	1.8	3.1	61	
	E	14:52			14:52											61	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 9, 2020
Soil Classification:	(SM) - Silty Sand	**Test Hole No.:**	DRI-2
Depth of Test Hole:	10 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	A.L. & C.P.	**Depth of Water in Rings:**	5 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3.5 in
Depth to Water Table:	300+ ft		

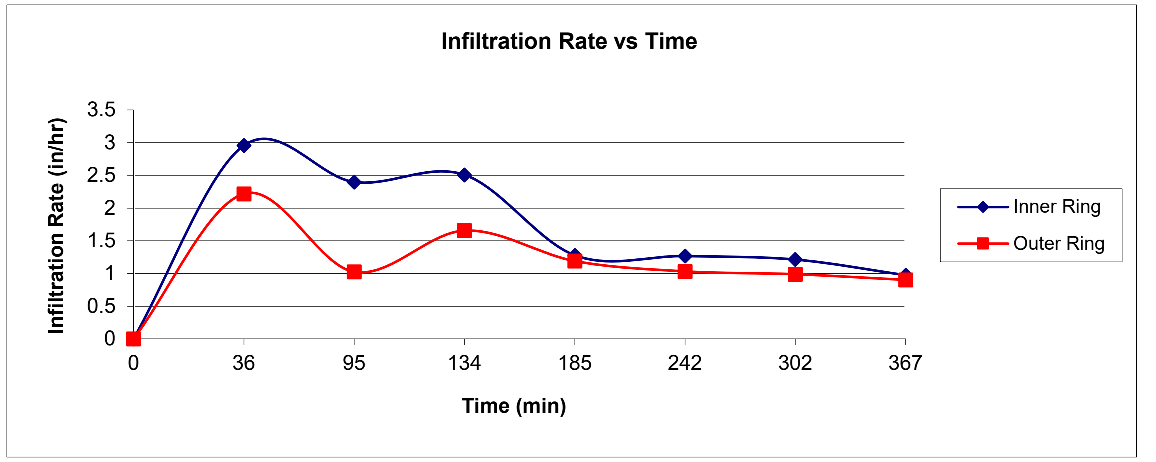


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELAPSED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:40	15	15	9:40	15	15	3.32	10.36	0.399	1.244	48.7	50.6	3.3	3.4	58	
	E	9:55			9:55											58	
2	S	9:55	15	30	9:55	15	30	1.65	8.33	0.198	1.000	24.2	40.7	1.6	2.7	58	
	E	10:10			10:10											58	
3	S	10:10	30	60	10:10	30	60	2.57	13.35	0.309	1.603	18.9	32.6	1.3	2.2	59	
	E	10:40			10:40											59	
4	S	10:45	30	90	10:45	30	90	2.98	9.39	0.358	1.127	21.9	22.9	1.5	1.5	59	
	E	11:15			11:15											59	
5	S	11:15	30	120	11:15	30	120	2.38	8.20	0.286	0.984	17.5	20.0	1.2	1.3	59	
	E	11:45			11:45											59	
6	S	11:45	30	150	11:45	30	150	1.86	9.88	0.223	1.186	13.7	24.1	0.9	1.6	60	
	E	12:15			12:15											60	
7	S	12:15	30	180	12:15	30	180	1.75	11.68	0.210	1.402	12.8	28.5	0.9	1.9	60	inner bottle leak
	E	12:45			12:45											60	
8	S	12:49	60	240	12:49	60	240	3.90	17.68	0.468	2.122	14.3	21.6	1.0	1.4	60	outer refill
	E	13:49			13:49											60	
9	S	13:49	60	300	13:49	60	300	3.22	17.81	0.387	2.138	11.8	21.7	0.8	1.5	60	
	E	14:49			14:49											61	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 11, 2020
Soil Classification:	(SM) Silty Sand	**Test Hole No.:**	DRI-3
Depth of Test Hole:	12 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	B.M.	**Depth of Water in Rings:**	4 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3 in
Depth to Water Table:	300+ ft		

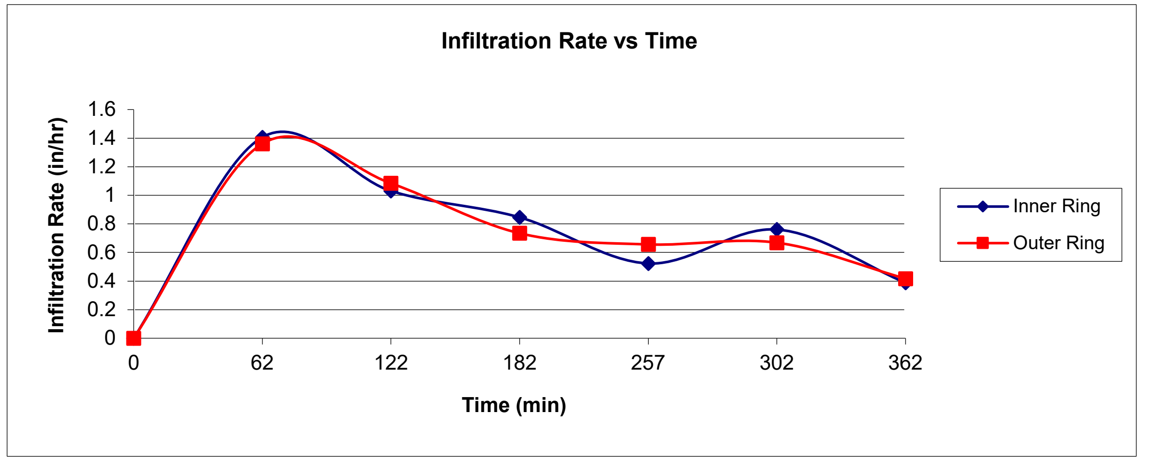


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	8:39	36	36	8:39	36	36	7.21	16.27	0.866	1.953	44.1	33.1	3.0	2.2	55	
	E	9:15			9:15											55	
2	S	9:15	59	95	9:15	59	95	9.59	12.31	1.151	1.478	35.8	15.3	2.4	1.0	55	
	E	10:14			10:14											55	
3	S	10:14	39	134	10:14	39	134	6.63	13.15	0.796	1.579	37.4	24.7	2.5	1.7	55	
	E	10:53			10:53											56	
4	S	10:53	51	185	10:53	51	185	4.42	12.36	0.531	1.484	19.1	17.8	1.3	1.2	56	
	E	11:44			11:44											56	
5	S	11:48	57	242	11:48	57	242	4.89	11.98	0.587	1.438	18.9	15.4	1.3	1.0	56	
	E	12:45			12:45											56	
6	S	12:45	60	302	12:45	60	302	4.93	12.08	0.592	1.450	18.1	14.7	1.2	1.0	57	
	E	13:45			13:45											57	
7	S	13:45	65	367	13:45	65	367	4.29	11.94	0.515	1.433	14.5	13.5	1.0	0.9	57	
	E	14:50			14:50											57	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 11, 2020
Soil Classification:	(SM) Silty Sand	**Test Hole No.:**	DRI-4
Depth of Test Hole:	12 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	B.M.	**Depth of Water in Rings:**	3 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3 in
Depth to Water Table:	300+ ft		

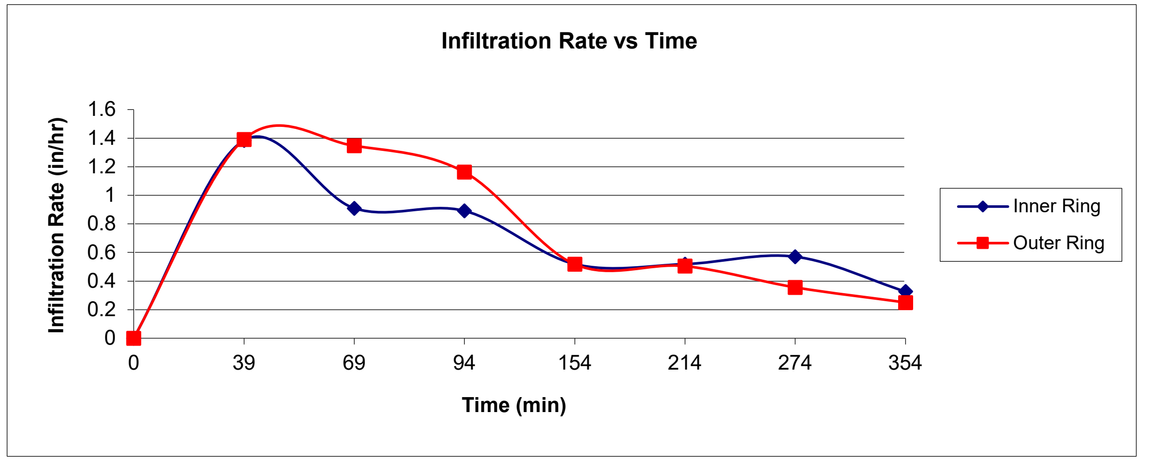


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	8:58	62	62	8:58	62	62	5.91	17.19	0.709	2.064	21.0	20.3	1.4	1.4	56	
	E	10:00			10:00											56	
2	S	10:00	60	122	10:00	60	122	4.19	13.25	0.503	1.591	15.4	16.2	1.0	1.1	56	
	E	11:00			11:00											56	
3	S	11:00	60	182	11:00	60	182	3.44	8.98	0.413	1.078	12.6	11.0	0.8	0.7	57	
	E	12:00			12:00											57	
4	S	12:00	75	257	12:00	75	257	2.66	10.03	0.319	1.204	7.8	9.8	0.5	0.7	57	
	E	13:15			13:15											58	
5	S	13:15	45	302	13:15	45	302	2.32	6.13	0.279	0.736	11.4	10.0	0.8	0.7	58	
	E	14:00			14:00											58	
6	S	14:00	60	362	14:00	60	362	1.58	5.10	0.190	0.612	5.8	6.2	0.4	0.4	59	
	E	15:00			15:00											59	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 11, 2020
Soil Classification:	(SM) Silty Sand	**Test Hole No.:**	DRI-5
Depth of Test Hole:	12 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	J.S.	**Depth of Water in Rings:**	3.5 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3 in
Depth to Water Table:	300+ ft		

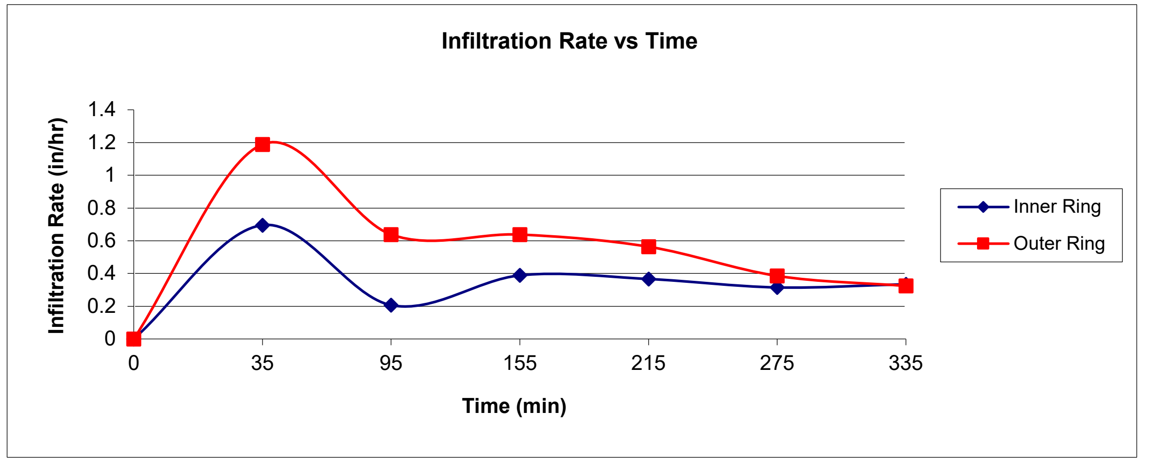


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:31	39	39	9:31	39	39	3.66	11.06	0.439	1.328	20.7	20.8	1.4	1.4	55	
	E	10:10			10:10											55	
2	S	10:10	30	69	10:10	30	69	1.85	8.23	0.222	0.988	13.6	20.1	0.9	1.3	55	
	E	10:40			10:40											56	
3	S	10:40	25	94	10:40	25	94	1.51	5.92	0.181	0.711	13.3	17.3	0.9	1.2	56	
	E	11:05			11:05											56	
4	S	11:05	60	154	11:05	60	154	2.11	6.33	0.253	0.760	7.7	7.7	0.5	0.5	56	
	E	12:05			12:05											56	
5	S	12:10	60	214	12:10	60	214	2.11	6.17	0.253	0.741	7.7	7.5	0.5	0.5	56	
	E	13:10			13:10											57	
6	S	13:10	60	274	13:10	60	274	2.32	4.35	0.279	0.522	8.5	5.3	0.6	0.4	57	
	E	14:10			14:10											57	
7	S	14:10	80	354	14:10	80	354	1.77	4.08	0.212	0.490	4.9	3.7	0.3	0.3	57	
	E	15:30			15:30											57	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 11, 2020
Soil Classification:	(SM) Silty Sand	**Test Hole No.:**	DRI-6
Depth of Test Hole:	11 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	J.S.	**Depth of Water in Rings:**	3 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3 in
Depth to Water Table:	300+ ft		

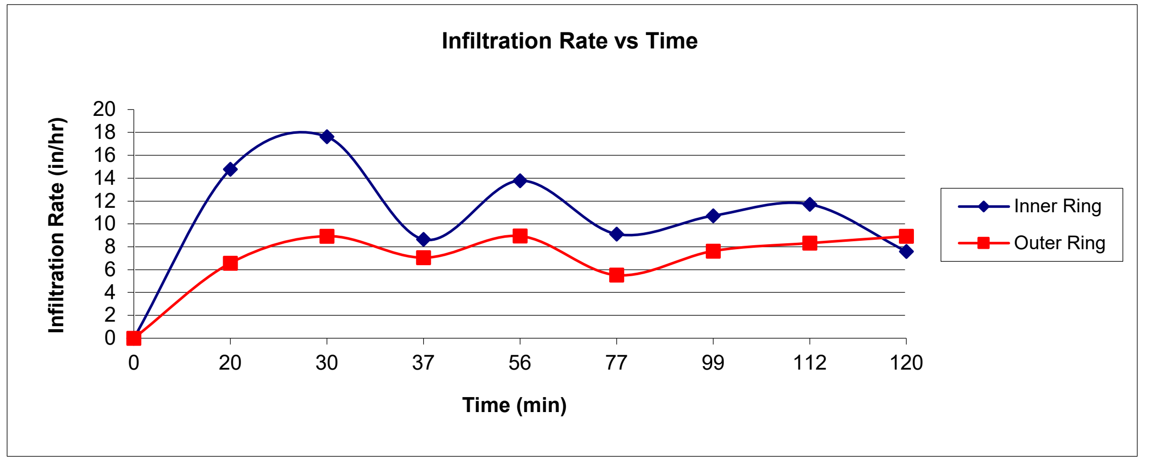


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:55	35	35	9:55	35	35	1.65	8.48	0.198	1.018	10.4	17.7	0.7	1.2	55	
	E	10:30			10:30											55	
2	S	10:30	60	95	10:30	60	95	0.84	7.80	0.101	0.936	3.1	9.5	0.2	0.6	55	
	E	11:30			11:30											55	
3	S	11:30	60	155	11:30	60	155	1.58	7.80	0.190	0.936	5.8	9.5	0.4	0.6	55	
	E	12:30			12:30											56	
4	S	12:30	60	215	12:30	60	215	1.49	6.89	0.179	0.827	5.5	8.4	0.4	0.6	56	
	E	13:30			13:30											56	
5	S	13:30	60	275	13:30	60	275	1.28	4.71	0.154	0.565	4.7	5.8	0.3	0.4	56	
	E	14:30			14:30											56	
6	S	14:30	60	335	14:30	60	335	1.36	3.96	0.163	0.475	5.0	4.8	0.3	0.3	57	
	E	15:30			15:30											57	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 12, 2020
Soil Classification:	(SW) Well Graded Sand	**Test Hole No.:**	DRI-7
Depth of Test Hole:	11 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	F.J.	**Depth of Water in Rings:**	3 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	4 in
Depth to Water Table:	300+ ft		

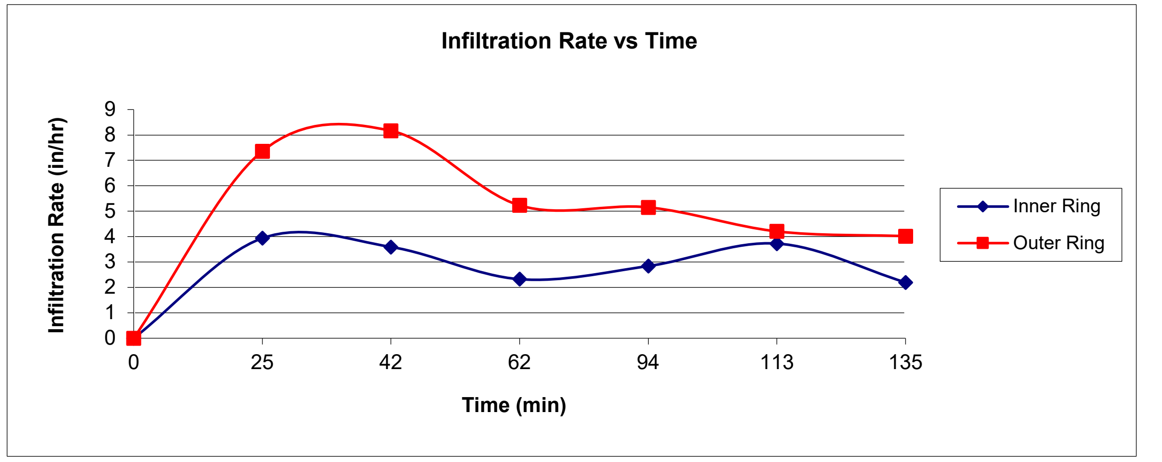


TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:45	20	20	9:45	20	20	20.05	26.79	2.407	3.216	220.8	98.1	14.8	6.6	55	
	E	10:05			10:05											55	
2	S	10:05	10	30	10:05	10	30	11.94	18.19	1.433	2.184	262.9	133.2	17.6	8.9	55	
	E	10:15			10:15											55	
3	S	10:15	7	37	10:15	7	37	4.10	10.05	0.492	1.206	129.0	105.2	8.6	7.0	55	
	E	10:22			10:22											56	
4	S	10:35	19	56	10:35	19	56	17.75	34.59	2.131	4.152	205.7	133.4	13.8	8.9	56	both refill
	E	10:54			10:54											56	
5	S	11:04	21	77	11:04	21	77	12.97	23.62	1.557	2.836	136.0	82.4	9.1	5.5	56	outer refill
	E	11:25			11:25											56	
6	S	11:44	22	99	11:44	22	99	15.98	34.20	1.918	4.106	160.0	113.9	10.7	7.6	57	outer refill
	E	12:06			12:06											57	
7	S	12:15	13	112	12:15	13	112	10.32	22.04	1.239	2.646	174.8	124.2	11.7	8.3	57	outer refill
	E	12:28			12:28											57	
8	S	12:37	8	120	12:37	8	120	4.12	14.53	0.495	1.744	113.4	133.0	7.6	8.9	57	
	E	12:45			12:45											57	

DOUBLE RING INFILTROMETER TEST DATA

Project:	NEC Beaumont Ave & Oak Valley	**Client:**	Santiago Holdings, LLC
Project No.:	13627.1	**Test Date:**	March 12, 2020
Soil Classification:	(SW-SM) Well Graded Sand w/ Silt	**Test Hole No.:**	DRI-8
Depth of Test Hole:	14 ft	**Test Hole Diameter:**	12 in. inner, 24 in. annular
Liquid Used:	Tap Water	**Date Excavated:**	March 9, 2020
Area of Rings:	Inner = 0.785 ft^2 , Annular 2.36 ft^2	**pH:**	7.8
Tested By:	F.J.	**Depth of Water in Rings:**	3.5 in
Liquid Level Maintained Using:	Vacuum Seal	**Ring Penetration:**	3 in
Depth to Water Table:	300+ ft		



TEST PERIOD

TRIAL NO.	INNER				ANNULAR SPACE			WATER USED (lbs.)		WATER USED (gal)		INFILTRATION RATE (gal/sf.day)		INFILTRATION RATE (in/hr)		LIQUID TEMP (°F)	REMARKS
	TIME		TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	TIME	TIME INTERVAL (minutes)	TOTAL ELASPED TIME (minutes)	inner	annular space	inner	annular space	inner	annular space	inner	annular space		
1	S	9:55	25	25	9:55	25	25	6.67	37.50	0.801	4.502	58.8	109.9	3.9	7.4	55	outer refill
	E	10:20			10:20											55	
2	S	10:23	17	42	10:23	17	42	4.14	28.28	0.497	3.395	53.6	121.9	3.6	8.2	55	outer refill
	E	10:40			10:40											55	
3	S	10:52	20	62	10:52	20	62	3.15	21.31	0.378	2.558	34.7	78.1	2.3	5.2	56	outer refill
	E	11:12			11:12											56	
4	S	11:19	32	94	11:19	32	94	6.16	33.60	0.739	4.034	42.4	76.9	2.8	5.2	56	outer refill
	E	11:51			11:51											56	
5	S	12:00	19	113	12:00	19	113	4.80	16.28	0.576	1.954	55.6	62.8	3.7	4.2	57	
	E	12:19			12:19											57	
6	S	12:19	22	135	12:19	22	135	3.28	18.01	0.394	2.162	32.8	60.0	2.2	4.0	57	
	E	12:41			12:41											57	

APPENDIX E

Seismic Design Spectrum

Project: APN's 404-190-001 & -003
Project Number: 13627.1
Client: Santigo Holdings, LLC, c/o Thatcher Engineering & Associates
Site Lat/Long: 33.9478/-116.9786
Controlling Seismic Source: Southern San Andreas

REFERENCE	NOTATION	VALUE
Site Class	A, B, C, D, E, or F	D measured
Site Class D - Table 11.4-1	F_a	1.0
Site Class D - 21.2.3.(ii)	F_v	2.5
$0.2*(S_{D1}/S_{DS})$	T_0	0.116
S_{D1}/S_{DS}	T_S	0.582
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}	0.773
Equation 11.4-2 - F_V*S_1	S_{M1}	1.159

REFERENCE	NOTATION	VALUE
F_v (Table 11.4-2)[Used for General Spectrum]	F_v	1.7
Design Maps	S_s	1.993
Design Maps	S_1	0.682
Equation 11.4-1 - F_A*S_S	S_{MS}	1.993
Equation 11.4-3 - $2/3*S_{MS}$	S_{DS}	1.329
Design Maps	PGA	0.812
Table 11.8-1	F_{PGA}	1.1
Equation 11.8-1 - F_{PGA}*PGA	PGA_M	0.893
Section 21.5.3	80% of PGA_M	0.715
Design Maps	C_{RS}	0.919
Design Maps	C_{R1}	0.893

RISK COEFFICIENT

Cr - At Perods <=0.2, Cr=Crs	C_{RS}	0.919
Cr - At Periods >=1.0, Cr=Cr1	C_{R1}	0.893

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

Period	Cr
0.200	0.919
0.300	0.916
0.400	0.913
0.500	0.909
0.600	0.906
0.680	0.903
1.000	0.893

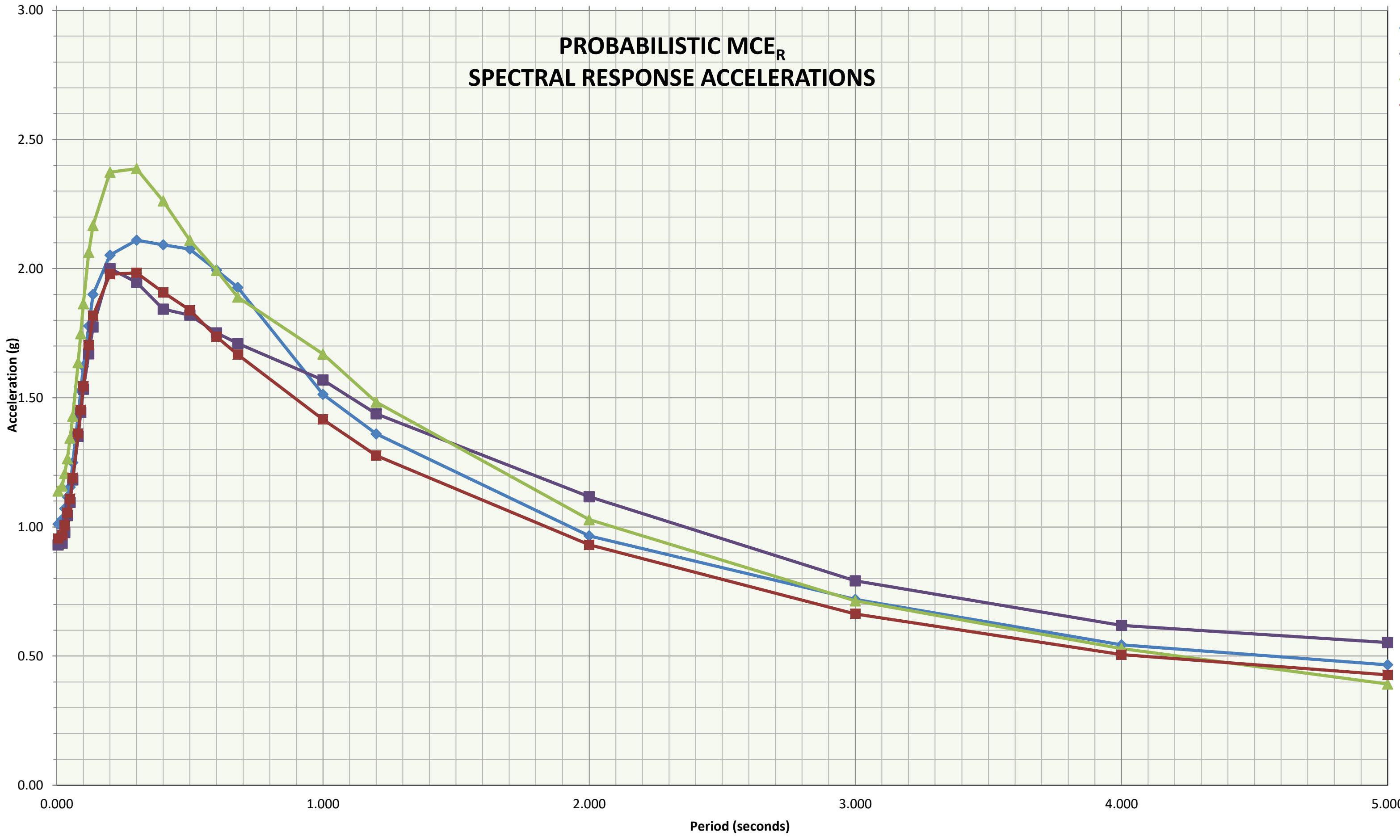
PROBABILISTIC SPECTRA
2% in 50 year Exceedence

Period	B - A	C - B	C - Y	Mean	Risk Coefficient (C_R)	Probabilistic MCE
0.005	1.01	0.93	1.14	1.04	0.919	0.955
0.020	1.03	0.94	1.16	1.05	0.919	0.969
0.030	1.07	0.98	1.21	1.10	0.919	1.007
0.040	1.12	1.04	1.26	1.15	0.919	1.056
0.050	1.15	1.10	1.34	1.21	0.919	1.109
0.060	1.25	1.18	1.43	1.30	0.919	1.190
0.080	1.43	1.35	1.64	1.48	0.919	1.361
0.090	1.53	1.44	1.75	1.58	0.919	1.453
0.100	1.62	1.53	1.86	1.68	0.919	1.545
0.120	1.78	1.67	2.06	1.85	0.919	1.704
0.136	1.90	1.77	2.17	1.98	0.919	1.818
0.200	2.05	2.00	2.37	2.15	0.919	1.979
0.300	2.11	1.95	2.39	2.17	0.916	1.984
0.400	2.09	1.84	2.26	2.09	0.913	1.908
0.500	2.08	1.82	2.11	2.02	0.909	1.839
0.600	1.99	1.75	1.99	1.92	0.906	1.736
0.680	1.93	1.71	1.89	1.85	0.903	1.667
1.000	1.51	1.57	1.67	1.59	0.893	1.416
1.200	1.36	1.44	1.48	1.43	0.893	1.276
2.000	0.97	1.12	1.03	1.04	0.893	0.931
3.000	0.72	0.79	0.71	0.74	0.893	0.663
4.000	0.54	0.62	0.53	0.57	0.893	0.506
5.000	0.47	0.55	0.39	0.48	0.893	0.427

B-A - Boore-Atkinson (2008) NGA USGS 2008 MRC
C-B - Campbell-Bozorgnia (2008) NGA USGS 2008 MRC
C-Y - Chiou-Youngs (2007) NGA USGS 2008 MRC

Probabilistic PGA: 0.955
Is $Sa_{(max)}$<$1.2F_a$? NO

Project No: 13627.1



Analysis Information

Spectral Response @ 5 % Damping with Maximum Rotated Component.

Probability of Exceedence:
2% in 50 years

Project No: 13627.1

LOR GEOTECHNICAL GROUP, INC.

DETERMINISTIC SPECTRUM AND LOWER LIMIT

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations*.

Controlling Source: Southern San Andreas

Period	DETERMINISTIC (RAW)	DETERMINISTIC MCE 84 FRACTILE
0.005	0.665	0.665
0.020	0.679	0.679
0.030	0.711	0.711
0.040	0.748	0.748
0.050	0.784	0.784
0.060	0.838	0.838
0.080	0.944	0.944
0.090	0.998	0.998
0.100	1.049	1.049
0.120	1.140	1.140
0.136	1.203	1.203
0.200	1.290	1.290
0.300	1.353	1.353
0.400	1.350	1.350
0.500	1.371	1.371
0.600	1.357	1.357
0.680	1.346	1.346
1.000	1.240	1.240
1.200	1.156	1.156
2.000	0.883	0.883
3.000	0.685	0.685
4.000	0.531	0.531
5.000	0.414	0.414

Is Sa(max)<1.2Fa? NO

Deterministic PGA: 0.665

*Attenuation Equations

Boore - Atkinson (2008) NGA USGS 2008 MRC

Campbell - Bozorgnia (2008) NGA USGS 2008 MRC

Chiou - Youngs (2007) NGA USGS 2008 MRC

Project No: 13627.1

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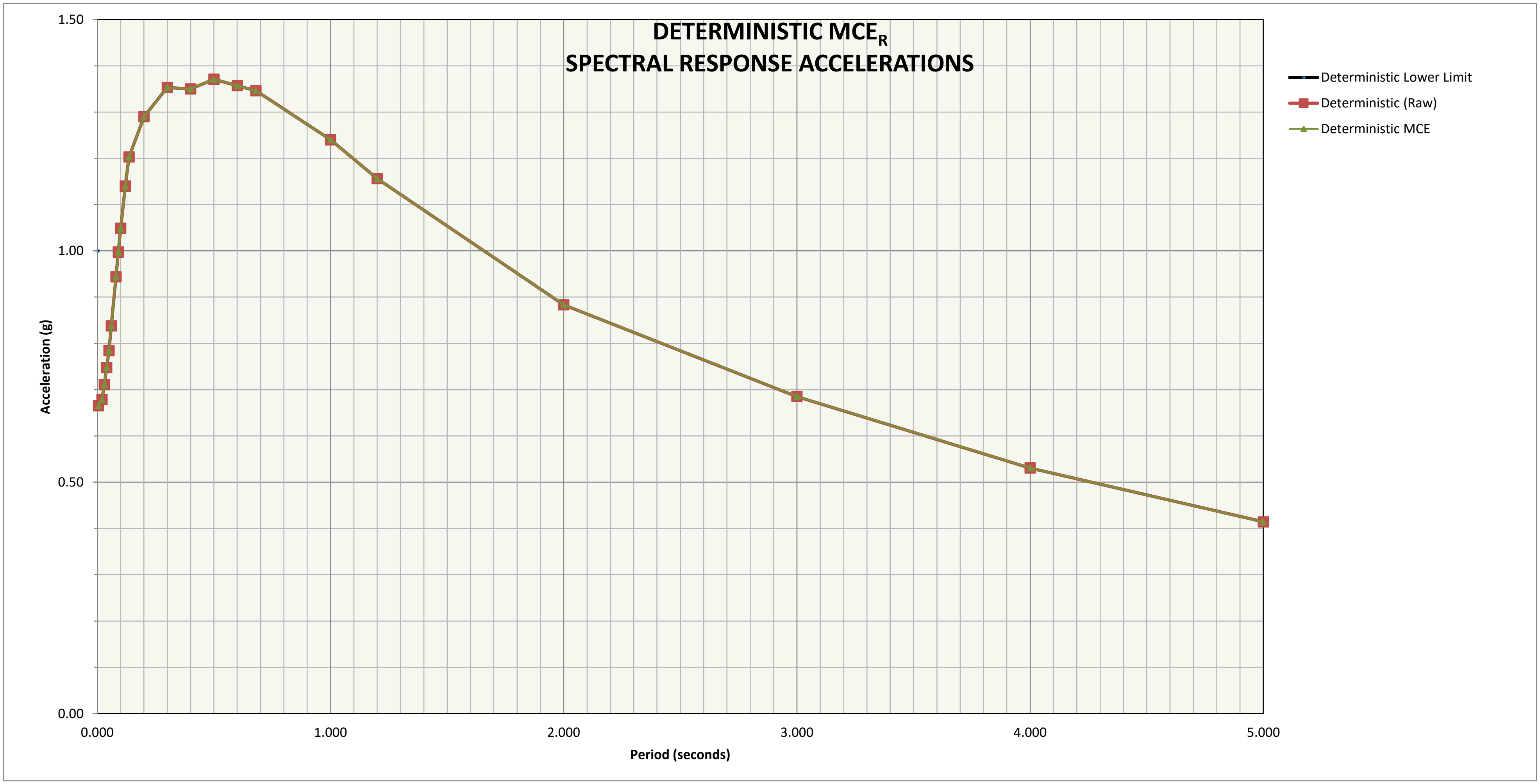

Project No: 13627.1

LOR GEOTECHNICAL GROUP, INC.

SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.005	0.955	0.665	0.665	0.453
0.020	0.969	0.679	0.679	0.480
0.030	1.007	0.711	0.711	0.590
0.040	1.056	0.748	0.748	0.644
0.050	1.109	0.784	0.784	0.699
0.060	1.190	0.838	0.838	0.754
0.080	1.361	0.944	0.944	0.864
0.090	1.453	0.998	0.998	0.919
0.100	1.545	1.049	1.049	0.973
0.120	1.704	1.140	1.140	1.063
0.136	1.818	1.203	1.203	1.063
0.200	1.979	1.290	1.290	1.063
0.300	1.984	1.353	1.353	1.063
0.400	1.908	1.350	1.350	1.063
0.500	1.839	1.371	1.371	1.063
0.600	1.736	1.357	1.357	1.063
0.680	1.667	1.346	1.346	0.909
1.000	1.416	1.240	1.240	0.827
1.200	1.276	1.156	1.156	0.771
2.000	0.931	0.883	0.883	0.589
3.000	0.663	0.685	0.663	0.442
4.000	0.506	0.531	0.506	0.337
5.000	0.427	0.414	0.414	0.276

Period	ASCE 7 SECTION 11.4.6 General Spectrum	80% General Response Spectrum
0.005	0.566	0.453
0.010	0.600	0.480
0.030	0.737	0.590
0.040	0.806	0.644
0.050	0.874	0.699
0.060	0.943	0.754
0.080	1.080	0.864
0.090	1.148	0.919
0.100	1.217	0.973
0.110	1.285	1.028
0.120	1.329	1.063
0.136	1.329	1.063
0.150	1.329	1.063
0.160	1.329	1.063
0.170	1.329	1.063
0.180	1.329	1.063
0.190	1.329	1.063
0.200	1.329	1.063
0.300	1.329	1.063
0.400	1.329	1.063
0.500	1.329	1.063
0.580	1.329	1.063
0.640	1.208	0.966
0.680	1.137	0.909
0.850	0.909	0.727
0.900	0.859	0.687
0.950	0.814	0.651
1.000	0.773	0.618
1.200	0.644	0.515
2.000	0.386	0.309
3.000	0.258	0.206
4.000	0.193	0.155
5.000	0.155	0.124

ASCE 7-16: Section 21.4

	Calculated Value	Design Value
SDS:	~~0.957~~	1.063
SD1:	1.381	1.381
SMS:	~~1.435~~	1.594
SM1:	2.071	2.071
Site Specific PGAm:	~~0.665~~	0.715
Site Class:	D measured	

Seismic Design Category - Short* D

Seismic Design Category - 1s* D

* Risk Categories I, II, or III

Project No: 13627.1

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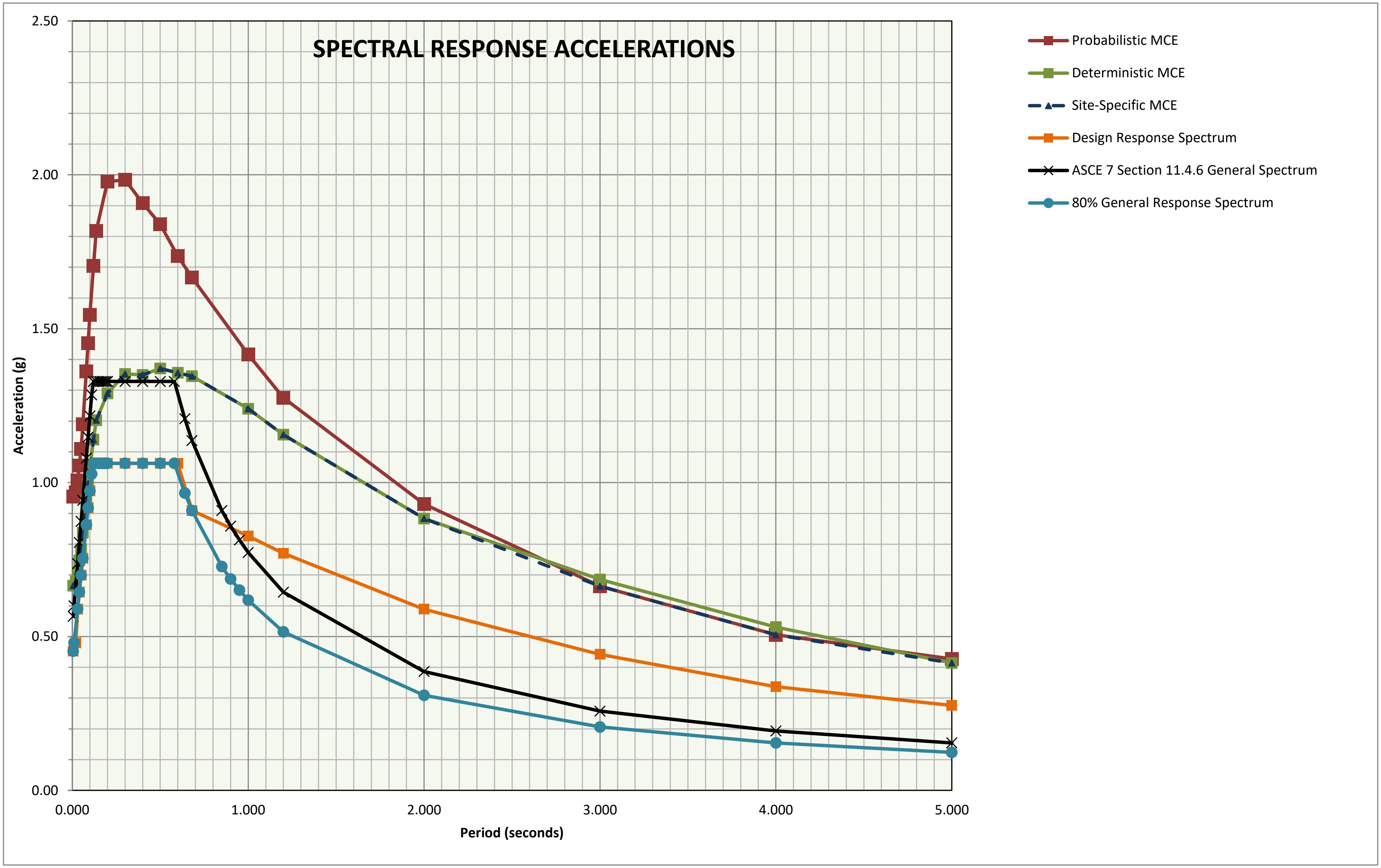

Project No: 13627.1

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INFILTRATION BASIN DRAWDOWN CALCULATIONS

DA B (DRI 2 & DRI 8)

AVERAGE RATE = 2.75"/HR / SAFETY FACTOR OF 3 = 0.92"/HR X 72 HOURS = MAX DEPTH 66" (Though some storage of water will occur in rock above and below pipe, max depth counted toward DCV shall be 66")

DA C (DRI 7 & DRI 5)

AVERAGE RATE = 4.6"/HR / SAFETY FACTOR OF 3 = 1.53"/HR X 72 HOURS = MAX DEPTH 110"

DA B (DRI 3 & DRI 4)

AVERAGE RATE = 0.65"/HR / SAFETY FACTOR OF 3 = 0.216"/HR X 72 HOURS = MAX DEPTH 15" (Though some storage of water will occur in rock above and below pipe, max depth counted toward DCV shall be 15")