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Environmental Scientists

January 27, 2014
J.N. 13-283
Revision 1

Mr. Chris Hermann
HERMANN DESIGN GROUP, INC.
78365 Highway 111, PMB 332
La Quinta, California 92253

Subject: **Revised Geotechnical Recommendations for Design and Construction of Retaining Walls, *Whitewater Park Expansion: Amphitheater Project*, 71560 San Jacinto Drive, Rancho Mirage, California.**

Reference: Preliminary Geotechnical Investigation, *Whitewater Park Expansion*, 71560 San Jacinto Drive, City of Rancho Mirage, Riverside County, California, report by Petra Geotechnical, Inc. dated November 8, 2013.

Dear Mr. Hermann:

Petra Geotechnical, Inc. (Petra) is pleased to submit herewith our recommendation for design and construction of new retaining walls on the subject properties. Specifically, we are in receipt of a set of Architectural plans for the project prepared by McAuliffe & Associates, Inc. dated January 14, 2014. Sheets A2.0, A2.5 through A2.7 and A3.0 depict retaining wall locations and cross sections. Based on these plans, we understand that a majority of these retaining walls are retaining relatively low height (retaining less than 6 feet) backfill.

One exception with respect to retaining height is a ramp/ retaining wall that are depicted on Sheet A3.0 (northerly of Grid J' and between Grids 5' and 7') and on Sheet A2.6 (Detail 4). This wall is retaining approximately 7.5 feet of backfill. The second exception is a retaining wall along Grid 13' between Grids D' and J' (Sheet A3.0) with details shown on Sheet A4.5. This wall is retaining approximately 12 feet of backfill. Based on these information and those contained in the referenced report, the following presents our recommendations for design and construction of these retaining walls.

RECOMMENDATIONS

Earthwork

All vegetation within the area of proposed wall construction should be stripped and removed from the site. All trash and any construction debris should also be removed from the area of construction.

Laborers should manually remove any vegetation and other deleterious materials during clearing and grubbing operations.

Provided that the wall construction to commence after the completion of the site grading as recommended in the referenced report, it is expected that excavation of the site down to the proposed grades of the wall footings will expose competent compacted fill soils or native soils. These materials are suitable to support the proposed retaining walls provided that the footings are supported uniformly and entirely by any of these materials. As such, any transition from one material to another below the proposed footings should be eliminated. Similarly, if any unsuitable surficial fill is exposed at proposed grades, they should be excavated down to competent soils and then replaced as properly compacted fill.

The majority of the proposed retaining walls are expected to be supported on the compacted fill soils. Transition from compacted fill to native materials may be encountered. Native materials, if encountered in retaining wall footings excavation should be overexcavated to a minimum of 2 feet below the bottom of the footings and replaced as compacted fill. Should native materials encountered in the wall shear key trenches only (if shear keys are considered), these materials should be overexcavated a minimum of one foot below the bottom of the shear key and replaced with compacted fill.

After the walls are constructed, they should be backfilled with compacted fill. As the backfill progresses upward, the newly compacted fill should be placed on a series of level benches that tie into the competent previously compacted fill. Each bench should have an approximate horizontal width of 2 to 3 feet and a corresponding near vertical backcut that is no more than approximately 2 to 3 feet high. The actual dimensions should be determined during grading.

To provide drainage, subdrains should be installed behind each of the new retaining walls. The subdrains should be installed in accordance with the drainage recommendations provided in the "Retaining Wall Design Recommendations" section of this report.

Fill Placement

All fill should be placed in 4- to 6-inch-thick lifts, watered or air dried as necessary to achieve near optimum moisture conditions and then compacted in place to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D1557.

Geotechnical Observations

Exposed bottom surfaces in each removal/ footing excavation area should be observed and approved by the project geotechnical consultant prior to placing new fill or footings. 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 verify proper placement and adequate compaction of fill, as well as to verify compliance with the other recommendations presented herein.

Stability of Temporary Excavation Sidewalls

During site grading, a temporary excavation with sidewalls varying from approximately 2 to 5 feet (or higher) in height may be created during construction of the retaining walls. Based on the physical characteristics of these onsite materials, temporary slopes may be excavated vertically but should not exceed a height of approximately 4 feet. Where excavations exceed this height, the lower 4 feet may be cut vertical and the upper portions above a height of 4 feet should be cut back at a maximum gradient of 1:1, horizontal to vertical, or flatter.

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 that 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.

2013 CBC Seismic Design Parameters

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2013 CBC. The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height and is based on the 2012 International Building Code. For structural design in accordance with the 2013 CBC, the online USGS Seismic Design Map tool was utilized to provide ground-motion parameters for the subject site. Based on the latitude, longitude and site classification, seismic design parameters and spectral response for both short periods and 1-second periods are calculated including Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter.

The following 2013 CBC seismic design coefficients should be used for the proposed structures. These criteria are based on the site class as determined by existing subsurface geologic conditions, on the proximity of the site to the nearby faults and on the maximum moment magnitude of the nearby faults.

2013 CBC Section 1613 Seismic Design Coefficients	
Site Latitude	33.7463
Site Longitude	-116.4138
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.500 g
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.656 g
Site Class Definition (Table 1613.5.2)	D
Site Coefficient, F_a (Table 1613.5.3(1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3(2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-36)	1.500 g
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-37)	0.984 g
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	1.000 g
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.656 g
Site Coefficient, F_{PGA} (Figure 22-7 ASCE 7-10)	1.0
Peak Ground Acceleration PGA_M (Equation 11-8.1 ASCE 7-10)	0.570 g

Retaining Wall Design Recommendations

Allowable Bearing and Lateral Resistance Values

Provided that remedial grading is performed as recommended herein, the retaining walls may be designed as conventional walls that are supported on shallow footings supported by either native soils or compacted fill. For this condition, an allowable bearing value of 1,500 pounds per square foot is recommended for design of 12-inch-wide continuous footings founded at a minimum depth of 12 inches into competent fill materials. This value may be increased by 20 percent for each additional foot of footing width and/or depth to a maximum value of 2,500 pounds per square foot. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third when designing for short duration wind and seismic forces.

For footings founded in competent native soils or compacted fill, 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, should be utilized; 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. A coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting fill materials to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces.

The above values are based on footings placed directly against competent native soils or compacted fill. In the case where footing sides are formed, all backfill placed against the footings should be compacted to at least 90 percent of maximum dry density.

Active and At-Rest Earth Pressures

As of the date of this report, it is uncertain whether the proposed retaining walls will be backfilled with on-site soils or imported granular materials. For this reason, active and at-rest earth pressures are provided below for both conditions. However, considering that the onsite earth materials have a very low to low expansion potential, the use of imported granular materials for backfilling behind the retaining walls as described in the following sections is optional.

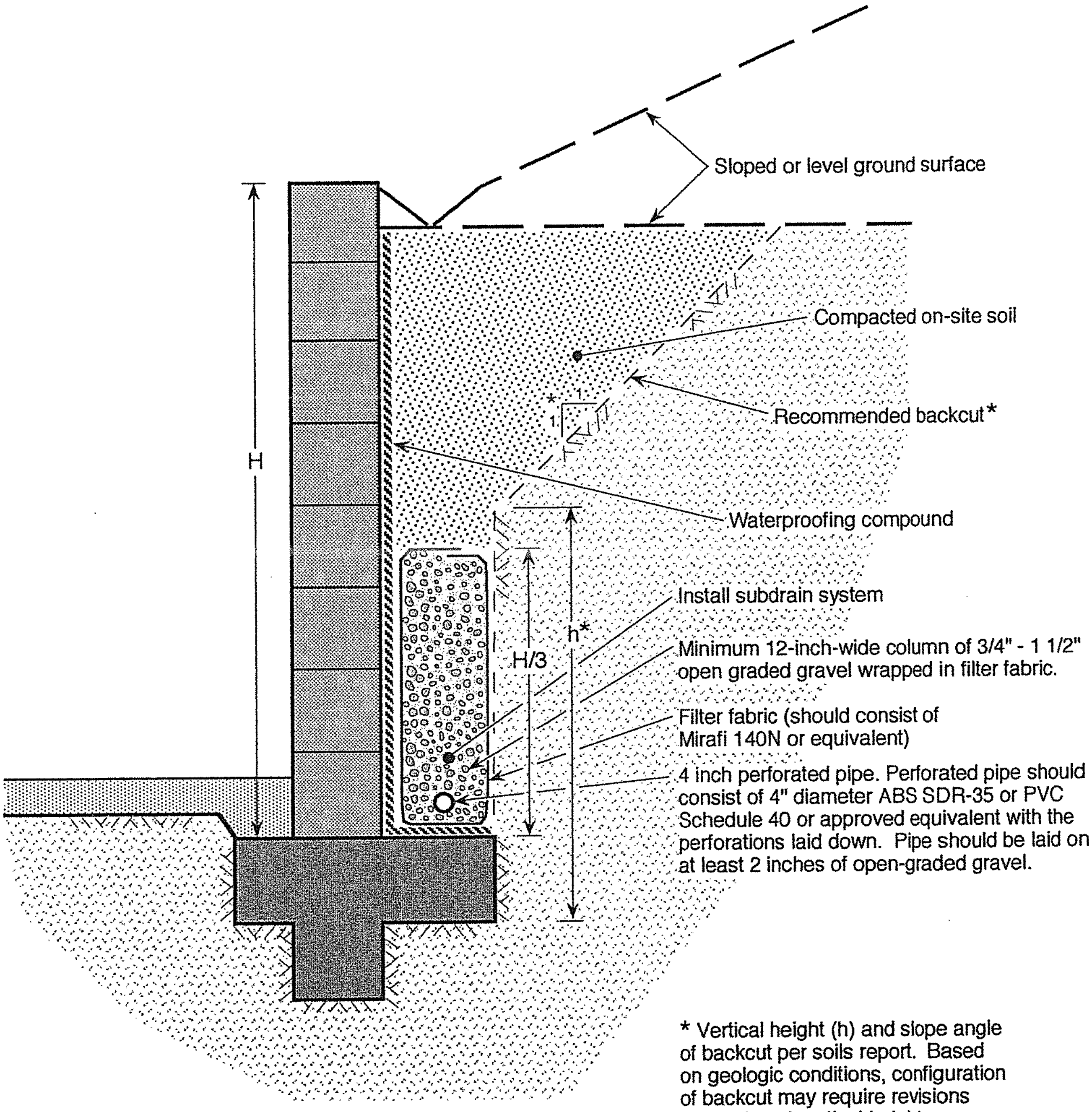
1. Onsite Soils Used for Backfill

Onsite soils are very low to low expansive. Therefore, 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. These values may be reduced to 35 and 51 pounds per cubic foot, respectively, for design of cantilevered walls that will be retaining 6 feet of backfill in height or less. For walls that are restrained at the top, at-rest earth pressures of 60 and 62 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

Imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures 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

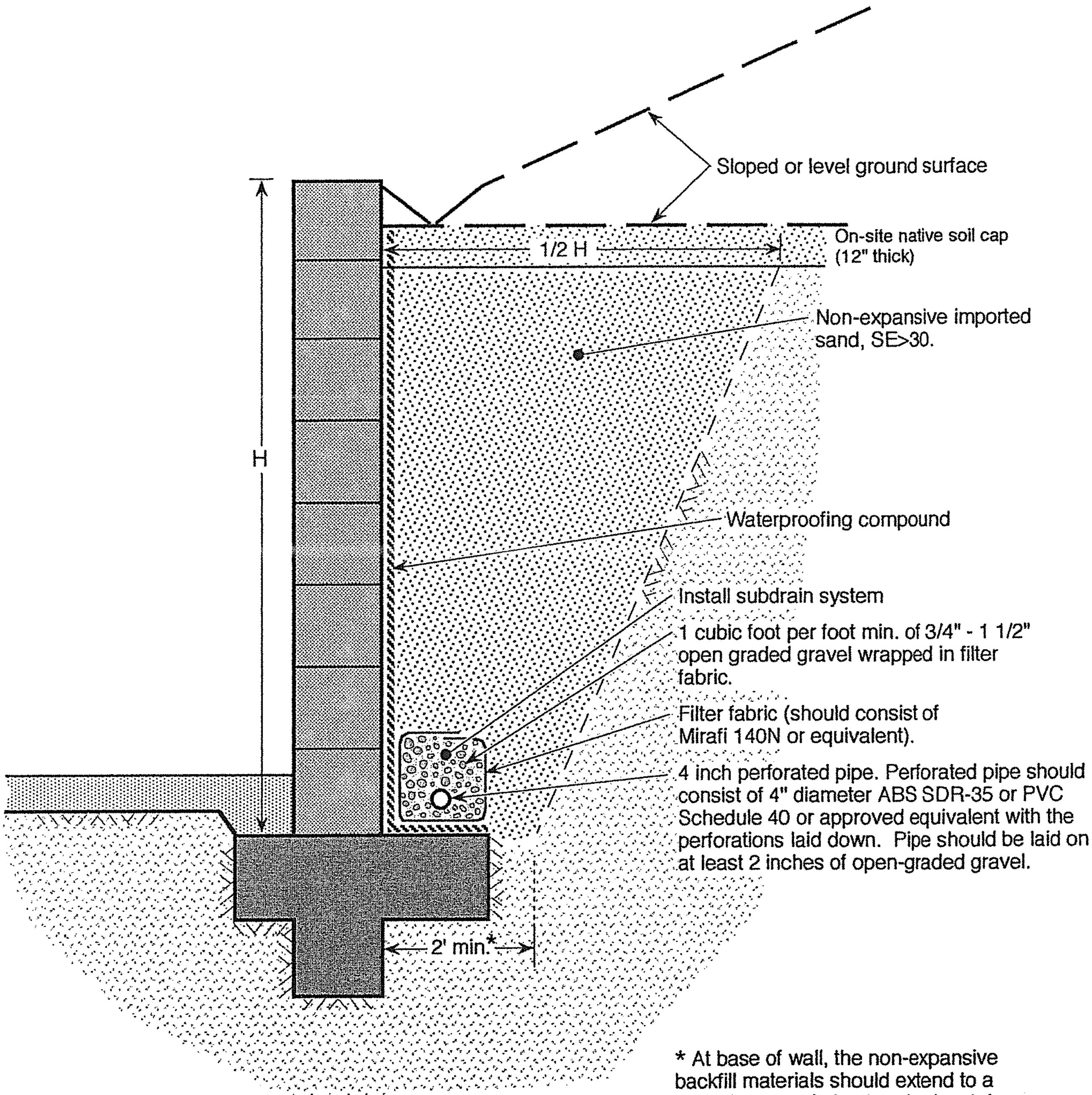
NATIVE SOIL BACKFILL



* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e. reduced vertical height, revised slope angle, etc.)

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-1

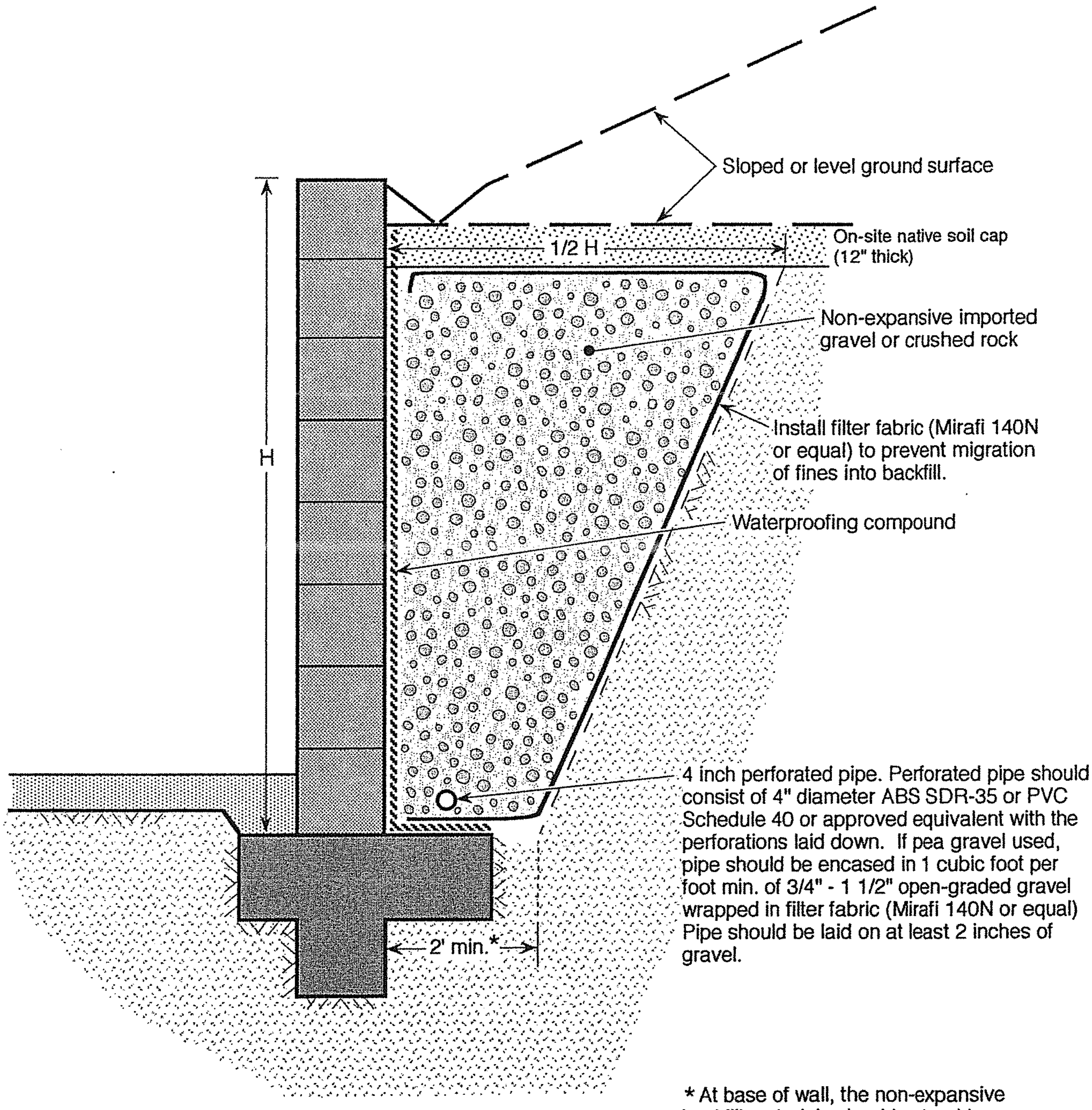
IMPORTED SAND BACKFILL



* 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.

PETRA	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.

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-3

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.

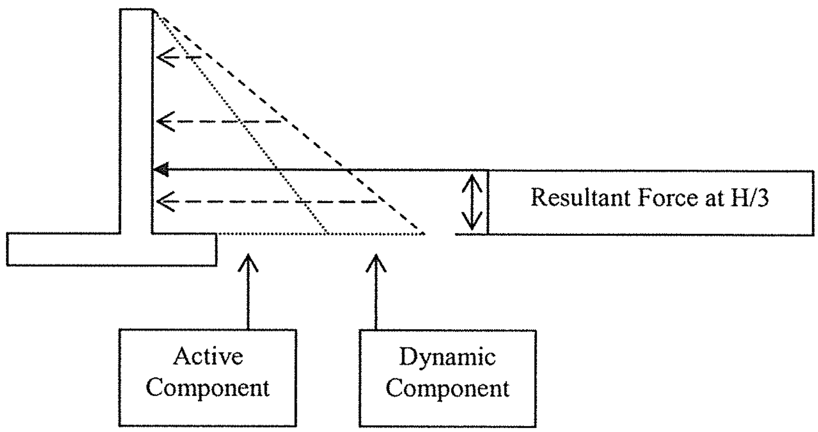
All structural calculations and details should be provided to the project geotechnical consultant for verification purposes prior to grading and construction phases.

Earthquake Loads on Retaining Walls

Section 1803.5.12 of the 2013 CBC requires the determination of lateral loads on retaining walls from earthquake forces for structures in seismic design categories D through E that are supporting in excess of 6 feet of backfill height. The following presents our recommendations for the determination of dynamic seismic lateral pressure for the design of retaining walls at the subject site.

It is customary to assume the horizontal ground acceleration vale k_h to be equal to half of the peak ground acceleration. (See, for example, County of Los Angeles, Department of Public Works, Manual for Preparation of Geotechnical Reports, July 1, 2013.) Thus, $k_h = ½ (PGA) = ½ (0.570 g) = 0.285 g$.

In our evaluation of the earthquake loads on retaining walls, we used the Mononobe-Okabe procedure to determine the total (static and dynamic combined) lateral loads on the retaining walls. Based on our analysis, a dynamic load equivalent to a fluid having a unit weight of 25 pcf should be used. Note that the active and seismic earth pressures have a triangular distribution with the largest load occurring at the bottom of the wall (Al Atik, Linda, and Sitar, Nickolas, 2007, Development of Improved Procedures for Seismic Design of Buried and Partially Buried Structures, PEER 2007/06, dated June).
The distribution of the seismic lateral load is as follows



Geotechnical Observation and Testing

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

Drainage and Waterproofing

Perforated pipe and gravel subdrains should be installed behind all 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 3/4-inch to 1½-inch open-graded gravel. If onsite soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-half 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 rocks are used as backfill, the open-graded gravel should extend above the wall footing to a minimum height of 1 foot above the footing. 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. 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. However, since the on site soil materials are expected to be low to moderately expansive, they 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 sand, pea gravel, crushed rock, or imported granular soils for backfill that exhibit a Very Low expansion potential (Expansion Index of less than 20). 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 that they are installed as shown on Figures RW-2 and RW-3.

Where on-site soils or imported sand are used for backfill, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve optimum or slightly above 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 that adequate compaction has been achieved.

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 mitigate 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 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.

We appreciate this opportunity to be of service. If you have questions, please contact this office.

Respectfully submitted,
PETRA GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
SIAMAK JAFROUDI
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Siamak Jafroudi, PhD, GE 2024
Senior Principal Engineer

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Environmental Scientists

November8 2013
J.N. 13-283

Mr. Chris Hermann
HERMANN DESIGN GROUP, INC.
78365 Highway 111, PMB 332
La Quinta, California 92253

Subject: **Preliminary Geotechnical Investigation, *Whitewater Park Expansion*, 71560 San Jacinto Drive, City of Rancho Mirage, Riverside County, California.**

Dear Mr. Hermann:

In accordance with your request and authorization, **Petra Geotechnical, Inc. (Petra)** is pleased to submit herewith our preliminary geotechnical investigation report for the Whitewater Park Expansion Project. The subject property is located at 71560 San Jacinto Drive (Figure 1). This work was performed in general accordance with the scope of work outlined in our Proposal No. 13-283P dated March 3, 2013. This report presents the results of our field exploration, laboratory testing, and our engineering judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects for the proposed expansion project.

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

Respectfully submitted,

PETRA GEOTECHNICAL, INC.

Siamak Jafroudi, GE
President, Senior Principal Engineer

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PRELIMINARY GEOTECHNICAL INVESTIGATION, WHITEWATER PARK EXPANSION, 71560 SAN JACINTO DRIVE, CITY OF RANCHO MIRAGE, RIVERSIDE COUNTY, CALIFORNIA

This report presents the results of Petra Geotechnical, Inc.'s (Petra) preliminary geotechnical investigation for the proposed park improvements. The project is known as the Whitewater Park Expansion, located at 71560 San Jacinto Drive, in the City of Rancho Mirage, Riverside County, California. This investigation included a review of published and unpublished literature, site reconnaissance and subsurface exploration, as well as a review of geotechnical maps pertaining to geologic hazards which may have an impact on the proposed construction.

Purpose and Scope of Services

The purposes of this study were to obtain preliminary information on the subsurface geologic and soil conditions within the project area, evaluate the field and laboratory data and provide conclusions and preliminary geotechnical recommendations for design and construction of the proposed site improvements as influenced by the subsurface conditions encountered.

The scope of our evaluation consisted of the following.

- Review of available published and unpublished geologic data, maps, available online aerial imagery and geotechnical reports concerning geologic and soil conditions within, and adjacent to the site which could have an impact on the proposed improvements.
- Reconnaissance of the property to evaluate existing onsite conditions.
- Excavate five (5) exploratory borings, utilizing a truck-mounted, hollow-stem auger drill rig, to evaluate the stratigraphy of the subsurface soils and collect representative undisturbed and bulk samples for laboratory testing.
- Log and visually classify soil materials encountered in the hollow-stem auger borings in accordance with the Unified Soil Classification System.
- Conduct two, relatively shallow (15 feet below grade) percolation tests to evaluate the infiltration rate of the near-surface onsite soils for preliminary design of two on-site dry wells.
- Conduct laboratory testing of representative samples (bulk and undisturbed) obtained from the hollow-stem auger borings to determine their engineering properties.
- Perform appropriate engineering and geologic analysis of the data with respect to the proposed improvements.

- Preparation of this report, including pertinent figures and appendices, presenting the results of our evaluation and recommendations for the proposed improvements in general conformance with the requirements of the 2010 California Building Code (CBC), as well as in accordance with applicable local jurisdictional requirements.

Location and Site Description

The subject site is an irregularly shaped public park located at 71560 San Jacinto Drive in the City of Rancho Mirage, Riverside County, California. The site is currently occupied by several structures including bathroom facilities, a maintenance building, a historic landmark building, as well as basketball, tennis and handball courts, shade structures, playgrounds, a fountain, and associated walking paths, parking areas, grass lawns and other landscaping. The project includes improvements to the existing park areas as well as development of the currently vacant dirt lot located immediately northwest of the developed area. The location of the site is shown on Figure 1.

Topographically, site elevations range from a high of approximately 232 feet above mean sea level (msl) near the northwest property corner to a low of approximately 222 feet below msl near the southeast corner, overall relief is on the order of 10 feet. Vegetation within the developed park area consists of large trees, bushes and grass. Vegetation in the undeveloped western portion of the site is comprised of very sparse desert scrub brush and sporadic native weeds and grasses.

Proposed Construction

The City proposes to expand the Park and implement several improvements including a small dog park, an amphitheatre, relocating and expanding parking areas, and redesigning the Park to provide for these improvements). The amphitheater area will consist of several buildings, which are listed below:

- Stage: 2,030 square feet
- Temporary Storage/Stage Wings: 910 square feet
- City Storage: 690 square feet
- Public Restrooms/Janitor Closet/Support Space: 950 square feet
- Dressing & Principal Dressing Rooms: 905 square feet
- Back of House Restrooms: 370 square feet

The amphitheater will be tiered in elevation and provide formal seating as well as informal sloped/tiered lawn seating. The amphitheater area is approximately 25,000 square feet, which includes the support buildings, stage and seating areas. Two landscaped open space areas will be constructed just west and southeast of the proposed amphitheater, which can be used for vendor booths during special events.

LEGEND

- Site Location

Reference: Base map from Bing.com

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COSTA MESA TEMECULA PALM DESERT SAN DIEGO SANTA CLARITA

SITE LOCATION MAP

Whitewater Park
San Jacinto Drive
Rancho Mirage, CA

DATE: Nov. 2013	J.N.: 13-283	Figure 1
DWG BY: AGW	SCALE: NTS	

Assumed Foundation Loading Conditions

The tentative foundation recommendations presented herein may be considered appropriate for lightly loaded foundations. It is our understanding that the proposed structures will consist of an amphitheatre, a storage room, public restrooms, janitor closet, dressing rooms, and back of house restrooms, using wood-framing and conventional slab-on-ground foundations. Building loads are assumed to on the order of 1 to 2 kips per foot for continuous footings and up to 10 kips for isolated pad footings. Supplemental design information will be required for more heavily loaded foundations, as would be the case with multi-story, masonry, or steel-framed structures.

Literature Review

Petra researched and reviewed available published and unpublished geologic data, maps and aerial imagery pertaining to regional geology, faulting and geologic hazards that may affect the site. The results of this review are discussed under Findings presented in a following section of this report.

Subsurface Exploration

A subsurface exploration program was performed under the direction of an engineering geologist from Petra on August 30, 2013. The exploration involved the excavation of five (5) exploratory borings (B-1 through B-5) to a maximum depth of approximately 51.5 feet below existing grades. Borings B-2 and B-5 were developed and used for preliminary percolation/infiltration testing. The borings were advanced utilizing a track-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Earth materials encountered within the exploratory borings were classified and logged by an engineering geologist in accordance with the visual-manual procedures of the Unified Soil Classification System (USCS), ASTM Test Standard D2488. The approximate locations of the exploratory borings are shown on Figure 2 and logs for the borings are presented in Appendix A.

Disturbed bulk samples and relatively undisturbed ring samples of in-situ soil materials were collected from the exploratory borings for classification, laboratory testing and engineering analyses. Undisturbed samples were obtained using a 3-inch outside diameter modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a free-fall, 140-pound automatic trip hammer. The central portions of the driven-core samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches into the soil were recorded for each 6-inch driving increment; however, the number of blows required to drive the sampler for the final 12 inches was noted in the boring logs as *Blows per Foot* (Appendix A).

Percolation/Infiltration Testing

Two percolation tests were conducted at a depth of 20 feet below grade using borings B-2 and B-5, as a pilot test for determination of infiltration rate of the near-surface onsite soils for preliminary design of the proposed dry wells. This test was performed in general accordance with the Riverside County Department of Environmental Health and/or the 2009 Uniform Plumbing Code ([2009 UPC], International Association of Plumbing & Mechanical Officials, 2009) guidelines for percolation testing.

The falling-head percolation test data was utilized in determining the test infiltration rate, I_t, expressed in units of inches/hour, utilizing the Porchet Method (RCFCWCD, 2011). Test results are summarized in the Geologic/Geotechnical Considerations section below and presented in Appendix C.

Laboratory Testing

The laboratory testing program included the determination of in-situ dry density and moisture content, maximum dry density and optimum moisture content, direct shear strength, R-value and preliminary soil corrosivity screening (soluble sulfate and chloride content, pH and minimum resistivity). A description of laboratory test methods and summaries of the laboratory test data are presented in Appendix B and the in-situ dry density and moisture content results are presented on the boring logs (Appendix A).

FINDINGS

Regional Geologic Setting

The proposed development is located in the Coachella Valley, which is part of the Salton Trough geomorphic province of California. The Salton Trough geomorphic province encompasses the Coachella, Imperial and Mexicali Valleys, which extend from northeast of Palm Springs near San Gorgonio Pass to the Gulf of California. The geologic structure of the trough is a result of extensional forces within the earth's crust. The Coachella Valley is generally bounded by the San Jacinto and Santa Rosa Mountains on the west, the San Bernardino and the Little San Bernardino Mountains on the north, the Cottonwood Mountains and the Mecca Hills on the east, and the Salton Sea to the south. Alluvial (Streams), aeolian (wind-blown), and lacustrine (lake) sediments are the dominant geologic units of the Coachella Valley.

The watershed of the Coachella Valley empties into the Salton Sea at the lowest part of the basin. This basin was periodically filled with water to form the ancient Lake Cahuilla, depending on which side of its delta the Colorado River would drain. The sediments of the delta form a topographic high that separates the Salton basin, which is below sea level, from the Gulf of California (Sea of Cortez).

More specifically, the site lies adjacent to the intermittent Whitewater River, dry during our investigation, and is bisected by a northwest to southeast trending contact between alluvial and river deposits.

Local Geology and Subsurface Soil Conditions

The project site is underlain by fluvial and alluvial deposits consisting of poorly-graded sands, silty sands, and to lesser extent, sandy silts and silt. In the developed park area these sediments are mantled by a relatively thin layer of artificial fill. In general, the deposits were found to be dry to moist and medium dense to dense with occasional loose lenses.

Groundwater

Free groundwater was not encountered within any of the exploratory borings advanced onsite to the maximum depth explored of 51.5 feet below grades. Based on our review of Figure 2 of 2013 Coachella Valley Water District Engineers Report, the regional ground water table has been measured to be over 200 feet below ground surface in a monitoring well near Country Club Drive and Bob Hope Drive in Palm Springs.

Faulting

The Coachella Valley is a seismically active area and numerous northwest-trending active faults have been documented within the area. The San Andreas fault zone is the most prominent fault within the Coachella Valley, and is considered to be "active". An "active" fault is defined as a fault that has had displacement within the Holocene epoch, or last 11,000± years. Based on our review, the site is not located within a *Fault Hazard Zone* (Bryant and Hart, 2007), as defined by the state of California in the Alquist-Priolo Earthquake Fault Zoning Act and no evidence for faulting was observed within the site during our study.

CONCLUSIONS AND RECOMMENDATIONS

General

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

Grading and Foundation Plan Review

It must be emphasized that the recommendations provided throughout this report are based solely on the conceptual information provided to us, and that no definitive grading plans, structural plans or details were available for review as of the date of this report. As such, the conclusions and recommendations provided herein should be considered as tentative. Once such plans and details become available, our firm should be retained to review these documents to determine the applicability of our recommendations to the actual construction proposed. Additional investigation, recommendations and/or modification of the recommendations provided herein will be provided, if necessary, depending on the results of the grading plan and/or structural plan review.

Geologic/Geotechnical Considerations

Infiltration Rate

The pilot percolation tests conducted on-site yielded a test infiltration rate, I_t, of 6 inches per hour at the B-2 location and 1.3 inches per hour at the B-5 location. This rate is unfactored and should be considered preliminary in nature. Additional testing may be necessary once the basin location(s), depths, etc. have been determined to finalize the design infiltration rate, I_d.

Groundwater

Based on our review, adverse effects on the proposed construction due to shallow regional groundwater conditions are currently not anticipated. However, seepage and perched groundwater conditions may occur onsite due to excess irrigation, migration from adjacent drainage areas and developments during and/or after periods of above normal or heavy precipitation. Thus, seepage and perched water conditions may occur in the future, and should be anticipated. Should manifestations of seepage and/or perched water conditions develop within the site in the future, Petra should assess the conditions and provide mitigation recommendations, as necessary.

Slope Erosion

The site is bounded to the north by the Whitewater River. This streambed fills with water during heavy rains and flash-flood conditions which could lead to erosion of the southern bank which bounds the site to the north. Over time, erosion of these banks could encroach back into the subject site or lead to over-steepened and unstable slopes.

Flooding

The site is bounded to the north by the Whitewater River. Seasonal heavy rains and flash flood conditions could lead to overflow of the river bank located along the northern boundary of the site and subsequent flooding.

Fault Rupture

The closest known fault is the San Bernardino Segement of the San Andreas fault, which is located approximately 7 miles northeast of the site. The site is not located within a currently designated State of California Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). In addition, no known active faults have been identified on or trending toward the site. While fault rupture would most likely occur along established fault traces, fault rupture could occur at other locations. The potential for active fault rupture at the site is considered to be very low.

Seismic Shaking

The site is located within an active tectonic area with several significant faults capable of producing moderate to strong earthquakes. The San Andreas fault and the San Jacinto fault are both in close proximity to the site and capable of producing strong ground motions. Onsite structures will likely be subjected to strong motions over the life of the project.

Secondary Effects of Seismic Activity

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure, as well as earthquake-induced flooding. Various general types of ground failures, which might occur as a consequence of severe ground shaking at the site, include ground subsidence, ground lurching and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoil and groundwater conditions, in addition to other factors. Based on the site conditions and relatively flat topography, ground subsidence, ground lurching and lateral spreading are considered unlikely at the site.

Seismically induced flooding that might be considered a potential hazard to a site normally includes flooding due to tsunami or seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. The nearest enclosed body of water is the Salton Sea, situated approximately 25 miles from the site and approximately 449 feet below average site elevation. No major reservoir is located near, or

Based on a review of the Riverside County Land Information System, the potential for liquefaction occurring in the general area of the site is deemed to be moderate. However; based on the lack of shallow groundwater, generally dense underlying native soils, and recommended engineered fills, the potential for liquefaction is considered low.

Earthwork

General Earthwork Recommendations

Earthwork should be performed in accordance with the applicable provisions of the 2010 CBC. Grading should also be performed in accordance with the following site-specific recommendations prepared by Petra.

Geotechnical Observations and Testing

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

Clearing and Grubbing

All vegetation onsite and any trash or debris in areas to be graded should be removed from the site; this includes organic rich soils excavated from the grass areas of the existing park. During site grading, fill soils should be cleared of any deleterious materials that are missed during the initial clearing and grubbing operations. Any cavities or excavations created upon removal of subsurface structures should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment and then backfilled with properly compacted fill.

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

Ground Preparation – Foundation Areas

Based on soil conditions observed in the exploratory borings, surface soils over a majority of the site are medium dense to dense below in the upper approximately 1 to 3 feet. In areas where structures are to be supported by conventional shallow slab-on-grade foundations, spread footings and/or post-tension foundations, the existing ground should be over-excavated to depths that expose competent native soils exhibiting an in-place relative compaction of 85 percent or more, based on Test Method ASTM D1557.

Based on above, the required depths of over-excavation together with the exposed bottom treatment are anticipated to vary from approximately 2 to 3 feet, excluding undocumented artificial fills. The horizontal limits of over-excavation should extend to a minimum distance of 5 feet beyond the proposed perimeter foundation lines or to a horizontal distance equal to the depth of over-excavation, whichever is greater.

Due to the variability of the surficial soil conditions, the required depths of over-excavation will have to be determined during grading on a case-by-case basis. In addition to removal of unsuitable soils and prior to placing compacted fill, the exposed bottom surfaces in all over-excavated areas should be observed and approved by the project geotechnical consultant. Following this approval, the exposed bottom surfaces should be scarified to a depth of approximately 8 inches, watered as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then re-compacted in-place to a minimum relative compaction of 90 percent.

Ground Preparation – Roadways, Parking and Sheet-Graded Areas

The existing ground in proposed roadway areas to be paved with asphaltic concrete should be over-excavated and re-compacted in a similar manner as recommended above, however the depth of overexcavation can be reduced to two feet, below existing ground, excluding undocumented artificial fills. In areas to be graded to a sheet flow condition for drainage purposes and *where no structures are planned*, the existing ground should be scarified to a depth of 8 inches, watered as necessary to achieve a moisture content that is equal to, or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent.

Cut Areas

Cuts that extend to depths greater than approximately 1 to 2 feet below existing grade are anticipated to expose, competent native soils. However, due to variability in moisture content and cohesionless nature of the earth materials encountered across the site, cuts in structural areas should be overexcavated to a minimum depth of 3 feet and replaced with fill compacted in-place to a minimum relative compaction of 90 percent. Shallower removals may be appropriate where exposed soil conditions, following the cut, are deemed to be suitable as determined by the engineering geologist.

Protection of Adjacent Properties

In order to protect the existing structures located along the property lines; if temporary excavations with sidewalls are created as a result of overexcavation and recompaction of low-density surficial soils during remedial grading of the site; it is recommended that the sidewalls of the excavation be laid back at a slope ratio of 1:1, horizontal to vertical, with the top of the cut located at least 12 inches away from the property line structures.

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

Exposure to moisture through excessive offsite irrigation or rainfall during the period that the excavation remains open could result in sidewall instability and should therefore be prevented. 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, the 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.

Fill Placement and Testing

All fill should be placed in lifts not exceeding 6 to 8 inches in thickness, watered as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to a minimum relative compaction of 90 percent. 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. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D1557.

2. Section 1804.3 of the 2010 California Building Code requires that "the ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall." Further, "swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation." These provisions fall under the purview of the design civil engineer. However, exceptions to allow modifications to these criteria are provided within the same section of the code as "Where climatic or soil conditions warrant, the slope of the ground away from the building foundations is permitted to be reduced to not less than one unit in 48 units horizontal (2 percent slope)." This exemption provision appears to fall under the purview of the Geotechnical Engineer of Record.

   It is our understanding that the state-of-the-practice for projects in various cities and unincorporated areas of riverside County, as well as throughout Southern California, has been to construct earthen slopes at 2 percent gradient away from the foundations and at 1 percent minimum for earthen swale gradients. Structures constructed and properly maintained under those criteria have performed satisfactorily. Therefore, considering the semi-arid climate, site soil conditions and an appropriate irrigation regime, Petra considers that the implementation of 2 percent slopes away from the structures and 1 percent swales to be suitable for the subject lots.

   The City should be cautioned that the slopes away from the structures and swales must be properly maintained, are not to be obstructed, and that future improvements must not to alter established gradients unless replaced with suitable alternative drainage systems. Further, where the flow line of the swale exists within five feet of the structure, adjacent footings shall be deepened appropriately to maintain minimum embedment requirements, measured from the flow line of the swale.

3. Concrete flatwork surfaces located within 10 feet of building foundations should be inclined at a minimum gradient of 2 percent away from building foundation and similar structures. Concrete flatwork surfaces located more than 10 feet from building foundations may be inclined at a minimum gradient of 1 percent away from building foundation and similar structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations.

4. For the landscape areas, a watering program should be implemented that maintains a uniform, near optimum moisture condition in the soils. Overwatering and subsequent saturation of the soils will cause excessive soil expansion and heave and, therefore, should be avoided. However, allowing the soils to dry out will cause excessive soil shrinkage. As an alternative to a conventional irrigation system, a drip irrigation system is strongly recommended for all planter areas. The owner is advised that all drainage devices should be properly maintained throughout the lifetime of the development.

Utility Trench Backfill

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Onsite earth materials contain excessive amounts of fines and cannot be densified adequately by flooding and jetting techniques. As such, trench backfill materials should be placed in lifts no greater than approximately 6 inches in thickness, watered or air-dried as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to document that adequate compaction has been achieved.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand exhibiting a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required. However, observation, probing and, if deemed necessary, testing, should be performed by a representative of the project geotechnical consultant to observe that an adequate degree of compaction has been achieved and that the backfill 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 permeable or non-permeable materials is expected to mitigate the potential for water to migrate through the backfilled trenches from outside of the building to the areas beneath the foundations and floor slabs.

Where an exterior and/or interior utility trench is proposed in a direction that parallels a building footing, the bottom of the trench should not extend below a 1:1 plane projected downward from the bottom edge of the adjacent footing. Where this condition occurs, the adjacent footing should be deepened or the utility constructed and the trench backfilled and compacted prior to constructing the footing.

Seismic Design Considerations

Seismic Design Coefficients

Structures proposed within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2010 California Building Code (CBC). The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height. For structural design in accordance with the 2010 CBC, a computer program, Earthquake Ground Motion Parameters Version 5.1.0, developed by the United States Geological Survey (USGS, 2007) was utilized to provide ground motion parameters for the subject site. The program includes hazard curves, uniform hazard response spectra and design parameters for sites in the 50 United States, Puerto Rico and the United States Virgin Islands. Based on the latitude, longitude and site classification, seismic design parameters and spectral response for both short periods and 1-second periods are calculated including Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter. The program is based on USGS research and publications in cooperation with the California Geological Survey for evaluation of California faulting and seismicity.

The following 2010 CBC seismic design coefficients should be used for the proposed structures. These criteria are based on the site class as determined by existing subsurface geologic conditions, on the proximity of the site to the nearest fault and on the maximum moment magnitude and slip rate of the nearest fault.

2010 CBC Section 1613 Seismic Design Coefficients	
Site Latitude	33.7463
Site Longitude	-116.4138
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.50
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.60
Site Class Definition (Table 1613.5.2)	D
Site Coefficient, F_a (Table 1613.5.3(1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3(2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-36)	1.50
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-37)	0.90
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	1.00
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.60

Conformance to the above seismic design parameters does not constitute any kind of assurance that structural damage, slope instability, and/or ground failure will not occur onsite in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all structural damage. The combined effects of local seismic events are not addressed in the 2010 CBC and regular maintenance and repair following locally significant seismic events (i.e., >M5.5) will likely be necessary, as is the case in southern California as a whole.

Tentative Foundation Design Recommendations

Foundation Systems

Based on the nature of the earth materials that underlie the site at depth and the anticipated conditions following site grading, the proposed structures may be founded on conventional foundations

Allowable Soil Bearing Capacities

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of both continuous footings and isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Static Settlement

Based on the allowable bearing values provided above, total static settlement of the footings is anticipated to be less than 1 inch. Differential settlement is expected to be less than ½ 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.

Liquefaction and Dynamic Settlement

The potential for dry sand dynamic settlement or liquefaction is considered low due to the relatively dense soils and the lack of shallow groundwater. However, a differential settlement of 1 inch over a distance of 40 feet in addition to that of the static conditions, should be considered in design.

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 0.35 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. 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.

Minimum Foundation Design Guidelines

Based on our observations of near-surface soils within the site during our investigation indicate that these material exhibit expansion potentials that are within the Very Low range (Expansion Index from 0 to 20) and are classified as non-expansive. As such, the design of slabs-on-grade is considered to be exempt from the procedures outlined in Sections 1803.5.3 and 1808.6.2 of the 2010 CBC and may be performed using any method deemed rational and appropriate by the project structural engineer. However, it should be noted that the recommendations presented in this section are appropriate for the STATIC loading and settlement criteria presented herein.

The following minimum recommendations are presented herein for conditions where the project design team may require geotechnical engineering guidelines for design and construction of footings and slabs on-grade the project site.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for reducing the effects of variability in composition and behavior within the site soils and long-term differential settlement. These recommendations 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 recommendations 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 recommendations for reinforcement provided herein are performance-based and intended only as guidelines to achieve adequate performance under the anticipated soil conditions. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion) as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

Conventional Slab-on-Ground Foundations

Given the very low expansion potential from 0 to 20 exhibited by onsite soils, we recommend that footings and floor slabs be designed and constructed in accordance with the following minimum criteria.

Footings

1. Exterior continuous footings supporting one- and two-story structures should be founded at a minimum depth of 12 and 18 inches below the lowest adjacent final grade, respectively. Interior continuous footings may be founded at a minimum depth of 10 inches below the tops of the adjacent finish floor slabs.

2. All continuous footings should have minimum widths of 12 and 15 inches for one- and two-story construction, respectively. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

3. 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. 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.

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

5. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

<u>Building Floor Slabs</u>

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

2. Living area concrete floor slabs and areas to receive moisture sensitive flooring 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 Orange Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

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

3. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

Accommodation of Dynamic Settlement

It is the responsibility of the structural engineer to verify that the foundation system is sufficiently rigid to accommodate the potential for liquefaction-induced differential settlement on the order of 1 inches over a span of 40 feet. Options for a strengthened foundation system to accommodate such settlement include, but are not necessarily limited to, extending slab reinforcement into the footings, a matt foundation or post-tension slab-on-grade system. Post-tensioned slab on-grade recommendations are provided below.

Post-Tensioned Slabs-on-Grade System

In consideration of the very low expansion potential exhibited by onsite soils, any rational and appropriate procedure may be chosen by the project structural engineer for the design of post-tensioned slabs-on-grade. Should the design engineer choose to follow the most current procedure published by the Post-Tensioning Institute (PTI), the following minimum design criteria are provided.

Design Parameters for PTI Procedure

Soil Information	Very Low Expansion Potential
Approximate Depth of Constant Suction, feet	9
Approximate Soil Suction, pF	3.9
Inferred Thornthwaite Index:	-20
Average Edge Moisture Variation Distance, e_m in feet:	
Center Lift	9.0
Edge Lift	4.7
Anticipated Swell, y_m in inches:	
Center Lift	0.20
Edge Lift	0.40

Modulus of Subgrade Reaction

The modulus of subgrade reaction for design of load bearing partitions may be assumed to be 125 pounds per cubic inch.

Minimum Design Recommendations

The soil values provided above may be utilized by the project structural engineer to design post-tensioned slabs-on-ground in accordance with Section 1808.6.2 of the 2010 CBC and the PTI publication. Thicker floor slabs and larger footing sizes may be required for structural reasons and should govern the design if more restrictive than the minimum recommendations provided below:

1. Perimeter footings for both one-story and two-story structures should be founded at a minimum depth of 12 inches below the lowest adjacent finished ground surface. Interior footings may be founded at a

minimum depth of 10 inches below the tops of the adjacent finish floor slabs. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. Alternatively, post-tensioned tendons may be utilized in the perimeter continuous footings in lieu of the reinforcement bars.

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

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

4. The thickness of the floor slabs should be determined by the project structural engineer with consideration given to the expansion potential of the onsite soils; however; we recommend that a minimum slab thickness of 4 inches be considered.

5. As an alternative to designing 4-inch-thick post-tensioned slabs with perimeter footings as described in Items 1 and 2 above, the structural engineer may design the foundation system using a thickened slab design. The minimum thickness of this uniformly thick slab should be 8 inches. The engineer in charge of post-tensioned slab design may also opt to use any combination of slab thickness and footing embedment depth as deemed appropriate based on their engineering experience and judgment.

6. Living area concrete floor slabs and areas to receive moisture sensitive flooring 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 Orange Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

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

7. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2010 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

Footing Observations

Building-footing trenches should be observed by the project geotechnical consultant to observe and document excavation into competent bearing soils. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square to the degree possible. Loose, sloughed or moisture-softened soil should be removed prior to concrete placement.

General Corrosivity Screening

The following sections represent an interpretation of current codes and specifications that are commonly used in our industry as they relate to the adverse impact of chemical components of the site soils on various components of the proposed structures. As a screening level study, limited chemical testing was performed on representative samples of onsite soils to identify potential corrosive characteristics of these soils. A variety of test methods are available to quantify the corrosive potential of soils. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies

Petra does not practice corrosion engineering; therefore, the opinion and engineering judgment provided herein should be considered as general guidelines only. Further analyses would be warranted for cases where buried metallic building materials such as copper and ductile iron are planned for the project. For these conditions, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer, as deemed appropriate.

Concrete in Contact with Site Soils

Soils containing soluble sulfates beyond certain threshold levels as well as acidic soils are considered to be detrimental to integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates concentration in soils determined in accordance with California Test Method No. 417.

Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in accordance with California Test Method No. 643.

The results of our very limited laboratory tests indicate that on-site soils within the subject site contain a water soluble sulfate content of 0.12 percent by weight. Based on Section 1904.3 of the 2010 CBC, concrete that will be exposed to sulfate-containing soils should comply with the provisions of Section 4.3 of ACI 318.

According to Table 4.2.1 of ACI 318-08 (a precursor to Section 4.3), an exposure class of S1 is appropriate for onsite soils. As such, a **Moderate** exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As directed by Table 4.3.1 of ACI 318-08, Type V cement (in accordance with ASTM C150) would be required for this condition. In addition, the maximum water/cement ratio of the fresh concrete should not exceed 0.45, and concrete minimum unconfined compressive strength should not be less than 4,500 psi.

The results of limited in-house testing of representative samples indicate that soils within the subject site are neutral with respect to pH (pH of 7.2). Based on this finding and according to Section 8.22.2 of Caltrans' 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type II Modified cement may be used.

These recommendations should be verified by the project structural engineer and the contractor responsible for concrete placement for concrete used in footings and interior slabs-on-ground, foundation walls and concrete exposed to weather.

Metals Encased in Concrete

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

The results of limited screening tests performed indicate that onsite soils contain a water-soluble chloride concentrations of 115 parts per million (ppm). Section 1904.4 of CBC 2010 requires that reinforcement in concrete be protected from the corrosive effects of chloride exposure in accordance with Section 4.4 of ACI 318. It should be noted that Section 4.4 of ACI 318-08 pertains to freeze-and-thaw conditions that are not applicable to the subject project; however, regardless of the level of chlorides in soils in contact with concrete, Table 4.2.1 of ACI 318-08 assigns an exposure class of C1 for concrete that will be exposed to moisture but not necessarily to external sources of chlorides. As such, a **Moderate** exposure to chloride may be expected for metallic elements encased in concrete, which is, in turn, placed in contact with the onsite soil materials.

One method of protecting reinforcement in concrete where moderate chloride concentrations are present in the soils is to increase the thickness of the concrete cover over the reinforcement. However, Table 8.22.1 of Caltrans BDS 2003 provides no minimum concrete cover when chloride concentration is less than 500 ppm (as is the case for the subject site). This recommendation should be verified by the project structural engineer.

Metallic Elements in Contact with Site Soils

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to building components that are in contact with site soils. The minimum electrical resistivity indicates the relative concentration of soluble salts in the soil and, therefore, can be used to estimate soil corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643.

The minimum electrical resistivity for onsite soils was found to be 1,700 ohm-cm based on limited testing. This result indicates that on-site soils are **Corrosive** to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings or panel foundations that are expected to be placed in direct contact with site soils should be protected against detrimental effects of the moderately corrosive soils as stipulated by a qualified corrosion engineer.

Post-Grading Recommendations

Site Drainage

Positive-drainage devices, such as sloping flatwork, graded-swales and/or area drains, should be provided around buildings to collect and direct water away from the structures. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Drainage should be directed to an appropriate discharge area. The ground surface adjacent to the structures should also be sloped at a gradient of 2 percent or more away from the foundations for a horizontal distance of 5 feet or more.

Utility Trenches

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

Utility-trench sidewalls deeper than about 3 feet should be laid back at a ratio of 1:1 (h:v) or flatter or shored. A trench box may be used in lieu of shoring. If shoring is anticipated, the project geotechnical consultant should be contacted to provide design parameters.

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

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

Preliminary Pavement Design

The proposed site improvements will include construction of new asphalt-paved parking and roadways. Based upon our experience we have developed the following preliminary recommendations for flexible pavement design based on a preliminary R-value of 60 and using Traffic Index (TI) values of 5.0 and 6.0. The pavement section thicknesses presented below are considered as minimums for the subject site, and may be superseded by the requirements of the design engineer or jurisdictional agency, if more stringent.

Suggested Minimum Flexible Pavement Thickness

Traffic Index	R-Value	Hot Mix Asphalt (inches)	Aggregate Base (inches)
5.0	60	3.0	4.0
6.0	60	3.0	4.0

All aggregate base material should be compacted to a minimum relative compaction of 95 percent (ASTM D-1557) prior to placing asphalt pavement. Base material should conform to the requirements for Untreated Base Materials, Section 200-2 of the latest edition of Standard Specifications for Public Works Construction (Greenbook).

The asphalt pavement design presented herein is based on the assumption that the pavement will be placed directly over engineered, compacted fill. R-value and traffic index parameters presented herein have also been assumed. We recommend that bulk samples of the actual subgrade materials be retrieved and tested after rough grading is completed. Once actual as-graded conditions are confirmed, additional testing and modified design recommendations may be presented.

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. Due to typical project scheduling constraints, it may not be feasible to collect additional samples of subgrade soils for testing to verify their expansion potential immediately prior to pouring concrete. For this reason, we recommend that all exterior concrete flatwork such as sidewalks, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private and/or public vehicular driveways and/or access roads within and adjacent to 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. If sufficient time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results generated may dictate that a somewhat less conservative design could be used.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, 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. Concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load in public access areas, segments of road that will be paved with concrete (such as bus stops and cross-walks) or access roads and driveways, that will be subject to heavy truck loadings should have a minimum thickness of 5 inches and be provided with control joints spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design of the public and access roads.

Reinforcement

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

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

Edge Beams (Optional)

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

Subgrade Preparation

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

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

Drainage

Drainage from 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 of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

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

GRADING PLAN REVIEW AND CONSTRUCTION SERVICES

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

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

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

LIMITATIONS

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

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

The professional opinions contained herein have been derived in accordance with current standards of practice and no warranty is expressed or implied. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

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

Respectfully submitted,
PETRA GEOTECHNICAL, INC.

11/12/13
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Senior Associate Geologist
CEG 1952 Exp 11/30/13

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W:\2013\200\13-283 Hermann Design Group (Whitewater Park Expansion)\100\Preliminary Geotechnical Investigation.doc

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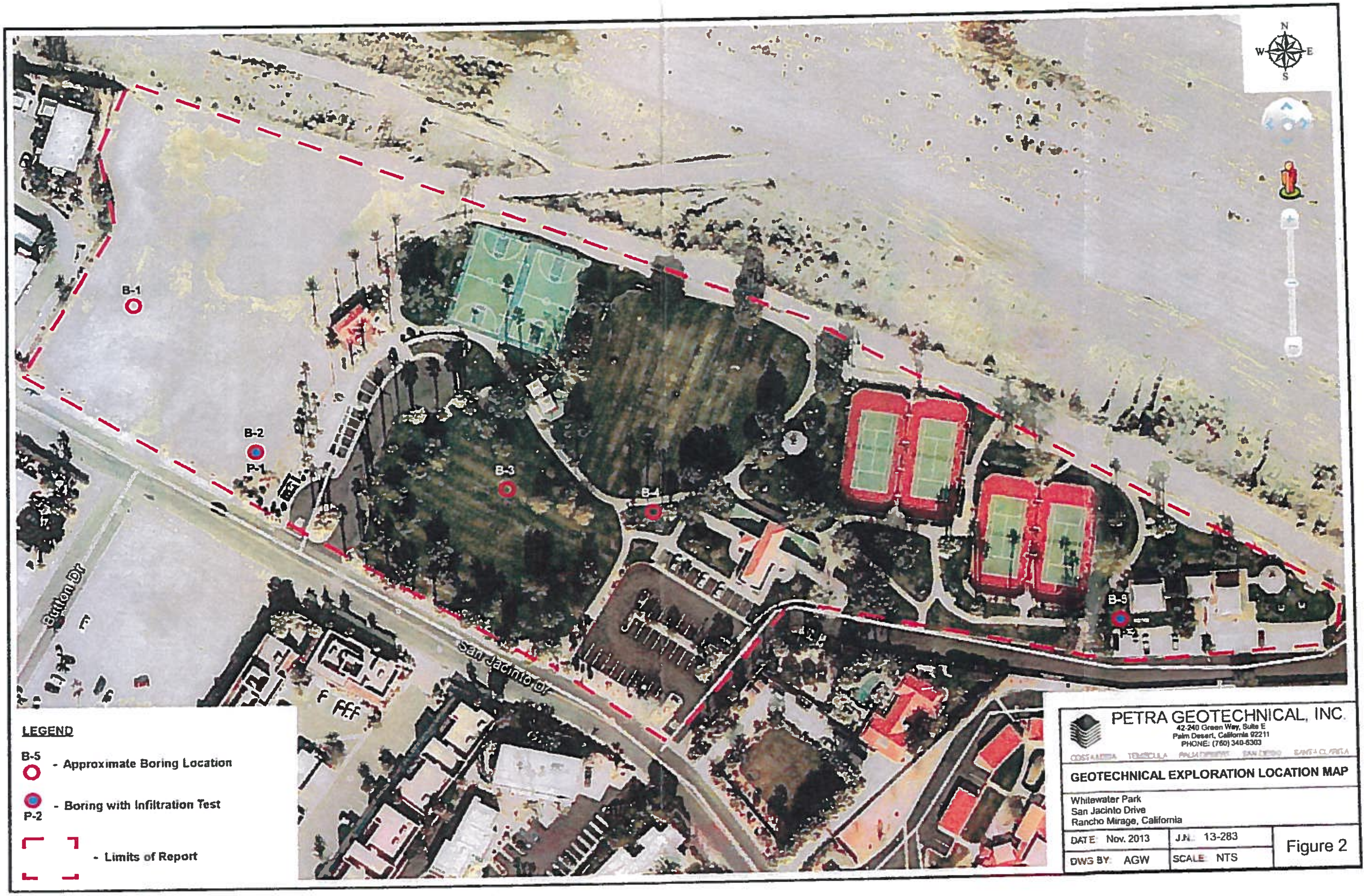
APPENDIX A

EXPLORATION LOGS

PETRA



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COSTA MESA TEMECULA PALM DESERT SAN DIEGO SANTA CLARITA

GEOTECHNICAL EXPLORATION LOCATION MAP

Whitewater Park
San Jacinto Drive
Rancho Mirage, California

DATE: Nov. 2013	J.N.: 13-283	Figure 2
DWG BY: AGW	SCALE: NTS	

Reference: Base map from Google Earth, 2013

EXPLORATION LOG

Project: Whitewater Park		Boring No.: B-1
Location: Rancho Mirage		Elevation: 232
Job No.: 13-283	Client: Hermann and Associates	Date: 8/30/13
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: AGW

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
1 2 3		**Quaternary Alluvium (Qal):** Silty SAND; yellowish brown, medium dense, fine to coarse sand, fine to coarse gravel.							Max, EI, Corr
		Total Depth = 3 feet No Groundwater.							

PLATE A-1

EXPLORATION LOG

Project: Whitewater Park		Boring No.: B-2
Location: Rancho Mirage		Elevation: 227
Job No.: 13-283	Client: Hermann and Associates	Date: 8/30/13
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: AGW

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Quaternary Alluvium (Qal):							
1–2		Poorly-graded SAND with silt; yellowish brown, dry, very dense, fine to coarse sand, fine gravel.		84			1.4	101.7	
3–4		Poorly-graded SAND; yellowish brown, dry, very dense, fine to coarse sand, fine to coarse gravel.		46			1.0	117.9	
5–6				50					
7		medium dense.		24					
9		fine to coarse sand.		31			2.2	116.3	
11				26					
15		dense.		42			1.4	121.2	
19		Silty SAND; gray, moist, medium dense, fine to medium sand.							
20				17			3.3	103.9	
		Total Depth = 21.5 feet No Groundwater.							

PLATE A-2

EXPLORATION LOG - V2 13-283 WHITEWATER PARK.GPJ PETRA.GDT 11/12/13

EXPLORATION LOG

Project: **Whitewater Park**		Boring No.: **B-3**	
Location: **Rancho Mirage**		Elevation: **225**	
Job No.: **13-283**	Client: **Hermann and Associates**	Date: **8/30/13**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **AGW**	

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0–2		**Artificial Fill, Undocumented (Afu):** Silty SAND; dark gray, moist, dense, fine to medium sand.		36			15.2	104.1	
2–4		**Quaternary Alluvium (Qal):** Poorly-graded SAND with silt; yellowish brown, moist, medium dense, fine to coarse sand.		33			8.2	119.2	
4–11.5		Silty SAND; dark yellowish brown, moist, medium dense, fine to medium sand.		21			18.4	102.2	
		loose.		5					
				9			17.9	99.2	
				6					
11.5–19		Poorly-graded SAND; yellowish brown, moist, loose, fine to coarse sand.							
		dense, fine to coarse sand, fine gravel.		47			2.7	124.0	
19–21.5		Silty SAND; dark yellowish brown, moist, medium dense, fine to medium sand.		17			8.5	97.8	

Total Depth = 21.5 feet
No Groundwater.

EXPLORATION LOG - V2 13-283 WHITEWATER PARK.GPJ PETRA.GDT 11/12/13

Petra Geotechnical, Inc.

EXPLORATION LOG

Project:	**Whitewater Park**	Boring No.:	**B-4**
Location:	**Rancho Mirage**	Elevation:	**222**
Job No.:	**13-283**	Date:	**8/30/13**
Client:	**Hermann and Associates**	Logged By:	**AGW**
Drill Method:	**Hollow-Stem Auger**	Driving Weight:	**140 lbs / 30 in**

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**Artificial Fill, Undocumented (Afu):** Poorly-graded SAND with silt; dark yellowish brown, moist, medium dense, fine to medium.							
1				28			6.8	107.6	
3		**Quaternary Alluvium (Qal):** Silty SAND; dark yellowish brown, moist, dense, fine to medium sand.		45			8.1	106.9	Direct Shear
5		dense.		56			10.6	106.3	
7		medium dense.		16					
8		Poorly-graded SAND with silt; dark yellowish brown, moist, medium dense.							
9				29			15.6	108.7	
10		Silty SAND; dark yellowish brown, moist, medium dense, fine to medium sand.		11					
12		Poorly-graded SAND; yellowish brown, moist, medium dense, fine to medium sand.							
13		Silty SAND; gray, moist, medium dense, fine to medium sand.					16.9	101.0	
15				32					
18		Poorly-graded SAND with silt; dark yellowish brown, moist, medium dense, fine to medium sand.							
20				29			6.4	100.2	
23		Silty SAND; dark yellowish brown, wet, loose, fine sand.							
25				27			34.2	87.7	
26		Poorly-graded SAND with silt; yellowish brown, moist, medium dense, fine to coarse sand.							

EXPLORATION LOG

Project: **Whitewater Park**		Boring No.: **B-4**	
Location: **Rancho Mirage**		Elevation: **222**	
Job No.: **13-283**	Client: **Hermann and Associates**	Date: **8/30/13**	
Drill Method: **Hollow-Stem Auger**	Driving Weight: **140 lbs / 30 in**	Logged By: **AGW**	

Depth (Feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
30–33		very dense, fine to coarse sand, fine to coarse gravel.		53	■		8.0	104.0	
33		Silty SAND; yellowish brown, moist, medium dense, fine sand.							
35				28	■		9.2	96.0	
40		dense.		36	■		14.7	95.1	
43		Poorly-graded SAND; yellowish brown, moist, dense, fine to coarse sand.							
45				43	■		3.1	116.9	
46		Silty SAND; dark yellowish brown, moist, dense, fine sand.							
48		Poorly-graded SAND; yellowish brown, dry, dense, fine to coarse sand.							
50				40	■		3.2	103.0	
51.5		Total Depth = 51.5 feet No Groundwater.							

Depth scale: 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51

EXPLORATION LOG

Project: Whitewater Park			Boring No.: B-5
Location: Rancho Mirage			Elevation: 223
Job No.: 13-283		Client: Hermann and Associates	Date: 8/30/13
Drill Method: Hollow-Stem Auger		Driving Weight: 140 lbs / 30 in	Logged By: AGW

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0–3		**Artificial Fill, Undocumented (Afu):** Silty SAND; gray, moist, medium dense, fine sand.		26			8.4	104.6	Max, EI
3–6		**Quaternary Alluvium (Qal):** Poorly-graded SAND with silt; gray, moist, very dense, fine sand.		51			3.3	106.0	
				57			6.8	108.0	
6–10		Poorly-graded SAND; gray, moist, medium dense, fine sand.		22					
				36			7.4	110.0	
10–13		Poorly-graded SAND with silt; dark yellowish brown, moist, medium dense, fine to medium sand.		16					
13–19		Poorly-graded SAND; gray, moist, very dense, fine to medium sand.		63			7.6	105.4	
19–21.5		Silty SAND; dark yellowish brown, moist, dense, fine sand.		37			18.9	106.2	

Total Depth = 21.5 feet
No Groundwater.

PLATE A-6

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

PETRA

APPENDIX B

Laboratory Test Criteria

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488). The samples were re-examined in the laboratory and the classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring Logs (Appendix A).

In-Situ Moisture and Density

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

Maximum Dry Density

Maximum dry density and optimum moisture content were determined for selected samples of the onsite soils in accordance with ASTM D1557. The test results are presented in Plate B-1.

Expansion Potential

Expansion potentials were determined for selected samples of the onsite soils in accordance with ASTM D4829. The test results are presented in Plate B-1.

Corrosivity

Chemical analyses were performed on a selected sample of the onsite soils to determine concentrations of soluble sulfate and chloride, as well as pH and resistivity. The tests were performed in accordance with California Test Method Nos. 417 (sulfate), 422 (chloride) and 643 (pH and resistivity). Test results are included in Plate B-1.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for disturbed (bulk) samples remolded to approximately 90 percent of maximum dry density. These tests were performed in general accordance with ASTM D3080. Three specimens were prepared for each test. The test specimens were artificially saturated, and then sheared under varied normal loads at a maximum constant rate of strain of 0.01 inches per minute. Results are graphically presented on Plate B-2.

R-Value

An R-value test was performed on a selected sample of soil anticipated to be prevalent at grade at the completion of grading. The purpose of this test was to provide preliminary data with respect to design of structural pavement sections. The R-value test was performed in accordance with California Test Method No. 301. Test data are presented in Plate B-1.

APPENDIX B

MAXIMUM DRY DENSITY

Boring/Depth (feet)	Soil Type	Maximum Dry Density[1] (pcf)	Optimum Moisture[1] (%)
B-1 @ 0-3	Silty SAND with gravel (SM)	136.0	6.0
B-5 @ 0-4	Silty SAND (SM)	121.0	12.0

EXPANSION POTENTIAL

Boring/Depth (feet)	Soil Type	Expansion Index[2]	Expansion Potential[2]
B-1 @ 0-3	Silty SAND with gravel (SM)	0	Very Low
B-5 @ 0-4	Silty SAND (SM)	3	Very Low

CORROSIVITY

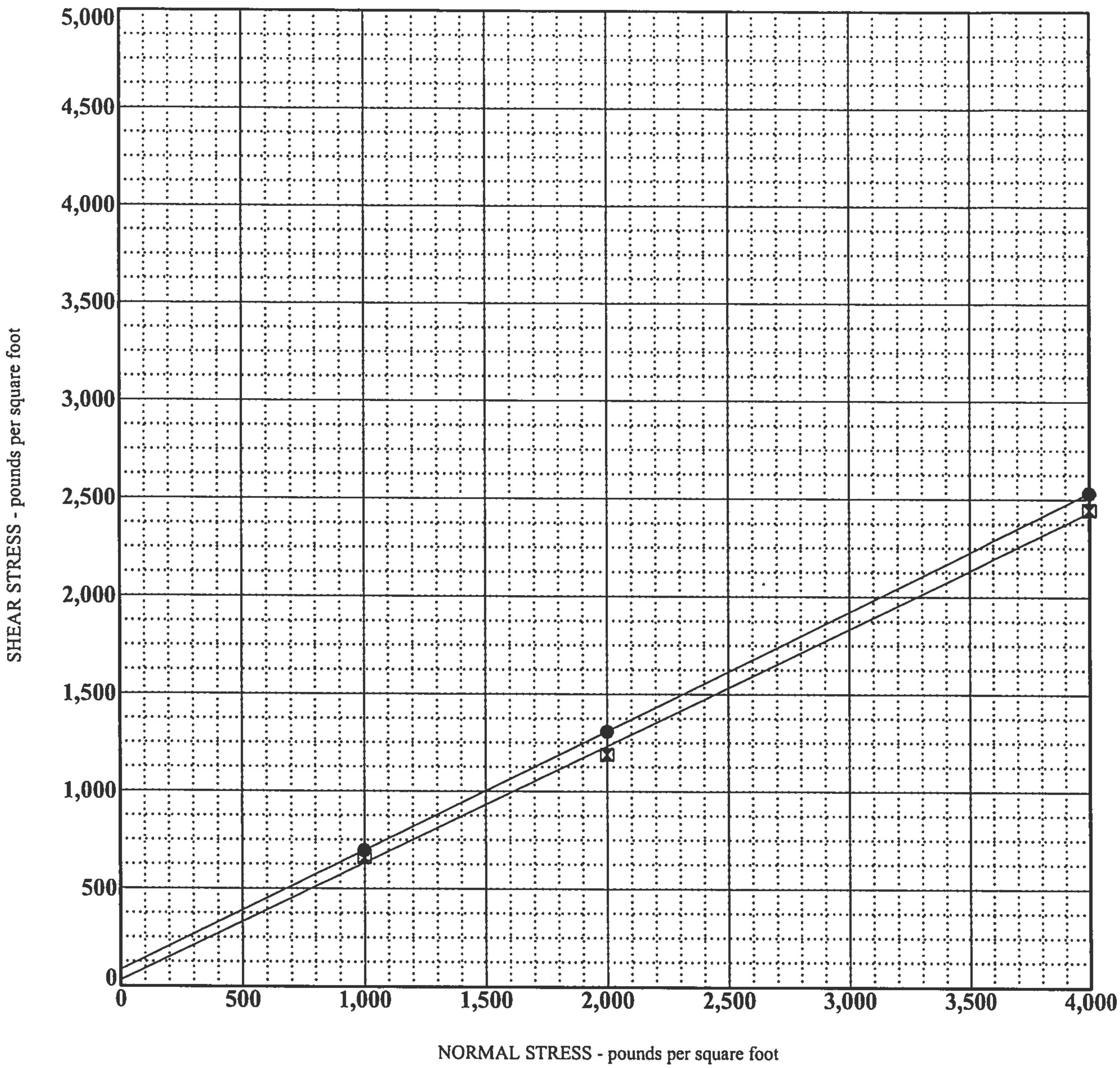
Boring/Depth (feet)	Sulfate[3] (%)	Chloride[4] (ppm)	pH[5]	Resistivity[5] (ohm-cm)	Corrosivity Potential
B-3 @ 0-4	0.12	115	7.2	1,700	concrete: Moderate steel: Corrosive

R-VALUE

Boring/Depth (feet)	Soil Type	R-Value[6]
B-1 @ 0-3	Silty SAND with gravel (SM)	72

(1) PER ASTM D1557
(2) PER ASTM D4829
(3) PER CALIFORNIA TEST METHOD NO. 643
(4) PER CALIFORNIA TEST METHOD NO. 422
(5) PERCALIFORNIA TEST METHOD NO. 643
(6) PER CALIFORNIA TEST METHOD NO. 301

Plate B-1



SAMPLE LOCATION	DESCRIPTION	FRICTION ANGLE (°)	COHESION (PSF)
● B-4 @ 3.0	Poorly-graded SAND (Peak)	31	85
⊠ B-4 @ 3.0	Poorly-graded SAND (Ultimate)	31	30

NOTES:

DIRECT SHEAR 13-283 WHITEWATER PARK.GPJ PETRA.GDT 11/11/13

J.N. 13-283	DIRECT SHEAR TEST DATA	November, 2013
PETRA GEOTECHNICAL, INC.	UNDISTURBED TEST SAMPLES	PLATE B-1

R - VALUE DATA SHEET

P.N. 13-283

PROJECT NUMBER 38982 BORING NUMBER: B-1 @ 0'-3'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN		
	a	b	c
Mold Number	1	2	3
Water added, grams	60	74	51
Initial Test Water, %	7.5	8.7	6.7
Compact Gage Pressure,psi	330	150	350
Exudation Pressure, psi	365	106	734
Height Sample, Inches	2.59	2.54	2.51
Gross Weight Mold, grams	3153	3144	3126
Tare Weight Mold, grams	1965	1969	1977
Sample Wet Weight, grams	1188	1175	1149
Expansion, Inches x 10exp-4	3	1	7
Stability 2,000 lbs (160psi)	18 / 28	19 / 34	14 / 24
Turns Displacement	4.35	4.49	4.02
R-Value Uncorrected	73	67	78
R-Value Corrected	74	67	78
Dry Density, pcf	129.3	128.9	129.9
	DESIGN CALCULATION DATA		
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.27	0.34	0.23
G. E. by Expansion	0.10	0.03	0.23

Equilibrium R-Value 72 by EXUDATION

Examined & Checked: 9 /26/ 13

REMARKS:

Gf = 1.25

5.2% Retained on the 3/4" Sieve.

Partial Free Drainage.

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 38982
PN: 13-283

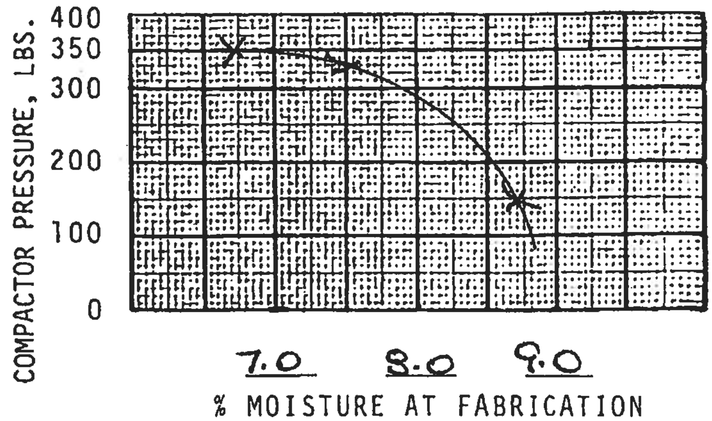
BORING NO. B-1 @ 0'-3'

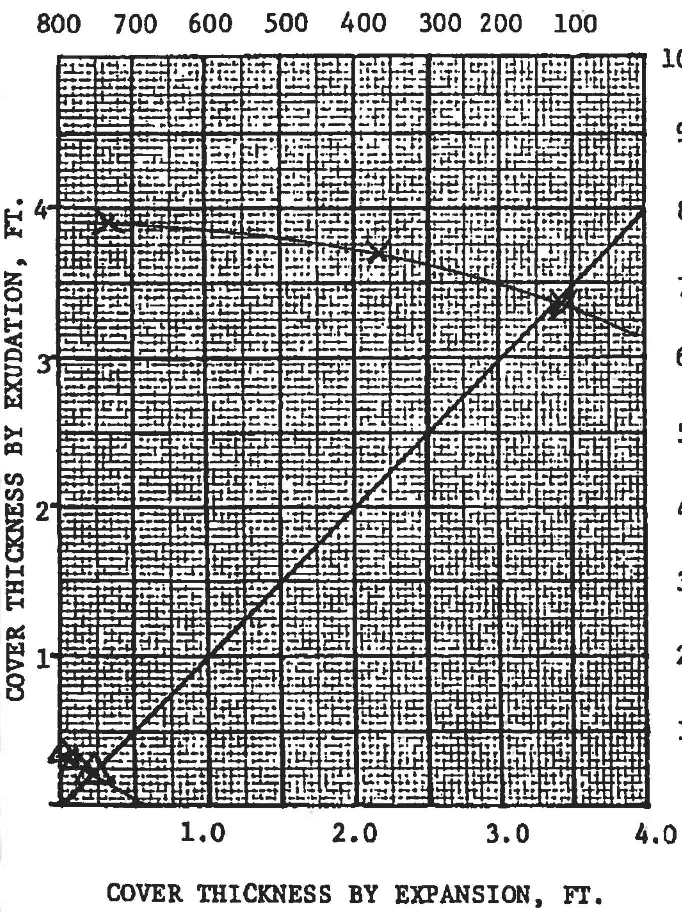
DATE 9/26/13

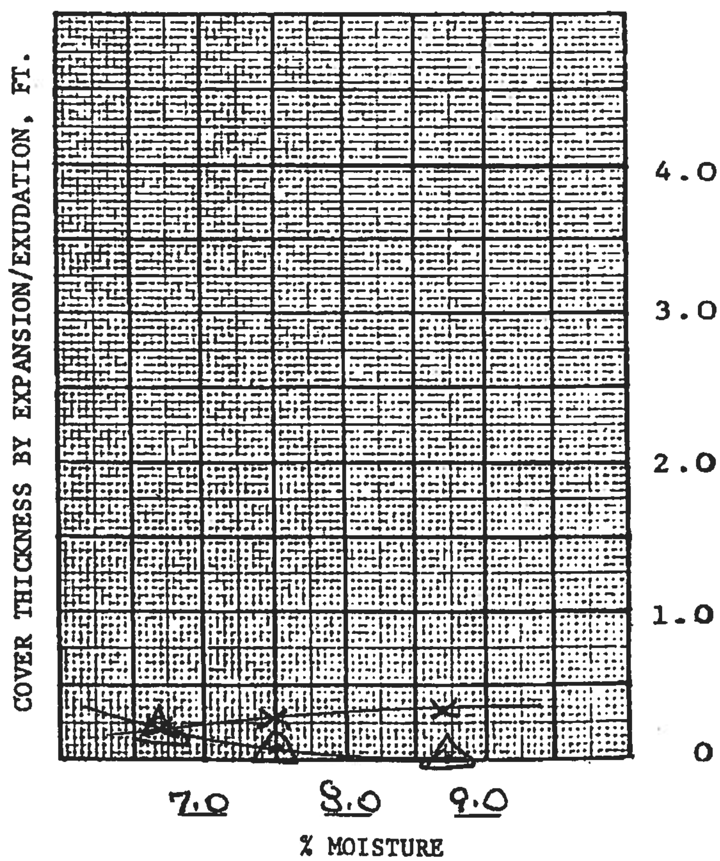
TRAFFIC INDEX Assume 4.0

R-VALUE BY EXUDATION 72

R-VALUE BY EXPANSION







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS

Gf=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

APPENDIX C

INFILTRATION RATE TEST DATA

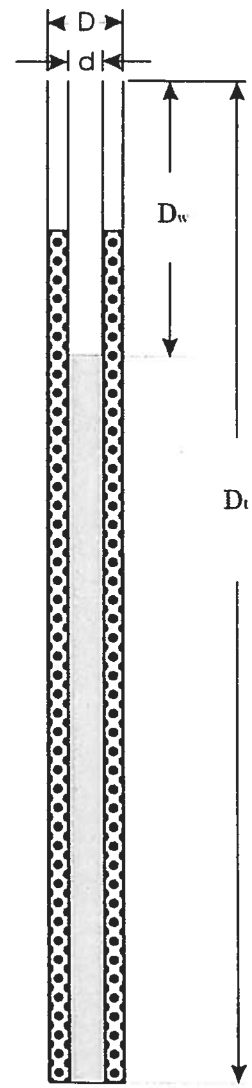
PETRA

PERCOLATION TEST SUMMARY

Job No. 13-283
Project Name: Whitewater Park
Date: August 30, 2013

Test Number: P-1

Depth to Bottom,ft (D_t): 20
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
5	10.0	14.2	50.40	98.61
5	10.0	14.0	48.00	92.76
5	10.0	14.0	48.00	92.76
5	10.0	14.0	48.00	92.76
5	10.0	13.9	46.80	89.90
5	10.0	13.9	46.80	89.90
5	10.0	14.0	48.00	92.76
5	10.0	13.9	46.80	89.90
5	10.0	13.9	46.80	89.90
5	10.0	13.8	45.60	87.06
5	10.0	13.9	46.80	89.90
5	10.0	14.0	48.00	92.76

Percolation Rate: **89.9 gal/day/ft^2**
Infiltration Rate: **6.0 in/hr**

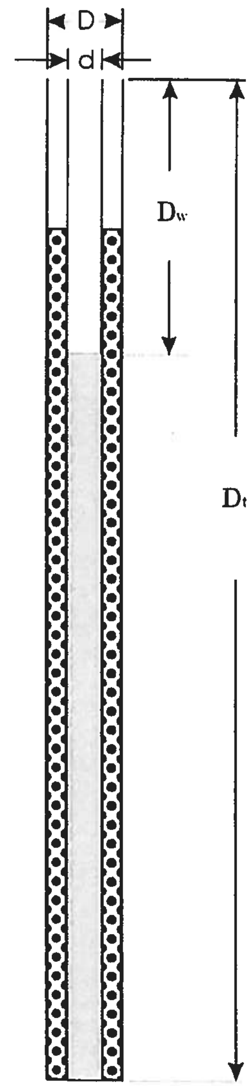
PETRA GEOTECHNICAL, INC.

PERCOLATION TEST SUMMARY

Job No. 13-283
Project Name: Whitewater Park
Date: August 30, 2013

Test Number: P-2

Depth to Bottom,ft (D_t): 20
Diameter of Hole, in (D): 8
Diameter of Pipe, in (d): 3
Agg. Correction (% Voids): 45



Time Interval (min)	Depth to Water Surface Dw (ft)		Change in Head (in)	Perc Rate gal/day/ft^2
	1st Reading	2nd Reading		
10	10.0	11.9	22.80	19.52
10	10.0	12.0	24.00	20.66
10	10.0	11.9	22.80	19.52
10	10.0	11.8	21.60	18.39
10	10.0	12.1	25.20	21.81
10	10.0	12.4	28.80	25.35
10	10.0	12.1	25.20	21.81
10	10.0	11.9	22.80	19.52
				#DIV/0!
				#DIV/0!
				#DIV/0!
				#DIV/0!

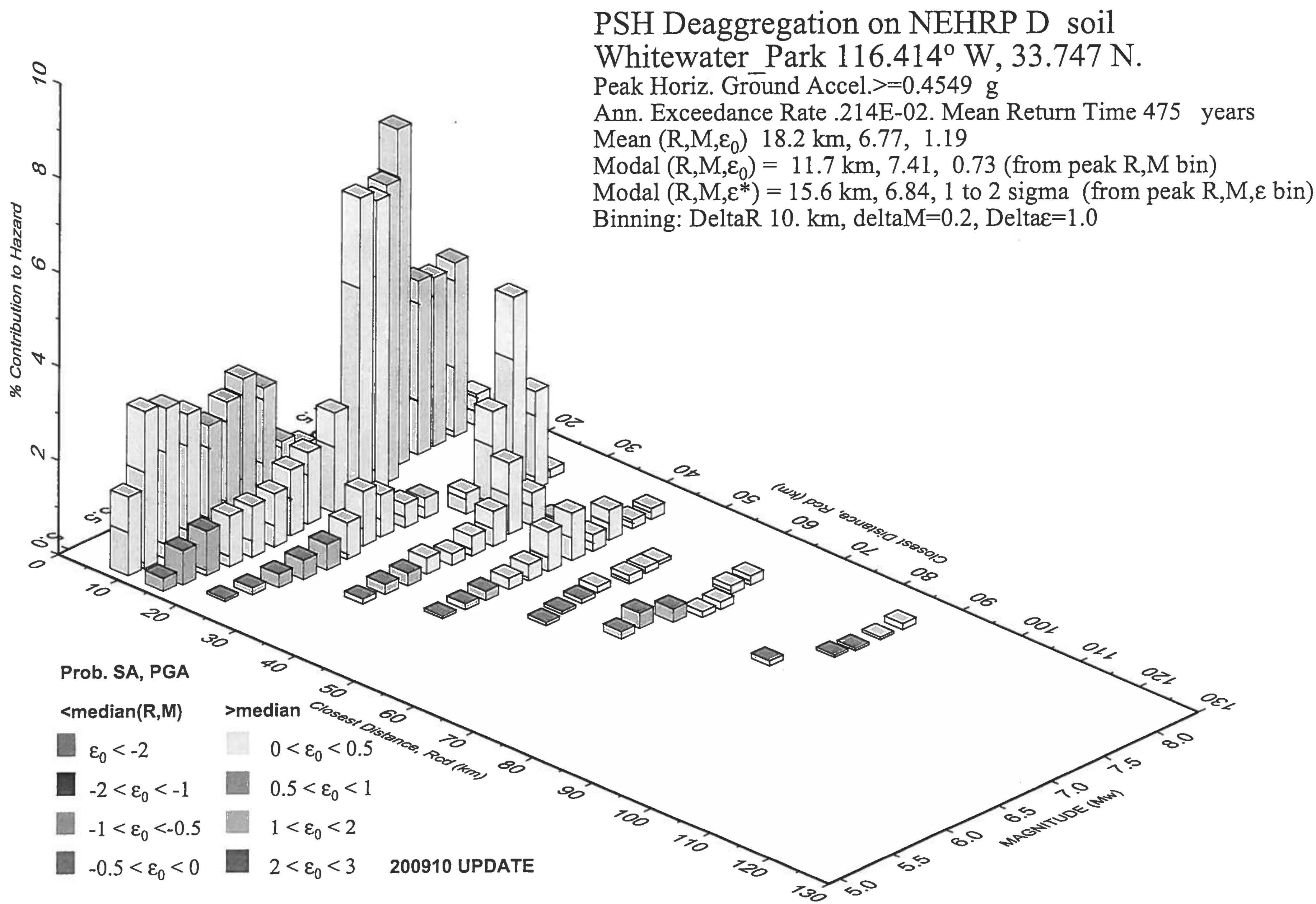
Percolation Rate: **19.5 gal/day/ft^2**
Infiltration Rate: **1.3 in/hr**

PETRA GEOTECHNICAL, INC.

APPENDIX D

SEISMICITY

PETRA



GMT 2013 Sep 24 17:58:03 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 200. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Whitewater_Park long: 116.414 W., lat: 33.747 N.
Vs30(m/s)= 200.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.4549 g. Weight * Computed_Rate_Ex 0.214E-02
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00306
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.7	5.05	1.671	0.754	0.917	0.000	0.000	0.000	0.000
14.7	5.05	0.269	0.269	0.001	0.000	0.000	0.000	0.000
8.7	5.20	3.336	1.294	1.920	0.121	0.000	0.000	0.000
15.1	5.20	0.720	0.675	0.045	0.000	0.000	0.000	0.000
22.1	5.21	0.058	0.058	0.000	0.000	0.000	0.000	0.000
8.8	5.40	3.199	0.946	1.957	0.297	0.000	0.000	0.000
15.6	5.40	0.947	0.837	0.110	0.000	0.000	0.000	0.000
23.3	5.41	0.151	0.151	0.000	0.000	0.000	0.000	0.000
8.8	5.60	2.861	0.678	1.787	0.395	0.000	0.000	0.000
16.0	5.60	1.080	0.860	0.220	0.000	0.000	0.000	0.000
24.2	5.61	0.286	0.286	0.000	0.000	0.000	0.000	0.000
8.9	5.80	2.411	0.481	1.492	0.438	0.000	0.000	0.000
16.2	5.80	1.090	0.754	0.336	0.000	0.000	0.000	0.000
24.8	5.80	0.409	0.402	0.006	0.000	0.000	0.000	0.000
34.7	5.81	0.114	0.114	0.000	0.000	0.000	0.000	0.000
8.2	6.01	2.640	0.418	1.689	0.534	0.000	0.000	0.000
16.3	6.00	1.142	0.682	0.460	0.000	0.000	0.000	0.000
25.2	6.01	0.546	0.513	0.033	0.000	0.000	0.000	0.000
34.9	6.01	0.221	0.221	0.000	0.000	0.000	0.000	0.000
44.5	6.01	0.060	0.060	0.000	0.000	0.000	0.000	0.000
7.5	6.20	2.907	0.357	1.787	0.764	0.000	0.000	0.000
15.6	6.20	1.401	0.605	0.796	0.000	0.000	0.000	0.000
25.0	6.21	0.779	0.663	0.116	0.000	0.000	0.000	0.000
35.2	6.20	0.271	0.271	0.000	0.000	0.000	0.000	0.000
44.8	6.21	0.133	0.133	0.000	0.000	0.000	0.000	0.000
7.3	6.40	2.487	0.247	1.361	0.879	0.000	0.000	0.000
14.9	6.40	1.517	0.477	1.014	0.026	0.000	0.000	0.000
24.0	6.41	1.169	0.798	0.371	0.000	0.000	0.000	0.000
34.8	6.41	0.348	0.338	0.010	0.000	0.000	0.000	0.000
44.8	6.40	0.195	0.195	0.000	0.000	0.000	0.000	0.000
54.9	6.41	0.075	0.075	0.000	0.000	0.000	0.000	0.000
6.9	6.60	1.120	0.141	0.622	0.351	0.006	0.000	0.000
15.1	6.64	2.116	0.723	1.371	0.022	0.000	0.000	0.000
23.7	6.59	0.897	0.526	0.371	0.000	0.000	0.000	0.000
35.3	6.61	0.246	0.205	0.041	0.000	0.000	0.000	0.000
44.8	6.62	0.233	0.219	0.014	0.000	0.000	0.000	0.000
54.2	6.60	0.073	0.073	0.000	0.000	0.000	0.000	0.000
64.0	6.61	0.161	0.161	0.000	0.000	0.000	0.000	0.000
7.0	6.80	0.853	0.098	0.462	0.286	0.007	0.000	0.000
15.6	6.84	6.491	1.913	4.470	0.108	0.000	0.000	0.000
24.1	6.80	0.502	0.289	0.214	0.000	0.000	0.000	0.000
35.3	6.81	0.414	0.284	0.130	0.000	0.000	0.000	0.000
44.9	6.80	0.346	0.282	0.064	0.000	0.000	0.000	0.000
53.8	6.80	0.096	0.094	0.002	0.000	0.000	0.000	0.000
63.6	6.80	0.333	0.325	0.008	0.000	0.000	0.000	0.000
8.3	6.99	0.812	0.102	0.480	0.226	0.003	0.000	0.000
15.6	6.99	6.271	1.618	4.107	0.546	0.000	0.000	0.000
24.1	6.99	0.425	0.202	0.220	0.002	0.000	0.000	0.000
35.4	7.00	0.749	0.376	0.373	0.000	0.000	0.000	0.000
44.9	6.99	0.836	0.546	0.290	0.000	0.000	0.000	0.000
53.2	6.99	0.123	0.106	0.018	0.000	0.000	0.000	0.000
65.4	7.02	0.281	0.237	0.044	0.000	0.000	0.000	0.000
82.2	7.00	0.114	0.114	0.000	0.000	0.000	0.000	0.000

9.0	7.20	0.514	0.062	0.301	0.152	0.000	0.000	0.000
14.1	7.18	6.327	1.187	4.019	1.121	0.000	0.000	0.000
27.0	7.22	0.452	0.147	0.280	0.025	0.000	0.000	0.000
34.5	7.20	1.514	0.682	0.832	0.000	0.000	0.000	0.000
44.9	7.21	1.004	0.528	0.476	0.000	0.000	0.000	0.000
54.4	7.22	0.180	0.121	0.059	0.000	0.000	0.000	0.000
67.1	7.20	0.121	0.096	0.025	0.000	0.000	0.000	0.000
9.0	7.40	0.384	0.043	0.214	0.128	0.000	0.000	0.000
11.7	7.41	7.146	1.018	4.091	2.037	0.000	0.000	0.000
28.3	7.39	2.062	0.742	1.198	0.122	0.000	0.000	0.000
34.9	7.39	0.699	0.292	0.406	0.001	0.000	0.000	0.000
45.1	7.40	0.361	0.182	0.179	0.000	0.000	0.000	0.000
53.9	7.40	0.134	0.080	0.054	0.000	0.000	0.000	0.000
67.2	7.37	0.158	0.119	0.039	0.000	0.000	0.000	0.000
86.2	7.38	0.056	0.052	0.003	0.000	0.000	0.000	0.000
9.2	7.55	0.227	0.024	0.122	0.081	0.000	0.000	0.000
11.6	7.62	3.693	0.480	2.074	1.118	0.021	0.000	0.000
28.3	7.58	4.310	1.338	2.670	0.302	0.000	0.000	0.000
36.1	7.55	0.054	0.023	0.030	0.000	0.000	0.000	0.000
44.5	7.58	0.643	0.281	0.362	0.000	0.000	0.000	0.000
53.4	7.54	0.078	0.042	0.035	0.000	0.000	0.000	0.000
65.2	7.57	0.180	0.116	0.065	0.000	0.000	0.000	0.000
86.8	7.54	0.055	0.046	0.009	0.000	0.000	0.000	0.000
11.4	7.78	3.584	0.404	1.916	1.207	0.057	0.000	0.000
28.3	7.78	2.082	0.518	1.296	0.267	0.000	0.000	0.000
44.3	7.81	0.182	0.066	0.116	0.000	0.000	0.000	0.000
65.2	7.76	0.197	0.113	0.084	0.000	0.000	0.000	0.000
86.9	7.78	0.055	0.041	0.014	0.000	0.000	0.000	0.000
11.4	7.97	3.696	0.391	1.845	1.350	0.111	0.000	0.000
27.5	7.98	0.180	0.039	0.112	0.029	0.000	0.000	0.000
44.3	7.99	0.236	0.092	0.140	0.003	0.000	0.000	0.000
87.0	7.98	0.131	0.092	0.039	0.000	0.000	0.000	0.000
11.3	8.19	0.666	0.064	0.309	0.265	0.028	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 18.2 km; M= 6.77; eps0= 1.19. Mean calculated for all sources.
Modal src-site R= 11.7 km; M= 7.41; eps0= 0.73 from peak (R,M) bin
MODE R*= 15.6km; M*= 6.84; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 4.470

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California A-faults	51.88	20.6	7.36	1.08
CA Compr. crustal gridded	36.72	13.0	5.86	1.30
San Gorgonio Zone gridded	7.13	15.4	7.02	1.09

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
San Jacinto;A+C aPriori	2.92	28.4	7.49	1.29	-143.1
S. S.Andr.;CO aPriori	3.17	16.2	6.97	1.23	73.4
S. S.Andr.;SSB+BG aPriori	2.36	11.3	7.31	0.78	-0.6
S. San Andreas;CO MoBal	11.08	16.2	6.95	1.25	73.4
S. San Andreas Unsegmented A-flt	3.41	13.5	7.67	0.71	-1.3

#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

PSHA Deaggregation. %contributions. site: Whitewater_Park long: 116.414 W., lat: 33.747 N.
Vs30(m/s)= 200.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.4549 g. Weight * Computed_Rate_Ex 0.105E-02
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000
#This deaggregation corresponds to Boore-Atkinson 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.7	5.05	0.397	0.348	0.048	0.000	0.000	0.000	0.000
14.7	5.05	0.071	0.071	0.000	0.000	0.000	0.000	0.000

8.8	5.20	0.783	0.653	0.130	0.000	0.000	0.000	0.000
15.6	5.21	0.236	0.236	0.000	0.000	0.000	0.000	0.000
22.5	5.22	0.028	0.028	0.000	0.000	0.000	0.000	0.000
8.9	5.40	0.726	0.557	0.169	0.000	0.000	0.000	0.000
16.2	5.40	0.350	0.350	0.000	0.000	0.000	0.000	0.000
23.8	5.41	0.093	0.093	0.000	0.000	0.000	0.000	0.000
9.0	5.60	0.642	0.468	0.174	0.000	0.000	0.000	0.000
16.6	5.60	0.421	0.421	0.000	0.000	0.000	0.000	0.000
24.8	5.61	0.192	0.192	0.000	0.000	0.000	0.000	0.000
34.2	5.62	0.041	0.041	0.000	0.000	0.000	0.000	0.000
9.0	5.80	0.553	0.376	0.177	0.000	0.000	0.000	0.000
16.9	5.80	0.441	0.434	0.006	0.000	0.000	0.000	0.000
25.4	5.80	0.283	0.283	0.000	0.000	0.000	0.000	0.000
34.9	5.81	0.106	0.106	0.000	0.000	0.000	0.000	0.000
8.2	6.01	0.603	0.371	0.231	0.000	0.000	0.000	0.000
16.7	6.00	0.450	0.427	0.024	0.000	0.000	0.000	0.000
25.6	6.01	0.375	0.375	0.000	0.000	0.000	0.000	0.000
35.0	6.01	0.202	0.202	0.000	0.000	0.000	0.000	0.000
44.5	6.01	0.060	0.060	0.000	0.000	0.000	0.000	0.000
7.5	6.20	0.639	0.341	0.298	0.000	0.000	0.000	0.000
15.8	6.20	0.500	0.438	0.062	0.000	0.000	0.000	0.000
25.3	6.21	0.517	0.502	0.015	0.000	0.000	0.000	0.000
35.3	6.20	0.244	0.244	0.000	0.000	0.000	0.000	0.000
44.8	6.21	0.131	0.131	0.000	0.000	0.000	0.000	0.000
54.6	6.21	0.037	0.037	0.000	0.000	0.000	0.000	0.000
7.4	6.40	0.526	0.241	0.284	0.000	0.000	0.000	0.000
15.2	6.40	0.494	0.403	0.091	0.000	0.000	0.000	0.000
24.0	6.41	0.810	0.596	0.214	0.000	0.000	0.000	0.000
34.9	6.40	0.296	0.296	0.000	0.000	0.000	0.000	0.000
44.9	6.40	0.189	0.189	0.000	0.000	0.000	0.000	0.000
54.9	6.41	0.075	0.075	0.000	0.000	0.000	0.000	0.000
64.2	6.43	0.039	0.039	0.000	0.000	0.000	0.000	0.000
7.6	6.60	0.282	0.137	0.145	0.000	0.000	0.000	0.000
15.4	6.65	1.133	0.521	0.611	0.001	0.000	0.000	0.000
23.8	6.59	0.725	0.413	0.312	0.000	0.000	0.000	0.000
35.4	6.61	0.234	0.199	0.035	0.000	0.000	0.000	0.000
44.8	6.62	0.231	0.217	0.014	0.000	0.000	0.000	0.000
54.2	6.60	0.073	0.073	0.000	0.000	0.000	0.000	0.000
64.0	6.61	0.161	0.161	0.000	0.000	0.000	0.000	0.000
7.6	6.80	0.219	0.096	0.123	0.000	0.000	0.000	0.000
15.8	6.84	3.714	1.083	2.584	0.046	0.000	0.000	0.000
24.4	6.80	0.382	0.242	0.140	0.000	0.000	0.000	0.000
35.4	6.81	0.391	0.273	0.118	0.000	0.000	0.000	0.000
44.9	6.80	0.343	0.280	0.063	0.000	0.000	0.000	0.000
53.8	6.80	0.096	0.094	0.002	0.000	0.000	0.000	0.000
63.6	6.80	0.333	0.325	0.008	0.000	0.000	0.000	0.000
82.3	6.79	0.041	0.041	0.000	0.000	0.000	0.000	0.000
8.6	6.99	0.235	0.098	0.137	0.000	0.000	0.000	0.000
15.7	6.99	3.434	0.846	2.309	0.279	0.000	0.000	0.000
24.6	6.99	0.308	0.178	0.127	0.002	0.000	0.000	0.000
35.5	7.00	0.704	0.342	0.362	0.000	0.000	0.000	0.000
44.9	6.99	0.823	0.535	0.288	0.000	0.000	0.000	0.000
53.2	6.99	0.123	0.105	0.018	0.000	0.000	0.000	0.000
65.4	7.02	0.281	0.237	0.044	0.000	0.000	0.000	0.000
82.2	7.00	0.114	0.114	0.000	0.000	0.000	0.000	0.000
9.0	7.20	0.155	0.057	0.098	0.000	0.000	0.000	0.000
14.5	7.16	3.058	0.651	1.944	0.463	0.000	0.000	0.000
27.4	7.22	0.365	0.124	0.216	0.025	0.000	0.000	0.000
34.5	7.20	1.322	0.533	0.789	0.000	0.000	0.000	0.000
44.9	7.21	0.961	0.489	0.472	0.000	0.000	0.000	0.000
54.4	7.22	0.177	0.119	0.058	0.000	0.000	0.000	0.000
67.1	7.20	0.120	0.095	0.025	0.000	0.000	0.000	0.000
73.3	7.24	0.033	0.028	0.005	0.000	0.000	0.000	0.000
83.5	7.21	0.038	0.037	0.000	0.000	0.000	0.000	0.000
9.0	7.40	0.114	0.040	0.074	0.000	0.000	0.000	0.000

11.7	7.40	3.207	0.553	2.045	0.609	0.000	0.000	0.000
28.3	7.39	1.488	0.420	0.947	0.121	0.000	0.000	0.000
35.1	7.39	0.588	0.221	0.366	0.001	0.000	0.000	0.000
45.1	7.39	0.325	0.148	0.177	0.000	0.000	0.000	0.000
53.9	7.40	0.128	0.075	0.053	0.000	0.000	0.000	0.000
67.2	7.37	0.157	0.118	0.039	0.000	0.000	0.000	0.000
72.4	7.40	0.039	0.031	0.008	0.000	0.000	0.000	0.000
86.2	7.38	0.056	0.052	0.003	0.000	0.000	0.000	0.000
9.2	7.55	0.067	0.023	0.044	0.000	0.000	0.000	0.000
11.5	7.63	1.729	0.267	1.146	0.316	0.000	0.000	0.000
28.4	7.58	3.120	0.759	2.002	0.358	0.000	0.000	0.000
36.4	7.55	0.041	0.017	0.024	0.000	0.000	0.000	0.000
44.5	7.58	0.567	0.218	0.349	0.000	0.000	0.000	0.000
53.4	7.54	0.073	0.038	0.035	0.000	0.000	0.000	0.000
65.2	7.57	0.175	0.111	0.064	0.000	0.000	0.000	0.000
71.4	7.54	0.038	0.027	0.011	0.000	0.000	0.000	0.000
86.8	7.54	0.054	0.046	0.009	0.000	0.000	0.000	0.000
11.4	7.79	1.213	0.154	0.770	0.289	0.000	0.000	0.000
28.3	7.79	1.216	0.266	0.778	0.172	0.000	0.000	0.000
44.3	7.81	0.142	0.043	0.099	0.000	0.000	0.000	0.000
65.2	7.76	0.186	0.102	0.084	0.000	0.000	0.000	0.000
86.9	7.77	0.053	0.039	0.014	0.000	0.000	0.000	0.000
122.3	7.83	0.037	0.037	0.000	0.000	0.000	0.000	0.000
11.4	7.97	1.454	0.176	0.902	0.376	0.000	0.000	0.000
27.6	7.98	0.106	0.021	0.068	0.017	0.000	0.000	0.000
44.3	7.99	0.187	0.060	0.124	0.003	0.000	0.000	0.000
87.0	7.98	0.125	0.086	0.039	0.000	0.000	0.000	0.000
122.3	8.00	0.032	0.032	0.000	0.000	0.000	0.000	0.000
11.3	8.19	0.262	0.029	0.153	0.080	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 49.4
Mean src-site R= 23.6 km; M= 6.93; eps0= 1.33. Mean calculated for all sources.
Modal src-site R= 15.8 km; M= 6.84; eps0= 1.25 from peak (R,M) bin
MODE,R*= 15.9km; M*= 6.84; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 2.584

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California A-faults	30.30	24.3	7.32	1.18
CA Compr. crustal gridded	12.09	17.9	5.93	1.64
San Gorgonio Zone gridded	3.16	18.6	6.99	1.18

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
San Jacinto;A+C aPriori	2.08	28.4	7.49	1.19	-143.1
S. S.Andr.;CO aPriori	1.87	16.2	6.97	1.18	73.4
S. S.Andr.;SSB+BG aPriori	1.03	11.3	7.31	0.88	-0.6
S. San Andreas;CO MoBal	6.59	16.2	6.94	1.19	73.4
S. San Andreas Unsegmented A-flt	1.55	15.3	7.62	0.91	-1.3

#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: Whitewater_Park long: 116.414 W., lat: 33.747 N.
Vs30(m/s)= 200.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.4549 g. Weight * Computed_Rate_Ex 0.217E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00042
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.6	5.05	0.331	0.275	0.056	0.000	0.000	0.000	0.000
13.3	5.05	0.021	0.021	0.000	0.000	0.000	0.000	0.000
8.7	5.20	0.758	0.575	0.183	0.000	0.000	0.000	0.000
14.0	5.21	0.079	0.079	0.000	0.000	0.000	0.000	0.000
8.7	5.40	0.860	0.572	0.288	0.000	0.000	0.000	0.000
14.9	5.41	0.152	0.152	0.000	0.000	0.000	0.000	0.000

21.7	5.42	0.008	0.008	0.000	0.000	0.000	0.000	0.000
8.8	5.60	0.815	0.490	0.326	0.000	0.000	0.000	0.000
15.4	5.60	0.200	0.200	0.000	0.000	0.000	0.000	0.000
22.6	5.61	0.022	0.022	0.000	0.000	0.000	0.000	0.000
8.8	5.80	0.668	0.366	0.302	0.000	0.000	0.000	0.000
15.6	5.80	0.199	0.197	0.002	0.000	0.000	0.000	0.000
23.2	5.80	0.032	0.032	0.000	0.000	0.000	0.000	0.000
8.1	6.01	0.707	0.361	0.346	0.000	0.000	0.000	0.000
16.0	6.00	0.214	0.210	0.004	0.000	0.000	0.000	0.000
24.1	6.01	0.047	0.047	0.000	0.000	0.000	0.000	0.000
7.4	6.20	0.803	0.340	0.464	0.000	0.000	0.000	0.000
15.4	6.20	0.288	0.269	0.019	0.000	0.000	0.000	0.000
24.5	6.21	0.078	0.078	0.000	0.000	0.000	0.000	0.000
34.7	6.21	0.008	0.008	0.000	0.000	0.000	0.000	0.000
7.3	6.40	0.729	0.242	0.487	0.000	0.000	0.000	0.000
14.7	6.40	0.338	0.284	0.054	0.000	0.000	0.000	0.000
24.2	6.40	0.097	0.097	0.000	0.000	0.000	0.000	0.000
34.1	6.41	0.016	0.016	0.000	0.000	0.000	0.000	0.000
6.6	6.60	0.354	0.131	0.215	0.007	0.000	0.000	0.000
14.0	6.60	0.244	0.202	0.042	0.000	0.000	0.000	0.000
24.3	6.60	0.036	0.036	0.000	0.000	0.000	0.000	0.000
33.6	6.61	0.006	0.006	0.000	0.000	0.000	0.000	0.000
6.7	6.80	0.254	0.091	0.157	0.006	0.000	0.000	0.000
13.4	6.80	0.260	0.202	0.058	0.000	0.000	0.000	0.000
23.8	6.80	0.036	0.036	0.000	0.000	0.000	0.000	0.000
33.4	6.80	0.008	0.008	0.000	0.000	0.000	0.000	0.000
8.1	6.99	0.218	0.091	0.124	0.002	0.000	0.000	0.000
14.5	6.99	0.167	0.130	0.037	0.000	0.000	0.000	0.000
22.8	6.99	0.036	0.035	0.001	0.000	0.000	0.000	0.000
33.7	7.00	0.006	0.006	0.000	0.000	0.000	0.000	0.000
9.0	7.20	0.128	0.053	0.075	0.000	0.000	0.000	0.000
14.9	7.18	0.183	0.141	0.042	0.000	0.000	0.000	0.000
23.9	7.19	0.019	0.018	0.001	0.000	0.000	0.000	0.000
9.0	7.40	0.093	0.037	0.056	0.000	0.000	0.000	0.000
13.1	7.40	0.271	0.187	0.084	0.000	0.000	0.000	0.000
26.3	7.39	0.017	0.016	0.000	0.000	0.000	0.000	0.000
9.2	7.55	0.053	0.021	0.032	0.000	0.000	0.000	0.000
12.5	7.60	0.127	0.082	0.044	0.000	0.000	0.000	0.000
27.0	7.57	0.023	0.022	0.001	0.000	0.000	0.000	0.000
11.3	7.78	0.057	0.038	0.020	0.000	0.000	0.000	0.000
28.4	7.79	0.008	0.008	0.000	0.000	0.000	0.000	0.000
11.3	7.97	0.061	0.039	0.022	0.000	0.000	0.000	0.000
11.3	8.21	0.009	0.006	0.003	0.000	0.000	0.000	0.000

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Summary statistics for above PSHA PGA  deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%):   10.2
 Mean src-site R=   10.8 km; M= 6.13; eps0=   1.30. Mean calculated for all sources.
Modal src-site R=    8.7 km; M= 5.40; eps0=   1.27 from peak (R,M) bin
 MODE R*=   8.6km; M*= 5.20; EPS.INTERVAL: 1 to 2 sigma  % CONTRIB.=  0.575

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category:                   % contr.  R(km)    M   epsilon0 (mean values).
CA Compr. crustal gridded             8.18    10.3   5.88    1.26
Individual fault hazard details if its contribution to mean hazard > 2%:
Fault ID                           % contr.   Rcd(km)  M   epsilon0 Site-to-src azimuth(d)
San Jacinto;A+C aPriori               0.01    28.4   7.50    2.68    -143.1
S. S.Andr.;CO aPriori                 0.02    16.2   7.03    2.34      73.4
S. S.Andr.;SSB+BG aPriori             0.05    11.3   7.31    1.96      -0.6
S. San Andreas;CO MoBal               0.07    16.2   7.01    2.36      73.4
S. San Andreas Unsegmented A-flt     0.01    11.4   7.75    2.66      -1.3
#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008  *********#

PSHA Deaggregation. %contributions. site: Whitewater_Park long: 116.414 W., lat: 33.747
N.
 Vs30(m/s)= 200.0 (some WUS atten. models use Site Class not Vs30).
```

NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 475 yrs. Exceedance PGA =0.4549 g. Weight * Computed_Rate_Ex 0.864E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.01019
#This deaggregation corresponds to Chiou-Youngs 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.7	5.05	0.943	0.662	0.281	0.000	0.000	0.000	0.000
14.8	5.05	0.177	0.177	0.000	0.000	0.000	0.000	0.000
8.7	5.20	1.794	1.103	0.691	0.000	0.000	0.000	0.000
15.1	5.20	0.405	0.405	0.000	0.000	0.000	0.000	0.000
21.7	5.21	0.029	0.029	0.000	0.000	0.000	0.000	0.000
8.8	5.40	1.613	0.857	0.756	0.000	0.000	0.000	0.000
15.4	5.40	0.445	0.441	0.004	0.000	0.000	0.000	0.000
22.5	5.41	0.050	0.050	0.000	0.000	0.000	0.000	0.000
8.8	5.60	1.403	0.649	0.754	0.000	0.000	0.000	0.000
15.6	5.60	0.458	0.436	0.022	0.000	0.000	0.000	0.000
23.1	5.60	0.072	0.072	0.000	0.000	0.000	0.000	0.000
8.9	5.80	1.190	0.460	0.729	0.000	0.000	0.000	0.000
15.8	5.80	0.450	0.416	0.034	0.000	0.000	0.000	0.000
23.7	5.80	0.094	0.094	0.000	0.000	0.000	0.000	0.000
8.1	6.01	1.331	0.410	0.911	0.009	0.000	0.000	0.000
16.1	6.00	0.477	0.424	0.053	0.000	0.000	0.000	0.000
24.4	6.01	0.124	0.124	0.000	0.000	0.000	0.000	0.000
7.5	6.20	1.465	0.356	1.084	0.024	0.000	0.000	0.000
15.5	6.20	0.612	0.480	0.132	0.000	0.000	0.000	0.000
24.5	6.21	0.184	0.184	0.000	0.000	0.000	0.000	0.000
7.4	6.40	1.232	0.247	0.942	0.044	0.000	0.000	0.000
14.8	6.40	0.684	0.417	0.267	0.000	0.000	0.000	0.000
23.8	6.41	0.261	0.256	0.006	0.000	0.000	0.000	0.000
34.2	6.41	0.036	0.036	0.000	0.000	0.000	0.000	0.000
6.8	6.60	0.485	0.138	0.326	0.021	0.000	0.000	0.000
15.0	6.64	0.735	0.428	0.306	0.001	0.000	0.000	0.000
22.8	6.59	0.136	0.124	0.012	0.000	0.000	0.000	0.000
6.9	6.80	0.380	0.097	0.258	0.025	0.000	0.000	0.000
15.6	6.84	2.510	1.013	1.489	0.008	0.000	0.000	0.000
23.1	6.80	0.085	0.076	0.008	0.000	0.000	0.000	0.000
8.2	6.99	0.359	0.099	0.243	0.018	0.000	0.000	0.000
15.5	7.00	2.680	0.844	1.621	0.215	0.000	0.000	0.000
22.9	6.99	0.081	0.070	0.012	0.000	0.000	0.000	0.000
33.4	6.99	0.039	0.039	0.000	0.000	0.000	0.000	0.000
9.0	7.20	0.231	0.058	0.157	0.015	0.000	0.000	0.000
13.9	7.17	2.659	0.509	1.574	0.576	0.000	0.000	0.000
25.7	7.20	0.068	0.050	0.018	0.000	0.000	0.000	0.000
34.0	7.21	0.187	0.156	0.031	0.000	0.000	0.000	0.000
44.8	7.23	0.042	0.042	0.000	0.000	0.000	0.000	0.000
9.0	7.40	0.178	0.041	0.121	0.015	0.000	0.000	0.000
11.6	7.41	4.094	0.546	2.250	1.298	0.000	0.000	0.000
28.2	7.40	0.558	0.335	0.222	0.000	0.000	0.000	0.000
34.0	7.41	0.106	0.079	0.027	0.000	0.000	0.000	0.000
45.0	7.41	0.035	0.035	0.000	0.000	0.000	0.000	0.000
9.2	7.55	0.107	0.024	0.073	0.010	0.000	0.000	0.000
11.5	7.63	2.253	0.251	1.126	0.854	0.021	0.000	0.000
28.3	7.59	1.365	0.598	0.767	0.001	0.000	0.000	0.000
44.4	7.61	0.075	0.063	0.012	0.000	0.000	0.000	0.000
11.4	7.78	1.926	0.179	0.887	0.803	0.057	0.000	0.000
28.3	7.80	0.660	0.224	0.400	0.035	0.000	0.000	0.000
44.3	7.83	0.038	0.022	0.016	0.000	0.000	0.000	0.000
11.4	7.97	2.154	0.166	0.903	0.974	0.111	0.000	0.000
27.4	7.98	0.073	0.018	0.044	0.012	0.000	0.000	0.000
44.3	8.01	0.051	0.033	0.018	0.000	0.000	0.000	0.000
11.3	8.19	0.394	0.029	0.152	0.185	0.028	0.000	0.000
11.3	8.36	0.030	0.002	0.012	0.015	0.001	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 40.5

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 Mean src-site R=   13.6 km; M= 6.74; eps0=   0.98. Mean calculated for all sources.
Modal src-site R=   11.6 km; M= 7.41; eps0=   0.61 from peak (R,M) bin
 MODE R*=  11.6km; M*= 7.41; EPS.INTERVAL: 1 to 2 sigma  % CONTRIB.=  2.250

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category:                  % contr.  R(km)    M   epsilon0 (mean values).
California A-faults                 21.06    15.6   7.42    0.90
CA Compr. crustal gridded           16.45    10.8   5.80    1.06
Individual fault hazard details if its contribution to mean hazard > 2%:
Fault ID                          % contr.   Rcd(km)  M   epsilon0 Site-to-src azimuth(d)
San Jacinto;A+C aPriori              0.83    28.4   7.50    1.52    -143.1
S. S.Andr.;CO aPriori                1.28    16.2   6.98    1.29      73.4
S. S.Andr.;SSB+BG aPriori            1.28    11.3   7.32    0.66      -0.6
S. San Andreas;CO MoBal              4.42    16.2   6.96    1.32      73.4
S. San Andreas Unsegmented A-flt     1.85    12.1   7.71    0.54      -1.3
#*********End of deaggregation corresponding to Chiou-Youngs 2008        *********#

******************** Southern California ***************************************
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APPENDIX E

STANDARD GRADING SPECIFICATIONS

PETRA

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.

3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide sufficient survey control for determining locations and elevations of processed areas, keys, and benches.

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

IV. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

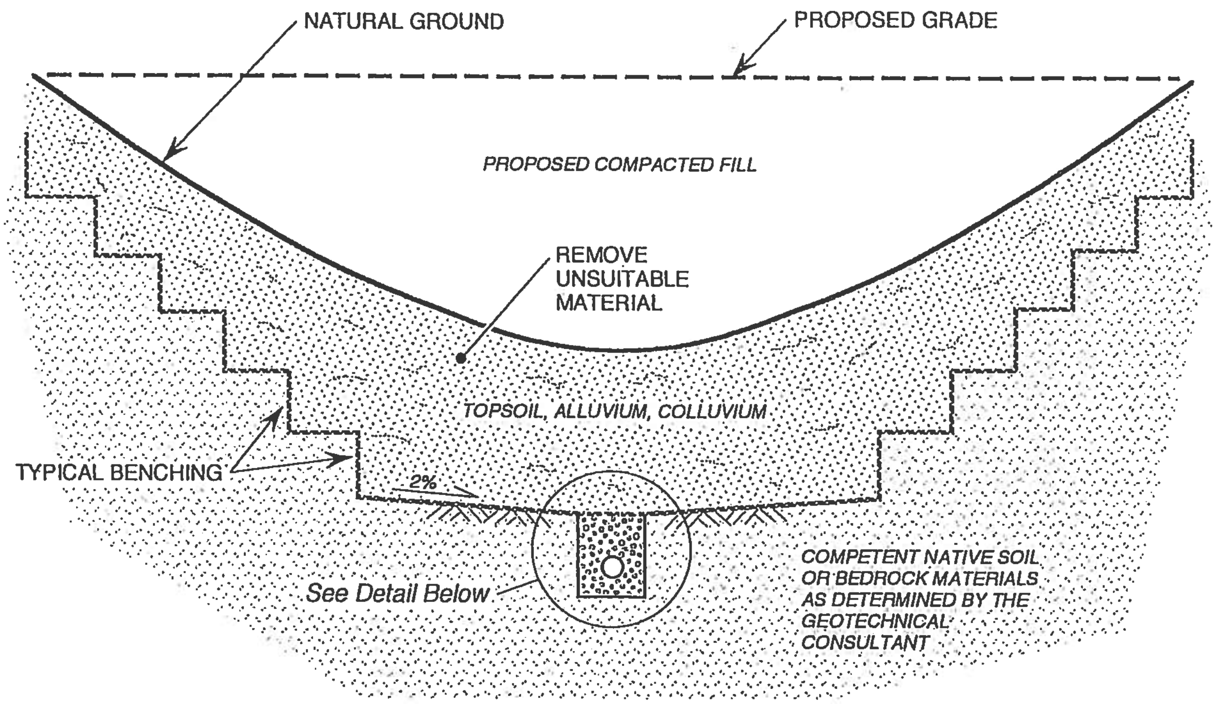
B. Oversize Materials

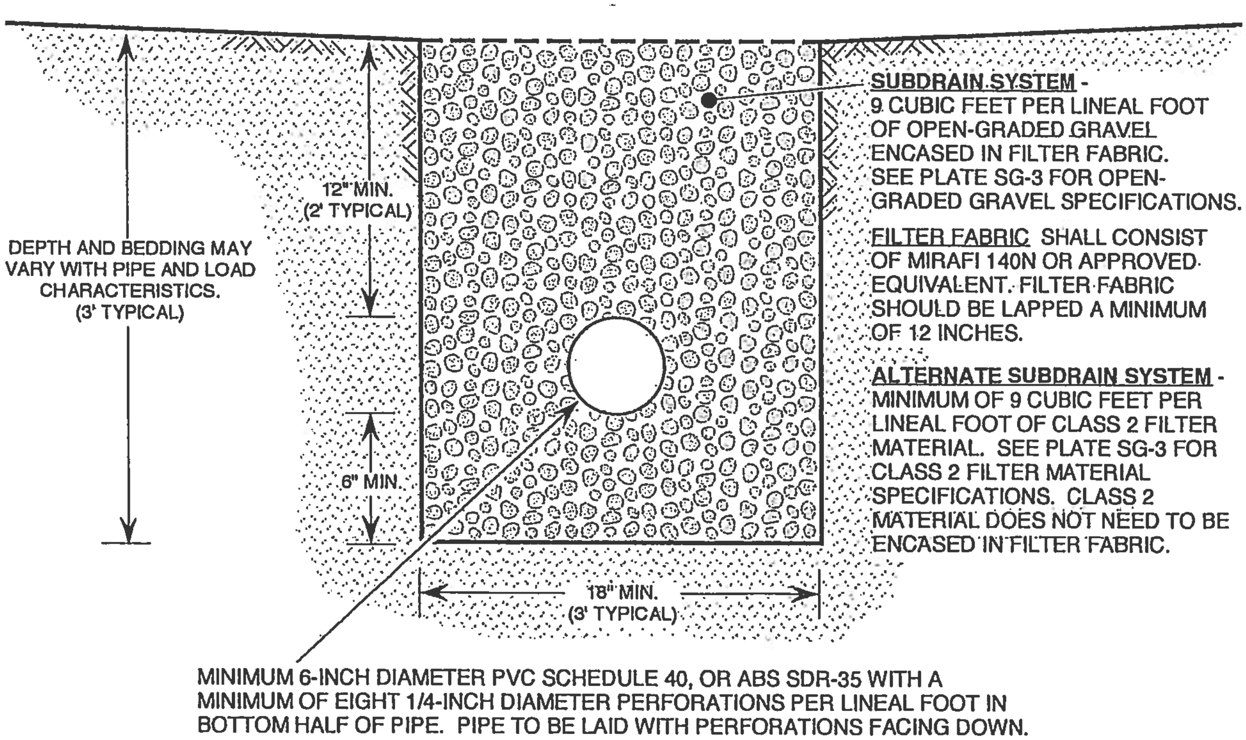
Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.

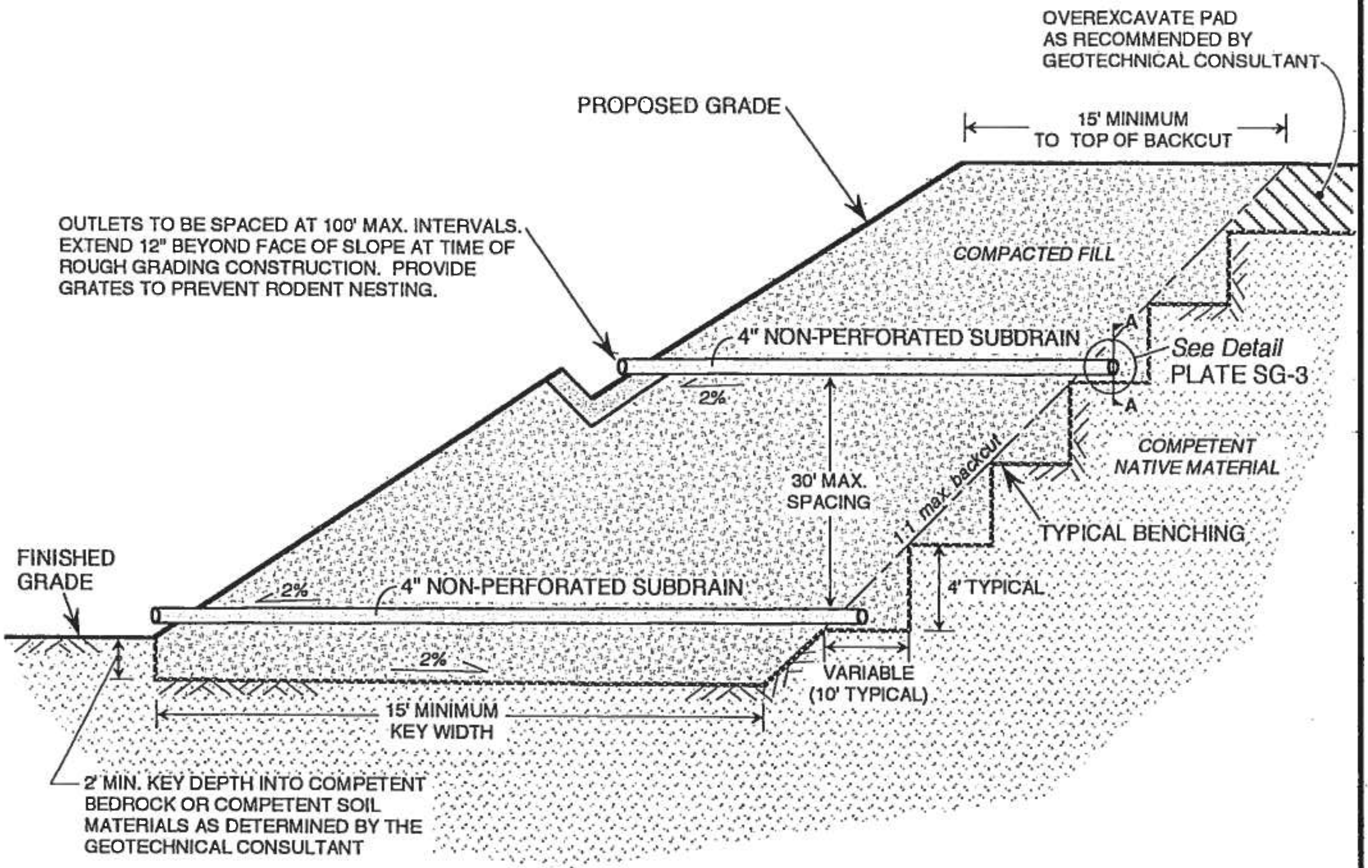




NOTES:

1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.

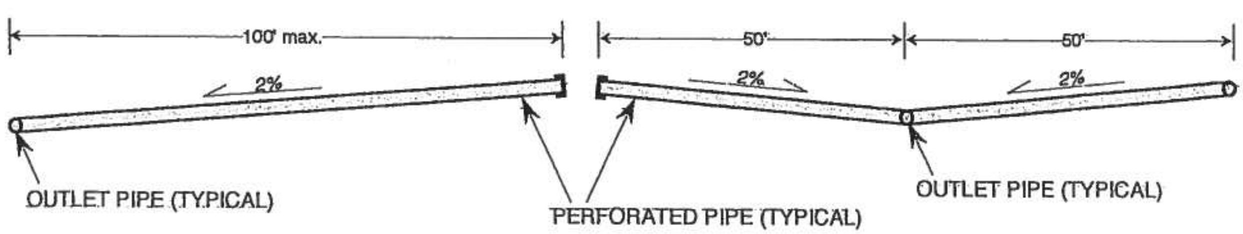
PETRA	CANYON SUBDRAIN DETAIL	PLATE SG-1



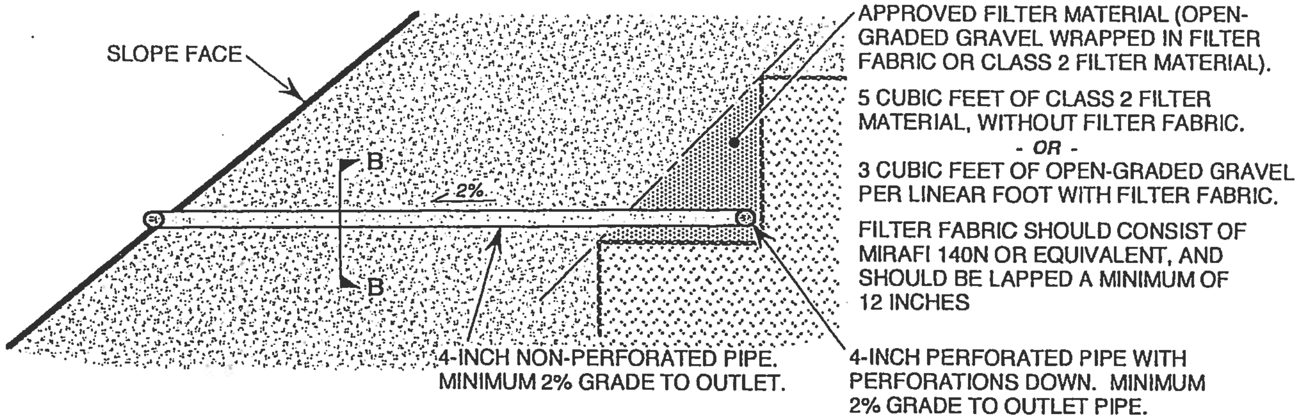
NOTES:

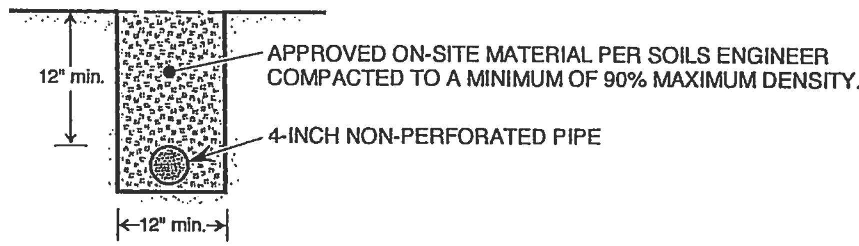
1. 30' MAXIMUM VERTICAL SPACING BETWEEN SUBDRAIN SYSTEMS.
2. 100' MAXIMUM HORIZONTAL DISTANCE BETWEEN NON-PERFORATED OUTLET PIPES. *(See Below)*
3. MINIMUM GRADIENT OF 2% FOR ALL PERFORATED AND NON-PERFORATED PIPE.

SECTION A-A (PERFORATED PIPE PROFILE)



PETRA	BUTTRESS OR STABILIZATION FILL DETAIL	PLATE SG-2





SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:

1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:

OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFI 140N OR EQUIVALENT)

OPEN-GRADED GRAVEL

SIEVE SIZE	PERCENT PASSING
1 1/2-INCH	88 - 100
1-INCH	5 - 40
3/4-INCH	0 - 17
3/8-INCH	0 - 7
No. 200	0 - 3

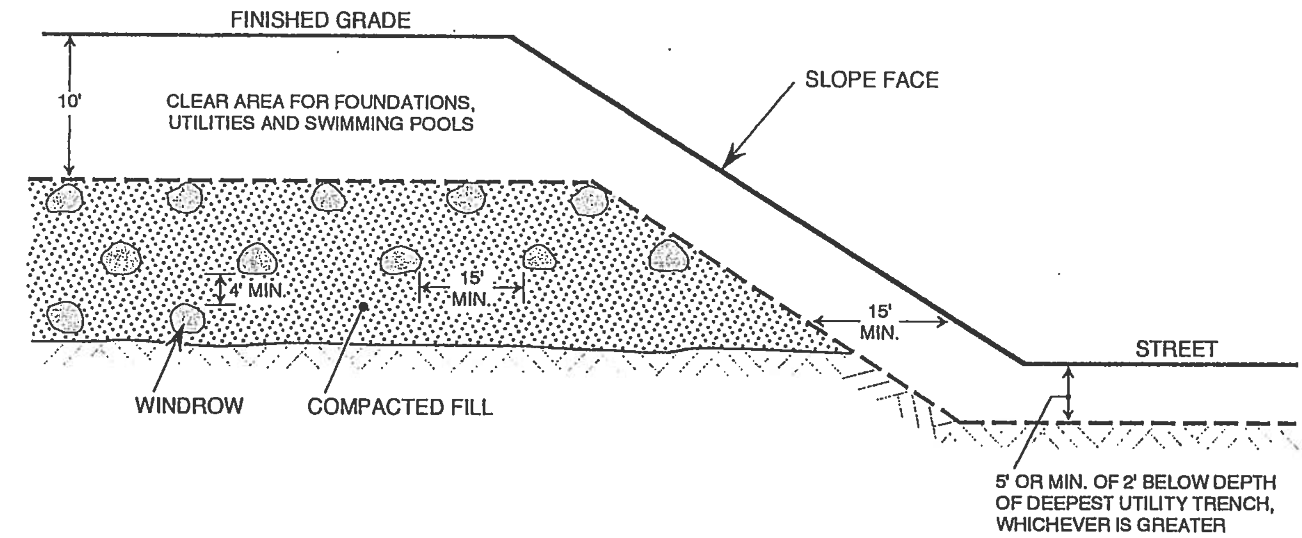
ALTERNATE:

CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS STANDARD SPECIFICATION 68-1.025.

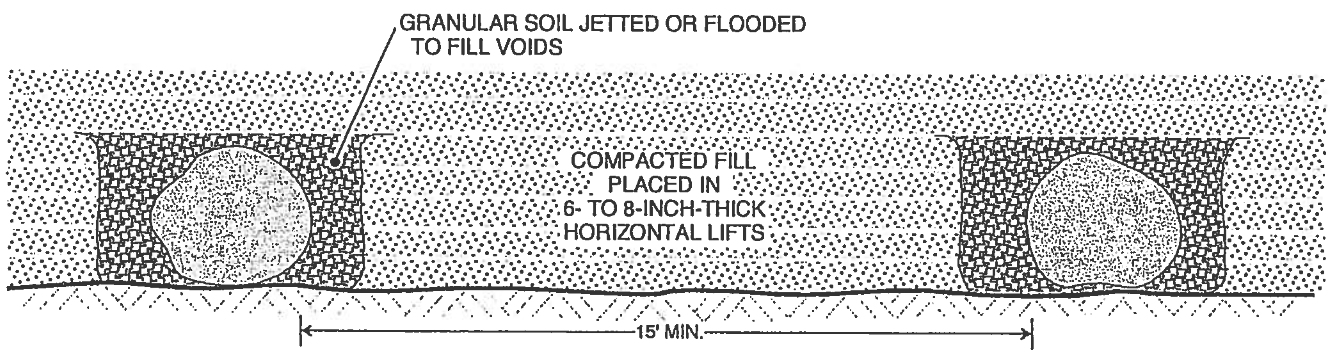
CLASS 2 FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1-INCH	100
3/4-INCH	90 - 100
3/8-INCH	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. -30	5 - 15
No. -50	0 - 7
No. 200	0 - 3

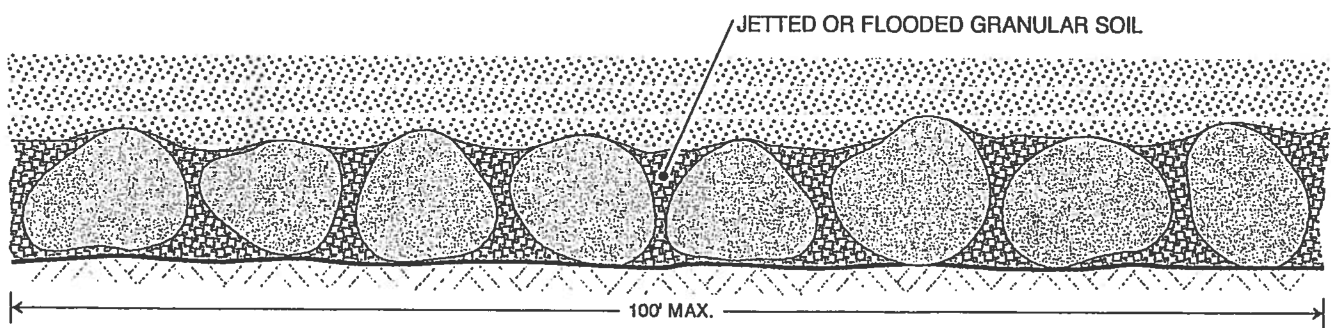
PETRA | BUTTRESS OR STABILIZATION FILL SUBDRAIN | PLATE SG-3



TYPICAL WINDROW DETAIL (END VIEW)

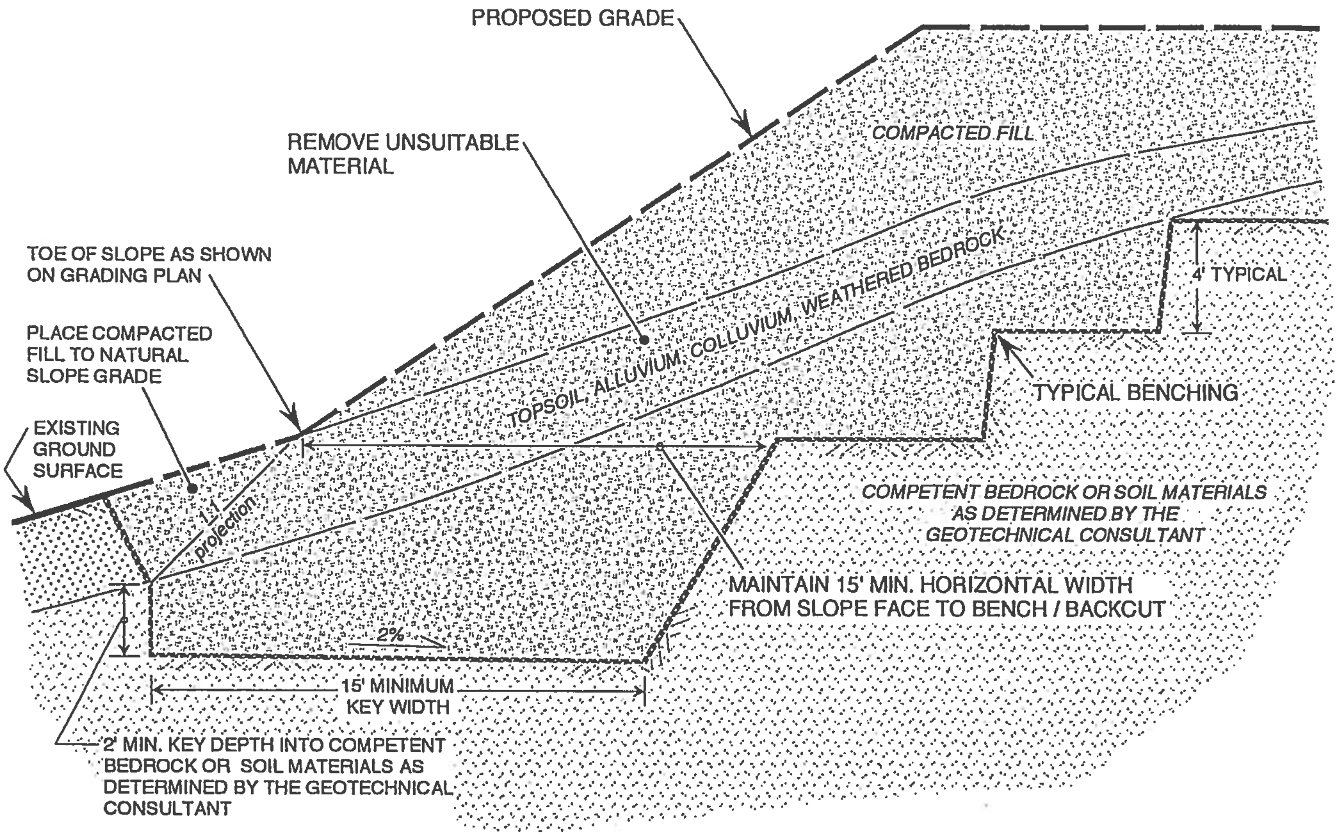


TYPICAL WINDROW DETAIL (PROFILE VIEW)



NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER

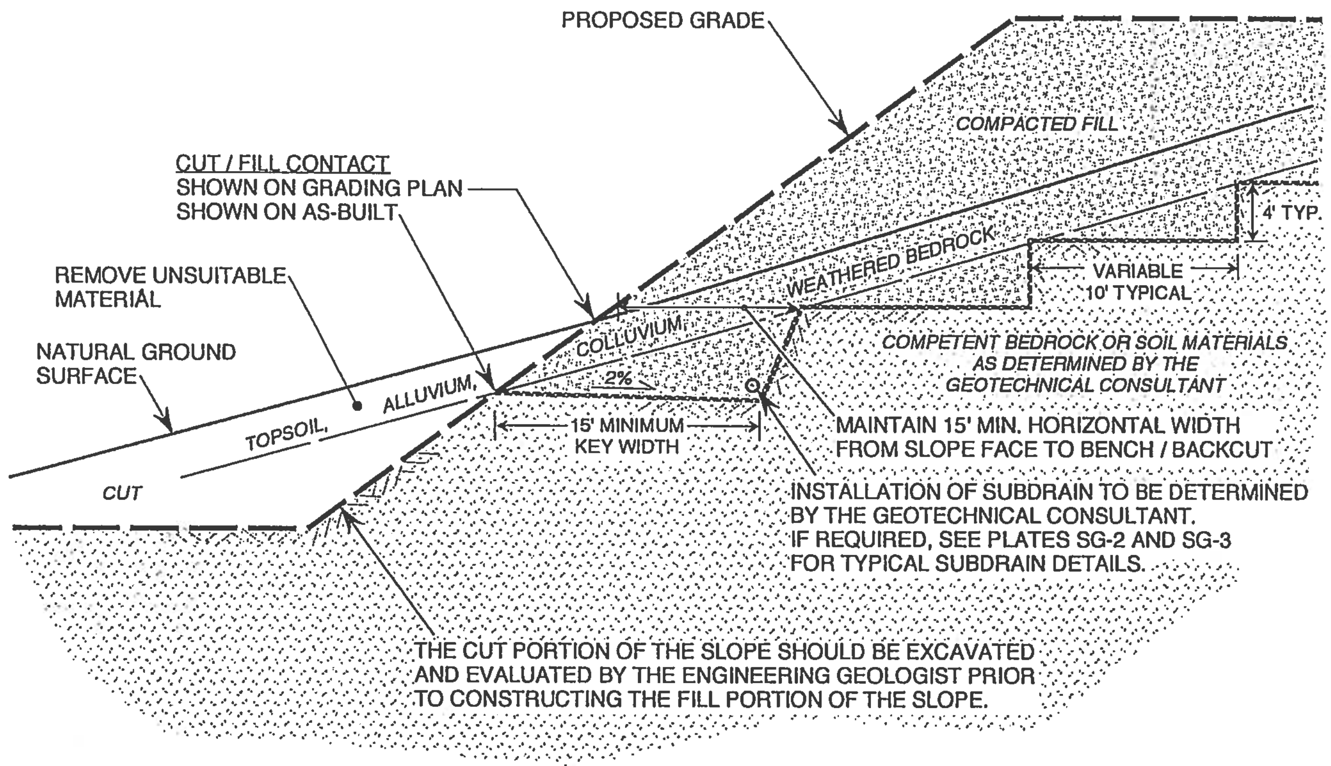
PETRA	TYPICAL ROCK DISPOSAL DETAIL	PLATE SG-4



<u>NOTES</u>:

1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.

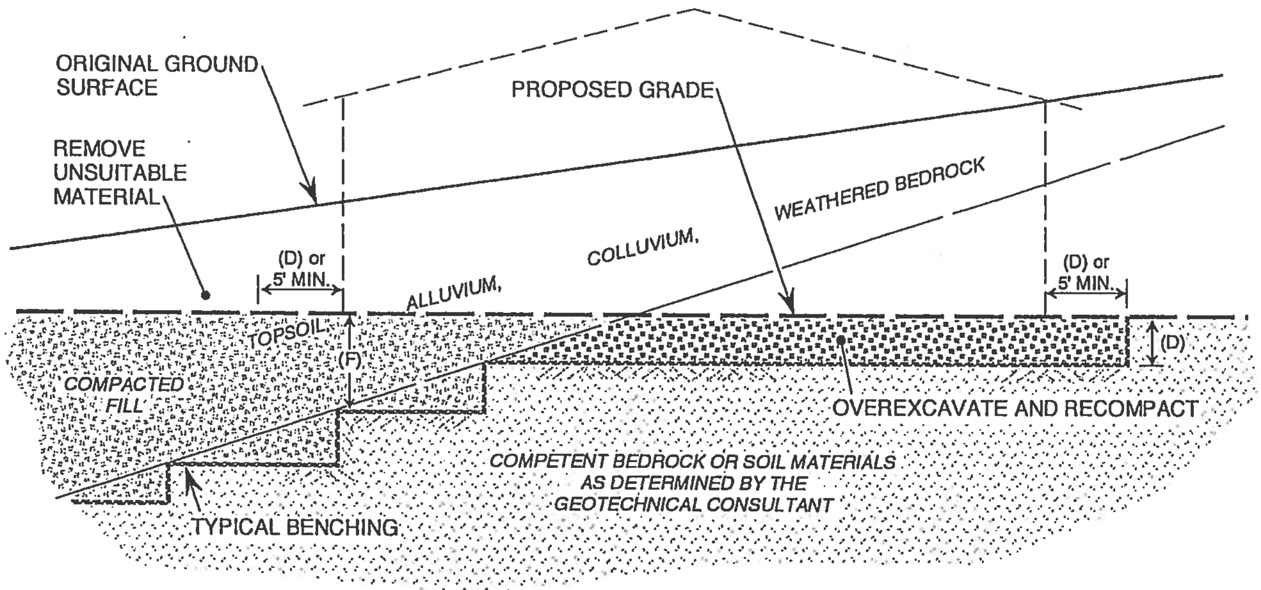
PETRA	FILL SLOPE ABOVE NATURAL SLOPE	PLATE SG-5



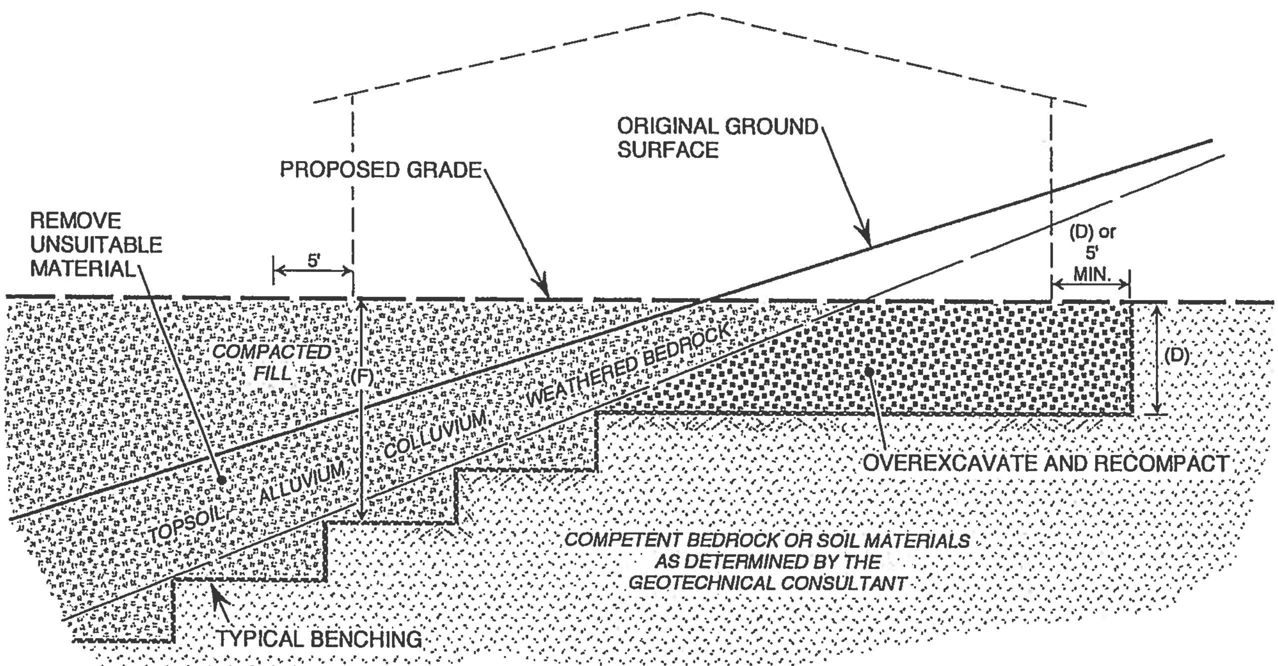
PETRA	FILL SLOPE ABOVE CUT SLOPE	PLATE SG-6

CUT LOT

UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD

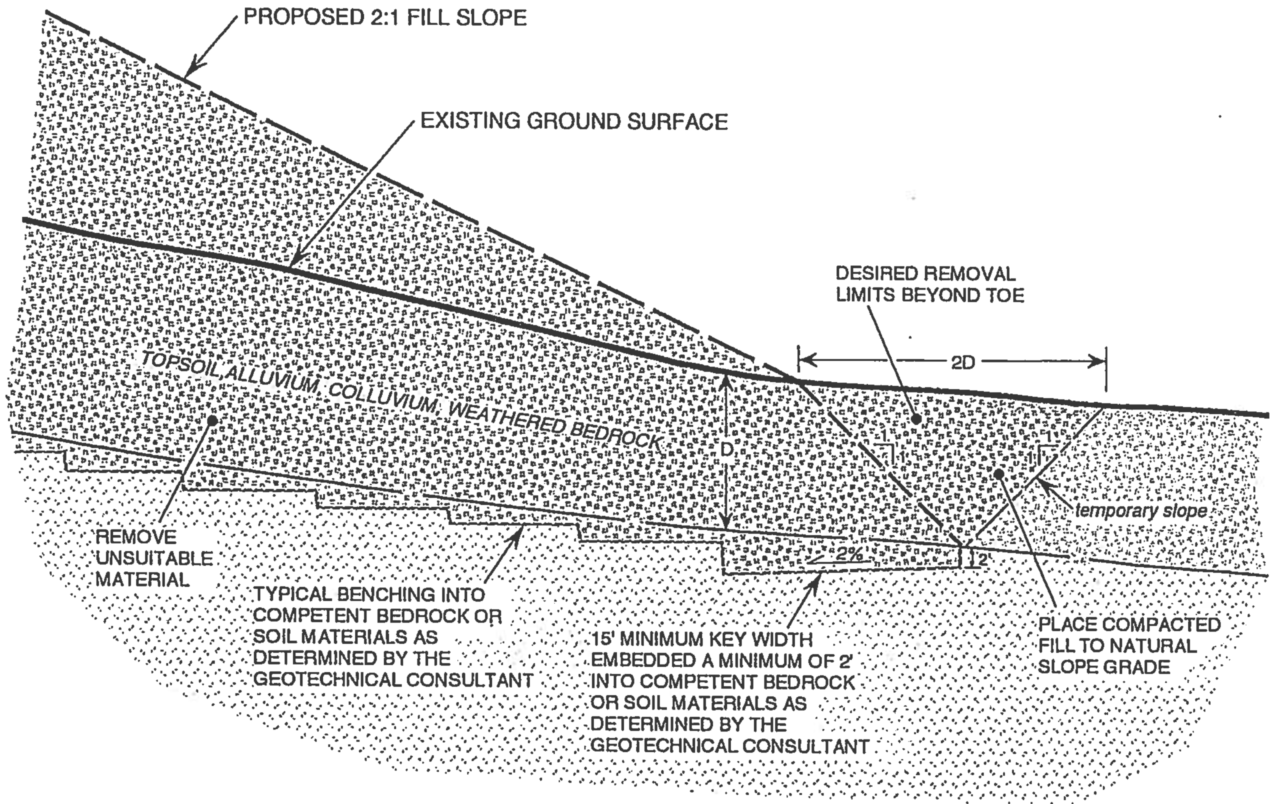


CUT-FILL TRANSITION LOT



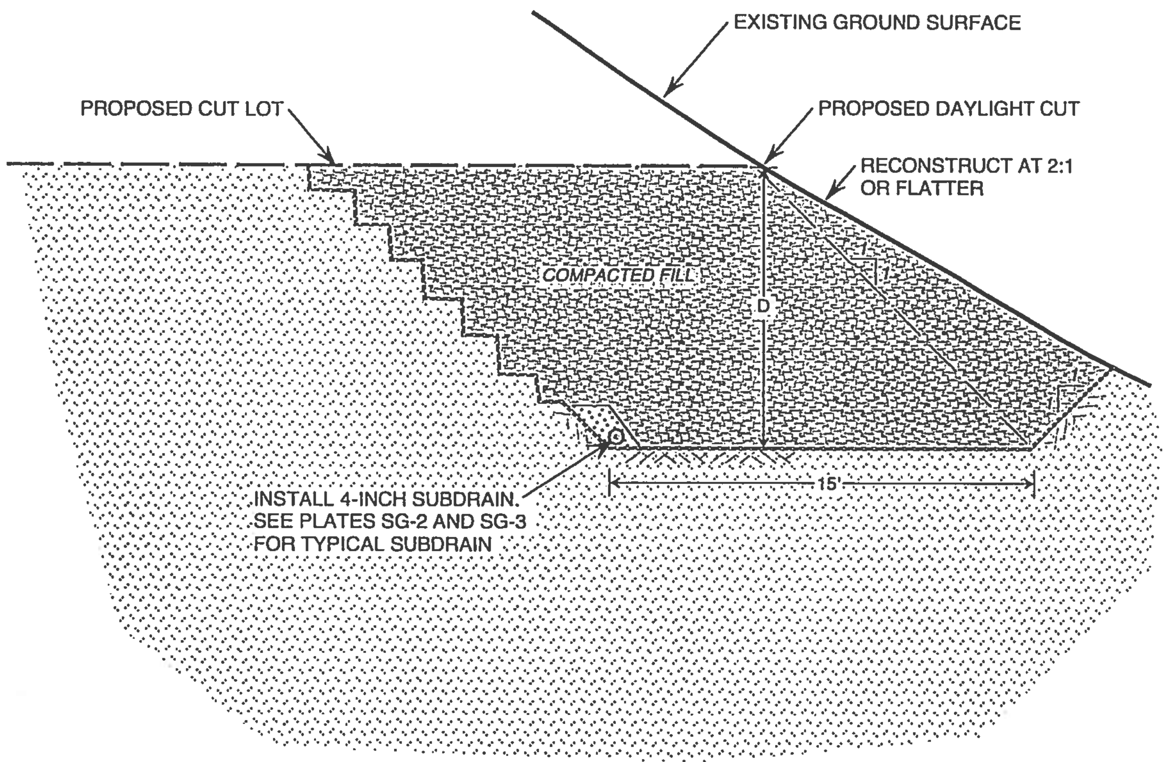
MAXIMUM FILL THICKNESS (F)	DEPTH OF OVEREXCAVATION (D)
FOOTING DEPTH TO 3 FEET	EQUAL DEPTH
3 TO 6 FEET	3 FEET
GREATER THAN 6 FEET	1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM

PETRA	CUT LOTS AND CUT-FILL TRANSITION LOTS	PLATE SG-7



D = RECOMMENDED DEPTH OF REMOVAL
PER GEOTECHNICAL REPORT

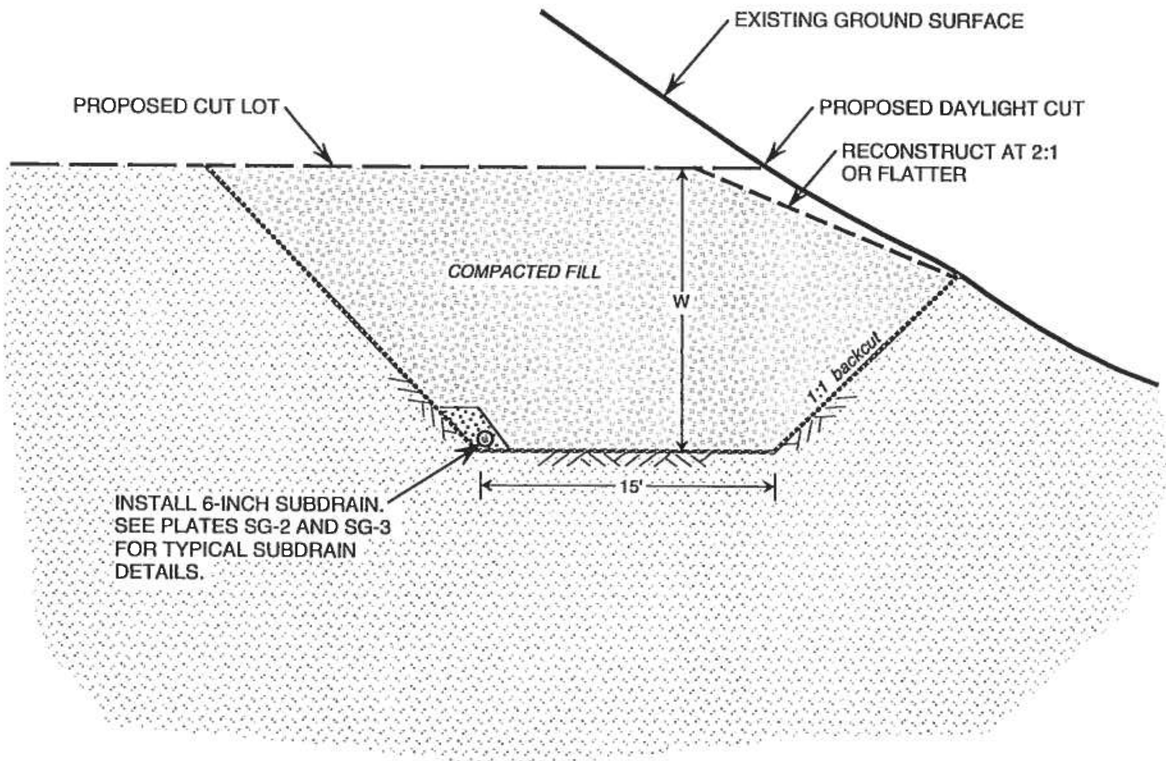
PETRA	TYPICAL REMOVALS BEYOND TOE OF PROPOSED FILL SLOPE	PLATE SG-8



NOTE:

1. "D" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA | SHEAR KEY ON DAYLIGHT CUT LOTS | PLATE SG-9



NOTE:

1. "W" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA	SHEAR KEY ON DAYLIGHT CUT LOTS	PLATE SG-9

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PETRA
past + present + future
it's in our science

Engineers, Geologists
Environmental Scientists

January 27, 2014
J.N. 13-283
Revision 1

Mr. Chris Hermann
HERMANN DESIGN GROUP, INC.
78365 Highway 111, PMB 332
La Quinta, California 92253

Subject: **Revised Geotechnical Recommendations for Design and Construction of Retaining Walls, *Whitewater Park Expansion: Amphitheater Project*, 71560 San Jacinto Drive, Rancho Mirage, California.**

Reference: Preliminary Geotechnical Investigation, *Whitewater Park Expansion*, 71560 San Jacinto Drive, City of Rancho Mirage, Riverside County, California, report by Petra Geotechnical, Inc. dated November 8, 2013.

Dear Mr. Hermann:

Petra Geotechnical, Inc. (Petra) is pleased to submit herewith our recommendation for design and construction of new retaining walls on the subject properties. Specifically, we are in receipt of a set of Architectural plans for the project prepared by McAuliffe & Associates, Inc. dated January 14, 2014. Sheets A2.0, A2.5 through A2.7 and A3.0 depict retaining wall locations and cross sections. Based on these plans, we understand that a majority of these retaining walls are retaining relatively low height (retaining less than 6 feet) backfill.

One exception with respect to retaining height is a ramp/ retaining wall that are depicted on Sheet A3.0 (northerly of Grid J' and between Grids 5' and 7') and on Sheet A2.6 (Detail 4). This wall is retaining approximately 7.5 feet of backfill. The second exception is a retaining wall along Grid 13' between Grids D' and J' (Sheet A3.0) with details shown on Sheet A4.5. This wall is retaining approximately 12 feet of backfill. Based on these information and those contained in the referenced report, the following presents our recommendations for design and construction of these retaining walls.

RECOMMENDATIONS

Earthwork

All vegetation within the area of proposed wall construction should be stripped and removed from the site. All trash and any construction debris should also be removed from the area of construction.

Laborers should manually remove any vegetation and other deleterious materials during clearing and grubbing operations.

Provided that the wall construction to commence after the completion of the site grading as recommended in the referenced report, it is expected that excavation of the site down to the proposed grades of the wall footings will expose competent compacted fill soils or native soils. These materials are suitable to support the proposed retaining walls provided that the footings are supported uniformly and entirely by any of these materials. As such, any transition from one material to another below the proposed footings should be eliminated. Similarly, if any unsuitable surficial fill is exposed at proposed grades, they should be excavated down to competent soils and then replaced as properly compacted fill.

The majority of the proposed retaining walls are expected to be supported on the compacted fill soils. Transition from compacted fill to native materials may be encountered. Native materials, if encountered in retaining wall footings excavation should be overexcavated to a minimum of 2 feet below the bottom of the footings and replaced as compacted fill. Should native materials encountered in the wall shear key trenches only (if shear keys are considered), these materials should be overexcavated a minimum of one foot below the bottom of the shear key and replaced with compacted fill.

After the walls are constructed, they should be backfilled with compacted fill. As the backfill progresses upward, the newly compacted fill should be placed on a series of level benches that tie into the competent previously compacted fill. Each bench should have an approximate horizontal width of 2 to 3 feet and a corresponding near vertical backcut that is no more than approximately 2 to 3 feet high. The actual dimensions should be determined during grading.

To provide drainage, subdrains should be installed behind each of the new retaining walls. The subdrains should be installed in accordance with the drainage recommendations provided in the "Retaining Wall Design Recommendations" section of this report.

Fill Placement

All fill should be placed in 4- to 6-inch-thick lifts, watered or air dried as necessary to achieve near optimum moisture conditions and then compacted in place to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D1557.

Geotechnical Observations

Exposed bottom surfaces in each removal/ footing excavation area should be observed and approved by the project geotechnical consultant prior to placing new fill or footings. 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 verify proper placement and adequate compaction of fill, as well as to verify compliance with the other recommendations presented herein.

Stability of Temporary Excavation Sidewalls

During site grading, a temporary excavation with sidewalls varying from approximately 2 to 5 feet (or higher) in height may be created during construction of the retaining walls. Based on the physical characteristics of these onsite materials, temporary slopes may be excavated vertically but should not exceed a height of approximately 4 feet. Where excavations exceed this height, the lower 4 feet may be cut vertical and the upper portions above a height of 4 feet should be cut back at a maximum gradient of 1:1, horizontal to vertical, or flatter.

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 that 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.

2013 CBC Seismic Design Parameters

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Section 1613 of the 2013 CBC. The method of design is dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system and on the building height and is based on the 2012 International Building Code. For structural design in accordance with the 2013 CBC, the online USGS Seismic Design Map tool was utilized to provide ground-motion parameters for the subject site. Based on the latitude, longitude and site classification, seismic design parameters and spectral response for both short periods and 1-second periods are calculated including Mapped Spectral Response Acceleration Parameter, Site Coefficient, Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter and Design Spectral Response Acceleration Parameter.

The following 2013 CBC seismic design coefficients should be used for the proposed structures. These criteria are based on the site class as determined by existing subsurface geologic conditions, on the proximity of the site to the nearby faults and on the maximum moment magnitude of the nearby faults.

2013 CBC Section 1613 Seismic Design Coefficients	
Site Latitude	33.7463
Site Longitude	-116.4138
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.500 g
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.656 g
Site Class Definition (Table 1613.5.2)	D
Site Coefficient, F_a (Table 1613.5.3(1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3(2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-36)	1.500 g
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-37)	0.984 g
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	1.000 g
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.656 g
Site Coefficient, F_{PGA} (Figure 22-7 ASCE 7-10)	1.0
Peak Ground Acceleration PGA_M (Equation 11-8.1 ASCE 7-10)	0.570 g

Retaining Wall Design Recommendations

Allowable Bearing and Lateral Resistance Values

Provided that remedial grading is performed as recommended herein, the retaining walls may be designed as conventional walls that are supported on shallow footings supported by either native soils or compacted fill. For this condition, an allowable bearing value of 1,500 pounds per square foot is recommended for design of 12-inch-wide continuous footings founded at a minimum depth of 12 inches into competent fill materials. This value may be increased by 20 percent for each additional foot of footing width and/or depth to a maximum value of 2,500 pounds per square foot. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third when designing for short duration wind and seismic forces.

For footings founded in competent native soils or compacted fill, 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, should be utilized; 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. A coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting fill materials to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces.

The above values are based on footings placed directly against competent native soils or compacted fill. In the case where footing sides are formed, all backfill placed against the footings should be compacted to at least 90 percent of maximum dry density.

Active and At-Rest Earth Pressures

As of the date of this report, it is uncertain whether the proposed retaining walls will be backfilled with on-site soils or imported granular materials. For this reason, active and at-rest earth pressures are provided below for both conditions. However, considering that the onsite earth materials have a very low to low expansion potential, the use of imported granular materials for backfilling behind the retaining walls as described in the following sections is optional.

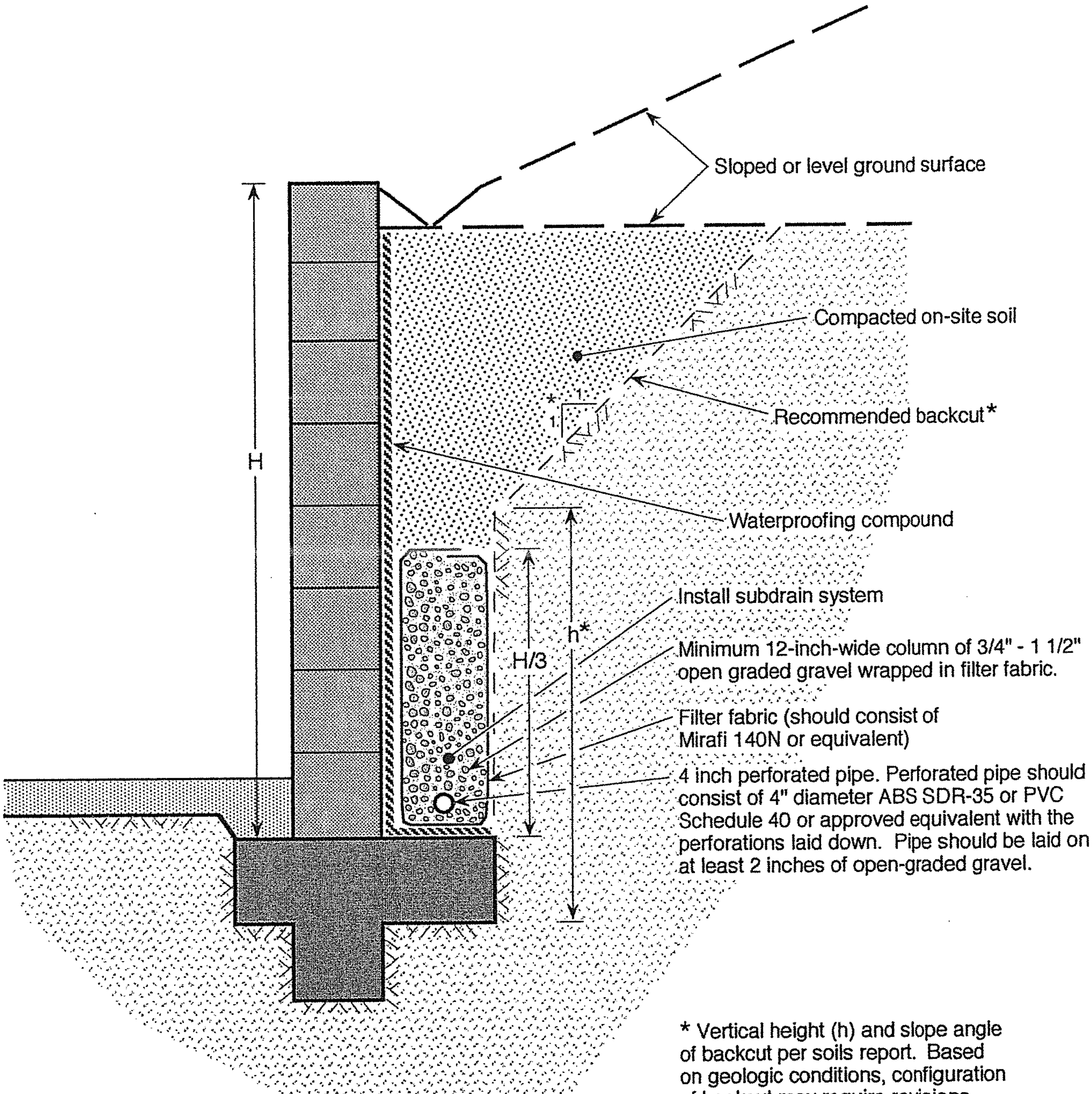
1. **Onsite Soils Used for Backfill**

   Onsite soils are very low to low expansive. Therefore, 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. These values may be reduced to 35 and 51 pounds per cubic foot, respectively, for design of cantilevered walls that will be retaining 6 feet of backfill in height or less. For walls that are restrained at the top, at-rest earth pressures of 60 and 62 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**

   Imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures 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

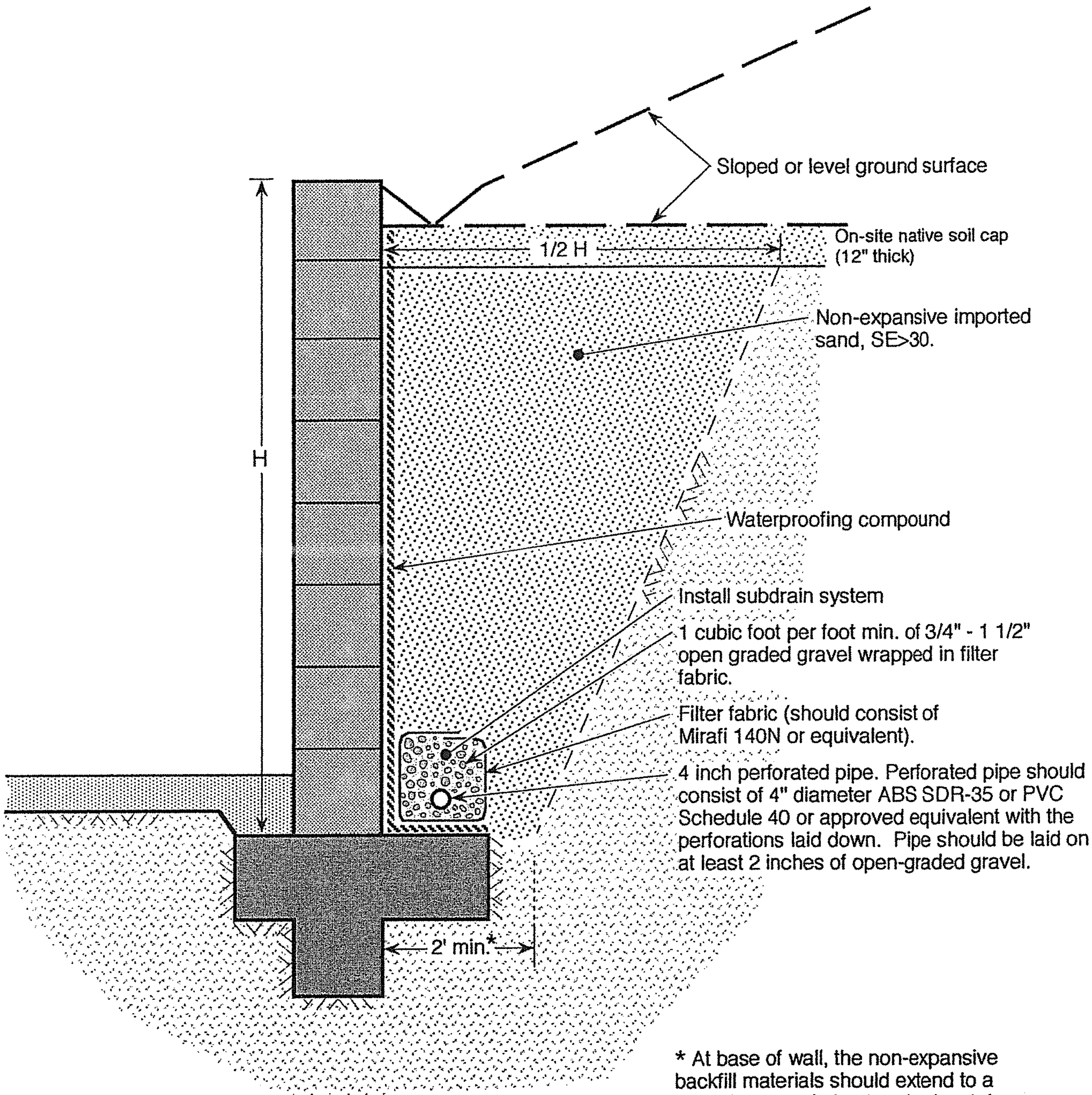
NATIVE SOIL BACKFILL



* Vertical height (h) and slope angle of backcut per soils report. Based on geologic conditions, configuration of backcut may require revisions (i.e. reduced vertical height, revised slope angle, etc.)

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-1

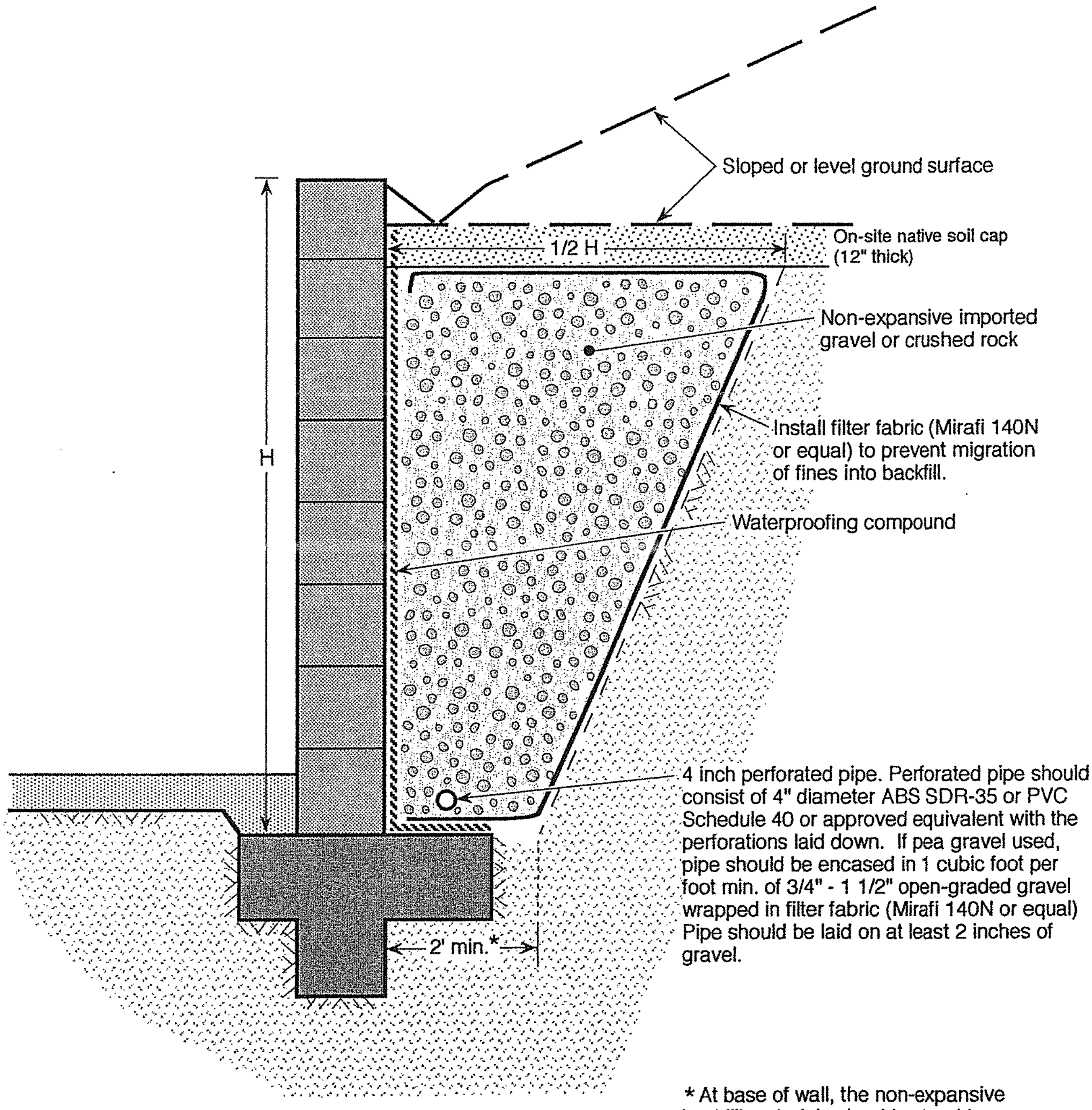
IMPORTED SAND BACKFILL



* 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.

PETRA	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.

PETRA	RETAINING WALL BACKFILL AND SUBDRAIN DETAILS	FIGURE RW-3

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.

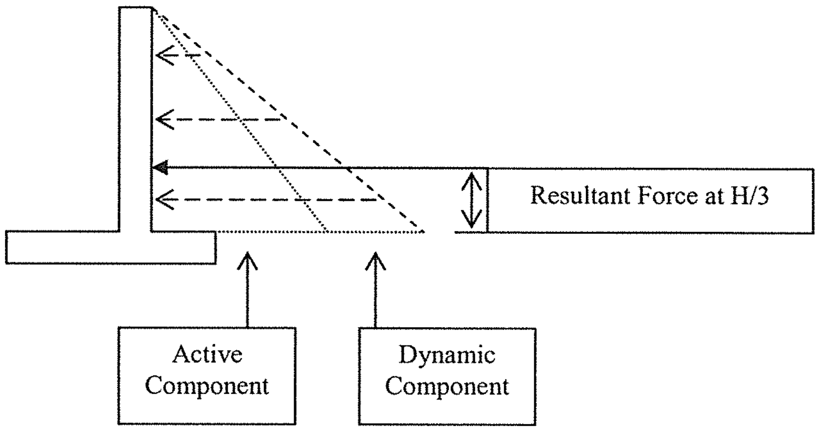
All structural calculations and details should be provided to the project geotechnical consultant for verification purposes prior to grading and construction phases.

Earthquake Loads on Retaining Walls

Section 1803.5.12 of the 2013 CBC requires the determination of lateral loads on retaining walls from earthquake forces for structures in seismic design categories D through E that are supporting in excess of 6 feet of backfill height. The following presents our recommendations for the determination of dynamic seismic lateral pressure for the design of retaining walls at the subject site.

It is customary to assume the horizontal ground acceleration vale k_h to be equal to half of the peak ground acceleration. (See, for example, County of Los Angeles, Department of Public Works, Manual for Preparation of Geotechnical Reports, July 1, 2013.) Thus, $k_h = ½ (PGA) = ½ (0.570 g) = 0.285 g$.

In our evaluation of the earthquake loads on retaining walls, we used the Mononobe-Okabe procedure to determine the total (static and dynamic combined) lateral loads on the retaining walls. Based on our analysis, a dynamic load equivalent to a fluid having a unit weight of 25 pcf should be used. Note that the active and seismic earth pressures have a triangular distribution with the largest load occurring at the bottom of the wall (Al Atik, Linda, and Sitar, Nickolas, 2007, Development of Improved Procedures for Seismic Design of Buried and Partially Buried Structures, PEER 2007/06, dated June).
The distribution of the seismic lateral load is as follows



Geotechnical Observation and Testing

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

Drainage and Waterproofing

Perforated pipe and gravel subdrains should be installed behind all 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 3/4-inch to 1½-inch open-graded gravel. If onsite soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-half 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 rocks are used as backfill, the open-graded gravel should extend above the wall footing to a minimum height of 1 foot above the footing. 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. 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. However, since the on site soil materials are expected to be low to moderately expansive, they 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 sand, pea gravel, crushed rock, or imported granular soils for backfill that exhibit a Very Low expansion potential (Expansion Index of less than 20). 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 that they are installed as shown on Figures RW-2 and RW-3.

Where on-site soils or imported sand are used for backfill, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve optimum or slightly above 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 that adequate compaction has been achieved.

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 mitigate 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 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.

We appreciate this opportunity to be of service. If you have questions, please contact this office.

Respectfully submitted,
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