SECTION H

GEOTECHNICAL REPORT

AYALA PARK
SPLASH PAD PROJECT

FOR

BLOOMINGTON RECREATION AND PARK DISTRICT
BLOOMINGTON, CALIFORNIA

PROJECT NO.: 30.30.0169

GEOTECHNICAL EXPLORATION
AYALA PARK SPLASH PAD PROJECT
17909 MARYGOLD AVENUE
CITY OF BLOOMINGTON, SAN BERNARDINO COUNTY, CALIFORNIA
PROJECT SERVICE REQUEST #SD008

Prepared For **SAN BERNARDINO COUNTY DEPARTMENT OF PUBLIC WORKS – SPECIAL DISTRICTS**
222 W. Hospitality Lane, Second Floor
San Bernardino, California 92415-0450

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Project No. 038.0000022346

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San Bernardino County
Department of Public Works - Special Districts
222 W. Hospitality Lane, Second Floor
San Bernardino, California 92415-0450

Attention: Chuck Hernandez
Project Manager

Subject: Geotechnical Exploration
Ayala Park Splash Project
17909 Marygold Avenue
City of Bloomington, San Bernardino County, California
Project Service Request #SD008

In accordance with our March 18, 2024, proposal, Leighton Consulting, Inc. (Leighton) has completed this geotechnical exploration in support of design of the proposed Splash Pad project within the existing Ayala Park located at 17909 Marygold Avenue in the City of Bloomington, California. The purpose of our exploration was to evaluate geologic hazards and geotechnical conditions of the site with respect to the proposed improvements and to provide geotechnical recommendations for design and construction of the proposed Splash Pad project.

This site is not located within a designated Alquist-Priolo Earthquake Fault Zone. However, as is the case for most of southern California, strong ground shaking has and will occur at this site. Groundwater levels in this area have been estimated to be 287 feet or deeper below the surface. Native site soils encountered during exploration have been characterized primarily as medium dense to dense silty sand, sands, and sands with silt and gravel; liquefaction is highly unlikely to occur at this site due to deep groundwater levels and the dense nature of encountered soils.

The proposed improvements are feasible from a geotechnical standpoint, provided that the geotechnical recommendations presented in this report are implemented during design and construction.

We appreciate this opportunity to be of additional service to the Special Districts Department. If you have any questions or if we can be of further service, please contact us at your convenience at ***866-LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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TABLE OF CONTENTS

Section **Page**

<u>List of Figures (Behind References)</u>

<u>Appendices</u>

1.0 INTRODUCTION

1.1 Site Location and Description

As depicted on Figure 1, *Site Location Map*, the proposed Splash Pad project is located within the existing Ayala Park located at 17909 Marygold Avenue in the City of Bloomington, San Bernardino County, California (latitude 34.0737° and longitude -117.4134°). Specifically, the site is bounded to the north by Marygold Avenue, to the east by single family homes, to the west by a cable installation equipment storage yard and a single-family residence, and to the south by a multi-family development. The approximately 4.45-acre property, which has been developed as the existing park, has been mapped as Assessor Parcel Number (APN) 0252-051-77 by the County of San Bernardino.

Based on our review of aerial imagery, dating back to 1938, much of the site was previously vacant, with the exception of the northern portion, which used to contain a single-family residence. Sometime between 2002 and 2018, aerial imagery appears to show the site used as a storage yard for vehicles and equipment. Between 2018 and 2019, the residence to the north was demolished and had remained vacant until the construction of the park, which appears to have taken place between 2020 and 2023. The site has retained the same layout since 2023 (NETR, 2024).

1.2 Proposed Improvements

Based on the provided *Project Service Request #SD008 for the Ayala Park Splash Pad Project,* dated February 29, 2024, we understand that the San Bernardino County Department of Public Works, Special Districts is currently proposing to add a new splash pad and an associated infiltration system within the existing park. Based on the provided *Exhibit B – Location Map*, the proposed splash pad will have an approximate 40-foot-diameter footprint and be located east of the existing dog park towards the northern portion of the park. At the time of this report, the project is early in its design phase and no structural plans or equipment plans were available. Based on our experience with similar projects, we anticipate that proposed splash pad improvements will generally include an underground water delivery system, aquatic play equipment, water features, fencing, a permeable water surface/anti-slip surface, flatwork improvements, landscaping improvements, and possible shade structures.

The project site is relatively flat and generally slopes gently towards the south with elevations within the proposed improvement areas of approximately 1,1322 feet above mean sea level. Grading plans were not available at the time of this report but based on the relatively flat and level surface of the park, we anticipate cuts and fill on the order of 3 feet or less will be required to construct the proposed improvements onsite.

1.3 Purpose and Scope of Exploration

The purpose of our work was to evaluate the subsurface conditions at the site relative to the proposed development and provide geotechnical recommendations to aid in project design and construction. The scope of this evaluation included the following tasks:

- *Background Review* – We reviewed readily available reports, literature, aerial photographs, and maps relevant to the site available from our in-house library or in the public domain. We evaluated geological hazards and potential geotechnical issues that may significantly affect the site. The documents reviewed are listed in the *References* section in the rear of this text.

- *Site Reconnaissance* – We performed a visual site reconnaissance to mark the locations proposed for hollow-stem auger test borings and to assess access throughout the site. Once the locations were marked, DigAlert (811) was notified for utility clearance. The services of a private utility locator were also retained in an effort to identify any private utility lines that were not marked by DigAlert and possibly in conflict with our proposed boring locations.

- *Field Exploration* – Field exploration was performed on April 16, 2024 and consisted of two (2) hollow-stem auger borings for geotechnical logging and sampling (designated as LB-1 and LP-1). Geotechnical borings were drilled to depths of 51.5 feet and 5 feet below ground surface (bgs), respectively. The approximate locations of the borings are shown on Figure 2, *Exploration Location Map.* Logs of the exploration are included in Appendix A - *Field Exploration Logs.*

  During advancement of the hollow-stem auger borings, bulk samples and drive samples were obtained for geotechnical laboratory testing. Drive samples were collected using a Modified California ring-lined sampler with sampling conducted in accordance with ASTM Test Method D 3550 and by the Standard Penetration Test (SPT) method in accordance with ASTM Test Method D 1586 within the hollow-stem auger borings. The ring and SPT samplers were driven for a total penetration of 18 inches using a 140-pound automatic hammer falling 30 inches. The number of blows per 6 inches of penetration was recorded on boring logs, see Appendix A – *Field Exploration Logs.* Bulk samples were collected from the upper 5 feet.

The borings were logged in the field by a member of our technical staff under the supervision of a State of California licensed Professional Engineer. Each soil sample collected was reviewed and described in general accordance with the Unified Soil Classification System. The samples were sealed and packaged for transportation to our in-house laboratory.

An infiltration test was conducted within boring LP-1, which was located within the proposed splash pad footprint. Testing was conducted at LP-1 at a depth of approximately 5 feet bgs to estimate infiltration characteristics of the underlying soil at the location and depth requested by the design team.

- *Laboratory Testing* – Geotechnical laboratory testing was performed on selected soil samples collected during our field exploration to determine engineering properties of encountered subsurface soils. The results of laboratory testing are presented in Appendix B – *Geotechnical Laboratory Testing.*
- *Engineering Analysis* – Geotechnical analysis was performed on the collected and available data to develop conclusions and preliminary recommendations for design and construction of the improvements as currently planned.
- *Report Preparation* – Results of our geotechnical study have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed splash pad improvements as currently planned.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located within the northern part of the Peninsular Ranges physiographic province, on the south of the San Gabriel Mountains. It occupies a broad, flat plain within the Upper Santa Ana Valley. This valley alluvial plain descends southward from the San Gabriel Mountains to the Santa Ana River.

Frontal faults of the San Gabriel Mountains have a predominantly dip-slip (reverse) component, typically northward at relatively shallow inclinations resulting in the uplift of the plutonic and metamorphic basement rock of the San Gabriel Mountains through the ground surface. This uplift has accommodated the erosion of confined channels within the San Gabriel Mountains, and sediment generated from these mountains has been transported and deposited onto the valley alluvial plain. Coalescing alluvial fans form a nearly continuous apron of sediments derived from the largely plutonic and metamorphic rocks of the adjacent San Gabriel Mountains about 4 miles to the north. These sediments consist of up to approximately 1200-foot thickness of gravel, sand, silt, and clay overlying crystalline and sedimentary basement rock (Fife, et al., 1976).

In general, surface drainage in the project region is directed towards the south. Cucamonga, Deer, Day, and Lytle Creeks drain the San Gabriel Mountains from the north, locally flowing through reinforced-concrete flood-control channels, then onto the lower alluvial plain to the Santa Ana River, and ultimately draining into Prado Dam Flood Control Basin.

As regionally mapped on Figure 3, *Regional Geology Map*, the site and surrounding alluvial fan are mapped as Holocene and late Pleistocene young alluvial fan deposits (Qyf) generally consisting of coarse grained sand to bouldery sediment.

2.2 Subsurface Soil Conditions

Based on results of our research and subsurface exploration site soils encountered to the depths explored consist of the following:

- **Undocumented Fill (Afu)**: We are unaware of any documentation of previous fill placement for this site, so we have characterized fill onsite as undocumented. Undocumented artificial fill was encountered in our borings to be generally about 3 to 3.5 feet thick. Fill encountered in our borings consisted of silty sands with variable amounts of silts and gravels.

- **Young Alluvial Fan Deposits (Qyf)**: The site has been regionally mapped as exposing native Young Alluvial Fan Deposits. Native onsite soils encountered in our borings below undocumented fill consisted of dense to very dense silty sands, sand with silts and gravels, and occasional interbedded hard sandy silts to explored depths. Moisture contents from the underlying soils within the upper 10 feet ranged from values of 4 to 13 percent while the dry densities ranged from 119 to 126 pounds per cubic foot (pcf).

More detailed descriptions of subsurface soils encountered are presented on our boring logs in Appendix A.

2.3 Groundwater

Groundwater was not encountered in our borings drilled onsite to a maximum explored depth of 51½ feet bgs. Historical data from groundwater elevation contour maps dating back to 1933 (CDWR, 1970) indicate groundwater levels in the area of the site are on the order of approximately 845 feet above mean sea level, which correlates to a depth of about 287 feet bgs from the lowest elevation at the site. Based on groundwater data from nearby State Well No. 01S05W20N001S, located approximately 1.1 miles southwest of the site with measurements dating from October 1, 1989, to March 3, 2022, the shallowest groundwater reading identified was measured on January 1, 2010, which was at an elevation of 834 feet above mean sea level (msl). This elevation correlates to a groundwater depth of approximately 298 feet bgs based on the lowest elevation at the project site (CDWR, 2023).

Based on the available regional groundwater level data reviewed, shallow groundwater (≤50 bgs) does not appear to be a significant geotechnical constraint to this project.

2.4 Faulting and Seismicity

Southern California is a seismically active area. As such, the site will be subject to seismic hazards from numerous sources in the area. The severity of potential seismic hazards is related to site-specific geology, distances from seismic sources, and the magnitude of earthquake events. Principal seismic hazards evaluated on a site-specific basis included: potential for surface rupture along active or potentially active fault traces, magnitude of seismic shaking, and the susceptibility to ground failure (liquefaction, lurching, and seismically induced landslides).

Based on our review of available in-house literature, there are no currently known active surface faults that traverse or trend towards this site, and this site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (CGS, 1995), nor a fault zone delineated by the County or City.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in this region could include soil liquefaction and earthquake-induced settlement. Site-specific potential for secondary seismic hazards is discussed in the following subsections:

2.5.1 Liquefaction Potential: The site has not been evaluated by the State of California for liquefaction hazards. San Bernardino County's Geologic Hazards Overlay map (FH 29 C) for the Fontana area indicates that the site is outside of a liquefaction susceptibility zone. However, groundwater was not encountered in our borings drilled to a maximum of 51½ feet below existing grade and collected data indicated that groundwater depths at and near this site have been historically 287 feet or deeper beneath the site. In addition, encountered granular alluvial soils onsite were generally medium dense to very dense, which are typically not susceptible to liquefaction. Based on the absence of shallow groundwater and the dense nature of the sands onsite, liquefaction is unlikely to occur at the site.

2.5.2 Seismically Induced Settlement: Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 287 feet below ground surface were used in the analysis. Based on our analysis, a potential for approximately 0.4 inch of seismic settlement is estimated at the site. Differential settlement due to seismic loading is assumed to be less than ½ inch over a horizontal distance of 30 feet based on the MCE.

Results of our seismic settlement analysis is presented in Appendix C.

2.6 Infiltration Testing

Infiltration testing was conducted within one of our borings excavated onsite (LP-1) to estimate the infiltration characteristics of the near surface onsite soils at the depths tested. Our test was conducted at a depth of approximately 5 feet below the surface.

Well permeameter tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. A test consists of excavating a boring to the depth of the test (or deeper as long as it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand or gravel is then placed in the boring bottom to temporarily support a perforated well casing pipe system. Once the well casing pipe has been installed, coarse sand or gravel is poured in the annular space outside of the well casing within the test zone to prevent the boring from caving/collapsing or spalling when water is added. Water is added into the boring to an initial water height, as water within the boring infiltrates into the soil, measurements are taken of the height of the water column within the boring at equally timed intervals (known as a falling head test). The infiltration rate as measured during intervals of the test is defined as the flow rate of water infiltrated, divided by the surface area of the infiltration interface. The test was conducted based on the USBR 7300-89 test method.

Results of the infiltration testing are summarized below and are provided in Appendix A. Further discussion of infiltration testing and related considerations are included in Section 3.4.

The following table summarizes raw infiltration rate estimates from our testing.

Table 1. Raw Infiltration Rates

Boring	Test Depth (ft)	Soil Classification	Raw Infiltration Rates (in/hr)
LP-1	5	SM	0.1

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

This site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture. However, as is the case for most of southern California, strong ground shaking has and will occur at this site. Groundwater levels are on the order of 287 feet below the surface or deeper based on available data. Encountered native site soils were medium dense to very dense granular soils; therefore, liquefaction is highly unlikely to occur at this site.

3.2 Earthwork

Project earthwork is expected to include overexcavation and recompaction of undocumented fill soils and onsite alluvial soils as described in the following subsections:

3.2.1 Earthwork Observation and Testing: Leighton should observe and test all grading and earthwork to check that the site has been properly prepared, to assess that selected fill materials are satisfactory, and to evaluate the placement and compaction of fills to be performed in accordance with our recommendations and the project specifications. Any imported soil or aggregate material to be evaluated for its suitability as onsite fill material should be submitted to a Leighton geotechnical laboratory at least two working days in advance of earth material placement and compaction. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, we should be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Conclusions and recommendations presented in this report are contingent upon construction geotechnical observation services.

3.2.2 Surface Drainage: Water should not be allowed to pond or accumulate anywhere except in approved outlet structures or drainage facilities and should be setback at least 15 feet from any proposed or existing structures. Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns and drainpipes approved at the time of fine grading should be maintained throughout the life of proposed structures.

3.2.3 **Site Preparation:** Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

Based on encountered site conditions, we recommend that existing fill soils under proposed structures be excavated and recompacted. Underground obstructions encountered should be removed. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted where interfering with proposed construction. Trees to be removed should be grubbed out.

Areas planned for structures with shallow foundations should be overexcavated to a minimum depth of 2 feet below the bottoms of the proposed footings or at least 3 feet below existing grade; whichever is deeper. Areas outside proposed structures/monument, planned for asphalt and/or concrete pavement, should be overexcavated to a minimum depth of 18 inches below existing or finish grade, or 12 inches below proposed pavement sections; whichever is deeper.

Resulting removal excavation bottom surfaces should be observed by Leighton prior to placement of backfill or new construction. Existing fill soils be excavated from the proposed structure foundations, regardless of depth. After overexcavations are completed and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve).

3.2.4 **<u>Fill Placement and Compaction</u>:** Onsite soils free of organics and debris are suitable for use as compacted structural fill provided it is free of oversized material greater than 8 inches in its largest dimension. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill should be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 3 percent above optimum moisture content, and compacted to a minimum 90% relative compaction as determined by ASTM D1557 standard test method (modified Proctor compaction curve). Aggregate base for pavement sections should be compacted to a minimum of 95% relative compaction.

Cobbles were encountered during advancement of our explorations onsite. Oversize material and debris, if encountered, should be removed from soils prior to placement as compacted fill.

3.2.5 **<u>Shrinkage or Bulking</u>:** The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Based on our laboratory test results for the underlying soils at the site we estimate the following earth volume changes will occur during grading:

Shrinkage and Subsidence	
Shrinkage	Approximately 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.1 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.2.6 **<u>Pipeline Backfilling</u>:** Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable *Standard Specifications for Public Works Construction* (Greenbook), 2018 Edition standards. Backfill in and above the pipe zone should be as follows:

- **Pipe Zone**: Pipe bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). Imported clean/uniform sand with a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30 can also be used in the pipe zone. CLSM or uniform sand bedding should be placed to 1 foot over the top of the conduit, and vibrated. CLSM should not be jetted but sand should be flooded and jetted.

  We recommend that open-graded crushed rock or similar material not be used as bedding or shading material unless special provisions are implemented to limit the migration of surrounding soil into the open-graded rock. If gravel or open-graded rock is approved and used as bedding or shading, it should be wrapped in Mirafi 140N filter fabric, or equivalent.

- **Over Pipe Zone**: Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material greater than 3 inches in largest dimension. As an option, the whole trench can be backfilled with one-sack CLSM same as presented above for the pipe bedding zone. Oversized rock (cobbles and/or boulders) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone. Gravel larger than ¾ inch in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. Native soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90% relative compaction (relative to the laboratory modified Proctor maximum dry density), relative to the ASTM D1557 laboratory maximum dry density within structure footprint and hardscape areas, or 85% under landscape areas. Backfill above the pipe zone should not be flooded or jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested by Leighton.

3.3 Infiltration System Design

One infiltration test was performed to estimate the infiltration rate of onsite soils within the upper 5 feet of onsite soils. Soils encountered within the upper 5 feet generally consisted of dense silty sands (SM), and generally became more granular (sand with gravel, SP) below 5 feet. Based on our infiltration test results presented in Appendix A, raw infiltration rates were measured at approximately 0.1 inch per hour within LP-1. Infiltration within the near surface soils represented by our testing is not recommended and water should be diverted into approved drainage systems associated with the proposed Splash Pads.

Deeper soils consisting of clean sands with gravels encountered below 5 feet appear that they would provide favorable results, but confirmation testing at the depth and location of the proposed infiltration system chosen should be conducted.

3.4 Seismic Design Parameters

The site will experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2022 CBC:

Table 2. 2022 CBC Site-Specific Seismic Parameters

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude: 34.0737, -117.4134	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.596 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.600 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.700*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.596 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.020* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.064 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.680* g
Mapped *MCE*$_G$ peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.662 g
Site Coefficient for Mapped *MCE*$_G$ *PGA* (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.728 g

* See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters SM_1 and SD_1 in the table are increased by 50%.**

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 8.1 (Mw) at a distance on the order of 8.57 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

3.5 Foundations

Based on our preliminary exploration and our experience in the region, conventional shallow spread footings may be used to support the proposed improvements. Anticipated foundation loads were not available during preparation of this report. Overexcavation and recompaction of footing subgrade soils should be performed as detailed in Section 3.3 of this report. Specific spread footing recommendations are presented below:

3.5.1 **<u>Minimum Embedment and Width</u>:** Based on our preliminary exploration, footings for this proposed monument structure should have a minimum embedment of 18 inches below lowest adjacent exterior grade or interior finished grade; whichever is deeper/lower. Minimum footings widths should be at least 24 inches for isolated rectangular column footings or 12 inches for continuous bearing wall (strip) footings.

3.5.2 **<u>Allowable Bearing Capacity</u>:** A net allowable bearing capacity of 2,500 psf may be used for design of isolated rectangular footings. These values are based on the minimum embedment depth and width recommended in Section 3.6.1, above, and are governed by properly compacted fill settlement. These allowable bearing values may be increased by 250 psf per foot increase in embedment depth and/or width to a maximum allowable bearing pressure of 4,000 psf, and are for total dead load and sustained live loads, which can be increased by one-third when considering short-duration wind or seismic loads. Footing reinforcement should be designed by the project Structural Engineer.

3.5.3 **<u>Lateral Load Resistance</u>:** Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.40. The passive resistance may be computed using an equivalent fluid pressure of 270 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. These friction and passive values have already been reduced by a factor of safety of 1.5, and can be increased by one third when considering short-duration wind or seismic loads. For spread footings and slabs-on-grade bearing on properly compacted fill over undisturbed native soils, full friction and passive resistance can be combined to resist lateral loads; although some lateral displacement is required to mobilize full passive resistance.

3.5.4 **<u>Uplift Load Resistance</u>:**

If required to resist seismic uplift loads, properly compacted backfill soils over spread footings can be used, modeled with both dead weight and soil shear strength resisting short term dynamic uplift forces. Properly compacted backfill soils may be assumed to have a moist unit weight of 120 pcf. A friction angle of 33° can be used to model properly compacted backfill soil's shear strengths. A factor-of-safety has not been applied to these values.

3.5.5 **Settlement Estimates:** The above recommended allowable bearing capacity is generally based on a total allowable, post-construction total settlement of 1 inch. Differential settlement due to static loading is generally estimated at ½ inch over a horizontal distance of 30 feet. Once developed by the Structural Engineer, we can review total dead and sustained live loads for the proposed structure.

3.5.6 **Pole Foundations** Lateral bearing resistance for any proposed light pole foundations may be based on an allowable lateral earth pressure of Class of Material 4 on Table 1806.2 of the 2022 California Building Code (CBC), which can be doubled in accordance with 1806A.3.4, ignoring the upper 18 inches of soil in non-paved areas. This lateral bearing value assumes that the pole can tolerate at least a 0.5-inch deflection at the ground surface due to short term loading. Lateral bearing resistance should be computed in accordance with Section 1807.3.2.1 (unconstrained laterally) of the 2022 CBC. These recommendations assume that the foundations will be embedded against firm intact soil.

As an alternative, the following parameters may be used in lateral loading analysis of concrete caisson piles: effective unit weight of 120 pcf, friction angle of 32 degrees, and k value of 90 pci. These parameters are intended for analyses such as with the Ensoft LPILE program, which solves the beam on elastic foundation problem using independent nonlinear lateral springs, commonly referred to as p-y curves, to model the relationship between soil resistance and pile deflection. Additional parameters to be considered by the structural engineer for lateral pile analysis include head fixity, allowable deflection, and section bending stiffness assuming concrete cracking.

We recommend an allowable resistance in compression for these foundations consisting of 150 psf for allowable skin friction, ignoring the bottom one diameter, and an allowable end bearing of 2,500 psf (assuming a cleaned-out bottom). We recommend that the piles be at least 4 pile diameters long. These values are for isolated single piles.

3.6 Concrete Slab-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2022 CBC requirements. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Slab-on-grade subgrade soil should be moisture conditioned to or within 3% over optimum moisture content, to a minimum depth of 18 inches within structure footprints, and compacted to 90% of the modified Proctor (ASTM D1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.

- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 4 inches should be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 24 inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 175 pounds per square inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾ inch.

- **Slab-On-Grade Control Joints**: Slab-on-grade crack control joint locations and spacing should be designed by the project Structural Engineer (SE).

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

3.7 Sulfate Attack and Ferrous Corrosion Protection

3.7.1 Sulfate Exposure: Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. A potentially high sulfate content could also cause corrosion of reinforcing steel in concrete. Section 1904A of the 2022 California Building Code (CBC) defers to the American Concrete Institute's (ACI's) ACI 318-19 for concrete durability requirements. Table 19.3.1.1 of ACI 318-19 lists "*Exposure categories and classes*," including sulfate exposure as follows:

Table 3. Sulfate Concentration and Exposure

Soluble Sulfate in Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	ACI 318-19 Sulfate Class
0-150	0.00 - 0.10	S0 (negligible)
150-1,500	0.10 - 0.20	S1 (moderate*)
1,500-10,000	0.20 - 2.00	S2 (severe)
>10,000	>2.00	S3 (very severe)

*or seawater

3.7.2 <u>Ferrous Corrosivity</u>: Many factors can modify corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "*Effects of Soil Characteristics on Corrosion*" (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as follows:

Table 4. Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in modifying corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

3.7.3 **Corrosivity Test Results:** To evaluate corrosion potential of soils sampled from this site, we tested a bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Table 5. Results of Corrosivity Testing

Locations	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
LB-1	0 - 5	140	40	7.6	10,100

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- **Sulfate Exposure**: Based on our previous experience and Table 19.3.1.1 of ACI 318-19, in our opinion, sulfate exposure should be considered "negligible" with an Exposure Class S0 for native silty sands sampled at the site. Based on Table 19.3.2.1 of ACI 318-19, for this Exposure Category S0, there would be no restrictions on cement type ("cementitious material") nor water/cement ratio, and an f_c' (28-day compressive strength) of at least 2,500 pounds per square inch (psi) is required at a minimum for structural concrete.
- **Ferrous Corrosivity**: As shown above, minimum soil resistivity of 10,100 ohm-centimeters was measured in our laboratory test. Based on these, the soils may be considered "very mildly corrosive" to ferrous materials.

  Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials.

3.8 Pavement Section Design

Based on design procedures outlined in the 2017 Caltrans *Highway Design Manual* and an assumed design R-value of 40 for silty sand subgrade variations, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below:

Table 6. Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
4.5 (automobile parking)	3	4

Traffic Indices (TIs) used in our pavement design are considered reasonable values for typical parking lot areas, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving, will result in premature pavement failure. Traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel-load analysis or a traffic study.

Portland Cement Concrete (PCC) sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing will help reduce severity of cracking.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Excavations

Based on our field observations, caving of cohesionless and loose fill soils will likely be encountered in unshored trench excavations. To protect workers entering excavations, excavations should be performed in accordance with OSHA and Cal-OSHA requirements, and the current edition of the California Construction Safety Orders, see:

http://www.dir.ca.gov/title8/sb4a6.html

Contractors should be advised that sand and fill soils should initially be considered Type C soils as defined in the California Construction Safety Orders. As indicated in Table B-1 of Article 6, Section 1541.1, Appendix B, of the California Construction Safety Orders, excavations less-than (<) 20 feet deep within Type C soils should be sloped back no steeper than 1½:1 (horizontal:vertical), where workers are to enter the excavation. This may be impractical near adjacent existing utilities and structures; so shoring may be required depending on trench locations. Stiff undisturbed native clays will stand steeper.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and Leighton Consulting, Inc. should be maintained to facilitate construction while providing safe excavations.

4.2 Temporary Shoring

Temporary cantilever shoring can be designed based on the active equivalent fluid pressure of 30 pounds-per-cubic-foot (pcf) in alluvium. If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure expressed in pounds-per-square-foot (psf), may be assumed to be 20 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 15 feet in depth at this site.

4.3 Geotechnical Services During Construction

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading, foundation and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all excavation,
- During compaction of all fill materials,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

5.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please also refer to Appendix D, *GBA's Important Information About This Geotechnical-Engineering Report*, presenting additional information and limitations regarding geotechnical engineering studies and reports.

This report was prepared for the San Bernardino County Department of Public Works' Special Districts based on their needs, directions and requirements at the time of our exploration, in accordance with generally accepted geotechnical engineering practices at this time in San Bernardino County for public sites. This report is not authorized for use by, and is not to be relied upon by, any party except the San Bernardino County Department of Public Works' Special Districts, and their design and construction management team, with whom Leighton Consulting, Inc. has contracted for this work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc.

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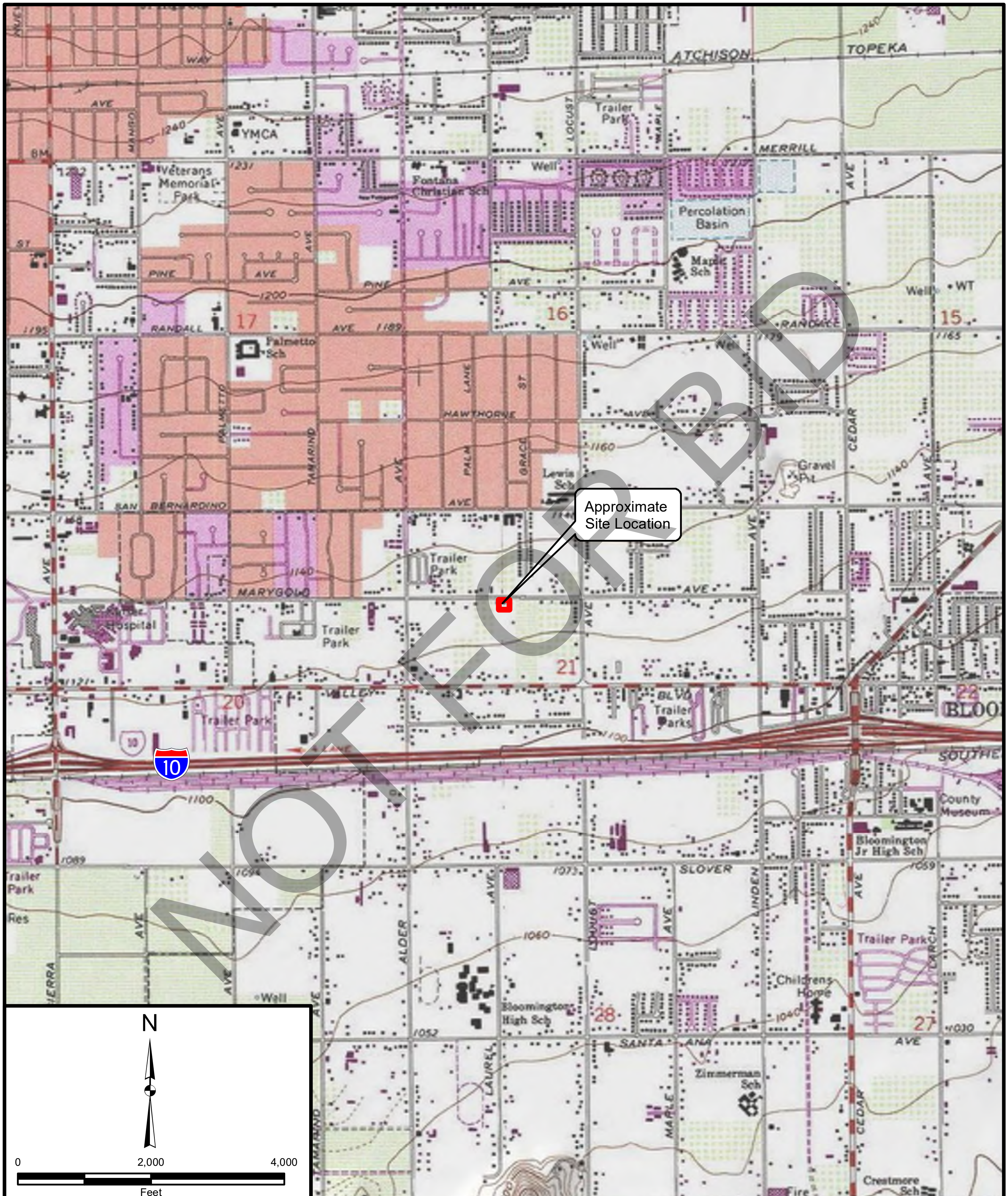


Project: 22346	Eng/Geol: JT
Scale: 1 " = 2,000 '	Date: May 2024

Reference: Copyright:© 2013 National Geographic Society, i-cubed

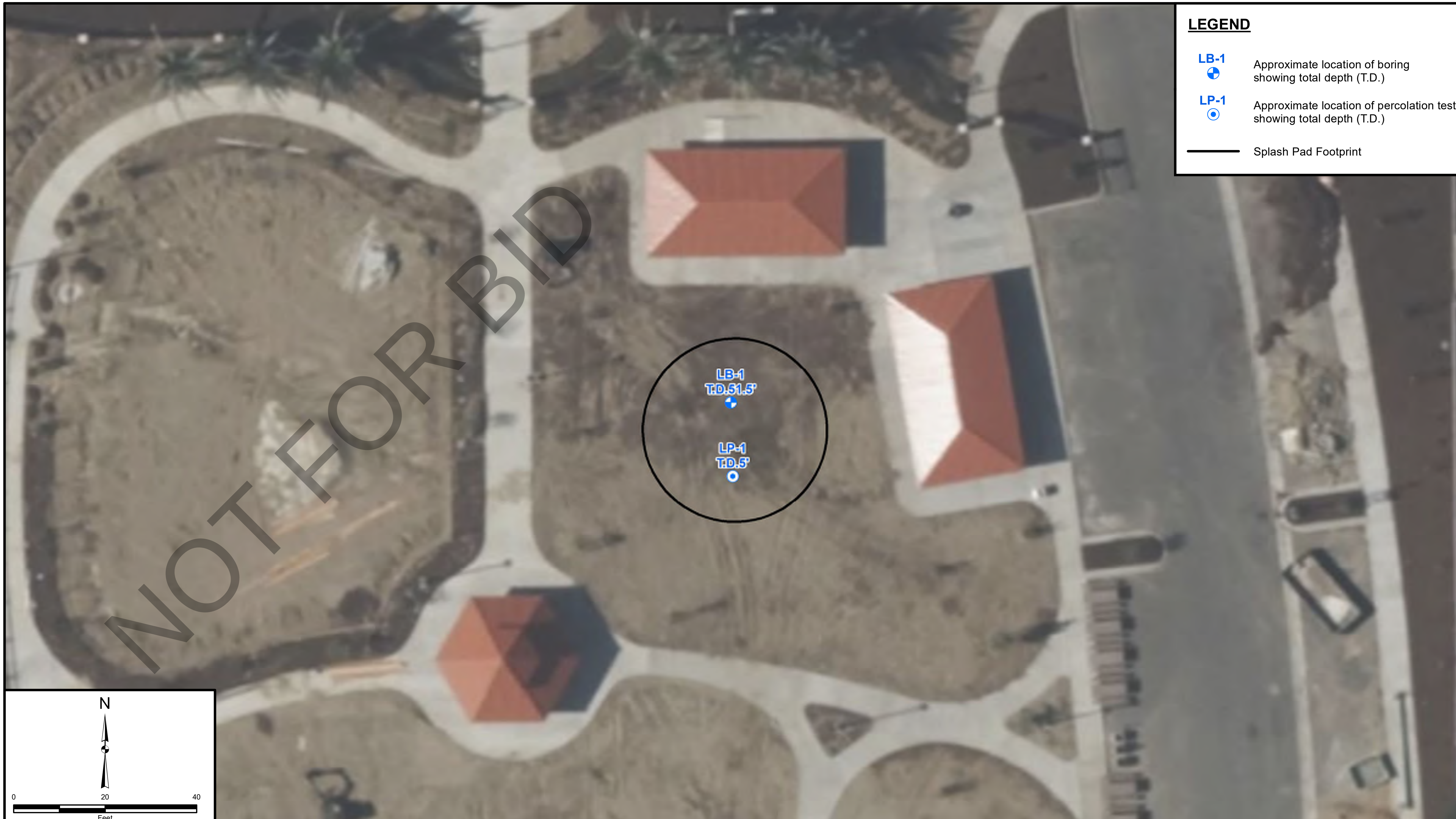
SITE LOCATION MAP

Ayala Park Splash Pad Project
San Bernardino County Department of Public Works - Special Districts
9984 Grace Street, Bloomington, California 92316

FIGURE 1

Leighton

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\22346 - Ayala Park Splash Pads Bloomington\GIS\Maps\22346_F01_SLM_2024-05-03.mxd on 5/1/2024 2:30:09 PM Author: KVM (btran)



Project: 22346	Eng/Geol: JT
Scale: 1 " = 20 '	Date: May 2024

Service Layer Credits: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus DS © 2024 TomTom

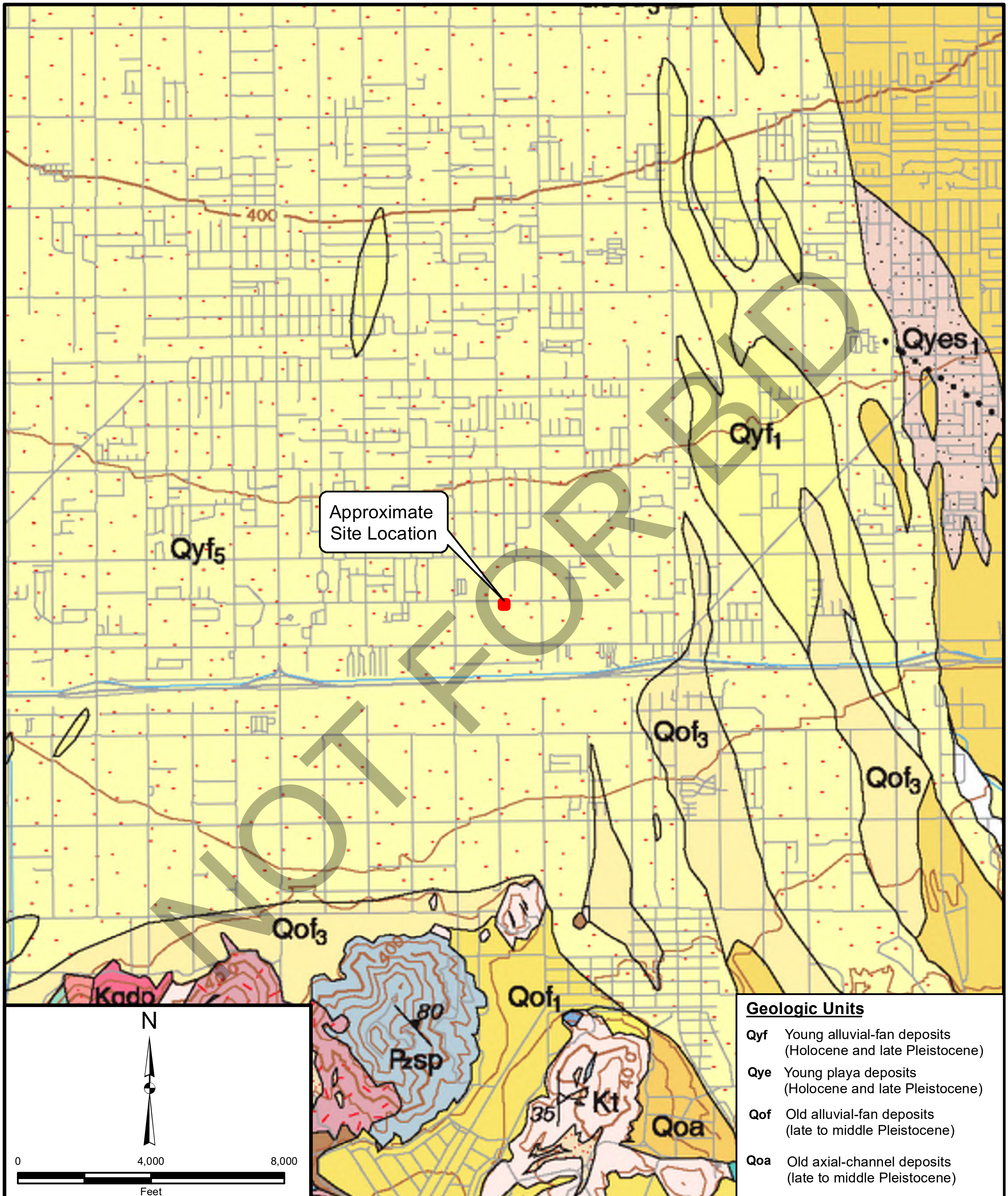
EXPLORATION LOCATION MAP

Ayala Park Splash Pad Project
San Bernardino County Department of Public Works - Special Districts
9984 Grace Street, Bloomington, California 92316

FIGURE 2

Leighton

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\22346 - Ayala Park Splash Pads Bloomington\GIS\Maps\22346_F02_ELM_2024-05-03.mxd on 5/1/2024 2:27:22 PM Author: KVM (btran)



Project: 22346	Eng/Geol: JT
Scale: 1 " = 4,000 '	Date: May 2024

Reference: Preliminary Geologic Map of The San Bernardino Quadrangle, California by Douglas Morton and Fred Miller, 2003

REGIONAL GEOLOGY MAP

Ayala Park Splash Pad Project
San Bernardino County Department of Public Works - Special Districts
9984 Grace Street, Bloomington, California 92316

FIGURE 3

Leighton

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\22346 - Ayala Park Splash Pads Bloomington\GIS\Maps\22346_F03_RGM_2024-05-03.mxd on 5/1/2024 3:25:26 PM Author: KVM (btran)

APPENDIX A

FIELD EXPLORATION

Our field exploration consisted of geologic reconnaissance and a subsurface exploration program consisting of two (2) geotechnical borings. These subsurface exploration locations are plotted on Figure 2, *Exploration Location Map*, and describe in more detail below:

Hollow Stem Auger Borings: On April 16, 2024, two (2) borings were drilled with a truck-mounted drill rig, logged and sampled to depths ranging from approximately 5 feet to 51.5 feet bgs. After sampling and logging, all borings were immediately backfilled with soil cuttings generated during drilling. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Near surface bulk soil samples were collected from these borings. Boring logs are included as part of this appendix.

Subsurface Variations and Limitations: These attached subsurface exploration logs and related information depict subsurface conditions only at the approximate locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. Passage of time may result in altered subsurface conditions due to possible environmental changes. In addition, any stratification lines depicted on these logs represent an approximate boundary between soil types, but these transitions can be gradual.

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000022346	**Date Drilled**	4-16-24
Project	Ayala Park Splash Pad Project	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1133'
Location	See Figure 2 Exploration Location Map	**Sampled By**	AA

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0		B-1				SM	**Undocumented Artificial Fill (afu):** @Surface: Grass over SILTY SAND (SM), dark brown, very moist, fine to coarse sand, trace rootlets, 22 percent fines (lab)	MD, SA, EI, CR
1130			R-1	14 26 33	119	13	SM	@2.5': Same as above, dense, very moist, trace rootlets, 22 percent fines (lab)	-200
								Young Alluvial Fan Deposits (Qyf):	
	5		R-2	22 40 48	126	4	SP	@5': Poorly-graded SAND with GRAVEL (SP), very dense, brown, moist, medium to coarse sand, 20 percent gravel, 5 percent fines (field estimate), auger grinding on cobbles	
1125			R-3	21 41 48		4	SP	@7.5': partial recovery, auger grinding on cobbles, same as above, dense, slightly moist, 25 percent gravel, 5 percent fines (field estimate)	
	10		R-4	16 13 20	126	5	SP	@10': Same as above, medium dense, moist, coarse sand, 15 percent gravel, 5 percent fines (field estimate)	
1120									
	15		S-1	14 9 6			SP	@15': Same as above, medium dense, coarse sand, 15 percent gravel, 5 percent fines (field estimate)	
1115									
	20		R-5	16 50/6	127	6	SP	@20': Same as above, very dense, moist, 20 percent gravel, 5 percent fines (field estimate)	
1110									
	25		S-2	15 17 27			SP-SM	@25': Poorly-graded SAND with SILT and GRAVEL (SP-SM), dense, grayish brown, slightly moist, medium to coarse sand, 15 percent gravel, 10 percent fines (field estimate)	
1105									
	30								

NOT FOR BID

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000022346	**Date Drilled**	4-16-24
Project	Ayala Park Splash Pad Project	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1133'
Location	See Figure 2 Exploration Location Map	**Sampled By**	AA

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-6	50/6			SP-SM	@30': Poorly-graded SAND with SILT and GRAVEL (SP-SM), very dense, slightly moist, 15 percent gravel, 10 percent fines (field estimate), trace cobbles	
1100										
	35			S-3	8 12 24			ML	@35': SANDY SILT (ML), hard, brown, slightly moist, fine sand, 75 percent fines (field estimate)	
								SP	@36': Poorly-graded SAND with GRAVEL (SP), dense, light brown, slightly moist, fine to coarse sand, 15 percent gravel, 5 percent fines (field estimate)	
1095										
	40			R-7	24 29 26			SP	@40': Same as above, no gravel, dense, fine to medium sand, 5 percent fines (field estimate)	
								ML	@41': SANDY SILT (ML), hard, brown, slightly moist, fine sand, very low toughness, 60 percent low plasticity fines (field estimate)	
1090										
	45			S-4	9 15 17			SP	@45': Poorly-graded SAND (SP), dense, light brown, slightly moist, 5 percent fines (field estimate)	
1085										
	50			R-8	22 34 50/5			SP	@50': Same as above, very dense, fine to coarse sand, trace gravel, 5 percent fines (field estimate)	
									TOTAL DEPTH = 51.5 FEET **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **BACKFILLED WITH SOIL CUTTINGS**	
1080										
	55									
1075										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

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*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LI-1

Project No.	038.0000022346	**Date Drilled**	4-16-24
Project	Ayala Park Splash Pad Project	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1132'
Location	See Figure 2 Exploration Location Map	**Sampled By**	AA

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							SM	**Undocumented Artificial Fill (afu):** @Surface: Grass over SILTY SAND (SM), dark brown, very moist, fine to coarse sand, trace roots, 35 percent fines (field estimate)	
1130				S-1	8 16 16			SM	@1': Same as above, dense, slightly moist, 21 percent fines (lab)	-200
				S-2	12 14 16			SM	**Young Alluvial Fan Deposits (Qyf):** @3.5': Same as above, medium dense, moist, 24 percent fines (lab)	-200
	5								**TOTAL DEPTH = 5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **CONVERTED TO INFILTRATION BORING** **BACKFILLED WITH SOIL CUTTINGS AFTER COMPLETION OF DRILLING**	
1125										
	10									
1120										
	15									
1115										
	20									
1110										
	25									
1105										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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Results of Falling Head Infiltration Test

Project: 22346
Exploration #/Location: *IT-1*
Depth Boring drilled, bgs (ft): 5
Tested by: AA
USCS Soil Type in test zone: SM
Weather (start to finish): Sunny
Water Source/pH: Tap
Measured boring diameter: 8 in. 4 in. Well Radius
Depth to GW or aquitard, bgs: 100 ft
Well Prep: 2" diameter slotted pipe. #3 Sand placed around slotted area

Initial estimated Depth to Water Surface (in.): 24
Average depth of water in well, "h" (in.): 34
approx. h/r: 8.4
Tu (Fig. 8) (ft): 98.0
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity: 0.3
Casing outer diameter, in.: 2.3
Casing inner diameter, in.: 2.1
Cross-sectional area, in.^2: 17.3

	ft	in.	Total (in.)
Depth to bottom of well measured from top of a	5. ft	0. in.	60
Casing stickup measured above top of auger (	0. ft	2. in.	2
Depth to top of sand from top of casing	0. ft	0. in.	

Depth of well bottom below top of casing (in): 62

Use of Barrels: No
Use of Flow Meter: No
Test Type: Falling Head

Field Data / **Calculations**

Date	Time	Depth to WL in Boring (measured from top of casing)		Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3)			Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:											from supply	from Δh	Total						
4/16/2024	9:50	ft	in.																	
4/16/24	10:00	1.23					10	12.8	47.2											
4/16/24	10:10	1.45				10	20	15.4	44.6	-2.64	46	0	46	46	5	274	1204	0.9	0.04	0.21
4/16/24	10:20	1.63				10	30	17.6	42.4	-2.16	44	0	37	37	4	224	1144	0.9	0.04	0.18
4/16/24	10:30	1.81				10	40	19.7	40.3	-2.16	41	0	37	37	4	224	1090	0.9	0.04	0.19
4/16/24	10:40	1.98				10	50	21.8	38.2	-2.04	39	0	35	35	4	212	1037	0.9	0.04	0.19
4/16/24	10:50	2.15				10	60	23.8	36.2	-2.04	37	0	35	35	4	212	986	0.9	0.05	0.20
4/16/24	11:00	2.29				10	70	25.5	34.5	-1.68	35	0	29	29	3	174	939	0.9	0.04	0.17
4/16/24	11:10	2.45				10	80	27.4	32.6	-1.92	34	0	33	33	3	199	894	0.9	0.05	0.21
4/16/24	11:20	2.78				10	90	31.4	28.6	-3.96	31	0	68	68	7	411	820	0.9	0.14	0.46
4/16/24	11:30	3.08				10	100	35.0	25.0	-3.6	27	0	62	62	6	374	725	0.9	0.15	0.48
4/16/24	11:40	3.17				10	110	36.0	24.0	-1.08	25	0	19	19	2	112	666	0.9	0.05	0.16
4/16/24	11:50	3.25				10	120	37.0	23.0	-0.96	23	0	17	17	2	100	640	0.9	0.04	0.14
4/16/24	12:00	3.33				10	130	38.0	22.0	-0.96	23	0	17	17	2	100	616	0.9	0.05	0.15
			concluded test low infiltration																	
																			Minimum Rate:	0.14
																			Raw Rate for design, prior to application of adjustment factors:	**0.15**

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying proposed improvements, and to aid in verifying soil classification.

Percent Passing No. 200 Sieve: Percent fines (silt and clay) passing the No. 200 U.S. Standard Sieve was determined for soil samples in accordance with ASTM D 1140 Standard Test Method. Samples were dried and passed through a No. 4 sieve, then a No. 200 sieve. Result of this grain size analysis, as percent by dry weight passing the No. 200 U.S. Standard Sieve, is tabulated in this appendix and entered on our test pit logs.

Particle Size (Sieve) Analysis: Particle size analysis of bulk soil samples by passing sieves was evaluated using the ASTM D 6913 Standard Test Method. Results of these analysis are presented on the *Particle-Size Distribution ASTM D 6913* sheets in this appendix.

Modified Proctor Compaction Curve: A laboratory modified Proctor compaction curve (ASTM D1557) was established for bulk soil-sample to evaluate the modified Proctor laboratory maximum dry density and optimum moisture content. Results of this test are presented on the following *Modified Proctor Compaction Test* sheet in this appendix.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested a bulk soil sample collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

Expansion Index (EI): An Expansion Index (EI) test was performed on a representative shallow bulk soil sample from this site, in general accordance with the ASTM D4829 Standard Test Method. Results of this test are presented on the following "*Expansion Index of Soils"* table.

APPENDIX C

SUMMARY OF SEISMIC ANALYSIS

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: Ayala park Splash Pads; Case 1; PGAm 0.662; design GW 287; No overex 0

Project No.: 38.0000022346

May 2024

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	298	287		0	1132	845
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)

General Parameters:

a_{max} = 0.66g
M_W = 8.1
MSF eq: 1
MSF = 0.82
Hammer Efficiency = 83
C_E = 1.38
C_B = 1
C_S for SPT? TRUE
Unlined, but room for liner
Rod Stickup (feet) = 3
Ring sample correction = 0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Leighton

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Ayala park Splash Pads; Case 1; PGAm 0.662; design GW 287; No overex 0

Project No.: 38.0000022346

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8		**22**	120	59	2	1	38.4	300	67.6	77.9	>Range	300	0.43	0.52	**NonLiq**	77.9	0.01		0.00	0.4
LB-1	3.8 to 6.3	5	2.5		5	120	88	2	1	57.2	600	100.9	100.9	>Range	600	0.43	0.52	**NonLiq**	100.9	0.01		0.00	0.4
LB-1	6.3 to 8.8	7.5	2.5		5	120	89	2	1	57.9	900	97.5	97.5	>Range	900	0.42	0.52	**NonLiq**	97.5	0.01		0.00	0.4
LB-1	8.8 to 12.5	10	3.8		5	120	33	2	1	21.5	1200	33.3	33.3	>Range	1200	0.42	0.51	**NonLiq**	33.3	0.15		0.07	0.4
LB-1	12.5 to 17.5	15	5.0		5	120	15	1	1.23	18.5	1800	23.4	23.4	0.264	1800	0.42	0.51	**NonLiq**	23.4	0.23		0.14	0.3
LB-1	17.5 to 22.5	20	5.0		5	120	100	2	1	65.0	2400	79.7	79.7	>Range	2400	0.41	0.50	**NonLiq**	79.7	0.02		0.01	0.2
LB-1	22.5 to 27.5	25	5.0		10	120	44	1	1.3	57.2	3000	62.7	64.9	>Range	3000	0.41	0.49	**NonLiq**	64.9	0.03		0.02	0.2
LB-1	27.5 to 32.5	30	5.0		10	120	100	2	1	65.0	3600	68.5	70.8	>Range	3600	0.40	0.49	**NonLiq**	70.8	0.02		0.01	0.2
LB-1	32.5 to 37.5	35	5.0		5	120	36	1	1.3	46.8	4200	45.6	45.6	>Range	4200	0.38	0.47	**NonLiq**	45.6	0.03		0.02	0.1
LB-1	37.5 to 42.5	40	5.0		60	120	55	2	1	35.8	4800	32.6	44.1	>Range	4800	0.37	0.44	**NonLiq**	44.1	0.04		0.02	0.1
LB-1	42.5 to 47.5	45	5.0		5	120	32	1	1.3	41.6	5400	35.8	35.8	>Range	5400	0.35	0.42	**NonLiq**	35.8	0.13		0.08	0.1
LB-1	47.5 to 52.0	50	4.5		5	120	94	2	1	61.1	6000	49.9	49.9	>Range	6000	0.33	0.40	**NonLiq**	49.9	0.03		0.02	0.0

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 34.0737, -117.4134

Date	5/7/2024, 9:45:30 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.596	MCE_R ground motion. (for 0.2 second period)
S_1	0.6	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.596	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.064	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.662	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.728	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.99	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.151	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.596	Factored deterministic acceleration value. (0.2 second)
S1RT	0.761	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.845	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.662	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.855	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.925	Mapped value of the risk coefficient at short periods
C_{R1}	0.9	Mapped value of the risk coefficient at a period of 1 s
C_V	1.419	Vertical coefficient

DISCLAIMER

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update…

Latitude

Decimal degrees

34.0737

Longitude

Decimal degrees, negative values for western longitudes

-117.4134

Site Class

259 m/s (Site class D)

Spectral Period

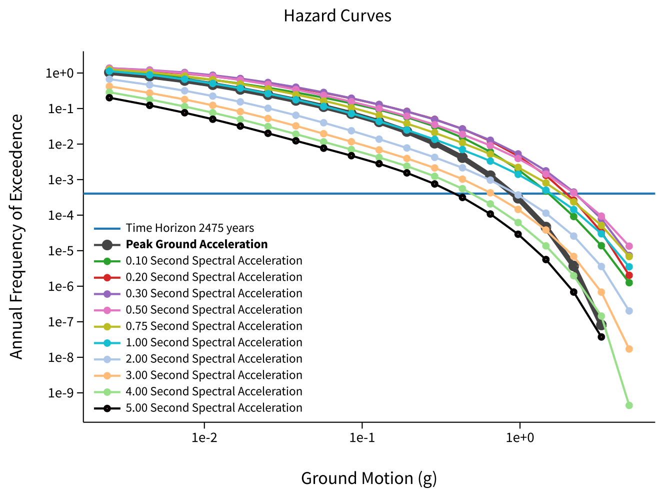
Peak Ground Acceleration

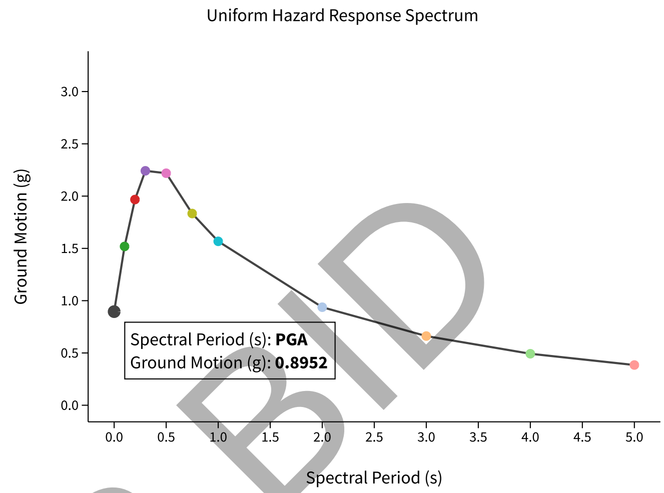
Time Horizon

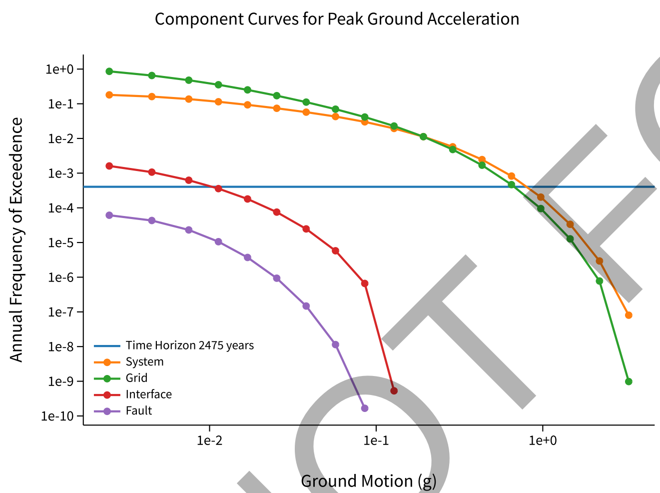
Return period in years

2475

Hazard Curve





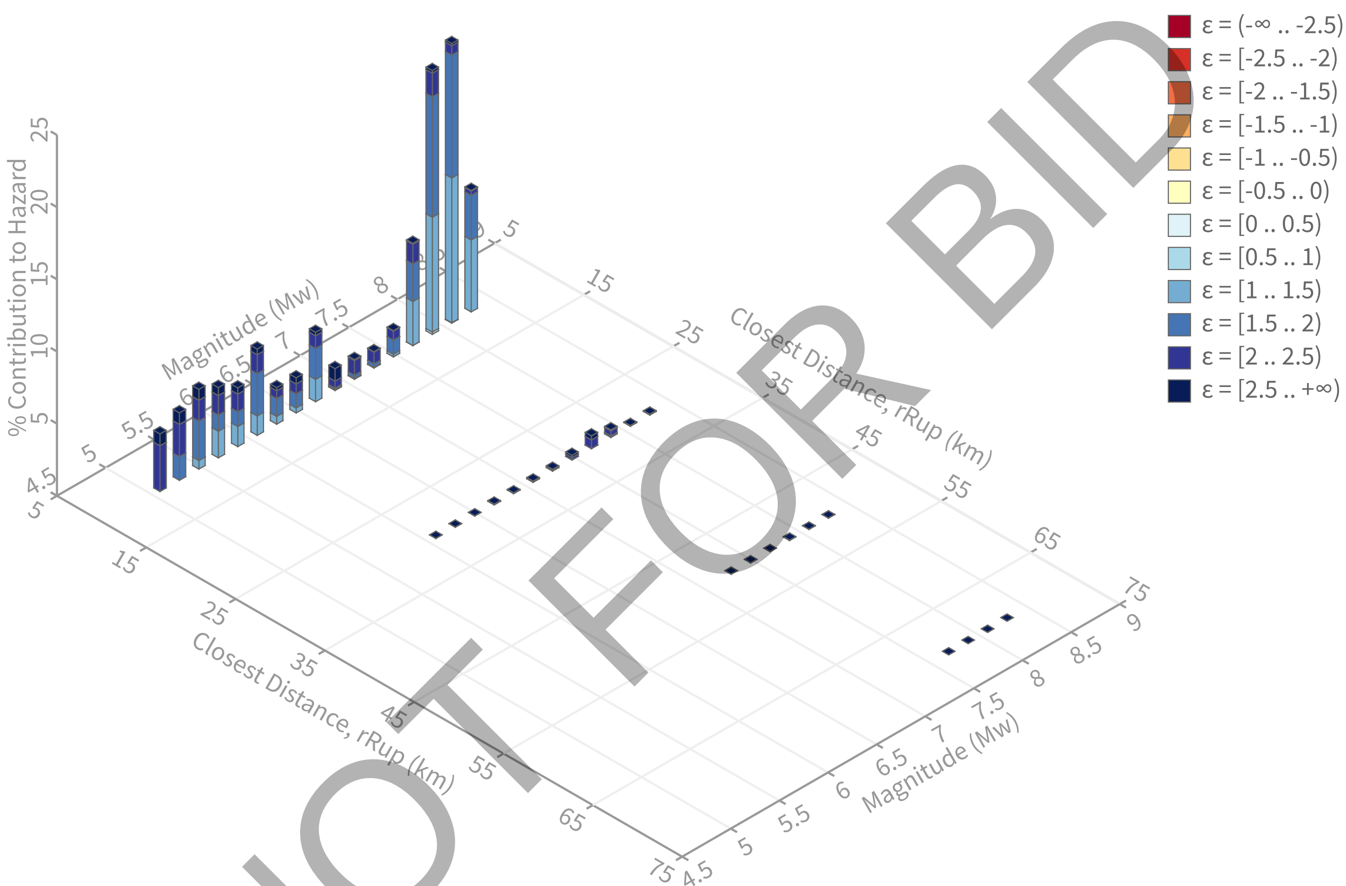


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.89522091 g

Recovered targets

Return period: 3232.5551 yrs
Exceedance rate: 0.00030935281 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 7.11
r: 10.54 km
ε_0: 1.75 σ

Mode (largest m-r bin)

m: 8.1
r: 10.93 km
ε_0: 1.5 σ
Contribution: 19.37 %

Mode (largest m-r-ε_0 bin)

m: 8.1
r: 8.57 km
ε_0: 1.27 σ
Contribution: 10.03 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							33.80
San Jacinto (San Bernardino) [3]		8.62	8.03	1.41	117.333°W	34.113°N	59.54	13.13
San Andreas (San Bernardino N) [4]		16.19	7.85	1.89	117.323°W	34.199°N	30.74	9.81
San Jacinto (Lytle Creek connector) [2]		7.71	7.99	1.34	117.365°W	34.129°N	35.86	3.83
Fontana (Seismicity) [0]		5.58	6.61	1.60	117.455°W	34.107°N	313.81	2.44
UC33brAvg_FM32	System							33.34
San Jacinto (San Bernardino) [3]		8.62	8.03	1.41	117.333°W	34.113°N	59.54	13.03
San Andreas (San Bernardino N) [4]		16.19	7.86	1.89	117.323°W	34.199°N	30.74	9.94
San Jacinto (Lytle Creek connector) [2]		7.71	7.99	1.34	117.365°W	34.129°N	35.86	3.85
Fontana (Seismicity) [0]		5.58	6.61	1.60	117.455°W	34.107°N	313.81	2.00
UC33brAvg_FM31 (opt)	Grid							16.44
PointSourceFinite: -117.413, 34.105		6.28	5.59	1.76	117.413°W	34.105°N	0.00	4.88
PointSourceFinite: -117.413, 34.105		6.28	5.59	1.76	117.413°W	34.105°N	0.00	4.88
PointSourceFinite: -117.413, 34.150		9.23	5.83	2.12	117.413°W	34.150°N	0.00	1.78
PointSourceFinite: -117.413, 34.150		9.23	5.83	2.12	117.413°W	34.150°N	0.00	1.78
UC33brAvg_FM32 (opt)	Grid							16.42
PointSourceFinite: -117.413, 34.105		6.28	5.59	1.76	117.413°W	34.105°N	0.00	4.88
PointSourceFinite: -117.413, 34.105		6.28	5.59	1.76	117.413°W	34.105°N	0.00	4.88
PointSourceFinite: -117.413, 34.150		9.24	5.83	2.12	117.413°W	34.150°N	0.00	1.78
PointSourceFinite: -117.413, 34.150		9.24	5.83	2.12	117.413°W	34.150°N	0.00	1.78

APPENDIX D

GBA'S IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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