SECTION H

GEOTECHINCAL
PART 1

GLEN HELEN LIGHTING PROJECT

FOR

SAN BERNARDINO, CALIFORNIA

PROJECT NO.: 30.30.0146

NOT FOR BID

GEOTECHNICAL EXPLORATION FOR THE PROPOSED GLEN HELEN LIGHTING PROJECT, GLEN HELEN REGIONAL PARK 2555 GLEN HELEN PARKWAY, SAN BERNARDINO CALIFORNIA

Prepared For **SAN BERNARDINO COUNTY- DEPARTMENT OF PUBLIC WORKS – SPECIAL DISTRICTS**
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Project No. 12108.009

August 11, 2023

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Leighton

Leighton Consulting, Inc.
A Leighton Group Company

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August 11, 2023

Project No. 12108.009

San Bernardino County
Department of Public Works – Special Districts
222 W. Hospitality Lane, Second Floor
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Attention: Mr. Charles Brammer
Senior Project Manager

Subject: Geotechnical Exploration for the Proposed Glen Helen Lighting Project, Glen Helen Regional Park
2555 Glen Helen Parkway, San Bernardino, California
Project No. 30.30.0146

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has conducted geotechnical exploration in support of the proposed Glen Helen Lighting Project (Project No. 30.30.0146) within the existing Glen Helen Regional Park located at 2555 Glen Helen Parkway in the City of San Bernardino, California. The purpose of our study has been to evaluate the geologic and geotechnical conditions within the area of and as they relate to the proposed improvements, and to provide geotechnical recommendations for foundation design and construction of proposed improvements.

This report presents our findings and conclusions regarding this project. Based upon our geotechnical investigation, the proposed improvements are feasible from a geotechnical viewpoint, provided our recommendations are incorporated into the design and construction of the project. The most significant geotechnical issues at the site are the potential for strong seismic shaking, potentially liquefiable soils, and groundwater within the upper 50 feet at select areas underlying the site. These and other geotechnical issues are discussed in this report.

We appreciate the opportunity to be of service to San Bernardino County. If you have any questions, or if we can be of further service, please call us at your convenience at (909) 484-2205.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Location

The proposed Glen Helen Lighting project is located at 2555 Glen Helen Parkway within the existing Glen Helen Regional Park in the City of San Bernardino, California (see Figure 1, *Site Location Map*). The project is generally located at the mouth or southeast end of Cajon Canyon. There are two areas where the lighting project is proposed to be installed. The first area (“Parking Lot” Area) consists of three relatively flat and undeveloped areas bound to the northwest by Forest Lane, to the southwest by the mountains on the south side of Cajon Canyon, to the southeast by the Interstate I-15 freeway, and to the northeast by railroad tracks. Glen Helen Road runs through the middle of the Parking Lot Area in a northwest-southeast direction. The second area (“Interior Park” Area) consists of several smaller parking lots within the main park area and select segments of the existing interior Glen Helen Park Road that combine to approximately 0.9 mile. The Interior Park Area is bounded to the southeast and southwest by the Mormon Battalion Mountain, to the northeast by Cajon Canyon, and to the northwest by Glen Helen Parkway (see Figure 2, *Geotechnical Map*).

Based on our review of historical aerial imagery dating from 1938 the Parking Lot Area has been undeveloped since at least 1938, while Glen Helen Regional Park (Interior Park Area) seems to have been developed sometime between 1980 and 1966 and brought to its current configuration sometime between 1985 and 1995. Prior to 1966, the area seems to have been utilized for agriculture, and prior to 1959, the area was undeveloped.

The Parking Lot Area is relatively flat with elevations ranging from 2,060 feet above mean sea level (msl) to 2,108 feet above msl, with a gentle slope to the south. The Interior Park Area contains hilly terrain with elevations ranging from 1,988 feet above msl to 2,045 feet above msl, sloping to the north.

1.2 Proposed Improvements

Based on review of the provided *Project Service Request #SD004 Questionnaire for the Glen Helen Lighting Project Geo Technical Services, Project # 30.30.0146,* dated June 2, 2023, we understand that the San Bernardino County Department of Public Works, Special Districts is proposing to install multiple 30-foot-tall light

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poles attached to concrete bases with solar light heads within parking lot areas and walkway areas at Glen Helen Regional Park. Based on the provided Lighting Maps, we understand there are two main areas where the lighting project is proposed to be installed, referred to as “Parking Lot” and “Park Interior”. The proposed improvement locations are depicted in Figure 2, *Geotechnical Map*.

Improvement Plans showing the proposed lighting systems and associated improvements were not available at the time of this study. However, based on experience with similar projects we anticipate the light poles will be founded on cast-in-drilled-hole (CIDH) piles, generally be 24- to 36-inch-diameter. We are unaware of any proposed ancillary structures as part of the proposed improvements.

The area of Glen Helen Regional Park is located within an Earthquake Fault Zone of Required Investigation for the San Jacinto Fault established by the California Geologic Survey (CGS 1995, 2023) in accordance with the Alquist Priolo Earthquake Fault Zoning Act (CGS, 2018). As currently planned, this project will consist of light standard installations, and no structures for human occupancy are proposed as part of this project. A structure for human occupancy is defined as “*any structure used or intended for supporting or sheltering any use or occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year*” (CGS, 2018). As a structure for human occupancy is not planned, the project does not fall under the requirements of the AP-act (CGS 2018, Plate 1).

1.3 Purpose of Investigation

The purpose of our study has been to evaluate geologic and geotechnical conditions, within the area of the proposed improvements, to explore subsurface conditions, and to provide recommendations for design and construction of the proposed improvements.

1.4 Scope

The scope of our geotechnical investigation has included the following tasks:

- **Research**: We reviewed pertinent, readily available geologic literature covering the site. Our review included published geologic maps and reports available and historical aerial photographs covering the site from our in-house library and from the public domain. Documents reviewed are listed in the attached *References*.

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- **Pre-field Investigation Activities**: Leighton contacted Dig Alert (811) a minimum 48 hours prior to drilling to locate and mark existing underground utilities prior to subsurface exploration. Leighton also contracted a private utility locator to scan each boring location for shallow buried private utilities prior to our subsurface investigation in an effort to identify any unmark utilities.
- **Field Exploration**: Our field exploration included eleven (11) hollow-stem auger borings, logging earth materials encountered, and collecting soil samples. On July 20, 21, and 25, 2023, we advanced hollow-stem auger borings (LB 1 through LB-5 and LB-7 through LB-12) at representative locations (see Figure 2, *Geotechnical Map*). The depths of these borings ranged from approximately 26½ to 51½ feet below the existing ground surface (bgs). Boring LB-6 was omitted due to presence of utilities conflicts within the boring vicinity.

  Encountered earth materials were logged in the field by our field representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using both a ring-lined Modified California split-barrel sampler and an unlined, 2-inch outside diameter Standard Penetration Test (SPT) split-spoon sampler. Sampling resistance blow counts were obtained by dropping a 140-pound, automatic-trip hammer through a 30-inch free fall onto a sampling rod anvil. Modified California and SPT samplers were driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). Representative bulk soil samples were also collected at shallow depths.

  Borings were backfilled with soil cuttings up to existing surfaces and capped with cold-patch asphalt to approximately match the surrounding ground surface within paved areas. Boring logs are presented in Appendix A, *Geotechnical Boring Logs*. The approximate boring locations are shown on the accompanying Figure 2, *Geotechnical Map*.

- **Laboratory Tests**: Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of sampled soils at this site, and to aid in evaluating soil classification.

  Tests are performed at our in-house geotechnical laboratory. Tests performed include:

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- In situ moisture and dry density
- Maximum dry density and optimum moisture content
- Grain Size Analyses
- Atterberg Limits
- Expansion Potential
- Direct Shear
- R-Value
- Soil corrosivity screening of resistivity, sulfate content, chloride content and pH

In-situ moisture and density of the collected samples are provided on the Geotechnical Boring Logs. Other laboratory test results are provided in Appendix B.

- **Engineering Analysis**: Data obtained from background review and field exploration was evaluated and analyzed to provide the geotechnical conclusions and recommendations presented in Section 3.0 of this report.

- **Report Preparation**: Results of our geotechnical exploration have been summarized in this report, presenting our findings, conclusions, and geotechnical foundation design recommendations.

The scope of work for this report does not include an evaluation of surface fault rupture hazards.

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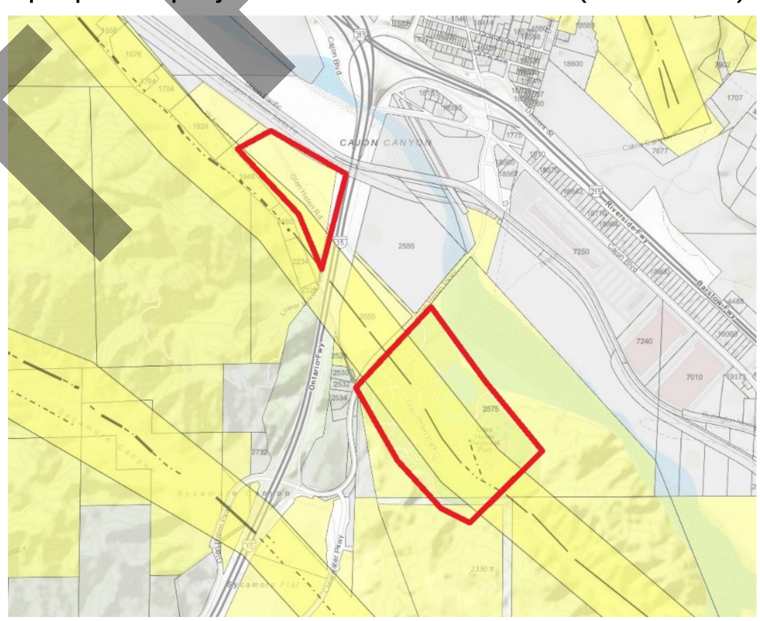
2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

This area is within the San Bernardino Basin in the northern portion of the Peninsular Ranges geomorphic province of California. Prominent mountain ranges surround this valley, including the San Gabriel Mountains on the northwest, San Bernardino Mountains on the north and east, the San Jacinto Mountains to the east, and the Temescal and Santa Ana Mountain ranges to the south.

Uplift of the San Bernardino and San Gabriel Mountain ranges are the result of the interaction between the North American and Pacific tectonic plates. This plate boundary is defined by the San Andreas transform system, which follows northwesterly along the foot of the San Bernardino Mountains near the project site.

The San Jacinto fault zone is a component of the San Andreas transform system. A section of the San Jacinto fault zone, which traces in a northwest/southeast direction, is mapped approximately 200 feet southwest of the Parking Lot Area and runs through the middle of the Interior Park Area. Figure 4, *Regional Fault and Historical Seismicity Map*, presents the site location in relation to active faults and epicenters of relatively large (> Mw 4.0) historical earthquakes. As noted, this fault has been zoned by CGS in accordance with the Alquist Priolo Earthquake Fault Zoning Act. A snippet showing the Alquist Priolo designated Fault Zone areas in relation to the proposed project sites is shown below (CGS 2023):



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The site is situated on alluvial wash deposits and young alluvial fan deposits. These deposits were formed from the transport and deposition of erosional materials from hills and mountains. These onsite deposits typically consist of silty sands, sands, silts and gravels. See Figure 3, *Regional Geology Map* for regional depiction of earth units at the surface in the project area.

2.2 Subsurface Soil Conditions

Based upon our subsurface exploration, the site is underlain by undocumented artificial fill (Afu) and alluvial wash deposit (Qw) or young alluvial fan deposits (Qyf).

- **Undocumented Artificial Fill (Afu):** Artificial fill onsite presumed to be associated with previous site grading is present across the approximate upper 1 to 7.5 feet within the explored locations. Artificial fill within the Parking Lot Area were generally encountered up to a depth of 0 to 2 feet and encountered within the upper 2.5 to 7.5 bgs at the Park Interior Area. Soils generally consisted of dry to slightly moist, dark brown and grayish, silty sands. Based on field sampling blow counts, artificial fill soils are very loose to medium dense. We are unaware of any documentation of previous fill engineering and placement for this site, so we have characterized all fill onsite as undocumented.

- **Young Alluvial Fan Deposits (Qyf):** Published regional geologic mapping has indicated that the portions of the project site are underlain by late Holocene young alluvial fan deposits consisting of moderately consolidated coarse sand and bouldery alluvial deposits. Young alluvial fan deposits encountered during our subsurface exploration consisted of loose to medium dense silty sands and silty clayey sands with some layers of trace gravel to depths reaching 51½ feet bgs with the material becoming generally denser with increasing depth.

- **Alluvial Wash Deposits (Qw):** Published regional geologic mapping has indicated that the portions of the project site are underlain by Alluvial Wash Deposits consisting of moderately unconsolidated coarse sands and gravel to boulder deposits. Alluvial Fan Deposits were found to underlie artificial fill within select borings within the Interior Park Area and within all borings conducted within the Parking Lot Area. Alluvial Wash Deposits extended to the maximum explored depth of 51½ feet bgs. These native deposits

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consisted of silty sands and poorly/well graded sands with interbedded gravel/cobble layers.

More detailed descriptions of the subsurface conditions are presented on the boring logs in Appendix A, *Geotechnical Boring Logs*.

2.2.1 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, cracking of building foundations and slabs-on-grade could result.

Based on the granular nature of near surface soils and recovered near surface samples collected during our exploration, near-surface onsite soils are anticipated to exhibit a “very low” expansion potential. Expansion Index testing yielded results in the very low range.

2.2.2 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) provisions, adopted by the 2022 CBC (CBC. 2022 and ACI, 2014).

Near-surface soil samples were tested during this exploration for soluble sulfate content. Based on our experience with similar soils within the area and laboratory testing results, the results of these tests indicated a sulfate content less than 0.02 percent by weight. As such, the near surface soils are expected to pose negligible potential for sulfate reaction with concrete (Exposure Class S0)

If the concrete is expected to be in contact with reclaimed water, Type V cement and a water/cement ratio of 0.45 should be used.

2.2.3 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil’s electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity between 1,000 and 2,000 ohm-cm is considered corrosive, and

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soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH. Based on results of the tested near surface soils, the onsite soils are considered to be severely corrosive to moderately corrosive to ferrous metals, based on minimum resistivity. Results of tested samples are presented below:

Boring No.	Depth (feet)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (Ω-cm)
LB-5	0-5	6.80	<150	10	980
LB-7	0-5	6.80	<150	10	3000
LB-8	0-5	7.20	150	40	4400
LB-9	0-5	6.80	<150	10	3100

2.3 Groundwater

Groundwater was encountered in four (4) of our borings excavated onsite within the Interior Park Area. Water depth at each boring is summarized in the following table:

Boring No.	Groundwater Depth (ft, bgs)	Ground Elevation (ft, above msl)	Groundwater Elevation (ft, above msl)
LB-1	25	1,992	1,967
LB-4	22	1,999	1,978
LB-5	29.5	2,042	2,012.5
LB-7	18.5	1,988	1,969.5

Groundwater was not encountered in any of our borings (LB-8 through LB-12) excavated within the Parking Lot Area to a maximum depth of 51½ feet bgs. Ground surface elevations are about 50 to 100 feet higher in the Parking Lot Area.

Recent groundwater data from the California Department of Water Resources (CDWR, 2023) indicated groundwater for well no. 342136N1174048W001 with a ground surface elevation of 2012.6 feet above mean sea level (msl), located

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approximately 2,000 feet northeast of the Interior Park Area, indicated the shallowest recorded groundwater to be at an elevation of 1977 feet above msl in April 19, 1995, based on measurements taken from January 1986 through October 1997. Based on the above, the historically high groundwater level is 11 feet below the lowest ground surface elevation onsite.

As mentioned above, the ground elevation within Parking Lot Area is approximately 50 to 100 feet higher than the ground elevation within the Interior Park Area, therefore, the historically high groundwater level is considered to be greater than 50 feet below existing ground surface within the Parking Lot Area.

The presence of groundwater should be taken into consideration during design and construction if light pole bases are proposed to extend to depths where groundwater was encountered within the Interior Park Area.

2.4 Liquefaction Potential

Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur there must be:

(1) loose, clean granular soils,
(2) shallow groundwater, **and**
(3) strong, long-duration ground shaking

The State of California has not evaluated the site for liquefaction hazards, but San Bernardino County has mapped the site to be within a zone of high liquefaction susceptibility (San Bernardino County, 2010). Historically high groundwater levels have been estimated to be as shallow as 11 feet bgs within the Interior Park Area and we have used this water elevation in our liquefaction analysis.

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Our analysis was based on the modified Seed Simplified Procedure as detailed by Youd et al. (2001) and Martin and Lew (1999), which compares the seismic demand on a soil layer (Cyclic Stress Ratio, or CSR) to the capacity of the soil to resist liquefaction (Cyclic Resistance Ratio, or CRR) (Youd et al., 2001). A minimum required factor of safety of 1.3 was used in our analysis, with factor of safety defined as CRR/CSR. As required, our analysis assumes that the design earthquake would occur while the groundwater is at its estimated historically highest level. In the SPT method, soil resistance to liquefaction is estimated based on several factors, including SPT sampling blow counts normalized and corrected for several factors including fines content, and overburden pressure. Soil plasticity and moisture content are also considered in an evaluation of liquefaction. Parameters utilized in our analysis include Standard Penetration Test (SPT) results from the borings, visual descriptions of soil samples retrieved, and geotechnical laboratory test results.

Based on our analysis, no subsurface layers are considered susceptible to liquefaction within the Parking Lot Area due to deep historical groundwater elevation and relatively dense nature of the onsite soils. Within The Interior Park Area, a potentially liquefiable layer was encountered at 20 feet bgs within Boring LB-2, at 10 to 15 feet bgs within Boring LB-3 and at 10 feet within Boring LB-4 when considering a design groundwater depth of 11 feet bgs. The liquefiable layer consists of loose to firm silty sands and sandy silts within the larger matrix of denser sands.

A key aspect of liquefaction is what effect it may have on the proposed improvements in terms of surface manifestations, and seismic settlement. These are addressed in the following sections. With this analysis, the potential for surface manifestations of liquefaction, such as bearing failures and sand boils, is low, based on Ishihara (1995), described below.

A summary of the liquefaction analysis is included in Appendix C.

2.5 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

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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 58 feet below ground surface were used in the analysis for The Parking Lot Area and 11 feet bgs used for Interior Park Area. Based on our analysis, a potential for approximately 1.1 inches of seismic settlement is estimated at the Parking Lot Area and 4.3 inches at the Interior Park Area. Results of our seismic settlement analysis is presented in Appendix C.

2.6 Bearing Failures/Surface Manifestation

We performed an analysis of the potential for bearing failures/structural damage due to liquefaction (surface manifestations) based on the work of Ishihara (1995) and as described in Martin and Lew (1999). This method is based on empirical data and considers the thickness of non-liquefiable soil below the ground surface and foundations, compared to the thickness of underlying liquefiable soils. Our analysis based on this method indicates that the potential for structural damage due to liquefaction is low, due to the depth of potentially liquefiable soils.

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3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

This site is located within a currently designated Alquist-Priolo Earthquake Fault Zone for the San Jacinto Fault. The fault has been mapped about 200 feet south west of the Parking Lot Area and through Glen Helen Regional Park. Our study did not include an evaluation of surface rupture along the fault. However, strong ground shaking has and will occur at this site. Historic groundwater levels have been estimated to be on the order of 11 feet below the surface within the Park Area and deeper than 50 feet within the Parking Lot Area based on available well data surrounding the immediate area and conducted geotechnical borings. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, liquefaction potential, and moderate seismic settlement. Surface rupture along the San Jacinto Fault through the Parking Lot Area and Glen Helen Regional Park is also possible. However, as the proposed Lighting Project does not include the construction of any structures intended for human occupancy, we expect that this project is not subject to the Alquist-Priolo Earthquake Fault Zoning Act.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soil and the anticipated response both during and after construction. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to evaluate the effect upon the recommendations presented herein. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the County of San Bernardino, and other governing agencies.

3.2 Earthwork and Grading

Grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix D, unless specifically revised or amended below or by future recommendations based on final development plans.

3.2.1 Site Preparation

Prior to construction, the areas of the proposed improvements should be cleared of vegetation, asphalt pavement, and debris, which should be

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disposed of offsite. Any underground obstructions onsite should be removed. Resulting cavities should be properly backfilled and compacted. In addition, any uncontrolled fill should be removed and replaced as compacted fill. Efforts should be made to locate any 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 as recommended in Sections 3.2.3 and 4.3.

3.2.2 Overexcavation and Recompaction

To reduce the potential for adverse total settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

For areas planned for asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), and areas to receive fill should be overexcavated to a minimum depth of 18 inches below existing grade or 12 inches below proposed subgrade, whichever is deeper. Deep foundations for light standards need may be constructed within undisturbed soils.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to at least 2 percent above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D1557 laboratory maximum density.

3.2.3 Fill Placement

The onsite soil is suitable for use as compacted structural fill, provided it is free of debris and oversized material (greater than 12 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be accepted by Leighton Consulting.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, if necessary, to a minimum of 2 percentage points above optimum, and compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. The upper 6 inches of subgrade soils in vehicle pavement areas should be compacted to a minimum 95 percent relative

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compaction, and aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.2.4 Import Fill Soil

If import soil is to be placed as fill, it should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.3 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.

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

2022 CBC Seismic Design Parameters

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC	
	Parking Lot Area	Interior Park Area
Site Latitude and Longitude:	34.2180, -117.4143	34.2071, -117.4069
Site Class Definition (1613A.2.2, ASCE 7-16 Ch 20)	C	C
Mapped Spectral Response Acceleration at 0.2s Period (1613A.2.1), S_s	2.403 g	2.440 g
Mapped Spectral Response Acceleration at 1s Period (1613A.2.1), S_1	0.975 g	0.978 g
Short Period Site Coefficient at 0.2s Period (T1613A.2.3(1)), F_a	*1.2*	*1.2*
Long Period Site Coefficient at 1s Period (T1613A.2.3(2)), F_v	*1.4*	*1.4*
Adjusted Spectral Response Acceleration at 0.2s Period (1613A.2.3), S_{MS}	2.884 g	2.928 g
Adjusted Spectral Response Acceleration at 1s Period (1613A.2.3), S_{M1}	1.365 g	1.369 g
Design Spectral Response Acceleration at 0.2s Period (1613A.2.4), S_{DS}	1.922 g	1.952 g
Design Spectral Response Acceleration at 1s Period (1613A.2.4), S_{D1}	0.910 g	0.913 g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), *PGA*	1.012 g	1.027 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), F_{PGA}	1.2	1.2
Peak Ground Acceleration, mod w/ site effects (1803A.5.12; 11.8.3.2), PGA_M	1.214 g	1.233 g

As an added check, PGA and hazard deaggregation were also estimated using the United States Geological Survey's (USGS) 2008 Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake for the Parking Lot Aera has a PGA of 1.41g with a magnitude of approximately 7.91 (M_W) at a distance on the order of 2.4934 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years). The Interior Park Area has a PGA of 1.38g with a magnitude of approximately 7.91 (M_W) at a distance on the order of 3.35 kilometers for the Maximum Considered Earthquake. Deaggregation results are included in Appendix C.

3.4 Foundations Design Recommendations

The proposed lighting structures may be supported by drilled cast-in-place reinforced concrete piers. Therefore, we present geotechnical design parameters for drilled cast-in-place concrete piers. Geotechnical parameters for pier design are presented in the following paragraphs.

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3.4.1 Downward Pier Capacity

Proposed light poles can be supported on drilled cast-in-place concrete pier foundations, if caving soils are controlled by temporary casing or other effective means that do not reduce or eliminate skin friction. Friction parameters presented in this section are based on the assumption that drilling mud will not be used to install piers. We recommend that piers penetrate at least 6 feet below grade. For the proposed light poles within the general area of Borings LB-2, LB-3, and LB-4, we recommend that piers penetrate at least 8 feet below existing grade due to encountered loose soils. Actual pier length should be per structural design.

An allowable skin friction of 250 pounds-per-square-foot (psf) may be used, with end bearing ignored. The top 18 inches of penetration should be discounted in non-paved area. End bearing should not be used due to the caving potential and difficulty of cleaning the bottom of the excavation. This allowable skin friction value may be increased by one-third for wind and seismic loading. Piers should have a minimum center-to-center spacing at least three pier diameters. A group action reduction in capacity will be required for more closely spaced piers.

3.4.2 Lateral Pier Capacity

Lateral bearing resistance for light standard pile foundations may be based on an allowable lateral earth pressure of Class of Material 4 on Table 1806A.2 of the 2022 CBC, which can be doubled in accordance with 1806A.3.4 and 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 1807A.3.2 of the 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 30 degrees, and k value of 95 pci. These parameters are intended for analysis such as with the Ensoft LPILE program, which solves the beam on elastic foundation problem using independent nonlinear lateral springs,

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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.

These values are for isolated single piles. A group action reduction in capacity would apply for closely spaced piles.

3.4.3 Construction Considerations

The drilling contractor should be experienced in and equipped to manage caving. Bottoms of proposed per shafts should be reasonably clean and free of loose soil before reinforcing streel is installed and concrete is placed. All pier installation should be observed by Leighton in accordance with section 1705A.8 of the 2022 CBC. Leighton should observe pier drilling and determine if piers are founded in suitable, undisturbed native materials and construction in accordance with the recommendations presented in this report. Piers should generally be constructed in accordance with Section 205-3.3.2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). Concrete should be tremmied or placed by a concrete pump pipe extending to the bottom of the frilled shaft, keeping the tremie or pump pipe below the surface of the concrete to avoid entrapment of water and/or loose soil in the concrete.

It is the contractor's responsibility to ensure stability and safety of drilling operations. Site safety is the contractor's responsibility.

3.5 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. Concrete should be designed in accordance with ACI 318-14, Section 4.2 (ACI, 2014), adopted by the 2022 CBC (Section 1904A.2).

Based on our laboratory testing, the onsite soil is considered moderately to severely corrosive to ferrous metals. Metallic utilities should be avoided, or corrosion protection of underground metallic utilities should be provided. Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density

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polyethylene (HDPE) or other non-ferrous pipe when possible. As an alternative, ferrous pipe can be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate the pipe from on-site soils. It is critical that coatings, tape and bags be properly protected during installation and trench backfill construction, such that they are not damaged. Corrosion information presented in this report should be provided to your underground utility contractors.

3.6 Pavement Design

We are unaware of any proposed pavement improvements associated with this project, if pavement improvements are planned later, the following recommendations should be considered. Based on the design procedures outlined in the 2017 Caltrans Highway Design Manual, and using an assumed design R-value of 50, flexible pavement sections may consist of the following for the traffic index indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer.

Table 1 – Flexible Pavement Design

Traffic Index	Asphaltic Concrete (AC) Thickness (Inches)	Class 2 Aggregate Base Thickness (inches)
5 or less (auto access)	3.0	4.0
7 (bus/truck access and fire lanes)	4.0	4.5

If asphalt pavement is to be constructed prior to construction, the full pavement thickness should be placed to support heavy construction traffic.

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,

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and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

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4.0 CONSTRUCTION CONSIDERATIONS

4.1 Trench Excavations

Based on our field observations, caving of cohesionless and loose fill soils will may 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 C, 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.

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 38 pounds-per-cubic-foot (pcf). 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 25 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 10 feet in depth at this site.

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4.3 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction* (SSPWC, "Greenbook"), 2018 Edition. Utility trenches may be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

(1) **Granular Bedding:** a uniform sand material with a Sand Equivalent (SE) greater-than-or-equal-to (≥) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer). The bedding/shading sand should be densified in-place by mechanical means, or in areas where the trench walls and bottom soil have a minimum sand equivalent of 15, the bedding sand may be jetted. Bedding sand should be placed in accordance with the Standard Specifications for Public Works Construction – Greenbook (Public Works Standard, Inc.), current edition.

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit and vibrated.

We recommend that open-graded crushed rock or similar material not be used as bedding material, unless special provisions are implemented to limit the migration of surrounding soil into the open-graded material, including surrounding the open-graded material with filter fabric (Mirafi 140N or equivalent), or mixing sand with the open-graded material. Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment.

Trench backfill over the pipe bedding zone may consist of native and clean fill soils. All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned to slightly above optimum, and mechanically compacted to at least 90 percent of the laboratory derived maximum density as determined by ASTM Test Method D 1557.

4.4 Excavation Characteristics

Based on the results of this exploration, it is anticipated that onsite soils can be excavated using conventional excavation equipment and that the excavated caisson shafts may exhibit localized side collapse especially in cohesionless or clean sand materials. If required, temporary casing should be used to support the open excavation during construction. Other construction requirements should comply with applicable provisions of Section 1810A of the CBC.

4.5 Limitations and Additional Geotechnical Services During Construction

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. 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. However, additional geotechnical study and analysis may be required based on final development plans. Leighton Consulting should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton Consulting during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton Consulting will provide geotechnical observation and testing during construction. Please refer to the GBA "Important Information about Your Geotechnical Engineering Report" presented at the end of this report.

This report was prepared for the sole use of San Bernardino County Department of Public Works – Special Districts for application to the design of the proposed project in accordance with generally accepted geotechnical engineering practices at this time in California.

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.

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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.

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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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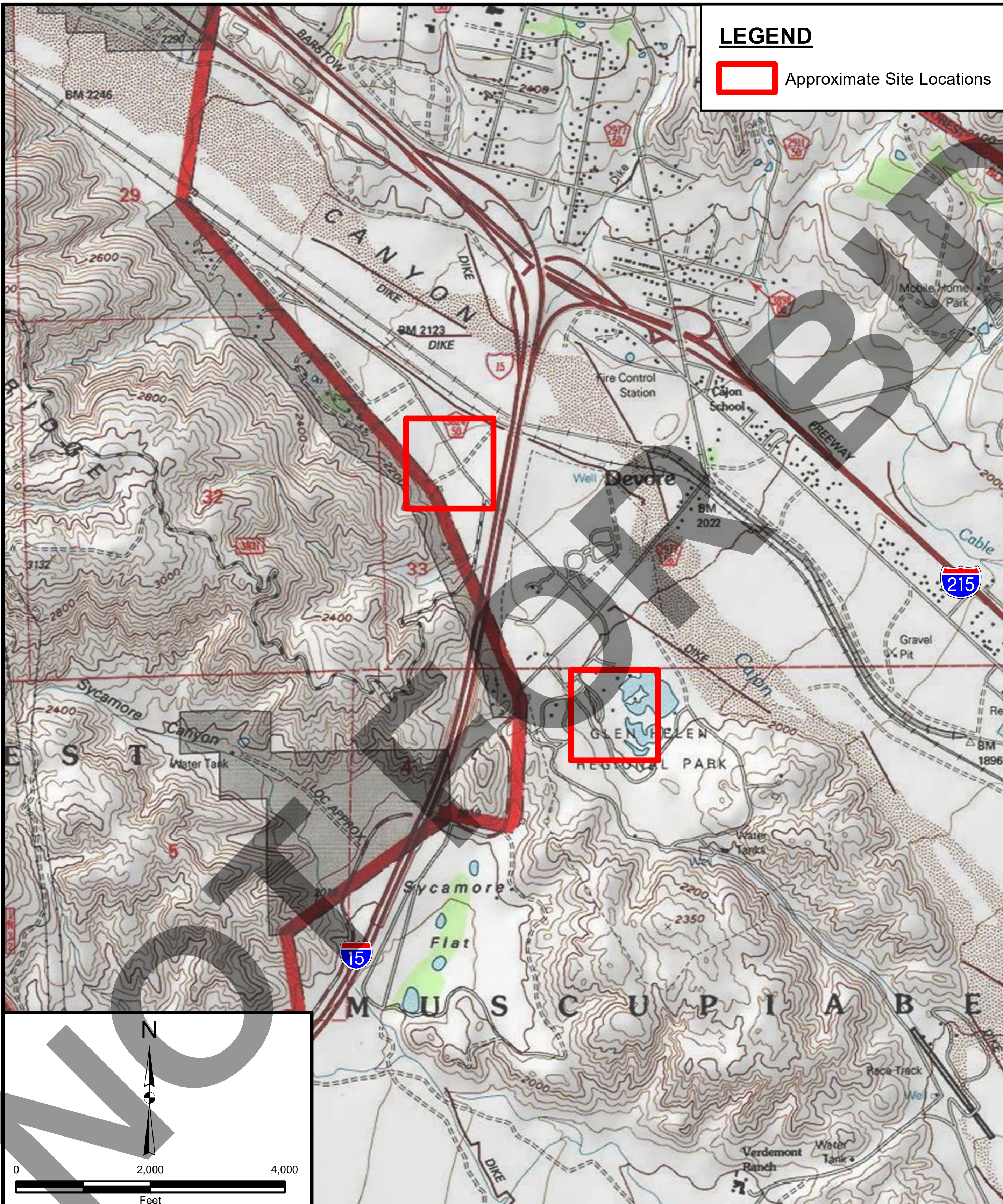

Project: 12108.009	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: August 2023

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SITE LOCATION MAP

Glen Helen Lighting Project
Glen Helen Regional Park
2555 Glen Helen Parkway
San Bernardino, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\12108\009\Maps\12108-009_F01_SLM_2023-07-27.mxd on 7/27/2023 8:54:09 AM Author: KVM (btran)