

**GEOTECHNICAL INVESTIGATION
PROPOSED NEW RETAINING WALLS
AND BUILDING/GARAGE IMPROVEMENTS
2155 CRESTVIEW DRIVE, LAGUNA BEACH, CALIFORNIA**

SEASIDE SHORES, LLC

*November 17, 2023
J.N. 23-299*

ENGINEERS + GEOLOGISTS + ENVIRONMENTAL SCIENTISTS

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SEASIDE SHORES, LLC
2155 Crestview Drive
Laguna Beach, California 92651

c/o Shawna Miller, Coulter Milner & Co
5882 Bolsa Avenue, Suite 120
Huntington Beach, California 92649

Subject: Geotechnical Investigation, Proposed New Retaining Walls and Building/Garage Improvements, 2155 Crestview Drive, Laguna Beach, California

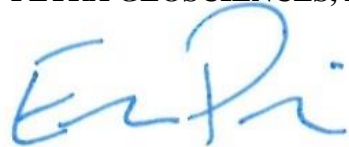
Dear Ms. Miller:

Submitted herewith is our geotechnical investigation report for the property located at 2155 Crestview Drive within the city of Laguna Beach, California. This work was performed in general accordance with the scope of services outlined in our Proposal No. 23-299P, dated September 7, 2023. This report presents the results of our field investigation, laboratory testing, and our engineering and geologic judgment, opinions, conclusions, and recommendations pertaining to geotechnical design aspects of the subject improvements.

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions regarding the contents of this report or require additional information.

Respectfully submitted,

PETRA GEOSCIENCES, INC.



Evan Price
Senior Associate Geologist

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FIGURE 1 – SITE LOCATION MAP

FIGURE 2 – BORING LOCATION MAP

APPENDIX A – EXPLORATION LOGS

APPENDIX B – LABORATORY TEST PROCEDURES / LABORATORY DATA SUMMARY

APPENDIX C – SEISMIC DESIGN ANALYSIS

**GEOTECHNICAL INVESTIGATION
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INTRODUCTION

Petra Geosciences, Inc. (Petra) is submitting herewith the results of our geotechnical investigation of the subject property. The purposes of this investigation were to determine the nature of subsurface soil and bedrock conditions, to evaluate their in-place characteristics, and to then provide geotechnical recommendations with respect to site clearing and grading, and for design and construction of new retaining walls as well as possible building and garage improvements. It should be noted that Petra was not contracted to investigate or analyze the site slope nor to investigate or evaluate the existing residence and associated improvements. Accordingly, this report specifically excludes any findings related to the performance of the existing railroad-tie retaining walls and concrete and stone retaining devices and their possible effects on the surficial stability of the rear yard slope as well as the structural integrity of the existing residential structure and associated improvements.

SITE LOCATION AND DESCRIPTION

The subject property is a hillside lot located at 2155 Crestview Drive, Laguna Beach in the County of Orange. It is the site of a 3-level residence built into the sloping lot with a detached 2-car garage. Overall relief at the site is approximately 50 feet. Existing unreinforced stone and concrete walls covering the upper (northeasterly) portion of the front descending slope of the site and on the western corner of the site are failing. Also, the low height wooden retaining walls along the lower (southwesterly) portion of the slope, below the residence exhibit distress in the form of tilting in the downslope direction. Further, there are failing stone walls near the driveway and near the west end of the residence, behind the balcony. Additional site improvements include concrete and stone walls and facades, concrete walkways, stone stairways, patio slabs, surface drainage controls, and landscape planter areas.

PROPOSED CONSTRUCTION AND GRADING

Based on our conversations with you, it is our understanding that the unreinforced stone and concrete walls covering the upper (northeasterly) portion of the front descending slope of the site and on the western corner of the site that are failing and will be removed and replaced with new retaining walls. Improvements may also be made to existing foundations of the residence and garage. An addition to the front of the existing garage may also be proposed. Although detailed grading plans or wall plans for the project have not yet been prepared, it is our understanding that the proposed new retaining walls in the front yard will be constructed near, or slightly above (upslope) the location of the lower most retaining wall northeasterly adjacent to the existing house and then backfilled to create a larger, level area below the existing driveway

while leaving the surrounding grades and lower retaining wall unchanged. Further, additional retaining walls will be constructed to replace the failing stone walls near the driveway and near the west end of the residence, behind the balcony. Additional site improvements may also include the elimination of additional concrete and stone walls and facades, concrete walkways, stone stairways, patio slabs, surface drainage controls, and landscape planter areas.

SITE RECONNAISSANCE AND LIMITED SUBSURFACE EXPLORATION

Our field investigation for the subject site was performed on September 22, 2023 and consisted of drilling eleven (11) hand-auger borings (HA-1 through HA-11) to depths up to approximately 2 feet deep. Borings HA-1 and HA-8, and HA-9 were drilled around the vicinity of the detached garage. Borings HA-2, HA-3, HA-10, and HA-11 were drilled in the area in the area of the existing retaining walls, between the house and the garage, near where the proposed retaining walls are anticipated to be located. Borings HA-6 and HA-7 were drilled in a planter and landscape area in the rear of the residence, near the existing railroad tie retaining walls. HA-7 encountered shallow refusal on concrete debris found in the planter backfill. Boring HA-4 was attempted to be drilled in the vicinity of the existing stone and concrete retaining wall in the northeastern most corner of the lower patio area but encountered shallow refusal on stone and concrete. Boring HA-5 was drilled in the crawl space in the lower level of the existing residence. Soil materials encountered in the borings were visually classified and logged in general accordance with the Unified Soil Classification System. The approximate locations of the borings are shown on Figure 2 and descriptive exploration logs are presented in Appendix A of this report.

Associated with the subsurface exploration was the collection of bulk and relatively undisturbed samples of the fill soils and bedrock for possible laboratory testing. The relatively undisturbed samples were obtained at various depths using a 3-inch outside diameter, modified California split-spoon sampler lined with 1-inch brass ring liners that was driven into the ground by repetitive drops of a hand operated drop hammer. The central portion of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

LABORATORY TESTING

To evaluate the engineering properties of the soils underlying the subject site, Petra performed several laboratory tests on selected samples considered representative of the materials encountered. Laboratory tests included expansion potential, Atterberg Limits, soluble sulfate and chloride content, pH and minimum resistivity. Moisture content and unit dry density were also determined for in-place soils in representative strata. A summary of laboratory test results is provided in the exploration logs (Appendix A) and in Appendix B of this report.

FINDINGS

Regional Geology

Geologically, the subject parcel lies within the Peninsular Ranges Geomorphic Province. More specifically, the subject parcel is located within the San Joaquin Hills which is a part of the northwestern flank of the Peninsular Ranges. The San Joaquin Hills are a broad gently rolling upland cut by moderately steep sided canyons with marine wave-cut terraces that gently slope toward the southwest and cover the area adjacent to the Pacific coast.

Local Geology and Subsurface Conditions

Based on our limited subsurface investigation, the site is underlain by bedrock belonging to the San Onofre Formation that is overlain by engineered fill to depths of approximately a few inches to 2 feet. The fill consists of clayey sand material that is brown, and dry to moist. This fill material is believed to be associated with the original grading/construction of the existing residence and/or road cut for Crestview Drive. The exact time of this grading is unknown as the original construction of the existing residence and road cut predates available historic aerial photographs dating back to 1931. The bedrock, where encountered in the borings and observed in surficial exposures, generally consists of light yellowish-brown massive clayey to silty sandstone. The bedrock materials were noted to be very hard and slightly to moderately fractured. Local bedding was not observed at the site, although published geologic maps (Edgington & Tan, 1976) show bedding of the San Onofre formation generally dipping to the southwest to southeast at angles ranging from 65 to 77 degrees.

Groundwater and Seepage

No groundwater or seepage was encountered within our exploratory excavations; furthermore, published literature indicates that the depth to historically high groundwater in the area of the subject site is generally considered to be greater than 50 feet below the surface.

Faulting

Based on our review of the referenced geologic maps and literature, no active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) defines an *active fault* as one that “has had surface displacement within Holocene time (about the last 11,000 years).” The main objective of the AP Act is to prevent the construction of dwellings on top of active faults that could displace the ground surface resulting in loss of life and property.

According to the 2014 USGS PSHA (probabilistic seismic hazard assessment) Interactive Deaggregation web site tool and/or the 2010 CGS 'Fault Activity Map of California', the closest active fault to the site is the offshore segment of the Newport-Inglewood fault is located approximately 2.5 miles or 4 kilometers to the southwest of the subject site. The Newport-Inglewood fault consists of a series of parallel and en-echelon, northwest-trending faults and folds extending from the southern edge of the Santa Monica Mountains southeast to the offshore area of Newport Beach. This zone has a history of moderate to high seismic activity and has generated several historic earthquakes greater than magnitude 4.0, including the March 11, 1933 Long Beach earthquake (magnitude 6.3), the October 21, 1941 earthquake (magnitude 4.9), and the June 18, 1944 earthquake (magnitude 4.5).

In addition, the San Joaquin Hills Blind Thrust Fault is believed to underlie the San Joaquin Hills (Grant, et al, 1999) and was incorporated into the State of California probabilistic seismic hazard database by the California Geological Survey (Cao, et al. 2003). Although the San Joaquin Hills thrust has not been observed directly at the surface, structural modeling indicates that this fault has a slip rate of approximately 0.5 millimeters per year and a recurrence interval of approximately 1,650 to 3,100 years for moderate-sized earthquakes. Recent blind thrust earthquakes, including the 1987 magnitude 5.9 Whittier Narrows and the 1994 magnitude 6.7 Northridge events, have demonstrated the significance of these features with respect to the tectonic setting of southern California.

Seismic Hazard Zones

Through the Seismic Hazards Mapping Act, the California Geological Survey (formerly the California Division of Mines and Geology) has established Seismic Hazard Zones for the more densely populated areas of southern and northern California. According to the Seismic Hazard Zone map for the Laguna Beach 7.5-minute quadrangle (CDMG, 1998), the subject site is not located within an area that has been mapped as being potentially susceptible to earthquake-induced liquefaction or landsliding. Given the generally dense nature of the soil and bedrock materials underlying the site, the lack of a shallow groundwater condition, and the relatively flat topography of the site, this zonation is considered appropriate.

Seismically Induced Flooding

The types of seismically-induced flooding that are generally considered as potential hazards to a particular site normally include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site lies several miles away from and at least 274 feet above the Pacific Ocean, and since it does not lie in close proximity to an enclosed body of water or directly downstream of a major water retention structure, the probability of flooding from a tsunami, seiche or dam-break is considered to be very low.

CONCLUSIONS AND RECOMMENDATIONS

General

From a soils engineering and engineering geologic standpoint, the subject property is considered suitable for the proposed development provided grading and construction are performed in accordance with local ordinances and codes, CBC requirements and recommendations provided in this report.

Final Grading & Wall Plan Review

The following earthwork and foundation design recommendations have been prepared without a grading or a wall plan. Once a grading and/or wall plans are available, they should be reviewed by the project geotechnical consultant with respect to the geotechnical aspects of the planned site development so that additional recommendations can be prepared as deemed appropriate.

Primary Geotechnical Concerns

Existing Undocumented Fill and Unsuitable Soils

The existing undocumented fill is not considered suitable as a bearing media for new fill or structural foundations. In addition, it is expected that existing surficial soils will be disturbed during the demolition of the existing walls and other improvements. Therefore, the existing undocumented fill will require complete removal to competent bedrock prior to re-placement as engineered fill to design grade. Recommendations for remedial grading and for design and construction of foundations are provided in the “Earthwork” and “Foundation Design Guidelines” sections of this report.

Stability of Temporary Excavations and Backcuts

Temporary excavations with sidewalls of varying height are anticipated to accommodate construction of the retaining walls. The sidewalls of these temporary excavations are primarily expected to expose undocumented fill over competent bedrock. Based on the physical characteristics of the on-site soil materials, temporary backcuts may be tentatively planned at a vertical gradient within the bedrock up to a height of 5 feet. The upper portion of cuts in bedrock exceeding 5 feet as well as all cuts in fill should be tentatively planned at a slope ratio of 1:1 (horizontal to vertical).

Temporary unsupported sidewalls constructed at the recommended maximum slope ratio are expected to remain stable during the remedial grading, however, all temporary slopes should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, revised temporary slope configurations may be necessary.

Recommendations for remedial grading are provided in the “Earthwork” and “Foundation Design Guidelines” sections of this report.

Protection of Adjacent Properties and Structures

Grading and construction of perimeter improvements such as site walls may occur along the property boundaries where site constraints may restrict grading. Provisions to help maintain the stability of adjacent properties should be considered during grading and are described in the “Earthwork and Grading” section of this report.

Foundation Setbacks

A structural setback from the face of the adjacent descending slope is required by the 2022 California Building Code (CBC) equal to one third the slope height (maximum 40 feet).

Slope Creep

The probability exists for development of a creep condition on the descending slope with the passage of time. Creep is an imperceptibly slow, nearly continuous downward and outward movement of slope soils that cannot be eliminated; however, the use of a properly designed foundation system can significantly reduce the adverse effects of slope creep on structures built on slopes.

Based on the earth materials encountered during our investigation, 2022 CBC setback requirements are expected to govern over estimated creep setbacks at this site.

Earthwork

General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with all applicable requirements of the City of Laguna Beach Building Department, the 2022 California Building Code (CBC), and in accordance with the following recommendations prepared by this firm.

Site Clearing

All structural materials associated with the existing improvements to be abandoned or remodeled, including footings and underlying drainage devices, should be removed from the site. Clearing operations should also include the removal of all landscape vegetation and existing structural features. Trees and large shrubs, when removed, should be grubbed out to include their stumps and major root systems. Existing underground utility lines located within proposed grading areas should also be removed and the resultant excavations

backfilled with engineered fill. Should any unusual soil conditions or subsurface structures be encountered during grading, they should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Ground Preparation

All of the existing undocumented fill (af) within the proposed area of improvements should be overexcavated to competent bedrock prior to placement of new engineered fill to design finish grades. Based on our exploratory borings the maximum depth of the existing undocumented fill is approximately 2 feet. Prior to replacing the overexcavated soils as engineered fill, the exposed bottom surfaces should first be scarified to a depth of 6 inches, watered or dried as necessary to achieve a uniform moisture content that is equal to or slightly greater than optimum, and then re-compacted in place to a minimum relative compaction of 90 percent of the applicable laboratory maximum dry density as determined in accordance with ASTM D 1557. This procedure should be followed in all fill areas, in areas to remain at existing grade, and in shallow cut areas where all of the unsuitable materials are not removed in their entirety.

Excavation Characteristics

Existing fill materials are expected to be readily excavatable with conventional earthmoving equipment. Portions of the bedrock material may require additional means of excavation beyond conventional earthmoving equipment such as coring, jack-hammers, or other specialized excavation equipment.

Fill Placement and Testing

All fill should be placed in lifts not exceeding 6 inches in thickness, watered or air dried as necessary to achieve at or above optimum moisture conditions, and then compacted in place to a minimum relative compaction of 90 percent of the applicable laboratory maximum dry density in accordance with ASTM Test Method D 1557. Each fill lift should be treated in a similar manner. Subsequent lifts should not be placed until the preceding lift has been approved by the project geotechnical consultant.

Imported soils, if any, should consist of clean granular materials exhibiting a Low expansion potential (Expansion Index between 21 and 50) and be free of deleterious materials, oversize rock and any organic materials. Soils to be imported should be approved by the project geotechnical consultant prior to importation.

Benching

Engineered fills placed against slope surfaces inclining at 5:1 or steeper (horizontal to vertical), should be placed on a series of level benches excavated into competent soil materials. The geotechnical consultant

should be notified as benching progresses in order to observe the stability of the benched slope and to monitor for unanticipated conditions.

Stability of Temporary Excavation Sidewalls and Backcuts

During site grading, a temporary excavation with sidewalls of varying height will be required for the proposed retaining wall at the toe of the slope. Based on the physical characteristics of the on-site materials, temporary slopes not exceeding a height of approximately 5 feet may be tentatively planned at a vertical gradient within bedrock. However, where these sidewalls exceed this height, the lower 5 feet may be cut vertical and the upper portions above a height of 5 feet should be cut back at a maximum gradient of 1:1, horizontal to vertical, or flatter. All fill materials should be cut back at a maximum gradient of 1:1 also.

If these cuts cannot be laid back at the configuration recommended above without encroaching into other properties, shoring will be required. If shoring is required, geotechnical design parameters for same can be provided upon request.

Temporary slopes excavated at the above slope configurations are expected to remain stable during construction; however, the temporary excavations should be observed by a representative of the project geotechnical consultant for any 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 temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures, and weather conditions at the time of construction. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed.

No temporary excavations along the property lines should be left open without proper protections to mitigate safety hazards. **The grading contractor is solely responsible for ensuring the safety of construction personnel and the general public, and for appointing a designated “Competent Person” to observe and classify temporary excavation sidewalls pursuant to 29 CFR Part 1926 (OSHA Safety and Health Regulations for Construction).**

Geotechnical Observations

Exposed bottom surfaces in each removal overexcavation area should be observed and approved by the project geotechnical consultant prior to placing fill. No fills should be placed without prior approval from the geotechnical consultant. The project geotechnical consultant should also be present on-site during

grading operations to observe fill placement and perform field density testing of the engineered fill, as well as to evaluate compliance with the other recommendations presented herein.

Post-Grading Considerations

Site Drainage

Positive drainage devices such as sloped concrete flatwork, graded swales and area drains should be provided around the new construction to collect and direct all water to a suitable discharge area. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. The owner is advised that the drainage system should be properly maintained throughout the life of the proposed development. The purpose of this drainage system will be to reduce water infiltration into the subgrade soils and to direct surface water away from building foundations, and walls. The following recommendations should be implemented during construction:

1. Area drains should be installed within all planter and landscape areas that are located within 10 feet of building foundations to reduce excessive infiltration of water into the foundation soils. Per the 2022 CBC, the ground surfaces of planter and landscape areas that are located within 10 feet of building foundations should be sloped at a minimum gradient of 5 percent away from the foundations and towards the nearest area drains. The ground surfaces of planter and landscape areas that are located more than 10 feet away from building foundations may be sloped at a minimum gradient of 2 percent away from the foundations and towards the nearest area drains.
2. Per the 2022 CBC, concrete flatwork surfaces that are located within 10 feet of building foundations should be inclined at a minimum gradient of 2 percent away from the building foundations and towards the nearest area drains. Concrete flatwork surfaces that are located more than 10 feet away from building foundations may be sloped at a minimum gradient of 1 percent away from the foundations and towards the nearest area drains.
3. A watering program should be implemented for the landscaped areas that maintain a uniform, near optimum moisture condition in the soils. Overwatering and subsequent saturation of the soils may cause excessive soil expansion (heave) or settlement and should thus be avoided. On the other hand, allowing the soils to dry out may cause excessive soil shrinkage. As an alternative to a conventional irrigation system, drip irrigation is strongly recommended for all planter areas. The owner is advised that all irrigation and drainage devices should be properly maintained throughout the lifetime of the development.

Slope Landscaping and Maintenance

A permanent slope maintenance program should be initiated that should include the care of drainage and erosion control provisions, rodent control, and repair of leaking irrigation systems. The owner should be advised that potential problems can develop when drainage on the graded level building pad and slope is altered in any way.

Bottomless Trench Drains

Infiltration systems are not recommended for this site.

Utility Trench Backfill

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Onsite soils cannot be densified adequately by flooding and jetting techniques; therefore, trench backfill materials should be placed in lifts no greater than approximately 6 inches in thickness, watered or air dried as necessary to achieve a uniform moisture content that is equal to or slightly above optimum moisture, and then mechanically compacted in-place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to document that adequate compaction has been achieved.

For shallow trenches where pipe may be damaged by mechanical compaction equipment, such as under the building floor slab, imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped in place. No specific relative compaction will be required; however, observation, probing, and, if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to document that the sand backfill is adequately compacted and will not be subject to excessive settlement.

Where utility trenches enter the footprint of the building, they should be backfilled through their entire depths with on-site fill materials, sand-cement slurry or concrete rather than with any sand or gravel shading. This “plug” of less- or non-permeable materials will reduce the potential for water to migrate through the backfilled trenches from outside of the building to the areas beneath the foundations and floor slabs.

If clean, imported sand is to be used for backfill of exterior utility trenches, it is recommended that the upper 12 inches of trench backfill materials consist of properly compacted on-site soil materials. This is to reduce infiltration of irrigation and rainwater into granular trench backfill materials.

Where an interior or exterior utility trench is proposed parallel to a building footing, the bottom of the trench should not be located below a 1:1 plane projected downward from the outside bottom edge of the adjacent footing. Where this condition exists, the adjacent footing should be deepened such that the bottom of the utility trench is located above the 1:1 projection.

Foundation Design Considerations

Near-Fault Site Determination

Based on our review of the referenced geologic maps and literature, no active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) defines an *active fault* as one that “has had surface displacement within Holocene time (about the last 11,000 years).” The main objective of the AP Act is to prevent the construction of dwellings on top of active faults that could displace the ground surface resulting in loss of life and property.

However, it should be noted that according to the USGS Unified Hazard Tool website and/or 2010 CGS Fault Activity Map of California, the Newport -Inglewood fault, located approximately 2.5 miles southwest of the site, 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 a distance of less than 9.5 miles (15 km) from the surface projection of this fault system, which is capable of producing a magnitude 7 or larger events with a slip rate along the fault greater than 0.04 inch per year. As such, the site should be considered as a **Near-Fault Site** in accordance with ASCE 7-16, Section 11.4.1.

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 States 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, V_{s30} , within the upper 30 meters (approximately 100 feet) of site soils.

A seismic risk category of II was assigned to the proposed structures in accordance with 2022 CBC, Table 1604.5. Based on our engineering geology judgement, the bedrock at the site appears to exhibit the characteristics of a Site Class B, i.e., competent rock with moderate fracturing and weathering, however, no direct, small-strain shear wave measurement of shear wave velocity was performed. Therefore, an average shear wave velocity in the range of 2,500 to 5,000 feet per second for the upper 100 feet was considered for the site based on engineering judgment and geophysical experience. As such, in accordance with ASCE 7-16, Table 20.3-1, Site Class B (B - Estimated as per SEAOC/OSHPD software) has been assigned to the subject site.

The following table, Table 1, provides parameters required to construct the design acceleration response spectrum based on the 2022 CBC guidelines. Please note that for Site Class B - Estimated, Site Coefficients, F_a , F_v , and F_{PGA} should be taken as unity (1.0), as reflected in Table 1. A printout of the computer output is attached in Appendix C.

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TABLE 1
Seismic Design Parameters

Ground Motion Parameters	Specific Reference	Parameter Value	Unit
Site Latitude (North)	-	33.5289	°
Site Longitude (West)	-	-117. 7660	°
Site Class Definition	Section 1613.2.2 ⁽¹⁾ , Chapter 20 ⁽²⁾	B-Estimated ⁽⁴⁾	-
Assumed Seismic Risk Category	Table 1604.5 ⁽¹⁾	II	-
M _w - Earthquake Magnitude	USGS Unified Hazard Tool ⁽³⁾	7.3 ⁽³⁾	-
R – Distance to Surface Projection of Fault	USGS Unified Hazard Tool ⁽³⁾	3.96 ⁽³⁾	km
S _s - Mapped Spectral Response Acceleration Short Period (0.2 second)	Figure 1613.2.1(1) ⁽¹⁾	1.325 ⁽⁴⁾	g
S ₁ - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(2) ⁽¹⁾	0.47 ⁽⁴⁾	g
F _a – Short Period (0.2 second) Site Coefficient	Table 1613.2.3(1) ⁽¹⁾	1 ⁽⁴⁾	-
F _v – Long Period (1.0 second) Site Coefficient	Table 1613.2.3(2) ⁽¹⁾	1 ⁽⁴⁾	-
S _{MS} – MCE _R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (0.2 second)	Equation 16-36 ⁽¹⁾	1.325 ⁽⁴⁾	g
S _{M1} - MCE _R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (1.0 second)	Equation 16-37 ⁽¹⁾	0.47 ⁽⁴⁾	g
S _{DS} - Design Spectral Response Acceleration at 0.2-s	Equation 16-38 ⁽¹⁾	0.883 ⁽⁴⁾	g
S _{D1} - Design Spectral Response Acceleration at 1-s	Equation 16-39 ⁽¹⁾	0.313 ⁽⁴⁾	g
T ₀ = 0.2 S _{D1} / S _{DS}	Section 11.4.6 ⁽²⁾	0.071	s
T _s = S _{D1} / S _{DS}	Section 11.4.6 ⁽²⁾	0.354	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⁾	0.581 ⁽⁴⁾	g
F _{PGA} - Site Coefficient Adjusted for Site Class Effect ⁽²⁾	Table 11.8-1 ⁽²⁾	1 ⁽⁴⁾	-
PGA _M –Peak Ground Acceleration ⁽²⁾ Adjusted for Site Class Effect	Equation 11.8-1 ⁽²⁾	0.581 ⁽⁴⁾	g
Design PGA ≈ (2/3 PGA _M) - Slope Stability ^(†)	Similar to Eqs. 16-38 & 16-39 ⁽²⁾	0.387	g
Design PGA ≈ (0.4 S _{DS}) – Short Retaining Walls ^(‡)	Equation 11.4-5 ⁽²⁾	0.353	g
C _{RS} - Short Period Risk Coefficient	Figure 22-18A ⁽²⁾	0.909 ⁽⁴⁾	-
C _{RI} - Long Period Risk Coefficient	Figure 22-19A ⁽²⁾	0.925 ⁽⁴⁾	-
SDC - Seismic Design Category ^(§)	Section 1613.2.5 ⁽¹⁾	D ⁽⁴⁾	-
References: ⁽¹⁾ California Building Code (CBC), 2022, 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 2/3 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.			

Footing Setbacks from Descending Slopes

Horizontal structural setback between the outside bottom edge of the proposed structure(s) footings and the face of the adjacent descending slope will be required to conform to the minimum requirements of Figure 1808.7.1 of the 2022 CBC, Chapter 18, Division I. Section 1808.7.2 and Figure 1808.7.1 of the 2022 CBC.

Although no specific foundation setback requirements are required for garden walls, planters, and decking or patios within the rear yard, any structures proposed in close proximity of the rear yard slope may be subject to future vertical and lateral movements due to slope creep and soil heave and contraction. Therefore, structures that are located along the top of the descending slope should be supported on deepened foundations. Recommendations for these deepened foundations are provided in subsequent sections of this report.

The exterior improvements should also be designed with reinforcement, thickened edges, movement joints, and either positive separations or doweled joints to help reduce the potential for distress due to movements caused by soil expansion and contraction. It should be noted that due to the depths of existing fill and the expansive nature of the fill beneath the subject site, it is not considered feasible to completely mitigate all effects of slope creep and soil heave on proposed improvements.

Allowable Bearing Capacity, Estimated Settlement and Lateral Resistance

Allowable Soil Bearing Capacities

An allowable bearing value of 2,500 pounds per square foot may be used for 24-inch-wide pad footings and 12-inch-wide continuous footings **founded at a minimum depth of 12 inches into competent bedrock**. This value may be increased by 20 percent for each additional foot of depth, to a maximum value of 5,000 pounds per square foot. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Estimated Footing Settlement

Based on the allowable bearing values provided above, total static settlement of the footings under the anticipated loads is expected to be on the order of ½ inch. Differential settlement is expected to be less than 1/4 inch over a horizontal span of 40 feet. The majority of settlement is likely to take place as footing loads are applied or shortly thereafter.

Lateral Resistance

A passive earth pressure of 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 investigation indicate that these materials predominantly exhibit expansion indices that range from 21 to 50. As such, the site soils are classified as "expansive" as defined in Section 1803.5.3 of the 2022 California Building Code (2022 CBC). The design of foundations and slabs on-ground should therefore be performed in accordance with the procedures outlined in Sections 1808.6.1 and 1808.6.2 of the 2022 CBC.

General

Briefly, Section 1808.6.1 of the 2022 CBC requires that foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Section 1808.6.2 of the 2022 CBC requires that non-prestressed slabs on-grade or mat foundations constructed on expansive soils be designed in accordance with the latest Code-adopted edition of *WRI/CRSI Design of Slab-on-Ground Foundations*. The 2022 CBC also requires that post-tensioned slabs on-grade or mat foundations placed on expansive soils be designed in accordance with the latest Code-adopted edition of *PTI DC 10.5*, with the provision that the analyses used to determination of moments, shears and deflections are performed accordingly. It should be noted that, under certain conditions, the 2022 CBC allows for alternative, rational methods of analysis and design of such slabs provided that these methods account for soil-structure interaction, the deformed shape of the soil support, plate or stiffened plate action of the slab, as well as both center lift and edge lift conditions.

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 Slabs on-Grade System

As stated above, onsite soils should be considered expansive per Section 1803.5.3 of the 2022 CBC. For soils that are considered expansive, Section 1808.6.2 of the 2022 CBC specifies that non-prestressed slab-on-grade foundations constructed on expansive materials should be designed in accordance with the latest Code-adopted edition of the Wire Reinforcement Institute (WRI) publication “Design of Slab-on-Ground Foundations”. The design procedures outlined in the WRI publication are based on the weighted plasticity index of the various soil layers existing within the upper 15 feet of the building site.

Based on the recent laboratory testing by our firm, a weighted plasticity index of 14 can be assumed for the subject site. The WRI publication states that the weighted plasticity index of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the supporting soil or bedrock materials. Since the structures will be constructed on level building pads, and in consideration of the estimated unconfined compressive strength of the onsite soils, it is recommended that the weighted plasticity index, as provided herein, be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 9 of the WRI publication). **In summary, it is recommended that an effective plasticity index of 17 be utilized by the project structural engineer to design slabs on-ground with an interior grade beam system in accordance with the WRI publication.**

Footings

1. Exterior continuous footings supporting one- and two-story structures should be founded at a minimum depth of 15 inches below the lowest adjacent final grade, respectively. Interior continuous footings may be founded at a minimum depth of 12 inches below the top of the adjacent finish floor slabs. Interior continuous footings width and spacing should be designed by the project structural engineer. **In addition, all residential footings should be deepened as necessary, such that they extend at least 12 inches into bedrock.**
2. In accordance with Table 1809.7 of 2022 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. **In addition, all residential footings should be deepened as necessary, such that they extend at least 12 inches into bedrock.** 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 for support of colonnades, roof overhangs, upper-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. **In addition, all residential footings should be deepened as necessary, such that they extend at least 12 inches into bedrock.** 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.
6. 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. Further, where excessive soils settlement issues have been identified for this site elsewhere in the report, it is strongly recommended to tie all footings both interior and exterior with a network of grade beams to reduce the potential differential settlement or isolated bearing distress issues below any independent footings.
7. The spacing and layout of the interior concrete grade beam system required below floor slabs should be determined by the project architect or structural engineer in accordance with the WRI publication using the effective plasticity index value provided previously.
8. To reduce the potential for distress due to differential settlement between the existing building footings and any new addition footings, the new footings should be doweled into the existing footings. This connection between the new and existing footings should be designed by the project structural engineer.
9. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2022 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 18 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid-depth.

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.

It should be noted that some of the non-climatic site parameters, which may impact slabs on-grade performance, are not known at this time, as it is the case for many projects at the design stage. Some of these site parameters include unsaturated soils diffusion conditions pre- and post-construction (e.g., casting the slabs at the end of long, dry or wet periods, maintenance during long, dry and wet periods, etc.), landscaping, alterations in site surface gradient, irrigation, trees, etc. While the effects of any or a combination of these parameters on slab performance cannot be accurately predicted, maintaining moisture content equilibrium within the soils mass and planting trees at a distance greater than half of their mature height away from the edge of foundation may reduce the potential for the adverse impact of these site parameters on slabs on-grade performance.

2. To reduce the potential for distress due to differential settlement between the existing building slab and any new slabs of a building addition, the new slabs should be doweled into the existing slab. This connection between the new and existing slabs should be designed by the project structural engineer.
3. 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.

In general, 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. Foot traffic on the membrane should be reduced to a minimum. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement. To comply with Section 1907.1.1 of the 2016 CBC, the living area concrete floor slab should also be underlain with capillary break consisting of a minimum of 4 inches of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 sieve. The capillary break should be placed below the 10-mil moisture vapor retarder.

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 contractor with experience in slab construction and curing should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing.

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 $\frac{3}{4}$ -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. Prior to placing concrete, the subgrade soils below living area floor slabs should be prewatered to achieve a moisture content that is at least 1.2 times the optimum moisture content. This moisture should penetrate to a depth of approximately 12 inches into the subgrade.
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 2022 CBC) by the structural engineer responsible for foundation design based on his/her calculations and engineering experience and judgment.

Foundation Observations

All foundation excavations should be observed by a representative of the project geotechnical consultant to verify that they have been excavated into competent materials. These observations should be performed prior to the placement of forms or reinforcement. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture-softened materials and/or any construction debris should be removed prior to the placement of concrete. Excavated soils derived from footing and utility trenches should not be placed in slab-on-grade areas unless they are compacted to at least 90 percent of maximum dry density.

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.

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 chlorides 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 2
Soil Corrosivity Screening Results

Test	Test Results	Classification	General Recommendations
Soluble Sulfates (Cal 417)	0.0084%	SO ¹ - Not Applicable	Type II cement; minimum $f'_c = 2,500$ psi; no water/cement ratio restrictions.
pH (Cal 643)	8.5	Strongly Alkaline ²	Type I-P (MS) Modified or Type II Modified cement
Soluble Chloride (Cal 422)	232.5 ppm	C1 ³ C2 ⁴	Residence: No max water/cement ratio, $f'_c = 2,500$ psi Spas/Decking: water/cement ratio 0.40, $f'_c = 5,000$ psi
Resistivity (Cal 643)	1,800 ohm-cm	Highly Corrosive ⁵	Consult a corrosion engineer

Notes:

1. ACI 318-14, Section 19.3
2. The United States Department of Agriculture Natural Resources Conservation Service, formerly Soil Conservation Service
3. ACI 318-14, Section 19.3
4. Exposure classification C2 applies specifically to swimming pools and appurtenant concrete elements
5. Pierre R. Roberge, "Handbook of Corrosion Engineering"

Retaining Wall Design Recommendations

Construction on Level Ground

Footings for retaining walls proposed on level ground and at least 15 feet from the top of any descending slope may be designed in accordance with the bearing and lateral resistance values provided previously for building footings; however, when calculating passive resistance, the resistance of the upper 6 inches of the soils should be ignored in areas where the footings will not be covered with concrete flatwork, or where the thickness of soil cover over the top of the footing is less than 12 inches. **Additionally, all wall footings should be deepened as necessary, such that they extend at least 12 inches into bedrock.**

Construction along Top of the Adjacent Descending Slope

A horizontal structural setback between the outside bottom edge of the proposed retaining wall footings and the face of the adjacent descending slope will be required to conform to the minimum requirements of Figure 1808.7.1 of the 2022 CBC, Chapter 18, Division I. Section 1808.7.2 and Figure 1808.7.1 of the 2022 CBC based on the total height of the adjacent slope below the proposed wall.

On the basis of this condition, any retaining wall sections proposed near the top of the descending slope should be supported on either deepened footings or caissons in order to achieve the required 2022 CBC setback. Recommendations for deepened footings are provided below. If caissons are required/desired, geotechnical design recommendations can be provided upon request.

Deepened Footings: Footings for any retaining wall sections proposed near the top of the rear yard descending slope should be founded at a depth that will provide a **minimum** footing setback of at least $h/3$ measured along a horizontal line projected from the outside bottom edges of the footings to the daylight contact with the slope face in order to extend the footings the required setback distance. It should be noted that additional footing depths may be required to achieve the necessary passive resistance against lateral movement as determined by the project structural engineer based on the soil parameters provided below. **Additionally, all wall footings should be deepened as necessary, such that they extend at least 12 inches into bedrock.**

Footings for retaining wall sections at the above recommended minimum setbacks may be designed using the allowable bearing values recommended previously for building footings; however, when calculating passive resistance, the passive earth pressure should be reduced to 150 pounds per square foot, per foot of depth, to a maximum value of 1,500 pounds per square foot. In addition, the lateral resistance should be ignored for the upper portions of the wall footings located within the creep zone (upper 2 feet).

Active and At-Rest Earth Pressures

Active and at-rest earth pressures to be utilized for design of any retaining walls to be constructed within the will be dependent on whether on-site soils or imported granular materials are used for backfill. For this reason, active and at-rest earth pressures are provided below for both conditions. However, considering that

the onsite earth materials have a low expansion potential, it is our recommendation that imported granular materials be used for backfilling behind the retaining walls as described in the following sections.

1. On-Site Soils Used for Backfill

If on-site soils are used as backfill, active earth pressures equivalent to fluids having densities of 40 and 61 pounds per cubic foot should be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 60 and 92 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system (see Figure RW-1). All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

2. Imported Sand, Pea Gravel or Rock Used for Wall Backfill

Where imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, or pea gravel or crushed rock are be used for wall backfill, the lateral earth pressures may be reduced provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater (see Figures RW-2 and RW-3). For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 45 and 62 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are also for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the recommended active and at-rest earth pressures.

Earthquake Loads on Retaining Walls

Note 1 of Section 1803.5.12 of the 2022 CBC indicates that the dynamic seismic lateral earth pressures on foundation walls and retaining walls supporting more than 6 feet of backfill height due to design earthquake ground motions be determined.

Per Section 1613.1 of the 2022 CBC, structures shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7. More specifically, Equation 11.4-5 of ASCE 7-10 provides the design peak ground acceleration (PGA) as follows:

$$PGA = S_a = S_{DS} [0.4 + 0.6 (T/T_0)]$$

Where:

S_a , is the spectral response acceleration,

S_{DS} , is design, 5 percent damped, spectral response acceleration at short periods,

T , is the fundamental period of the building,

T_0 , is taken as $0.2 S_{D1}/S_{DS}$, and

S_{D1} , is the design, 5 percent damped, spectral response acceleration at a period of 1 second.

The fundamental period of a short, masonry retaining wall that has no damping is infinitesimal. For comparison, the fundamental period of a ten-story and a one-story, relatively ductile, building is very close to 1 and 0.1 seconds, respectively. As such, the fundamental period of a short, masonry wall can be taken as zero. Therefore, equating T with zero in above equation yields $PGA = 0.4 S_{DS}$. From Table 1 presented in our geotechnical investigation report (Petra, 2023), this value is 0.353g for this site. This value was used in the Seed and Whitman (1970) simplified calculation for level conditions behind retaining structures.

According to the research of Sitar, et al. (2012), the simplified Seed and Whitman calculation is appropriate for use for both cantilever retaining walls and restrained walls.

Basement Walls

For restrained basement walls, the horizontal ground acceleration value k_h may be assumed to equal the peak ground acceleration. Thus, $k_h = a_g = 0.353g$.

From Seed and Whitman (1970), the lateral load on a retaining structure can be determined by the following equation:

$$P_D = \gamma \left(\frac{3}{4} \right) K_h$$

Where:

P_D = Dynamic Lateral Earth Pressure,
 γ = weight of soil = 120 pcf, and
 K_h = horiz. ground acceleration

thus, $P_D = (120 \text{ pcf}) \left(\frac{3}{4} \right) (0.353) = 31.77 \text{ pcf}$, **use 32 pcf**.

The distribution of the seismic lateral load is discussed below.

Cantilever Retaining Walls

From the County of Los Angeles Department of Public Works Manual for the Preparation of Geotechnical Reports (Dec., 2006), the horizontal ground acceleration value k_h for cantilever retaining walls may be assumed to be equal to half of the peak ground acceleration. Thus, $k_h = \frac{1}{2} (a_g) = (0.5) (0.353g) = 0.177g$.

From Seed and Whitman (1970), the lateral load on a retaining structure can be determined by the following equation:

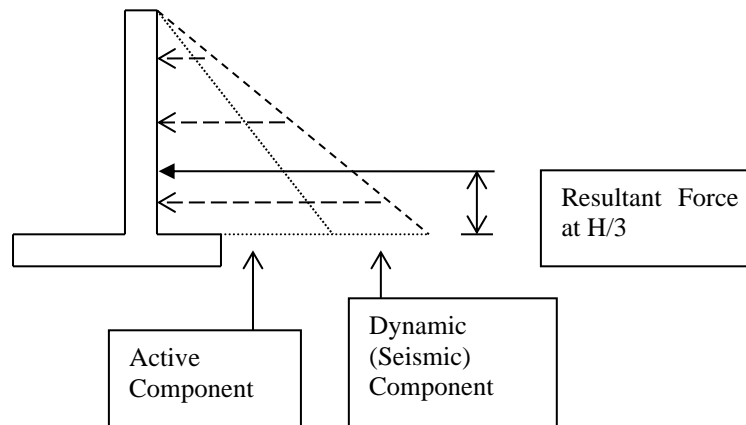
$$P_D = \gamma \left(\frac{3}{4} \right) K_h$$

where P_D = Dynamic Lateral Earth Pressure,
 γ = weight of soil = 120 pcf, and
 K_h = horiz. ground acceleration

thus, $P_D = (120 \text{ pcf}) \left(\frac{3}{4} \right) (0.177) = 15.88 \text{ pcf}$, **use 16 pcf**.

For cantilever and retrained retaining walls, Sitar, et al. (2012), indicates that the seismic earth pressure has a triangular distribution with the largest load occurring at the bottom of the wall.

The distribution of the seismic lateral load is as follows:



Subdrainage

Perforated pipe and gravel subdrains should be installed behind all basement and retaining walls to prevent entrapment of water in the backfill (see Figures RW-1 through RW-3). Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of $\frac{3}{4}$ -inch to 1½-inch open-graded gravel. If on-site soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-third the wall height or to a minimum height of 1.5 feet above the footing, whichever is greater. If imported sand, pea gravel, or crushed rock is used as backfill, subdrain details shown on Figures RW-2 and RW-3 should be utilized. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

If a limited area exists behind the walls for installation of a pipe and gravel subdrain, a geotextile drain mat such as Mirafi Miradrain, or equivalent, can be used in lieu of drainage gravel. The drain mat should extend the full height and lengths of the walls and the filter fabric side of the drain mat should be placed up against the backcut. The perforated pipe drain line placed at the bottom of the drain mat should consist of 4-inch minimum diameter PVC Schedule 40 or SDR-35. The filter fabric on the drain mat should be peeled back and then wrapped around the drain line.

Waterproofing

The portions of retaining walls supporting backfill should be coated with an approved waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Recommended active and at-rest earth pressures for design of retaining walls are based on the physical and mechanical properties of the on-site soil materials. On-site soil materials may be difficult to compact when placed in the relatively confined areas located between the walls and temporary backcut slopes. Therefore, to facilitate compaction of the backfill, consideration should be given to using pea gravel or crushed rock behind the proposed retaining walls. For this condition, the reduced active and at-rest pressures provided previously for sand, pea gravel, or crushed rock backfill may be considered in wall design provided they are installed as shown on Figures RW-2 and RW-3.

Where the onsite soils materials or imported sand (with a Sand Equivalent of 30 or greater) are used as backfill behind the proposed retaining walls, the backfill materials should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to document field density tests.

If imported pea gravel or rock is used for backfill, the gravel should be placed in approximately 2- to 3-foot-thick lifts, thoroughly wetted but not flooded, and then mechanically tamped or vibrated into place. A representative of the project geotechnical consultant should observe the backfill procedures and probe the backfill to determine that an adequate degree of compaction is achieved.

To reduce the potential for the direct infiltration of surface water into the backfill, imported sand, gravel, or rock backfill should be capped with at least 12 to 18 inches of on-site soil. Filter fabric such as Mirafi 140N or equivalent, should be placed between the soil and the imported gravel or rock to prevent fines from penetrating into the backfill. If a thicker cap is desired (for planting or other reasons), consultation with the project structural engineer may be required to ascertain if the wall design is appropriate for the additional lateral pressure that a thicker cap of native material may impose.

Geotechnical Observation and Testing

All grading and construction phases associated with retaining wall construction, including backcut excavations, observation of the footing and pier excavations, installation of the subdrainage systems, and placement of backfill should be provided by a representative of the project geotechnical consultant.

EXTERIOR CONCRETE FLATWORK

General

Near-surface compacted fill soils within the site are variable in expansion behavior and are expected to exhibit very low to low expansion potential. For this reason, we recommend that all exterior concrete flatwork such as sidewalks, patio slabs, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private vehicular driveways and/or access roads within the site be designed by the project architect and/or structural engineer with consideration given to mitigating the potential cracking and uplift that can develop in soils exhibiting expansion index values that fall in the low category.

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

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Private driveways that will be designed for the use of passenger cars for access to private garages should also be at least 5 inches thick and provided with construction joints or expansion joints every 10 feet or less.

Reinforcement

All concrete flatwork having their largest plan-view panel dimension exceeding 5 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. The reinforcement should be properly positioned near the middle of the slabs.

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

Edge Beams (Optional)

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches

below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

Subgrade Preparation

Compaction

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Drainage

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

Tree Wells

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

LONG-TERM EFFECT OF SOIL EXPANSION

As mentioned previously in this report, the site is underlain by expansive fill materials. Due to their inherent composition, these materials invariably exhibit the potential to undergo a certain amount of long-term volume changes such as settlement, heave, and lateral movement. When water is introduced to expansive soils by such sources as landscape irrigation, swimming pools and rainfall, the expansive soils tend to absorb an excessive amount of moisture, which, in turn, causes the expansive soils to expand and heave. This heave causes an upward and lateral movement of hardscaped areas or, if the movement is restricted, causes distress and fracturing to hardscape features constructed in these areas.

Expansive soils not only expand as the moisture contents increase, but also contract as their moisture contents decrease. Therefore, as a result of seasonal variations in the moisture contents, this repeated cycle of expansion and contraction causes a loss in density and shear strength. In addition, these cycles cause progressive outward and downward movement of the surficial materials located on or in close proximity to descending slopes (slope creep). The progressive outward and downward movement due to slope creep may, in turn, cause distress and tilting to such structures as masonry block walls, retaining walls, and concrete flatwork that are constructed in these areas.

Although the recommendations provided in this report are intended to reduce the potential for distress of structures resulting from the effects of expansive soils, our experience has shown that even with the implementation of these recommendations, a certain amount of cracking and/or horizontal and vertical movements is unavoidable and can be anticipated during the lifetime of the proposed development. The homeowner should be made fully aware that the property is underlain by expansive soils and that these soils can cause the concerns noted above.

FUTURE IMPROVEMENTS

Should any new structures or improvements be proposed at any time in the future other than those discussed herein, our firm should be notified so that we may provide design recommendations. Design recommendations are particularly critical for any new improvements that may be proposed on or near the slope descending from the rear yard, and in areas where they may interfere with the proposed permanent drainage facilities.

Potential problems can develop when drainage on the pad is altered in any way (i.e., excavations or placement of fills associated with construction of new walkways, patios, block walls and planters). Therefore, it is recommended that we be engaged to review the final design drawings, specifications and grading plan prior to any new construction. If we are not given the opportunity to review these documents

with respect to the geotechnical aspects of new construction and grading, we can take no responsibility for misinterpretation of our recommendations presented herein.

REPORT LIMITATIONS

It should be noted that Petra was not contracted to investigate or analyze the site slope nor to investigate or evaluate the existing residence and associated improvements. Accordingly, this report specifically excludes any findings related to the performance of the existing railroad-tie retaining devices and their possible effects on the surficial stability of the rear yard slope as well as the structural integrity of the existing residential structure, stone and concrete retaining structures, and associated improvements.

This report has been prepared without the aid of a grading and wall plan depicting the proposed grading and construction. As such, the recommendations provided in this report should be considered tentative until a finalized precise grading and wall plan are available and reviewed by the project geotechnical consultant. Additional recommendations and/or modification of the recommendations provided herein may be necessary depending on the results of a finalized precise grading plan review.

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

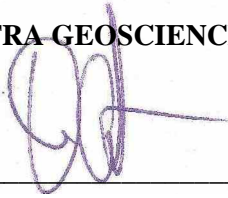
This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guarantee or warranty.

This report should be reviewed and updated after a period of one year or if the project concept changes from that described herein. Additionally, this report has not been prepared for use by parties or projects other than those named or described herein as it may not contain sufficient information for other parties or other purposes.

This opportunity to be of service is appreciated. Should you have any questions or require additional information, please call this office at (714) 549-8921.

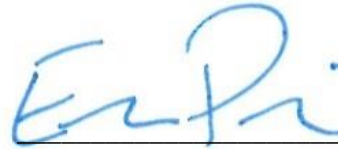
Respectfully submitted,

PETRA GEOSCIENCES, INC.

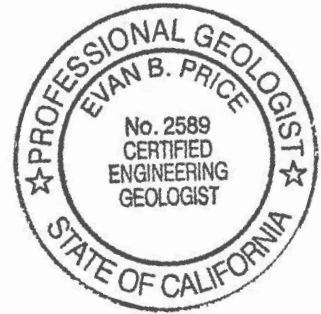


Don Obert
Associate Engineer
RGE 2872

EBP/DO/lv



Evan Price
Senior Project Geologist
CEG 2589



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REFERENCES

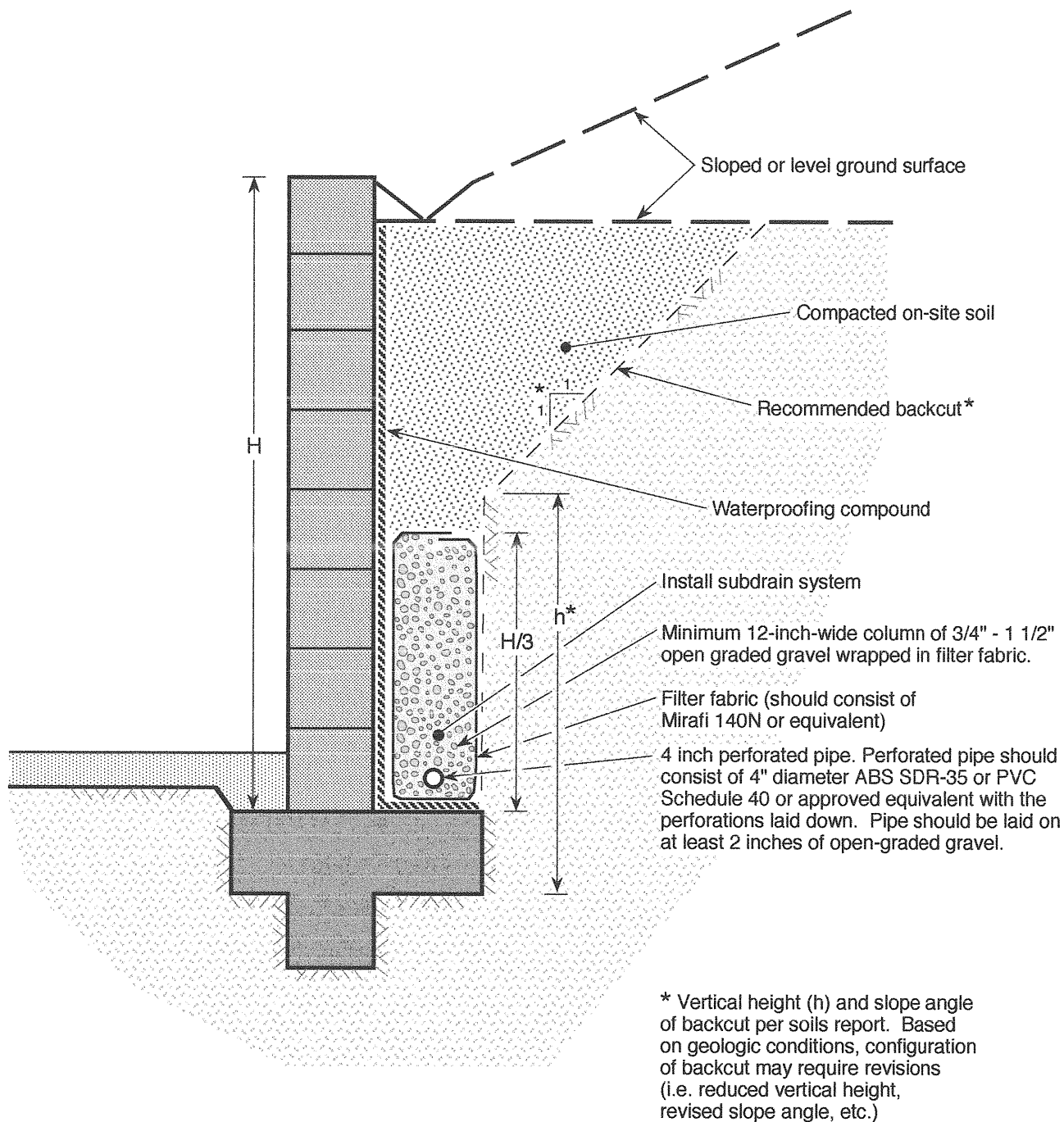
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FIGURES

NATIVE SOIL BACKFILL

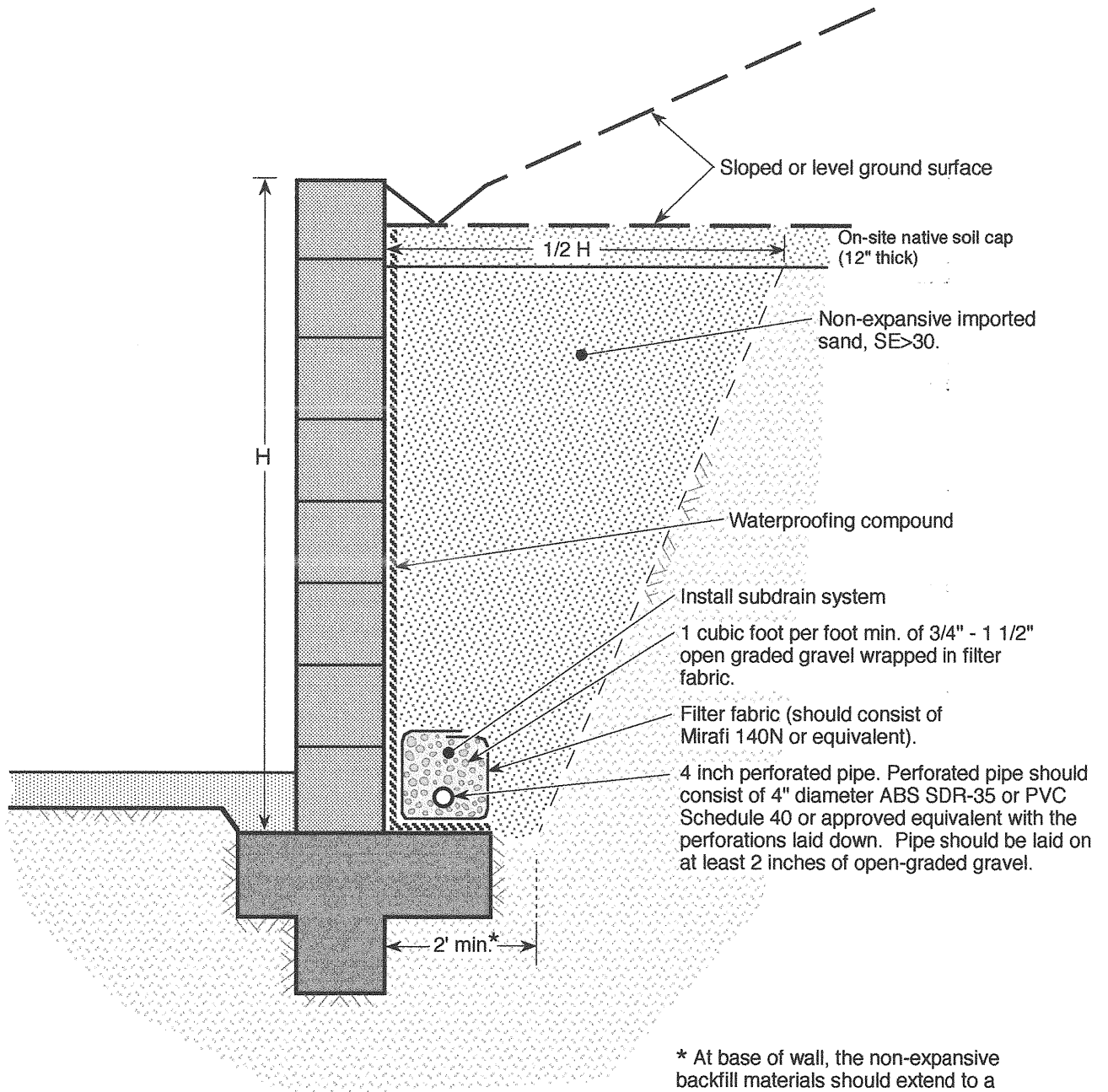


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**RETAINING WALL BACKFILL
AND SUBDRAIN DETAILS**

FIGURE RW-1

IMPORTED SAND BACKFILL



* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.

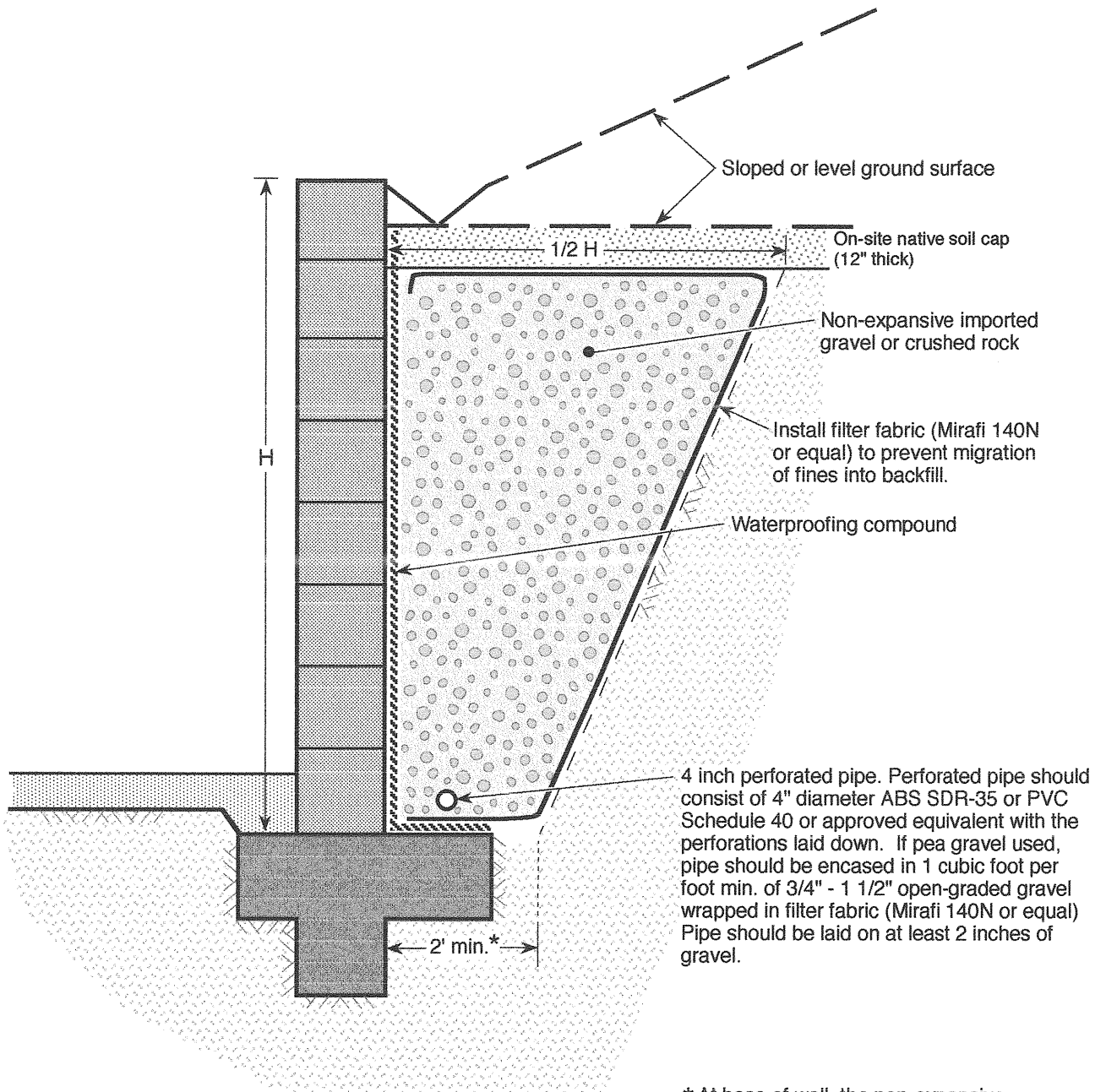


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**RETAINING WALL BACKFILL
AND SUBDRAIN DETAILS**

FIGURE RW-2

IMPORTED GRAVEL OR CRUSHED ROCK BACKFILL



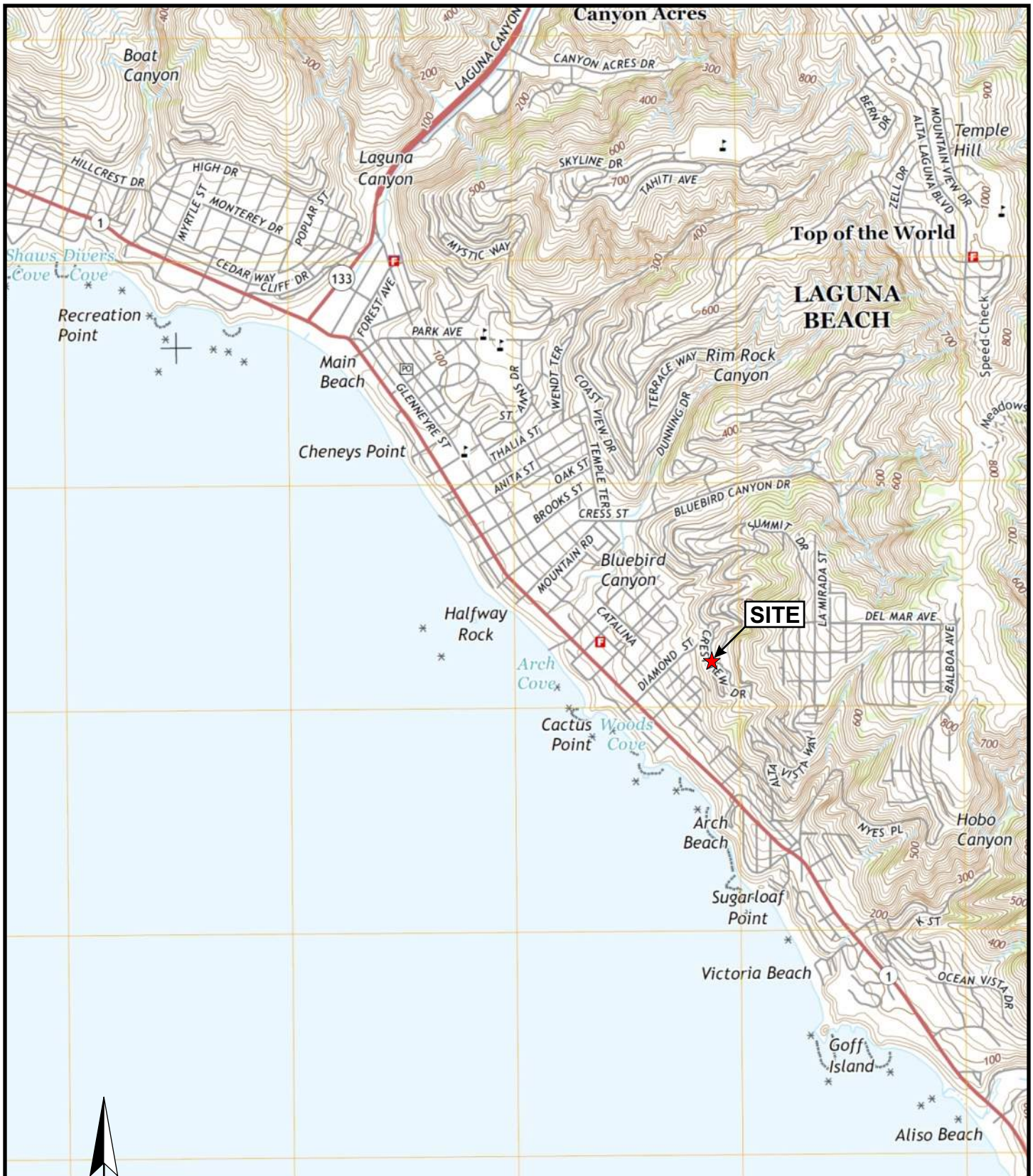
* At base of wall, the non-expansive backfill materials should extend to a min. distance of 2' or to a horizontal distance equal to the heel width of the footing, whichever is greater.



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**RETAINING WALL BACKFILL
AND SUBDRAIN DETAILS**

FIGURE RW-3



Scale: 1" = 2,000'

Base Map: Portion of USGS Laguna Beach Quadrangle
7.5-Minute Topographic Series, 2015

PETRA GEOSCIENCES, INC.

3186 Airway Avenue, Suite K
Costa Mesa, California 92626
PHONE: (714) 549-8921
COSTA MESA TEMECULA LOS ANGELES PALM DESERT CORONA ESCONDIDO

SITE LOCATION MAP

2155 Crestview Drive
Laguna Beach, California



DATE: November 2023
J.N.: 23-299

Figure 1



APPENDIX A



EXPLORTION LOGS

EXPLORATION LOG

Project: Seaside Shores, LLC					Boring No.: HA-1				
Location: 2155 Crestview, Laguna Beach					Elevation: 324±				
Job No.: 23-299		Client: Coulter Milner & Co			Date: 9/22/23				
Drill Method: Hand Auger		Driving Weight: N/A			Logged By: SS				
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples			Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	X	ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, slightly moist, medium dense, fine- to coarse-grained, some gravel.							
		BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, dry to slightly moist, dense, fine- to coarse-grained. Total Depth = 2.1' Groundwater not encountered during drilling Backfilled with cuttings.							
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PLATE

EXPLORATION LOG

Project: Seaside Shores, LLC				Boring No.: HA-2					
Location: 2155 Crestview, Laguna Beach				Elevation: 311±					
Job No.: 23-299		Client: Coulter Milner & Co		Date: 9/22/23					
Drill Method: Hand Auger		Driving Weight: N/A		Logged By: SS					
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples			Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0		ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, dry to slightly moist, medium dense, fine- to coarse-grained, some gravel.							
		BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive.							CL, pH, RES, SO4, ATT, EI
		Total Depth = 2' Groundwater not encountered during drilling Backfilled with cuttings.							
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PLATE

EXPLORATION LOG

Project: Seaside Shores, LLC		Boring No.: HA-3						
Location: 2155 Crestview, Laguna Beach		Elevation: 305±						
Job No.: 23-299	Client: Coulter Milner & Co		Date: 9/22/23					
Drill Method: Hand Auger	Driving Weight: N/A		Logged By: SS					
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples		Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)
0		ARTIFICIAL FILL, undocumented (afu) <u>Clayey Sand (SC):</u> Brown, slightly moist, medium dense, fine- to coarse-grained. Total Depth = 1' Groundwater not encountered during drilling Backfilled with cuttings.				6.8		
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PLATE



EXPLORATION LOG

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PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project: Seaside Shores, LLC		Boring No.: HA-5							
Location: 2155 Crestview, Laguna Beach		Elevation: 292±							
Job No.: 23-299	Client: Coulter Milner & Co	Date: 9/22/23							
Drill Method: Hand Auger	Driving Weight: N/A	Logged By: SS							
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples			Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0		ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown to light brown, dry, dense, fine- to coarse-grained.							
		BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive.							
		Total Depth = 2'							
5		Groundwater not encountered during drilling Backfilled with cuttings.							
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EXPLORATION LOG

Project: Seaside Shores, LLC				Boring No.: HA-6					
Location: 2155 Crestview, Laguna Beach				Elevation: 282±					
Job No.: 23-299		Client: Coulter Milner & Co		Date: 9/22/23					
Drill Method: Hand Auger		Driving Weight: N/A		Logged By: SS					
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples			Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	ARTIFICIAL FILL	ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, slightly moist, medium dense, fine- to coarse-grained, few gravel. BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive. Total Depth = 1.5' Groundwater not encountered during drilling Backfilled with cuttings.							
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PLATE

EXPLORATION LOG

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PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:		Seaside Shores, LLC				Boring No.:		HA-8		
Location:		2155 Crestview, Laguna Beach				Elevation:		325±		
Job No.:		23-299		Client:		Coulter Milner & Co		Date:		9/22/23
Drill Method:		Hand Auger		Driving Weight:		N/A		Logged By:		SS
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples			Laboratory Tests			
				Blows per 6 in.	C o r e	B u i k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests	
0		ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, slightly moist, medium dense, fine- to coarse-grained, some gravel.								
		BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive.								
5		Total Depth = 2.1' Groundwater not encountered during drilling Backfilled with cuttings.								
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PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

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PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project: Seaside Shores, LLC		Boring No.: HA-10						
Location: 2155 Crestview, Laguna Beach		Elevation: 305±						
Job No.: 23-299	Client: Coulter Milner & Co	Date: 9/22/23						
Drill Method: Hand Auger	Driving Weight: N/A	Logged By: SS						
Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples		Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)
0		ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, slightly moist, medium dense, fine- to coarse-grained, some gravel.						
		BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive.						
5		Total Depth = 1.6' Groundwater not encountered during drilling Backfilled with cuttings.						
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EXPLORATION LOG

Project: Seaside Shores, LLC		Boring No.: HA-11	
Location: 2155 Crestview, Laguna Beach		Elevation: 314±	
Job No.: 23-299	Client: Coulter Milner & Co		Date: 9/22/23
Drill Method: Hand Auger	Driving Weight: N/A		Logged By: SS

Depth (Feet)	Lith- ology	Material Description	W A T E R	Samples		Laboratory Tests		
				Blows per 6 in.	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)
0		ARTIFICIAL FILL, undocumented (afu) Clayey Sand (SC): Brown, slightly moist, medium dense, fine- to coarse-grained, some gravel. BEDROCK - San Onofre Formation (Tso) Silty Sandstone: Light brown to yellow, fine- to coarse-grained, moderately hard to hard, massive. Total Depth = 1.1' Groundwater not encountered during drilling Backfilled with cuttings.						
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PLATE

APPENDIX B

LABORATORY TEST PROCEDURE

LABORATORY DATA SUMMARY

LABORATORY TEST PROCEDURES

Soil Classification

Soil materials encountered within the property were classified and described in accordance with the Unified Soil Classification System and in general accordance with the current version of Test Method ASTM D 2488. The assigned group symbols are presented in the exploration logs, Appendix A.

In Situ Moisture Content and Dry Unit Weight

In-place moisture content and dry unit weight of selected, relatively undisturbed soil samples were determined in accordance with the current version of the Test Method ASTM D 2435 and Test Method ASTM D2216, respectively. Test data are presented on the exploration logs, Appendix A.

Expansion Index

An expansion index test was performed on a selected bulk sample of the on-site soils in accordance with the current 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 a selected bulk sample 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 current versions of California Test Method Nos. CTM 417, CTM 422 and CTM 643, respectively. The results of these tests are included on Plate B-1.

Atterberg Limits

The Atterberg limits (liquid limit and plastic limit) were determined for a selected bulk sample of representative onsite soils materials in general accordance with the current version of Test Method ASTM D 4318. The results of these tests are included on Plate B-1.

LABORATORY DATA SUMMARY													
Boring Number	Sample Depth (ft)	Soil Description	Max. Dry Density ¹ (pcf)	Optimum Moisture ¹ (%)	Expansion Index ²	CBC Soil Classification ³	Atterberg Limits ⁴			Sulfate Content ⁵ (%)	Chloride Content ⁶ (mg/L)	pH ⁷	Minimum Resistivity ⁷ (Ohm-cm)
							LL	PL	PI				
HA-2	1.5	Clayey Sand (SC)	-	-	26	Low	33	19	14	0.0084	232.5	8.5	1,800

Note: Laboratory data pertaining to in-place soil moisture content and dry density are provided on the exploration logs included in Appendix A of this report.
 (-) Test not performed

Test Procedures:	¹ Per ASTM Test Method D 1557	⁵ Per Caltrans Test Method 417
	² Per ASTM Test Method D 4829	⁶ Per Caltrans Test Method 422
	³ Per ASTM Test Method D 4829 Table 1, Per CBC 2013	⁷ Per Caltrans Test Method 643
	⁴ Per ASTM Test Method D 4318	⁸ Per ASTM Test Method D 1140

APPENDIX C

SEISMIC DESIGN ANALYSIS

⚠ This is a beta release of the new ATC Hazards by Location website. Please [contact us](#) with feedback.

ℹ The ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

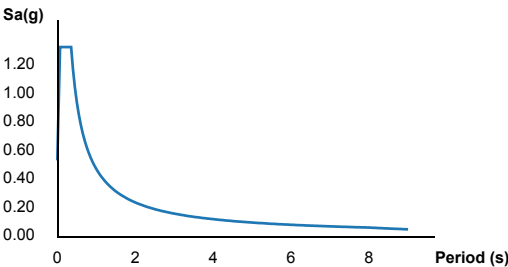
ATC Hazards by Location

Search Information

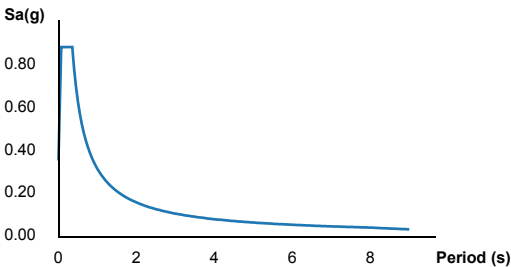
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Elevation: 301 ft
Timestamp: 2023-11-14T04:12:34.751Z
Hazard Type: Seismic
Reference Document: ASCE7-16
Risk Category: II
Site Class: B-estimated



MCE_R Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S _S	1.325	MCE _R ground motion (period=0.2s)
S ₁	0.47	MCE _R ground motion (period=1.0s)
S _{MS}	1.325	Site-modified spectral acceleration value
S _{M1}	0.47	Site-modified spectral acceleration value
S _{DS}	0.883	Numeric seismic design value at 0.2s SA
S _{D1}	0.313	Numeric seismic design value at 1.0s SA

▼Additional Information

Name	Value	Description
SDC	D	Seismic design category
F _a	1	Site amplification factor at 0.2s
F _v	1	Site amplification factor at 1.0s
CR _S	0.909	Coefficient of risk (0.2s)
CR ₁	0.925	Coefficient of risk (1.0s)
PGA	0.581	MCE _G peak ground acceleration
F _{PGA}	1	Site amplification factor at PGA
PGA _M	0.581	Site modified peak ground acceleration
T _L	8	Long-period transition period (s)
SsRT	1.325	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.457	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.869	Factored deterministic acceleration value (0.2s)
S1RT	0.47	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.508	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.979	Factored deterministic acceleration value (1.0s)

PGAd	1.176	Factored deterministic acceleration value (PGA)
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The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

Disclaimer

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