PETRA GEOSCIENCES INC.

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UPDATED GEOTECHNICAL DUE-DILIGENCE ASSESSMENT
PARCEL H, ASSESSOR PARCEL NUMBERS (APN) 224-141-23-00 AND 224-141-25-00
NORTHWEST CORNER OF STANLEY AVENUE AND CONWAY DRIVE
CITY OF ESCONDIDO, SAN DIEGO COUNTY, CALIFORNIA

ESCONDIDO NORTH, LLC

April 7, 2021
J.N. 20-437

ENGINEERS + GEOLOGISTS + ENVIRONMENTAL SCIENTISTS

April 7, 2021
J.N. 20-437

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

Attention: Mr. John Kaye

Subject: Updated Geotechnical Due-Diligence Assessment; Parcel H, Assessor Parcel Numbers 224-141-23-00 and 224-141-25-00, Northwest Corner of Stanley Avenue and Conway Drive, City of Escondido, San Diego County, California

Dear Mr. Kaye:

Petra Geosciences, Inc. (Petra) is submitting herewith our updated geotechnical due-diligence assessment report for Parcel H, APNs 224-141-23-00 and 224-141-25-00, located adjacent the northwest corner of Stanley 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. 20-437P dated February 5, 2021. This update report includes shallow percolation testing to evaluate the infiltration characteristics of the soils in the area of the proposed WQMP basin, as well as updated seismic and foundation design parameters for the subject site that comply with the current 2019 California Building Code (CBC). The updated report presented herein reiterates the findings, conclusions and recommendations of the previous due-diligence evaluation, dated April 30, 2014, except where superseded by updated information and supplemental data.

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

Grayson R. Walker
Principal Engineer
GE 871

JC/GRW/lv

TABLE OF CONTENTS

Page

ATTACHMENTS

FIGURE 1 – SITE LOCATION MAP

FIGURE 2 – EXPLORATION LOCATION MAP

APPENDIX A – BORING LOGS

APPENDIX B – LABORATORY TEST CRITERIA/LABORATORY TEST DATA

APPENDIX C – SEISMIC DESIGN DATA

APPENDIX D – PERCOLATION TEST DATA

UPDATED GEOTECHNICAL DUE-DILIGENCE ASSESSMENT
PARCEL H, APNs 224-141-23-00 and 224-141-25-00
NORTHWEST CORNER OF STANLEY AVENUE AND CONWAY DRIVE
CITY OF ESCONDIDO, SAN DIEGO COUNTY, CALIFORNIA

INTRODUCTION

Petra Geosciences, Inc. (Petra) is presenting herewith the results of our geotechnical due-diligence assessment for the proposed development of Parcel H, APNs 224-141-23-00; and 224-141-25-00, located adjacent the northwest corner of Stanley 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. The updated report presented herein reiterates the findings, conclusions and recommendations of the previous due-diligence evaluation, dated April 30, 2014, except where superseded by updated information and supplemental data.

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.

- Drilled 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 D2488-90.

- Preformed appropriate laboratory testing of representative samples (bulk and undisturbed) obtained from the exploratory borings to determine their engineering properties.

- 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 Stanley Avenue and Conway Drive in the City of Escondido, San Diego County, California. The site, which encompasses approximately 6.8 acres, is an irregular-shaped property comprised of two parcels of land identified as APN's 224-141-23-00 and 224-141-25-00. Topographically, site elevations ranged from approximately 818 feet above mean sea level (msl) within the south portion of the site to approximately 773 feet msl within the drainage in the central portion of the site. The location of the site is shown on Figure 1.

Four (4) existing residential structures were observed onsite during our site reconnaissance. The residential structures are one-story single-family residences with three located within the north-northwest portion of the site adjacent Conway Drive and the forth located in the south 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 Petra (Petra, 2014). In addition, we reviewed the Pasco Laret Suiter and Associates, Conway+Stanley 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+Stanley 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+Stanley Option A Site Plan the tract will consist of a cul-de-sac street (street A), seventeen (17) residential lots, a water quality basin and a public utility easement. The F&H Density bonus Site Plan depicts the tract to consist of a cul-de-sac street (Street A), twenty (20) residential lots (Lot 13 is for 8 low-income units), a water quality basin, and a private storm drain 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

Petra (2014) advanced five (5) exploratory test pits (TP-1 through TP-5) to a maximum depth of approximately 12 feet below existing grades, and/or practical refusal. Based on the test pits advanced, it was reported that up to three (3) feet of loose to medium dense topsoil was observed overlying the majority of the site. Colluvium was observed within test pit five (5) and consists of fine to coarse silty sands that were yellowish brown, dry, medium dense and slightly to moderately porous. Up to ten (10) feet of dense to very dense older alluvial deposits were encountered within test pits TP-2, -3 and -4 underlying the topsoil within the central portion of the site. Cretaceous-age granitic bedrock was observed within the bottom of each test pit. The granitic rock was reddish brown and gray, hard to very hard and moderately weathered.

The granitic bedrock was observed at varying depths of three (3) feet below the ground surface (bgs) within the north portion of the site to eleven (11) feet bgs within the central portion of the site.

Percolation Boring

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 northwest portion of the property in the general location of the water quality basin "Lot B". 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. The zone consisting of the bottom 5 feet of the borehole 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 0.16 inches per hour. Test data for the percolation test is attached in Appendix D.

Laboratory Testing

The laboratory testing during our previous evaluation of the site (Petra, 2014), included the determination of in-situ dry density and moisture content, in-situ and maximum dry density and in-situ and 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 and the in-situ dry density and moisture content results are presented on the test pit logs (Appendix A).

FINDINGS

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 previous evaluation of the site (Petra, 2014). The earth materials encountered within the exploratory test pits consisted of topsoil, colluvium, older alluvial deposits and Cretaceous age bedrock of the Southern California Batholith. These units, from younger to older, are described below.

Topsoil: Topsoil mantles the majority of the site. These soils were comprised of fine to coarse silty sands that were medium brown and yellowish brown, dry to slightly moist and loose.

Colluvium: Colluvium was observed within test pit five (5) and consists of fine to coarse silty sands that were yellowish brown, dry, medium dense and slightly to moderately porous.

Older Alluvial Deposit (Qoal): Older alluvial deposits were encountered within test pits 2, 3 and 4 underlying the topsoil. These soil deposits were observed to be yellowish brown and reddish brown, dry and dense to very dense. These soils were fine to coarse grained with some clay and slightly to moderately porous with strong cementation.

Granitic Bedrock: Cretaceous-age granitic bedrock was observed within the bottom of each test pit. The granitic rock was reddish brown and gray, hard to very hard and moderately weathered.

Groundwater

The site is located within the Escondido Valley Groundwater Basin, (California Department of Water Resources, [CDWR], 2004). Two historic groundwater well was 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.

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 which has been mapped approximately 12 miles northeast of the site (Kennedy and Tan, 2005).

Strong Ground Motions

The site is located in a seismically active area of southern California and will likely be subjected to very strong seismically related ground shaking over 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 2019 California Building Code (CBC) and the seismic parameters included in the recommendations section herein.

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.

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.

Compressible Soils

A significant geotechnical factor affecting the project site is the presence of near-surface compressible topsoil, colluvium 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.

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 attention of the project geotechnical consultant for corrective recommendations, as warranted.

Ground Preparation – Unsuitable Soil Removals

Based on the earth materials encountered within the exploratory test pits, surficial soils (i.e. topsoil) over a majority of the site 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. Existing fill soils may also be present, particularly in the area of the existing houses. Such fill should also be considered subject to over-excavation and re-compaction. 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 1 to 3 feet. Removal of existing fills may extend to depths of 5 feet or possibly more. A minimum of 5 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 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 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.

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)	-	33.1616	°
Site Longitude (West)	-	-117.0822	°
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)	0.940 (4)	g
S_1 - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(2) (1)	0.342 (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)	1.127 (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)	0.752 (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)	0.407	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)	0.488 (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)	0.301	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.

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.

Building Floor Slabs

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 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.

Footing Observations

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)	0.06 %	S0[1]	No specific requirements
pH (Cal 643)	7.1	Moderately Alkaline	Type I-P (MS) Modified or Type II Modified cement
Soluble Chloride (Cal 422)	105 ppm	C1[2] C2[4]	Residence: No special recommendations Pools/Decking: water/cement ratio 0.40, f'c = 5,000 psi
Resistivity (Cal 643)	2,500 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 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.

4/7/2021

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W:\2020-2025\2020\400\20-437 Escondido North, LLC (Parcel H, Escondido)\Reports\20-437 100 Update Geotechnical Due-Diligence Report.docx

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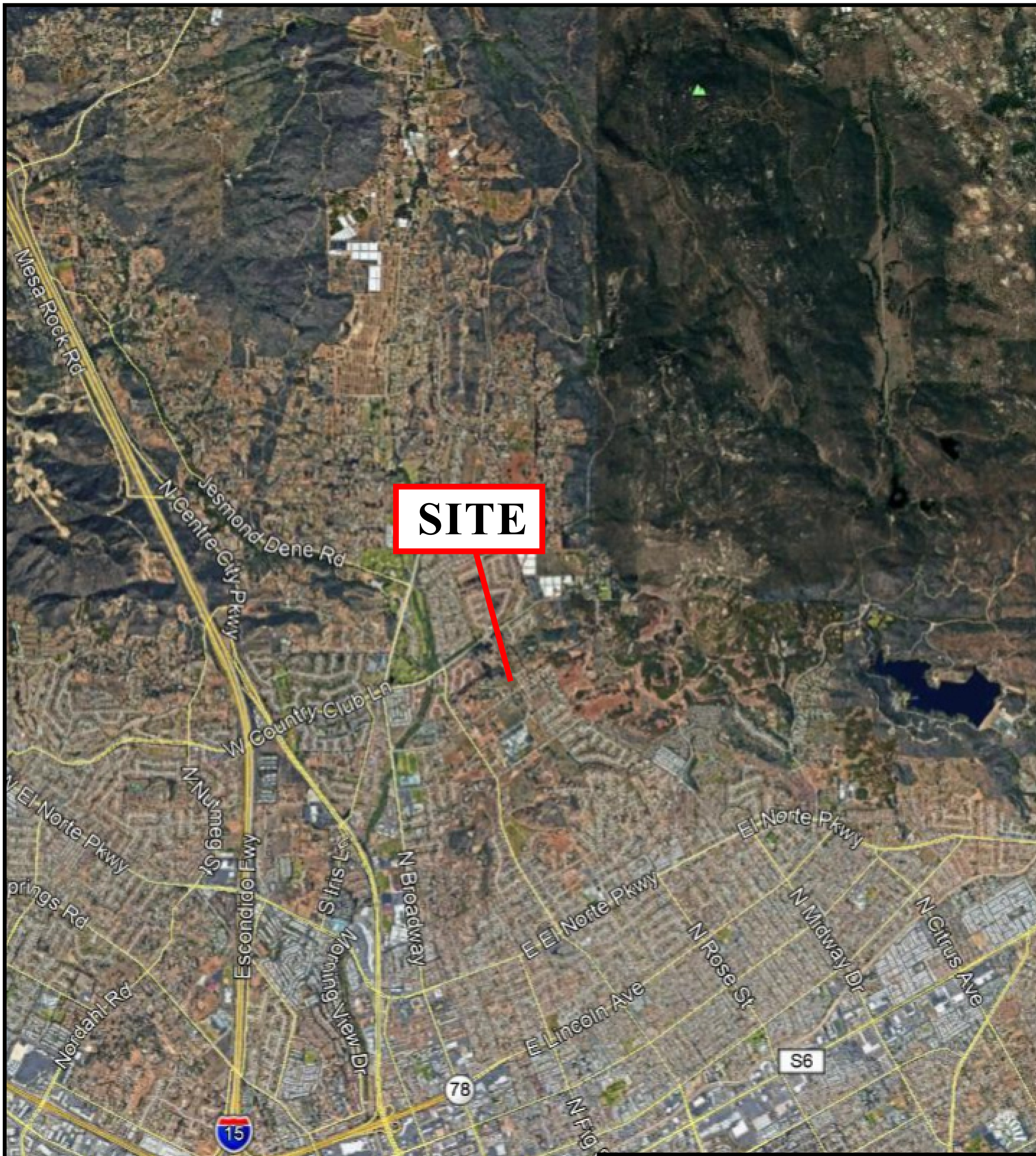
FIGURES



Base Map Reference: Google Earth (2021) Map

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COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

SITE LOCATION MAP

Parcel H
Escondido, California

PETRA GEOSCIENCES INC.	DATE: April 2021	Figure 1
	J.N.: 20-437	

EXPLANATION

Approximate Location of Percolation Boring
P-1

Approximate Location of Exploratory Test Pit (Petra, 2014)
TP-5

N

Base Map Reference: Google Earth (2021) Map

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EXPLORATION LOCATION MAP

Parcel H
Escondido, California

PETRA GEOSCIENCES INC.

DATE: April 2021

J.N.: 20-437

Figure 2

APPENDIX A

EXPLORATION LOGS

Key to Soil and Bedrock Symbols and Terms

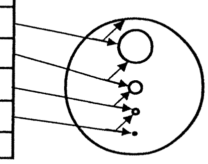
PETRA

Unified Soil Classification System

Major Divisions	Note	Subdivision	Group	Symbol	Description
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

Abbreviation	Test	Abbreviation	Test
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

EXPLORATION LOG

Project: **Parcel H**		Boring No.:	**P-1**
Location: **Escondido**		Elevation:	**±763'**
Job No.: **20-437**	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 foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ALLUVIUM (Qal)** Silty Sand (SM): Grayish-brown, dry, loose, fine- to medium-grained.							
		BEDROCK - Monzogranite (Kmm) Orangish-brown and dark gray, moist, fine- to medium-grained, moderately hard, highly weathered.							
5		Same as above.		10 30 32					
		Same as above.		7 10 12					
10		Total Depth= 10' Infiltration test installed within boring using 3" perforated pipe and gravel After test had concluded, boring backfilled with cuttings.							
15									
20									
25									
30									

Petra Geosciences, Inc.

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

LABORATORY TEST PROCEDURES

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D 2488). The samples were re-examined in the laboratory and the classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Test Pit Logs (Appendix A).

In-Situ Moisture and Density

Moisture content and unit dry density of in-place soils were determined in representative strata. Test data are summarized in the Boring Logs (Appendix A).

Expansion Index

Expansion Index (E.I.) testing was performed on a selected bulk samples of the onsite soils in general accordance with ASTM D 4829. The test results and expansion potentials are presented in Appendix B.

Soil Corrosivity

Chemical analyses were performed on a selected sample to determine concentrations of soluble sulfate and chloride, as well as pH and resistivity. These tests were performed in accordance with California Test Method Nos. 417 (sulfate), 422 (chloride) and 643 (pH and resistivity). Test results are presented in Appendix B.

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**				
TP-3	1-4	Silty Sand	-	-	3	Very Low	-	-	-	0.06	105	7.1	2,500

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

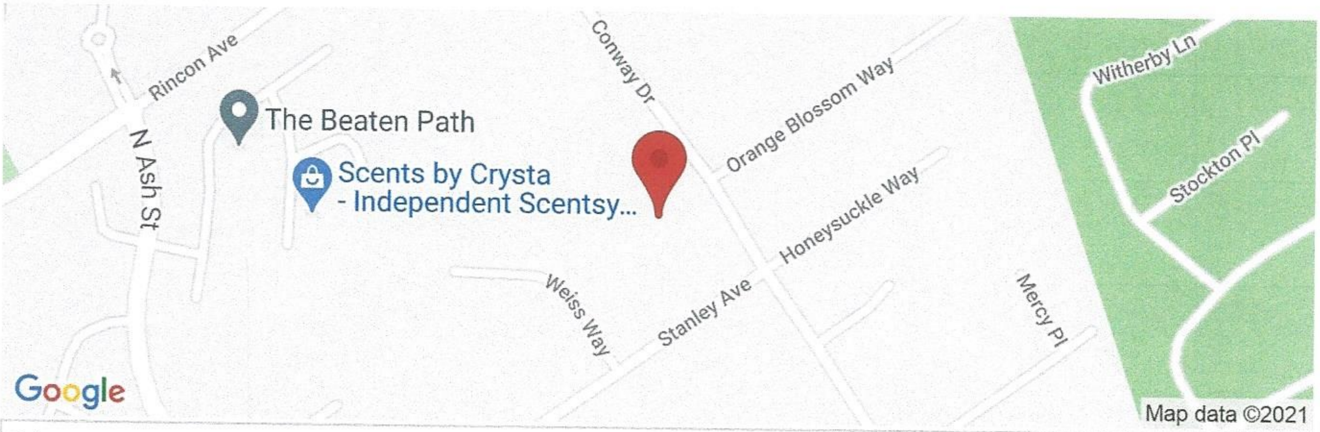
APPENDIX C

SEISMIC DESIGN DATA

OSHPD

Parcel H (20-437)

Latitude, Longitude: 33.1616, -117.0822



Date	3/29/2021, 7:33:31 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	0.94	MCE_R ground motion. (for 0.2 second period)
S_1	0.342	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.127	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.752	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	0.407	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.488	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.94	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.022	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.342	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.371	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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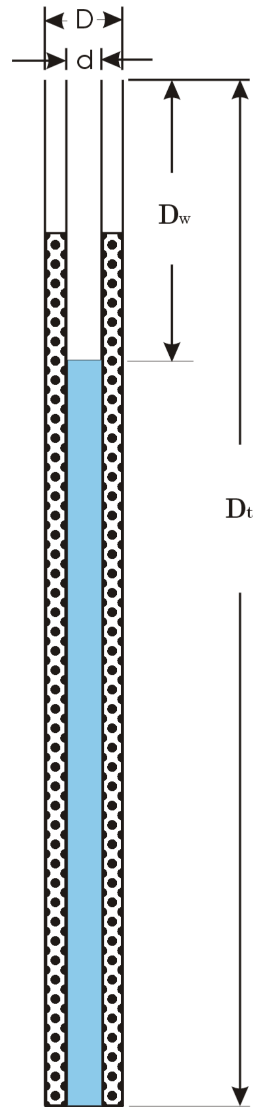
APPENDIX D

PERCOLATION TEST DATA

Test Number: P-1

Shallow Percolation Test Method

Total Depth of Boring, D_t (ft):	8.6
Diameter of Hole, D (in):	8
Diameter of Pipe, d (in):	3
Agg. Correction (% Voids):	42
Pre-soak depth (ft):	3



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	3.30	3.48	2.16	13.89	1.01
30	3.48	3.66	2.16	13.89	1.04
30	3.66	3.90	2.88	10.42	1.44
30	3.90	4.12	2.64	11.36	1.39
30	4.12	4.32	2.40	12.50	1.32
30	4.00	4.20	2.40	12.50	1.29
30	4.05	4.24	2.28	13.16	1.23
30	3.98	4.18	2.40	12.50	1.28
30	4.00	4.19	2.28	13.16	1.22
30	4.03	4.23	2.40	12.50	1.30
30	4.03	4.22	2.28	13.16	1.23
30	4.05	4.24	2.28	13.16	1.23

Percolation Rate: 13.16 Minutes/Inch

1.23 gal/day/ft^2

Infiltration Rate: 0.16 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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PHONE: (714) 549-8921
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

PERCOLATION TEST SUMMARY

Parcel H
Escondido, California

PETRA GEOSCIENCES INC.

DATE: April 2021
J.N.: 20-437

Figure 1