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DESIGN-PHASE GEOTECHNICAL INVESTIGATION PROPOSED GLAMPING PROJECT, APN 0629-181-01-0000 2107 OLD WOMAN SPRINGS ROAD, FLAMINGO HEIGHTS AREA SAN BERNARDINO COUNTY, CALIFORNIA

ROBOTT LAND COMPANY

January 11, 2021
J.N. 19-309

ENGINEERS + GEOLOGISTS + ENVIRONMENTAL SCIENTISTS

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ROBOTT LAND COMPANY
350 S. Reeves Drive, #100
Beverly Hills, California 90212

Attention: Mr. Larry Roth
Mr. Steve Botthof

Subject: Design-Phase Geotechnical Investigation, Proposed *Glamping Project*, APN 0629-181-01-0000, 2107 Old Woman Springs Road, Flamingo Heights Area, San Bernardino County, California

Dear Mr. Roth and Mr. Botthof:

Petra Geosciences, Inc. (Petra) is presenting herein our design-phase geotechnical evaluation report for the proposed Glamping project located in the unincorporated Flamingo Heights community in San Bernardino County. This report presents our findings and professional opinions with respect to the geotechnical feasibility of the proposed development, as well as a summary of geotechnical constraints that should be taken into consideration during the design and construction phases of the project. Recommendations for mitigation of geotechnical issues, and for the design and construction of the proposed development and appurtenances are provided as considered appropriate from a geotechnical engineering standpoint.

Please note that this geotechnical evaluation report does not address soil contamination or other environmental issues that may affect the property.

Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Alan Pace, CEG
Senior Associate Geologist

TABLE OF CONTENTS

Page

TABLE OF CONTENTS

DESIGN-PHASE GEOTECHNICAL INVESTIGATION
PROPOSED GLAMPING PROJECT, APN 0629-181-01-0000
2107 OLD WOMAN SPRINGS ROAD, FLAMINGO HEIGHTS AREA
SAN BERNARDINO COUNTY, CALIFORNIA

INTRODUCTION

Petra Geosciences, Inc. (Petra) is presenting herein the results of our geotechnical evaluation for the subject property. The main purpose of this investigation is to provide support for land planning activities by determining the nature of subsurface soil conditions and presenting general geotechnical design recommendations with respect to site clearing and grading and design and construction of new building foundations, pavement surfaces and other improvements.

This investigation included a review of published and unpublished literature and geotechnical maps and aerial photographs with respect to active and potentially active faults located on the site that may have an impact on the proposed construction.

A State of California active fault zone is present along the eastern boundary of the site adjacent to Old Woman Springs Road. The fault zone has been investigated by others and a building restriction zone for habitable structures (structures with greater than 2,000 person hours of occupancy) is in place. The report was reviewed and approved by the San Bernardino County Geologist. The building restriction zone will be incorporated into the project design. Petra has shown the approximate location of the restriction zone on Figure 1, Site Location Map.

OBJECTIVES AND SCOPE OF SERVICES

The objectives of this investigation were to identify and characterize geotechnical conditions that would impact site development, and to provide geotechnical recommendations for design and construction of the project as currently planned. To accomplish these objectives, our scope of services included the following:

1. Review of available published reports and maps pertaining to soil and geologic conditions in the area of the subject site, including a state-approved Fault Rupture Hazard Investigation (LandMark Geo-Engineers and Geologists, 2007).
2. Review of readily available regional geologic, fault and groundwater maps and reports, and aerial imagery (see references).
3. Review of exploration logs and laboratory test data produced by our firm during this investigation.
4. Performing engineering analyses in accordance with the current (2019) edition of the California Building Code (CBC).
5. Provide seismic design parameters for the proposed project in accordance with the 2019 CBC.

6. Preparation of this report presenting the results of our geotechnical evaluation and preliminary geotechnical recommendations for site grading and structural foundation design in conformance with state and local jurisdictional requirements.

SITE LOCATION AND DESCRIPTION

The subject site is located northeast of the intersection of La Brisa Drive and Old Woman Springs Road Avenue in the community of Flamingo Heights, San Bernardino County, California. Topographically the site slopes toward the north. Several unpaved roads/trails traverse the site. Overhead or buried utilities were not observed crossing the site. Most of the site is occupied by vegetation consisting of weeds, brush, junipers, and Joshua Trees. The subject site is shown on Figure 1.

The site lies to the west of Pipes Canyon Wash running north-northeast. The wash is approximately 15 to 50 feet below the subject site's elevation. Most of the adjacent properties are vacant with exception of several scattered single-family residences to the west and south of the site.

PROPOSED DEVELOPMENT

As of the date of this report, no rough grading plans have been provided. Based on review of a conceptual drawing, we anticipate that the project will consist of approximately 30 to 50 upscale camping sites, associated roads, landscaping, onsite septic, and stormwater retention facilities. Additionally, the conceptual plans show a restaurant, maintenance facilities, pool, and outdoor amphitheater with lawn seating for entertainment events.

CURRENT PRELIMINARY GEOTECHNICAL EVALUATION

Subsurface Exploration

A subsurface investigation was performed by our firm as part of this design-phase geotechnical evaluation. The field exploration was performed on August 5, 2020 and included the excavation of four geotechnical borings (identified herein as B-1 through B-4) and two percolation borings (identified herein as P-1 and P-2). Borings B-1 through B-4 ranged in depth from 24 to 58 feet. The two percolation borings were excavated to depths of approximately 10 and 5 feet, respectively. All borings were advanced using a truck-mounted hollow-stem auger. All borings were backfilled with cuttings from the excavation upon completion. The approximate locations of the borings are shown on the attached Geotechnical Map (Figure 2).

Associated with the subsurface exploration was the collection of bulk samples and relatively undisturbed samples for laboratory testing. Bulk samples consisted of selected materials obtained at various depth intervals from the borings. Relatively undisturbed samples were obtained from the borings using a 3-inch outside diameter (OD) modified California split-spoon soil sampler lined with brass rings. The soil sampler was mechanically driven to a depth of 18 inches with successive 30-inch drops of a 140-pound automatic trip hammer. The number of blows required to drive the sampler for each 6-inch increment are noted in the boring logs in Appendix A. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

Standard Penetration Tests (SPT) were also performed at selected depth intervals in accordance with ASTM D1586. This method consists of mechanically driving an unlined, 2.0-inch OD standard penetrometer sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts are also noted on the exploration logs. Disturbed (bulk) soil samples from the unlined standard penetrometer sampler were placed in sealed plastic bags and transported to our laboratory for testing.

Infiltration Test Results

Two infiltration test borings (P-1, P-2) were excavated to depths of 10 and 5 feet bgs, respectively, to assess an infiltration rate of the near-surface onsite soils for preliminary design of detention basins/underground chambers to manage storm water runoff. These tests used the Falling Head Test Method (RCFCD, 2011). Infiltration rates were then calculated using the Porchet Method (RCFCD, 2011), commonly called the "inversed auger-hole method." The test locations were situated within the northern portion of the subject property. The infiltration test was conducted in conjunction with the geotechnical borings, and the soils encountered at the test location consisted of fine to coarse silty sands. The test locations, Borings P-1 and P-2, are shown on Figure 2. The un-factored infiltration rate results are summarized below in Table 1, and are provided in Appendix E.

TABLE 1

Summary of Infiltration Rates

Percolation Test Location	Percolation Rate (gallons/day/feet)	Infiltration Rate (inches/hour)
P-1	310.8	45.3
P-2	100.3	14.7

Please note that no factor-of-safety has been applied to the reported percolation and infiltration rates. As such, these values should be reduced by a proper factor of safety at the discretion of the project civil engineer based on the site condition. Further, standard percolation/infiltration tests are performed using clean, potable water. However, surface runoff carries fines and debris with it, which may reduce the percolation/infiltration rates further.

Laboratory Testing

To assist in a preliminary evaluation of the engineering properties of the on-site earth materials, laboratory testing was performed on selected representative bulk and relatively undisturbed samples of soil materials obtained during the field evaluation. Laboratory testing included determination of the following:

- In-situ dry density and moisture content
- Maximum dry density and optimum moisture content
- Grain Size Analysis
- Expansion Index
- Remolded Direct Shear
- Hydro-collapse
- Chloride content
- Minimum resistivity

A description of laboratory test procedures and summaries of the laboratory test data are provided in Appendix B of this report. The results of the in-situ dry density and moisture content determinations are presented in the exploratory boring logs (Borings P-1, P-2, B-1 through B-4, Appendix A). An evaluation of the laboratory test data is reflected throughout the "Conclusions and Recommendations" section of this report.

FINDINGS

Geologic Setting

The property lies in the Mojave Desert Geomorphic province along the northern margin of the San Bernardino Mountains. The San Bernardino Mountains comprise a portion of the Transverse Ranges Geomorphic Province. The Transverse Ranges Geomorphic Province in the vicinity of the site is bounded on the north by the east-bending Pinto Mountain fault and the south by the Mission Creek branch of the San Andreas Fault. The Transverse Ranges are bounded to the east by the Coxcomb Mountains, to the west by the Pacific Ocean. The Transverse Ranges are characterized by east-west trending mountains and canyons.

Groundwater

Limited groundwater data is available in this region. The three nearest wells with published groundwater data (ranging from 0.8 to 1.7 miles distance from the site) show groundwater depths below ground surface (bgs) ranging from 190 to 350 feet during the period from 1990 to 2019 (Mojave Water Agency, 2020). Table 2 shows the selected well numbers, distance from project site, and minimum depth to groundwater.

TABLE 2

Depth to Groundwater

Well Number	Minimum Depth to GW (ft.)	Distance from Project (mi.)
01N05E02A01	388	0.8
02N05E27R01	192	1.1
02N05E36C01	270	1.7

Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific ground motion analysis. Alternatively, a design response spectrum can be developed for certain sites based on the code guidelines. To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used two computer applications. Specifically, the first computer application, which was jointly developed by Structural Engineering Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD), the SEA/OSHPD Seismic Design Maps Tool website, https://seismicmaps.org, is used to calculate the ground motion parameters. The second computer application, the United Stated Geological Survey (USGS) Unified Hazard Tool website, https://earthquake.usgs.gov/hazards/interactive/, is used to estimate the earthquake magnitude and the distance to surface projection of the fault.

To run the above computer applications, site latitude and longitude, seismic risk category and knowledge of site class are required. The site class definition depends on the direct measurement and the ASCE 7-16 recommended procedure for calculating average small-strain shear wave velocity, Vs30, within the upper 30 meters (approximately 100 feet) of site soils.

A seismic risk category of II was assigned to the proposed project in accordance with 2019 CBC, Table 1604.5. No shear wave velocity measurement was performed at the site, however, the subsurface materials at the site appears to exhibit the characteristics of stiff soils condition for Site Class D designation.

Therefore, an average shear wave velocity of 800 feet per second for the upper 100 feet was assigned to the site based on engineering judgment and geophysical experience. As such, in accordance with ASCE 7-16, Table 20.3-1, Site Class D (D- Default as per SEA/OSHPD software) has been assigned to the subject site.

The following table, Table 3, provides parameters required to construct the seismic response coefficient, C_s, curve based on ASCE 7-16, Article 12.8 guidelines. A printout of the computer output is attached in Appendix C.

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TABLE 3

Seismic Design Parameters

Ground Motion Parameters	Specific Reference	Parameter Value	Unit
Site Latitude (North)	-	34.215	°
Site Longitude (West)	-	-116.433	°
Site Class Definition	Section 1613.2.2 [1], Chapter 20 [2]	D-Default [4]	-
Assumed Seismic Risk Category	Table 1604.5 [1]	II	-
M_w - Earthquake Magnitude	USGS Unified Hazard Tool [3]	7.4 [3]	-
R – Distance to Surface Projection of Fault	USGS Unified Hazard Tool [3]	10.8 [3]	km
S_s - Mapped Spectral Response Acceleration Short Period (0.2 second)	Figure 1613.2.1(1) [1]	1.95 [4]	g
S_1 - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(2) [1]	0.675 [4]	g
F_a – Short Period (0.2 second) Site Coefficient	Table 1613.2.3(1) [1]	1.2 [4]	-
F_v – Long Period (1.0 second) Site Coefficient	Table 1613.2.3(2) [1]	Null [4]	-
S_{MS} – MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (0.2 second)	Equation 16-36 [1]	2.34 [4]	g
S_{M1} - MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (1.0 second)	Equation 16-37 [1]	Null [4]	g
S_{DS} - Design Spectral Response Acceleration at 0.2-s	Equation 16-38 [1]	1.56 [4]	g
S_{D1} - Design Spectral Response Acceleration at 1-s	Equation 16-39 [1]	Null [4]	g
$T_o = 0.2\ S_{D1}/\ S_{DS}$	Section 11.4.6 [2]	Null	s
$T_s = S_{D1}/\ S_{DS}$	Section 11.4.6 [2]	Null	s
T_L - Long Period Transition Period	Figure 22-14 [2]	8 [4]	s
PGA - Peak Ground Acceleration at MCE_G [*]	Figure 22-9 [2]	0.82	g
F_{PGA} - Site Coefficient Adjusted for Site Class Effect [2]	Table 11.8-1 [2]	1.2 [4]	-
PGA_M –Peak Ground Acceleration [2] Adjusted for Site Class Effect	Equation 11.8-1 [2]	0.986 [4]	g
Design PGA ≈ (⅔ PGA_M) - Slope Stability [†]	Similar to Eqs. 16-38 & 16-39 [2]	0.66	g
Design PGA ≈ (0.4 S_{DS}) – Short Retaining Walls [‡]	Equation 11.4-5 [2]	0.62	g
C_{RS} - Short Period Risk Coefficient	Figure 22-18A [2]	0.924 [4]	-
C_{R1} - Long Period Risk Coefficient	Figure 22-19A [2]	0.911 [4]	-
SDC - Seismic Design Category [§]	Section 1613.2.5 [1]	Null [4]	-

References:

[1] California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume I and II.

[2] American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standards 7-16.

[3] USGS Unified Hazard Tool - https://earthquake.usgs.gov/hazards/interactive/

[4] SEI/OSHPD Seismic Design Map Application – https://seismicmaps.org

Related References:

Federal Emergency Management Agency (FEMA), 2015, NEHERP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-1050).

Notes:

* PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).

† PGA Calculated at the Design Level of ⅔ of MCE; approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).

‡ PGA Calculated for short, stubby retaining walls with an infinitesimal (zero) fundamental period.

§ The designation provided herein may be superseded by the structural engineer in accordance with Section 1613.2.5.1, if applicable.

<u>Discussion - General</u>

Owing to the characteristics of the subsurface soils, as defined by Site Class D-Default designation, and proximity of the site to the sources of major ground shaking, the site is expected to experience strong ground shaking during its anticipated life span. Under these circumstances, where the code-specified design response spectrum may not adequately characterize site response, the 2019 CBC typically requires a site-specific seismic response analysis to be performed. This requirement is signified/identified by the "null" values that are output using SEA/OSHPD software in determination of short period, but mostly, in determination of long period seismic parameters, see Table 3.

For conditions where a "null" value is reported for the site, a variety of design approaches are permitted by 2019 CBC and ASCE 7-16 in lieu of a site-specific seismic hazard analysis. For any specific site, these alternative design approaches, which include Equivalent Lateral Force (ELF) procedure, Modal Response Spectrum Analysis (MRSA) procedure, Linear Response History Analysis (LRHA) procedure and Simplified Design procedure, among other methods, are expected to provide results that may or may not be more economical than those that are obtained if a site-specific seismic hazards analysis is performed. These design approaches and their limitations should be evaluated by the project structural engineer.

<u>Discussion – Seismic Design Category</u>

Please note that the Seismic Design Category, SDC, is also designated as "null" in Table 3. For condition where the mapped spectral response acceleration parameter at 1 – second period, S_1, is less than 0.75, the 2019 CBC, Section 1613.2.5.1 allows that seismic design category to be determined from Table 1613.2.5(1) alone provided that all 4 requirements concerning <u>fundamental period of structure</u>, <u>story drift</u>, <u>seismic response coefficient</u>, and <u>relative rigidity of the diaphragms</u> are met. Our interpretation of ASCE 7-16 is that for conditions where one or more of these 4 conditions are not met, seismic design category should be assigned based on: 1) 2019 CBC, Table 1613.2.5(1), 2) structure's risk category and 3) the value of S_{DS}, at the discretion of the project structural engineer.

<u>Discussion – Equivalent Lateral Force Method</u>

Should the Equivalent Lateral Force (ELF) method be used for seismic design of structural elements, the value of Constant Velocity Domain Transition Period, T_s, is estimated to be 0.5 seconds and the value of Long Period Transition Period, T_L, is provided in Table 3 for construction of Seismic Response Coefficient – Period (C_s -T) curve that is used in the ELF procedure.

As stated herein, the subject site is considered to be within a Site Class D-Default. A site-specific ground motion hazard analysis is not required for structures on Site Class D-Default with $S_1 \geq 0.2$ provided that the Seismic Response Coefficient, C_s, is determined in accordance with ASCE 7-16, Article 12.8 and structural design is performed in accordance with Equivalent Lateral Force (ELF) procedure.

SITE-SPECIFIC HAZARD ANALYSIS

Liquefaction Hazards Analysis

General Procedure

In April 1991, the State of California enacted the Seismic Hazards Mapping Act (Public Resources Code, Division 2, Chapters 7-8). The purpose of the Act is to protect the public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure. The Act defines mitigation as "… *those measures that are consistent with established practice and reduce seismic risk to acceptable levels.*" Acceptable level of risk is defined as "*that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project [California Code of Regulations; Section 3721 (a)]*." In the context of that Act, mitigation of the potential liquefaction hazards at this site to appropriate levels of risk can be accomplished through appropriate foundation and/or subsurface improvement design.

Based on site exploration and available historic water table elevation data, this site is not considered susceptible to seismically-induced liquefaction. Local groundwater depth records from 190 feet to 350 feet bgs and lower are available from 1990 to 2019, no known sources bodies of perched water at higher elevation, and little to no likelihood of increasing to within 50 feet bgs of the project elevation, this site can be considered non-liquefiable based on the screening criteria presented in SP 117. Seismically induced settlement of the dry sandy soils above the ground water is possible. This is due primarily to the documented presence of unconsolidated granular (sandy) soils in the area and the proximity of seismic sources.

Historical High Groundwater Level

As noted previously herein, groundwater was observed in the nearest nearby wells at depths ranging from 190 to 350 feet bgs and deeper from 1990 to 2019 (Mojave Water Agency, 2020). These wells are located between 0.8 and 1.7 miles from the subject site. This groundwater depth range is consistent with the general geology of the local region. No regions of perched water are known to exist in the vicinity of the subject site, and the likelihood of an increase in the elevation of the water table to within 50 vertical feet of the proposed structures' foundation systems is very remote at best.

Secondary Seismic Hazards

Seismically Induced Flooding

The types of seismically induced flooding that may be considered as potential hazards to a particular site normally includes flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site lies a considerable distance inland from the Pacific Ocean, and since it does not lie in close proximity to an enclosed body of water, the probability of flooding from a tsunami or seiche is considered to be low.

Seismically Induced Dry Sand Settlement

Due to the 200-foot plus depth to groundwater and the observed soil conditions, the site was evaluated for dry sand settlement under the Maximum Considered Earthquake. Blow counts from the exploration were converted to equivalent SPT blow counts and the data input into LiquefyPro (CivilTech, 2012). Detailed calculation output is provided in Appendix C. The results of our analysis indicate that dry sand settlement on the order of 2 to 3 inches could occur across the site. Differential settlement across the site can be assumed at 1 to 2 inches. Based on the concept plan, Petra estimates total settlement to the proposed structures from seismic dry sand settlement to be on the order of 1 to 2 inches and differential settlement to be on the order of ½ to 1 inch over a 100-foot span.

Geotechnical Issues Not Related to Seismicity

Expansive Soils

The onsite surficial soil materials have been visually classified as alluvial deposits consisting of granular and non-plastic soils. They consist primarily of sands (SP, SW) and silty sands (SM) to the depths explored in the borings. No expansive soils were observed. A tested sample, considered representative of the site's soils, was non-expansive (EI = 0). Imported soil, if utilized, should be limited to non-expansive soil. Specifications for import soils are discussed in a subsequent section of this report.

Collapsible Soils

Soils subject to collapse typically exhibit a high strength when dry; however, when moisture is introduced, the grain structure is rearranged resulting in a relatively rapid volume reduction (collapse). The collapse phenomenon is relatively common in arid environments such as the Flamingo Heights Area. Collapsible soils generally result from rapid deposition close to the source of the sediment such as debris flows, but can occur in wind-blown sands also. When saturated, the grain structure of these soils condenses or collapses resulting in subsidence and settlement under relatively low loads. A rise in the groundwater table or an increase in surface-water infiltration, with or without the weight of structures can initiate settlement and cause the foundations and walls of constructed facilities to crack.

Our initial test results showed excessive collapse potential at around 4 percent. Subsequent additional tests on five samples showed collapse potential ranging from 0.2 to 1.9 percent. Hydro-collapse settlement ranging from 1 to 2 percent across the site can be anticipated in any areas not receiving remedial, engineered grading. Structural sites that have been graded in accordance with the recommendations contained herein are likely to see less than 1 percent (roughly ½ inch) of total hydro-collapse settlement with differential settlements around ¼ inch over 40 feet. Based on our understanding of the proposed accommodations, hydro-collapse settlement will not affect the serviceability of these free-standing structures.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General Feasibility

From a soils engineering and engineering geologic standpoint, the subject property is considered suitable for the proposed development provided the following recommendations are incorporated into the design criteria and project specifications. In addition, the proposed grading and construction are not expected to affect the stability of adjoining properties in an adverse manner provided grading and construction are performed in accordance with current standards of practice, all applicable grading ordinances and the recommendations presented in this report.

Grading Plan Review

This report has been prepared without reference to a finalized grading plan, a foundation plan, or specifications concerning the proposed grading and construction. We have based our recommendations on a plot plan (with annotations) prepared by Fomotor Engineering (2020) supplied by the client. **As such, the recommendations provided in this report should be considered tentative until grading and foundation plans are finalized and reviewed by our firm.** Additional recommendations and/or modification of the recommendations provided herein may be necessary depending upon the results of our grading and foundation plan review.

Primary Geotechnical Constraints

The following are geotechnical issues which, based on the results of our review of previous reports, site reconnaissance, subsurface evaluation, laboratory testing and engineering analysis, are considered to have the potential to affect site development:

- Site Clearing: Prior to commencement of remedial grading within the site, any existing stockpiled soil, landscape cuttings, household wastes and other debris should be hauled offsite. No existing improvements were observed on the site. Any unknown remnant underground structures discovered during grading, such as building foundations, utility pipelines, existing onsite septic tanks and seepage pits, leach lines or other structures found below existing grade, should be removed in their entirety within the project limits and disposed of offsite. In the event buried construction materials or miscellaneous debris is encountered during grading operations, hand labor may be needed.

- Remedial Grading Requirements: Based on the conditions noted in our borings, the near-surface native alluvial materials at the site are subject to seismically-induced dry sand settlement. In an effort to limit the potential total and differential settlement to construction tolerances, it is recommended that the near-surface soils within the site where construction of permanent structures is planned be over-excavated to a minimum depth of at least 5 feet below the finished grades. The excavated material should subsequently be replaced as engineered compacted fill as required to establish the planned finished pad grades for the permanent structures. Additional grading recommendations are provided in the Earthwork Guidelines section below.

- Subsurface Obstructions: Any abandoned subsurface obstructions encountered during remedial grading should be removed entirely or, if appropriate, properly abandoned in place. Any cavities resulting from obstruction removal should be backfilled as described in the Earthwork Guidelines section of this report.

- Boundary Conditions: Since the site is primarily undeveloped and large scale engineered grading is not anticipated for this project, it is unlikely that structures requiring in-place protection will be a concern during grading. Nevertheless, in the course of any remedial grading, should it be necessary to avoid disturbance to sensitive improvements (masonry walls, fencing, roadway right-of-way, utility installations, etc.), maintain three feet or the depth of the overexcavation, whichever is deeper, horizontal distance minimum between the improvement and the grading. Temporary excavation backcuts subjacent to sensitive structures should be maintained at a gradient of 1.5:1 (horizontal to vertical) or flatter until competent ground is exposed.

- Suitability of Onsite Soils for Use in Engineered Fills: Onsite soils are considered suitable for use in engineered fills provided they are free of organics, demolition debris or other deleterious materials. Hand labor may be required to remove organic material (especially roots) or accumulated debris during grading operations.

- Expansion and Corrosion Potential of Site Soils: Given the granular nature of the near-surface soil they are expected to be non-expansive (Expansion Index ≤ 20); a representative sample test had an EI = 0.

  Lab data provided herein indicate that site soils are slightly alkaline. Corrosion is considered **Not Applicable** to concrete (**Exposure Class S0).** Corrosion is considered **Moderate** to metal encased in concrete (**Exposure Class C1**). Soils are considered mildly corrosive to buried metallic building materials due to resistivity and pH. See General Corrosivity Section following for a specific discussion of corrosion and special conditions that may affect the ratings provided in this section.

In the event that imported soil material is required to establish the planned finished grade elevations, potential import sources should be evaluated by the project geotechnical consultant prior to importing to the site to verify that only non-expansive and non-corrosive soil materials are used. A final assessment of soil expansion potential and corrosivity should be performed at the completion of grading.

- Adjustment of Earthwork Quantities: Based on the data collected at the site a shrinkage factor of 9 to 14 percent for compacted onsite soils is recommended for earthwork estimation. In addition, a subsidence value of 0.2 feet may be used in calculating earthwork quantities.

- Static and Dynamic Settlement Potential: A total static settlement of approximately 1 inch, and a differential settlement of approximately of ¾ an inch over a distance of 40 feet are estimated. In addition, based on our analysis using data collected from our borings, an estimated 1 to 2 inches of dry sand (dynamic) total settlement should be anticipated after remedial grading, with a corresponding differential settlement of approximately 1 inch or less for the free-field condition. Permanent structures should be designed to accommodate this magnitude of settlement; non-structural improvements and freestanding guest quarters should not be adversely impacted.

- Strong Ground Motion: The site is located in a seismically active area of southern California and will likely be subjected to strong seismically-related ground shaking during the anticipated life span of the project. Structures within the site should therefore be designed and constructed to resist the effects of strong ground motion in accordance with the current edition of the California Building Code.

Earthwork Guidelines

General Specifications

All earthwork should be performed in accordance with current industry standards of practice in the area following all requirements of the California Building Code (CBC) and local authorities, as well as with the recommendations provided in this report.

Site Clearing

Any remnant structural materials associated with the previously agricultural development within the site, as well as appurtenant exterior improvements (including buried utilities that are not to be protected in place) should be demolished and removed from the site. During site grading, laborers should be provided to clear from fill soils any roots, tree branches, and other deleterious materials missed during initial clearing and grubbing operations.

Our firm should be notified to observe general clearing operations. Should any unusual soil conditions or buried structures be encountered during demolition operations or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of our firm for corrective recommendations.

Ground Preparation

To mitigate the potential for excessive static and dynamic settlements, it is recommended that onsite soils be over-excavated to the following minimum depths, whichever condition is deeper:

- Permanent Structures (Restaurant, Store, etc.): at least 5 feet below the existing ground surface or at least 5 feet below proposed subgrade elevations or 3 feet below the bottom of the deepest footing, whichever is deeper. Lateral limits of overexcavation should extend a minimum of 5 feet outside of the footprint of the structures.

- Pool and Patio Flatwork, Fire Pit area, and Landscape Wall Footing Areas: at least 2 feet below the existing ground surface or at least 2 feet below proposed subgrade elevations, whichever is deeper. Lateral limits of overexcavation should extend a minimum of 2 feet outside of the footprint of the structures.

- Proposed Streets, Sidewalks, and Parking Areas: No Portland Cement Concrete (PCC) or Asphaltic Concrete (AC) streets, parking areas, or sidewalks are noted on the conceptual plan. The general and Fire Department access road construction material is proposed as “Stabilized Deconstructed Granite.” In General, areas of the project proposed for vehicular traffic or parking should be overexcavated at least 2 feet below the existing ground surface or at least 2 feet below proposed subgrade elevations, whichever is deeper. The Fire Department Access roadways may need a designed section (Depths of stabilized deconstructed granite, aggregate base (if necessary), and depth of compacted subgrade), which will be predicated on the Fire Department requirements or other specified loading as required by the responsible civil design professionals.

- Proposed Camping Pads: Insufficient information is available at this time to provide detailed recommendations for ground preparation for the various Glamping accommodations (Teepee Site, Glamping Loft Site, and Glamping Site). It is our understanding that they will be utilizing freestanding structures. Sites of concentrated activity such as these would benefit from the same overexcavation and fill recommendations as provided for Proposed Streets, Sidewalks, and Parking Areas, above, as a base for any walking surface ground covering that may be selected. Ancillary structures, if permanent, should follow the same recommendations as for Permanent Structures, above. Petra can provide enhanced recommendations, if necessary, when additional information becomes available.

Excavation Characteristics

Based on the results of our subsurface evaluation, excavation of native soil within the site is expected to be readily accomplished with conventional earthmoving equipment.

Stability of Temporary Excavation Sidewalls

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed. No temporary excavations along the property lines should be left open, and the backfill should be placed as soon as possible. **The grading contractor is solely responsible for ensuring the safety of construction personnel and the general public.**

Protection of Adjacent Properties

The sidewalls of temporary excavations should be maintained three feet from adjacent property lines or structures sensitive to settlement. If grading is required immediately adjacent to the property line or structure, the overexcavation can be cut at 1½:1 (horizontal to vertical) down from the property line to the desired overexcavation depth. Additional recommendations for slot cutting or other excavation support can be provided if future grading plan review reveals proposed project features that necessitate such techniques.

During the preparation of the grading plan for the subject site, the project civil engineer should take into consideration the location and elevation of the footings of any existing structures that are to be protected in-place. **Grades within the site should not be lowered to the extent that they will have an adverse impact on the lateral stability of adjacent properties or sensitive structures**.

Fill Placement

Remedial grading should be performed as recommended in the preceding paragraphs. Depth of grading will range between 5 feet at the location of any permanent structures to 2 feet for roadways, flatwork, wall foundations, or permanent similar improvements. Removals should extend laterally outside the footprint of the building a minimum distance equal to the depth of the overexcavation. Ultimate removal depths must be determined based on observation and testing by the geotechnical consultant during grading operations. Following removal of unsuitable surficial materials, exposed bottom surfaces in areas approved for engineered fill placement should be first scarified to a depth of 12 inches, flooded and compacted with a heavy vibratory roller in two directions prior to placement of additional fill. Minimum compaction of the upper 12 inches of the removal bottom should meet or exceed 90 percent relative compaction. All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve slightly above-optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557.

Imported Soils

If imported soils are required to complete the planned grading, these soils should consist of clean granular materials devoid of rock exceeding a maximum dimension of 2 inches. Import soils should not contain any organics, trash or similar deleterious materials. Imported soils should also exhibit an expansion index of 20 or less. Prospective import soils should be observed, tested and approved by our firm **prior to importation of any soil to the site.** It is recommended that the project environmental consultant should also be notified so that they can confirm the suitability of the proposed import material from an environmental standpoint.

Geotechnical Observations and Testing During Grading

Exposed bottom surfaces in each remedial removal area should be observed and approved by a representative of our firm **prior to placing fill**. In addition, a representative of our firm should be present onsite during grading operations to observe proper placement and adequate compaction of all engineered fills, as well as to document compliance with the other recommendations presented herein.

Volumetric Changes - Bulking, Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when onsite soils are excavated and replaced as properly compacted fill. Based on data obtained during our exploration, a shrinkage factor on the order of 9 to 14 percent may generally be anticipated. The actual shrinkage that will occur during grading will depend, in part, on the average degree of relative compaction achieved. A maximum subsidence of approximately 0.2 feet should be expected as a result of the scarification and compaction of the exposed bottom surfaces within the removal areas.

The above estimates of shrinkage and subsidence are intended for use by project planners in estimating earthwork quantities and should not be considered absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that will occur during grading.

Post-Grading Considerations for Utility Trenches, Precise Grading and Drainage

Utility Trenches

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Onsite earth materials cannot be densified adequately by flooding and jetting techniques. Therefore, trench backfill materials should be placed in lifts no greater than approximately 12 inches in thickness, watered or air-dried as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of our firm should probe and test the backfills to determine whether adequate compaction has been achieved.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand having a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of our firm to document that an adequate degree of compaction has been achieved. If clean, imported sand is to be used for backfill of exterior utility trenches, it is recommended that the upper 12 inches of trench backfill materials consist of properly compacted onsite soil materials. This is to reduce infiltration of irrigation and rain water into granular trench backfill materials.

Where an exterior and/or interior utility trench is proposed in a direction parallel to a building footing, the bottom of the trench should not extend below a 1:1 (horizontal to vertical) plane projected downward from the bottom edge of the adjacent footing. Where this condition occurs, the adjacent footing should be deepened or the utility constructed and the trench backfilled and compacted prior to footing construction. Where utility trenches cross under a building footing, these trenches should be backfilled with on-site soils at the point where the trench crosses under the footing to reduce the potential for water to migrate under the floor slabs.

<u>Precise Grading and Site Drainage</u>

It is likely that surface drainage systems consisting of sloping concrete flatwork and graded earth swales will be constructed on the subject site to collect and direct all surface water. In addition, the ground surface around the proposed camping areas should be sloped to provide a positive drainage gradient away from the camping structures. The purpose of the drainage systems is to prevent ponding of surface water within the level areas of the site and against building foundations and associated site improvements. It is recommended that the following recommendations be implemented during construction:

1. Area drains should be extended into all planters and landscape areas that are located within 10 feet of buildings, camping areas, and masonry block walls to mitigate excessive exfiltration of irrigation water into the surrounding soils.

2. It is our understanding that the state-of-the-practice for projects in various cities and unincorporated areas of various counties throughout Southern California has been to construct earthen slopes at 2 percent gradient away from the proposed camping pads; drainage swales, driveways, and internal streets should be at 1 percent minimum for earthen swale gradients. With consideration of the arid climate, site soil conditions and an appropriate irrigation regime, Petra considers that the implementation of 2 percent slopes away from the structures and developed camping pads and 1 percent swales to be suitable for the subject project.

3. It should be emphasized that all surface drainage controls must be properly maintained and unobstructed, and that future improvements not alter established gradients unless replaced with suitable alternative drainage systems. Further, where the flowline of any swale exists within five feet of any paved surface, the adjacent footings shall be deepened appropriately to maintain minimum embedment requirements as measured from the flowline elevation of the swale.

4. Concrete flatwork surfaces located within 10 feet of any masonry wall foundations should be inclined at a minimum gradient of 2 percent away from top of footings. Neither rain nor excess irrigation water should be allowed to collect or pond against building or wall foundations.

5. For the landscaped areas, a watering program should be implemented that maintains a uniform, near optimum moisture condition in the soils. Overwatering and subsequent saturation of the soils should be avoided. As an alternative to a conventional irrigation system, drip irrigation systems are strongly recommended for all planter areas. Xeriscaping certain areas may also be suitable for the project and environmental conditions.

6. It is assumed that the proposed finished grade elevations around the site perimeter will match existing offsite grades. No slopes of significant height are currently anticipated. This would preclude substantial soil erosion or loss of topsoil within the developed site. There is the potential for localized erosion during grading operations; however, it is expected that this will be mitigated through the implementation of a Storm Water Pollution Prevention Plan (SWPPP) for the site as required by the oversight agencies.

7. The Pipes Canyon Wash, which runs north-northeast, borders the proposed development to the east. A gentle, but irregular, slope descends from the development area to the bottom of the wash at gradients varying between 6:1 and 10:1. There may be localized areas that exceed 3:1 (horizontal to vertical), the CBC mandated minimum slope at which foundation setbacks must be established. It appears from the concept plan that only the pool may be close enough to the slope to potentially require setbacks as mandated in Section 1808.7 and Figure 1808.7.1 of the CBC (2019). Grading plan review is suggested to determine the actual setback requirements for the pool or any other currently unspecified structures, but foundation setback requirements in general for a slope 3:1 or steeper require a setback equal to the height of the slope (H) divided by six with a maximum of 20 feet for pools and H divided by three with a maximum of 40 feet for other permanent structures, including landscape walls.

8. Site grading should not permit collected water to flow down the face of the slope. All graded areas should be finished with a minimum of 2% slope away from the face of the slope or with appropriate drainage berms to intercept runoff water, and suitable non-erosive collection infrastructure constructed to transport water to the onsite basin.

FOUNDATION DESIGN GUIDELINES

Allowable Bearing Capacity, Estimated Settlement, and Lateral Resistance

Allowable Soil Bearing Capacities

Pad Footings

An allowable soil bearing capacity of 2,000 pounds per square foot may be utilized for design of isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade for pad footings that are not a part of the slab system and are used for support of such features as roof overhang, second-story decks, patio covers, etc. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 3,000 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Continuous Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of 1 foot wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,000 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Estimated Footing Settlement

Based on the allowable bearing values provided above, total static settlement of the footings under the anticipated loads is expected to be on the order of 1 inch. Differential settlement is expected to be less than 1 inch over a horizontal span of 40 feet. The majority of settlement is likely to take place as footing loads are applied or shortly thereafter.

Lateral Resistance

A passive earth pressure of 200 pounds per square foot per foot of depth, to a maximum value of 2,000 pounds per square foot, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.3 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be increased by one-third when designing for transient wind or seismic forces. Isolated poles not affected by ½ inch motion at the ground surface due to short term lateral loads may be designed for a passive earth pressure of 400 pounds per square foot per foot of depth, to a maximum value of 2,000 pounds per square foot.

It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Guidelines for Footings and Slabs on-Grade Design and Construction

The results of our laboratory tests performed on representative samples of near-surface soils within the site during our investigation indicate that these material predominantly exhibit expansion indices that are less than 20. As indicated in Section 1803.5.3 of 2019 California Building Code (2019 CBC), these soils are considered non-expansive and, as such, the design of slabs on-grade is considered to be exempt from the procedures outlined in Sections 1808.6.2 of the 2019 CBC and may be performed using any method deemed rational and appropriate by the project structural engineer. However, the following minimum recommendations are presented herein for conditions where the project design team may require geotechnical engineering guidelines for design and construction of footings and slabs on-grade the project site.

The design and construction guidelines that follow are based on the above soil conditions and may be considered for reducing the effects of variability in fabric, composition and, therefore, the detrimental behavior of the site soils such as excessive short- and long-term total and differential heave or settlement. These guidelines have been developed on the basis of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these guidelines has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future heave or settlement.

It should also be noted that the suggestions for dimension and reinforcement provided herein are performance-based and intended only as preliminary guidelines to achieve adequate performance under the anticipated soil conditions. However, they should not be construed as replacement for structural engineering analyses, experience and judgment. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to slab and footing dimensions, and reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion) as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

Conventional Slabs on-Grade System

Given the expansion index of less than 20, as generally exhibited by onsite soils, we recommend that footings and floor slabs be designed and constructed in accordance with the following minimum criteria.

<u>Footings</u>

1. Exterior continuous footings supporting one- and two-story structures should be founded at a minimum depth of 12 inches below the lowest adjacent final grade, respectively. Interior continuous footings may be founded at a minimum depth of 10 inches below the top of the adjacent finish floor slabs.

2. In accordance with Table 1809.7 of 2019 CBC for light-frame construction, all continuous footings should have minimum widths of 12 inches for one- and two-story construction. We recommend all continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

3. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced with a similar manner as provided above.

4. Interior isolated pad footings, if required, should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs for one- and two-story buildings. Pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

5. Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

6. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2019 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

<u>Building Floor Slabs</u>

1. Concrete floor slabs should be a minimum 4 inches thick and reinforced with No. 3 bars spaced a maximum of 24 inches on centers, both ways. Alternatively, the structural engineer may recommend the use of prefabricated welded wire mesh for slab reinforcement. For this condition, the welded wire mesh should be of sheet type (not rolled) and should consist of 6x6/W2.9xW2.9 (per the Wire Reinforcement Institute, WRI, designation) or stronger. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid-depth. Care should be exercised to prevent warping of the welded wire mesh between the chairs in order to ensure its placement at the desired mid-slab position.

   Slab dimension, reinforcement type, size and spacing need to account for internal concrete forces (e.g., thermal, shrinkage and expansion) as well as external forces (e.g., applied loads), as deemed necessary.

2. Conditioned area concrete floor slabs and areas to receive moisture sensitive floor covering should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote

uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

3. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all floor slab areas should be thoroughly moistened to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches below the bottoms of the slabs.

4. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2019 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

General Corrosivity Screening

As a screening level study, limited chemical and electrical tests were performed on samples considered representative of the onsite soils to identify potential corrosive characteristics of these soils. The following sections present the test results and an interpretation of current codes and guidelines that are commonly used in our industry as they relate to the adverse impact of chemical contents and electrical resistance of the site soils on various components of the proposed structures in contact with site soils.

A variety of test methods are available to quantify corrosive potential of soils for various elements of construction materials. Depending on the test procedures adopted and the characteristics of the leachate that is used to extract the target chemicals from the soils and the test equipment; the results can vary appreciably for different test methods in addition to those caused by variability in soil composition. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies. In drawing conclusions from the results of our chemical and electrical laboratory testing and providing mitigation guidelines to reduce the detrimental impact of corrosive site soils on various components of the structure in contact with site soils, heavy references were made to 2019 California Building Code (2019 CBC) and American Concrete Institute publication (2019 Building Code Requirements for Structural Concrete, ACI 318-14). Where relevant information was not available in these codes, references were made to guidelines developed by California Department of Transportation (Caltrans), Post-Tensioning Institute (PTI DC10.5-12) and other reputable institutions and/or publications. Specifically, the reference to Caltrans approach were made because their risk management protocol for highway bridges are considered comparable to those for residential or commercial structures and that Post Tensioning Institute (PTI), in part, accepts and uses Caltrans' relevant corrosivity criteria for post-tensioned slabs on-grade.

It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron pipes) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer may not be informed of these choices. Therefore, for conditions where such elements are considered, we recommend that other, relevant project design professionals (e.g., the architect, landscape architect, civil and/or structural engineer) also consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

Concrete in Contact with Site Soils

Soils containing soluble sulfates beyond certain threshold levels, as well as acidic soils are considered to be detrimental to long-term integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates (SO_4^{2-}) concentration in soils determined in accordance with California Test Method No. 417. Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in accordance with California Test Method No. 643.

Article 1904.1 of Section 1904 of the 2019 CBC indicates that structural concrete shall conform to the durability requirements of ACI 318. Concrete durability is impacted by exposure to water soluble chemicals and its resistance to fluid penetration. Section 19.3 of Chapter 19 of ACI 318-14 provides guidelines for assigning exposure categories and classes for various conditions. **Exposure Category S**, which is subdivided to four **Exposure Classes of S0, S1, S2 and S3**, applies to concrete in contact with soil or water containing deleterious amounts of water soluble ions. Table 4 below provides a summary of demarcation levels from the referenced sources. In this table, acidity classification is adopted from the United States

Department of Agriculture Natural Resources Conservation Service, formerly Soil Conservation Service classification of soil pH ranges

TABLE 4

Soluble Sulfates and Acidity Levels Classification for Corrosion of Concrete in Contact with Soils

Source	Tests	Soluble Sulfates (SO_4^{2-}) Concentration	Acidity (pH)	Classification/Severity
CBC/ACI	Soluble Sulfates by Cal 417	0.00 – 0.10 % by mass		**S0/Not Applicable**
		0.10 – 0.20 % by mass		**S1/Moderate**
		0.20 – 2.00 % by mass		**S2/Severe**
		> 2.00 % by mass		**S3/Very Severe**
Caltrans	Soluble Sulfates by Cal 417 pH by Cal 643	0 – 1,499 ppm	7.1 -14	**Neutral to Very Strongly Alkaline**
		1,500 – 1,999 ppm	5.6 - 7	**Moderately Acid to Neutral**
		2,000 – 15,000 ppm	3 – 5.5	**Ultra-Acid to Moderately Acid**

The results of our limited in-house laboratory tests indicate that on-site soils tested contain a water-soluble sulfate content of 0.0015 percent by weight. Based on Table 19.3.1.1 of ACI 318-14, the **Exposure Class S0** is appropriate for onsite soils. For this exposure class, Table 19.3.2.1 of ACI 318-14 provides that no restriction for cement type or maximum water-cement ratio for the fresh concrete would be required. Further, this table indicates that the concrete minimum unconfined strength should not be less than 2,500 psi.

The results of limited in-house testing of a representative sample indicate that soils within the subject site are slightly alkaline with respect to pH (a pH of 7.7). Based on this finding and according to Table 8.22.2 of Caltrans' 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type II Modified cement may be used.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer and the contractor responsible for concrete placement for structural concrete used in exterior and interior footings, interior slabs on-ground, isolated slabs and flatwork, walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

Metals Encased in Concrete

Soils containing a soluble chloride concentration beyond a certain threshold level are considered corrosive to metallic elements such as reinforcement bars, tendons, cables, bolts, anchors, etc. that are encased in concrete that, in turn, is in contact with such soils. For the purpose of this study, soluble chlorides (Cl) in soils were determined in accordance with California Test Method No. 422.

As stated earlier, Article 1904.1 of Section 1904 of the 2019 CBC indicates that structural concrete shall conform to the durability requirements of ACI 318. Concrete durability is impacted by exposure to water soluble chemicals and its resistance to fluid penetration. Section 19.3 of Chapter 19 of ACI 318-14 provides guidelines for assigning exposure categories and classes for various conditions. **Exposure Category C**, which is subdivided to three Exposure Classes of C0, C1, and C2, applies to non-prestressed and prestressed concrete exposed to conditions that require additional protection against corrosion of reinforcement.

According to Table 19.3.1.1 of ACI 318-14, the **Exposure Class C0** is appropriate for reinforced concrete that remains dry or protected from moisture. Similarly, the **Exposure Class C1** is appropriate for reinforced concrete that is exposed to moisture but not to external sources of chlorides. And, lastly, the **Exposure Class C2** is appropriate for reinforced concrete that is exposed to moisture and external sources of chlorides as “*deicing chemicals, salt, brackish water, seawater, or spray from these sources*”.

Based on our understanding of the project, it is our professional opinion that the **Exposure Class C1** is appropriate for a majority of reinforced concrete to be placed at the site in contact with site soils. It should be noted, however, that the **Exposure Class C2** is more appropriate for reinforced concrete that is planned for pool walls and decking, should such features be considered for the project.

The results of our limited laboratory tests performed indicate that onsite soils contain a water-soluble chloride concentration of 120 parts per million (ppm). No maximum water/cement ratio for the fresh concrete is prescribed by ACI 318 for **Exposure Class C1** condition. Table 19.3.2.1 of ACI 318-14 indicates that concrete minimum unconfined compressive strength, f'_c, should not be less than 2,500 psi. For **Exposure Class C2** condition, Table 19.3.2.1 of ACI 318-14 requires that the maximum water/cement ratio of the fresh concrete should not exceed 0.40 and concrete minimum unconfined compressive strength, f'_c, should not be less than 5,000 psi.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer for reinforced concrete placement for structural concrete used in exterior and interior footings, interior slabs on-ground, walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

It should be noted that another source of elevated chloride-ion concentration can be the chloride content of water that is used to prepare the fresh concrete at the plant. The protection against high chloride concentration in fresh concrete should therefore be provided by concrete suppliers for the project.

Metallic Elements in Contact with Site Soils

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to building metallic components that are in contact with site soils. The minimum electrical resistivity measurement provides a simple indication of relative concentration of soluble salts in the soil and, therefore, is widely used to estimate soil corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643. The soil corrosion severity rating is adopted from the Handbook of Corrosion Engineering by Pierre R. Roberge.

The minimum electrical resistivity for onsite soils was found to be 10,800 ohm-cm based on limited testing. The result indicates that on-site soils are **Mildly Corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils may need to be protected against detrimental effects of corrosive soils. Such protection could include the use of galvanized tubing, coated pipes, or wrapping or encasing these metallic objects in special protection wrappings or conduits. It should be noted that at this time Petra is not aware of any plans to incorporate such items for the proposed buildings. Should such elements be considered for these building, we recommend that a corrosion engineer to be consulted to provide appropriate recommendations for long term protection of metallic elements in contact with site soils.

Exterior Concrete Flatwork

The guidelines that follow should be considered as minimums and are subject to review and revision by the project architect, structural engineer and/or landscape consultant as deemed appropriate.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Private driveways that will be designed for the use of passenger cars and RV parking should also be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load in public access areas, segments of road that will be paved with concrete (such as bus stops and cross-walks) or access roads and driveways, which serve multiple residential units or garages, that will be subject to heavy truck loadings should have a minimum thickness of 5 inches and be provided with control joints spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design of the interior roads.

Reinforcement

All concrete flatwork having their largest plan-view panel dimension exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designation in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs.

> *The reinforcement recommendations provided herein are intended as guidelines to achieve adequate performance for anticipated soil conditions. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

Edge Beams (Optional)

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

Subgrade Preparation

Compaction

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent. Where all-weather fire department access roads (including stabilized deconstructed granite road material), concrete public roads, concrete segments of roads and/or concrete access driveways are proposed, the upper 6 inches of subgrade soil should be compacted to a minimum 95 percent relative compaction.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the local retention basin or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas since they introduce excessive water into the subgrade soils and allow root invasion, both of which can cause heaving and cracking of the flatwork.

GRADING AND FOUNDATION PLAN REVIEW

It must be emphasized that the recommendations provided throughout this report are based solely on conceptual design information provided by the Client, and that no finalized grading plans, structural plans or details were available for review as of the date of this report. As such, the conclusions and recommendations provided herein should be considered as tentative. Once such plans and details become available, our firm should be retained to review these documents to determine the applicability of our recommendations to the actual construction proposed. Additional recommendations and/or modification of the recommendations provided herein will be provided if necessary depending on the results of the grading plan and/or structural plan review. Additional field exploration may be required once the layout, configuration, and building loads are provided.

Furthermore, no specific plans have been provided regarding the Amphitheater featuring lawn seating. Graded slopes in excess of 5:1 (horizontal to vertical) may require additional geotechnical design with respect to slope stability, terracing, and drainage to meet the requirements of the CBC and local regulations. **Petra** should review the proposed grading plans for this and other features when available. Additional or modified recommendations may be required by proposed grading design.

FUTURE IMPROVEMENTS AND GRADING

If additional improvements are considered in the future, our firm should be notified so that we may provide design recommendations to mitigate movement, settlement and/or tilting of the structures. Potential problems can develop when drainage on the pads is altered in any way such as placement of fill and construction of new walkways, patios, landscape walls, or planters. Therefore, it is recommended that we be engaged to review the final design drawings, specifications and grading plan prior to any new construction. If we are not provided the opportunity to review these documents with respect to the geotechnical aspects of new construction and grading, it should not be assumed that the recommendations provided herein are wholly or in part applicable to the proposed construction.

REPORT LIMITATIONS

This report is based on the proposed project and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory investigation are believed representative of the project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guarantee or warranty. This report should be reviewed and updated after a period of one year or if the project concept changes from that described herein.

The information contained herein has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

This report is subject to review by the controlling authorities for this project. Should you have any questions, please do not hesitate to call.

Respectfully submitted,

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1/11/2021

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