Appendix D

Geotechnical Investigation
Proposed Ten Building Commercial/Light Industrial Development
Associated Infrastructure and Parking, 9.28-Acre Parcel South of Avenue
15 on Little Morongo Land, Parcel Number 37138,

City of Desert Hot Springs, California,

prepared by
Petra Geosciences
September 13, 2016

GEOTECHNICAL INVESTIGATION, PROPOSED TEN BUILDING COMMERCIAL/LIGHT INDUSTRIAL DEVELOPMENT ASSOCIATED INFRASTRUCTURE AND PARKING 9.28–ACRE PARCEL SOUTH OF AVENUE 15 ON LITTLE MORONGO LANE PARCEL NUMBER 37138, CITY OF DESERT HOT SPRINGS, CALIFORNIA

GABRIEL LUJAN & ASSOCIATES

September 13, 2016
J.N. 16-208

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GABRIEL LUJAN & ASSOCIATES
736-947 Cook Street, Suite 104
Palm Desert, California 92211

Attention: Mr. Gabriel Lujan

Subject: Geotechnical Investigation, Proposed Ten Building Commercial/Light Industrial Development, Associated Infrastructure and Parking, 9.28–Acre Parcel South of Avenue 15 on Little Morongo Lane, Parcel Number 37138, City of Desert Hot Springs, California

Dear Mr. Lujan:

Presented herewith is our geotechnical investigation report for the proposed buildings at the subject site. This work was performed in general accordance with the scope of services outlined in our Revised Proposal No. 16-208P, dated June 29, 2016. This report presents the results of our field investigation, laboratory testing, and our engineering and geologic judgment, opinions, conclusions and recommendations pertaining to geotechnical design aspects of the proposed development.

It has been a pleasure to be of service to you on this project. Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Alan Pace
Senior Associate Geologist

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PURPOSE AND SCOPE OF SERVICES

Petra Geosciences, Inc. (Petra) is presenting herewith the results of our geotechnical investigation for the subject property. The purpose of this investigation was to 1) obtain information regarding surface and subsurface geologic conditions within the project area, 2) evaluate the engineering properties of the onsite soil materials, and 3) provide conclusions and recommendations for design and construction of the proposed 10 building commercial/light industrial development. To accomplish these objectives, our scope of services included the following:

1. Performed an initial site reconnaissance.
2. Reviewed existing published and unpublished reports and maps concerning soil and geologic conditions within and adjacent to the site.
3. Reviewed plans and project information made available to us at the time of our investigation. Only a preliminary site plan was available at this time.
4. Coordinated with the local underground utilities locating service (Underground Service Alert) and Client to obtain an underground utility clearance and all available onsite utility line plans, respectively, prior to commencement of the subsurface investigation.
5. Coordinated field activities and exploration point locations with the project architect and civil engineer (if one had been appointed as of the date of our field investigation).
6. Advanced six exploratory test pits in conformance with the client responsibilities, exclusions, limitations of Terms and Conditions of Agreement provided herein. The test pits were excavated using a rubber-tired backhoe provided by Petra's subcontractor.
7. Logged and field classified soil materials encountered in the exploratory test pits in accordance with the visual-manual procedures outlined in the Unified Soil Classification System and the American Society for Testing and Materials (ASTM) Procedure D 2488 and ASTM D 2487. All field activities were performed by, or under the direct supervision of, a State of California Certified Engineering Geologist.
8. Collected bulk and undisturbed soil samples for laboratory analysis. Relatively undisturbed samples were retrieved at 3- to 10-foot depth intervals utilizing a 2.4-inch inside diameter, modified California split-spoon sampler in general accordance with ASTM Test Method D 3550.
9. Performed laboratory analyses on soil samples retrieved during our field investigation. Lab tests may include all or a portion of the following:

- In-situ moisture content and dry density determination
- Maximum dry density and optimum moisture content
- Hydrometer (grain size analyses)
- Expansion index
- Shear strength
- Soluble sulfate and chloride content
- Soil pH and minimum resistivity

10. In accordance with the requested scope of services, our study included performing two soil percolation tests to support the preliminary design of onsite storm water retention/dissipation systems. Since the design of the onsite storm water system has not as yet been finalized, the field testing included in this proposal will be considered as a pilot study and conducted based on an assumed design and percolation depth of approximately 5 to 10 feet below ground surface. The tests were conducted as follows:

    - After the completion of sampling and logging, two surface level percolation test were conducted in two of the excavated test pits. Once the test was set up, the test hole was filled with water and allowed to pre-soak.

    - Following pre-soaking, falling-head percolation test data was collected at a pre-determined time interval until a stable percolation rate had been established.

    - Upon completion of percolation testing, the test pits were backfilled with onsite soil.

    - Numerical analyses was performed on the field data in order to determine the absorption rate of the geologic materials exposed on the test pit bottom and sidewalls. The Porchet Method was applied in order to consider only vertical percolation.

    The results of our pilot percolation study were incorporated into this design-phase report for the site.

11. Performed engineering analyses of field and laboratory data. Our analyses included determination of seismic design parameters per the applicable sections of the 2013 California Building Code (CBC), geotechnical foundation design parameters (including allowable bearing capacity), and potential total and differential settlement due to seismic (i.e., liquefaction, if applicable) and static loading conditions.

12. If requested, attend one meeting with the Client and other members of the design team to discuss the results of our investigation and preliminary recommendations for site grading and building foundation design.

13. Prepared a single design-phase geotechnical report for the project site. This report presents the results of our subsurface investigation and recommendations for site grading, building foundation design and construction, and design and construction of exterior hardscape and pavement in accordance with applicable sections of the 2013 California Building Code. It is assumed that one electronic draft copy of the report, plus three (3) final hard copies and one final electronic copy (in PDF format) will be required for distribution to the design team and regulatory agency.

LOCATION AND SITE DESCRIPTION

The site is in a natural undeveloped condition with sparse desert vegetation. The site covers approximately a 9.28 acre parcel of land located to the east of Little Morongo Road and to the west of a currently dry storm water channel. The triangular shaped property is fairly flat with elevations ranging from 944 on the northern edge to 902 feet on the southern section. An elevated levee and unmaintained dirt access road were present along the eastern portion of the site along the drainage canal. Overhead transmission lines are situated along the western and southern borders of the site.

PROPOSED CONSTRUCTION AND GRADING

Based on the conceptual site plan prepared by Egan and Egan, Inc., we understand that proposed development at the site includes construction of 10 buildings ranging in size from 32,164 square-feet to 31,008 square-feet for a total of 127,786 square-feet, and associated infrastructure and parking. The proposed site plan can be seen on Figure 3.

Structural details for the proposed buildings have not as yet been provided to our firm; however, it is anticipated that they will be one- to two- stories in height and of wood frame or light gauge metal frame construction with slabs constructed on grade. For this type of construction, it is anticipated that relatively light foundation loads will be imposed on the subgrade soils.

Based on the conceptual site plan that has been provided by Egan and Egan, Inc., and given the relatively flat topography of the proposed development design, earthwork within the site is generally expected to entail design cuts and fills of up to approximately 5± feet in order to achieve proposed grades. It should be noted, however, that the ultimate fill thicknesses throughout the site will be greater due to the recommended remedial grading (i.e., removal and recompaction of existing unsuitable surficial soils) as presented in subsequent sections of this report.

FIELD INVESTIGATION

Our subsurface exploration was performed on August 17, 2016 and included excavating six exploratory test pits (identified herein as Test Pits TP-1 through TP-6) to depths ranging from approximately 4 to 13 feet below the existing ground surface. The test pits were excavated utilizing a rubber-tire backhoe. The locations of our test pits are shown on Figure 2 and descriptive exploration logs are provided in Appendix A.

Earth materials encountered in each of the exploratory borings were field classified and logged in accordance with Unified Soil Classification System procedures. In addition, our subsurface exploration included the collection of bulk samples and relatively undisturbed samples of the subsurface soils for laboratory testing purposes. Bulk samples consisted of selected earth materials obtained at various depth intervals from selected borings. Relatively undisturbed samples were collected using a 3-inch, outside-diameter, modified California split-spoon soil sampler lined with 1-inch high brass rings. The modified sampler was driven with successive 30-inch drops from a standard 140-pound sampling hammer. Blow counts for each 6-inch driving increment were recorded on the field logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing. Logs of the borings are presented in Appendix A.

LABORATORY TESTING

In order to evaluate the engineering properties of the onsite soils, a number of laboratory tests were performed on selected samples considered representative of the materials encountered during our investigation. Laboratory tests included the determination of in-place moisture content and dry density, maximum dry density and optimum moisture content, expansion potential, hydrometer (grain size analyses), remolded direct shear, soluble sulfate and chloride content, and soil pH and minimum resistivity. A description of laboratory test methods is provided in the Laboratory Test Procedures section of this report (Appendix B) and summaries of the test data are presented on the Exploration Logs (Appendix A) and in Appendix B.

FINDINGS

Regional Geology

The site is located in Desert Hot Springs area of the upper Coachella Valley at the juncture of three natural geomorphic provinces of California; the Transverse Ranges, the Peninsular Ranges, and the Colorado Desert. The Coachella Valley lies within the northern portion of the Salton Trough, a large northwest-trending structural depression that extends approximately 180 miles from San Gorgonio Pass to the Gulf of California. Part of this basin, including the Salton Sea, lies below sea level and has progressively been filling with sediments eroded from local bounding mountain ranges, sediments from the Colorado River, and by incursions by the Gulf of California since at least the late-Miocene Epoch. Sediments within the Salton Trough are estimated to be over two to five miles thick (Kohler and Fuis, 1986; Fuis and Kohler, 1984; Biehler, et. al., 1964). It is considered the dominant feature of the Colorado Desert Geomorphic Province, and is well known for its exposures of the San Andreas Fault and related fault systems that form the margin between the Pacific and North American Plates.

Desert Hot Springs is located approximately 100 miles to the east of Los Angeles. The area is bordered by the Little San Bernardino Mountains to the north and east, the San Gorgonio Mountains to the west, and the Palm Springs area of the Coachella Valley to the south. The site lies on an alluvial fan of the San Gorgonio and

Little San Bernardino Mountains that consists of sand and gravel of mafic and gneissic detritus derived from the rising San Bernardino Mountains to the north. (Dibblee, 2004). The site does not lie within an Alquist-Priolo Earthquake Fault Zone (CDMG, 1974a; 1974b). No seismic hazard zone maps have been prepared for the Desert Hot Springs Quadrangle.

Bedrock materials in the vicinity of the site can generally be described as alluvial fan sediments, moderately lithified, with only the uppermost part of the roughly 4,500 foot thick formation exposed in the immediate area. The age of the bedrock is considered to be in the upper Miocene. (Dibblee, 2004).

Local Geology and Subsurface Conditions

Artificial Fill

The artificial fill encountered consisted of gray, fine- to medium-grained sand to silty sand that was generally dry and in a loose to medium dense state. The fill ranged from approximately 0 to 1 foot in depth.

Alluvial Fan Deposits

Older Alluvial Fan deposits were encountered in all of the exploratory test pits to the maximum depth explored. These deposits consisted primarily of medium- to coarse-grained sand to silty sand with increasing numbers of gravels and boulders with depth. The sands were observed to be dry, medium dense, and were generally gray in color.

Bedrock: Timeoteo Formation

Bedrock was not observed in our exploratory test pits, but is projected to be located at depths in excess of 100 feet below the existing ground surface.

Groundwater

No groundwater or seepage was encountered within exploratory test pits, at least to the total depth explored (14 feet). The site is located within the Coachella Valley Groundwater Basin (California Department of Water Resources [CDWR], Water Data Library, 2014). Groundwater depth varies within the area and flow direction beneath the subject site is unknown; although, it is believed to be toward the south-southeast and the Salton Sea. Two groundwater wells were listed within the immediate area of the subject site on the CDWR Water Data Library (2016). Based on our review of a well located approximately 0.7 miles east-southeast of the site (Well 339320N1165142W001) between 2015 and 2016, groundwater was reported at depths of 195± to 204± feet below the ground surface. A similar well located approximately 0.7 miles east-northeast of the site (Well 339390N1165135W001) indicated that between 2015 and 2016, groundwater was at depths of 217± to

232 ± feet below the ground surface. From this information, we can extrapolate that groundwater levels at the site may be located between roughly 232± and 244± feet in depth below existing ground surface.

Faulting

Based on our review of the referenced geologic maps and literature, no active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act (Bryant and Hart, 2007). The Alquist-Priolo Earthquake Fault Zoning Act (AP Act) defines an *active fault* as one that "has had surface displacement within Holocene time (about the last 11,000 years)." The main objective of the AP Act is to prevent the construction of dwellings on top of active faults that could displace the ground surface resulting in loss of life and property.

According to the maps of known active-fault near-source zones (ICBO, 1998), the active San Gogonio Pass segment of the San Andreas fault is located approximately 2.6 miles or 4.2 kilometers to the southwest of the subject site. The San Andreas Fault is a major dextral right lateral strike-slip fault zone that extends for approximately 1,100 kilometers from the Gulf of Mexico to the coast of Northern California. This zone has a history of moderate to high seismic activity and has generated several historic earthquakes greater than magnitude 4.0, including the January 9, 1857 Fort Tejon earthquake (magnitude 7.9 approximately).

Liquefaction and Seismically-Induced Settlement

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

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

In view of the depth to groundwater and moderately dense to very dense alluvial fan materials that underlie the site, the potential for manifestation of liquefaction induced features or significant dynamic settlement resulting from liquefaction is considered negligible.

Seismically-Induced Flooding

The types of seismically induced flooding that are generally considered as possible hazards to a particular site include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir retention structure upstream of the site. Since the site lies approximately 923 feet above the Pacific Ocean, and does not lie in close proximity to an enclosed body of water or downstream of a major reservoir retention structure, the probability of flooding from a tsunami, seiche or dam-break is considered to be negligible.

Earthquake Loads

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific acceleration response spectrum. To provide the design team with the parameters necessary to construct the site-specific acceleration response spectrum for this project, we used the computer applications that are available on the United States Geological Survey (USGS) website, http://geohazards.usgs.gov/. Specifically, the Design Maps website http://geohazards.usgs.gov/designmaps/us/application.php was used to calculate the ground motion parameters.

To run the above computer applications, site latitude, longitude, risk category and knowledge of "Site Class" are required. The site class definition depends on the average shear wave velocity, Vs_{30}, within the upper 30 meters (approximately 100 feet) of site soils. A shear wave velocity of 250 meters per second for the upper 100 feet was used for the site based on engineering experience and judgment.

The following table, Table 1, provides parameters required to construct the site-specific acceleration response spectrum based on 2013 CBC guidelines. Printouts of the computer output are attached in Appendix C.

Table 1

SEISMIC DESIGN PARAMETERS

Ground Motion Parameters	Reference	Parameter Value	Unit
Latitude (North)	-	33.93774	°
Longitude (West)	-	-116.52675	°
Site Class Definition	Table 20.3-1, ASCE 7-10	D	-
Assumed Risk Category	Table 1604.5, CBC 2013	II	-
S_s - Mapped Spectral Response Acceleration	Figure 1613.3.1(1), CBC 2013	2.782	g
S_1 - Mapped Spectral Response Acceleration	Figure 1613.3.1(2), CBC 2013	1.031	g
F_a - Site Coefficient	Table 1613.3.3(1), CBC 2013	1.0	-
F_v - Site Coefficient	Table 1613.3.3(2), CBC 2013	1.5	-
S_{MS} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration	Equation 16-37, CBC 2013	2.782	g
S_{M1} - Adjusted Maximum Considered Earthquake Spectral Response Acceleration	Equation 16-38, CBC 2013	1.547	g
S_{DS} - Design Spectral Response Acceleration	Equation 16-39, CBC 2013	1.855	g
S_{D1} - Design Spectral Response Acceleration	Equation 16-40, CBC 2013	1.031	g
T_o - (0.2 S_{D1}/ S_{DS})	Section 11.3, ASCE 7-10	0.111	s
T_s - (S_{D1}/ S_{DS})	Section 11.3, ASCE 7-10	0.556	s
T_L - Long Period Transition Period	Figure 22-12, ASCE 7-10	8	s
F_{PGA} - Site Coefficient	Figure 22-7, ASCE 7-10	1.0	-
PGA_M - Peak Ground Acceleration at MCE [1]	Equation 11.8-1, ASCE 7-10	0.659	g
PGA – Design Level – (0.4 S_{DS}) [2]	Equation 11.4-5, ASCE 7-10	0.551	g
C_{RS} - Short Period Risk Coefficient	Figure 22-17, ASCE 7-10	0.954	-
C_{R1} - Long Period Risk Coefficient	Figure 22-18, ASCE 7-10	0.920	-
Seismic Design Category [3]	Section 1613.3.5, CBC 2013	E	-

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

[2] PGA Calculated at the Design Level of 2/3 of MCE which is approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).

[3] Seismic Design Category may be calculated by the structural engineer in accordance with the alternate design procedures of Section 1613.3.5.1 based on structural characteristics in addition to the ground motion parameters, this may supersede the category listed herein.

References: USGS Seismic Design Web Application – http://geohazards.usgs.gov/designmaps/us/application.php
California Building Code (CBC), 2013, California Code of Regulations, Title 24, Part 2, Volume I and II.
American Society of Civil Engineers (ASCE/SEI), 2010, Minimum Design Load for Buildings and Other Structures, Standards 7-10.
Federal Emergency Management Agency (FEMA), 2009, NEHERP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-750).

CONCLUSIONS AND RECOMMENDATIONS

General Feasibility

From a soils engineering and engineering geologic point of view, the subject site is considered suitable for the proposed development provided the recommendations of this report are incorporated into the design criteria and project specifications. Furthermore, it is our opinion that the proposed grading and construction will not adversely affect the geologic stability of adjoining properties in an adverse manner provided grading and construction are performed in accordance with current standards of practice, all applicable grading ordinances and the recommendations presented in this report.

Grading Plan Review

This report has been prepared based on a conceptual site plan provided by Egan and Egan, Inc. rather than a detailed grading plan depicting the proposed grades and construction. As such, the recommendations provided in this report should be considered tentative until a finalized grading plan is available and reviewed by our firm. Additional recommendations and/or modification of the recommendations provided herein may be necessary depending on our review of the final grading plan.

Earthwork and Grading

General Specifications

All earthwork and grading should be performed in accordance with all applicable requirements of the grading and excavation codes of the City of Desert Hot Springs and in compliance with all applicable provisions of the 2013 California Building Code (CBC). Grading should also be performed in accordance with the recommendations provided in this report.

Site Clearing

Any structural materials present at the site should be demolished and removed in their entirety. Any existing utility lines that are not to remain should be removed to the property limit and the remaining portion should be properly abandoned. The resultant void should be backfilled with properly compacted fill.

Clearing operations should also include the removal existing vegetation. Large shrubs and trees, when removed, should be grubbed out to include their stumps and major root systems. During site grading, laborers should be provided to clear from fill soils any roots, tree branches, and other deleterious materials missed during initial clearing and grubbing operations.

The project geotechnical consultant should be notified at the appropriate times to observe general clearing operations. Should any unusual soil conditions or buried structures be encountered during demolition operations or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Ground Preparation – Building Area

The upper portion of the site soils should be removed and replaced as compacted fill. The depth of over-excavation should extend a minimum of 5 feet below the existing grade or 1 feet below the deepest footing, whichever is deeper. In order to provide both vertical and lateral support of the footings, the horizontal limits of over-excavation and recompaction should extend to a minimum horizontal distance equal to the depth of overexcavation or 5 feet beyond the outer edges of perimeter footings including any footings supporting overhead canopies or structures connected to the building, whichever is further. After completion of over-excavation and prior to fill placement, the exposed bottom surfaces should be scarified to a minimum depth of 6 inches, thoroughly flooded to above optimum moisture content, and then recompacted to a minimum relative compaction of 90 percent of the applicable maximum dry density as determined in accordance with ASTM Test Method D 1557.

Ground Preparation - Parking Lot and Exterior Hardscape Areas

Within the paved parking lot and exterior hardscape areas exposed soils should be overexcavated to a depth of at least 24 inches, and the underlying 6 inches processed in-place. That is, prior to replacing the excavated soils as compacted fill, the exposed ground surface should be scarified to a depth of at least 6 inches, thoroughly flooded to above optimum moisture content, and then recompacted in-place to a minimum relative compaction of 90 percent. The horizontal limits of overexcavation should extend to a minimum horizontal distance of 2 feet beyond the perimeter of the proposed improvements, where possible. Where removals are limited by property lines, the removals should be performed within about 1 foot from the property lines.

It must be emphasized that the depths of remedial grading as provided above are estimates only and are based on conditions encountered at the exploratory test pit locations. Subsurface conditions can and usually do vary between points of exploration. For this reason, the actual removal depths will have to be determined on the basis of in-grading observations and testing performed by a representative of the project geotechnical consultant. Remedial grading and ground preparation should be performed prior to placing compacted fill.

Excavation Characteristics

Based on the results of our subsurface investigation, all soils within the site are expected to be readily excavatable with conventional earthmoving equipment.

Some oversized rock will likely be generated during remedial grading (i.e., rock greater than 12-inch maximum dimension). Since deep fill areas are not anticipated to be present, rock disposal methods, such as windrows or rock blanket, will not be possible. As such, oversized materials will need to be either disposed of offsite, broken down to acceptable size and then used in the fill areas, or utilized in another manner acceptable to the geotechnical consultant and the governing agency.

Imported Soils

If imported soils are required to complete the planned grading, these soils should consist of clean materials devoid of rock exceeding a maximum dimension of 6 inches, as well as organics, trash and similar deleterious materials. To avoid making revisions to the foundation design, imported soils should also exhibit a very low expansion potential (Expansion Index 0-20). Prospective import soils should be observed, tested and approved by the geotechnical consultant prior to importing the soils to the site. It is recommended that the project environmental consultant should also be notified so that they can confirm the suitability of the proposed import material from an environmental standpoint.

Stability of Temporary Excavation Sidewalls

Average remedial removals within the subject site are anticipated to be about 5 feet below the existing ground surface. As a result, it is likely that excavations with sidewalls ranging up to approximately 5 feet in height will be temporarily created during grading. Based on our subsurface investigation, the sidewalls of these temporary excavations may expose non-cohesive silty sands and sands. Therefore, temporary backcut slopes ***located away from the site boundaries and foundations of existing offsite structures*** may be cut vertical to a maximum height of 3 feet. If excavation sidewalls are required to exceed a height of 3 feet, the lower 3 feet may be cut vertical and the upper portions above a height of 3 feet should be laid back at a slope ratio of 1:1, horizontal to vertical, or flatter. If clean sands or gravels are encountered then temporary excavations may not be able to stand vertically, and should be sloped at a minimum ratio of at least 1:1 or flatter as required to maintain stability.

Sidewalls of temporary excavations that are excavated to the above configurations are expected to remain sufficiently stable during grading. However, all temporary excavations should be observed by a representative of our firm for any evidence of potential instability. Depending upon the results of these observations, revised temporary slope configurations may become necessary.

Other factors that should be considered with respect to the stability of temporary excavation sidewalls include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, and weather conditions at the time of construction. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed. No temporary excavations along the property lines should be left open, and the backfill should be placed as soon as possible. **The grading contractor is solely responsible for ensuring the safety of construction personnel and the general public**

Grading Adjacent to Property Boundaries and Existing Structures

As stated previously, remedial removal depths within the subject site are anticipated to be about 5 feet below the existing ground surface. Based on the relatively non-cohesive nature of on-site soils, temporary backcut slopes adjacent to these boundaries will generally be restricted to a slope ratio of 1:1 (horizontal to vertical) or flatter. If encroachment into these adjacent properties is not possible during grading, a relatively narrow wedge of potentially compressible soil will be left in-place along the property perimeter that will extend into the site to a horizontal distance equal to the vertical depth of the required remedial removals. If unsuitable soils cannot be removed using slot-cutting techniques and must be left in place, some degree of distress may result to the proposed improvements if they are constructed within the zone of influence of these unsuitable soils (generally regarded as a 1:1 projection upward from the bottom of the temporary backcut slope).

Considering the location of the proposed building structure as shown on the conceptual site plan and anticipated zone of the influence of the unsuitable soils that will be left in place along the site perimeter, it appears unlikely that the foundations for the proposed structures will be located within the zone of influence of these unsuitable soils. However, foundations for masonry block walls (both retaining and non-retaining) that may be proposed along the property lines may be underlain by these low-density soil materials. If permission to encroach into the adjacent properties during grading cannot be obtained, the perimeter walls should be supported on deepened continuous footings or shallow caissons to transfer the loads of proposed retaining walls below the unsuitable soils to be left in-place.

If any walls or structures are proposed within the zone of compressible soils additional recommendations may be provided by the geotechnical consultant. Plans for any proposed perimeter structures should be forwarded to the geotechnical consultant so that we may provide the appropriate recommendations.

Fill Placement

All fill should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve above-optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each soil type should be determined in general accordance with the current revision of Test Method ASTM D 1557.

Geotechnical Observations and Testing During Grading

Exposed bottom surfaces in each remedial removal area should be observed and approved by a representative of the geotechnical consultant prior to placing fill. In addition, a representative of the geotechnical consultant should be present onsite during grading operations to observe and test the fill materials, as well as to document compliance with the other recommendations presented herein.

Volumetric Changes - Bulking, Shrinkage and Subsidence

An approximate shrinkage factor estimated at approximately 10 to 15 percent is expected to occur when excavated on-site soils are replaced as properly compacted fill. A subsidence estimated at between 0.10 feet is also expected to occur when exposed bottom surfaces in removal areas are scarified and recompacted as recommended herein.

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

Post-Grading Considerations

Utility Trenches

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Onsite earth materials cannot be adequately densified by flooding and jetting techniques. Therefore, trench backfill materials should be placed in lifts no greater than approximately 12 inches in thickness, watered or air-dried as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to evaluate whether compliance with project specifications is being attained.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand having a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to evaluate whether an adequate degree of compaction is being attained.

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

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

Site Drainage

Positive surface drainage systems consisting of a combination of sloped concrete flatwork/asphalt pavement, sheet flow gradients, swales and surface area drains (where needed) should be provided around the buildings and within the planter areas to collect and direct all surface waters to an appropriate drainage facility as determined by the project civil engineer. The ground surfaces of planter and landscape areas that are located within 10 feet of building foundations should be sloped at a minimum gradient of 5 percent away from the foundations and towards the nearest area drains. The ground surfaces of planter and landscape areas that are located more than 10 feet away from building foundations may be sloped at a minimum gradient of 2 percent away from the foundations and towards the nearest area drains.

Concrete flatwork surfaces that are located within 10 feet of building foundations should be inclined at a minimum gradient of 2 percent away from the building foundations and towards the nearest area drains. Concrete flatwork surfaces that are located more than 10 feet away from building foundations may be sloped at a minimum gradient of 1 percent away from the foundations and towards the nearest area drains. Surface waters should not be allowed to collect or pond against building foundations and within the level areas of the site. All drainage devices should be properly maintained throughout the lifetime of the development. Future changes to site improvements, or planting and watering practices, should not be allowed to cause saturation of site soils adjacent to the structures.

WQBMP infiltration systems may be proposed at the site. The WQBMP concept is to maintain the site's pre-development water runoff rates and volumes by implementing design techniques that detain surface runoff and direct the water into systems that infiltrate to the subsurface. This concept is in direct conflict with the geotechnical drainage recommendations presented above to collect surface runoff and transfer this water to an approved drainage device away from buildings and structures. As regulatory guidelines must be met, the project civil engineer will be required to design the site to meet the appropriate regulatory standards; however the project civil engineer must also be cognizant of the geotechnical concerns with water infiltration adjacent to buildings and structure and design accordingly. A minimum setback of at least 15 feet between infiltration systems and building foundations is suggested to help alleviate potential negative consequences arising from this conflict. If this setback distance can not be maintained, then modified foundation recommendations may be necessary to reduce the potential for distress that could be caused by infiltration of water near the foundation systems.

FOUNDATION DESIGN CONSIDERATIONS

Allowable Bearing Capacity, Estimated Settlement and Lateral Resistance

Allowable Soil Bearing Capacities

Pad Footings

Based on the laboratory test results an allowable soil bearing capacity of 2,000 pounds per square foot, including dead and live loads, may be utilized for design of 24-inch-square pad footings that are a part of the slab system and are placed directly on top of the compacted final grade. This value may be increased by 20 percent for each foot of embedment and by 10 percent for each additional foot of width, to a maximum value of 3,000 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade for pad footings that are not a part of the slab system and are used for support of such features as roof overhang, second-story decks, patio covers, etc. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads, and may be increased by one-third for short duration wind and seismic forces.

Continuous Footings

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

Estimated Footing Settlement

Under the above bearing values, maximum total static settlements of footings is expected to be on the order of ¾ of an inch with a differential settlement of approximately 1/2 of an inch over a span of 40 feet. The majority of this estimated footing settlement will occur during building construction or shortly thereafter as the loads are applied.

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 or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Drilled Pier Foundations

Cast-in-drilled-hole (CIDH) concrete piers may be required for light pole, signs, posts, and utility poles. Shallow piers are usually designed based on rigid body mechanics. Piers or piles generally act as rigid bodies when the length to width ratio is less than 10. Equations for lateral resistance design are given in 2013 CBC Section 1807. A soil strength of 200 psf should be used in these equations for the lateral resistance. The code limits the use of these equations to piers that are less than 12 feet in length. For vertical loads (compression or tension), a skin friction of 100 pounds per square foot per foot of depth may be utilized for the design of the pier foundations. An end bearing of 2,000 pounds per square foot may be utilized, and may be combined with the skin friction.

Guidelines for Footings and Slabs on-Grade Design and Construction

The results of our laboratory tests performed on representative samples of near-surface soils within the site during our investigation indicate that these materials exhibit expansion potentials that are within the Very Low range (Expansion Index from 0 to 20). 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 2013 CBC and may be performed using any method deemed rational and appropriate by the project structural engineer. However, the following minimum recommendations are presented herein for conditions where the project design team may require geotechnical engineering guidelines for design and construction of footings and slabs on-grade the project site.

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

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

Conventional Slabs on-Grade System

Given the very low expansion potential 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 should be founded at a minimum depth of 12 inches below the lowest adjacent final grade. Interior continuous footings may be founded at a minimum depth of 10 inches below the top 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. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across large door or similar openings. The grade beam should be reinforced with a similar manner as provided above.

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

5. Exterior isolated pad footings intended for support of roof overhangs and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings

may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

6. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2013 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

Building Floor Slabs

1. Concrete floor slabs should be a minimum 4 inches thick and reinforced with No. 3 bars spaced a maximum of 24 inches on centers, both ways. Alternatively, the structural engineer may recommend the use of prefabricated welded wire mesh for slab reinforcement. For this condition, the welded wire mesh should be of sheet type (not rolled) and should consist of 6x6/W2.9xW2.9 (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-slab. 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. Where carpet, tile, wood flooring or other moisture sensitive coverings are planned in office areas concrete floor slabs should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

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

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

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

General Corrosivity Screening

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

A variety of test methods are available to quantify corrosive potential of soils for various elements of construction materials. Depending on the test procedures adopted, characteristics of the leachate that is used to extract the target chemicals from the soils and the test equipment; the results can vary appreciably for different test methods in addition to those caused by variability in soil composition. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies. In drawing conclusions from the results of our chemical and electrical laboratory testing and providing mitigation guidelines to reduce the detrimental impact of corrosive site soils on various components of the structure in contact with site soils, heavy references were made to 2013 CBC and American Concrete Institute, 2011 Structural Concrete Building Code (ACI 318-11). Where relevant information was not available in these codes, references were made to guidelines developed by California Department of Transportation (Caltrans), mainly because their risk tolerance for highway bridges are considered comparable to those for residential or commercial structures and that Post Tensioning Institute (PTI), in part, accepts and uses Caltrans' relevant corrosivity criteria for post-tensioned slabs on-grade.

It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer is not informed of these choices. Therefore, for conditions where such elements are considered, 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.

<u>Concrete in Contact with Site Soils</u>

Soils containing soluble sulfates beyond certain threshold levels as well as acidic soils are considered to be detrimental to long-term integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfates (SO_4) concentration in soils determined in general accordance with California Test Method No. 417. The soil soluble sulfate severity rating is adopted from ACI 318 publication. Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in general accordance with California Test Method No. 643. The soil acid severity rating is adopted from The United States Department of Agriculture, Natural Resources Conservation Service classification.

The results of our limited in-house laboratory tests indicate that on-site soils contain a water-soluble sulfate content of 0.0006 percent by weight. Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to sulfates in site soil should be assigned exposure classes in general accordance with the durability requirements of ACI 318.

Based on the test results and in reference to Table 4.2.1 of ACI 318-11, an exposure class of **S0** is appropriate for onsite soils. Accordingly, a severity level of **Not Applicable** for exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As such, Table 4.3.1 of ACI 318-11 provides that no restriction for cement type or maximum water-cement ratio for the fresh concrete would be required. However, this table indicates that the concrete minimum unconfined compressive strength should not be less than 2,500 psi.

Further, the results of limited in-house testing of representative samples indicate that soils within the subject site are moderately alkaline with respect to pH 8.0 (a pH of 7.9 to 8.4). 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.

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

<u>Metals Encased in Concrete</u>

Soils containing a soluble chloride concentration beyond a certain threshold level are considered corrosive to metallic elements such as reinforcement bars, tendons, 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 (Cl) in soils were determined in general accordance with California Test Method No. 422.

Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to chlorides from "*deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where concrete has steel reinforcement*" should be assigned exposure classes in general accordance with the durability requirements of ACI 318. According to Table 4.2.1 of ACI 318-11, an exposure class of **C0** with a severity designation of **Not Applicable** is appropriate for reinforced concrete that remains dry or protected from moisture. Similarly, an exposure class of **C1** with a severity designation of **Moderate** is appropriate for reinforced concrete that is exposed to moisture but not to external sources of chlorides. And, lastly, an exposure class of **C2** with a severity designation of **Severe** is appropriate for reinforced concrete that is exposed to moisture and external sources of chlorides as enumerated above.

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

The results of our limited laboratory tests performed indicate that onsite soils contain a water-soluble chloride concentration of 204 milligrams per liter (mg/L). Article 1904.2 of Section 1904 of the 2013 CBC requires that concrete mixtures conform to the most restrictive maximum water-cementitious material ratios, maximum cementitious admixture, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Article 1904.1. No maximum water/cement ratio for the fresh concrete is prescribed by ACI 318 for class **C1** (or **Moderate** severity) exposure condition. However, Table 4.3.1 of ACI 318-11indicates that concrete minimum unconfined compressive strength, f'_c, should not be less than 2,500 psi. For class **C2** (or **Severe**) exposure condition, Table 4.3.1 of ACI 318-11requires that the maximum water/cement ratio of the fresh concrete should not exceed 0.40 and concrete minimum unconfined compressive strength, f'_c, should not be less than 5,000 psi.

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

Metallic Elements in Contact with Site Soils

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

The minimum electrical resistivity for onsite soils was found to be 12,000 ohm-cm based on limited testing. The result indicates that onsite soils are "mildly corrosive" to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils may need to be protected against detrimental effects of "mildly corrosive" soils based on recommendations provided by a qualified corrosion engineer.

Masonry Block Screen Walls

Where there is room to perform remedial grading, the footings for masonry block screen walls (if any) may be designed in accordance with the bearing and lateral resistance values provided previously for building footings.

However, where remedial grading cannot be performed due to site constraints, a reduced bearing value of 1,200 pounds per square foot should be used for 12-inch-wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. No increase in bearing value may be used for wider or deeper footings for this condition. 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. In addition, a reduced passive earth pressure of 175 pounds per square foot per foot of depth, to a maximum value of 1,750 pounds per square foot, should be used to resist lateral loads. A coefficient of friction of 0.35 times the dead load forces may still be used between concrete and the supporting soils to determine lateral sliding resistance. The above values may be combined without reduction provided the lateral sliding resistance does not exceed one-half the dead load. An increase of one-third of the above values may also be used when designing for short duration wind or seismic forces.

As a minimum, the wall footings should be embedded at a minimum depth of 12 inches below the lowest adjacent final grade. The footings should also be reinforced with a minimum of two No. 4 bars, one top and one bottom. In order to reduce the potential for unsightly cracking related to the possible effects of differential settlement and/or expansion, positive separations (construction joints) should also be provided in the block walls at each corner and at horizontal intervals of approximately 20 to 25 feet. The separations should be provided in the blocks and not extend through the footings. The footings should be poured monolithically with continuous rebars to serve as effective “grade beams” below the walls.

Retaining Wall Design Recommendations

General

The enclosed conceptual plan does not show that retaining walls are proposed and they are not anticipated based on the relatively flat layout of the site. Recommendations for retaining walls may be provided upon request if plans change.

Exterior Concrete Flatwork

General

Near-surface compacted fill soils within the site can be variable in expansion behavior but are expected to exhibit very low to low expansion potential. Therefore, we recommend that all exterior concrete flatwork such as sidewalks, patio slabs, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private 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.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Concrete pavement in driveway and parking lot areas should be at least 5 inches thick and provided with construction joints or expansion joints every 10 feet or less. 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 general 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.*

<u>Edge Beams (Optional)</u>

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.

<u>Subgrade Preparation</u>

<u>Compaction</u>

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.

<u>Pre-Moistening</u>

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

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient as previously discussed, or as prescribed by the 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.

Tentative Pavement Design Recommendations

The final pavement section should be designed once rough grading has occurred and the R-Value of the resulting subgrade can be determined. For tentative development of an approximate pavement section to use for rough grading purposes we would suggest that the pavement section thickness consist of at least 4.0 inches of asphaltic concrete over 6.0 inches of aggregate base. Final pavement design recommendations should be provided based on sampling and testing at the completion of rough grading and the values of traffic indices that should be provided by the project civil engineer.

The project civil engineer should confirm with the City before specifying any pavement section that may be less than the presumed minimum. Subgrade soils should be properly compacted, smooth, and non-yielding prior to pavement construction. The subgrade soils should be compacted to at least 90 percent of ASTM D 1557.

Aggregate base materials should be Crushed Aggregate Base, Crushed Miscellaneous Base, or Processed Miscellaneous Base conforming to Section 200-2 of the Standard Specifications for Public Works Construction (Greenbook). It should be noted that base thickness reported herein is based on the use of Crushed Aggregate base material. For conditions where either Crushed Miscellaneous Base or Processed Miscellaneous Base Materials are used, a 10 percent increase in base section thickness should be incorporated in the design and construction of the structural pavement section. The base materials should be brought to uniform moisture near optimum moisture then compacted to at least 95 percent of the applicable maximum density standard as determined per ASTM D 1557. Asphaltic concrete materials and construction should conform to Section 203 of the Greenbook.

FIELD PERCOLATION TESTING

In accordance with the Client's requested scope of services, our study included performing a surface level soil percolation study in two selected test pits to support the design of a storm water dissipation system that may be proposed within the site. The tests was performed at the location of Test Pits TP-2 and TP-4 in general conformance with County of Riverside Low Impact Development Best Management Practice Guideline. The percolation rates acquired were converted to infiltration rates by means of the Porchet Method. Percolation test results are included in Appendix D and are summarized in the table below.

Percolation Test Results

Test Pit No.	Soil Type (USCS)	Test Pit Depth (feet)	Infiltration Rate* (inches/hour)
TP-2	Sand (SP)	4.0	6.2
TP-4	Sand (SP)	4.0	8.4

* Unadjusted filtration rate without factor-of-safety. An appropriate factor-of-safety should be used by the designer.

Summary of Percolation Test Findings

1. Percolation Test Pit TP-2 encountered 0 to 1 foot of artificial fill overlying native alluvial fan deposits, which extended to the maximum depth explored. Percolation Test Pit TP-4 encountered alluvial fan deposits to the maximum depth explored. The alluvial fan deposits in which percolation testing was performed consisted predominately of gray, coarse-grained gravelly sand.

2. The test procedure utilized yielded information more suitable for relatively shallow infiltration systems such as shallow dry wells. The test data would not be applicable to the use of deep subsurface seepage pits. Evaluation of the feasibility of drywells would require further deep percolation testing.

3. It should be noted that the percolation test was conducted with relatively clean water. Storm runoff and nuisance water, which typically contains sediments and other impurities, may reduce the soil absorption rate.

4. If the test results provided above indicate that a storm water absorption system may be feasible, it would be prudent to consider a back-up area for conditions where the main storm water absorption area has either lost its full absorption capacity, or an overflow of storm water takes place. This may be required from a regulatory standpoint.

FUTURE IMPROVEMENTS AND GRADING

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

REPORT LIMITATIONS

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

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

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

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

Respectfully submitted,

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9/13/16

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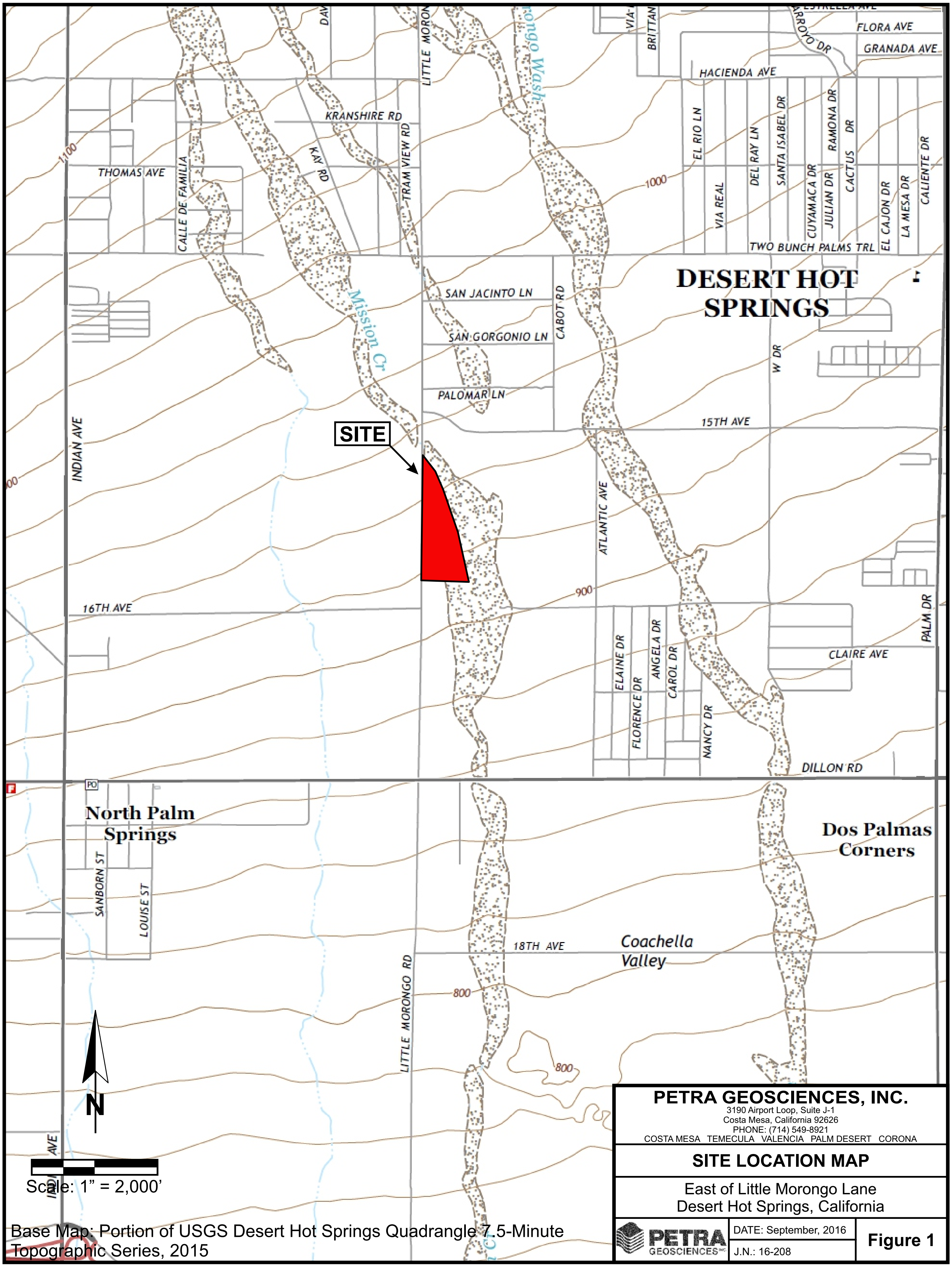
FIGURES



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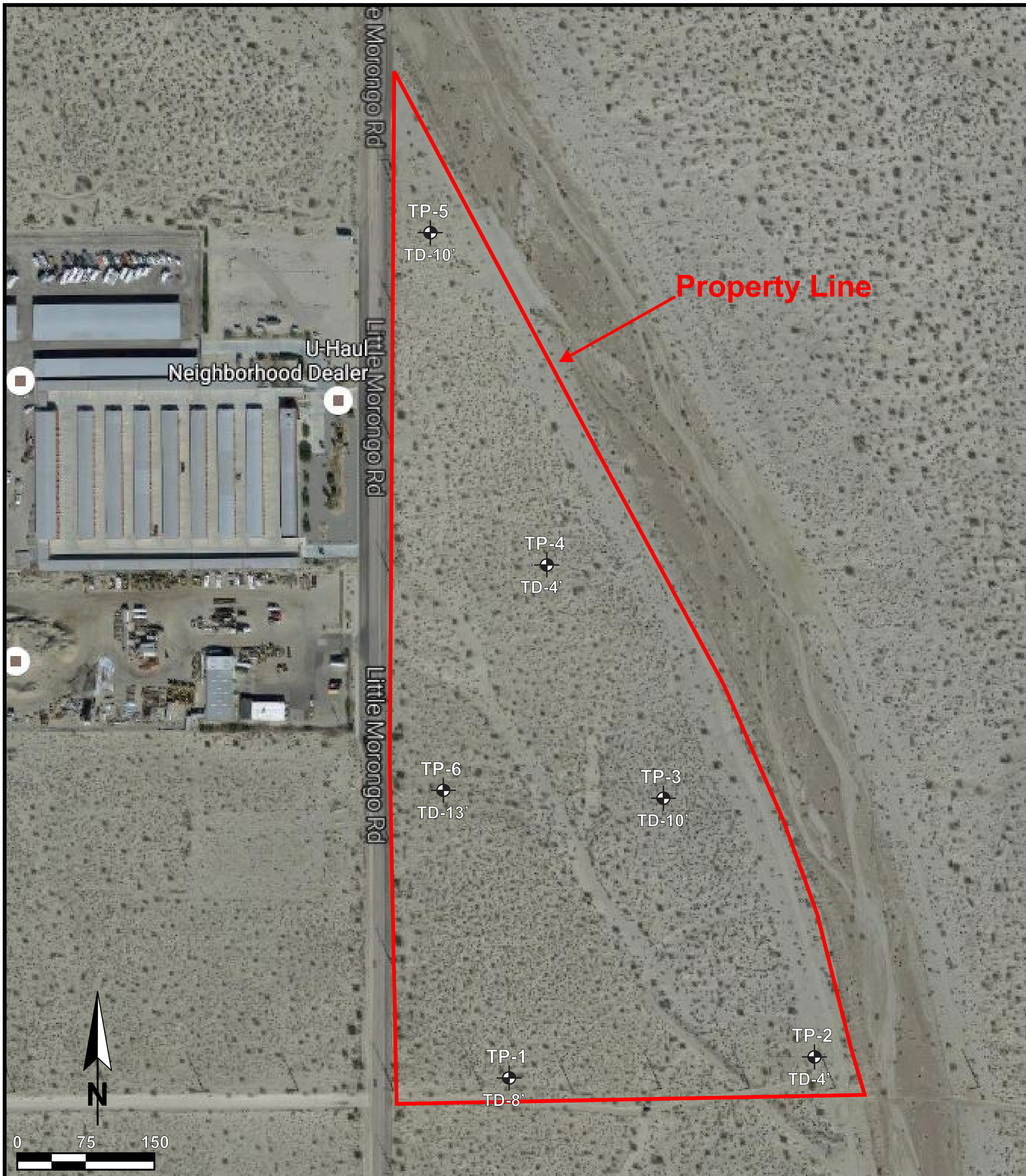
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

SITE LOCATION MAP

East of Little Morongo Lane
Desert Hot Springs, California

PETRA GEOSCIENCES
DATE: September, 2016
J.N.: 16-208

Figure 1



EXPLANATION

TP-6 / TD-13' — Approximate Location of Exploratory Test Pit

TD= Total Depth

Base Map Reference: Google Earth (2015) Map, Dated March, 2015

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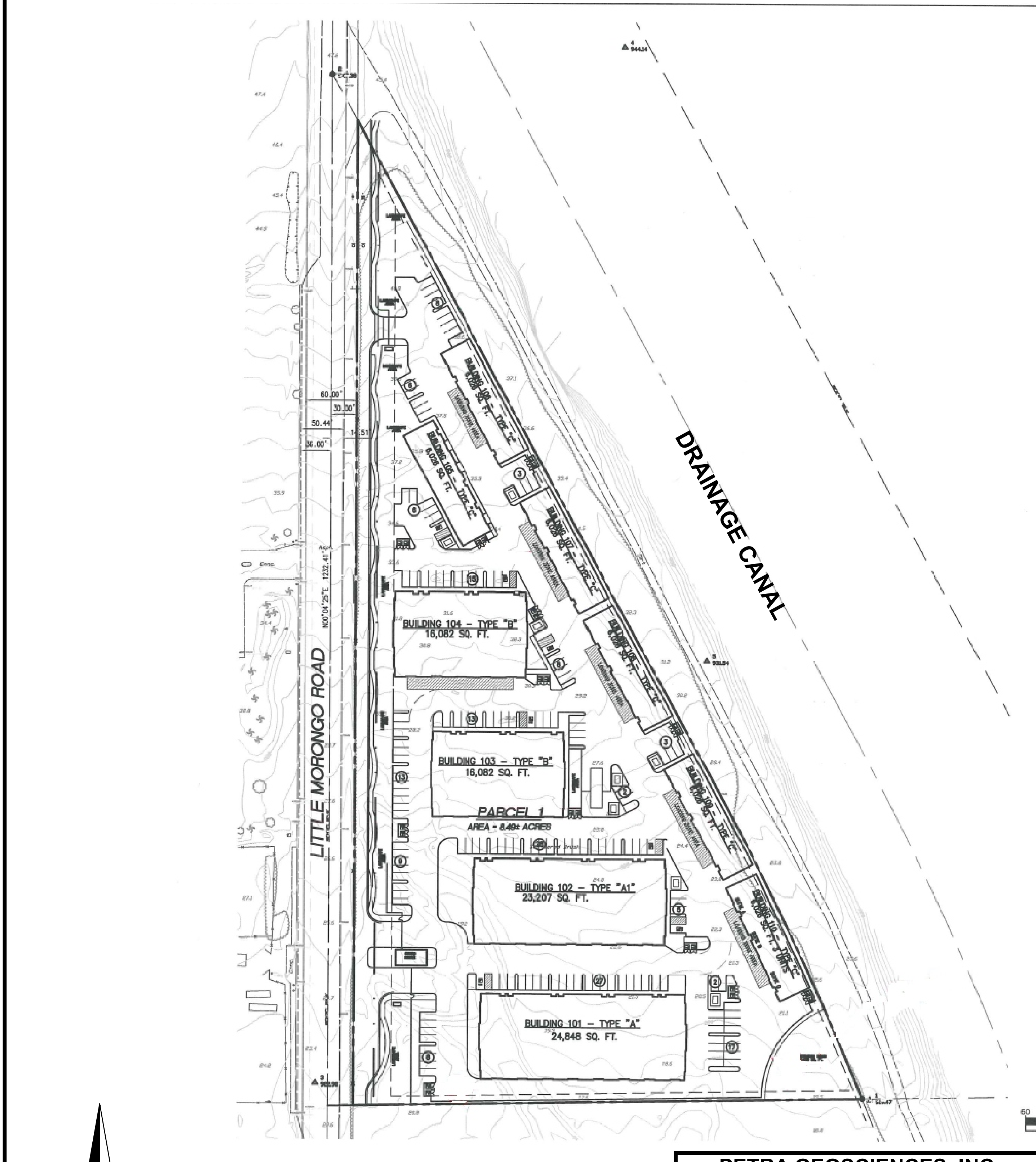
GEOTECHNICAL MAP

East of Little Morongo Lane
Desert Hot Springs, California

PETRA GEOSCIENCES INC

DATE: September, 2016
J.N.: 16-208

Figure 2



Base Map Reference: Egan and Egan, Inc. Tentative Parcel Map, 2016

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PROPOSED SITE PLAN

East of Little Morongo Lane
Desert Hot Springs, California

PETRA GEOSCIENCES INC.

DATE: September, 2016
J.N.: 16-208

Figure 3

APPENDIX A

EXPLORATION LOGS

Key to Soil and Bedrock Symbols and Terms

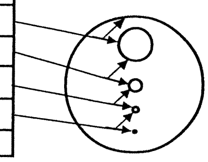
PETRA

Unified Soil Classification System

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

Grain Size

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



Laboratory Test Abbreviations

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

Modifiers

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

Sampler and Symbol Descriptions

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

Bedrock Hardness

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

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

TEST PIT LOG

Project: DHS Facility		Boring No.: TP-1
Location: E. of Little Morongo Rd., Desert Hot Springs		Elevation: 907
Job No.: 16-208	Client: Gabriel Lujan and Assoc.	Date: 8/17/16
Drill Method: Backhoe	Driving Weight: 140 lbs / 30 in	Logged By: KTM

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
		ALLUVIAL FAN DEPOSITS (Qal) Sand (SP): Gray; dry; loose to medium dense; fine- to coarse-grained sand. Same as above except with few (5%) gravel up to 1" in diameter and medium dense.		6			1.7	121.6	MAX, EI, HYD, SO4, PH, CL, RES, DSR
5		Same as above except with few (5%) cobbles up to 4" in diameter.							
		Same as above except with some (10%) boulders up to 1.5' in diameter.							
		Total Depth- 8' No groundwater encountered Boring backfilled with cuttings.							
10									
15									
20									

TEST PIT LOG 16-208.GPJ PETRA.GDT 9/13/16

PLATE A-1

TEST PIT LOG

Project: **DHS Facility**		Boring No.:	**TP-2**
Location: **E. of Little Morongo Rd., Desert Hot Springs**		Elevation:	**906**
Job No.: **16-208**	Client: **Gabriel Lujan and Assoc.**	Date:	**8/17/16**
Drill Method: **Backhoe**	Driving Weight: **140 lbs / 30 in**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		<u>ARTIFICIAL FILL (af)</u> <u>Sand (SP)</u>: Gray; dry; loose to medium dense; fine- to medium-grained sand.							
		<u>ALLUVIAL FAN DEPOSITS (Qal)</u> <u>Sand (SP)</u>: Gray; dry; medium dense; coarse-grained sand.							
		Same as above except slightly moist.		7			2.1	111.1	
5		Total Depth- 4' No groundwater encountered Open pit percolation test performed in test pit.							
10									
15									
20									

PLATE A-2

TEST PIT LOG

Project:	DHS Facility	Boring No.:	TP-3
Location:	E. of Little Morongo Rd., Desert Hot Springs	Elevation:	916
Job No.: 16-208	Client: Gabriel Lujan and Assoc.	Date:	8/17/16
Drill Method: Backhoe	Driving Weight: 140 lbs / 30 in	Logged By:	KTM

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		**ALLUVIAL FAN DEPOSITS (Qal)** Sand (SP): Gray; dry; loose to medium dense; fine- to coarse-grained sand.							
2		Same as above except coarse-grained.							
3		Same as above except grayish-brown and dry to slightly moist.		6	■		2.0	94.6	
5						▒			
6		Same as above except with numerous (15%) cobbles up to 4" in diameter and with few (5%) boulders up to 1.5' in diameter.							
8		Same as above.							
10		Total Depth - 10' No groundwater encountered Boring backfilled with cuttings.							

Depth scale: 5, 10, 15, 20

PLATE A-3

TEST PIT LOG

Project: DHS Facility		Boring No.: TP-4	
Location: E. of Little Morongo Rd., Desert Hot Springs		Elevation: 929	
Job No.: 16-208	Client: Gabriel Lujan and Assoc.	Date: 8/17/16	
Drill Method: Backhoe	Driving Weight: 140 lbs / 30 in	Logged By: KTM	

Depth (Feet)	Lith- ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ALLUVIAL FAN DEPOSITS (Qal) Sand (SP): Gray; dry; loose; medium- to coarse-grained sand.							
		Same as above except with few (5%) boulders up to 1.5' in diameter.							
5		Total Depth- 4' No groundwater encountered Open pit percolation test performed at bottom of test pit.							
10									
15									
20									

TEST PIT LOG 16-208.GPJ PETRA.GDT 9/13/16

PLATE A-4

TEST PIT LOG

Project: **DHS Facility**		Boring No.: **TP-5**	
Location: **E. of Little Morongo Rd., Desert Hot Springs**		Elevation: **938**	
Job No.: **16-208**	Client: **Gabriel Lujan and Assoc.**	Date: **8/17/16**	
Drill Method: **Backhoe**	Driving Weight: **140 lbs / 30 in**	Logged By: **KTM**	

Depth (Feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ALLUVIAL FAN DEPOSITS (Qal)							
		Sand (SP): Gray; dry; loose; fine- to medium-grained sand.							
		Same as above except medium- to coarse-grained.							
		Silty Sand (SM): Grayish-brown; dry; loose to medium dense; fine- to medium-grained sand.							
		Sand (SP): Gray; dry; medium dense; medium- to coarse-grained sand.		7					
		Same as above except with few (5%) boulders up to 1.5' in diameter.							
5		Same as above except with numerous (15%) gravel up to 3" in diameter.							
		Same as above except with numerous (15%) cobbles and boulders from 0.5' to 1.5' in diameter.							
		Same as above.							
10		Total Depth - 10' No groundwater encountered Boring backfilled with cuttings.							
15									
20									

TEST PIT LOG 16-208.GPJ PETRA.GDT 9/13/16

PLATE A-5

Petra Geosciences, Inc.

TEST PIT LOG

Project: DHS Facility		Boring No.: TP-6
Location: E. of Little Morongo Rd., Desert Hot Springs		Elevation: 920
Job No.: 16-208	Client: Gabriel Lujan and Assoc.	Date: 8/17/16
Drill Method: Backhoe	Driving Weight: 140 lbs / 30 in	Logged By: KTM

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	SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand (SM): Gray; dry; loose to medium dense; fine- to medium-grained sand.							
2	SP	Sand (SP): Gray; dry; medium dense; medium- to coarse-grained sand.							
3		Same as above except with one boulder up to 2' in diameter.		5	■		1.1	118.5	
4		Same as above except with few (5%) boulders up to 1.5' in diameter.							
5						▒			
8		Same as above except with some (8%) gravel up to 3" in diameter.							
10		Same as above.				▒			
13		Total Depth- 13' No groundwater encountered Boring backfilled with cuttings.							
15									
20									

TEST PIT LOG 16-208.GPJ PETRA.GDT 9/13/16

PLATE A-6

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

LABORATORY TEST PROCEDURES

Soil Classification

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

Moisture Content and In Situ Moisture Content and Dry Unit Weight

Moisture content of selected bulk samples and in-place moisture content and dry unit weight of selected, relatively undisturbed soil samples were determined in general accordance with the current version of the Test Method ASTM D 2435 and Test Method ASTM D 2216, respectively. Test data are presented in the exploration logs, Appendix A.

Laboratory Maximum Dry Unit Weight and Optimum Moisture Content

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

Expansion Index

An expansion index test was performed on a selected bulk sample of the on-site soils in general accordance with the current version of Test Method ASTM D 4829. The test results are presented on Plate B-1.

Corrosivity Screening

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

Grain Size Distribution

Grain-size analysis (including hydrometer and percent passing the No. 200 sieve) was performed on a sample of selected soils. The test was performed in general accordance with Test Method No. ASTM D 1140. The test result is presented on Plate B-3.

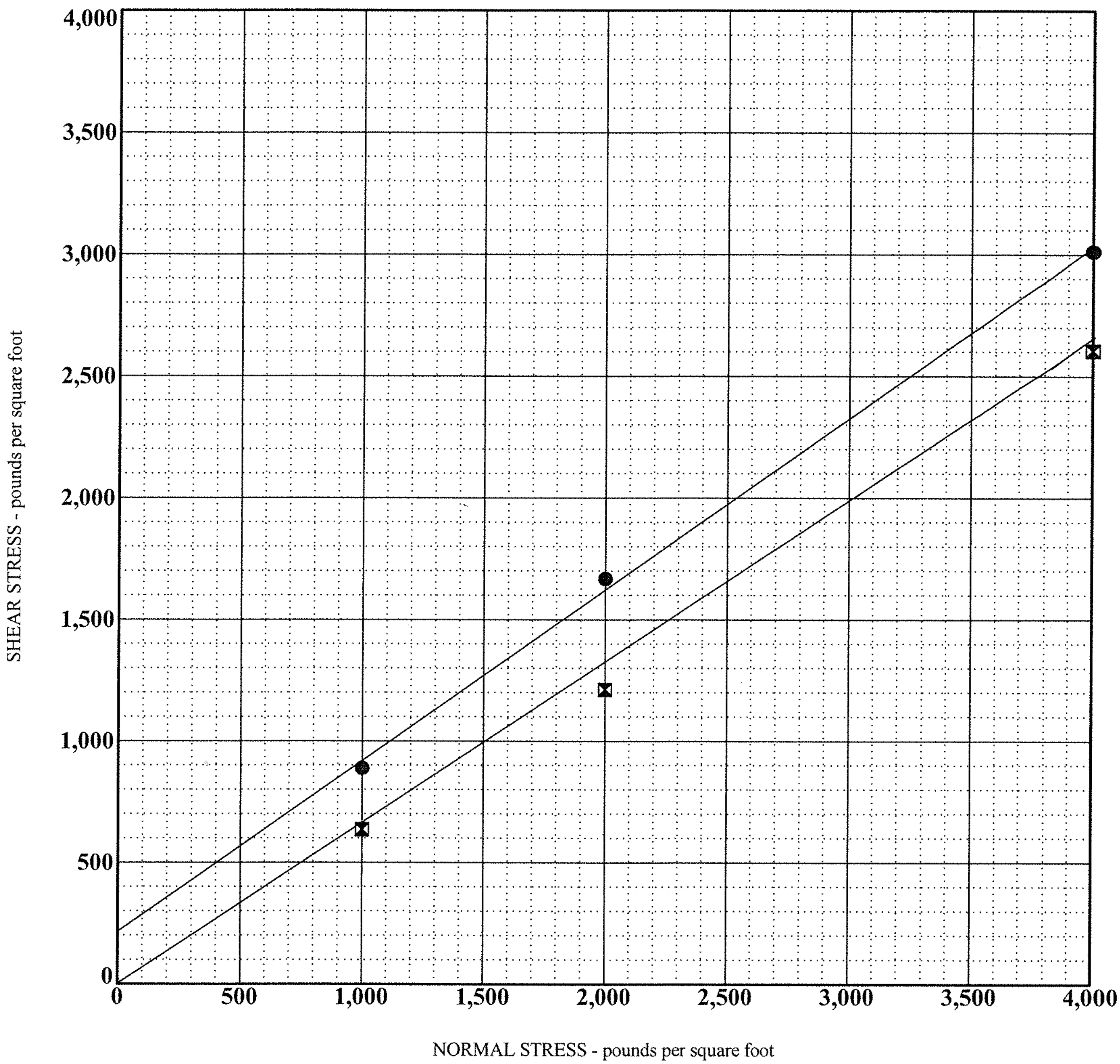
Direct Shear Strength

The Coulomb shear strength parameters (angle of internal friction and cohesion) were determined for remolded samples of on-site soils. The test was performed in general accordance with Test Method No. ASTM D 3080. The test specimens were saturated, and then sheared under varying normal loads at a maximum constant strain rate of 0.01 inches per minute. Results are graphically presented on Plate B-2.

LABORATORY DATA SUMMARY*													
Boring Number	**Sample Depth (ft)**	**Soil Description**	**Max. Dry Density [1] (pcf)**	**Optimum Moisture[1] (%)**	**Expansion Index[2]**	**CBC Soil Classification[3]**	**Atterberg Limits[4]**			**Sulfate Content[5] (%)**	**Chloride Content[6] (mg/L)**	**pH[7]**	**Minimum Resistivity[7] (Ohm-cm)**
							LL	**PL**	**PI**				
TP-1	0-5	Sand to Silty Sand (SP-SM)	123.5	11.5	2	Very Low	-	-	-	0.0006	204	8.0	12,000

*Note: Laboratory data pertaining to in-place soil moisture content and dry density are provided on the exploration logs included in Appendix A of this report.
**Note: NP = Non Plastic

Test Procedures:
[1] ASTM Test Method D 1557
[2] ASTM Test Method D 4829
[3] ASTM Test Method D 4829 Table 1, Per CBC 2010
[4] ASTM Test Method D 4318
[5] Caltrans Test Method 417
[6] Caltrans Test Method 422
[7] Caltrans Test Method 643



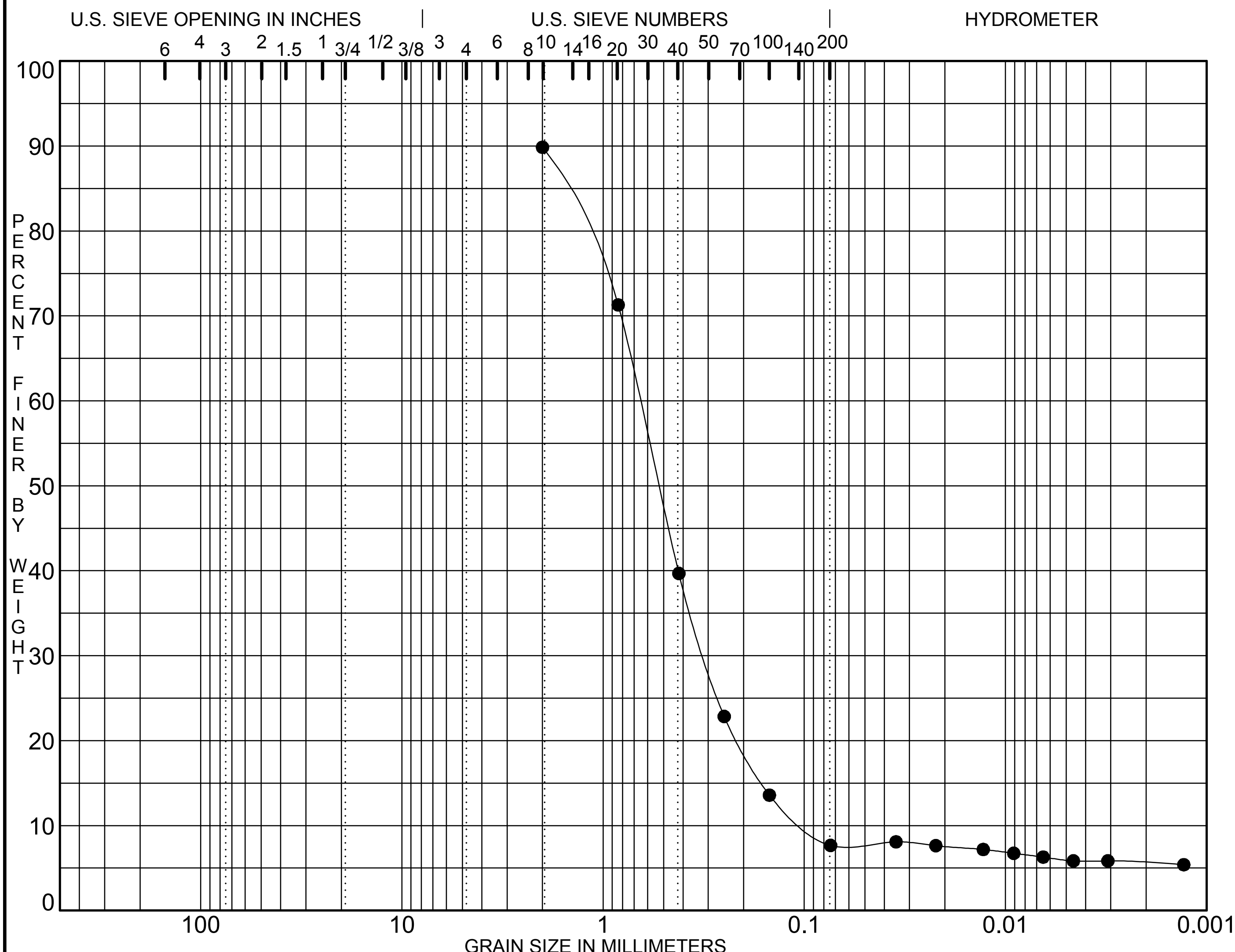
SAMPLE LOCATION	DESCRIPTION	FRICTION ANGLE (°)	COHESION (PSF)
● TP-1 @ 0.0	Poorly Graded Fine to Coarse-Grained Sand	35	216
⊠ TP-1 @ 0.0	Poorly Graded Fine to Coarse-Grained Sand @.25" DISPLACEMENT	33	0

NOTES:

Samples Remolded to 90% of Maximum Dry Density
All Samples Were Inundated Prior to Shearing

J.N. 16-208	**DIRECT SHEAR TEST DATA**	**September, 2016**
PETRA GEOSCIENCES, INC.		**PLATE B-2**

DIRECT SHEAR 16-208.GPJ PETRA.GDT 9/9/16



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
● TP-1	0.0	Poorly Graded Fine to Coarse-Grained Sand					2.55	11.3

Specimen Identification		D100	D60	D30	D50	%Gravel	%Sand	%Silt	%Clay
● TP-1	0.0	2.00	0.66	0.312	0.5269	0.0			5.9

GRAIN SIZE - V3 16-208.GPJ PETRA.GDT 9/13/16

J.N. **16-208**

PETRA GEOSCIENCES, INC.

GRAIN SIZE ANALYSIS

September, 2016

PLATE B-3

APPENDIX C

SEISMIC DESIGN PARAMETERS

Design Maps Detailed Report

ASCE 7-10 Standard (33.93496°N, 116.52652°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 2.812 g
From Figure 22-2 [2]	S_1 = 1.044 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.812 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 1.044 g, F_v = 1.500

Equation (11.4–1):	$S_{MS} = F_aS_S = 1.000 \times 2.812 = 2.812$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 1.500 \times 1.044 = 1.566$ g

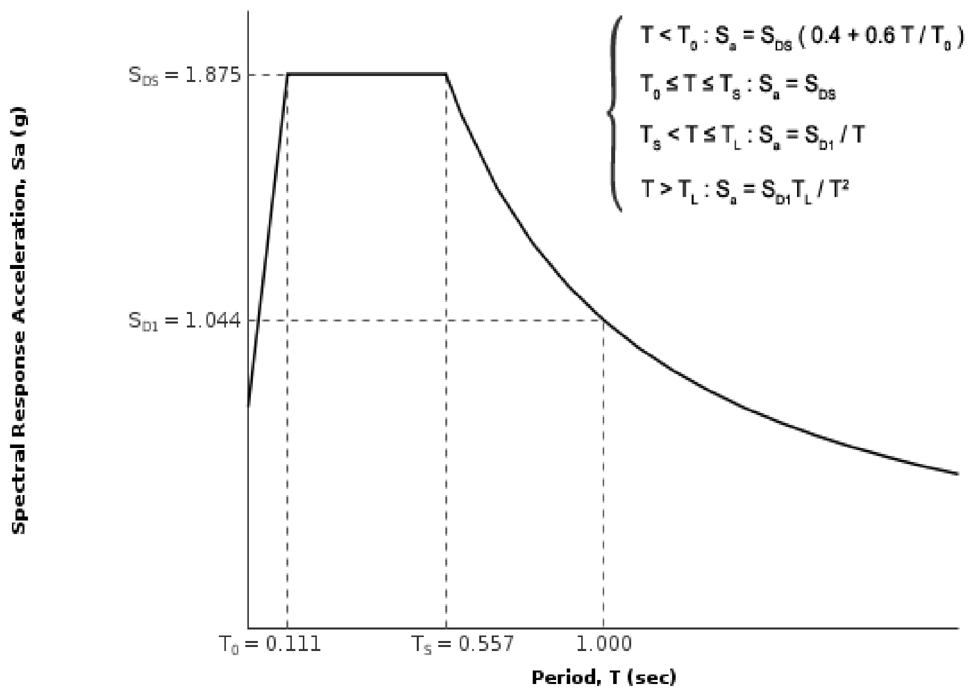
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 2.812 = 1.875$ g
Equation (11.4–4):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 1.566 = 1.044$ g

Section 11.4.5 — Design Response Spectrum

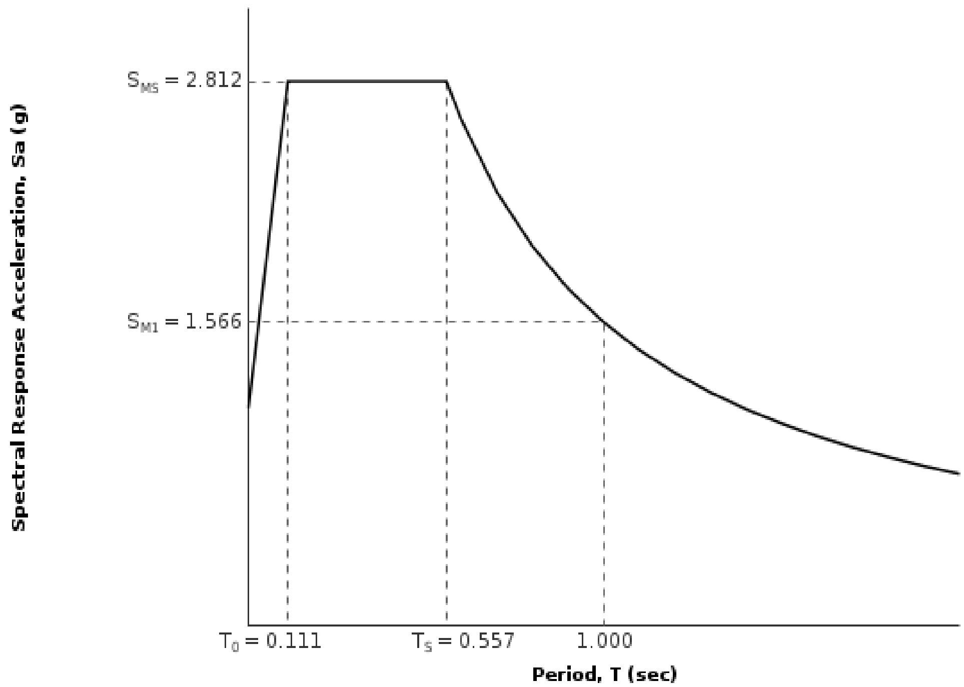
From Figure 22-12 [3]	T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 1.063
Equation (11.8–1):	$PGA_M = F_{PGA}PGA$ = 1.000 x 1.063 = 1.063 g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 1.063 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.951
From Figure 22-18 [6]	C_{R1} = 0.917

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.875 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 1.044 g, Seismic Design Category = D

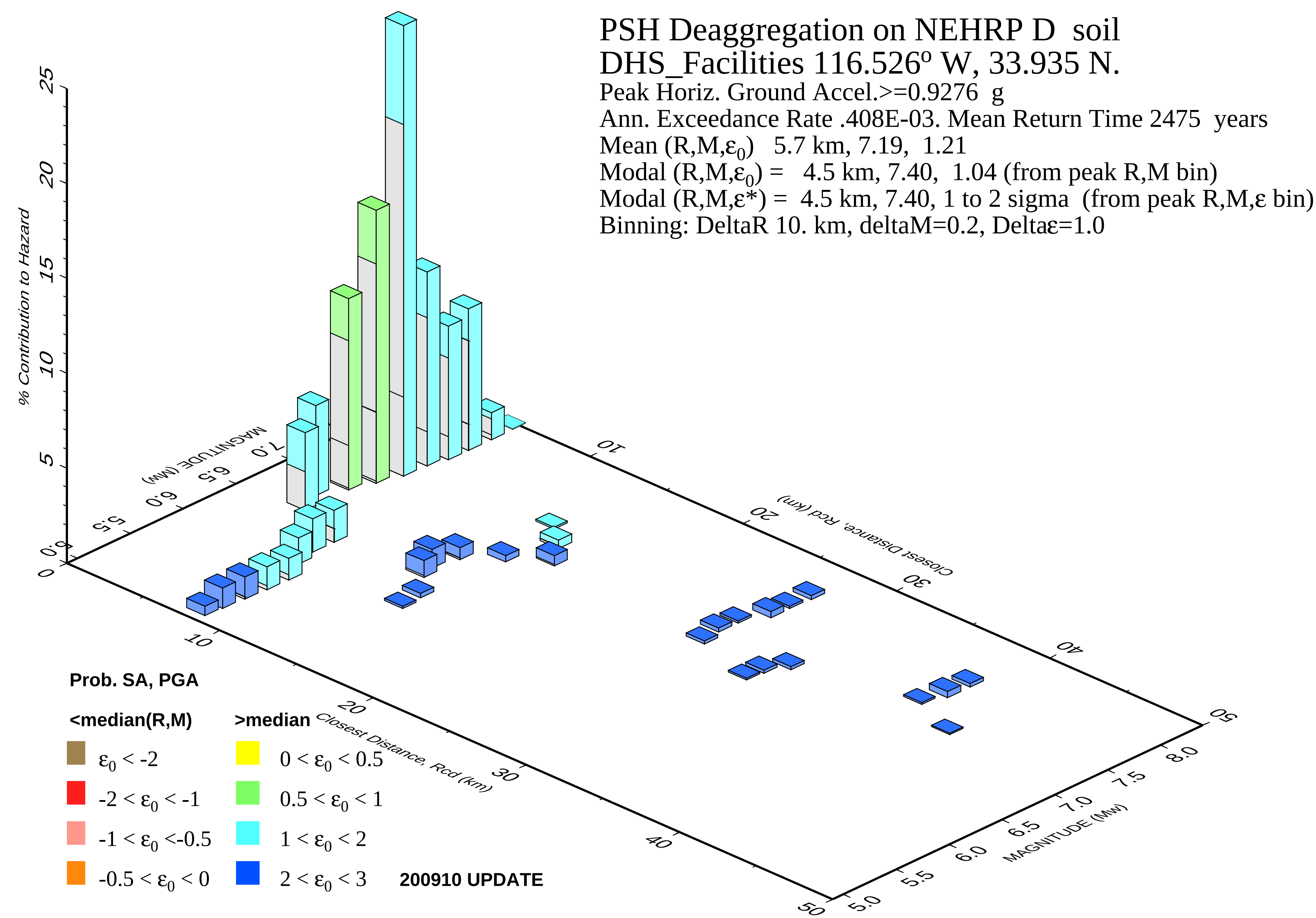
Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

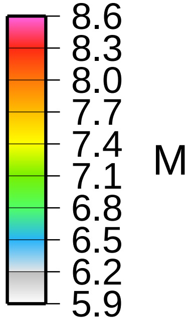
1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

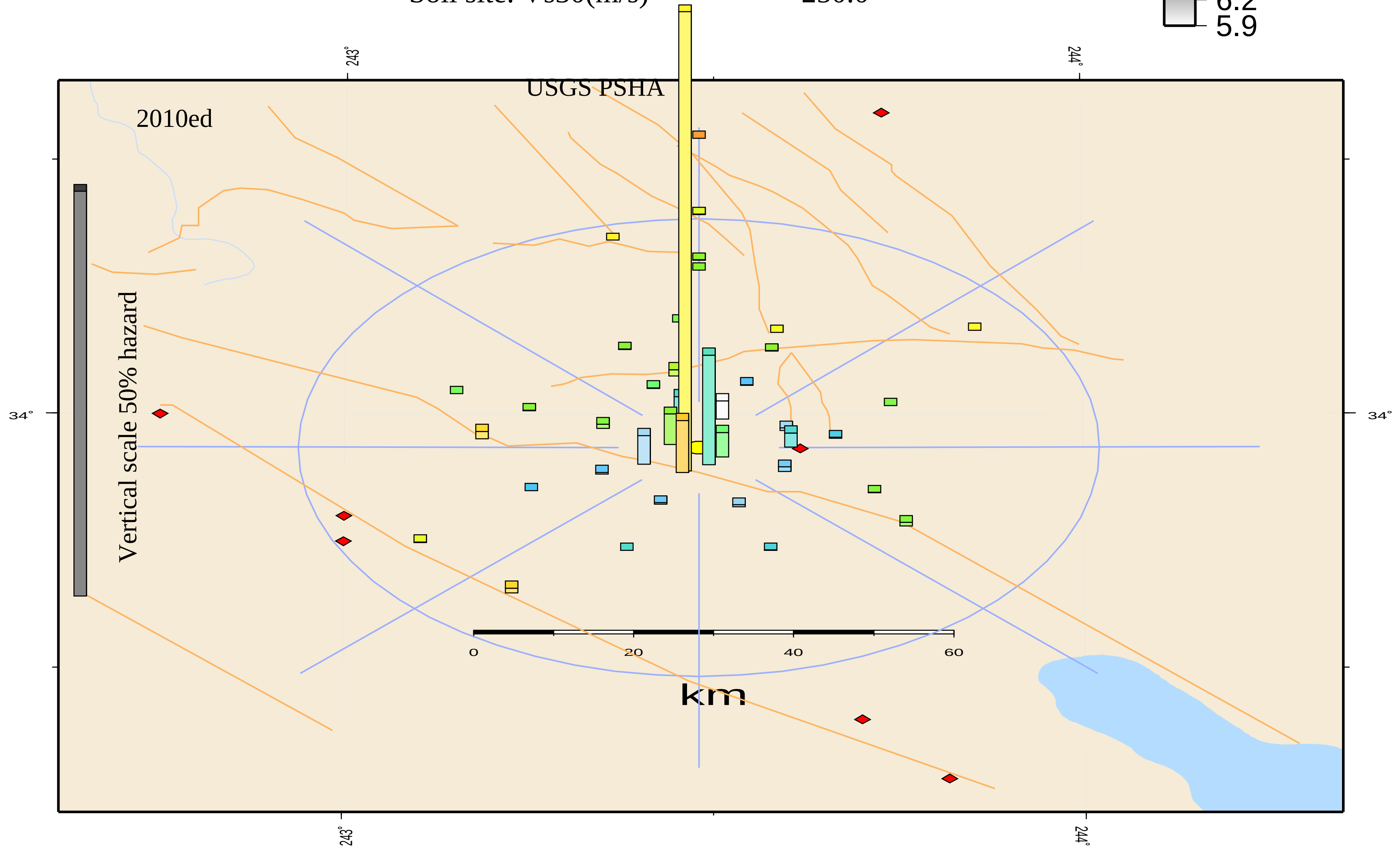


GMT 2016 Sep 13 19:12:44 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 250. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

DHS_Facilities Geographic Deagg. Seismic Hazard
for 0.00-s Spectral Accel, 0.9275 g

PGA Exceedance Return Time: 2475 year
Max. significant source distance 68. km.
View angle is 35 degrees above horizon
Gridded-source hazard accum. in 45° intervals
Soil site. Vs30(m/s) = 250.0





GMT 2016 Sep 13 19:12:44 **Site Coords:-116.526 33.9350 (yellow disk) Vs30= 250.0. Max annual ExcdRate .2315E-03 (column height prop. to ExRate). Red diamonds: historical earthquakes, M>6**

*** 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: DHS_Facilities long: 116.526 W., lat: 33.935 N.
Vs30(m/s)= 250.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.9276   g. Weight * Computed_Rate_Ex 0.408E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00035
#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
7.8	5.05	0.480	0.480	0.000	0.000	0.000	0.000	0.000
7.9	5.20	1.070	1.048	0.022	0.000	0.000	0.000	0.000
8.0	5.40	1.176	1.049	0.126	0.000	0.000	0.000	0.000
8.1	5.60	1.196	1.003	0.193	0.000	0.000	0.000	0.000
8.1	5.80	1.149	0.871	0.278	0.000	0.000	0.000	0.000
7.3	6.01	1.392	1.012	0.380	0.000	0.000	0.000	0.000
14.0	6.03	0.109	0.109	0.000	0.000	0.000	0.000	0.000
6.9	6.20	1.749	1.216	0.533	0.000	0.000	0.000	0.000
13.8	6.22	0.233	0.229	0.003	0.000	0.000	0.000	0.000
6.9	6.40	1.695	0.979	0.716	0.000	0.000	0.000	0.000
12.5	6.44	0.880	0.791	0.089	0.000	0.000	0.000	0.000
3.7	6.60	4.088	2.064	2.023	0.000	0.000	0.000	0.000
12.1	6.58	0.975	0.838	0.137	0.000	0.000	0.000	0.000
3.0	6.80	4.758	1.733	2.844	0.181	0.000	0.000	0.000
12.4	6.80	0.629	0.558	0.071	0.000	0.000	0.000	0.000
30.8	6.85	0.103	0.103	0.000	0.000	0.000	0.000	0.000
3.7	7.01	10.078	2.227	5.516	2.239	0.096	0.000	0.000
14.1	6.98	0.339	0.291	0.048	0.000	0.000	0.000	0.000
26.7	7.03	0.152	0.152	0.000	0.000	0.000	0.000	0.000
30.9	7.00	0.153	0.153	0.000	0.000	0.000	0.000	0.000
4.1	7.21	14.380	2.835	7.795	3.637	0.113	0.000	0.000
16.1	7.16	0.539	0.459	0.080	0.000	0.000	0.000	0.000
26.4	7.22	0.207	0.207	0.000	0.000	0.000	0.000	0.000
31.5	7.16	0.168	0.168	0.000	0.000	0.000	0.000	0.000
41.5	7.21	0.062	0.062	0.000	0.000	0.000	0.000	0.000
4.5	7.40	23.742	5.212	14.360	4.163	0.006	0.000	0.000
15.0	7.36	0.443	0.339	0.104	0.000	0.000	0.000	0.000
26.4	7.41	0.112	0.112	0.000	0.000	0.000	0.000	0.000
38.4	7.39	0.096	0.096	0.000	0.000	0.000	0.000	0.000
4.6	7.62	10.225	2.411	6.019	1.796	0.000	0.000	0.000
13.4	7.54	0.095	0.067	0.028	0.000	0.000	0.000	0.000
27.1	7.61	0.333	0.318	0.015	0.000	0.000	0.000	0.000
38.8	7.59	0.319	0.319	0.000	0.000	0.000	0.000	0.000
4.9	7.78	7.036	1.677	4.146	1.213	0.000	0.000	0.000
27.1	7.79	0.097	0.084	0.013	0.000	0.000	0.000	0.000
38.8	7.80	0.194	0.194	0.000	0.000	0.000	0.000	0.000
4.9	7.97	7.473	1.697	4.399	1.378	0.000	0.000	0.000
27.2	7.98	0.212	0.171	0.041	0.000	0.000	0.000	0.000
4.9	8.19	1.422	0.321	0.843	0.258	0.000	0.000	0.000
4.9	8.39	0.054	0.011	0.032	0.011	0.000	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=   5.7 km; M= 7.19; eps0=   1.21. Mean calculated for all sources.
Modal src-site R=   4.5 km; M= 7.40; eps0=   1.04 from peak (R,M) bin
MODE R*=   4.5km; M*= 7.40; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 14.360

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

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California A-faults	65.25	5.6	7.49	1.03
CA Compr. crustal gridded	11.11	8.0	5.89	1.88
San Gorgonio Zone gridded	20.49	3.4	7.02	1.29

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

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
S. S.Andr.;BG aPriori	7.70	4.1	7.06	0.68	-158.4

```
S. S.Andr.;SSB+BG aPriori          10.09     4.6   7.31    0.90     -158.4
S. S.Andr.;BG+CO aPriori            6.41     4.7   7.36    0.99     -158.4
S. S.Andr.;NSB+SSB+BG aPriori       3.48     4.7   7.45    1.02     -158.4
S. S.Andr.;SSB+BG+CO aPriori        3.04     4.8   7.51    1.08     -158.4
S. S.Andr.;SM+NSB+SSB+BG aPriori    2.13     4.8   7.72    1.11     -158.4
SSAndr.;NSB+SSB+BG+CO aPriori       2.89     4.8   7.61    1.11     -158.4
SSAnd;SM+NSB+SSB+BG+CO aPriori      2.85     4.9   7.82    1.11     -158.4
S. San Andreas;BG+CO MoBal          4.00     4.7   7.35    0.99     -158.4
S. San Andreas;SM+NSB+SSB+BG+CO     2.28     4.9   7.82    1.11     -158.4
S. San Andreas Unsegmented A-flt    6.43     5.1   7.71    1.19     -156.2
#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs  *********#

PSHA Deaggregation. %contributions. site: DHS_Facilities long: 116.526 W., lat: 33.935 N.
 Vs30(m/s)= 250.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08  See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475  yrs. Exceedance PGA =0.9276   g. Weight * Computed_Rate_Ex 0.172E-03
#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
    7.2    5.05    0.022    0.022    0.000    0.000    0.000    0.000    0.000
    7.5    5.21    0.067    0.067    0.000    0.000    0.000    0.000    0.000
    7.7    5.40    0.105    0.105    0.000    0.000    0.000    0.000    0.000
    7.8    5.60    0.137    0.137    0.000    0.000    0.000    0.000    0.000
    8.0    5.80    0.168    0.168    0.000    0.000    0.000    0.000    0.000
    7.1    6.02    0.268    0.268    0.000    0.000    0.000    0.000    0.000
    6.8    6.20    0.377    0.375    0.002    0.000    0.000    0.000    0.000
   13.9    6.23    0.071    0.070    0.002    0.000    0.000    0.000    0.000
    6.8    6.40    0.375    0.364    0.011    0.000    0.000    0.000    0.000
   12.5    6.45    0.412    0.362    0.050    0.000    0.000    0.000    0.000
    3.7    6.60    1.400    1.248    0.152    0.000    0.000    0.000    0.000
   12.4    6.58    0.563    0.485    0.077    0.000    0.000    0.000    0.000
    2.9    6.80    1.824    1.246    0.563    0.015    0.000    0.000    0.000
   12.9    6.80    0.394    0.354    0.041    0.000    0.000    0.000    0.000
   25.2    6.79    0.040    0.040    0.000    0.000    0.000    0.000    0.000
   30.8    6.85    0.103    0.103    0.000    0.000    0.000    0.000    0.000
    3.6    7.00    3.457    1.375    1.696    0.386    0.000    0.000    0.000
   14.8    6.98    0.233    0.206    0.027    0.000    0.000    0.000    0.000
   26.8    7.03    0.149    0.149    0.000    0.000    0.000    0.000    0.000
   30.9    7.00    0.153    0.153    0.000    0.000    0.000    0.000    0.000
    4.2    7.20    5.941    1.642    3.297    1.002    0.000    0.000    0.000
   16.3    7.16    0.402    0.342    0.060    0.000    0.000    0.000    0.000
   26.5    7.22    0.191    0.191    0.000    0.000    0.000    0.000    0.000
   31.5    7.16    0.168    0.168    0.000    0.000    0.000    0.000    0.000
   41.5    7.21    0.062    0.062    0.000    0.000    0.000    0.000    0.000
    4.6    7.40   11.420    2.768    7.046    1.606    0.000    0.000    0.000
   15.4    7.36    0.284    0.234    0.050    0.000    0.000    0.000    0.000
   26.5    7.41    0.101    0.101    0.000    0.000    0.000    0.000    0.000
   38.4    7.39    0.096    0.096    0.000    0.000    0.000    0.000    0.000
   41.7    7.40    0.040    0.040    0.000    0.000    0.000    0.000    0.000
    4.6    7.62    4.732    1.309    2.867    0.556    0.000    0.000    0.000
   14.0    7.54    0.053    0.045    0.008    0.000    0.000    0.000    0.000
   27.1    7.61    0.288    0.272    0.015    0.000    0.000    0.000    0.000
   38.8    7.59    0.318    0.318    0.000    0.000    0.000    0.000    0.000
   42.9    7.53    0.022    0.022    0.000    0.000    0.000    0.000    0.000
    4.9    7.78    3.182    0.814    1.988    0.379    0.000    0.000    0.000
   27.1    7.79    0.082    0.069    0.013    0.000    0.000    0.000    0.000
   38.8    7.80    0.188    0.188    0.000    0.000    0.000    0.000    0.000
    4.9    7.97    3.285    0.822    2.061    0.402    0.000    0.000    0.000
   27.2    7.98    0.170    0.129    0.041    0.000    0.000    0.000    0.000
    4.9    8.18    0.674    0.177    0.424    0.073    0.000    0.000    0.000
    4.9    8.39    0.023    0.005    0.015    0.003    0.000    0.000    0.000

Summary statistics for above PSHA PGA  deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%):   42.2
 Mean src-site R=    6.6 km; M= 7.31; eps0=   1.29. Mean calculated for all sources.
```

Modal src-site R= 4.6 km; M= 7.40; eps0= 1.07 from peak (R,M) bin
MODE R*= 4.6km; M*= 7.40; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 7.046

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

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California A-faults	30.54	6.6	7.49	1.17
San Gorgonio Zone gridded	7.76	3.6	7.02	1.34

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

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
S. S.Andr.;BG aPriori	2.57	4.1	7.05	0.98	-158.4
S. S.Andr.;SSB+BG aPriori	4.83	4.6	7.31	0.92	-158.4
S. S.Andr.;BG+CO aPriori	3.08	4.7	7.35	1.03	-158.4
S. S.Andr.;NSB+SSB+BG aPriori	1.77	4.7	7.45	1.03	-158.4
S. S.Andr.;SSB+BG+CO aPriori	1.47	4.8	7.51	1.13	-158.4
S. S.Andr.;SM+NSB+SSB+BG aPriori	1.01	4.8	7.72	1.17	-158.4
SSAndr.;NSB+SSB+BG+CO aPriori	1.37	4.8	7.61	1.16	-158.4
SSAnd;SM+NSB+SSB+BG+CO aPriori	1.31	4.9	7.82	1.19	-158.4
S. San Andreas;BG+CO MoBal	1.93	4.7	7.35	1.03	-158.4
S. San Andreas;SM+NSB+SSB+BG+CO	1.05	4.9	7.82	1.19	-158.4
S. San Andreas Unsegmented A-flt	2.92	5.2	7.70	1.30	-156.2

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

PSHA Deaggregation. %contributions. site: DHS_Facilities long: 116.526 W., lat: 33.935 N.
Vs30(m/s)= 250.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.9276 g. Weight * Computed_Rate_Ex 0.220E-04
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000
#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
7.3	5.05	0.023	0.023	0.000	0.000	0.000	0.000	0.000
7.7	5.21	0.080	0.080	0.000	0.000	0.000	0.000	0.000
7.9	5.41	0.142	0.142	0.000	0.000	0.000	0.000	0.000
8.0	5.60	0.172	0.172	0.000	0.000	0.000	0.000	0.000
8.0	5.80	0.163	0.163	0.000	0.000	0.000	0.000	0.000
7.3	6.01	0.175	0.175	0.000	0.000	0.000	0.000	0.000
12.9	6.04	0.004	0.004	0.000	0.000	0.000	0.000	0.000
6.8	6.20	0.230	0.230	0.000	0.000	0.000	0.000	0.000
13.6	6.21	0.010	0.010	0.000	0.000	0.000	0.000	0.000
6.8	6.40	0.268	0.267	0.001	0.000	0.000	0.000	0.000
13.1	6.40	0.025	0.025	0.000	0.000	0.000	0.000	0.000
3.7	6.60	0.660	0.651	0.010	0.000	0.000	0.000	0.000
11.9	6.60	0.014	0.014	0.000	0.000	0.000	0.000	0.000
2.9	6.80	0.589	0.567	0.022	0.000	0.000	0.000	0.000
11.4	6.81	0.024	0.024	0.000	0.000	0.000	0.000	0.000
3.3	7.01	0.742	0.596	0.146	0.000	0.000	0.000	0.000
13.0	6.99	0.009	0.009	0.000	0.000	0.000	0.000	0.000
3.6	7.21	0.887	0.639	0.249	0.000	0.000	0.000	0.000
13.1	7.19	0.009	0.009	0.000	0.000	0.000	0.000	0.000
3.9	7.41	0.748	0.595	0.152	0.000	0.000	0.000	0.000
12.1	7.40	0.015	0.015	0.000	0.000	0.000	0.000	0.000
3.7	7.58	0.304	0.249	0.055	0.000	0.000	0.000	0.000
11.4	7.55	0.005	0.005	0.000	0.000	0.000	0.000	0.000
4.8	7.80	0.036	0.036	0.000	0.000	0.000	0.000	0.000
4.9	7.97	0.044	0.044	0.000	0.000	0.000	0.000	0.000
4.9	8.22	0.005	0.005	0.000	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 5.4
Mean src-site R= 4.6 km; M= 6.81; eps0= 1.84. Mean calculated for all sources.
Modal src-site R= 3.6 km; M= 7.21; eps0= 1.62 from peak (R,M) bin
MODE R*= 3.7km; M*= 6.60; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 0.651

Principal sources (faults, subduction, random seismicity having > 3% contribution)
Source Category: % contr. R(km) M epsilon0 (mean values).
Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
S. S.Andr.;BG aPriori	0.18	4.1	7.08	1.91	-158.4
S. S.Andr.;SSB+BG aPriori	0.30	4.6	7.31	1.89	-158.4
S. S.Andr.;BG+CO aPriori	0.12	4.7	7.36	2.01	-158.4
S. S.Andr.;NSB+SSB+BG aPriori	0.05	4.7	7.45	2.05	-158.4
S. S.Andr.;SSB+BG+CO aPriori	0.04	4.8	7.51	2.19	-158.4
S. S.Andr.;SM+NSB+SSB+BG aPriori	0.02	4.8	7.72	2.31	-158.4
SSAndr.;NSB+SSB+BG+CO aPriori	0.03	4.8	7.61	2.27	-158.4
SSAnd;SM+NSB+SSB+BG+CO aPriori	0.02	4.9	7.82	2.34	-158.4
S. San Andreas;BG+CO MoBal	0.07	4.7	7.35	2.01	-158.4
S. San Andreas;SM+NSB+SSB+BG+CO	0.02	4.9	7.82	2.34	-158.4
S. San Andreas Unsegmented A-flt	0.00	0.0	0.00	0.00	-156.2

#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008 *********#

PSHA Deaggregation. %contributions. site: DHS_Facilities long: 116.526 W., lat: 33.935 N.
Vs30(m/s)= 250.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.9276 g. Weight * Computed_Rate_Ex 0.214E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00105
#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
7.9	5.05	0.435	0.435	0.000	0.000	0.000	0.000	0.000
8.0	5.20	0.923	0.923	0.000	0.000	0.000	0.000	0.000
8.1	5.40	0.929	0.918	0.011	0.000	0.000	0.000	0.000
8.1	5.60	0.887	0.831	0.056	0.000	0.000	0.000	0.000
8.2	5.80	0.818	0.737	0.081	0.000	0.000	0.000	0.000
14.5	5.80	0.035	0.035	0.000	0.000	0.000	0.000	0.000
7.4	6.01	0.949	0.838	0.111	0.000	0.000	0.000	0.000
14.0	6.02	0.085	0.085	0.000	0.000	0.000	0.000	0.000
7.0	6.20	1.141	0.986	0.155	0.000	0.000	0.000	0.000
13.8	6.21	0.152	0.150	0.002	0.000	0.000	0.000	0.000
7.0	6.40	1.052	0.840	0.212	0.000	0.000	0.000	0.000
12.5	6.44	0.444	0.404	0.039	0.000	0.000	0.000	0.000
3.6	6.60	2.027	1.575	0.452	0.000	0.000	0.000	0.000
11.7	6.58	0.398	0.340	0.059	0.000	0.000	0.000	0.000
3.0	6.80	2.346	1.359	0.879	0.107	0.000	0.000	0.000
11.7	6.80	0.211	0.185	0.027	0.000	0.000	0.000	0.000
3.7	7.01	5.715	1.462	2.715	1.442	0.096	0.000	0.000
12.7	6.97	0.096	0.080	0.016	0.000	0.000	0.000	0.000
4.0	7.19	6.373	1.520	3.231	1.509	0.113	0.000	0.000
15.6	7.16	0.127	0.127	0.000	0.000	0.000	0.000	0.000
4.5	7.39	12.666	2.754	7.104	2.802	0.006	0.000	0.000
14.4	7.37	0.145	0.141	0.004	0.000	0.000	0.000	0.000
4.6	7.61	5.254	1.201	2.912	1.142	0.000	0.000	0.000
12.9	7.54	0.038	0.033	0.005	0.000	0.000	0.000	0.000
27.1	7.61	0.045	0.045	0.000	0.000	0.000	0.000	0.000
4.9	7.78	4.004	0.886	2.284	0.834	0.000	0.000	0.000
4.9	7.97	4.144	0.830	2.338	0.976	0.000	0.000	0.000
27.2	7.99	0.042	0.042	0.000	0.000	0.000	0.000	0.000
4.9	8.19	0.742	0.139	0.419	0.185	0.000	0.000	0.000
4.9	8.39	0.030	0.005	0.017	0.008	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 52.4
Mean src-site R= 5.1 km; M= 7.13; eps0= 1.09. Mean calculated for all sources.
Modal src-site R= 4.5 km; M= 7.39; eps0= 0.92 from peak (R,M) bin
MODE R*= 4.6km; M*= 7.39; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 7.104

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

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California A-faults	33.73	4.8	7.48	0.88
CA Compr. crustal gridded	7.87	8.0	5.83	1.78
San Gorgonio Zone gridded	9.74	3.2	7.02	1.13

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

Fault ID % contr. Rcd(km) M epsilon0 Site-to-src azimuth(d)

S. S.Andr.;BG aPriori	4.95	4.1	7.06	0.48	-158.4
S. S.Andr.;SSB+BG aPriori	4.95	4.6	7.31	0.81	-158.4
S. S.Andr.;BG+CO aPriori	3.21	4.7	7.36	0.92	-158.4
S. S.Andr.;NSB+SSB+BG aPriori	1.65	4.7	7.45	0.97	-158.4
S. S.Andr.;SSB+BG+CO aPriori	1.53	4.8	7.51	1.01	-158.4
S. S.Andr.;SM+NSB+SSB+BG aPriori	1.10	4.8	7.72	1.03	-158.4
SSAndr.;NSB+SSB+BG+CO aPriori	1.49	4.8	7.61	1.03	-158.4
SSAnd;SM+NSB+SSB+BG+CO aPriori	1.52	4.9	7.82	1.02	-158.4
S. San Andreas;BG+CO MoBal	2.00	4.7	7.35	0.92	-158.4
S. San Andreas;SM+NSB+SSB+BG+CO	1.21	4.9	7.82	1.02	-158.4
S. San Andreas Unsegmented A-flt	3.51	5.0	7.72	1.09	-156.2

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#*********End of deaggregation corresponding to Chiou-Youngs 2008        *********#

******************** Southern California ****************************************
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APPENDIX D

PERCOLATION TEST RESULTS

JN 16-208
Test Date: 8/17/16

ABSORPTION RATE CALCULATIONS
Open Pit Falling Head Test Procedure

By: K.Morenz

Test No.	Equivalent Boring Diameter (in)	Equivalent Boring Depth (ft)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Δ Water Level (ft)	Time Interval (min)	Water Loss (cu ft)	Wetted Area (sq ft)	Percolation Rate (in/hr)	Infiltration Rate (in/hr)*
T-2 (Trial 1)	12.6	2.5	1.00	1.50	0.50	10.0	0.4	5.0	36.0	6.2
T-2 (Trial 2)	12.6	2.5	1.00	1.50	0.50	10.0	0.4	5.0	36.0	6.2
T-2 (Trial 3)	12.6	2.5	1.00	1.50	0.50	10.0	0.4	5.0	36.0	6.2
T-2 (Trial 4)	12.6	2.5	1.00	1.50	0.50	10.0	0.4	5.0	36.0	6.2
T-2 (Trial 5)	12.6	2.5	0.92	1.42	0.50	10.0	0.4	5.3	36.0	5.9
T-2 (Trial 6)	12.6	2.5	1.00	1.50	0.50	10.0	0.4	5.0	36.0	6.2

Unadjusted (pre-factor of safety) infiltration rate: **6.2** inches/hour

* Based on Porchet Method

Figure D-1

JN 16-208
Test Date: 8/17/16

ABSORPTION RATE CALCULATIONS
Open Pit Falling Head Test Procedure

By: K.Morenz

Test No.	Equivalent Boring Diameter (in)	Equivalent Boring Depth (ft)	Initial Depth to Water (ft)	Final Depth to Water (ft)	Δ Water Level (ft)	Time Interval (min)	Water Loss (cu ft)	Wetted Area (sq ft)	Percolation Rate (in/hr)	Infiltration Rate (in/hr)*
T-4 (Trial 1)	12.6	2.5	1.17	1.79	0.62	10.0	0.5	4.2	44.6	9.1
T-4 (Trial 2)	12.6	2.5	1.33	1.92	0.59	10.0	0.5	3.8	42.5	9.8
T-4 (Trial 3)	12.6	2.5	1.17	1.75	0.58	10.0	0.5	4.3	41.8	8.4
T-4 (Trial 4)	12.6	2.5	1.25	1.83	0.58	10.0	0.5	4.0	41.8	9.0
T-4 (Trial 5)	12.6	2.5	1.17	1.75	0.58	10.0	0.5	4.3	41.8	8.4
T-4 (Trial 6)	12.6	2.5	1.17	1.75	0.58	10.0	0.5	4.3	41.8	8.4

Unadjusted (pre-factor of safety) infiltration rate: **8.4** inches/hour

* Based on Porchet Method

Figure D-2