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GEOTECHNICAL DUE-DILIGENCE ASSESSMENT
PARCEL F, ASSESSOR PARCEL NUMBERS 224-142-30-00; -31-00; -32-00 AND -33-00
ADJACENT NORTHWEST CORNER OF LEHNER AVENUE AND CONWAY DRIVE
CITY OF ESCONDIDO, SAN DIEGO COUNTY, CALIFORNIA

ESCONDIDO NORTH, LLC

April 15, 2021
J.N. 20-439

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ESCONDIDO NORTH, LLC
30200 Rancho Viejo Road, Suite B
San Juan Capistrano, California 92675

Attention: Mr. John Kaye

Subject: Geotechnical Due-Diligence Assessment; Parcel F, Assessor Parcel Number's (APN's) 224-142-30-00; -31-00; -32-00 and -33-00, Adjacent Northwest Corner of Lehner Avenue and Conway Drive, City of Escondido, San Diego County, California

Dear Mr. Kaye:

Petra Geosciences, Inc. (Petra) is submitting herewith our geotechnical due-diligence assessment report for Parcel F, Assessor Parcel Number's (APN's) 224-142-30-00; -31-00; -32-00 and -33-00, located adjacent the northwest corner of Lehner Avenue and Conway Drive, in the city of Escondido, San Diego County, California. This work was performed in general accordance with the scope of work outlined in our Revised Proposal No. 20-439P dated February 5, 2021. This report presents the results of our field exploration, laboratory testing, and our engineering judgment, opinions, conclusions and recommendations pertaining to preliminary geotechnical design aspects for the proposed residential development.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report or should you require additional information, please contact this office.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Jonathan Cain
Senior Associate Geologist

John Montgomery Schultz
Associate Engineer

JC/JMS/lv

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INTRODUCTION

Petra Geosciences, Inc. (Petra) is presenting herewith the results of our geotechnical due-diligence assessment for the proposed development of Parcel F, APNs 224-142-30-00; -31-00, -32-00 and -33-00, located adjacent the northwest corner of Lehner Avenue and Conway Drive in the city of Escondido, San Diego County, California. This assessment included a review of published and unpublished literature, site reconnaissance and subsurface exploration, as well as a review of geotechnical maps pertaining to geologic hazards which may have an impact on the proposed residential construction.

PURPOSE AND SCOPE OF SERVICES

The purposes of this study were to compile and review pertinent geotechnical information within the project site area and to provide recommendations pertaining to feasibility of site development from a geotechnical engineering viewpoint.

The scope of our assessment consisted of the following:

- Performed a site reconnaissance and conducted geologic mapping of the property to evaluate existing onsite conditions.
- Reviewed available published and unpublished geologic data, maps, available online aerial imagery and geotechnical documents concerning geologic and soil conditions within, and adjacent to the site which could have an impact on the proposed improvements.
- Reviewed a previous report by another consultant (North County Compaction Engineering, 2004) for a previous investigation of a portion of the property to be developed.
- Excavated two trenches around bedrock outcrops within the northern portion of the site utilizing a rubber tire backhoe. The trenches were excavated to help determine excavation characteristics of the bedrock.
- Excavated one percolation test boring to approximately 10 feet below ground surface within the area of the proposed water quality basin using a truck-mounted drill rig equipped with hollow-stem augers. The boring was converted into a shallow percolation test hole to evaluate the infiltration characteristics of the soils in the area of the proposed WQMP basin.
- A falling head percolation test was conducted on the percolation boring in general compliance with City of Escondido and/or County of San Diego standards.
- Logged and field-classified soil materials encountered in the percolation boring in accordance with the visual-manual procedures outlined in the Unified Soil Classification System and the American Society for Testing and Materials (ASTM) Procedure D 2488.

- Preformed appropriate laboratory testing of representative samples (bulk) to determine their engineering properties. The samples tested were obtained from the percolation boring.

- Performed appropriate engineering and geologic analysis of the data with respect to the proposed improvements.

- Prepared this report, including pertinent figures and appendices presenting the results of our assessment and recommendations for the proposed improvements, in general conformance with the requirements of the 2019 California Building Code (CBC), as well as in accordance with applicable local jurisdictional requirements.

LOCATION AND SITE DESCRIPTION

The subject site is located at the northwest corner of Lehner Avenue and Conway Drive in the city of Escondido, San Diego County, California. The site, which encompasses approximately 5.23 acres, is a rectangular-shaped property comprised of four parcels of land identified as APN's 224-142-30-00; -31-00; -32-00; and -33-00. Topographically, site elevations range from approximately 806 feet above mean sea level (msl) within the northwest portion of the site to approximately 757 feet above msl within the southwest portion of the site. The site is bounded by Stanley Avenue along the north; Conway Drive along the east; Lehner Avenue along the south and residential development along the west. The location of the site is shown on Figure 1.

Existing residential structures were observed onsite during our site reconnaissance. The residential structures are single-family residences with six located within the east portion of the site adjacent Conway Drive. Two additional residential structures, a house and detached structure, are located in the northwest portion of the site, adjacent Stanley Avenue. The remainder of the site is vacant land. Site vegetation consists of native grasses and weeds with mature trees.

Literature Review

As part of this assessment, we reviewed the prior geotechnical due diligence evaluation by North County Compaction Engineering, Inc. (NCCEI, 2004). In addition, we reviewed the Pasco Laret Suiter and Associates, Conway + Lehner Option A Site Plan and the F&H Density Bonus Site Plan (Pasco, 2021a,b) for the subject site. Petra also reviewed available published and unpublished geologic data, maps and aerial imagery pertaining to regional geology, faulting and geologic hazards that may affect the site. The results of this review are discussed in the Findings section of this report.

Proposed Construction

Based on a Conway+Lehner Option A Site Plan and the F&H Density bonus Site Plan by Pasco Laret Suiter and Associates., the site is proposed to be developed as a residential tract with two configuration options. On the Conway+Lehner Option A Site Plan the tract will consist of a cul-de-sac street (street A), and twenty-two lots. The F&H Density bonus Site Plan depicts the tract to consist of a cul-de-sac street (Street B), twenty-one (21) residential lots, a water quality basin (Lot A), and a public utility easement. At this time, no specific development plans have been provided for our review. However, it is assumed the structures will utilize typical wood-frame construction with either conventional or post-tension slab-on-ground foundation systems. Building loads are assumed to be typical for this type of relatively light residential construction.

Subsurface Exploration

Previous Field Exploration

North County Compaction Engineering, Inc. (NCCEI, 2004) advanced four (4) exploratory test pits onsite to a maximum depth of 9 feet below existing onsite ground levels. Based on the test pits advanced, it was reported that up to six (6) feet of loose alluvial soils were observed in the lower (southern) half of the property. At the time of their field work, NCCEI reported this area of the property to consist of alluvial soils overlying dense to very dense, "granitic sands of the Southern California Batholith Formation" (CEI, 2004). Within the northern portion of the site, test pits reportedly encountered "granitic formation" at a depth of two (2) feet below the ground surface.

Petra's Exploration

A subsurface exploration program was performed under the direction of an engineering geologist from Petra on February 24, 2021. One percolation boring was drilled within the southeast portion of the property in the general location of the water quality basin "Lot A". The proposed bottom depth of the basin was unknown during the time of our assessment, so the 8-inch diameter boring was advanced to 10 feet below existing grade. Soils encountered in the percolation boring P-1 consisted of silty sands and highly weathered granitic bedrock.

A three-inch diameter perforated casing was installed within the borehole and the annular space packed with gravel. The hole was pre-soaked immediately after drilling and casing installation. During removal of the auger and before casing installation the bottom of the hole was infilled in the bedrock portion (resulting in slightly more than 8 feet of casing installed). Therefore the portion of the borehole consisting of the

bottom 5 feet of the soil zone was utilized for percolation testing. Percolation testing was conducted the following day by one of Petra's geologists.

The falling-head percolation test data from the boring (test P-1) was utilized in determining the test infiltration rate, I_t, expressed in units of inches/hour, utilizing the Porchet Method (RCFCWCD, 2011). The infiltration rate, I_t, was calculated for the test by determining the volumetric water flow through the wetted borehole surface area, expressed in terms of inches per hour. The falling-head percolation test yielded an un-factored infiltration rate of 0.09 inches per hour. Test data for the percolation test is attached in Appendix D.

On February 25, 2021, a second phase of the field assessment included the advancement of two (2) exploratory trenches utilizing a rubber tire back-hoe to evaluate the excavation characteristics of the granitic bedrock. The exploratory trenches were excavated within the northwest portion of the site around granitic bedrock outcrops which may expose hard bedrock at relatively shallow depths with a corresponding negative impact on the proposed grading and construction operations.

Laboratory Testing

Laboratory testing included the determination of in-situ maximum dry density and in-situ optimum moisture content; expansion index, and preliminary soil corrosivity screening (soluble sulfate and chloride content, pH and minimum resistivity). A description of laboratory test methods and summaries of the laboratory test data are presented in Appendix B.

FINDINGS

Literature Review

As discussed previously, we reviewed the geotechnical study by North County Compaction Engineering, Inc. (NCCEI, 2004). Noteworthy findings made from reviewing the report are discussed herein, with any comments by Petra provided in *italic* font.

NCCEI, 2004:

- NCCEI's report was for a large portion of the subject site and included three of the four subject parcels. The parcel that was not included was APN 224-142-30-00 (943 Stanley Avenue) which is located within the northwest corner of the subject property. *(Petra: Our exploratory trenches around the rock outcrops were located on the parcel not previously explored.)*

- NCCEI stated that the results of their investigation indicated that the proposed development was feasible provided the recommendations contained within their report were adhered to.

- NCCEI described the property as "L-shaped", consisting of three (3) lots occupied by six (6) single-family dwellings. Topographically, the property was reported to consist of a gentle sloping hillside on the north half, and the south half predominantly flat.

- NCCEI reported that the majority of the dwellings are on underground septic systems.

- NCCEI's field exploration program consisted of the excavation of four (4) exploratory test pits to a maximum depth of 9 feet below grade. Based on the test pits advanced, relatively shallow granitic formational materials were encountered at a depth of two (2) feet in the northern half of the property. Alluvial soils, six (6) feet in thickness, were reportedly excavated in the southern half of the site.

- NCCEI concluded that isolated boulders may be encountered during earthwork and recommended over-sized rock with a diameter greater than 6-inches should be sorted out of the fill and disposed of in special non-structural fill areas designated by the soils engineer at the time of grading. They recommended that oversized rock should be mixed with a substantial amount of fines, well-watered and mechanically compacted to minimize the probability of subsidence. Nesting of rock is not recommended, and rock over 36-inches in diameter should not be placed in non-structural fills. No structures should be constructed within 15 feet on any designated non-structural area.

- Groundwater seepage was encountered by NCCEI in one test pit in the south-central portion of the site at the time of their field investigation, at a depth of approximately 5 feet. The excavation (Test Pit #3) revealed that seepage was encountered one (1) foot above the granitic formation. NCCEI concluded that seepage was likely due to collection of seasonal rains in a depression and recommended additional test pits in this area prior to the start of earthwork to re-evaluate groundwater conditions. *(Petra: The potential, albeit small, for this seepage to be associated with an onsite septic system should also be considered. In any case, the grading contractor should be aware of this information.)*

- NCCEI stated that, although groundwater was encountered, it is their opinion that soil liquefaction is unlikely to occur if loose alluvial soils are removed to competent formational soil and re-compacted in accordance with recommendations provided in their report. *(Petra: We concur that liquefaction should not be a concern if soils are removed to the competent bedrock and recompacted.)*

- NCCEI indicated that structures planned for the proposed development should not straddle cut/fill transitions. CEI recommended that the cut portion of the pad be over-excavated to a minimum depth of one (1) foot below the base of the footing. The limits of work should extend a minimum of five (5) feet beyond the aerial extent of the proposed dwelling. *(Petra: It should be noted that the minimum depth of overexcavation for a cut/fill transition should not be less than ½ of the depth of the deepest fill under the structure.)*

- Conventional foundation design parameters provided in the 2004 NCCEI report pertained to one-story and two-story residential construction, for non-expansive soils with average strength parameters. An allowable soil bearing capacity of 1,990 pounds per square foot (psf) was recommended. A minimum foundation setback of 8 feet from the top of slope was provided. *(Petra: Updated foundation design recommendations in accordance with the 2019 CBC are provided later within this report.)*

- NCCEI concluded that cut and compacted fill slopes up to a height of 20 feet, with a maximum gradient of 2:1 (horizontal to vertical), will be stable related to deep seated failure provided they are maintained properly. A slope stability analysis factor of safety of 1.5 was reported by NCCEI utilizing Taylor's

Charts for cut and compacted fill slopes. *(Petra: If necessary, slope stability analysis should be re-evaluated once grading plans are available for review.)*

- A (seismic) Soil Profile of SD (NCCEI, 2004) was provided for a Type "A" fault (Elsinore/Julian) at a distance greater than 15 kilometers (from the site) based upon the 1997 Uniform Building Code (UBC). *(Petra: Based on the relative age of the previous investigation (2004), foundation and seismic design recommendations have been updated to current code requirements, 2019 California Building Codes [CBC].)*

- The NCCEI report stated that all concrete (under the rigid pavement heading) should have a minimum compressive strength of 3,250 pounds per square inch (psi). *(Petra: It should be noted that per the City of Escondido [2013] Construction Specifications, concrete for footings shall have a minimum compressive strength of 3,000 psi at 28 days, and shall be composed of 1 part cement, 3 parts sand, 4 parts of 1inch maximum size rock, and not more than 7½ gallons of water per sack of cement.)*

- The results of the laboratory testing by NCCEI indicate that near surface soils onsite are non-expansive. *(Petra: it should be noted that no sulfate, chloride, or other corrosion testing was conducted for the project by NCCEI [2004]. Testing was conducted during this assessment to provide preliminary foundation recommendations. However, additional soils testing should be performed to include: expansion index and corrosion testing, conducted at the conclusion of earthwork [grading] to provide specific recommendations per the most recent California Building Code [CBC], for the design of foundations and other concrete elements anticipated to be in contact with onsite soils.)*

Regional Geologic Setting

Geologically, the site lies within the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Range region extends from the tip of Baja California to the Transverse Ranges and the Los Angeles Basin and is characterized by northwest trending mountain ranges separated by subparallel fault zones. In general, the province is underlain primarily of plutonic rock of the Southern California Batholith. The Peninsular Range Geomorphic Province is generally characterized by alluviated basins and elevated erosion surfaces.

More specifically, the subject site lies within the rolling foothills east of Escondido. According to the 7.5 Minute Geologic Map of the Valley Center Quadrangle (Kennedy 1999), Pleistocene-age Older Alluvial Flood Plain Deposits which are moderately well consolidated, poorly sorted, permeable flood plain deposits underlie the site. These Older Alluvial Flood Plain Deposits are underlain by Cretaceous-age Granitic rocks of the Southern California Batholith.

Local Geology and Subsurface Soil Conditions

Several geologic units were encountered during our limited assessment. The earth materials encountered within the exploratory trenches and percolation boring consisted of fills, older alluvial deposits and

Cretaceous age bedrock of the Southern California Batholith. These units, from younger to older, are described below.

Artificial Fill (af): These soils were comprised of fine-grained sand with silt, and silty sands that were brown and dark brown, dry to slightly moist and medium dense.

Older Alluvial Deposit (Qoal): Older alluvial deposits were fine to medium grained silty sands observed to be dark yellowish brown, light brown and brown, moist and medium dense.

Granitic Bedrock: Cretaceous-age granitic bedrock was observed within the bottom of each trench and the percolation boring. The granitic rock was dark orangish brown, yellowish brown and gray, moderately hard to very hard and slightly to highly weathered.

Laboratory Testing

Limited laboratory testing was conducted on bulk soil samples collected from percolation boring P-1. Results are discussed in appropriate sections below, and data is provided in Appendix B.

Groundwater

The site is located within the Escondido Valley Groundwater Basin, (California Department of Water Resources, [CDWR], 2004). Two historic groundwater wells were listed within the vicinity of subject site on the CDWR water data library (CDWR, 2021). Based on our review, historic groundwater levels within the vicinity range between 2± and 28± feet below the ground surface. In general, groundwater depth varies within the area and though flow direction beneath the subject site is unknown, it is believed to be toward the west-southwest.

Compressible Soils

A significant geotechnical factor affecting the project site is the presence of near-surface compressible topsoil and older alluvial deposits. Such materials in their present state are not considered suitable for support of fill or structural loads. Accordingly, these materials will require removal to competent older alluvial deposit soils or granitic bedrock and replacement with properly compacted fill.

Flooding

Based on our review, storm water in the form of localized sheet flooding and/or channelized flows from adjacent properties has the potential to affect the site. Based on current site configurations, it is anticipated a drainage study will be performed by the project civil engineer. As such, the potential for localized surface flooding is considered low.

Expansive Soils

Based on our previous tests results, the silty sand soils encountered within the site were found to have a Very Low expansion potential (Elevation Index of 0-20). Since site grading remains to be completed, additional sampling and laboratory testing is recommended for expansion, as well as general corrosion potential, once rough grading is complete for the purposes of providing final foundation design recommendations.

Faulting

Based on our review of published geologic maps, no faults are known to project through the property, and no portion of the site lies within an Earthquake Fault Hazard Zone as designated by the State of California pursuant to the Alquist-Priolo Earthquake Zoning Act (CGS, 1977). No evidence for lineal topography was observed in aerial photographs reviewed. The closest known active earthquake fault is the Elsinore fault zone (Julian Section) which has been mapped approximately 12.5 miles northeast of the site (Kennedy and Tan, 2005).

However, it should be noted that according to the USGS Unified Hazard Tool website that fault would probably generate the most severe site ground motions and, therefore, is the majority contributor to the deterministic minimum component of the ground motion models. The subject site is located at a distance of more than 9.5 miles (15 km) from the surface projection of this fault system, which is capable of producing magnitude 7 or larger events with a slip rate along the fault greater than 0.04 inch per year. As such, the site doe does not need to be considered as a **Near-Fault Site** in accordance with ASCE 7-16, Section 11.4.1.

Landslides and Secondary Seismic Effects

The site and immediate area exhibits gently sloping topography that is not prone to landsliding. Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure. Various general types of ground failures, which might occur as a consequence of severe ground shaking at the site, include ground subsidence, ground lurching and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoil and groundwater conditions, in addition to other factors. Based on the site conditions, proposed grading and gentle topography across the site, landsliding, ground subsidence and lateral spreading are considered unlikely at the site. However, due to the close proximity of the site to the Elsinore Fault Zone, significant ground lurching should be anticipated during a seismic event.

Seismically induced flooding that might be considered a potential hazard to a site normally includes flooding due to tsunami or seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. Lake Dixon is the closest reservoir located approximately 1.6 miles east-southeast of the subject site. Drainage from the dam is to the southeast, therefore, the potential for seiche or inundation is considered negligible. Because of the inland location of the site, flooding due to a tsunami is also considered negligible at the site.

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 building in accordance with 2019 CBC, Table 1604.5. No shear wave velocity measurement were performed at the site, 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 1, 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.

TABLE 1

Seismic Design Parameters

Ground Motion Parameters	Specific Reference	Parameter Value	Unit
Site Latitude (North)	-	33.1599	°
Site Longitude (West)	-	-117.0809	°
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)	7.7 (3)	-
R – Distance to Surface Projection of Fault	USGS Unified Hazard Tool (3)	21 (3)	km
S_s - Mapped Spectral Response Acceleration Short Period (0.2 second)	Figure 1613.2.1(1) (1)	0.938 (4)	g
S_1 - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(2) (1)	0.341 (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)	1.126 (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)	0.750 (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)	0.406	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)	0.487 (4)	g
Design PGA ≈ (⅔ PGA_M) - Slope Stability (†)	Similar to Eqs. 16-38 & 16-39 (2)	0.325	g
Design PGA ≈ (0.4 S_{DS}) – Short Retaining Walls (‡)	Equation 11.4-5 (2)	0.30	g
C_{RS} - Short Period Risk Coefficient	Figure 22-18A (2)	0.919 (4)	-
C_{R1} - Long Period Risk Coefficient	Figure 22-19A (2)	0.921 (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.

Discussion - General

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 1.

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.

Discussion – Seismic Design Category

Please note that the Seismic Design Category, SDC, is also designated as "null" in Table 1. For the 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 fundamental period of structure, story drift, seismic response coefficient, and relative rigidity of the diaphragms 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.

Discussion – Equivalent Lateral Force Method

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 0.606 seconds and the value of Long Period Transition Period, T_L, is provided in Table 1 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 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.

Liquefaction and Seismically-Induced Settlement

Assessment of liquefaction potential for a particular site requires knowledge of a number of regional as well as site-specific parameters, including the estimated design earthquake magnitude, the distance to the assumed causative fault and the associated probable peak horizontal ground acceleration at the site, subsurface stratigraphy and soil characteristics, and groundwater elevation. Parameters such as distance to causative faults, estimated probable peak horizontal ground acceleration can readily be determined using published references, or by utilizing a commercially available computer program specifically designed to perform a probabilistic analysis. On the other hand, stratigraphy and soil characteristics can only be accurately determined by means of a site-specific subsurface evaluation combined with appropriate laboratory analysis of representative samples of onsite soils.

Liquefaction occurs when dynamic loading of a saturated sand or silt causes pore-water pressures to increase to levels where grain-to-grain contact is lost and material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common manifestation of liquefaction is the formation of sand boils – short-lived fountains of soil and water that emerge from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface.

In view of the recommended grading and shallow bedrock materials that underlie the site, the potential for manifestation of liquefaction induced features or significant dynamic settlement is considered negligible.

CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical engineering and engineering geologic point of view, the subject property is considered suitable for the proposed residential development provided the following conclusions and recommendations are incorporated into the design criteria and project specifications.

Earthwork

General Earthwork Recommendations

Earthwork should be performed in accordance with the applicable provisions of the 2019 CBC. Grading should also be performed in accordance with the following site-specific recommendations prepared by Petra based on the proposed residential development of the site.

Geotechnical Observations and Testing

Prior to the start of earthwork, a meeting should be held at the site with the owner, contractor and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Earthwork, which in this instance will generally entail removal and re-compaction of the near surface soils, should be accomplished under full-time observation and testing of the geotechnical consultant. A representative of the project geotechnical consultant should be present onsite during all earthwork operations to document proper placement and compaction of fills, as well as to document compliance with the other recommendations presented herein.

Clearing and Grubbing

Several residential structures and driveway/flatwork areas are located within the site. The possibility exists that underground structures such as foundations, pipes, utility lines, seepage pits, leach lines or other structures may be found below current grades. Additionally, the majority of the property has a light to occasionally moderate amount of vegetation cover and numerous mature trees. All surficial or buried vegetation, trees, and stumps (including the root ball), miscellaneous debris and/or other deleterious materials will require clearing and hauling offsite. It is anticipated that buried roots and/or any miscellaneous debris will need to be removed from the engineered fills by hand (root pickers) during grading operations.

The project geotechnical consultant should provide periodic observation and testing services during clearing and grubbing operations to document compliance with the above recommendations. In addition, should any unusual or adverse soil conditions be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations, as warranted.

Excavation Characteristics

Based on the results of our exploratory trenches within the northwest portion of the site, fills and surficial native soil deposits, including older alluvium and highly weathered bedrock, are expected to be readily

excavatable with conventional earthmoving equipment throughout the majority of the site. Our limited assessment of bedrock excavation characteristics is discussed below.

- The subject site is underlain by highly weathered to very hard, granitic bedrock which locally forms outcrops extending to several feet above the surrounding terrain within the northwest portion of the site. Exploratory trenches (T-1 and T-2) were excavated with a rubber tire backhoe in this area presumed to have hard bedrock at a relatively shallow depth, and which may be an issue during the construction phase of the project.

- Based on the data obtained in the area where the trenches were excavated the granitic rock encountered is anticipated to be rippable utilizing a D-9 dozer or equivalent, with a single shank in the center slot. However, isolated areas of very hard and/or unrippable rock maybe encountered within this portion of the site.

- The isolated areas of very hard granitic bedrock, if encountered, may require other techniques of removal. Subsurface boulders, floaters or corestones occur unpredictably at depth. Consequently, hoe rams, chemical rock braking and/or blasting may be required to remove large subsurface boulders that may be encountered in some cut areas.

- Where bedrock is exposed at finish grade, wheel trenchers and small backhoes may experience a moderate to very high degree of difficulty during excavation of footing and/or utility trenches, particularly where excavations in cut areas will exceed a depth of approximately 8 to 12 feet. Consequently, the use of jack hammers and/or hoe rams may locally be necessary to facilitate excavation of trenches in hard bedrock areas. Therefore, where hard, relatively resistant bedrock is exposed at finish grade within building pads and street areas, consideration should be given to capping cut lots exposing bedrock with at least 4 feet at back and 5 feet at front of lots with compacted fill and undercutting street areas to the depth of at least one foot below the deepest utility with large earthmoving equipment during grading.

Ground Preparation – Unsuitable Soil Removals

Based on the earth materials encountered within the site, surficial soils (i.e. existing fills, loose alluvium) are considered unsuitable for support of structures in their existing state, and therefore should be removed and recompacted, in areas proposed for settlement sensitive improvements. In areas where structures are to be supported by conventional shallow slab-on-grade foundations, spread footings, and/or post-tension foundations the existing ground should be over-excavated to depths that expose competent materials exhibiting an in-place relative compaction of 85 percent or more, based on ASTM Test Method D 1557.

Therefore, the required depths of remedial removals are anticipated to vary from approximately 3 to 5 feet. A minimum of 3 feet of compacted fill should cap all building pads. The horizontal limits of over-excavation should extend to a minimum distance of 5 feet beyond the proposed perimeter foundation lines or to a horizontal distance equal to the depth of remedial removals, whichever is greater.

All lots should be evaluated for shallow-to-deep-fill transitions. The areas of shallow fill that are less than one-half the depth of the deepest fill to reduce the potential for excessive differential settlement.

Due to the variability of the near surface earth materials that underlie the project site, the required depths of over-excavation will have to be determined during grading on a case-by-case basis. Therefore, prior to placing compacted fill, the exposed bottom surfaces in all over-excavated areas should be observed and approved by the project geotechnical consultant. Following this approval, the exposed bottom surfaces should be scarified to a depth of approximately 6 to 8 inches, watered as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then processed to a relative compaction of 90 percent or more based on ASTM D 1557.

Fill Placement and Testing

All fills should be placed in lifts not exceeding 8 inches in thickness, watered as necessary to achieve moisture contents that are equal to, or slightly above optimum moisture content, and then compacted to a minimum relative compaction of 90 percent or more. Each fill lift should be treated in a similar manner. Subsequent lifts should not be placed until the preceding lift has been tested and approved by the project geotechnical consultant. 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.

Import Soils for Grading

We assume the site will be designed to grade to balance earthwork volumes and that import soils will not be needed to achieve final design grades; however, if needed, any import soils should be free of deleterious materials, oversize rock and any hazardous materials. The soils should also be non-expansive (i.e. Expansion Index (EI) 20 or less) and essentially non-corrosive and approved by the project geotechnical consultant *prior* to being brought onsite. The geotechnical consultant should inspect the potential borrow site and conduct testing of the soil at least three days before the commencement of import operations.

Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Accordingly, it is estimated that a shrinkage factor on the order of approximately 5 to 10 percent will occur when near surface onsite earth materials are excavated and placed as compacted fill.

Subsidence from scarification and re-compaction of exposed bottom surfaces in over-excavated areas is expected to be on the order of approximately 0.05 to 0.10 feet.

The above estimates of shrinkage and subsidence are intended as aids for the civil engineer and project planners in determining earthwork quantities. However, these values should not be considered as absolute values and some contingencies should be made for balancing earthwork quantities on the basis of actual shrinkage and subsidence that occur during grading.

Temporary Excavations

Temporary excavations varying up to a height of 10 feet below existing grades may be required to accommodate the recommended overexcavation of unsuitable materials. Based on the physical properties of the onsite soils, temporary excavations which are constructed exceeding 4 feet in height should be cut back to a ratio of 1:1 (h:v) or flatter for the duration of the overexcavation of unsuitable soil material and replacement as compacted fill, as well as placement of underground utilities. However, the temporary excavations should be observed by a representative of the project geotechnical consultant for evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary. Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and/or storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures on adjacent properties and weather conditions at the time of construction. 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.

Preliminary Foundation Design Considerations

Foundation Systems

Either conventional or post-tension slab-on-ground foundation systems are deemed to be suitable for the proposed residences, providing the site is prepared as recommended in this report. Recommendations for the design and construction of both options are presented herein.

Allowable Soil Bearing Capacities

Pad Footings

An allowable soil bearing capacity of 1,500 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 2,500

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 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 2,500 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.

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 ¾ inch. Differential settlement is expected to be less than 1/2 inch over a horizontal span of 30 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 250 pounds per square foot per foot of depth, to a maximum value of 2,500 pounds per square foot, may be used to determine lateral bearing resistance for footings. In addition, a coefficient of friction of 0.30 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. 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 prior evaluation indicate that these materials 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 and 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 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 Slab 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.

Footings

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 WWF (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. Living 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. Garage floor slabs should be a minimum 4 inches thick and reinforced in a similar manner as living area floor slabs. Garage slabs should also be poured separately from adjacent wall footings with a positive separation maintained using ¾-inch-minimum felt expansion joint material. To control the propagation of shrinkage cracks, garage floor slabs should be quartered with weakened plane joints. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that provided in Item 2 above, should the garage slab be overlain with moisture sensitive floor covering.

4. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage 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.

5. 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.

Post-Tensioned Slabs on-Grade System (Optional)

In consideration of the expansion index of less than 20, as predominantly exhibited by onsite soils, any rational and appropriate procedure may be chosen by the project structural engineer for the design of post-tensioned slabs on-grade. Should the design engineer choose to follow the latest Code-adopted edition of the procedure published by the Post-Tensioning Institute (PTI DC 10.5), the following minimum design criteria are provided Table 2, below.

TABLE 2

Presumptive Post-Tensioned Slab on-Grade Design Parameters for PTI Procedure

Soil Information	
Approximate Depth of Constant Suction, feet	9
Approximate Soil Suction, pF	3.9
Inferred Thornthwaite Index:	-20
Average Edge Moisture Variation Distance, e_m in feet: Center Lift Edge Lift	 9.0 5.0
Anticipated Swell, y_m in inches: Center Lift Edge Lift	 0.35 0.65

Modulus of Subgrade Reaction

The modulus of subgrade reaction for design of load bearing elements depends on the size of the element and soil-structure interaction. However, as a first level of approximation, this value may be assumed to be 125 pounds per cubic inch.

Minimum Design Recommendations

The soil values provided above may be utilized by the project structural engineer to design post-tensioned slabs on-ground in accordance with Section 1808.6.2 of the 2019 CBC and the PTI publication. Thicker floor slabs and larger footing sizes may be required for structural reasons and should govern the design if more restrictive than the minimum recommendations provided below:

1. Exterior continuous footings for one- and two-story structures should be founded at a minimum depth of 12 inches below the lowest adjacent finished ground surface. Interior footings may be founded at a minimum depth of 10 inches below the tops 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. Alternatively, post-tensioned tendons may be utilized in the perimeter continuous footings in lieu of the reinforcement bars.

3. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in 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 thickness of the floor slabs should be determined by the project structural engineer with consideration given to the expansion index of the onsite soils; however; we recommend that a minimum slab thickness of 4 inches be considered.

7. As an alternative to designing 4-inch-thick post-tensioned slabs with perimeter footings as described in Items 1 and 2 above, the structural engineer may design the foundation system using a thickened slab design. The minimum thickness of this uniformly thick slab should be 7.5 inches. The engineer in charge of post-tensioned slab design may also opt to use any combination of slab

thickness and footing embedment depth as deemed appropriate based on their engineering experience and judgment.

8. Living 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.

9. Garage floor slabs should be designed in a similar manner as living area floor slabs. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that provided in Item 6 above, should the garage slab be overlain with moisture sensitive floor covering.

10. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage 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.

11. 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>Footing Observations</u>

Foundation footing trenches should be observed by the project geotechnical consultant to document into competent bearing-soils. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square; prior to placing concrete, all loose, sloughed or softened soils and/or construction debris should be removed. Excavated

soils derived from footing and utility trench excavations should not be placed in slab-on-grade areas unless the soils are compacted to a relative compaction of 90 percent or more.

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 common indicators associated with soil corrosivity include water-soluble sulfate and chloride levels, pH (a measure of acidity), and minimum electrical resistivity. Test methodology and results are presented in Appendix C.

> *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.*

In general, a soil's water-soluble sulfate levels and pH relate to the potential for concrete degradation; water-soluble chloride in soils impact ferrous metals embedded or encased in concrete, e.g., reinforcing steel; and electrical resistivity is a measure of a soil's corrosion potential to a variety of buried metals used in the building industry, such as copper tubing and cast or ductile iron pipes. Table 2, below, presents a single value of individual test results with an interpretation of current code indicators and guidelines that are commonly used in this industry. The table includes the code-related classifications of the soils as they relate to the various tests, as well as a general recommendation for possible mitigation measures in view of the potential adverse impact on various components of the proposed structures in direct contact with site soils. The guidelines provided herein should be evaluated and confirmed, or modified, in their entirety by the project structural engineer, corrosion engineer and/or the contractor responsible for concrete placement for structural concrete used in exterior and interior footings, interior slabs on-ground, garage slabs, wall foundations and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

TABLE 3

Soil Corrosivity Screening Results

Test	Test Results	Classification	General Recommendations
Soluble Sulfate (Cal 417)	0.0117 %	S0[1] **Not Applicable**	Type II cement; min. f_c' = 2,500 psi; no water/cement ratio restrictions.
pH (Cal 643)	6.87	Neutral	No Special Recommendation
Soluble Chloride (Cal 422)	180 ppm	C1[2] C2[4]	Residence: No special recommendations Pools/Decking: water/cement ratio 0.40, f'c = 5,000 psi
Resistivity (Cal 643)	1,800 ohm-cm	Highly Corrosive[3]	Protective wrapping/coating of buried pipes; corrosion resistant materials; or cathodic protection

Notes:
1. ACI 318-14, Section 19.3
2. ACI 318-14, Section 19.3
3. Pierre R. Roberge, "Handbook of Corrosion Engineering"
4. Exposure classification C2 applies specifically to swimming pools and appurtenant concrete elements

Post-Grading Recommendations

Laboratory Testing

Additional sampling and laboratory testing upon completion of rough grading operations is recommended to evaluate expansion and general corrosion potential for the purposes of providing final foundation design recommendations.

Site Drainage

Surface drainage systems consisting of sloping concrete flatwork, graded earth swales and/or an underground area drain system are anticipated to be constructed to collect and direct all surface waters to the adjacent streets and storm drain facilities. In addition, the ground surface around the proposed buildings should be sloped at a positive gradient away from the structures. The purpose of the precise grading is to prevent ponding of surface water within the level areas of the site and against building foundations and associated site improvements. The drainage systems should be properly maintained throughout the life of the proposed development.

Utility Trenches

Utility-trench backfill within street right-of-ways, utility easements, under sidewalks, driveways and building-floor slabs should be compacted to a relative compaction of 90 percent or more. Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative to document adequate compaction.

Utility-trench sidewalls deeper than about 4 feet should be laid back at a ratio of 1:1 (h:v) or flatter or shored. A trench box may be used in lieu of shoring. If shoring is anticipated, the project geotechnical consultant should be contacted to provide design parameters.

For trenches with vertical walls, backfill should be placed in approximately 1- to 2-foot thick loose lifts and then mechanically compacted with a hydra-hammer, pneumatic tampers or similar compaction equipment. For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch-thick loose lifts and then compacted by rolling with a sheepsfoot tamper or similar equipment.

Where utility trenches are proposed in a direction that parallels any building footing (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

Plan Review and Construction Services

This report has been prepared for the exclusive use of the client to assist the project team in the design of the proposed development. It is recommended that Petra be engaged to review the final-design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted and are incorporated into the project grading plans and specifications. If Petra is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that Petra be retained to provide soil-engineering services during grading and construction of the excavation and foundation preparation phases of the work. This is to observe compliance with the design, specifications, or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project design concept changes significantly (e.g., structural loads or types), we should be retained to review our original design recommendations and their applicability to the revised construction concept. If conditions are encountered during construction that appears to be different than those indicated in this report, this office should be notified immediately. If this is the case, design and construction revisions may be required.

LIMITATIONS

This report is based on the project, as described, and the preliminary geologic/geotechnical field data obtained from the prior investigation (NCCEI,2004) and our limited field tests performed at the locations shown. The materials encountered on the project site and utilized in our laboratory evaluation are believed representative of the total area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials and groundwater levels can vary in characteristics between points of excavation, both laterally and vertically.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty. The findings, conclusions and opinions contained in this report are to be considered tentative only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and Petra or the undersigned professionals assume no responsibility for its use. In addition, this report should be reviewed and updated after a period of 1 year or if the site ownership or project concept changes from that described herein.

The professional opinions contained herein have been derived in accordance with current standards of practice and no warranty is expressed or implied. This report 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.

We sincerely appreciate this opportunity to be of service. Please do not hesitate to call the undersigned if you have any questions regarding this report.

Respectfully submitted,
PETRA GEOSCIENCES, INC.

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4/15/2021

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FIGURES

N

Base Map Reference: Google Earth (2021) Map

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COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

SITE LOCATION MAP

Parcel F
Escondido, California

PETRA GEOSCIENCES INC.

DATE: April, 2021
J.N.: 20-439

Figure 1

EXPLANATION

P-1 Approximate Location of Percolation Boring By Petra

T-2 Approximate Location of Petra Exploratory Trenches By Petra

TP-4 Approximate Location of Test Pit By North County Compaction Engineering, Inc

N

Base Map Reference: Google Earth (2021) Map

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PERCOLATION LOCATION MAP

Parcel F
Escondido, California

PETRA GEOSCIENCES INC

DATE: April, 2021

J.N.: 20-439

Figure 2

APPENDIX A

EXPLORATION LOGS

Key to Soil and Bedrock Symbols and Terms

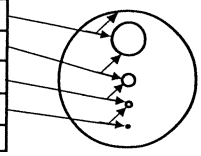
PETRA

Unified Soil Classification System

Coarse-grained Soils > 1/2 of materials is larger than #200 sieve (The No. 200 U.S. Standard Sieve is about the smallest particle visible to the naked eye)	GRAVELS more than half of coarse fraction is larger than #4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty Gravels, poorly-graded gravel-sand-silt mixtures
			GC	Clayey Gravels, poorly-graded gravel-sand-clay mixtures
	SANDS more than half of coarse fraction is smaller than #4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sands, little or no fines
		Sands with fines	SM	Silty Sands, poorly-graded sand-gravel-silt mixtures
			SC	Clayey Sands, poorly-graded sand-gravel-clay mixtures
Fine-grained Soils > 1/2 of materials is smaller than #200 sieve	SILTS & CLAYS Liquid Limit Less Than 50		ML	Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts & clays of low plasticity
	SILTS & CLAYS Liquid Limit Greater Than 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silt
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium-to-high plasticity
Highly Organic Soils			PT	Peat, humus swamp soils with high organic content

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized to
Fines		Passing #200	<0.0029"	Flour-sized and smaller



Laboratory Test Abbreviations

MAX	Maximum Dry Density	MA	Mechanical (Particle Size) Analysis
EXP	Expansion Potential	AT	Atterberg Limits
SO4	Soluble Sulfate Content	#200	#200 Screen Wash
RES	Resistivity	DSU	Direct Shear (Undisturbed Sample)
pH	Acidity	DSR	Direct Shear (Remolded Sample)
CON	Consolidation	HYD	Hydrometer Analysis
SW	Swell	SE	Sand Equivalent
CL	Chloride Content	OC	Organic Content
RV	R-Value	COMP	Mortar Cylinder Compression

Modifiers

Trace	< 1 %
Few	1 - 5%
Some	5 - 12 %
Numerous	12 - 20 %

Sampler and Symbol Descriptions

- Approximate Depth of Seepage
- Approximate Depth of Standing Groundwater
- Modified California Split Spoon Sample
- Standard Penetration Test
- Bulk Sample
- Shelby Tube
- No Recovery in Sampler

Bedrock Hardness

Soft	Can be crushed and granulated by hand; "soil like" and structureless
Moderately Hard	Can be grooved with fingernails; gouged easily with butter knife; crumbles under light hammer blows
Hard	Cannot break by hand; can be grooved with a sharp knife; breaks with a moderate hammer blow
Very Hard	Sharp knife leaves scratch; chips with repeated hammer blows

Notes:

Blows Per Foot: Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil or bedrock at the bottom of the hole with a standard (140 lb.) hammer dropping a standard 30 inches unless noted otherwise in Log Notes. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586

TEST PIT LOG

Project:	**Parcel F**	Boring No.:	**TP-1**
Location:	**Escondido**	Elevation:	**±798'**
Job No.: **20-439**	Client: **Escondido North, LLC**	Date:	**2/25/2021**
Drill Method: **Backhoe**	Driving Weight: **Nopt Applicable**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af)** Silty Sand (SM): Dark brown, slightly moist to moist, fine-grained sand.							
		BEDROCK - Monzogranite (Kmm) Yellowish-brown and gray, slightly moist, fine- to coarse-grained, very hard, massive, slightly weathered.							
5		Total Depth= 3.5' Refusal due to hard bedrock No groundwater encountered Test Pit backfilled with cuttings.							
10									
15									
20									
25									
30									
35									

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TEST PIT LOG

Project:	**Parcel F**	Boring No.:	**TP-2**
Location:	**Escondido**	Elevation:	**±798'**
Job No.: **20-439**	Client: **Escondido North, LLC**	Date:	**2/25/2021**
Drill Method: **Backhoe**	Driving Weight: **Not Applicable**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af)** Silty Sand (SM): Dark brown, dry, medium-dense, fine-grained, few roots, several pvc sprinkler pipes encountered.							
		OLDER ALLUVIUM (Qoal) Silty Sand (SM): Dark yellowish-brown to brown, moist, fine- to medium-grained sand.							
5		**BEDROCK - Monzogranite (Kmm)** Pale yellowish-brown, moist, medium- to coarse-grained, moderately hard, massive, highly weathered.							
10		Total Depth= 9.5' Still excavatable No groundwater encountered Test Pit backfilled with cuttings.							
15									
20									
25									
30									
35									

PLATE A-3

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EXPLORATION LOG

Project:	**Parcel F**		Boring No.: **P-1**
Location:	**Escondido**		Elevation: **±764'**
Job No.:	**20-439**	Client: **Escondido North, LLC**	Date: **2/24/2021**
Drill Method:	**8" Hollow Stem Auger**	Driving Weight: **140lbs/30"**	Logged By: **KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af)** Sand with Silt (SP-SM): Brown, moist, medium-dense, fine-grained.							
		OLDER ALLUVIUM (Qoal) Silty Sand (SM): Light brown, moist, medium-dense, fine- to medium-grained.							MAX, EI, SO4, CL, RES, pH
5		Same as above.		7 17 20					
		BEDROCK - Monzogranite (Kmm) Dark orangish-brown, moist, fine- to medium-grained, moderately hard, highly weathered, trace clay.		17 20 30					
10		Total Depth= 10' No groundwater encountered Infiltration test installed within boring using 3 inch perforated pipe and gravel After test had concluded, boring backfilled with cuttings.							
15									
20									
25									
30									

PLATE A-1

Petra Geosciences, Inc.

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

LABORATORY TESTING

Associated with the subsurface exploration was the collection of bulk and relatively undisturbed samples of soil materials for laboratory testing. The relatively undisturbed samples were obtained using a 3-inch, outside-diameter, modified California split-spoon soil sampler lined with 1-inch-high brass or stainless-steel rings. The driven ring samples were placed in sealed containers and all ring and bulk samples transported to our laboratory located at 1251 W. Pomona Road, Unit #103, Corona, CA 92882, for testing.

Our laboratory testing capabilities include Soil Classifications, Moisture Content and In-Situ Moisture Content and Dry Unit Weight, Organic Content, Laboratory Maximum Dry Unit Weight and Optimum Moisture Content, Expansion Index, Corrosivity Screening (Soluble Sulfate and Chloride Content, pH, Resistivity), Atterberg Limits, Grain Size Distribution, Direct Shear, Consolidation and Permeability; all in accordance with the latest procedures of American Society for Testing and Materials (ASTM) and California Department of Transportation (Caltrans).

To evaluate the engineering properties of site soils, laboratory testing was performed on selected samples of soil considered representative of those encountered. Appropriate tests were assigned by the project engineer and geologist based on project plans and specifications including the level of anticipated loads, when available, and subsurface stratigraphy. Test results were reviewed by the laboratory manager and engineer-in-charge of the laboratory or his qualified designee for completeness and accuracy. A description of laboratory test procedures and summaries of the test data are presented in the following pages.

LABORATORY TEST PROCEDURES

Laboratory Maximum Dry Unit Weight and Optimum Moisture Content

The maximum dry unit weight and optimum moisture content of the on-site soils were determined for selected bulk samples in accordance with latest version of Method B of ASTM D 1557. The results of these tests are presented on Plate B-1.

Expansion Index

Expansion index tests were performed on selected bulk samples of the on-site soils in accordance with the latest version of Test Method ASTM D 4829. The test results are presented on Plate B-1.

Corrosivity Screening

Chemical and electrical analyses were performed on selected bulk samples of onsite soils to determine their soluble sulfate content, chloride content, pH (acidity) and minimum electrical resistivity. These tests were performed in accordance with the latest versions of California Test Method Nos. CTM 417 (sulfate), CTM 422 (chloride), and CTM 643 (pH and resistivity) respectively. The results of these tests are included on Plate B-1.

LABORATORY DATA SUMMARY													
Boring Number	Sample Depth (ft)	Soil Description	Compaction[1]		Expansion[2]		Atterberg Limits[3]			Soluble Sulfate Content[4] (%)	Chloride Content[5] (ppm)	pH[6]	Minimum Resistivity[6] (Ohm-cm)
			Max. Dry Density (pcf)	Optimum Moisture (%)	Index	Potential	LL	PL	PI				
P-1	0-5	Silty Sand	127.0	8.0	0	Very Low	-	-	-	0.0117	180	6.87	1,800

Test Procedures:
[1] Per ASTM Test Method ASTM D 1557
[2] Per ASTM Test Method ASTM D 4829
[3] Per ASTM Test Method ASTM D 4318
[4] Per California Test Method CTM 417
[5] Per California Test Method CTM 422
[6] Per California Test Method CTM 643

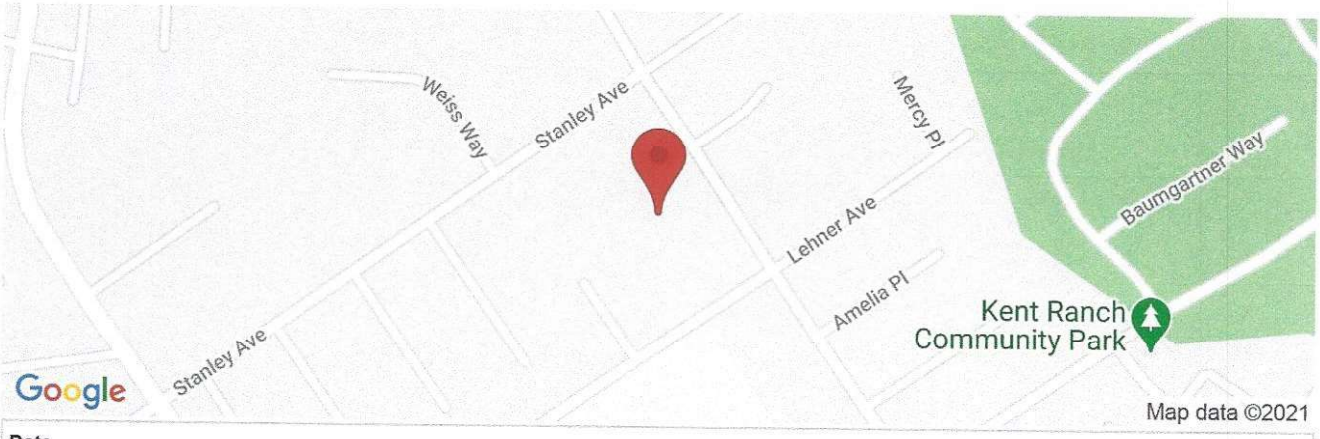
APPENDIX C

SEISMIC DESIGN DATA

OSHPD

Parcel F (20-439)

Latitude, Longitude: 33.1599, -117.0809



Date	3/29/2021, 7:35:00 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	0.938	MCE_R ground motion. (for 0.2 second period)
S_1	0.341	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.126	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.75	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.406	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.487	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.938	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.02	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.341	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.37	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.919	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.921	Mapped value of the risk coefficient at a period of 1 s

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APPENDIX D

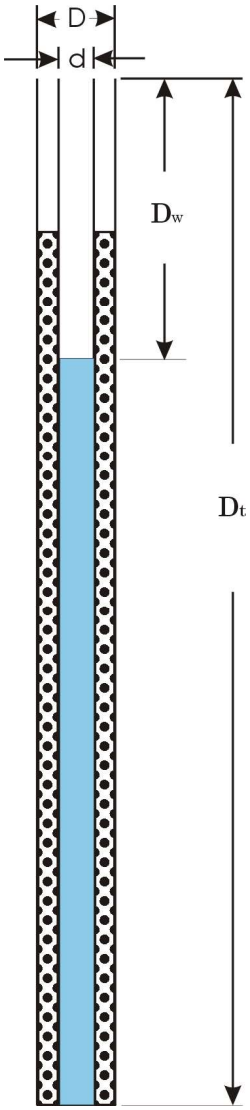
PERCOLATION TEST DATA

Test Number: P-1

Shallow Percolation Test Method

Total Depth of Boring, D_t (ft): 8.3
Diameter of Hole, D (in): 8
Diameter of Pipe, d (in): 3
Agg. Correction (% Voids): 42
Pre-soak depth (ft): 3.5

Time Interval (min)	Depth to Water Surface D_w (ft)		Change in Head (in)	Perc. Rate (min/in)	Perc. Rate (gal/day/ft^2)
	1st Reading	2nd Reading			
30	5.00	5.17	2.04	14.71	1.51
30	5.17	5.33	1.92	15.63	1.49
30	5.33	5.47	1.68	17.86	1.37
30	5.47	5.60	1.56	19.23	1.33
30	4.68	4.78	1.20	25.00	0.80
30	4.78	4.86	0.96	31.25	0.66
30	4.86	4.94	0.96	31.25	0.67
30	4.94	5.03	1.08	27.78	0.78
30	5.03	5.11	0.96	31.25	0.71
30	5.00	5.09	1.08	27.78	0.79
30	5.02	5.10	0.96	31.25	0.71
30	5.05	5.13	0.96	31.25	0.71



Percolation Rate: 31.25 Minutes/Inch
0.71 gal/day/ft^2

Infiltration Rate: 0.09 Inches/Hour*
(Porchet Method)

where Infiltration Rate, $I_t = \Delta H\ (60r) / \Delta t\ (r + 2H_{avg})$

$r = D / 2$

$H_o = D_t - D_o$

$H_f = D_t - D_f$

$\Delta H = \Delta D = H_o - H_f$

$H_{avg} = (H_o + H_f) / 2$

*Raw Number, Does Not Include a Factor of Safety

Reference: RCFCWCD, Design Handbook for LIDBMP, dated September, 2011 **or**
SARWQCB, Technical Guidance Document Appendix VII, dated December 20, 2013 **or**
CofSBASP, Technical Guidance Document Appendix D, dated May 19, 2011 **or**

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PERCOLATION TEST SUMMARY

Parcel F
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Figure 1