PDC
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March 12, 2008
Work Order 500654

Attention: Ms. Karissa Sylvester

Subject: **DRAFT EIR-LEVEL GEOTECHNICAL ASSESSMENT**
"The Preserve at San Juan"
Counties of Orange and Riverside, California

References: See Appendix A

Gentlemen:

In accordance with your request, Pacific Soils Engineering, Inc. (PSE) is pleased to submit this geotechnical assessment in support of EIR submittals for "The Preserve" project located north-northwest of State Highway 74, in the County of Orange and Riverside, California. PSE has completed an initial geotechnical subsurface investigation that included air track drilling, excavation, logging, and sampling of backhoe trenches and conducting seismic refraction surveys. Additionally, PSE reviewed the technical documents listed in Appendix A as well as stereo-pair vintage aerial photographs that are archived in PSE files.

In this document PSE first summarizes the investigative methodology and the geographic, geomorphic, geologic setting; and then provides engineering properties of the earth materials of The Preserve project and its environs. We then assess geological and geotechnical engineering issues applicable to EIR processing and offer potential mitigations, if necessary. Included in the text of this report are a Site Location Map (Figure 1), two Conceptual Land Use Plans (Figures 2 and 3), regional Geomorphic and Fault Map (Figure 4), regional Geologic Map (Figure 5), Simplified Fault Map of California (Figure 6), Geologic Map with the location of the proposed offsite improvements (Figure 7), Riverside County Fault Zone Map (Figure 8), and a detailed Elsinore Fault Map (Figure 9). Appendices include the cited References (Appendix A), Subsurface Investigation (Appendix B), Laboratory Test Results (Appendix C), Seismic

Refraction Survey (Appendix D), and a Seismic Hazards Analysis (Appendix E). A 300-scale Geologic Map is included as a pocket enclosure for both the "with-trade" and "no trade" land use options (Plates 1 and 2).

The project is considered feasible from a geologic and geotechnical perspective. The significant geotechnical issues that could impact the development as conceived are discussed in Section 4.0 of this document and include: faulting and seismic hazards; rock excavation characteristics; erosion/mass wasting; slope stability; and compressible/collapsible soils. All of these issues can be mitigated and alternatives for mitigation are presented in this report.

PSE appreciates the opportunity to provide you with geotechnical consulting services. If you have any questions or should you require any additional information, please contact the undersigned at (951) 582-0170.

Respectfully submitted,
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1.0 INTRODUCTION

1.1 Background and Purpose

This report presents the results of Pacific Soils Engineering, Inc's. (PSE's) geotechnical assessment in support of EIR processing for The Preserve development.

1.2 Scope of Work

PSE's scope of work consisted of the following:

- A review of the cited geologic literature, maps, and aerial photographs;
- Site geologic mapping;
- Advancement of thirty five (35) air-track drill holes;
- Excavation, logging, and sampling of forty-eight (48) backhoe test pits;
- Conducting 14 seismic refraction surveys
- Laboratory testing of collected soil samples;
- A limited seismic evaluation;
- Evaluation of the general remedial grading requirements;
- Evaluation of shallow groundwater conditions and the potential effects on the proposed construction;
- Consolidation of the geologic and geotechnical data and preparation of a geologic base map;
- Preparation of this report with exhibits that summarizes our findings and supports your EIR submittals.

1.3 Site Location and Existing Conditions

The site is located in the southeasterly portion of Orange County, California just west of State Highway 74 (Ortega Highway). It is bisected by Long Canyon Road and is located just west of Cariso Village and east of a United States Forest Service (USFS) fire fighting housing complex (Figure 1). The Mystic Oak Spa area is located directly to the west of the northern portion of the site. Three general areas are being considered and evaluated as part of this project. The North Parcel (Neilson Property) encompasses approximately 203 acres and is located to the north of Long Canyon Road. The South Parcel (Sanchez Property) is located south of Long Canyon Road and is approximately 436 acres in size. The third parcel is currently owed by the USFS and is located both north and south of Long Canyon Road. This property is approximately 234 acres in size and is being considered for inclusion into the proposed development in exchange for properties within both the North and South Parcels.

The properties consist of varied terrain. The very northern area has a steep high ridgeline, the southern most area is a deep canyon. The bulk of the proposed developable area is gently rolling hills and small irregular valleys on a large plateau. Elevations range from approximately 3,300 feet above mean sea level (MSL) in the northeast portion of the property to approximately 2,025 MSL in the southern major canyon bottom. Most of the proposed development area is between the elevations of 2,400 and 2,900 feet above MSL. Drainage is by sheet flow to the smaller draws and canyons that flow into Long Canyon.

The property is for the most part undeveloped with the exception of Long Canyon Road and the numerous dirt roads and trails which are present throughout the site. Within the North Parcel is a dirt runway that is actively used for landings and take offs by private aircraft. This area also has a few scattered buildings and associated local utilities. The majority of the properties are covered with low to moderate brush, consisting of chaparral and coastal sage scrub. Stands of coast live oak trees, and other trees exist in the larger canyon areas.

1.4 Proposed Development

The proposed Land Use Plans (Figures 1, 2 and 3) propose a master planned community that consists of 159 dwelling units, a spa facility and a site for construction of St. Michael's Abbey. The residential development will consist of large lots (one acre or larger) for construction of semi-custom or custom type homes. The lots are often separated or surrounded by islands of natural undeveloped land to achieve a rural appeal and relatively low density. Vineyards are proposed to surround portions of the properties.

Development of the residential lots will include the network of access roads and attendant utilities necessary to support the development. Primary access will be from Long Canyon Road with secondary and emergency access to Highway 74 at one other location to be determined. Portions of Highway 74 will be improved near the intersection of Long Canyon Road. Several of the primary utilities will also extend or require improvement off site. This includes a proposed water line and dry utilities that will be constructed in a 21 feet wide easement from the area of Lake Elsinore up to the proposed development (Figure 1). Sewer service will be achieved through construction of a new pipeline to the existing treatment facility located west of the site that is operated for the nearby County of Orange Los Pinos Conservation Camp (Figure 1).

Two development options are being proposed for this project. A “with trade” option (Figure 2) in which USFS property will be swapped with private property, and “no trade” option that maintains the current property boundaries.

The “with trade” option centers the entire development near the existing Long Canyon Road using the property that is currently owned by the USFS. The “with trade” option also includes additional property that would be utilized by St. Michael’s Abbey. The “no trade” option (Figure 3) differs in that a string of lots would be developed on a ridgeline near the western edge of the South Parcel, and development immediately adjacent Long Canyon Avenue would not occur. In addition a sinuous access road would need to be developed from the eastern portion of south parcel up to this isolated ridgeline.

2.0 INVESTIGATIVE METHODOLOGY

No previous geotechnical studies are known to have been conducted at the site. In April of 2005, PSE advanced thirty-five (35) air track holes; excavated, logged and sampled forty-eight (48) backhoe test pits, and conducted fourteen (14) seismic refraction surveys. The test pits ranged in depth from 6 to 18 feet, and the air track holes ranged from 10 to 41 feet in depth. Collected samples were delivered to the laboratory for testing to characterize the engineering properties of the onsite earth materials.

The purpose of the investigation was to evaluate the distribution of earth materials within the property and to determine their engineering and excavation properties. The approximate locations of the excavations and seismic survey locations are presented on Plates 1 and 2. The logs of the air track holes and trenches are presented in Appendix B. Appendix C contains the results of the laboratory testing performed on the samples collected from those excavations. Appendix D contains the findings of the seismic refraction survey lines. Other then geologic field mapping, no subsurface investigation was performed for the proposed offsite improvements (sewer line, water line and dry utilities).

3.0 GEOLOGIC CONDITIONS

3.1 Geologic Setting

The subject property is located in the Santa Ana Mountains of the Peninsular Range geomorphic province in southern California (Figure 4). The Santa Ana Mountains are composed of basement complex crystalline and semi-crystalline rocks of Mesozoic age unconformably overlain by upper Cretaceous and Cenozoic sedimentary rocks. The geologic relationships established in the Santa Ana Mountains indicate that the boundary between basement and superjacent sedimentary rock is of early or middle Cenozoic age and the northeastward transgression of Paleocene strata onto successively older units infers an early Tertiary southwestward tilt of the mountain mass. The emergence of the mountain mass continued into middle Miocene time when the relative depression of the Los Angeles Basin began. Deformation has continued since that event and has produced distinct erosional unconformities in upper Miocene, Pliocene, and upper Pleistocene strata (Schoellhamer et al., 1981).

3.2 Stratigraphy

The subject property is located within the basement complex of rocks that form the core of the Santa Ana Mountains (Figure 5). The two bedrock units that have been mapped within the property limits are the Cretaceous age Woodson Mountain Granodiorite (commonly referred to as granite) and the Jurassic age Bedford Canyon Formation, which is primarily composed of moderately metamorphosed argillite, slate, greywacke and quartzite. These bedrock units are locally mantled by thin surficial deposits of colluvium (soil) and/or alluvium in the more significant drainage bottoms.

The USFS property, the South Parcel, and most of the North Parcel are underlain entirely by the Woodson Mountain Granodiorite (Figure 5). The northern most portion of the North Parcel is underlain by the Bedford Canyon Formation. A hill top within the central portion of the North Parcel has also been mapped as Bedford Canyon Formation. The proposed easement for the offsite sewer line is underlain by granodiorite. The proposed easement for the water line is underlain primarily by granodiorite, except at the northern end, which is underlain by bedrock of the Bedford Canyon Formation (Morton and Miller, 1981). The mapped distribution of geologic units with the proposed development is shown on Plates 1 and 2. Presented below is a brief description of the geologic units mapped onsite.

No landslides, significant or active faults have been mapped, or are known to exist within the properties. No significant springs have been mapped within the property. However, minor seepage was observed from various locations during the spring of 2005 when PSE conducted the field investigation at the site.

3.2.1 Undocumented Artificial Fill (no map symbol):

Undocumented artificial fills consist of existing dirt trails and roads, boring and trench backfill, catchment berms and other minor miscellaneous improvements. The majority of these fills are not depicted on the accompanying Plates 1and 2 due to the relatively small size of these fills in comparison to the map scale. The undocumented fills are most likely derived from onsite sources and consist of fine to coarse-grained sand with varying amounts of pebbles. The undocumented fills are in a dry, loose condition.

3.2.2 Alluvial/Colluval Deposits (map symbol Qal):

The Quaternary age alluvial deposits are located primarily within the active drainage courses. Colluvium, where encountered, was generally less than a few feet in thickness and therefore was not mapped as a separate unit. The alluvial deposits are generally subangular, loose to medium dense. Where encountered the alluvium was less than a few feet in thickness. In the main canyon area pockets of alluvium may be thicker.

3.2.3 Quaternary Older Alluvium (map symbol: Qalo)

Within the main courses of Long Canyon and Decker Canyon the older alluvium has locally formed broad alluvial pockets that are likely up to 20 feet or more in thickness. It is our experience, in nearby areas, that the upper few feet of this alluvium consists of a sandy soil and is underlain by older silty sand that is often porous, with local zones of gravel and cobbles.

3.2.4 Woodson Mountain Granodiorite (map symbal Kgr):

The Cretaceous age Woodside Mountain Granodiorite is an intrusive rock exposed throughout most of the proposed development area. The rock is a grey, coarse grained, with white plagioclase, blueish grey quartz, black biotite and horneblend of relatively uniform composition and texture. The Woodson Mountain Granodiorite varies from hard rock outcrops and large core-stone boulders at the surface to areas of softer, decomposed granite to a depth of approximately 10 feet. This bedrock appears to be fairly massive with few continuous fracture or fault zones.

3.2.5 Bedford Canyon Formation (map symbol Tbc):

The Jurrassic age Bedford Canyon Formation were intruded by the younger Cretaceous igneous rocks. This formation is generally composed of low-grade metamorphic volcanic and sedimentary rocks typified by shale, greywacke, and quartzite. This rock is primarily found in the northern portion of the project where no development is planned. It underlines a knob within the north parcel and also underlies the north end of the offsite water line.

3.3 Geologic Structure/Tectonic Setting

3.3.1 Regional

The Santa Ana Mountains lie within the Peninsular Ranges Geomorphic Province extending from the San Gabriel Mountains in the north to the southern tip of Baja California, Mexico. This Province is typified by northwest-trending alluvium-filled basins, elevated Pleistocene surfaces undergoing active erosion and northwest trending mountain ranges formed along faults oriented in the same direction (Figure 4).

The approximate 125-mile long Elsinore Fault Zone (EFZ) is the most significant fault in relation to the site. Near Lake Elsinore it forms a right-oblique, transtensional, pull apart tectonic basin with local reverse and normal –slip components (Weber, 1977; Shlemon and Ginter, 2001). It is one of several northwest-trending continental borderland fault zones that extend from the Mojave Desert in the east to the Channel Islands in the west (Jennings, 1994). It consists of many individual faults and as part of the boundary separating the North America and Pacific Plates; these faults typically exhibit evidence of Holocene displacement as well as historic seismic activity.

Faults within the EFZ have classic geomorphic and stratigraphic chraracteristics of active (Holocene) faults. Right-lateral offsets of streams, canyons/interfluves and quaternary alluvial fans abound along its trace, as do young scarps and sag ponds (e.g. Glen Ivy Marsh).

Recent paleoseismic studies (Rockwell and others, 1986; Millman, 1988) in trenches at Glen Ivy Marsh (Table A) located about 8-miles to the northwest, identified at least five historical ground rupture events on the EFZ.

Table A Summary of Late Holocene Earthquake History at Glen Ivy Marsh (After Rockwell and Others, 1986)	
Date	**Size**
Historical post 1660A.D.	30-50 cm vertical separation, unknown
Between 1360 and 1660 A.D.	2-3 cm vertical separation; horizontal separation Unknown
About 1300 A.D.	Large event: 0 to 20 cm of vertical separation and at least 90 cm of horizontal separation unknown
1260-1275 A.D.	3-5 cm vertical separation, horizontal separation unknown
About 1060 A.D.	Large event: 10-30 cm vertical separation, horizontal separation unknown

In addition to the EFZ, many other significant faults occur within 100 km of the subject site (Blake, 2004). Selected faults are discussed with respect to proximity to the site. The most significant of these is the San Andreas Fault.

The San Andreas Fault is a major tectonic feature of western North America. The fault traverses 1,100 km from Cape Mendocino, north of San Francisco, to the Gulf of California in the south. It is interpreted to have formed as a transform fault which delineates the boundary between the North American and Pacific plates (Powell, 1993). The San Andreas fault is located, at its closest point, approximately 91 km northwest of The Preserve project.

3.3.2 <u>Local Faulting</u>

No active or significant faults have been mapped within the proposed project site. The Los Pinos Fault has been mapped to the northwest of the project and The San Juan Fault has been mapped to the southeast of the project (Figure 5). The EFZ is located at the northern end of the proposed offsite water line. This fault zone is discussed in more detail below.

Lake Elsinore and its environs occupy part of a fault-bounded valley (graben) formed by right-transtensional slip along elements of the Elsinore Fault Zone (EFZ)-an active northwest-trending fault zone within the Peninsula Ranges of Southern California (Figure 4, herein; Kennedy, 1977; Weber, 1977; Rockwell and others, 1986). The Willard and Wildomar Faults of the EFZ form the southwestern margin of the valley, and Glen Ivy North segment of the EFZ in part forms the northeast margin valley (Figure 4-9). Elements of the Willard Fault have been mapped near the northern end of the water line [Figure 7 and 9, herein; Engle, 1959; Weber, 1977; Kennedy, 1977; Morton and Weber, 1991; County of Riverside (2003); Mortan, 2004]. Mortan (2004), Weber (1977), Kennedy (1977), and The County of Riverside (2003) have mapped splays of the Willard Fault trending northwest at the northern end of the water line. The Wildomar Fault is about one-half mile to the east and the Glen Ivy North Fault is about 4 miles north.

Willard and Wildomar Faults

The northern extent of the Willard Fault has been mapped near the northern end of the water line by several authors (for example, Engel, 1959; CDMG, 1965; Weber, 1977; Kennedy, 1977; Morton, 2004). Youthful, scarp-like features have been identified at two locations near by Weber (1977) are suggestive of youthful faulting in younger and slightly older valley fill. A northwest-trending lineament mapped by Weber just north of the water line was not recognizable during review of aerial photographs or during PSE’s 2005 field investigation. Furthermore, the Willard Fault splays and associated photo-lineaments have been poorly defined and mapped in the area surrounding that project site (Smith, 1978).

The northwest-trending Wildomar Fault can be traced near the southwestern shoreline of Lake Elsinore (Figure 9, herein; Engel, 1959; Weber, 1977; Morton, 1999, 2004). This Fault is described as a series of right stepping, strike-slip faults with steep, west-dipping normal components. A prominent alignment of stags, fault-line scarps, and displaced outcrops are found along this fault (Engel, 1959; Weber, 1977). The trace has been arbitrarily extended along the southwestern boundary of Lake Elsinore, less then ½-mile east of the proposed water line. Shlemon and Ginter (2001) have concluded that the Wildomar Fault is now likely taking up most neotectonic slip on the south side of Lake Elsinore.

3.4 Ground Water

No significant springs have been mapped within the property, however minor seepage was observed from various bedrock locations during the spring of 2005 after periods of rain when PSE conducted a field investigation at the site. It is anticipated that the broad alluvial valleys may have locally perched pockets of ground water.

3.5 Mineral Deposits

No mining or significant mineral deposits are known to exist within the subject properties. However, the Old Dominion Mine is located approximately one mile northwest of the northern parcel. This mine is located within a fissure vein of the Bedford Canyon Formation. The vein is highly irregular in width and degree of mineralization present. The ore that was mined included argentiferous (silver), galena (primarily lead), sphalerite (contains zinc), minor gold within a quartz matrix, pyrrhorite, arsenopyrite, and siderite (Morton, et. al., 1976). Mining of the vein was conducted from to 1894 until 1943, with several hundred tons of ore being mined (California Division of Mines, 1959). In 1965, 13 drill holes were excavated in the area of the mine, which indicated that the vein of ore was erratic and discontinuous (Morton, el. al., 1976). The mine has not processed any ore since 1943.

4.0 GEOTECHGHNICAL ENVIRONMENTAL IMPACTS

Appendix G of the California Environmental Quality Act (CEQA) guidelines requires an evaluation of environmental conditions and the potential impacts of those conditions. That evaluation must classify the conditions as to whether there is “No Impact” or “Impact”. If the condition “impacts” the site, then it needs to be classified as 1) "potentially significant"; 2) "less than significant with mitigation"; or 3) "less than significant".

The threshold for determining geotechnical impacts as outlined in Appendix G of the CEQA guidelines are described as follows:

Would the project:

a) *Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:*
   - i) *Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault? Refer to Division of Mines and Geology Special Publication 42.*
   - ii) *Strong seismic ground shaking?*
   - iii) *Seismic-related ground failure, including liquefaction?*
   - iv) *Landslides?*

b) *Result in substantial soil erosion or the loss of topsoil?*

c) *Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?*

d) *Be located on expansive soil, as defined by Section 1802.3.2 of the California Building Code (2007), creating substantial risks to life or property?*

e) *Have soils incapable of adequately supporting the use of septic tanks or alternate waste water disposal systems where sewers are not available for the disposal of waste water?*

Presented in the following sections is PSE's evaluation of the potential geotechnical impacts to the site based on the above classifications.

4.1 Fault and Seismic Hazards

4.1.1 Fault Rupture Potential

There are no State-imposed Alquist-Priolo Fault-Rupture Hazard Zones mapped onsite. In addition no faults have been mapped or observed within the site. Thus, the likelihood of surface fault rupture at the site is considered remote. The proposed offsite water line and dry utilities cross a branch of the EFZ that is located within a County of Riverside defined

Earthquake Fault Hazard Zone; however, the presence and activity of this portion of the fault has not been well documented.

Level of Impact

The level of impact due to Fault Rupture Potential is considered to be "no impact" for the residential portion of the proposed development. There is a "less than significant impact with mitigation" for the proposed offsite water line.

Mitigation Measures

Mitigating fault rupture hazards for pipelines that cross fault zones is generally through the installation of automatic emergency shut-off valves and/or construction of flexible joints in the zone of potential fault rupture hazards. Future evaluations should include more precise location of zones that have a potential for fault rupture hazard and coordination with the local water agencies regarding potential mitigation measures preferred.

4.1.2 Seismic Ground Shaking

Southern California, in general, is a seismically active region and the proposed improvements are likely to be subjected to significant ground motion during the design life of the project. To provide estimates of the range of strong ground motion that the project could experience during its lifetime, PSE has performed a Probabilistic Seismic Hazard Analysis (PSHA) utilizing the FRISKSP software program. FRISKSP is a software program developed from United States Geologic Survey data by Blake (2004). Attenuation relationships by Sadigh, et al. 1997 (rock), Boore et. al. 1997 (hard rock and soft rock) and Campbell and Bozorgnia 1997 Rev. (rock and soft rock), were used to compute a mean plus one standard deviation peak ground acceleration (PGA). Equal weight was given to all relations. An average PGA of 0.52g (rock) and .58 (soft rock) was computed for earthquake ground motions having 10% probability of exceedance in 50 years, generally termed the "design basis earthquake".

In general accord with the 2007 California Building Code (CBC) a maximum "Considered earthquake" should also be considered using a 2% probability of exceedence in 50-years, particularly regarding essential structures. For the maximum considered earthquake an average PGA of 0.63g (rock) and .70 (soft rock) was determined. However, the CBC does not mandate the use of raw PGA for structural design, but rather recommends modified spectral accelerations and velocities (CBC, 2007)

Level of Impact

The level of impact due to seismic shaking is considered to be less than significant with mitigation.

Mitigation Measures

Seismic shaking can be mitigated through the design of the structures in compliance with prevailing seismic codes outlined in California Building Code (CBC, 2007). Remedial grading to further mitigate seismic hazards may also be required. Remedial grading could include:

- *The removal and replacement of loose and/or compressible soils with engineered fill; and*
- *The removal and recompaction of the cut and shallow fill portions of building lots that exhibit unfavorable geologic conditions. These lots will be identified as a part of future geotechnical grading plan reviews and during construction of the improvements based on exposed geologic conditions in the field.*

4.1.3 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement are the processes by which saturated sediments lose their strength during strong ground motion generated by earthquakes. The State of California (California Division of Mines, 1997) has mandated that the California Geological Survey identify areas that may be susceptible to liquefaction, and through the Seismic Hazards Mapping Act (SHMA) to provide quadrangle maps showing these zones and establish procedures for studies prior to project approval. SHMA quadrangle maps for "The Preserve" area have not yet been published. As a part of PSE's preliminary investigation of the site, an evaluation indicated no significant potential for liquefaction and/or dynamic settlement.

Level of Impact
The level of impact due to liquefaction and dynamic settlement is considered to be less than significant.

Mitigation Measures
Mitigation measures for liquefaction/dynamic settlement potential include removal and replacement with compacted, drained fills; ground modification; and/or designing for potential settlement of liquefiable materials.

4.1.4 Earthquake Induced Landsliding

The subject site has not been evaluated under SHMA. Proposed slopes and/or natural slopes steeper than 1.5:1 (horizontal to vertical) and cut slopes that expose unfavorable geologic conditions such as daylighted jointing, low strength, or poorly cemented soils, are potentially susceptible to the secondary seismic hazard of earthquake-induced rock falls or minor landsliding.

Rock falls generally will only occur on slopes steeper than 1.5:1 (H:V). For the "with-trade" option, no structural areas are proposed in areas below natural or proposed slopes that are steeper than 1.5:1. Therefore, there is no need for mitigation of the potential for rock fall. The "no-trade" option has an access road between the western and eastern portions of the South Parcel that is below natural slopes that are locally steeper than 1.5:1 (H:V) and therefore has a potential for rock fall.

Level of Impact
The level of impact due to earthquake induced landsliding is considered to be less than significant with mitigation.

Mitigation Measures

Areas susceptible to earthquake-induced landsliding can be mitigated utilizing common earthwork remedial grading techniques such as construction of drained shear keys, replacement with manufactured buttress fills, slope laybacks, or structural setbacks.

Areas subject to earthquake-induced rock fall can be mitigated by slope layback, setbacks from the toe of slope areas. In addition catchment nets, debris barriers, and other methods are available to reduce the risk of earthquake induced rock fall to an acceptable level.

4.1.5 Seiches, Tsunamis, and Dam Failures

Seiches are periodic oscillations within a large enclosed body of water. Any enclosed body of water such as an artificial lake, reservoir, or tank could be susceptible to seiche oscillations. A tsunami is a large oceanic wave generated from an earthquake or undersea landsliding.

Level of Impact

The elevation and distance from large bodies of water of the development precludes inundation resulting from tsunamis, seiches, and dam failure, and therefore, has no impact on the project.

Mitigation Measures

None.

4.2 Rock Excavation Characteristics

Generally, hard granitic rocks and Bedford Canyon formation underlie the entire site. These rocks will generally require blasting for cut areas deeper than fifteen (15) to twenty (20) feet below the existing ground surface. Difficult excavation maybe encountered five (5) to fifteen (15) feet below ground surface. Oversize rock will be generated from the cut areas that will require specialized grading techniques and disposal.

Level of Impact

As both conceptual land use plans include development within hard rock areas, the use of blasting to achieve design grades is likely to be required. Therefore, this condition is considered to have significant economic impact on the project development; however, less than significant impact with mitigation geotechnically.

Mitigation Measures

The impacts of the use of explosives include noise, vibration, and flying debris. Those impacts can be mitigated through the use of experienced blasting contractors who would submit a blasting plan to the appropriate agencies for approval. Additionally, numerous small charges are typically used in the blasting process and overburden is typically left-in-place to improve the effects of the detonation. This process, combined with the remote location of the site in relation to existing homes or other structures, can significantly reduce the impacts of noise, vibration, and flying debris. Other geotechnical concerns related to hard rock conditions that can be mitigated include excavation of deep utilites, swimming pools and other underground improvements. Mitigation for these positional improvements would include overexcavation or blasting to the anticipated depth.

4.3 Soil Erosion/Mass Wasting

Soil erosion or mass wasting is the process in which earthen materials are transported down slope by gravity. Large scale mass wasting is not present onsite and is not anticipated to be a hazard to the project. Deposits of topsoil of relatively minor thickness (a few feet) are present over a majority of the site. The relative lack of topsoil is due to the arid environment and the hardness of the bedrock. Seasonal runoff is the principal agent of erosion in addition to local, shallow, soil slumping.

In the area of the proposed offsite water pipeline the soils and/or shallow fills were locally being eroded due to concentrated runoff from the access roads which parallel the existing easement. This erosion has locally undermined an existing water pipeline.

Level of Impact
Due to the lack of significant topsoils at the site, and the proposed site improvements, soil erosion or mass wasting is deemed be less than significant with mitigation on the site.

Mitigation Measures
Control of surface drainage and diversion of flows to non-erodible devices are the principle mitigation measures typically employed and can be accomplished with compliance with design standards outlined in the CBC. Mitigation of slope surface erosion of highly granular soils can be accomplished by establishing appropriate surface drainage patterns, judicious landscaping, and, when necessary, use of surface erosion control products such as "jute mesh" or "straw waddles" in compliance with erosion control standards. In the steep areas of the proposed offsite pipelines more aggressive erosion control methods maybe necessary to reduce the potential impacts of concentrated water flow, that has currently undermined the existing water line. These more aggressive forms of erosion control could be use of flow diverters, rip-rap or other hardened, wall-like structures where erosion is anticipated to be concentrated. In addition, the use of soil cement, Line treat or other soil additives could also reduce the erodible nature of trench backfill or embankments, which support the proposed pipeline and utilities. Detailed mapping during later development stages should identify those areas that may require mitigation.

4.4 Slope Stability

Cut, fill, and natural slope stability can be affected by several factors including geologic structure, strength of materials, height, inclination, and orientation of design slopes. Bedding within the granitic bedrock is absent, and jointing is anticipated to be primarily related to weathering and not well defined or continuous at depth. Therefore, only the upper weathered zone is anticipated to have a significant potential for weak planar features that could be prone to mass movement with the slope angles proposed.

The “with trade” project plans indicate that all proposed cut slopes will have design cut slopes that are no steeper than a 2:1 (horizontal to vertical) inclination. The “no trade” project plans have cuts slopes which are proposed to be as steep as 1.5:1 inclination and up to 100 feet in height. In addition, geogrid reinforced walls are proposed along the access road from the western portion to the eastern portion of the South Parcel in order to minimize the height of proposed cut slopes in these areas. Natural slopes are generally flatter than 2:1 in inclination, however, locally same sections that are steeper than 1.5:1.

Evaluation of the interrelationships of the various combinations of slope configuration, geologic structure, and material strength characteristics will be required to assess each specific slope condition when more detailed project designs are evaluated.

4.4.1 Cut Slopes

The underlying bedrock is generally capable of supporting Code-compliant, 2:1 or flatter cut slopes. For the “no trade “ option cut slopes are proposed at inclinations steeper than 2:1. Slopes as steep as 1.5:1 are likely stable pending results of additional future geotechnical but will require a waiver from the County of Orange or the County of Riverside because they are not compliant with either County’s grading standards.

Level of Impact

The level of impact due to cut slope instability is considered to be less than significant with mitigation.

Mitigation Measures

Stability of proposed cut slopes may be of significant economic importance to the proposed development. All cut slopes will require evaluation during the design process as well as during construction. Mitigation of some slopes may be required and will likely include overexcavation and replacement with either drained stabilization fill or buttress fills. Stabilization fills should be utilized when cut slopes expose loose or highly erosive soils or highly fractured bedrock. At the developers discretions selected cut slopes may be converted with a replacement fill so that more desirable landscaping can be established.

4.4.2 Fill Slopes

Based on the engineering characteristics defined by our laboratory testing, the onsite earth materials are generally considered suitable for use as engineered fill and, when properly constructed and maintained, can be expected to perform satisfactorily in Code compliant embankments and fill slopes (typically 2:1 or flatter). For the "no trade" option numerous reinforced earth walls are proposed for the access road between the east and west portions of the South Parcel. These walls systems are considered structural features and therefore will require engineering analysis and specific design prior to construction.

Level of Impact

The level of impact due to the design and construction of fill slopes is considered to be less than significant with mitigation.

Mitigation Measures

For the "with trade" option fill slopes are designed at a 2:1 (H:V) or flatter ratios. These slopes should perform adequately with proper construction and maintenance. For the "no trade" option slopes fill locally steeper than 2:1 are being considered. These slopes should be *constructed with a block facing and reinforced with a geosynthetic fabric to enhance the strength of fill materials and improve the surficial and gross stability of these slopes. Other potential mitigation measures are available to mitigate and stabilize proposed slopes that are steeper than 2:1 inclination. Subsurface drainage devices should be installed below fills to intercept and direct water that may seep from the bedrock or be introduced from the surface.*

4.4.3 Natural Slope Stability

The proposed plans indicate that natural slopes will surround the perimeter of the project. In general, these natural slopes have an inclination of 2:1 or less with localized areas of steeper, approximately 1:1 slopes. Because of the nature of the bedrock these slope are considered grossly stable. However, the slopes steeper than 1.5:1 may have a potential for rock fall hazard.

Level of Impact

The level of impact due to natural slope stability is considered less than significant with mitigation.

Mitigation Measures

Steep natural slopes above the proposed project should be evaluated for rock fall potential. Mitigation techniques may include structural setbacks, rock catchment walls or fences, layback of natural slope areas, or a combination of these measures. Mitigation alternatives discussed above can be implemented to correct local instabilities, if they exist.

4.5 Compressible/Collapsible Soils

Based upon the data obtained from this firm's subsurface investigation and laboratory testing, highly weathered granite and alluvial deposits are likely to be compressible.

Hydro-collapse is the process in which loose dry soils undergo rapid consolidation (collapse) when wetted. Unmitigated, the presence of compressible and collapsible soils below fills and where exposed in cuts can produce significant settlements that can be manifested differentially on engineered structures.

<u>Level of Impact</u>

The level of impact due to collapsible soils is considered to be less than significant with mitigation.

<u>Mitigation Measures</u>

Typically, compressible and collapsible soils can be mitigated using a combination of removal and overexcavation of the susceptible soils and recompaction as engineered fills. PSE estimates that remedial grading removals will be on the order of one (1) to ten (10) feet. All undocumented fills will also require removal and recompaction.

4.6 <u>Expansive Soils</u>

The expansion potential of the vast majority of soils that will be encountered onsite during grading will likely range from "very low" to "low". Expansive soils can increase in volume upon the introduction of water, and decrease in volume (shrink) upon drying. These volume changes can produce stresses on engineered structures than can result in cosmetic distress and even structural damage.

<u>Level of Impact</u>

The level of impact due to expansive soils is considered to be less than significant with mitigation.

<u>Mitigation Measures</u>

The presence of expansive soils and bedrock is commonly and effectively mitigated by various techniques including: 1) proper design of foundations, slabs, streets, and other improvements subject to the influence of the soils; 2) overexcavation of the expansive soils/bedrock and replacement with less expansive fill soils; 3) utilizing selective grading techniques to place more highly expansive soils well below *foundation elements; 4) employment of presaturation techniques to lessen expansion potential; 5) control of surface and subsurface drainages to reduce moisture variations; and 6) combinations of these various techniques.*

4.7 Percolation Characteristics of Site Soils

Development plans for the site call for a sanitary sewage system to handle waste. At this time no septic systems are planned. If development plans are changed requiring the use of septic systems, then the site soils are of the type that generally exhibit favorable percolation characteristics. Specific studies would be required to identify the percolation characteristics of the site soils if development plans change.

Level of Impact

Currently septic systems are not planned for this site, therefore the percolation characteristics of the site soils are deemed to have "No Impact" on the site development.

4.8 Corrosion

The presence of soluble sulfates in soils can be detrimental to concrete. Low resistivity soils can have a detrimental effect on metals. Based upon the laboratory results, the onsite soils exhibit "negligible" sulfate exposure and are classified as "mildly corrosive" in accordance with NACE standards.

Level of Impact

The level of impact due to corrosive soils is considered to be less than significant with mitigation.

Mitigations Measures

Consultation with a Corrosion Engineer is recommended in order to mitigate the potential corrosive effects on metal portions of structures and should be accomplished in compliance with CBC. Final mitigations should be based on testing of as-graded soil conditions.

5.0 FUTURE GEOTECHNICAL ANALYSIS

Prior to approval of a Tentative Tract Map, a geotechnical report shall be prepared by a licensed Engineering Geologist and Geotechnical Engineer and submitted to the governing agency for review and approval. This report shall be prepared in accordance with the governing agency standards and shall evaluate the proposed development in relation to site soils and geologic conditions. Recommendations shall be provided to specifically identify and mitigate any hazards related to faulting and seismicity, collapsible soils, expansive soils, corrosion, and slope stability.

APPENDIX A

References

APPENDIX A

References

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APPENDIX B

Subsurface Investigation

APPENDIX B

Subsurface Investigation

PSE's subsurface investigation was conducted in April, 2005. An approximately 47,000 lbs track-hoe excavator was used to excavate forty-eight (48) test pits to evaluate the near surface earth materials. The test pits ranged from 6 to 18 feet in depth. They were generally extended until the earth materials were too difficult to excavate. Collected bulk samples were delivered to the laboratory for testing to characterize the engineering properties of the near surface earth materials. A representative of this firm logged each test pit. The logs are presented herein and the location of each excavation is shown on Plates 1 and 2

PSE advanced thirty-five (35) Air Track holes (Ingersol-Rand EMC-370) at a constant rate and time was recorded for the depth of penetration. The air track holes ranged from 10 to 41 feet in depth. Based on previous experience the depth to marginal rippability and the depth to likely blasting were estimated for each hole. A log of time versus depth is presented herein with estimated rippability charateristics.

Representative bulk samples were obtained from the exploratory excavations and delivered to PSE's laboratory for testing and analysis.

APPENDIX C

Laboratory Analysis

APPENDIX C

LABORATORY TESTING

The results of laboratory testing performed during this study are enclosed within this Appendix. Table C-1 presents a summary of laboratory test results.

The following laboratory tests were performed on representative samples in accordance with the applicable latest standards or methods from the ASTM, Uniform Building Code (UBC), and the California Department of Transportation.

Particle Size Analysis
Modified hydrometer grain size analyses (ASTM D 422-63 (02)) were conducted to aid in classification of the soils. The results of the hydrometer particle size analysis are presented in Table C-1.

Direct Shear Tests
Direct shear tests were performed on relatively "undisturbed" samples and samples that were remolded to 90 percent of the laboratory maximum density. Samples were tested after inundation and confinement for 24 hours. Tests were made under various normal loads at a constant rate of strain of 0.05 inches per minute. Shear test data is presented in Table II and on Plates C-1 and C-2.

Expansion Tests
Expansion index tests were performed on selected samples in accordance with the expansion index UBC Standard No. 18-2. Results are presented in Table C-1.

Compaction Characteristics
Maximum densities and optimum moistures were determined for selected samples in accordance with ASTM: D 1557-02. Results are presented in Table C-1.

Chemical Testing
Chemical tests were performed to analyze the corrosion potential of the on-site soils on ferrous metals and concrete. As part of this testing, sulfate contents were determined to

analyze the potential for sulfate attack on concrete products. Additionally, pH and electrical resistivity were also determined test results are summarized in Table C-1.

APPENDIX D

Seismic Refraction Survey

By
Subsurface Surveys & Associates, Inc.

APPENDIX E

PROBABILISTIC SEISMIC HAZARD ANALYSIS

Introduction

The 1933 Long Beach, 1971 San Fernando, 1992 Landers, 1994 Northridge, and 1999 Hector Mine earthquakes particularly illustrate both regional seismicity and the need to incorporate seismic considerations into project design. Current standards of practice and regulatory agencies dictate such. PSE therefore provides herein *probabilistic* estimates of free-field peak horizontal ground accelerations (PGA) that hypothetically could be generated by earthquakes along regional and local seismogenic faults, that are essential to assessment of hypothetical site effects such as liquefaction and dynamic settlement, and that may also be useful to some dynamic structural design methods. The PGA estimates given in this appendix are based on guidelines set forth in the 2007 California Building Code (CBC, 2007), CDMG (1997), and Martin and Lew (1999). Please see Appendix A for references cited in this appendix.

Selection of the appropriate design seismic parameters depends upon the kinds of geotechnical or structural analyses (for example, static or dynamic), the kind and sensitivity (normal-risk vs. critical-risk) of proposed structures, and the level of "acceptable risk" deemed suitable for the project. Normal-risk structures usually include those where the CBC (2007) concern is primarily life and safety during, rather than structural performance after, a major earthquake. Critical-risk facilities include, but are not necessarily limited to, schools, hospitals, dams, and other structures where the most superficial failure is intolerable (Krinitzsky, 1995). PSE assumes that "normal" and "critical" or essential risk structures are planned for The Preserve Project. Thus, both the "Design-Basis Earthquake" (DBE) that is the PGA that has a 10-percent chance of being exceeded in 50 years, which is typically applied to "normal risk" structures; and the maximum "considered earthquake" for "critical" risk structures that has a 2-percent chance of being exceeded in 50 years are given herein. However, the CBC does not mandate the use of raw PGA for structural design, but rather recommends modified spectral accelerations and velocities (CBC, 2007)

This firm reviewed published and unpublished literature about regional active faults, and about the potential for and possible magnitudes of future seismic events along those faults.

Also, articles that empirically relate proximity of postulated earthquakes to possible on-site PGA were reviewed. In this appendix, principal regional active faults and earthquakes are briefly described, and the probabilistic methodology used to estimate PGA is then spelled out.

Active Faults

Several definitions of an active fault--in this case seismogenically active--have evolved over the years. For this discussion, an active fault as defined by the California Code of Regulations (Title 14, Sec. 3601a) is:

> "A fault that has had surface displacement in the Holocene (about the last 11,000 years) hence constituting a potential hazard to structures…"

Regional Faults

Because the site is located in a seismically active region, numerous active faults capable of generating moderate to large earthquakes lie within 100-kilometers. The faults shown on Figures E-1 and E-2 are from Blake (2004) as modified from Cao, and others, (2003). Thus, by consensus, those faults possess the potential to give rise to moderate to large earthquakes, and hence moderate to strong PGA at the study site.

Two kinds of faults are in essence represented on Figure E-1 and E-2: 1) northwest trending, right-lateral, strike-slip faults that occur in a belt that extends from the Mojave Desert on the east to beyond the Channel Islands on the west (Jennings, 1994); and 2) left-oblique reverse or thrust faults that owe their existence to ongoing compression resultant from convergence of the North American and Pacific plates. The San Andreas Fault Zone (SAFZ), The San Jacinto Fault (SJF), and the Elsinore Fault Zone (EFZ), are the largest nearby active faults. The EFZ is the closest of these at a distance of around 3 miles to the east. This Fault zone includes the Willard and Wildomar Faults.

The SAFZ is the longest (>700 miles) and most prominent in California. It reaches from the Gulf of California to Cape Mendocino and has historically been reported to produce earthquakes up to magnitude 8. The section of this fault closest to the subject site is capable of generating a magnitude 7.4 earthquake (Wells and Coppersmith, 1994).

Table E-1 lists major seismogenic faults within about 100 kilometers of the study site as set forth by Cao and others (2003).

TABLE E-1 Regional Faults		
Abbreviated Fault Name	**Approximate Distance Km.**	**Approximate Distance Mi.**
Cleghorn	2.0	1.2
North Frontal Fault Zone (West)	6.8	4.2
San Andreas – Whole M-1a	9.7	6.0
San Andreas – SB-Coach. M-1b-2	9.7	6.0
San Andreas – San Bernardino M-1	9.7	6.0
San Andreas – SB-Coach. M-2b	9.7	6.0
San Andreas – Cho-Moj M-1b-1	14.5	9.0
San Andreas – Mojave M-1c-3	14.5	9.0
San Andreas – 1857 Rupture M-2a	14.5	9.0
San Jacinto – San Bernardino	16.0	9.9
Cucamonga	17.2	10.7
San Jacinto – San Jacinto Valley	34.2	21.2
Helendale – S. Lockhardt	35.7	22.2
San Jose	38.7	24.1
Sierra Madre	42.1	26.2
Clamshell – Sawpit	46.9	29.2
Chino – Central Ave. (Elsinore)	48.2	30.0
North Frontal Fault Zone (East)	49.9	31.0
Elsinore (Glen Ivy)	57.5	35.7
Whittier	57.5	35.7
Lenwood-Lockhart –Old Women Springs	59.4	36.9
Raymond	61.8	38.4
Pinto Mountain	63.9	39.7
Puente Hills Blind Thrust	64.4	40.0
Johnson Valley (Northern)	64.8	40.2
Landers	68.9	42.8
Gravel Hills – Happer Lake	72.7	45.2
Elsinore (Temecula)	74.1	46.0
San Jacinto-Anza	74.5	46.3
Upper Elysian Park Blind Thrust	74.7	46.4
Verdugo	77.2	48.0
Emerson So. – Copper MTN.	77.5	48.1
San Jacinto-Hills	81.9	50.9
Calico Hidalgo	82.8	51.5
Blackwater	84.3	52.4
Hollywood	84.4	52.4
San Gabriel	86.1	53.5
Sierra Mader (San Fernando)	87.5	54.4
Burnt MTN.	88.1	54.8
Eureka Peak	89.7	55.7
San Andreas – Coachella M-1 c-5	91.2	56.7
Newport –Inglewood (L.A. Basin)	92.4	57.4

TABLE E-1 Regional Faults		
Abbreviated Fault Name	**Approximate Distance Km.**	**Approximate Distance Mi.**
Pisgah – Bullion MTN. Mesquite LK	95.5	59.3
Newport –Inglewood (Offshore)	95.7	59.5
Northridge (E. Oak Ridge)	97.9	60.9

Regional Historical Earthquakes

The site is within seismically active southern California (Figures E-1 and E-2). In particular, the SAFZ has produced numerous historical 6.0Mw or greater earthquakes (Blake, 2004). For instance, the Fort Tejon earthquake (1857) of approximate magnitude 7.9 was centered near the intersection of the SAFZ and the Garlock Fault and is considered one of the strongest earthquakes ever recorded in the U.S.

Soil Profile Types

The underlying soil profiles are important variables used in typical ground acceleration attenuation formulae (Boore, et al., 1997). Usually the characteristics of the upper 30-m of the underlying soil/bedrock are estimated based on either Tables 3 and 4 from Boore, et al. (1997) or subjectively judged when employing the Campbell (1997, revised 1999) and Sadigh, et al. (1997) methodologies.

Based on review of the referenced reports, PSE judges that the preponderance of subsoil corresponds generally to the Boore, et al. site “hard rock” and would fit in the Campbell (1997, revised 1999) and Sadigh, et al. (1997) categories for each.

Probabilistic Peak Horizontal Ground Acceleration (PGA)

In recent years, particularly since 1998, the standard for seismic hazard (in this case PGA) assessment has increasingly become probabilistic-driven for both “normal” and for at least some “high-risk” structures. That is, the State of California, and the California Building Code of 2007 (CBC, 2007) have directed the industry toward or required probabilistic ground motion analyses. The rationale and basis for that direction is beyond the scope of this document; the reader is referred to the listed investigators.

Probabilistic methods of seismic risk determination attempt to account for uncertainties or likelihood in recurrence intervals, sizes, and locations of hypothetical earthquakes; and are increasingly being used for engineering analyses (Blake, 2004; Martin and Lew, 1999). Probabilistic analyses thus provide levels of hypothetical free-field ground acceleration for a finite exposure period. For example, a commonly accepted level of risk is the PGA with a 10-percent chance of being exceeded in 50 years. That PGA estimate is sufficient for most geotechnical or structural engineering analyses, including single-family residences. In general accord with the 2007 California Building Code (CBC) a maximum "Considered earthquake" should also be considered using a 2% probability of expedience in 50-years, particularly regarding critical structures. However, the CBC does not mandate the use of raw PGA for structural design, but rather recommends modified spectral accelerations and velocities (CBC, 2007)

One useful probabilistic method is FRISKSP that was derived from public domain USGS software by Blake (2004). Details of the mechanics for FRISKSP can be obtained from Blake, and are not recited herein. The fault inventory used to calculate hypothetical free-field probabilistic ground motions by FRISKSP is in essence the same as derived by the State of California for use in their seismic hazards mapping program and by Petersen, et al., (1996). FRISKSP selected 38 such faults within a 100-km radius (Table E-1). Three attenuation relationships, Boore, et al. (1997), Campbell (1997, revised 1999), and Sadigh, et al. (1997), were used to compute probabilistic horizontal free-field peak ground accelerations for rock and soft rock (Plates E-1 through E-3). These results were then averaged as shown in Table E-2.

Following the 1994 Northridge earthquake that occurred along a "blind thrust" fault, several investigators (for example, Abrahamson and Somerville, 1996; Somerville, et al., 1996) reported that peak horizontal accelerations generated by thrust faults are 20- to 30-percent higher than for strike-slip faults; and that in the case of dipping faults, the position of a particular site on either the hanging wall or footwall of a causative reverse fault played a greater role in increased ground acceleration than directivity. That is, in 1994, accelerations in alluvium were up to 50 percent greater on the hanging wall than what would have been predicted by using the many mean peak horizontal ground acceleration attenuation curves (Abrahamson and Somerville, 1996). To account for such, the attenuation relationships used for this assessment incorporate the differences in amplitude among reverse, thrust and strike-slip faults. In addition, the derived hypothetical accelerations represent the one standard deviation, which captures over a 50 percent increase in derived peak ground acceleration.

Table E-2 and E-3 presents the calculated horizontal ground accelerations representing the 10 and 2-percent chance of exceedance in 50- -years for hard rock and soft rock (taken from graphs on Plates E-1 through E-3).

TABLE E-2		
Investigators	**Hard rock**	
	10% in 50-years	**2% in 50-years**
Boore et al. (1997)	0.37g	0.43g
Campbell and Bozorgnia (1997 revised 1999)	0.54g	0.65g
Sadigh, et al. (1997)	0.66g	0.80g
Average	***0.52g***	***0.63g***

TABLE E-3		
Investigators	**Soft rock**	
	10% in 50-years	**2% in 50-years**
Boore et al. (1997)	0.47g	0.57g
Campbell and Bozorgnia (1997 revised 1999)	0.62g	0.73g
Sadigh, et al. (1997)	0.66g	0.80g
Average	***0.58g***	***0.70g***

It should be noted that these hypothetical numbers are based on recent standards of practice (for example, Martin and Lew, 1999, Petersen, et al., 1996; Cao, et al., 2003) and thus differ from numbers derived from past standards of practice. Owing to additional knowledge gained from recent earthquakes, as well as from recent and ongoing geological and seismological investigations, the sciences are expanding so rapidly that in some cases industry and government generated seismic guidelines can be obsolete after only a few years.

The maximum free-field PGA should not necessarily be used in empirical engineering formulas currently in use to determine earthquake-resistant engineering design. Page and others (1972) also noted that a single peak of intense motion (maximum or peak acceleration) might contribute less to cumulative damage potential than multiple cycles of less intense shaking. Further, the California Division of Mines and Geology (1997) cautions that the seismic coefficient "k" is not equivalent to peak ground acceleration, and that peak ground acceleration should not be used in pseudostatic slope stability analyses. Design of future improvements should be based on current design practices for similar works in the area. It is under the purview of the engineer, based upon information presented herein, to select suitable seismic parameters.

Closure

The PGA results are based upon many unavoidable geological and statistical uncertainties, yet are consistent with current standard-of-practice (Petersen, et al., 1996; Martin and Lew, 1999). As engineering seismology evolves, as more fault-specific geological data are gathered; and as legislative action continues, increased certainty and different methodologies may also evolve. Further, predictions of times of occurrence, locations and magnitudes of, as well as ground response to, future earthquakes are tenuous and subjective. Only probabilities and/or possibilities can be assessed on the basis of the existing geologic data, limited historical and seismic records, and empirical relationships among fault lengths, distances between the faults and the study site, and ground acceleration. However, enough seismic events of magnitude 6.0 or greater have occurred regionally to indicate that such events could recur within the life of the subject development.