CESPL-ED-GD (1110) 24 August 2012

MEMORANDUM FOR Aric Torreyson (Tetra Tech)

Subject: Geotechnical Recommendations, Robles High Flow Diversion DDR, Plans and Specifications (Contract W912PP-08-D-0009, Task Order CQ01)

The Corps of Engineers will be the Geotechnical Engineer of record for the subject project. The intent of this memorandum is to supplement the geotechnical report submitted by AMEC/Geomatrix ("Foundation Report for Robles Diversion Dam Modification Project," dated August 2, 2012). The Corps adopts the recommendations of that report except as stated herein.

1. Impervious Fill.
    a. The design should assume that the existing impervious fill/timber cutoff wall system is effective. While it is anticipated that the wall has deteriorated, there is no evidence that the structure was not built as designed; specifically, it is assumed that the impervious fill that was included in the original design is in place. The borings along the canal alignment indicate that there were ample supplies of sandy clay to clayey sand that would have functioned adequately as an impervious cutoff. Prior to advertisement, a limited investigation will be conducted to confirm the nature and extent of the fill.
    b. The specifications should require that the Contractor's method for removal of interfering portions of the timber wall, without compromising the integrity of the remaining fill, be presented in a submittal for Government approval.
    c. For the purpose of this submittal, the specifications should indicate that, if in the opinion of the Contracting Officer additional impervious fill is required, the Contractor shall provide a material which classifies as CL or CL-ML per ASTM D 2487. No borrow source will be designated. The soils shall be classified by a laboratory validated by the Materials Testing Center. The bid schedule should include a quantity estimate of material to be imported, noting that some may be salvaged from the required excavation.
    d. The drawings indicate that impervious fill is to be utilized under the fish ladder structure and under the structural keys. This value of this component is not evident and it is recommended that this detail be removed.

2. Weep holes.
    a. Grouted riprap. Weep holes in the grouted riprap are not recommended. Pressures will dissipate through natural occurring cracks in the grouted stone.
    b. Structural concrete. Dissipation of hydrostatic pressures under the spillway runout slab are necessary; a subdrain collector system is recommended. It is recommended that two collector pipes run parallel to the dam axis control line and drain through weep holes at the spillway walls. The precise location of the collector pipes is left to the discretion of the designer, though it is recommended that at least one be placed within 50 feet of the dam axis control line. The collector pipes should be constructed of perforated 4- to 6-inch schedule 40 PVC

and wrapped in a geotextile filter fabric. The pipes should centered in a sand bedding with a height and depth roughly equivalent to five pipe diameters. The gradation of the sand bedding should conform to the requirements of ASTM C 33 fine aggregate. Flap gates on the weep holes may be used at the designer's discretion. Clean-outs should be provided for both collector pipes.

3. Preparation for concrete slabs-on-grade. The recommendations in the AMEC report are acceptable. However, to clarify, the integrity of the impervious section is to remain intact underlying the spillway; no compacted granular fill is to be placed between the existing impervious fill and spillway or between the existing impervious fill and fish ladder. This consistent with the approach presented in the 60 percent plans.

4. Other specification-related comments will be provided during review of the pre-final submittal.

If there are any questions on the above or should you need further clarification on other issues, please contact me at (213)452-3587.

Douglas E. Chitwood, P.E., G.E.
Engineering Division

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FOUNDATION REPORT FOR ROBLES DIVERSION DAM MODIFICATION PROJECT

Submitted to:

Los Angeles District
U.S. Army Corps of Engineers
Los Angeles, California

Submitted by:

AMEC Geomatrix, Inc., Oakland, California

August 8, 2008

Project No. 9993.003.2

AMEC Geomatrix

DRAFT

August 8, 2008

Project 9993.003.2

Mr. Douglass Chitwood
Los Angeles District
U.S. Army Corps of Engineers
P.O. Box 532711
Los Angeles, California 90053-2325

Re: **Foundation Report for**
Robles Diversion Dam Modification Project
Ventura County, California

Dear Mr. Chitwood:

The enclosed report presents the results of a geotechnical study performed by AMEC Geomatrix for the Robles Diversion Dam Modification Project (project). Our study involved reviewing and summarizing existing investigations and available information, identifying data needs, performing a field reconnaissance of the site, and developing geotechnical recommendations and other considerations intended for the design of the project.

Geomatrix has appreciated this opportunity to work with you. Please contact the undersigned if you have any questions about this report or if Geomatrix can be of further service.

Sincerely yours,
AMEC GEOMATRIX, INC.

Michael L. Traubenik, GE
Principal Geotechnical Engineer

Faiz Makdisi, PE
Principal Engineer

mlt/fim/ldu
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Enclosure

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TABLE OF CONTENTS

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TABLE OF CONTENTS
(Continued)

FIGURES

APPENDIXES

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FOUNDATION REPORT FOR ROBLES DIVERSION DAM MODIFICATION PROJECT

Ventura County, California

1.0 INTRODUCTION

This report presents the results of a geotechnical study performed by AMEC Geomatrix, Inc. (AMEC Geomatrix) for Robles Diversion Dam Modification Project in Ventura County, California. The Robles Diversion Dam is located about 1.9 miles downstream of Mitilija dam (Figure 1); its purpose is to divert water from the Ventura River to the Robles-Casitas Diversion Canal.

According to construction drawings provided by the Los Angeles District, U.S. Army Corps of Engineers (USACE), the diversion dam was constructed in the late 1950s (Figure 2). The available drawings and specifications indicate that the diversion dam is a zoned earthfill and rockfill embankment (Figure 3) that was constructed using the various earth materials taken from the required excavations in the Ventura River bed and along the Robles-Casitas Diversion Canal, and other nearby borrow areas.

Based on observations made during a reconnaissance of the site conducted for this study (described later in this report) and a recent topographic map that was provided by the USACE, the diversion dam has been modified from its original plans. The available drawings indicate that original diversion dam had a height of between 5 and 10 feet above the original river bed, and had an original crest length of about 530 feet (Figures 2 and 3). A gated spillway that controls flows to the Ventura River and the Robles-Casitas Diversion Canal exist on the right abutment of the diversion dam, and the diversion dam's original left abutment consisted of a dike constructed of "compacted impervious embankment" fill. Based on measurements made during the site reconnaissance and the recent aerial and topographic map of the site (Figure 4), the crest length of the diversion dam has been shortened to about 350 feet by fill that has been added (or deposited) along the left abutment dike.

The spillway controlling flows to the Ventura River (i.e., service spillway) consists of a reinforced concrete structure that has three, 16-foot-wide by 9.5-foot-high radial/tainter gates, and one 10-foot-wide by 9.5-foot-high radial/tainter gate (Figure 3). Flow to the Robles-Casitas Diversion Canal is controlled by a reinforced concrete structure that has three, 11.5-foot-wide by 10.5-foot-high radial gates.

The overall Robles Diversion Dam Modification Project includes construction of a high flow bypass spillway, a stilling basin, a downstream rock ramp, and a technical spillway. The new facilities will be constructed adjacent to (and east of) the original Ventura River spillway structure (Figure 5). Cross sections through the spillways for two options are being considered by the USACE are shown on Figure 6.

The high flow bypass spillway will be constructed of reinforced concrete and will have four bays. Each bay will have tainter gates that are about 30-foot-wide by 12-foot-high. In addition, it is anticipated that the tainter gates to the existing spillway will be modified to retain an additional 2 feet of water. The remaining diversion dam embankment also will be raised and scour (overtopping) protection will be added.

This foundation report was prepared specifically for the design of the new high flow bypass spillway and stilling basin described above. Recommendations are not provided for the other structures/modifications that are being planned as part of the project.

1.1 PURPOSE AND SCOPE

The purpose of this study is to provide the geologic and the geotechnical information and recommendations needed for the design of the new high flow bypass spillway and stilling basin included in the overall Robles Diversion Dam Modification Project.

The scope of work consisted of the following tasks:

Task	Description
1	Review available information
2	Conduct a field reconnaissance
3	Perform engineering evaluations and develop recommendations for design
4	Prepare foundation report

The scope of work performed is described in the Amendment to Task Order 3 of Contract (W912PL-07-D-0004-0003) dated 3 August 2007.

1.2 PROJECT ORGANIZATION

The work described in this report was coordinated with the following individuals:

- Mr. Doug Chitwood - USACE
- ____________________

Key AMEC Geomatrix personnel who participated in this project include:

- Dr. Faiz Makdisi – Principal-in-Charge
- Mr. Michael L. Traubenik - Principal Geotechnical Engineer
- Mr. Timothy Keuscher – Principal Geotechnical Engineer
- Mr. Donald Wells, Senior Geologist

1.3 REPORT ORGANIZATION

The site of the planned facilities is described in Section 2. Observations made during the site reconnaissance are discussed in Section 3. Sections 4 and 5 summarize regional geologic and seismic setting, and site geology and subsurface conditions, respectively. Geotechnical recommendations and other considerations for the design of the new high flow bypass spillway and stilling basin are discussed in Sections 6. Finally, the basis for all the conclusions and recommendations presented in this report is provided in Section 7.

The appendices of this report are described below:

- **Appendix A – Annotated Photographs from Field Reconnaissance**
- **Appendix B - Logs of Borings and Test Pits from Previous Site Investigations**
  This appendix presents boring and test pit logs from previous investigations performed at the site by the USACE.

2.0 SITE AND DIVERSION DAM DESCRIPTION

The Robles Diversion Dam was constructed across the Ventura River about 1.9 miles downstream of Mitilija dam. In the discussions below, the vertical datum is based on NAVD 88 (ortho geoid 2003, U.S. Survey feet) and the site coordinates of North 1994080.95 and East 6173077.80 are based NAD (1983 U.S. Survey feet).

The diversion dam that currently exists spans about 350 feet across the river. According to the recent site topographic map provided by the USACE (Figure 4), the river channel is about 10 to 15 feet below its eastern and western banks. Bars of primarily coarse-grained material (gravel, cobbles and boulders) have formed near mid-channel both upstream and downstream of the diversion dam. At the time of the site reconnaissance conducted for this study (May 2008), a pool of water was present on the upstream side of the diversion dam. Downstream of the diversion dam, the main river channel is near the diversion dam's left abutment. Aerial photographs of the site show a pool of water just downstream of the service spillway (Figure

4). The Robles-Casitas Diversion Canal is up on the bank of the river channel on the right abutment of the diversion dam. Minor vegetation (bush and sparse grass) is growing in the river channel downstream of the dam. Private agricultural land is on the left river bank; no development is present on the right river bank.

As previously described, the available drawings and specifications indicate that the diversion dam is a zoned earthfill and rockfill embankment (Figure 3) that was constructed using the various earth materials taken from the required excavations for the diversion dam and along the Robles-Casitas Diversion Canal, and other nearby borrow areas. To help mitigate seepage beneath the diversion dam and spillways, a 15- to 20-foot deep trench (with sloping sidewalls) was to be excavated into the generally pervious deposits present in the river channel. The trench was to be backfilled with "compacted impervious backfill" (compacted to 95 percent of the maximum dry density determined by ASTM Designation D 698-42T) that was to be placed both upstream and downstream of a timber cutoff wall positioned in the center of the trench (Figure 3). The cutoff trench was to be constructed beneath the centerline of the diversion dam. At the spillways, the timber cutoff wall was to be positioned just upstream of the spillway concrete aprons. "As-built" drawings showing the actual construction of the diversion dam and cutoff trenches were not available for review.

3.0 SITE RECONAISSANCE

AMEC Geomatrix personnel and the USACE reconnoitered the Robles Diversion Dam on June 2, 2008 to evaluate the general conditions of the site, existing diversion earth dam, and concrete spillway structure. Our observations during the site visit are provided below.

At the time of our site reconnaissance, we observed a shallow pool of water on the upstream side of the diversion dam fed by the flowing Ventura River. Sparse to moderate amounts of vegetation were present in the pool of water upstream of the dam, on top of the crest and along the left bank. Surface soils observed during our site visit generally consisted of tan silty sand with large amounts of gravel, cobbles and boulders. Some of the boulders on top of the crest and downstream of the diversion dam were greater than 12 inches in diameter.

The diversion dam spans approximately 350 feet across the river, which is approximately 100 feet shorter than shown on plans provided by the USACE. As indicated by USACE personnel during the site visit, it appears the left bank of the river channel in the diversion dam area was filled in, which may explain why the existing diversion dam is shorter than indicated on the plans. The upstream face of the diversion dam has a slope gradient of approximately 3:1 to 4:1 (horizontal to vertical), and the downstream slope dips very gently such that it is difficult to determine the limits of the crest. The timber cutoff wall was exposed along most of on the northern edge of the crest and lines up with the spillway gates. The cutoff wall appeared to be approximately 6 inches wide with an approximately 10-inch wide timber cap. The top of the timber cutoff wall where exposed appeared to be in generally good condition. The left abutment consists of fill dike, and the right abutment consists of a gated concrete spillway and elaborate fish passageway.

Based on conversations with USACE representatives during the site visit, we understand that portions of the diversion dam were damaged during a large storm event in 1969. Reportedly during this incident, overtopping and scour eroded portions of the diversion dam near the left bank and damaged the cutoff wall. We further understand that while these structures were repaired, there are no repair records. Filling in the left bank of the river may have been part of those repairs. Lastly, we observed the conditions of the concrete spillway and found it to be in generally good condition with no notable distress or cracking.

4.0 GEOLOGIC AND SEISMIC SETTING

The Robles Diversion Dam is located on the Ventura River, approximately 1.6 miles downstream (south) of the confluence of the Matilija and the North Fork of the Matilija Rivers, and approximately 1.9 miles downstream from Matilija Dam (on the Matilija River). The region lies within the eastern Santa Ynez Mountains, which are part of the Western Transverse Ranges Province of Southern California. The Santa Ynez Mountains are a young east-west trending mountain range, composed of highly folded and faulted Cenozoic and late Mesozoic marine sedimentary rocks that have been deformed by slip on a series of generally east-west trending strike slip and reverse slip faults (Jennings and Strand, 1969). The diversion dam site lies near the southeast margin of the Santa Ynez Mountains, about 0.9 mile south of where the Ventura River emerges from a narrow canyon into a wider floodplain characterized by braided channels and extending to the Pacific Ocean (U.S. Army Corps of Engineers, 2004).

The geologic structure in the area surrounding the diversion dam site is characterized by a series of east-west trending, tightly folded anticlines and synclines, where the bedrock includes sandstone, siltstone, and shale of the late Eocene Cozy Dell Formation and Coldwater Formation, and the Oligocene Sespe Formation. The diversion dam site lies on the north limb of a syncline, where sandstone and siltstone beds within the Sespe Formation are overturned to dip steeply north. The Ventura River floodplain, upon which the diversion dam sits, is underlain by young unconsolidated fluvial terrace and channel deposits, including sand, gravel, and boulders overlying bedrock of the Sespe Formation (Figure 7; Dibblee, 1987; Tan and Jones, 2006).

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Major active faults in the region include the Santa Ynez fault, located about 3.1 miles north of the diversion dam, the San Cayetano fault, located about 7.5 miles east of the site, and the Mission Ridge-Arroyo Parida-Santa Ana fault, located about 2.5 miles south of the dam site. The San Andreas fault is located about 28 miles northeast of the site.

Major earthquakes that have resulted in strong ground shaking in the region include two magnitude 7.0+ earthquakes in 1812 (Santa Barbara Channel offshore) and 1927 (offshore of Lompoc), the 1857 M 7.9 earthquake on the San Andreas fault, and the 1952 M 7.3 earthquake on the White Wolf fault. These earthquakes are estimated to have caused ground shaking with peak ground accelerations (PGA) of up to about 0.2 g at the site (U.S. Army Corps of Engineers, 2004). Ten additional earthquakes of magnitude 5.5 to 6.0 have been recorded at distances of about 12.5 to 31 miles from the site. (http://redirect.conservation.ca.gov/cgs/rghm/quakes/historical/index.htm). Based on the historical seismicity, proximity, estimated maximum magnitude, and slip rate for major active faults near the site, the California Geological Survey indicates that the expected PGA for an earthquake return period of 475 years is about 0.56 g.

AMEC Geomatrix is currently performing site-specific probabilistic and deterministic seismic hazard analyses (PSHA and DSHA, respectively) and is developing design response spectra for the diversion dam site. Results of these studies are presented in a separate report (AMEC Geomatrix, 2008). The results of these studies show that the controlling deterministic source is the Santa Ynez fault, which is located about 5.2 km north of the project site. The fault trends generally east-west and dips steeply south; the expected sense of slip during potential earthquakes is dominantly strike slip, with a lesser component of reverse slip. The maximum credible earthquake (MCE) for the Santa Ynez fault is identified as a moment magnitude 7.2 earthquake. The median peak ground acceleration (PGA) at the site for the MCE, based on the average results from four Next General Attenuation (NGA) relationships (Atkinson and Boore, 2008; Boore and Atkinson, 2008; Campbell and Bozorgnia, 2008; Chiou and Youngs, 2008), and for an estimated shear wave velocity over the top 30 meters (V_{S30}) of 450 m/s, is equal to 0.50 g.

5.0 SITE SUBSURFACE CONDITIONS

As described above, the Ventura River channel has been mapped by Dibblee (1987) as containing stream channel deposits consisting of mostly gravel and sand (Figure 7); cobbles and boulders also are present in these deposits. The banks of the river have been mapped as alluvium (i.e., unconsolidated flood plain deposits of silt, sand and gravel).

Based on a construction drawing provided by the USACE, only one test pit (TP1) was excavated in the river channel near the centerline of the diversion dam prior to construction (refer to Appendix B). The test pit was excavated to a depth of about 17 feet. The test pit log indicates that mostly subrounded sandstone pebbles, cobbles and boulders were encountered. The boulders were reportedly hard and up to 3 feet in size. The pebbles, cobbles and boulders were overlain by a 2- to 3-foot-thick layer of topsoil that was described as silty and sandy, with a few pebbles and cobbles of sandstone.

Five test pits (i.e., TP-101 to TP-105) also were excavated across the river channel about 800 feet upstream of the diversion dam centerline. The test pits were excavated to depths ranging from about 18 to 33 feet. The test pit logs describe primarily gravel and sand deposits (refer to Appendix B). Boulders greater than 12 inches in size, reportedly comprising from about 3 percent to 25 percent of the total volume, were present in the gravel and sand deposits. Groundwater was encountered at depths ranging from 15 to 33 feet below the river channel at the time the pits were excavated (i.e., December 1954). Excavation of the pits was stopped when, or just after, groundwater was encountered.

Three additional test pits were excavated upstream and downstream of the existing diversion structure, indicated the presence of gravels boulders and sands, with a gravel and boulder content in excess of 65 percent.

Twenty borings also were drilled along the alignment of the Robles-Casitas Diversion Canal. One boring (i.e., boring DHC-1), drilled just downstream of the diversion canal spillway, reportedly encountered about 6½ feet of sandy and silty clay overlying sandstone boulders and cobbles in a clayey-sandy matrix. The boulders and cobbles were reported to a depth of about 20 feet (i.e., the total depth of the boring). Groundwater was not encountered in this boring.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS AND CONSIDERATIONS FOR STRUCTURES

This section presents the geotechnical engineering recommendations and considerations that apply to design of the new high flow bypass spillway and stilling basin described in Section 1. Recommendations are not provided for the other structures/modifications that are being planned as part of the project.

General spillway design considerations and recommendations are discussed first. Based on these general recommendations, specific recommendations are provided for the design of the spillway structures. When appropriate, earthwork and foundation design recommendations are presented separately.

The recommendations and other considerations presented in this report are intended for planning and design of the new high flow bypass spillway and stilling basin. This report may not provide all of the subsurface information that a contractor may need to construct the project. The recommendations presented herein were developed based on conceptual designs prepared by the USACE (refer to Figures 5 and 6).

6.1 General Considerations and Recommendations

The conceptual plans that were provided by the USACE indicate that the new spillways and stilling basins will require excavations that are as deep as 15 to 20 feet below from the top of the existing diversion dam crest (Figure 6). Based on the available construction drawings of the existing diversion dam and the conceptual plans provided by the USACE, if the spillways are constructed over the existing diversion dam centerline, the required excavations likely will remove virtually all of the rockfill comprising the upper portion of the diversion dam and likely will encounter the "compacted impervious backfill" that was placed as part of the diversion dam's cutoff trench (refer to Figures 2 and 3; the available construction drawings). Excavations also will likely encounter the generally coarse-grained alluvium that contains cobbles and boulders.

According to the available construction drawings, the required excavations also will encounter the existing timber cutoff wall along the centerline of the diversion dam. The new spillway structure designs probably will require an effective cutoff to mitigate high uplift forces and prevent excessive seepage from occurring beneath the spillways. If the condition and extent of the existing timber cutoff wall could be reasonably verified, it may be possible to incorporate the existing cutoff trench and wall into the design of the new spillways. However, based on information provided by the USACE, the diversion dam embankment was breached in floods that occurred in 1969, and "as-built" drawings of the limits of the damage and required repairs are not available. Therefore, the extent of the existing cutoff trench and condition of the timber cutoff wall is not known. The condition of the cutoff wall also is suspect after more than 50 years of service, and at the time this report was prepared, the design details for incorporating the existing cutoff wall into the new spillway structures had not been developed by the USACE. Such a detail may be a challenge to construct (e.g., excavating adjacent to the cutoff wall and cutting it cleanly where required). Based on these considerations, it is our opinion that it would be prudent to remove and reconstruct the cutoff trench and timber wall to ensure that the new spillways function as intended. This recommendation is made even though the bearing pressures from the new spillway structures probably will be less than the weight of the deposits that will be excavated during construction, and thus settlement of the soils comprising the cutoff trench and river channel alluvium would likely not be a concern. In summary, reconstructing the cutoff trench and cutoff wall potentially has the following advantages:

- Uncertainties that now exist regarding the condition and extent of the existing cutoff trench and timber cutoff wall would be eliminated.
- Potential construction difficulties that could arise from the use of the existing cutoff are eliminated.
- A new cutoff wall could be easily incorporated into the design of the new spillways.
- Uniform foundations conditions for the spillways could be "constructed".
- It is not known if the location and depth of the cut off trench and wall is adequate for the new improvements.

For the above reasons, the discussions and recommendations presented below assume the existing cutoff trench and wall will be removed and a new cutoff trench and wall will be constructed.

6.2 EARTHWORK

This section describes miscellaneous work necessary to prepare for construction of the new high flow bypass spillway and stilling basin. Excavation and groundwater conditions, fills and backfills, and drainage requirements also are discussed.

6.2.1 Clearing and Grubbing

All construction areas should be cleared of objectionable materials, including grass, weeds, concrete, gravel piles, old construction debris, and any other material that might interfere with the performance or completion of the work. All roots, buried logs, and other objectionable material should be grubbed. Old pipes, underground structures, debris, or waste should be removed if found anywhere on the site. Any holes created by the grubbing process below proposed structures and in areas to receive fill should be backfilled with compacted granular fill described in Section 6.2.6, Fill Material and Compaction Criteria. Excavations and trenches from abandoned utilities and pipelines that cross the footprints of new structures should be backfilled with the compacted impervious backfill described in Section 6.2.6, Fill material and Compaction Criteria. All objectionable material from clearing and grubbing should be removed from the site and properly disposed of.

Before the fill materials described in Section 6.2.7 are placed, the soil surface should be cleared and grubbed as described above. Where fill is to be placed, cobbles and boulders that lie on top of the ground surface and that are not surrounded by finer-grained soil should be removed. After the loose cobbles and boulders are removed, the exposed surface should be scarified or plowed thoroughly to a depth of 6 inches, thoroughly moistened, and then compacted by making at least 6 passes with compaction equipment weighing no less than 40,000 pounds. No fill material should be placed until a qualified engineering geologist or geotechnical engineer has reviewed the condition of the prepared surface upon which fill will be placed.

6.2.2 Excavation Conditions

Based on the available subsurface information, excavation of the earth materials comprising the existing diversion dam and the generally coarse-grained alluvial (containing cobbles and boulders) should be possible with heavy conventional earthmoving equipment and excavators. The alluvial deposits probably will be more consolidated (become denser) with depth; these deposits may be somewhat more difficult to excavate in the deeper excavations. These deposits also will likely become somewhat disturbed as they are being excavated and when the boulders are being "plucked" from excavation bottoms and sidewalls.

To avoid disturbing the bottom of the new cutoff trench and the spillway structure foundation subgrades, AMEC Geomatrix recommends that the contractor be required to exercise caution when excavating the final two feet of the foundation areas. Excavation equipment that can disturb excavation bottoms and structure subgrades (e.g., large dozers with rippers to loosen the earth) should be avoided when excavating to final subgrade. Final subgrade surfaces should be trimmed to minimize disturbance as much as possible.

In general, existing spillway structure foundations bearing on soils that lie above a line projected upward at an inclination of 45 degrees from the bottom of an adjacent excavation may require underpinning during construction or the excavation must be adequately supported. Should underpinning be necessary, we recommend that the contractor be responsible for its design and be required to submit an underpinning plan for review prior to construction.

6.2.3 Groundwater Conditions

Groundwater will be encountered during construction of the new cutoff trench and in other required excavations for the proposed spillway structures. Measures will be required to divert surface water flows of the Ventura River around the construction area. Measures to control groundwater also will be required. The combination of surface water and groundwater and the action of heavy earthmoving equipment will quickly disturb and degrade soil exposed in structure subgrades and the bottom of excavations. Wet or saturated deposits may cause difficulty during excavation, and equipment may get bogged down in the softer deposits. To minimize construction difficulties that typically occur during the winter rainy season or when groundwater is encountered, major earthwork operations should be planned for the normally dry summer and fall seasons, if possible.

The contractor should be made responsible for the design, construction, operation, maintenance, and removal of any system that is implemented to control the inflow of surface water and groundwater. The system should be designed to prevent migration and pumping of soil fines with discharge water. The contractor must plan the dewatering and excavation carefully so that stable and dry excavations are maintained throughout construction.

6.2.4 Temporary Slopes and Excavation Support Systems

The stability of the temporary excavation slopes made at the site will depend on the depth of the excavation, the strength and character of the soils exposed in the excavation, groundwater conditions, the construction schedule (i.e., the time the excavation or cut is allowed to stand open), and the contractor's operations and equipment, among other factors. For planning purposes and for preparing the engineer's construction cost estimates, temporary excavation slopes should be no steeper than 1½ (H):1(V). These temporary slopes apply for excavations that have a maximum depth of 15 feet. Flatter side slopes may be required (and should be anticipated) if the contractor intends to stockpile materials and/or use heavy equipment adjacent to the excavation. Flatter slopes also may be necessary if localized instability is observed during construction. Cut slopes exposed for extended periods likely will erode and/or ravel and require cleanup.

All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to all state and/or federal safety regulations and requirements. As is the case anywhere that excavations are made in soil, unexpected caving of excavations, temporary cut slopes, or trench walls could occur at any time or place. Workers in excavations and trenches must be adequately protected, by adequate means, at all times.

Excavations required for the construction of portions of the new cutoff trench and proposed spillway structures will be located near the existing spillway structures; some may be near pipelines or other improvements that must be protected. Excavations with inclined side slopes likely will be used during construction wherever possible. However, at some locations, sufficient room for sloped excavations may not exist and measures will be needed to support the adjacent ground and nearby existing facilities. Locations where such conditions exist and the structure excavations that could require ground support should be identified during design. Construction costs associated with ground support systems are sometimes underestimated when project-specific requirements are not identified. It should be noted that installing sheet piles to required depths in the coarse-grained alluvial soils (that contain cobbles and boulders) may be extremely difficult or not possible.

Excavations having vertical sidewalls deeper than 5 feet will require sheeting, shoring, or other effective means to adequately support the ground and to protect workers. Excavations shallower than 5 feet may require support depending on the location of the excavation, the anticipated soil and groundwater conditions, and/or the contractor's activities in the vicinity of the excavation. The stability of excavation walls and slopes will need to be evaluated during excavation. As is the case with any excavation in soils, unexpected caving of excavation walls and slopes could occur at any time or place, regardless of the depth. It should be noted that the boulders present in the alluvium may be "plucked" from excavation sidewalls causing overhangs and voids, and the excavation sidewalls to become disturbed. Such conditions will be subject to sloughing and caving. Project specifications should place full responsibility on the contractor for planning, design, construction, maintenance, and removal of excavation support systems.

Ground movement/settlement must be prevented to avoid damaging the nearby spillway structures and other improvements. All excavations should be adequately braced to prevent failure of the excavation walls and to mitigate potentially damaging ground movement/settlement. The ground support system should be installed without leaving the nearby spillways and other improvements unsupported. To help mitigate ground movement/settlement, stockpiling earth and other construction materials near open excavations should be avoided. In no case should stockpiling occur closer to excavations than federal or state regulatory agencies allow.

If removal of the support system might cause an excavation wall to collapse, the support system should be left in place. Locations where excavations may be subject to caving should be identified as the excavations are being made. Soils that tend to ravel and cave while being excavated probably will cave if the support system is removed while the excavation is being backfilled. If pressure-treated wood is used, it should be left in place and cut off about 2 feet below the ground surface. Wood that is subject to rotting should not be used.

6.2.5 Permanent Slopes

Permanent cut slopes in soil and fill slopes should be no steeper than 2(H):1(V). Where possible, flatter permanent slopes should be used to blend the final ground surface into the adjacent ground contours. All exposed ground surfaces and cut and fill slopes will be subject to water erosion and local raveling if not adequately protected. All permanent cut and fill surfaces should have erosion protection measures as soon as the final grades or cut and fill slopes are created.

All permanent fill slopes should be overbuilt by at least 1 to 2 feet and then cut to final grade to provide adequate compaction. As previously described, permanent fill slopes should be no steeper than 2(H):1(V).

6.2.6 Preparation for Concrete Slabs-On-Grade

Considering the character and nature of the coarse-grained alluvial soils, we recommend that all new spillway structures and slabs constructed for the project be founded on a pad of compacted granular material, such as the aggregate base material described in Section 6.2.7 below. The purposes of the pads are to: (1) provide a uniform bearing surface for the completed structure or slab; (2) provide a reasonable working surface for equipment (small cranes, concrete trucks, etc.) during construction; (3) create a smooth surface upon which to position concrete reinforcement for footings and slabs; and (4) provide drainage, if required.

Building pads under the new spillway structures and slabs should be at least 6 inches thick and should extend at least 1 foot beyond the outer edge of the slab or footing of the structure. All excavation bottoms should be cleaned of all debris and loose soil, cobbles and boulders before the pad for any structure is constructed.

Slabs for minor surface structures and equipment should also be placed on a 6-inch-thick pad of compacted granular material (e.g., crushed rock, permeable material, or aggregate base material) described in Section 6.2.7 below). Slabs should not be placed directly on the native soils. Before the granular material is placed on soil subgrades, the soil surface should be cleared and grubbed as previously described in Section 6.2.1. Cobbles and boulders that lie on top of the ground surface and that are not surrounded by finer-grained soil should be removed. After the loose cobbles and boulders are removed, the exposed surface should be scarified or plowed thoroughly to a depth of 6 inches, thoroughly moistened, and then compacted by making at least 6 passes with compaction equipment weighing no less than 40,000 pounds. Compacted surfaces should be regular and free of debris.

If a slab-on-grade is to be damp-proofed, it should be placed on 6 inches of free-draining crushed rock described in Section 6.2.7.

6.2.7 Fill Materials and Compaction Criteria

It is anticipated that five principal fill types could be used to construct the project. These are (from coarsest to finest):

1. crushed rock
2. permeable material
3. aggregate base material
4. impervious site fill
5. Controlled Low Strength Material.

Each type of material is described in the following text according to its (a) potential source, (b) uses, (c) typical specifications, (d) compaction requirements, and (e) special handling/processing requirements (if applicable).

It should be noted that the relative compaction requirements discussed below are based on the *maximum dry density* and optimum moisture content of the subject material as determined by ASTM Method D1557 (latest edition). When the *relative density* is discussed in the text, it is based on ASTM Methods D4253 and D4254 (latest edition).

Crushed Rock

Crushed rock should be an imported material or processed native alluvium that consists of durable rock and gravel that is free of deleterious and potentially hazardous material and substances, and free from slaking or decomposition under the action of alternate wetting and drying. This material may be used to construct drainage trenches (if required), or may be used to construct wall drains and building pads for the proposed spillway structures, or may be placed on the bottoms of excavations and trenches excavated in wet, unstable ground. Crushed rock should meet the following gradation requirements.

Standard Sieve Size	Percentage Passing
1 inch	100
¾ inch	90-100
No. 4	0-10
No. 200	0-2

These materials should have a durability index of not less than 40. If there is a concern that fines from the subgrade could migrate to the voids of the crushed rock, the crushed rock can be placed on, or surrounded by, a suitable geotextile fabric.

Crushed rock should be moistened thoroughly and compacted to a relative density of at least 75 percent using suitable plate- or roller-type vibratory compaction equipment.

Permeable Material

Permeable material should be an imported material that consists of durable crushed rock or gravel and sand that is free from slaking and decomposition under the action of alternate wetting and drying. Permeable material may be used for wall drains and/or subsurface trench drains. It also may be used beneath the slabs of the spillway structures if a permanent drain is required.

The material should have a durability index of not less than 40 and a sand equivalent value of not less than 75. Complete specifications for this material, which is commonly referred to as Class 2 Permeable Material, are given in the State of California, Department of Transportation (Caltrans) Standard Specifications, Section 68.

Permeable material should be moistened thoroughly and compacted to a relative density of at least 75 percent using plate- or roller-type vibratory compaction equipment.

Permeable material used behind retaining and other structural walls should have a thickness of not less than 12 inches. It should be placed against the wall at least 1 foot higher than the adjacent backfill to prevent contamination and should be continuous with any foundation drain system. A 2-foot-thick cap of relatively impervious fill should be placed over the permeable material at the top of the backfill to prevent infiltration of surface runoff.

Aggregate Base

Imported aggregate base material may be used to construct project access roads and the building pads for the proposed spillway structures, and for use as fill and backfill beneath and adjacent to structures for which settlement of the backfill must be minimized. This material should meet the requirements in the Caltrans Standard Specifications, Section 26, Class 2 Aggregate Base (¾-inch maximum particle size), except recycled materials should not be allowed. For a relatively impervious material, aggregate base having at least 10 percent fines (i.e., particles passing the No. 200 sieve) should be specified. Aggregate base material placed beneath structures should be compacted to no less than 95 percent of maximum dry density. The moisture content of the material should be within -1 percent and +3 percent of optimum, and the material should be placed in horizontal lifts that do not exceed 8 inches before being compacted.

Impervious Site Fill

Impervious site fill will be needed to construct the new cutoff trench and to backfill the new spillway structures. Based on the specifications for the construction of the existing diversion dam, impervious site fill was to consist of "...selected material, containing no stones larger than 3 inches in diameter,..." and "... a mixture of earth materials containing an excess of clays and silts suitable for providing an impermeable backfill when compacted." While the excavation to remove the existing cutoff trench could be the source of the impervious site fill, it is likely that an adequate quantity of imperious site fill will not be available from the existing cutoff trench and a suitable borrow source will be required for the new construction. Based on the available subsurface information and our understanding of the site's geologic conditions, earth materials suitable for impervious site fill should be present along the banks of the Ventura River.

During construction, careful monitoring and testing of the site fill will be essential to mitigate potentially damaging ground settlements. To mitigate ground settlement, fill derived from the site soils must be thoroughly processed and moisture conditioned prior to placement and compaction, as described in this section, or should not be used. As described above, imported aggregate base may be used as fill and backfill where settlement must be minimized. Aggregate base may be easier to compact and test than fill derived from the site soils.

Fill generated from a borrow source located along the banks of the Ventura River will likely consist of finer-grained clayey and sandy soils. These soils may require screening/processing to remove oversized particles of soil, cobbles and boulders. The processed native soils used as impervious site fill should have the following properties or characteristics:

- All fill particles should be less than 3 inches in size.
- Less than 30 percent of the material should be retained on the ¾-inch sieve.
- No less than 20 percent and no more than 50 percent of the material should pass the No. 200 sieve.
- The fines (i.e., material passing the No. 40 sieve) should have a plasticity index (PI) no greater than 15.

- The fill material should contain less than ½ percent by weight of organics and should be free of other deleterious, potentially hazardous, and objectionable material (e.g., concrete, plastic, and other wastes).

Proper compaction of the impervious site fill will depend on the fill moisture content at the time of compaction. None of the exposed soil surfaces should be allowed to dry out or become wet during or after placement. If the material is too dry, then it should be over excavated, moisture conditioned and recompacted. If it becomes wet, impervious site fill derived from the native clayey soils may soften; its surface may become slick. Placing and compacting impervious site fill material should be avoided during the winter rainy season when it may be difficult to control the moisture content of the fill.

The soils to be used as impervious site fill will likely be heterogeneous, and therefore, will require mixing, blending, and moisture conditioning to create a material that can be placed and adequately compacted. All fill should be scarified, plowed, disked, and/or bladed until it is uniform in consistency and free of large, unbroken chunks or clods of soil. Chunks and clods of soil and cobbles and boulders having any dimension greater than 3 inches either should be broken down by heavy earthmoving equipment (or other effective methods) or should be removed from the fill while the fill is placed. The moisture content of the mixed fill should be adjusted to between optimum and +3 percent of the optimum moisture content. Additional disking or blading may be necessary to obtain uniform gradation and moisture content.

Impervious site fill should be placed on the prepared surface in horizontal lifts that do not exceed 8 inches in thickness before compaction. The fill should be compacted with suitable equipment to no less than 93 percent of maximum dry density. The final surface of the compacted fill should be graded to promote good surface drainage.

When new fill is to be placed and compacted against existing fill and native slopes, the existing fill should be benched horizontally so that the new fill will be incorporated into the existing fill slope. To provide a firm foundation free of loose or disturbed material, a minimum of 2 feet normal to the existing fill slope should be removed and recompacted while the new fill is brought up in layers. Existing fill material cut in this manner should be recompacted along with the new fill material.

Controlled Low Strength Material

Controlled low strength material (CLSM) which is referred to in Section 19 of the Caltrans Standard Specifications (July 1999) as "Slurry Cement Backfill," should be considered as an alternative trench backfill material, especially where high strength and/or low permeability is required. CLSM consists of a fluid, workable mixture of aggregate, Portland cement, fly ash, and water. The use of CLSM has the advantages that a narrower trench can be used, thereby minimizing the quantity of soil to be excavated, and CLSM can be batched to flow into irregularities in the bottoms and walls of trenches. The Caltrans specification for the gradation of CLSM aggregate is:

Standard Sieve Size	Percentage Passing
1-½ inch	100
1 inch	80-100
¾ inch	60-100
3/8 inch	50-100
No. 4	40-80
No. 100	10-40

More restrictive gradation requirements may be desirable to limit the fines content and the size of the sand and gravel. AMEC Geomatrix recommends that (1) no more than 25 percent of the aggregate particles pass through the No. 200 sieve; and (2) the 28-day compressive strength of the CLSM be no less than 100 pounds per square inch (psi) and no more than 150 psi. If native soils are used for the CLSM aggregates, trial mixtures will be necessary to confirm the quality and properties of the resulting CLSM.

6.3 Foundation Recommendations

This section provides recommendations for shallow foundations (mat, spread- and strip-type footings) that may be used to support the spillway structures and for the consideration of uplift forces.

6.3.1 Shallow Foundations

AMEC Geomatrix understands that the proposed spillway structures will impart relatively light loads onto the underlying foundation soils. If the recommendations presented in this report are employed in the design and construction, the proposed spillway structures will be built on a building pad constructed at or near final grade. As previously described, the building pad should consist of compacted granular fill material (e.g., aggregate base material). Depending on the configuration of the new spillway structures and the cutoff trench, the spillway structure may overlie compacted impervious fill that is derived from deposits borrowed along the Ventura River.

Spillway structures constructed on the conditions described above can be supported on shallow foundations. Shallow foundations (e.g. mat, spread- and strip-type footings) for the proposed structures situated on a building pad constructed above the cutoff trench (i.e., impervious site fill) should be designed using allowable bearing pressures of 3,000 pounds per square foot (psf) for dead load (DL) and 4,000 psf for DL and live loads (LL). Foundations bearing on a building pad constructed on coarse-grained sand and gravel alluvial deposits should be designed using an allowable bearing pressure of 4,000 psf (DL) and 6,000 psf (DL + LL). Spread and strip-type footings should be a minimum of 2 feet wide and should extend at least 2 feet below adjacent grade. The allowable bearing pressures above assume that the structures will be placed on properly prepared subgrade as discussed in Section 6.2.6 and may be increased by one-third when considering seismic or other transient loads.

All footing excavations and bearing surfaces should be observed by a registered geotechnical engineer prior to placing the granular fill building pad, reinforcing steel and concrete. If soft or weak materials are encountered at the bearing elevations, the unsuitable materials should be excavated down to firm bearing materials and backfilled with compacted aggregate base.

It is anticipated that settlement of new spillway structures will be less than 1-inch under the maximum anticipated loads following construction. Most of the settlement is expected to be immediate. Variations in water levels retained by the structures may induce 1-inch of elastic rebound and settlements during operations.

Lateral loads imposed by the water retained behind the spillway structures or by an earthquake will be resisted by the passive resistance of the adjacent soil/fill acting on the sides of the footings and buried walls and by sliding frictional forces. Assuming an allowable wall/footing deflection, the passive soil resistance recommended for design should be calculated using the passive lateral earth presence distribution shown in Figure 8 and the chart presented in Figure 9. The diagram and chart shown on Figures 8 and 9, respectively, are for the impervious fill material described above. A coefficient of sliding resistance of $\mu = 0.45$ should be used when a footing is poured neat on the building pad that is constructed on impervious site fill. For footings poured neat on a building pad constructed on the sand and gravel alluvial deposits, a coefficient of sliding resistance of 0.55 should be used. These values assume no factor of safety (i.e., a factor of safety equal to 1.0).

6.3.2 Uplift Forces on Structures

The proposed spillway structures may be subjected to uplift forces that will depend on: (1) the position of the cutoff trench and cutoff wall beneath the spillway, (2) the seepage conditions that develop beneath the spillway, and (3) the planned operation of the spillways. Uplift pressures should be evaluated after the conceptual designs of the spillway structures are further developed. Uplift forces acting on the spillway structures due to hydrostatic conditions will have to be resisted by: (1) the weight of the structure, and (2) the total and buoyant weights of the soil acting on the extension of the foundation slabs/mats (if the extensions are incorporated into design). For preliminary design of the spillways, the effective weight of soil acting on an extension of the foundation should be 130 pcf and 65 pcf for soils above and below the assumed planned operational water level(s), respectively. The effective weight of the soil should be verified after the soil that will be used to backfill the spillway structures is identified. The frictional resistance of the soil on the sidewalls of the spillway structures should be neglected when evaluating uplift resistance.

Uplift forces must be controlled while the spillway structures are being constructed. The method used to control or resist uplift forces during construction should be chosen by the contractor.

6.4 Retaining Walls

Lateral earth pressures recommended for the design of the spillway retaining walls and the buried walls of the stilling basin structures are presented on Figure 8. The pressure distributions shown on Figure 8 are for walls that are backfilled with impervious site fill material described above. The walls of the spillway walls and basins should be designed to meet nonyielding (at rest) conditions, because it is likely that the tops of the walls cannot deflect (or be allowed to deflect) to develop active wall conditions.

The nonyielding wall pressure distribution shown on Figure 8 assumes that no permanent surcharge loads are applied adjacent to the retaining wall or buried structure. Such loads may be produced by other structures, by heavy equipment, or by storing/stockpiling materials during construction. If such loads are anticipated, the design must account for additional pressures. For example, if material is stockpiled adjacent to a spillway basin, a uniform surcharge load will produce an additional lateral uniform wall pressure equal to 0.40 times the anticipated surcharge load. Spread- or strip-type footings and slabs that may be constructed adjacent to a spillway retaining wall also will produce a load on the wall that must be considered in design. Walls that fall within a zone of influence defined by an imaginary line drawn from the bottom edge of the footing or slab downward at an angle of 45 degrees should be designed to accommodate the load on the footing. Transient loads produced, for example, by trucks, need not be considered in the design, unless they produce lateral pressures that exceed the pressures produced under earthquake loading conditions.

Retaining walls for sloping (upward or downward) backfill conditions must be designed using earth pressures different from those for level ground conditions (Figure 8). If the wall is backfilled with impervious fill, the slope of the backfill need not be considered when the toe of an upward slope is at a distance greater than about 1.5 times the retaining wall height. If sloping backfill conditions are required behind retaining walls, AMEC Geomatrix will provide lateral earth pressures appropriate for design.

Where settlement of wall backfill must be kept to a minimum (e.g., in an area that will be paved or where a "step" in the structure occurs), backfill placed adjacent to the retaining wall should consist of the aggregate base material or CLSM described in Section 6.2.6, Fill Materials and Compaction Criteria. If properly moisture conditioned and placed in loose lifts less than 8 inches thick, the aggregate base material should be compacted well using hand-held mechanical equipment and settlement of the aggregate base will be minimal.

If settlement of the wall backfill need not be limited, impervious fill derived from the on-site borrow excavations may be used. Compared to the aggregate base backfill, this fill may be more difficult to compact, especially when using hand-held equipment.

Backfill placed adjacent to retaining walls should be compacted to at least 90 percent, but no more than 92 percent, of maximum dry density because over-compaction could cause excessive stresses on the walls.

6.4.1 Drainage Requirements

Retaining walls that are not designed to resist hydrostatic pressures should be provided with drainage systems to prevent the buildup of hydrostatic pressures. The drainage system should consist of granular backfill and a 4-inch-diameter (minimum) perforated subdrain pipe. The granular backfill may consist of either crushed rock surrounded by a geotextile or permeable material. Weep holes may be used for retaining walls, if desired.

6.5 SLABS ON GRADE

Slabs for minor surface structures and equipment should be placed on prepared subgrade as described in Section 6.2.6.

For the design of slabs on grade, a modulus of subgrade reaction (K_{v1}) of 75 to 100 tons per cubic foot may be used for slabs that bear on impervious site fill and 150 to 200 for slabs that bear on alluvial deposits of primarily sand and gravel. These subgrade modulus values are for a one-foot wide, square plate acting on a uniform subgrade. Appropriate conversion relationships should be applied to the subgrade modulus to account for the actual dimensions of the slab being designed. The modulus of subgrade reaction (K_b) for a foundation of width b should be determined from the following formula:

$$K_b = K_{v1}\left(\frac{b+1}{2b}\right)^2$$

6.6 SEISMIC CONSIDERATIONS

A discussion of the seismic considerations is presented in this section, including the seismic design criteria and the potential for ground settlement and soil liquefaction caused by earthquake shaking. A discussion of the significant seismic sources and the estimated peak ground motions at the site are presented in Section 4.0.

6.6.1 Earthquake-Induced Lateral Wall Pressures

During an earthquake, additional lateral loads will be applied to the walls of all buried structures and to retaining walls. The seismic lateral earth pressure is approximately proportional to the peak ground surface acceleration. The seismic lateral earth pressure increment was evaluated using ground motion criteria described above. The increment, equal to 20H, is a uniform pressure distribution in pounds per square foot (psf) acting on the full height of the wall (H). This pressure distribution applies to walls designed for both active and at rest conditions. If other earthquake ground motion criteria are used to design the facilities, a different seismic lateral earth pressure may apply. Additional recommendations will be provided upon request.

6.6.2 Earthquake-Induced Ground Settlement and Liquefaction

According the seismic hazard maps for Southern California, the Robles Diversion Dam site is located within a zone of potential liquefaction. Given that median deterministic ground motions at the site are of the order of about 0.5 g, the saturate fine-grained cohesionless soils at the site may be susceptible to liquefaction. Because of the apparently dense granular soils encountered at the site, and the reported high percentage of gravels and boulders observed in the excavated test pits in the vicinity of the site, the hazard posed by densification or lateral spreading of the site soils caused by earthquake shaking is judged to be low. It should be noted that if the existing cut-off trench and timber wall were excavated and reconstructed prior to construction of the new spillway structures (as recommended in Section 6.1 above), the potential for liquefaction of the near-surface fine cohesionless soils will be further mitigated.

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7.0 RECOMMENDATIONS FOR FUTURE WORK

The location and depth of the cutoff trench/wall for the improvements could have significant impact on the long term performance of those improvements. It is not known whether seepage analyses were performed to determine the location and depth of the existing cutoff trench/wall. Consequently, we recommend seepage analyses be performed to evaluate the optimum position and depth of the cutoff trench/wall for the improvements. As part of these analyses, seepage conditions that are likely to develop during operation of the structures should be evaluated so that uplift forces can be reasonably estimated and incorporated into the project's design.

During final design, it would be prudent to further evaluate and characterize subsurface conditions underlying the proposed spillway and stilling basin structures, particularly if the existing cutoff trench and wall are incorporated into the design of the new structures. Possible borrow sources for the impervious site fill and other earth materials needed for construction of the project also should be identified and evaluated, and the properties of these materials should be estimated through laboratory testing. If additional information is collected or developed for the project, the recommendations presented in this report should be reviewed and modified, if necessary; supplemental recommendations may be prepared.

8.0 BASIS FOR RECOMMENDATIONS

This report was prepared for the exclusive use of USACE, the designers of the Robles Diversion Dam Project. The recommendations and other considerations presented in this report are intended for the planning and design of the new high flow bypass spillway and stilling basin included in the overall Robles Diversion Dam Modification Project described in Section 1.0. The recommendations were developed using subsurface information available for the site and our understanding of the site's geologic conditions. Additional field exploration work was not performed at the site. The recommendations are based on the assumption that soil conditions at the new high flow bypass spillway and stilling basin do not deviate appreciably from those described herein.

During construction of the project, if any variations or undesirable conditions are encountered during construction, AMEC Geomatrix should evaluate the effects these conditions may have on our recommendations and, if necessary, develop supplemental recommendations. Recommendations are made for the specific project described in this report. Changes in design of the structures should be evaluated by AMEC Geomatrix for their effects on these recommendations.

An AMEC Geomatrix representative should observe earthwork and foundation construction to confirm that subsurface conditions encountered during construction are comparable to those used for developing the recommendations presented in this report. Unanticipated subsurface conditions, which cannot be disclosed fully by completing field exploration work, commonly are encountered and frequently require additional expenditures to attain a properly constructed project. Some contingency funding is recommended in case conditions encountered during construction require additional exploration, testing, or design modifications.

In the performance of our professional services, AMEC Geomatrix, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. This report may not provide all of the subsurface information that may be needed by a contractor to construct the project. No warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings. We are responsible for the conclusions and recommendations contained in this report, which are based on data related only to the specific project and locations discussed herein. In the event conclusions or recommendations based on these data are made by others, such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review and concur with such conclusions or recommendations in writing.

9.0 REFERENCES

Dibblee, T.W., Jr., 1987, Geologic map of the Matilija Quadrangle, Ventura County, California: Dibblee Geological Foundation Map DF-12, scale 1:24,000.

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Tan, S.S., and Jones, T.A., 2006, Geologic Map of the Matilija 7.5' Quadrangle, Ventura County, California: A Digital Database, Version 1.0: California Geological Survey, Preliminary Release of Digital Geologic Map, scale 1:24,000.

U.S. Army Corps of Engineers, Los Angeles District, 2004, Draft Environmental Impact Statement/Environmental Impact Report for the Matilija Dam Ecosystem Restoration Project, July

FIGURES

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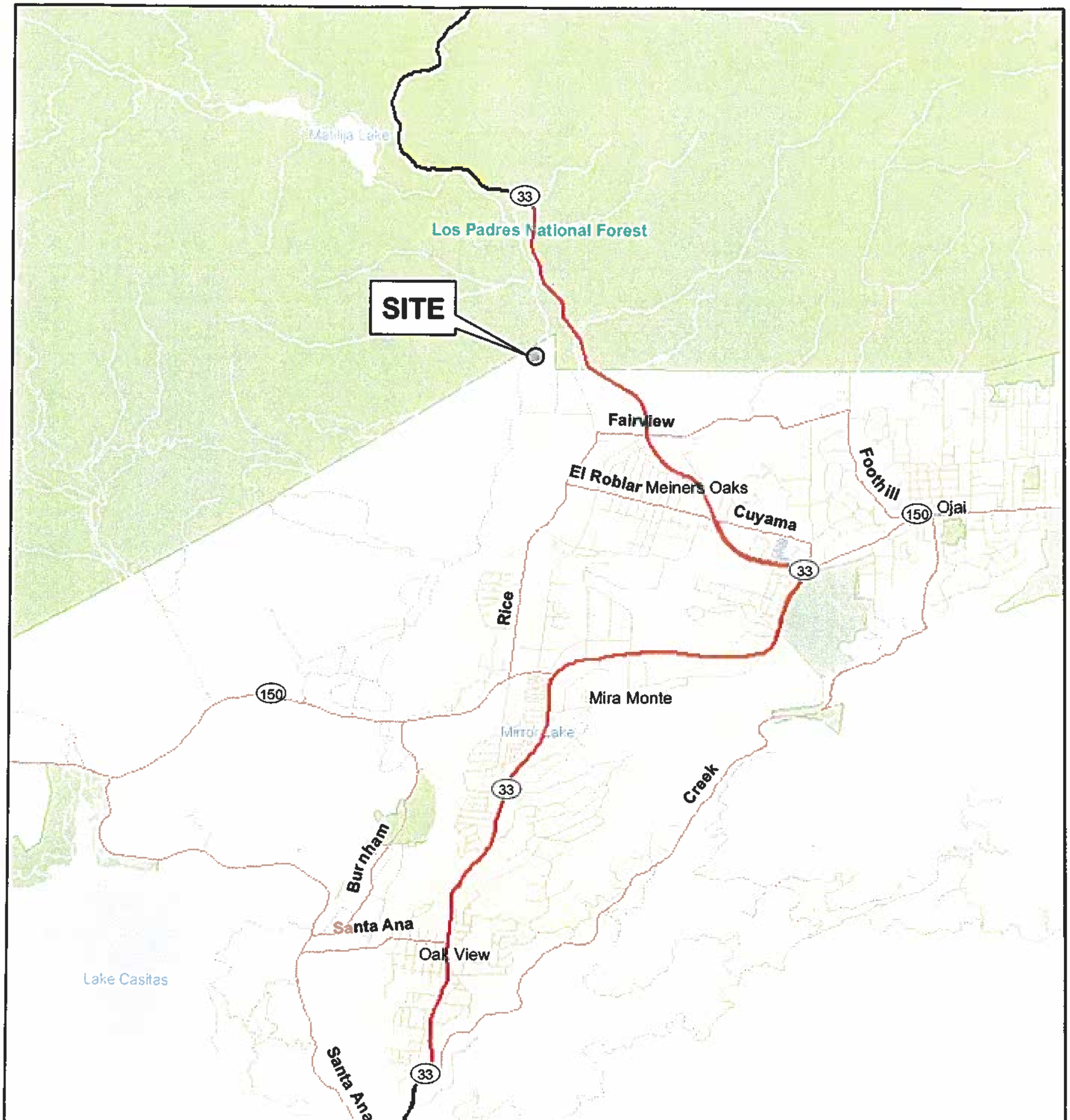

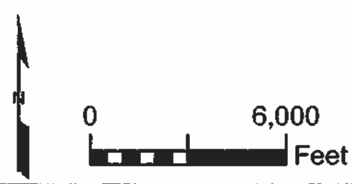

SITE LOCATION MAP
Robles Diversion Dam
Ventura County, California

By:	Date: 8/8/2008	Project No. 9993.003
AMEC Geomatrix		Figure 1

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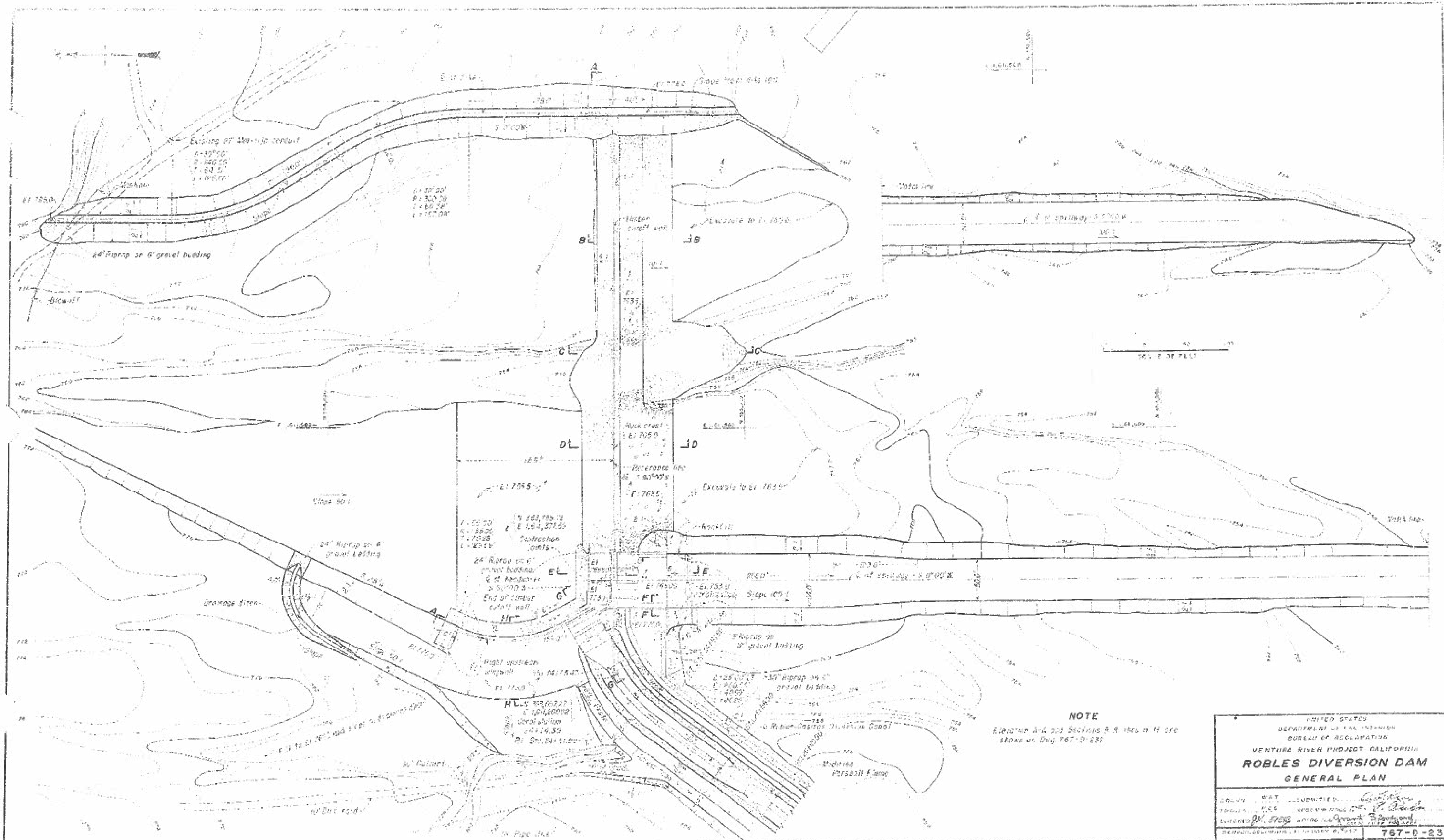

Figure 2

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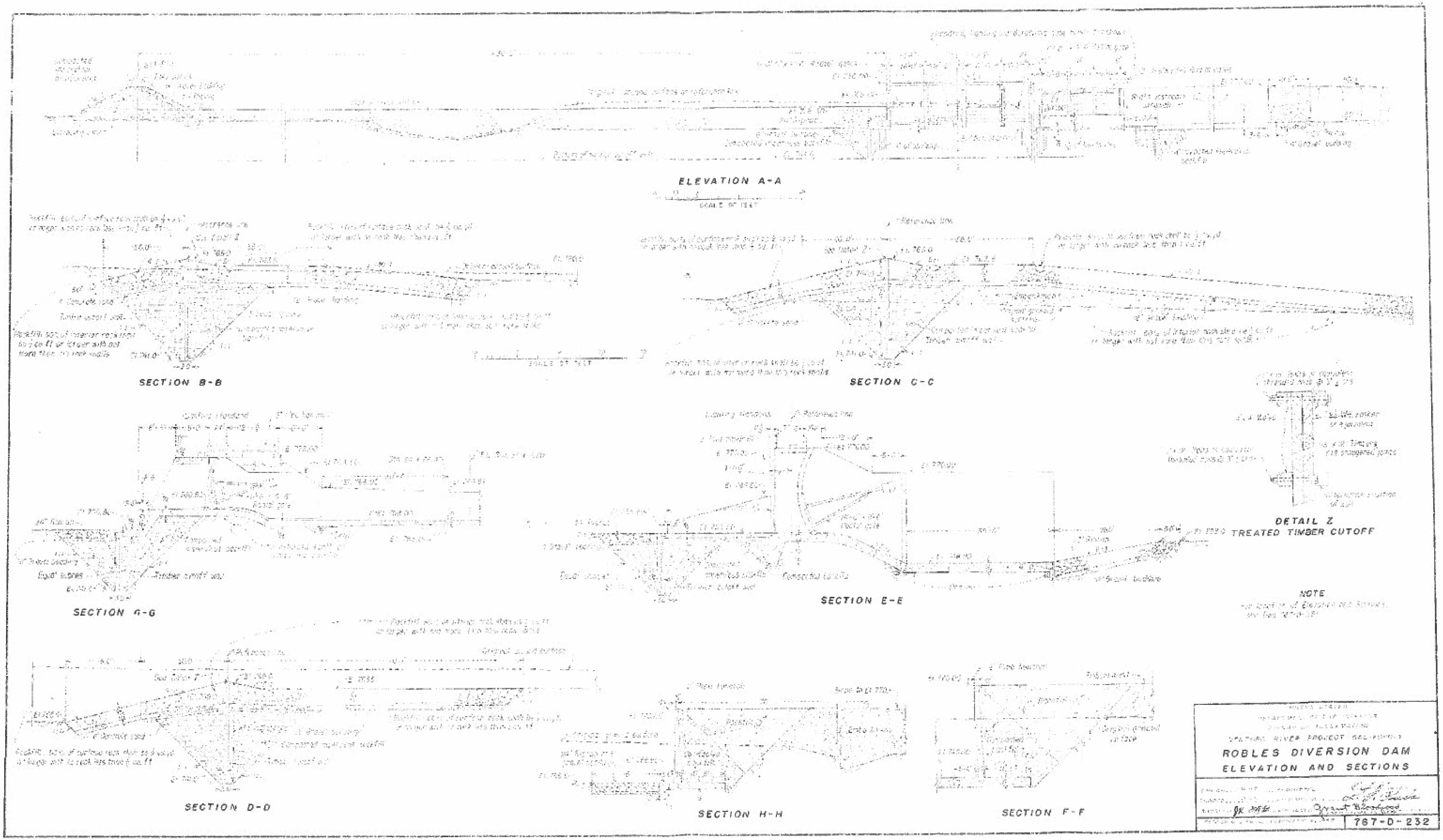

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Figure 3

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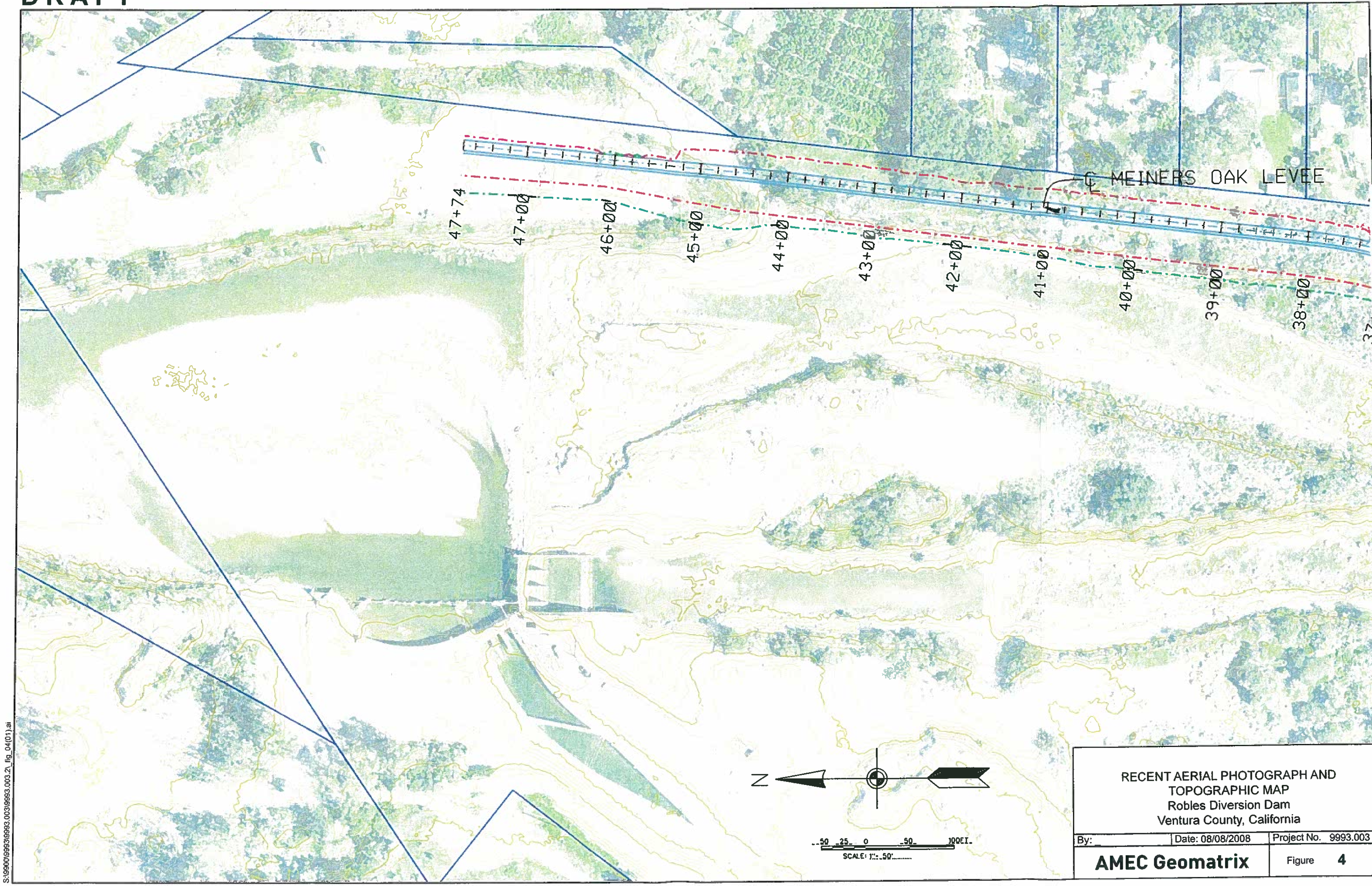

RECENT AERIAL PHOTOGRAPH AND TOPOGRAPHIC MAP
Robles Diversion Dam
Ventura County, California

By:	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure 4

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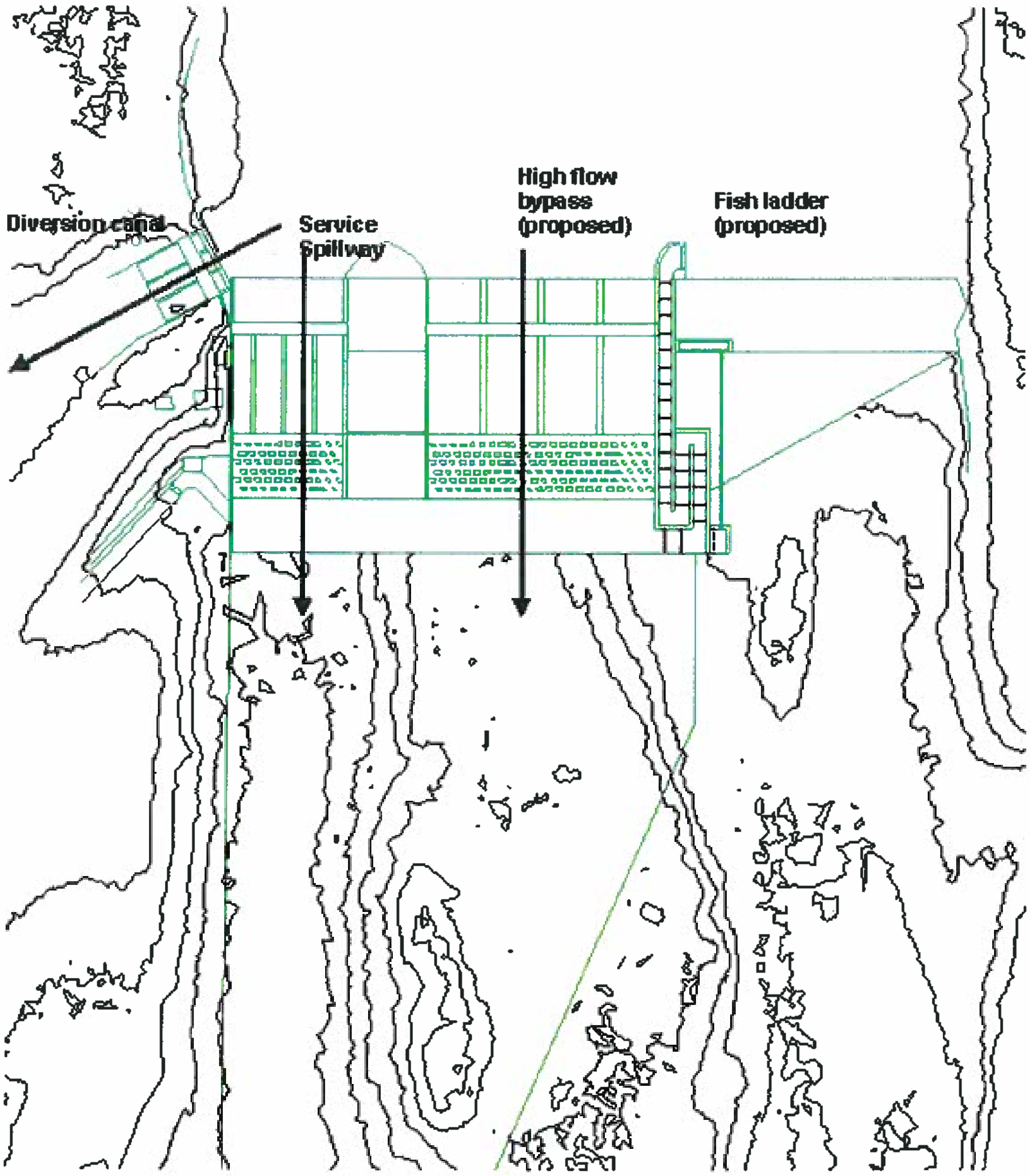

PLAN OF PROPOSED SPILLWAY MODIFICATIONS
Robles Diversion Dam
Ventura County, California

By:	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure 5

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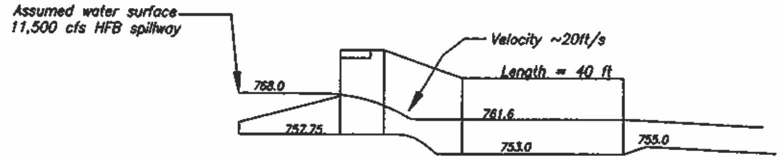
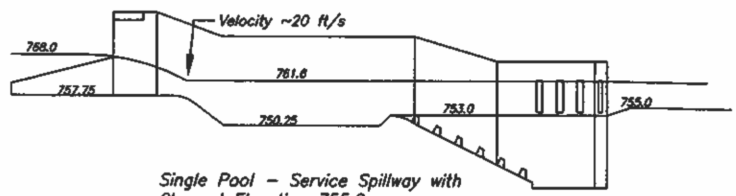
Option A

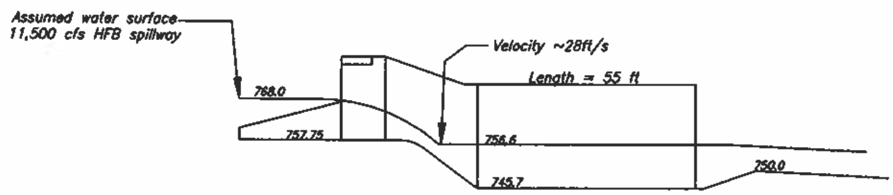


Single Pool – Downstream Channel Elev. 755.0 Type I Low Froude Number Basin (Fr=1.6)

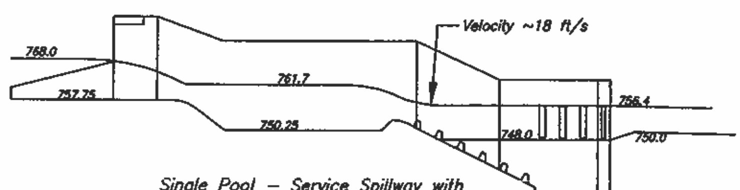


Single Pool – Service Spillway with Channel Elevation 755.0

Option B



Single Pool – Downstream Channle Elev. 750.0 Type I Low Froude Number Basin (Fr=2.5)



Single Pool – Service Spillway with Downstream Channel Elev. 750

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PROPOSED SPILLWAY SECTION
Robles Diversion Dam
Ventura County, California

By: _	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure **6**

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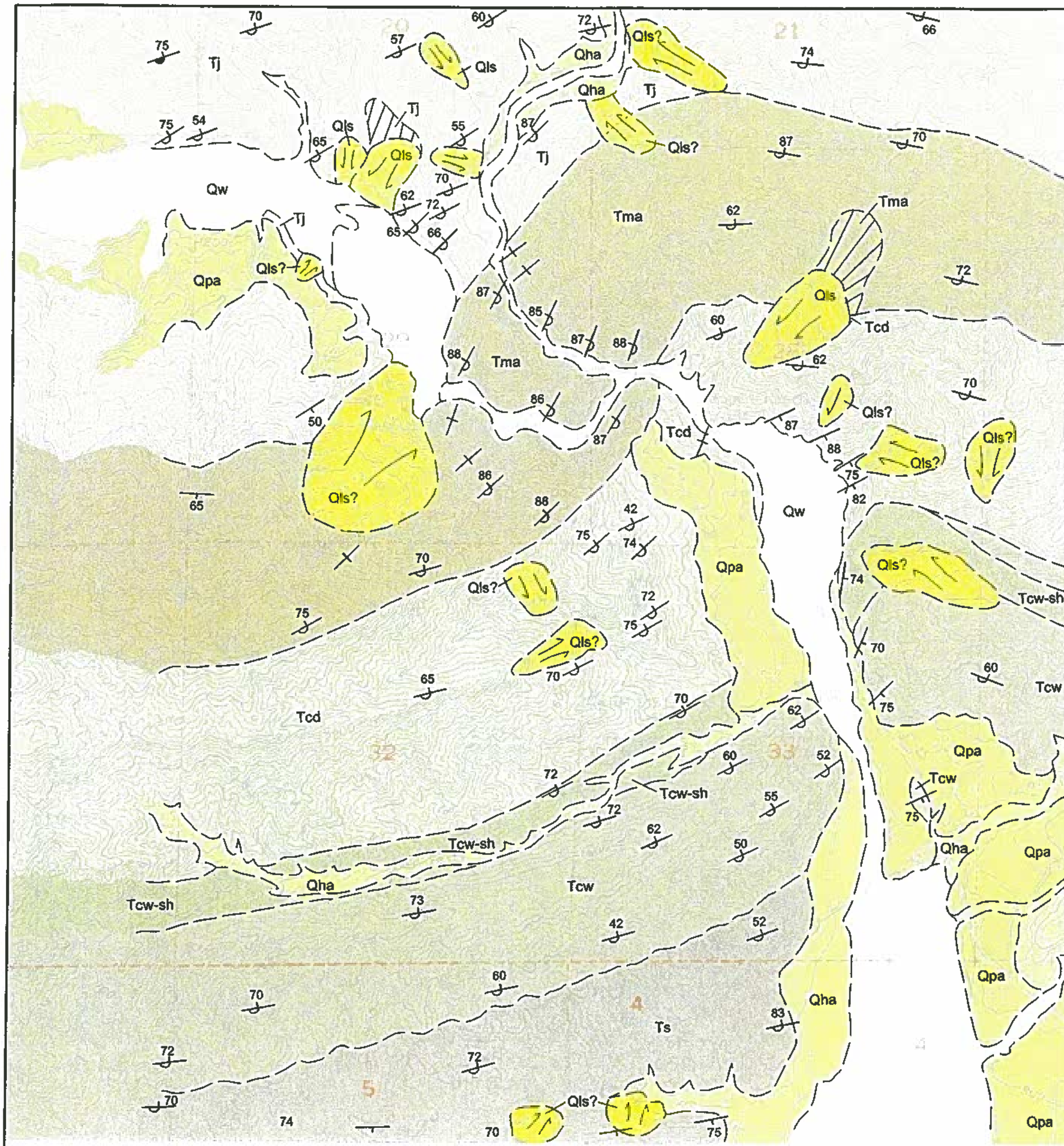

Unit Explanation

Symbol	Description
Qw	**Active wash deposits within major river channels (Holocene)** - Composed of unconsolidated silt, sand and gravel.
Qha	**Alluvial and colluvial deposits, undivided (Holocene)** - Located on the floors of valleys; includes active stream deposits in hill slope areas; composed of unconsolidated sandy clay with some gravel.
Qls	**Landslide deposits (Holocene to late Pleistocene)** - Includes numerous active landslides, composed of weathered, broken up rocks; extremely susceptible to renewed landsliding, including their head scarp areas.
Qpa	**Alluvial deposits, undivided (late Pleistocene)** - Consists of semi-consolidated silt, sand, clay, and gravel.
Ts	**Sespe Formation (Oligocene)** - Composed of sandstone; locally pebbly, siltstone and claystone; rocks are generally reddish in color.
Tcw / Tcw-sh	**Coldwater Sandstone (late Eocene)** - Composed of hard arkosic sandstone with siltstone and shale interbeds; locally reddish in color, similar to appearance of Sespe Formation. Tcw-sh consists predominantly of shale.
Tma	**Matilija Sandstone (middle to late Eocene)** - Composed of hard arkosic sandstone with micaceous shale interbeds.
Tj	**Juncal Formation (early to middle Eocene)** - Consists of micaceous shale with arkosic sandstone interbeds; generally susceptible to landsliding.

0 1 2 3 Thousand Feet

Contour Interval 40 Feet

Dotted Lines Represent Half-Interval Contours

GEOLOGIC MAP
Robles Diversion Dam
Ventura County, California

By:	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure 7

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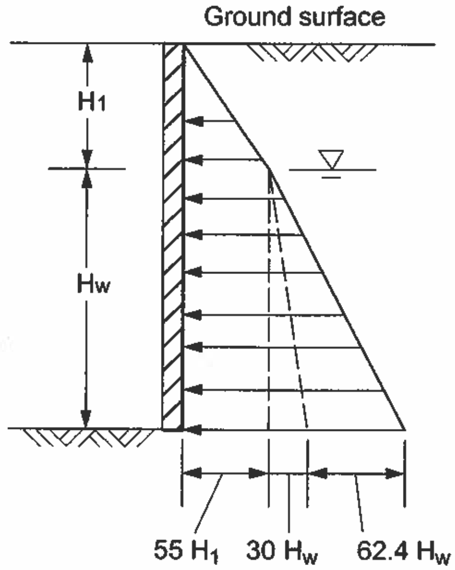
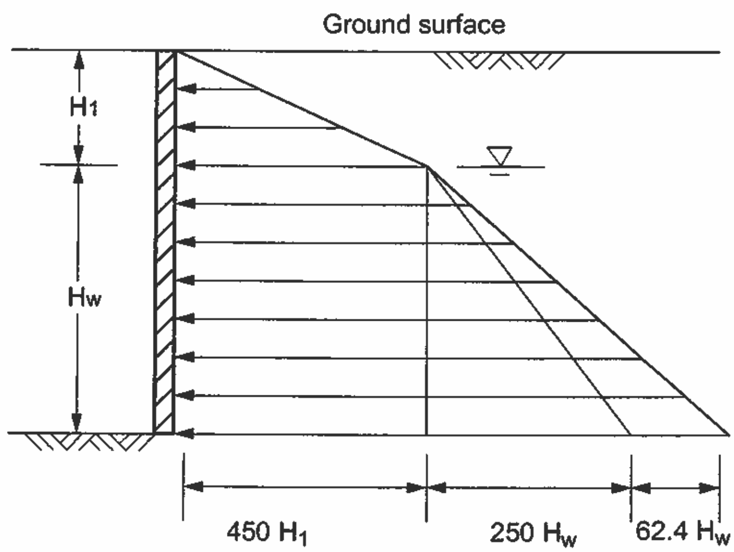


Non-yielding (At Rest) Wall Pressure (psf)



Passive Wall Pressure (psf)

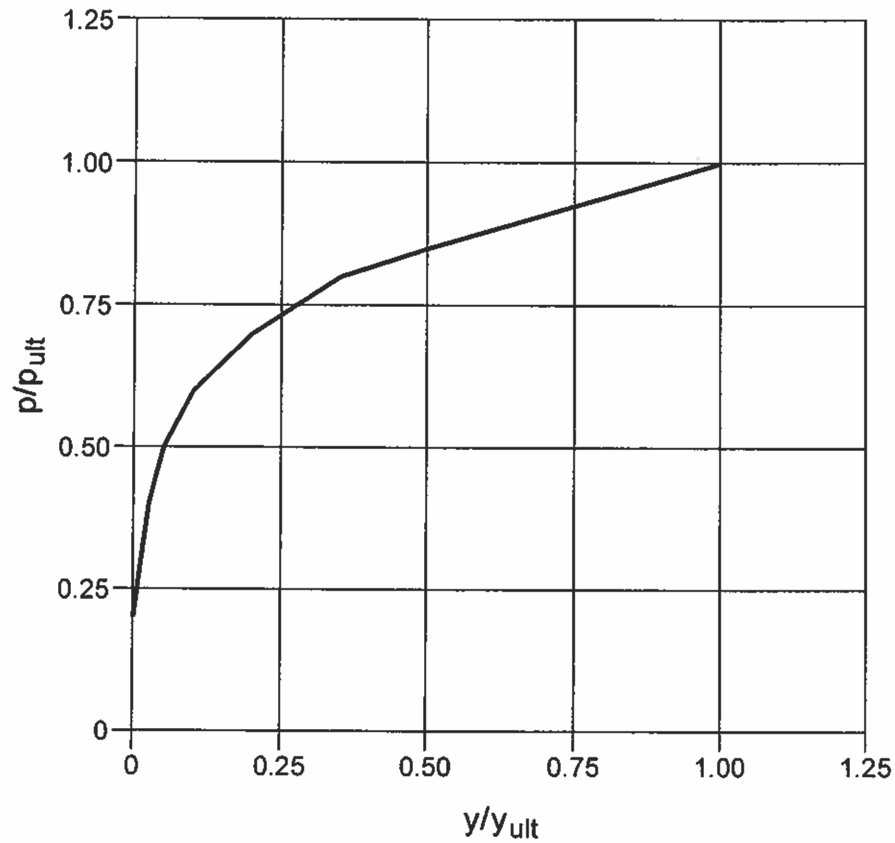
Notes

1. Above distributions apply to walls that are backfilled with native impervious site fill.
2. $H_W + H_1$ = height of earth retaining wall (in feet). H_W = height of groundwater above base of wall (in feet).
3. When multiplied by H_W and H_1 (in feet), coefficients yield lateral earth pressures in pounds per square foot (psf).
4. Passive pressure acting on wall, footing, or drilled pier must be calculated for assumed wall deflection using factors shown in Figure 9. Ignore upper foot of passive pressure when estimating lateral resistance of shallow foundations.
5. Pressure diagrams shown in this figure are appropriate for the design of buried structure walls and footings. Pressure diagrams needed for the design of the temporary excavation support systems were not developed as part of this study.

S:\9900\9993\9993.003\9993.003.2_fig_08.ai

LATERAL EARTH PRESSURE DISTRIBUTIONS FOR LEVEL BACKFILL CONDITIONS WITH HIGH GROUNDWATER Robles Diversion Dam Ventura County, California		
By: _	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure 8

DRAFT



p/p_{ult}	y/y_{ult}
0.20	0
0.40	0.025
0.50	0.05
0.60	0.10
0.70	0.20
0.80	0.35
0.85	0.5
1.0	≥1.0

Notes:

1. p_{ult} given in Figure 8.
2. $y_{ult} = 0.02H$

S:\9900\9993\9993.003\9993.003.2_fig_09.ai

NORMALIZED PASSIVE PRESSURE RESISTANCE VS. DEFLECTION AT TOP OF WALL
Robles Diversion Dam
Ventura County, California

By: -	Date: 08/08/2008	Project No. 9993.003
AMEC Geomatrix		Figure 9

DRAFT

APPENDIX A

Annotated Photographs

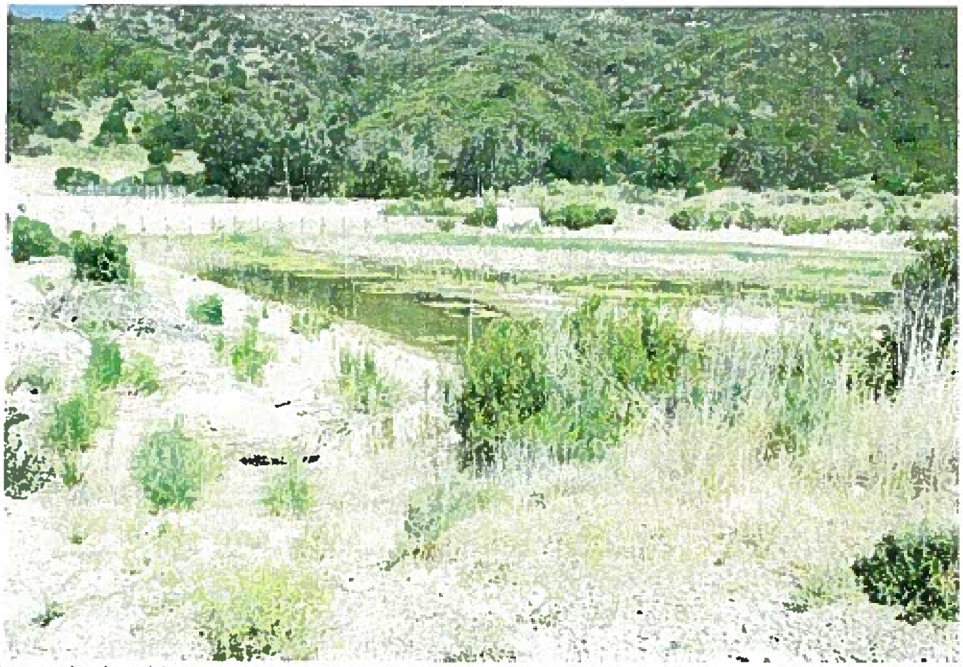


Figure 1 - Looking upstream and at the right bank from the crest of the diversion dam

Figure 2 - Shallow pool of water located upstream of the diversion dam

PHOTOS OF ROBLES DIVERSION DAM Whittier Narrows and Prado Dams Ventura County, California		
By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page 1 of 6

Figure 3 - Upstream face of the diversion dam; portions of the face contained soilcrete and large boulders

Figure 4- Upstream face of the diversion dam looking towards the right abutment

PHOTOS OF ROBLES DIVERSION DAM Whittier Narrows and Prado Dams Ventura County, California		
By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page **2 of 6**

Figure 5 - Looking at the left bank and top of the crest

Figure 6 - Silty sand with large amounts of gravel, cobbles, and boulders typical of the surface soils encountered at the site

PHOTOS OF ROBLES DIVERSION DAM Whittier Narrows and Prado Dams Ventura County, California		
By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page 3 of 6

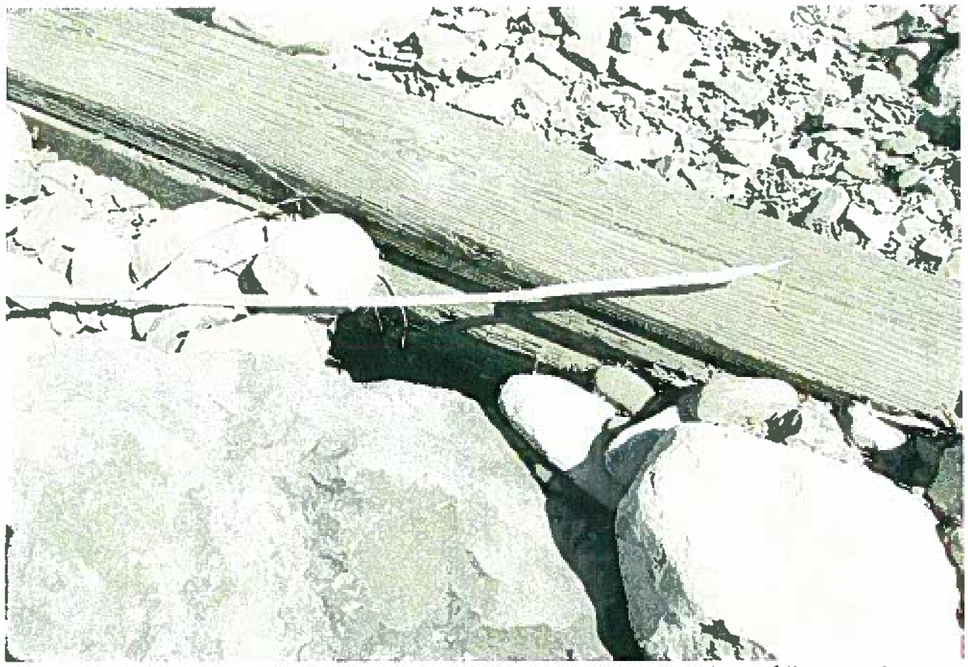


Figure 7 - Timber cutoff wall located at the northern edge of the crest

Figure 8 - Elaborate fish passageway located at the right bank

PHOTOS OF ROBLES DIVERSION DAM Whittier Narrows and Prado Dams Ventura County, California		
By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page 4 of 6

Figure 9 - Concrete spillway gates located at the right abutment

Figure 10 - Close-up of spillway gate

PHOTOS OF ROBLES DIVERSION DAM
Whittier Narrows and Prado Dams
Ventura County, California

By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page 5 of 6

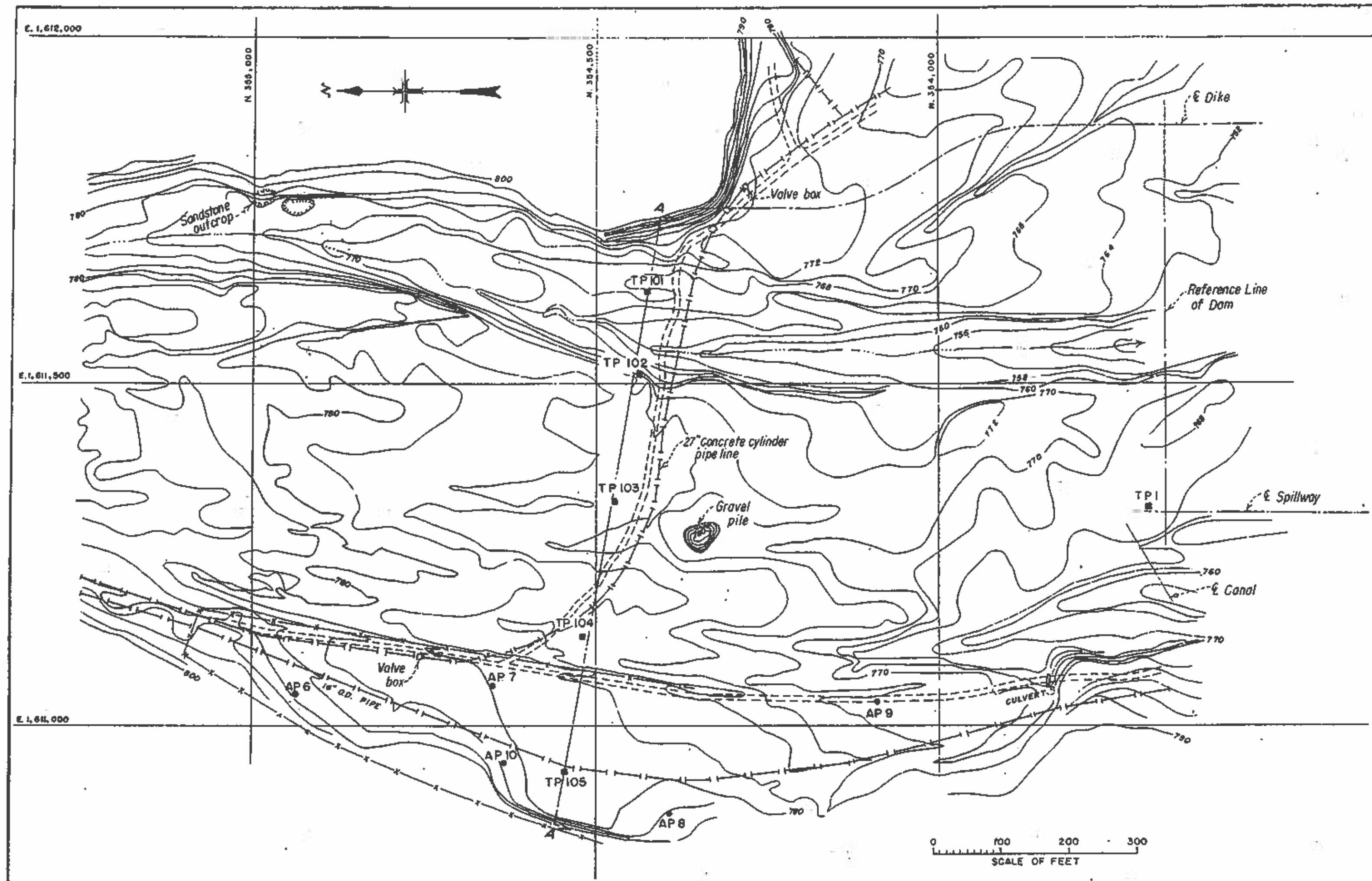
Figure 11 - Looking at the pool of water upstream of dam towards the left bank from the top of the fish passageway

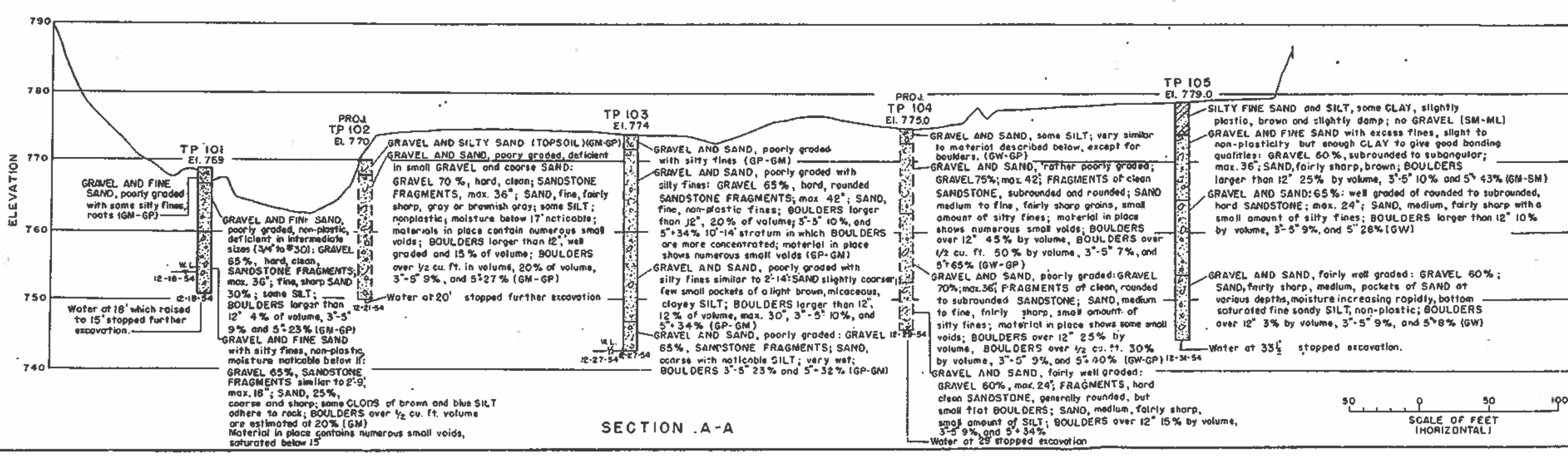
Figure 12 - Looking downstream of the diversion dam

PHOTOS OF ROBLES DIVERSION DAM Whittier Narrows and Prado Dams Ventura County, California		
By: mb	Date: 07/31/08	Project No.: 9993.002.3
AMEC Geomatrix		Page **6 of 6**

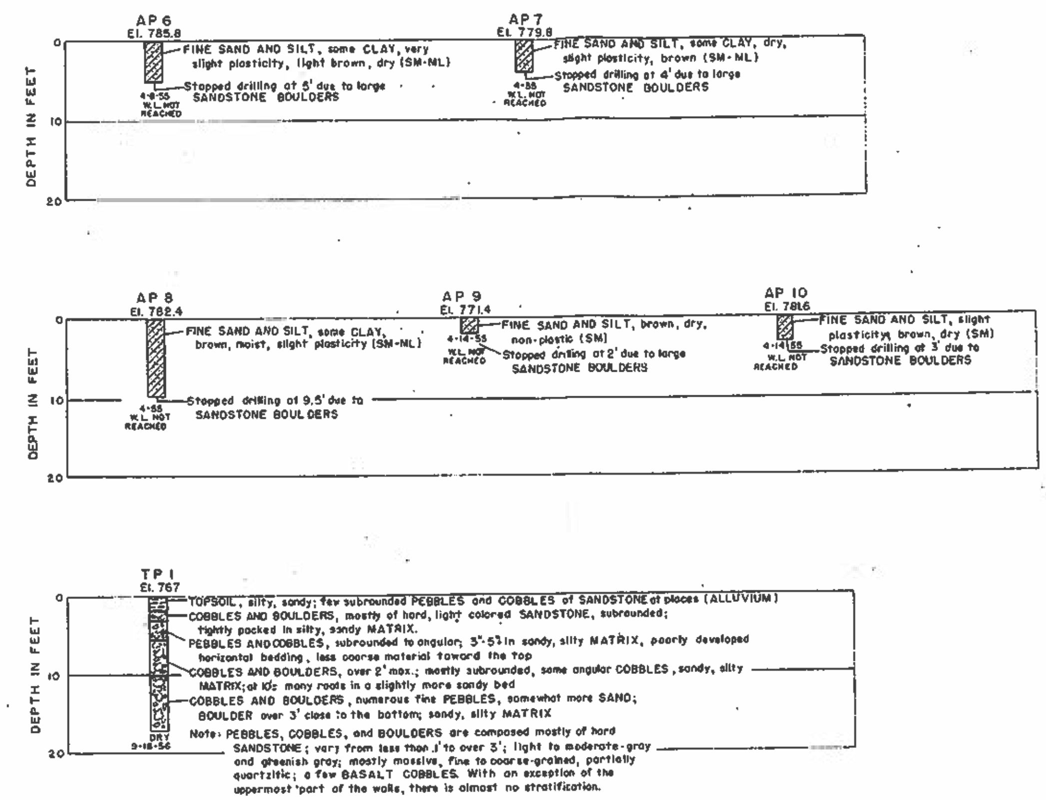
APPENDIX B

Logs of Borings and Test Pits from Previous Site Investigations





SECTION A-A



MISCELLANEOUS HOLES

EXPLANATION

● AP... Power Auger (30" dia.) bucket type

■ TP... Test Pit (16'x16') Clam Shell, except TP 104 (15' dia.) Clam Bucket, TP 105 (13' dia.) Clam Bucket and TP 1 size and method unknown

W.L. 12-18-54 ... Water level and date measured

Letter symbols in parentheses, following the soil descriptions in the logs are group symbols of the Unified Soil Classification System based on field identification. Copies of Drawing No. 103-D-347, Unified Soil Classification, may be obtained on request to Assistant Commissioner and Chief Engineer, Bureau of Reclamation, Denver Colorado

NOTE

Percentages of cobbles and boulders determined from field sample gradation analysis by weight for sizes under 12" and inspection by volume for sizes over 12"

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

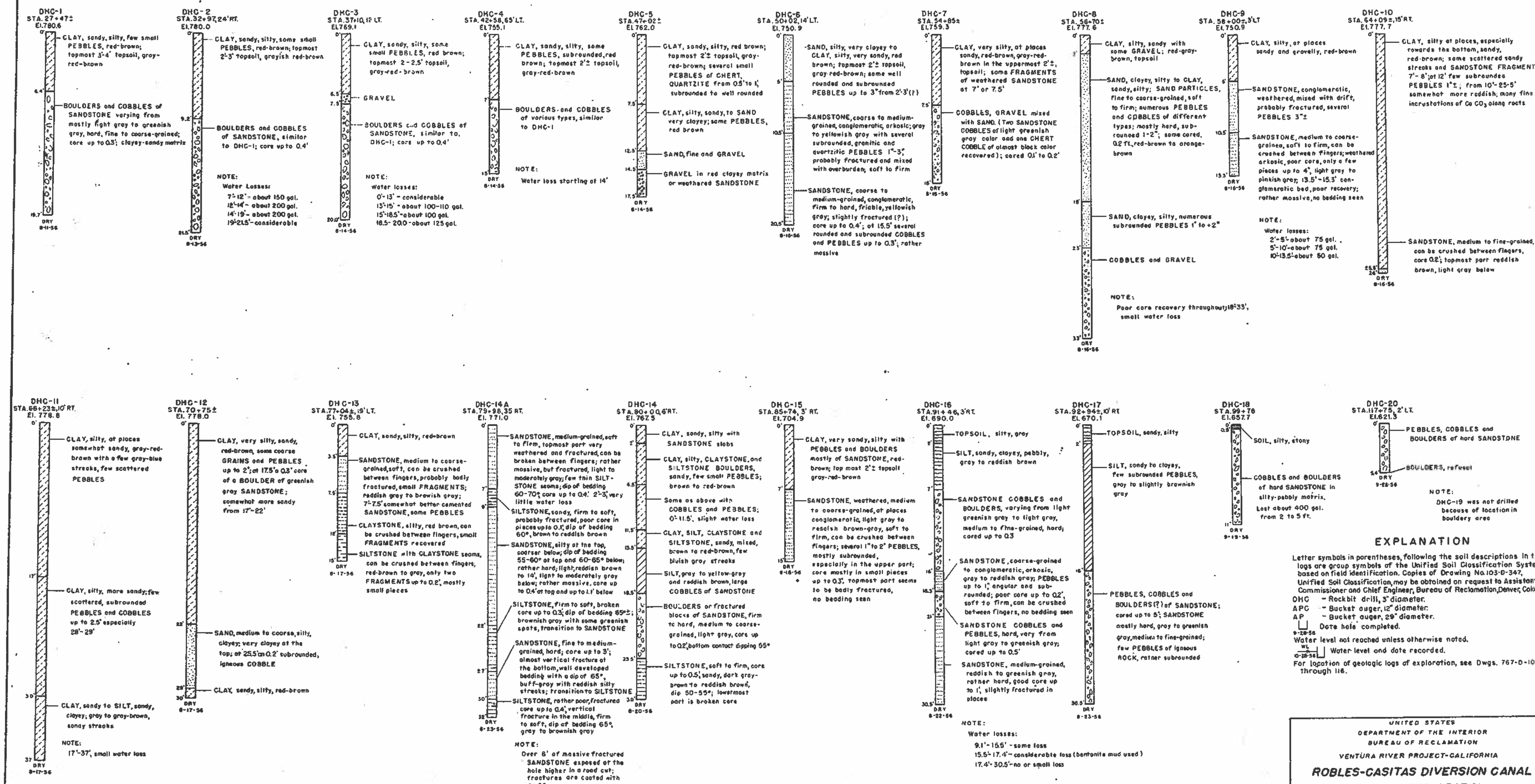
VENTURA RIVER PROJECT-CALIFORNIA

RÓBLES DIVERSION DAM

LOCATION AND LOGS OF EXPLORATION

DRAWN W.H. - J.F.M. SUBMITTED [signature]
TRACED J.F.M. RECOMMENDED [signature]
CHECKED [initials] APPROVED [signature]
CHIEF DESIGNING ENGINEER

DENVER, COLORADO, FEB. 4, 1957 | 767-D-221



EXPLANATION

Letter symbols in parentheses, following the soil descriptions in the logs are group symbols of the Unified Soil Classification System based on field identification. Copies of Drawing No.103-D-347, Unified Soil Classification, may be obtained on request to Assistant Commissioner and Chief Engineer, Bureau of Reclamation, Denver, Colorado.

DHC - Rockbit drill, 3" diameter.
APC - Bucket auger, 12" diameter.
AP - Bucket auger, 29" diameter.
9-28-56 Date hole completed.
Water level not reached unless otherwise noted.
WL 10-28-56 Water level and date recorded.

For location of geologic logs of exploration, see Dwgs. 767-D-107 through 116.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
VENTURA RIVER PROJECT-CALIFORNIA

ROBLES-CASITAS DIVERSION CANAL

LOGS OF EXPLORATION

DRAWN WE-EMW SUBMITTED [signature]
TRACED VMM RECOMMENDED [signature]
CHECKED [signature] APPROVED [signature] ACTING CHIEF DESIGNING ENGINEER

DENVER, COLORADO, FEB. 8, 1957 SHEET 1 OF 4 | 767-D-245

GROUND MOTION HAZARD EVALUATION FOR ROBLES DIVERSION DAM MODIFICATION PROJECT

Submitted to:

Los Angeles District
U.S. Army Corps of Engineers
Los Angeles, California

Submitted by:

AMEC Geomatrix, Inc., Oakland, California

January 19, 2009

Project No. 9993.003.2

AMEC Geomatrix

amec

January 19, 2009

Project 9993.003.2

Mr. Douglas Chitwood
Los Angeles District
U.S. Army Corps of Engineers
P.O. Box 532711
Los Angeles, California 90053-2325

Re: **Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project**
Ventura County, California

Dear Mr. Chitwood:

The enclosed report presents the results of a ground motion hazard study performed by AMEC Geomatrix, Inc. for the Robles Diversion Dam Modification Project (project). Our study involved characterizing potential seismic sources, evaluating site conditions, conducting site-specific probabilistic and deterministic seismic hazard analyses, and developing probabilistic and deterministic ground motion response spectra for use in design of the project. A draft copy of this report was submitted for your review on November 12, 2008. Review comments received on the draft were addressed in this report.

AMEC Geomatrix appreciates the opportunity to work with you. Please contact the undersigned if you have any questions about this report or if we can be of further service.

Sincerely yours,
AMEC GEOMATRIX, INC.

Donald L Wells

Donald L Wells, CEG
Senior Engineering Geologist

Faiz Makdisi, PE
Principal Engineer

dlw/fim/cw
I:\Doc_Safe\9000s\9993.003.2\3000 Report\Final GM Report\Cover Letter_GM rpt_Final_01-19-09.doc

Enclosure

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Tel (510) 663-4100
Fax (510) 663-4141
www.amecgeomatrixinc.com

AMEC Geomatrix

TABLE OF CONTENTS

TABLES

FIGURES

TABLE OF CONTENTS
(Continued)

FIGURES (Continued)

GROUND MOTION HAZARD EVALUATION
ROBLES DIVERSION DAM MODIFICATION PROJECT
Ventura County, California

1.0 INTRODUCTION

This report presents the results of a ground motion hazard evaluation performed by AMEC Geomatrix, Inc. (AMEC Geomatrix) for the Robles Diversion Dam Modification Project in Ventura County, California. The Robles Diversion Dam (RDD) is located about 1.9 miles downstream of Matilija dam (Figure 1); its purpose is to divert water from the Ventura River to the Robles-Casitas Diversion Canal. The overall Robles Diversion Dam Modification Project includes construction of a high flow bypass spillway, a stilling basin, a downstream rock ramp, and a technical spillway. The new facilities will be constructed adjacent to (and east of) the original Ventura River spillway structure.

1.1 PURPOSE AND SCOPE

The purpose of this study is to provide the geologic and the seismic information and estimated ground motions needed for the design of the new high flow bypass spillway and stilling basin included in the overall Robles Diversion Dam Modification Project. Specifically, this report presents the site-specific earthquake horizontal response spectra for ground motions that may be experienced at the Robles Diversion Dam located in Ventura County, California. The earthquake ground motions correspond to the following design level events: an Operating Basis Earthquake (OBE), a Maximum Design Earthquake (MDE), and a Maximum Credible Earthquake (MCE).

The scope of work consisted of the following tasks:

Task	Description
1	Review available information for site characterization
2	Characterize potential seismic sources
3	Perform probabilistic and deterministic seismic hazard evaluations and develop ground motion response spectra for the MCE and the OBE.
4	Prepare ground motion hazard report

The scope of work performed is described in the Amendment to Task Order 3 of Contract (W912PL-07-D-0004-0003) dated 3 August 2007.

The primary approach taken in this study is to conduct a probabilistic ground motion analysis to estimate the probability of exceedance of peak ground acceleration (PGA) and response spectral accelerations (Sa) at the site during selected exposure times. The results of the probabilistic approach were used to develop smooth site-specific response spectra for the OBE and MDE events, using 50-percent probability of exceedance in 100 years (144 year return period) and 10-percent probability of exceedance in 100 years (950 year return period), respectively. For the MCE event, we used the deterministic approach to estimate ground motions from nearby controlling seismic sources. The following sections describe the geologic and seismic setting, the site subsurface conditions, the probabilistic and deterministic ground-motion analyses conducted for the site, and the development of site-specific horizontal earthquake response spectra.

AMEC Geomatrix (AMEC) also has prepared a report to document the foundation conditions for Robles Diversion Dam and to provide geologic and geotechnical information and recommendations needed for the design of the new high flow bypass spillway and stilling basin included in the overall Robles Diversion Dam Modification Project. The draft Foundation Report was submitted in August, 2008.

1.2 Project Organization

The work described in this report was coordinated with the following individuals:

- Mr. Doug Chitwood - USACE

AMEC Geomatrix personnel who participated in this project include:

- Dr. Faiz Makdisi – Principal-in-Charge
- Mr. Donald Wells, Senior Geologist
- Ms. Alexis Lavine, Project Geologist
- Mr. Tawat Anantanavanich, Staff Engineer
- Mr. Trey Apel, Staff Geologist
- Mr. Serkan Bozkurt, GIS Analyst

1.3 Report Organization

The existing conditions, project team, organization of the report are described in Sections 1.1, 1.2, and 1.3, respectively. The geologic setting and site conditions, including surface and subsurface conditions, and the site soil classification, are described in Section 2.0. The tectonic and seismic setting of the site, and characterization of seismic sources is presented in

Section 3.0. The ground motion hazard analysis and response spectra for use in structural analysis of the diversion dam are presented in Sections 4.0. Limitations of the investigation are noted in Section 5.0. References are presented in Section 6.0.

2.0 GEOLOGIC SETTING AND SITE CONDITIONS

Robles Diversion Dam is located at 34.4651° N latitude and 119.290° W longitude, in Ventura County, Southern California. The geologic setting, site conditions, and site classification are described in the following sections.

2.1 GEOLOGIC SETTING

The Robles Diversion Dam is located on the Ventura River, approximately 1.6 miles downstream (south) of the confluence of the Matilija and the North Fork of the Matilija Rivers, and approximately 1.9 miles downstream from Matilija Dam (on the Matilija River) (Figure 1). The region lies within the eastern Santa Ynez Mountains, which are part of the Western Transverse Ranges Province of Southern California. The Santa Ynez Mountains are a young east-west trending mountain range, composed of highly folded and faulted Cenozoic and late Mesozoic marine sedimentary rocks that have been deformed by slip on a series of generally east-west trending strike slip and reverse slip faults (Jennings and Strand, 1969). The diversion dam site lies near the southeast margin of the Santa Ynez Mountains, about 0.9 mile south of where the Ventura River emerges from a narrow canyon into a wider floodplain characterized by braided channels and extending to the Pacific Ocean (U.S. Army Corps of Engineers, 2004).

The geologic structure in the area surrounding the diversion dam site is characterized by a series of east-west trending, tightly folded anticlines and synclines, where the bedrock includes sandstone, siltstone, and shale of the late Eocene Cozy Dell Formation and Coldwater Formation, and the Oligocene Sespe Formation. The diversion dam site lies on the north limb of a syncline, where sandstone and siltstone beds within the Sespe Formation are overturned to dip steeply north. The Ventura River floodplain, upon which the diversion dam sits, is underlain by young unconsolidated fluvial terrace and channel deposits, including sand, gravel, and boulders overlying bedrock of the Sespe Formation (Figure 2; Dibblee, 1987; Tan and Jones, 2006).

2.2 SITE CONDITIONS

The diversion dam that currently exists spans about 350 feet across the Ventura River. According to the recent site topographic map provided by the USACE (Figure 3), the river channel is about 10 to 15 feet below its eastern and western banks. Bars of primarily coarse-grained material (gravel, cobbles and boulders) have formed near mid-channel both upstream and downstream of the diversion dam. Downstream of the diversion dam, the main river channel is near the diversion dam's left abutment. As described in the Foundation Report, the available drawings and specifications indicate that the diversion dam is a zoned earthfill and rockfill embankment that was constructed using the various earth materials taken from the required excavations for the diversion dam and along the Robles-Casitas Diversion Canal, and other nearby borrow areas.

As described in the Foundation Report, the Ventura River channel has been mapped by Dibblee (1987) and Tan and Jones (2006; Figure 2) as containing stream channel deposits consisting of mostly gravel and sand; cobbles and boulders also are present in these deposits (Figure 3). The banks of the river have been mapped as alluvium (i.e., unconsolidated flood plain deposits of silt, sand and gravel). Also as described in the Foundation Report (AMEC Geomatrix, 2008), test pit and boring logs for exploration work conducted at and near the dam show that the alluvium consists of sand and sandstone pebbles, cobbles and boulders to the maximum depth of the explorations at about 30 feet. The cobbles and boulders compose up to 65 percent of the alluvium, with boulders up to 3 feet in diameter.

Based on the geomorphic setting of the dam, with respect to the width and depth of the alluvial floodplain and the proximity of bedrock along the valley margins, we estimate that the alluvium may be on the order of 40 feet thick at the site. The alluvium overlies sandstone, siltstone, and claystone of the Oligocene Sespe Formation (Figure 2).

It is our understanding that the dam is founded on the alluvial materials in the river channel. The ground motions for the dam and site are developed at the soil surface at foundation level in the free field.

2.3 Site Classification

The shear wave velocity (V_S) is the velocity that shear waves – such as those produced by an earthquake – will have as they pass through the soil or bedrock. Specifically, the subsurface soil and rock profile at a site can be classified as based on the average shear wave velocity of the materials in the uppermost 100 feet (30 meters, V_{S30}). The V_{S30} is currently the preferred parameter for characterizing (classifying) the site conditions in developing estimates of ground shaking from ground motion attenuation relationships.

No site-specific measurements of shear wave velocity were available for dam site. However, the shear wave velocity can be estimated based on comparison to other sites with similar geologic conditions where the shear wave velocity has been measured. Information on shear wave velocity of geologic materials is presented in Wills and Clahan (2006). This study indicates that coarse alluvium has a mean V_{S30} of 354 m/s, and that Tertiary sandstone units, such as the Sespe Formation, have a mean V_{S30} of 515 m/s. Shear wave velocity measurements at Casitas Dam to the south of RDD, indicate that the shear wave velocity of the Sespe Formation bedrock is about 550 m/s (Sirles,1988). The V_{S30} for RDD site is estimated for a profile consisting of 40 feet of coarse alluvium (V_S 350 m/s), overlying the Sespe Formation (V_S 550 m/s), which results in a V_{S30} of 450 m/s.

Additional site parameters that are used in ground motion attenuation relations are estimates of the depth to rock layers with shear wave velocity of 1000 m/s (Z_1) and 2,500 m/s ($Z_{2.5}$). These parameters can be obtained from direct measurements, if available, or alternatively default values may be obtained from empirical relationships between V_{S30}, Z_1, and $Z_{2.5}$. Velocity profiles developed by the U.S. Bureau of Reclamation for a ground motion evaluation of Casitas Dam (O'Connell, 1999) indicates the Z_1 is about 200 m, and $Z_{2.5}$ is about 3,500 m. Based on the similar setting of the sites along the Ventura River, the estimates from Casitas Dam are judged reasonable to use as site-specific estimates of these parameters for RDD.

3.0 SEISMIC SETTING AND SEISMIC SOURCE CHARACTERIZATION

The Robles Diversion Dam is located in the Western Transverse Ranges, a region dominated by uplift along reverse (thrust) faults and translation along right-lateral strike slip faults. Major active faults in the region include the Santa Ynez fault, located about 3.1 miles north of the diversion dam, the San Cayetano fault, located about 7.5 miles east of the site, and the Mission Ridge-Arroyo Parida-Santa Ana fault, located about 2.4 miles south of the dam site. The San Andreas fault is located about 28 miles northeast of the site. The San Andreas fault extends over 1200 km from the Gulf of California to Cape Mendocino, and is considered to be the most significant tectonic structure within the region and throughout California. It was the source of a great M ~8 earthquake in January 1857 that ruptured from Cholame (south of Parkfield) as far south as Cajon Pass (Agnew and Sieh, 1978), over a distance of about 225 miles (360 km).

Other major earthquakes that have resulted in strong ground shaking in the region include two magnitude 7.0+ earthquakes in 1812 (Santa Barbara Channel offshore) and 1927 (offshore of Lompoc), and the 1952 M_W 7.5 earthquake on the White Wolf fault. These earthquakes are estimated to have caused ground shaking with peak ground accelerations (PGA) of up to about 0.2 g at the site (U.S. Army Corps of Engineers, 2004). Ten additional earthquakes of magnitude 5.5 to 6.0 have been recorded at distances of about 12.5 to 31 miles from the site. (http://redirect.conservation. ca.gov/cgs/rghm/quakes/historical/index.htm). Based on the historical seismicity, proximity, estimated maximum magnitude, and slip rate for major active faults near the site, the California Geological Survey indicates that the expected PGA for an earthquake with a return period of 475 years is about 0.56 g.

4.0 GROUND MOTION HAZARD ANALYSIS

This section provides an assessment of earthquake-induced ground shaking potential for the Robles Diversion Dam site. As part of this assessment, both a probabilistic seismic hazard analysis (PSHA) and a deterministic seismic hazard analysis (DSHA) were performed to characterize earthquake ground shaking that may occur at the site during future seismic events in the region. The PSHA was conducted to estimate the probability of exceedance of peak ground acceleration (PGA) and response spectral accelerations (Sa) at the site during selected exposure times. The PSHA was conducted using the software program EZ-Frisk developed by Risk Engineering, and the DSHA was conducted using software developed by AMEC Geomatrix to estimate ground motions from recently published attenuation relationships.

The elements of the site-specific ground motion assessment presented in this section are the approach for the PSHA (Section 4.1), seismic source characterization (Section 4.2), selection of attenuation relationships (Section 4.3), results of the PSHA (Section 4.4), assessment of deterministic response spectra (Section 4.5), development of average horizontal maximum credible earthquake (MCE) and design response spectra (Section 4.6), and assessment of fault rupture near-field effects (Section 4.7).

4.1 APPROACH FOR PROBABILISTIC GROUND MOTION ANALYSIS

The probabilistic analysis, commonly termed a "probabilistic seismic hazard analysis" (PSHA) is based on an assessment of the recurrence of earthquakes on potential seismic sources in the Western Transverse Ranges and offshore Santa Barbara region and on ground motion attenuation relationships appropriate for the types of seismic sources in the region and the subsurface conditions interpreted for the project site. Results of the hazard analysis are expressed as relationships between amplitudes of peak ground acceleration and response spectral acceleration (at specified structural periods), and the annual frequencies or return periods (return period being the reciprocal of annual frequency) for exceeding those ground motion amplitudes.

The PSHA analysis procedure requires the specification of probability functions to describe the uncertainty in both the time and location of future earthquakes and the uncertainty in the ground motion level that will be produced at the project site. The basic elements of the analysis are:

1. identification of potential (active) seismic sources that could significantly contribute to seismic hazard at the project site;
2. specification of an earthquake recurrence relationship for each seismic source, defining the frequency of occurrence of various magnitude earthquakes up to the maximum magnitude possible on the source;
3. specification of attenuation relationships defining ground motion levels as a function of earthquake magnitude, distance, and style of faulting from an earthquake rupture; and
4. calculation of the probability of exceedance of peak ground acceleration and response spectral accelerations (i.e., seismic hazard) using inputs from the elements above, and development of equal-hazard (i.e., equal-probability-of-exceedance) response spectra from the results.

The probabilistic seismic hazard analysis conducted for this study is based on the seismic source model developed by the USGS and California Geological Survey (CGS) to prepare the 2002 ground shaking hazard maps for California (Petersen et al., 1996; Cao et al., 2003). The USGS/CGS seismic source model also incorporates background areal source zones that account for the possibility that earthquakes may be generated in the regions between the known fault sources. The background source zones are represented by spatially smoothed gridded seismicity calculated on a cell by cell approach (Frankel et al., 1996). The source parameters are combined in a logic tree approach that incorporates the full range of uncertainty in the individual parameters that are used as input into the ground motion hazard analysis.

The recurrence rates for these sources are based on a weighted average of a characteristic earthquake model (Schwartz and Coppersmith, 1984; Youngs and Coppersmith, 1985) and a Gutenberg-Richter (truncated) exponential model (Cornell and Van Marke, 1969), as described in Petersen al. (1996, 2002) and Cao et al. (2003).

4.2 SEISMIC SOURCE CHARACTERIZATION

Two types of potential crustal seismic sources are included in the seismic hazard model: 1) faults, and 2) areal source zones. The assessment of maximum magnitudes and earthquake recurrence rates for both types of sources are described in this section.

4.2.1 Fault Sources

The location and activity of mapped faults in the region surrounding the site are shown on Figure 4. Characterization of seismic sources for probabilistic analysis of faults entails describing a number of fault parameters and the uncertainty associated with each parameter. These parameters include the probability of activity, sense of slip, the three-dimensional geometry, downdip width within the seismogenic crust, rupture segmentation, maximum earthquake magnitude, fault slip rate, and recurrence (magnitude distribution).

As shown on Figure 4, the major active faults located near the site include the Mission Ridge-Arrroyo Parida-Santa Ana, Santa Ynez, San Cayetano, Oak Ridge, Red Mountain, Pitas Point-Ventura, Oak Ridge, and San Andreas faults. The distance, direction from the site, maximum magnitude, and slip rate for each of these faults and other nearby fault sources are summarized in Table 1. These parameters are based on the CGS seismic source model for California (http://www.consrv.ca.gov/cgs/rghm/psha/Pages/index.aspx).

The Working Group on California Earthquake Probabilities (WGCEP, 2008) has published updated source parameters for faults that will be used in developing the 2008 California ground shaking hazard maps; this report shows that the source parameters for the major active faults near the site (Santa Ynez, Mission Ridge-Arroyo Parida-Santa Ana, and San Cayetano faults) are not changed significantly from the 2002 fault characterization, except for incorporation of a multi-segment rupture on the Santa Ynez fault.

4.2.2 Maximum Earthquake Magnitudes

Maximum earthquake magnitudes have been assessed for each fault based on the current understanding of the tectonic environment and individual fault parameters (e.g., sense of slip, total length, maximum rupture length, maximum displacement per event, and rupture area). Empirical relationships among these fault parameters were utilized to estimate maximum earthquake magnitudes (as described in Cao et al., 2003; WGCEP, 2008). The mean maximum magnitudes from the weighted probabilistic distribution of maximum magnitudes are listed in Table 1.

4.2.3 Earthquake Recurrence

As described in Section 2.2, the project site has experienced several large earthquakes during the historical period (approximately 200 years). However, because this historical record is short relative to the earthquake recurrence intervals on individual faults, and because of uncertainties in the locations of older earthquakes, seismicity data alone are not adequate to define earthquake recurrence on individual faults. To estimate recurrence on individual faults, geologic evidence for the long-term rate of seismicity must be utilized as well. This seismicity rate is assessed from the geologic slip rate of the fault using the seismic moment rate approach (Youngs and Coppersmith, 1985).

Assuming that seismic moment is released through earthquake occurrence, the rate of release must be distributed through earthquakes of all magnitudes up to the maximum earthquake capability of the fault. Regional and fault-specific historical seismicity data, which are sufficient to provide estimates of the relative rate of occurrence among more frequent, smaller-magnitude events, are used to define the slope ("b"-value) of the magnitude-frequency relationship. When this magnitude-frequency slope is coupled with the total moment rate evaluated from geologic evidence, average recurrence for earthquakes of various magnitudes on a fault may be described. Assessed slip rates for regional faults are reported by the USGS and CGS (Cao et al., 2002; WGCEP, 2008) and the range of rates used in the PSHA are summarized in Table 1.

For this study, earthquake recurrence has been modeled in a way that accounts for our current understanding of fault behavior. The characteristic earthquake concept has been developed from geologic and seismicity data on interplate and intraplate faults (Schwartz and Coppersmith, 1984). The characteristic earthquake concept has direct implications to fault-specific earthquake recurrence relationships (Youngs and Coppersmith, 1985). Because of the tendency for a fault or fault segment to generate the characteristic event (which is essentially the maximum earthquake), moderate-size events occur less frequently than the characteristic event.

In a seismic hazard analysis, it is assumed that on a given fault, earthquakes of a certain magnitude may occur with equal likelihood at different locations along the length and depth of the fault. The extent of rupture on a fault varies with earthquake magnitude according to a selected relationship between rupture area and magnitude. The dimensions of an earthquake's rupture area are the length of the rupture along the fault strike and the width of the rupture in the dip of the fault plane (for vertical faults and events with ground surface rupture, width is defined by the depth of rupture). For smaller events, i.e., magnitudes 4 to 5, rupture area is small and the rupture dimensions are assumed to be equal. For larger earthquakes, the length-to-width ratio is greater than one; e.g., for magnitude 7, the ratio is assumed to be 2:1. It has been empirically observed that rupture area increases rapidly with increasing earthquake magnitude (e.g., Wells and Coppersmith, 1994), a trend that is important in the hazard analysis because the larger-magnitude earthquakes, having greater rupture areas, will tend to rupture portions of the faults closer to the site.

4.2.4 Gridded Background Seismicity

In seismic hazard analysis, gridded seismicity rates are used to represent the occurrence of background seismicity that cannot be associated with specific geologic structures. This includes random background seismicity and larger events that occur on faults that are not explicitly included in the seismic source model. In the USGS and CGS seismic source model, the background seismicity is represented by spatially smoothed gridded seismicity (Frankel et al., 1996, 2002).

4.3 ATTENUATION RELATIONSHIPS

A ground motion attenuation model relates the amplitudes of peak acceleration and response spectral acceleration to earthquake magnitude and source-to-site distance. Different attenuation models are required for different types of seismic sources. The Pacific Earthquake Engineering Research Center (PEER) sponsored a project to develop updated ground motion models (Next Generation Attenuation or NGA) for California. The results of this project are five ground motion models for the randomly oriented average horizontal component of ground motions, Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2008), and Idriss (2008). The models provide estimates of spectral accelerations in the period range of 0.01 second to 10 seconds (spectral periods of 0.1 to 100 Hz). Four of the models provide ground motion estimates as a function of the average shear wave velocity of the top 30 meters underlying the site, V_{S30}. The fifth model, Idriss (2008), provides estimates for soft rock sites with V_{S30} greater than or equal to 450 m/sec. Each of the PEER-NGA model developers consider that their model replaces previous models that they have developed. These models are also being used by the USGS to develop the current version of the National Seismic Hazard Maps.

The NGA attenuation relationships are defined in terms of M_W (moment magnitude). Except for the Boore and Atkinson (2008) relationships which use closest distance to the surface projection of the rupture as the distance measure, the NGA relationships all use closest distance to fault rupture as the distance measure.

The NGA models include the effect of style of faulting on median ground motions. All of the models include the effect of reverse faulting. Four of the models include factors for normal faulting. The Abrahamson and Silva (2008) and Chiou and Youngs (2008) normal faulting factors apply only to normal faulting with rakes in the range of -60° to -120°, and group normal-oblique faulting together with strike-slip faulting. The Boore and Atkinson (2008) and the Campbell and Bozorgnia (2008) normal faulting factors apply to both normal and normal-oblique faulting, rakes in the range of -30° to -150°. The Idriss (2008) model combines normal and strike-slip faulting into a single category.

The estimated median peak ground accelerations (*pga*) and 1-second pseudo-absolute spectral accelerations (*psa*) produced by the PEER-NGA ground motion models are very similar for strike-slip earthquakes, but show increased variability for normal faulting earthquakes, in part because of differences in modeling ground motions in the hanging wall of dipping faults. Explicit hanging wall models are included in the Abrahamson and Silva (2008), Campbell and Bozorgnia (2008), and the Chiou and Youngs (2008) models. The Boore and Atkinson (2008) model uses Joyner-Boore distance and thus implicitly includes a hanging-wall effect. The Idriss (2008) model does not include a hanging wall term. In the PEER-NGA models, the magnitude of the hanging wall effect generally decreases as the spectral period increases.

Preliminary analyses conducted during the development of the Chiou and Youngs (2008) model suggest that inclusion of the uncertainty in the predictor variables would increase the standard errors in the median motions to values in the range of 0.1 to 0.15 (natural log units). Using these results, an epistemic uncertainty of 0.15 (in natural log units) is assigned to each of the PEER-NGA models in the hazard analysis. This level of uncertainty captures the general uncertainty estimated for the Chiou and Youngs (2008) model, and similar estimates would be expected for the other PEER-NGA models because they are based on similar data sets. This uncertainty in the individual models combined with the variability among the models represents the uncertainty in median motions incorporated in the PSHA model. The basis for the selection of the relationships is presented in the following section.

As described in Section 2.0, the subsurface conditions at the site are characterized as dense coarse alluvial materials to a depth of about 40 feet, overlying sandstone/siltstone/claystone bedrock, with an estimated V_{S30} of 450 m/s. Therefore, four of the NGA attenuation relationships are selected for the crustal earthquake sources (faults and areal source zones) in this analysis, including the relationships of Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). The relationships of Idriss (2008) are not included because they are developed for soft rock sites, and may not be appropriate for alluvial and bedrock conditions at the RDD site. Each of the four selected PEER-NGA models was assigned equal weight in the analysis as they were developed using a common process and a common global data base.

4.4 RESULTS OF PROBABILISTIC SEISMIC HAZARDS ANALYSIS

The basic results of the PSHA are presented in Section 4.4.1 in terms of annual frequency of exceedance versus spectral acceleration (commonly referred to as hazard curves). Equal hazard response spectra for various probabilities of exceedance are presented in Section 4.4.2.

4.4.1 Calculations of Frequencies of Exceedance

The frequencies of exceedance of various values of peak ground acceleration and response spectral acceleration at the site for given structural periods were calculated by combining, for each fault and then for all the faults:

1. the annual frequency of earthquakes of various magnitudes on a fault obtained from the fault recurrence relationships;
2. given an earthquake of a certain magnitude on a certain fault, the probability distribution of the location of the earthquake on the fault obtained using the selected rupture area versus magnitude relationship and assuming equal likelihood of rupture along the length and some prescribed probabilities along the depth of the fault; and
3. given an earthquake of a certain magnitude occurring at a certain distance from the site, the probability distributions of ground motion at the site obtained from the selected attenuation relationships. For this study, attenuation relationships were utilized corresponding to peak ground acceleration (0.03 second) and ten structural periods (0.05, 0.1, 0.2, 0.3, 0.4, 0.5, 1, 2, 3, and 4 seconds) at a damping ratio of 5 percent.

Figure 5 illustrates typical results of the probabilistic analysis for ground motions at the site in terms of annual frequency of exceedance of horizontal peak ground acceleration; the contributions of individual seismic sources to the total hazard curve also are shown. This plot shows that major contributions to the PGA result from earthquakes occurring on the San Ynez, San Cayetano, and San Andreas faults. Similar results were obtained for spectral accelerations at other structural periods of vibration.

4.4.2 Equal-Hazard Response Spectra for Operating Basis Earthquake

Having obtained the annual frequency of exceedance of a certain level of horizontal response spectral acceleration, the probability of exceeding that level within any time period of interest is then obtained assuming a Poisson distribution, as follows:

$$P_E = 1 - \exp(-\mu t)$$

in which "P_E" is the probability of exceedance, "μ" is the annual frequency of events that exceed that level of ground motion, and "t" is the specified time period of interest.

Using the suite of probabilistic hazard analysis results, smooth equal-hazard (i.e., equal-frequency-of-exceedance) horizontal ground-motion response spectra were constructed for the dam site. For this study, the OBE design spectrum is selected to be associated with 50% probability of exceedance (P_E) in a 100-year time period (a return period of 144 years). We also provide five-percent damped horizontal equal hazard response spectra for additional P_E's of 10% in 50 years, 10% in 100 years, and 2% in 50 years (corresponding to return periods of 475, 950, and 2475 years, respectively) (Figure 6).

The contribution of earthquakes in different magnitude and distance ranges to the PGA for a return period of 144 years is shown on Figure 7. These plots also show that the major contribution to the ground motion hazard results from large magnitude (characteristic) earthquakes occurring on the San Ynez, San Cayetano, and San Andreas faults at distances of about 5, 12, and 40 km from the site, respectively.

Spectral acceleration values at the selected structural periods analyzed are presented in Table 2 for the OBE (144-year return period) and additional P_E's for the dam site. The value of peak horizontal ground acceleration associated with the OBE is 0.32 g, as indicated on the typical hazard results illustrated in Figure 5 and the equal hazard response spectra shown in Figure 6 and listed in Table 2.

Engineering Manual No. 1110-2-2100 of the U.S. Army Corps of Engineers (2005) defines Maximum Design Earthquake (MDE) ground motions, as the level of ground motion for which a structure is designed or evaluated. Generally, the probabilistically determined MDE for structures is designated as a ground motion level that has a 10 percent chance of being exceeded in a 100-year period (or a 950-year return period). The value of the PGA for the 950-year MDE as shown in Table 2 is 0.63g. The corresponding response spectrum is shown on Figure 6.

4.5 DETERMINISTIC RESPONSE SPECTRA FOR MAXIMUM CREDIBLE EARTHQUAKE

For critical structures the MDE is considered the same as the Maximum Credible Earthquake (MCE). For the Maximum Credible Earthquake (MCE) design ground motions, Engineering Regulation No. 1110-2-1806 of the U.S. Army Corps of Engineers (1995a) specifies that the event should be characterized by a deterministic event that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. Should the Robles Diversion dam be considered a critical structure, then the MDE could be selected as the MCE.

Median and 84th percentile deterministic response spectra are developed for maximum credible earthquakes that may occur on various active or potentially active faults that are located near the dam site. Fault sources and the maximum credible earthquake scenarios for the RDD site include the Santa Ynez fault (M_W7.4 at 5.2 km), San Cayetano fault (M_W 7.2 at 12.1 km), and Mission Ridge-Arroyo Parida-Santa Ana fault (M_W 6.8 at 3.8 km), and the San Andreas fault (M_W 8.1 at 45 km). These faults represent the largest contributions to the ground motion hazard in the PSHA.(Figure 7). The most recent earthquake on each of these faults is known to have occurred during the past 30,000 years (San Andreas fault), or the available information indicates that an earthquake may have occurred on the fault within the past 30,000 years (Santa Ynez, San Cayetano, and Mission Ridge Arroyo Parida-Santa Ana faults; Table 1).

The deterministic spectra were developed using the same weighted attenuation relationships used for the probabilistic analysis. Based on comparison of median deterministic results for the earthquake scenarios noted above (Figure 8; Table 3), the strongest ground motions at the site result from the M_W 7.4 earthquake occurring on the Santa Ynez fault[1]. Therefore, the deterministic spectra for the RDD site are taken as the median deterministic spectra for the Santa Ynez fault. The 84th percentile deterministic spectra also are listed in Table 3 for comparison to the median spectra.

The median and 84th percentile deterministic response spectra are shown along with the OBE, MDE, and additional equal-hazard spectra on Figure 9. The median deterministic response spectrum corresponds to ground motions with a return period between 144 and 475 years in the high-frequency range (less than about 0.4-second period), and corresponds to ground motions with a return period of about 475 years in the intermediate to long period range (greater than 0.4-second period). The 84th percentile response spectrum corresponds to ground motions with a return period between 950 and 2475 years in the high-frequency range (less than about 0.3-second period), and corresponds to ground motions with a return period of about 2475 years in the long period range (greater than 0.3-second period) (Figure 9).

Consideration of potential adjustments to the response spectra for near-field fault rupture effects (directivity and fault normal/fault parallel effects) are described in Section 4.6.

4.6 Modification of Response Spectra for Near-Field Fault Rupture Effects

Two other considerations in developing the design response spectra are near-field fault rupture effects, and the orientation of the structure with respect to the fault normal and fault parallel orientations for the response spectra.

Two kinds of near-field ground motion effects at periods of vibration longer than 0.5 second are associated with large magnitude ($\geq M_W 6.5$) earthquakes occurring on faults. For fault ruptures propagating toward the site, the first effect, termed the directivity effect, produces a large-velocity pulse at the beginning of the strong shaking and a resulting enhancement of the long-period horizontal spectral accelerations. For fault ruptures propagating away from the site, a de-amplification of long-period spectral acceleration is expected to occur.

The second near-field effect, called the fault-normal/fault-parallel effect, produces unequal longer-period ($\geq$ 0.5 second) spectral accelerations between the two orthogonal horizontal components of ground motion. Empirical observations, as well as theoretical considerations, suggest that in the near-field of an earthquake rupture, longer-period ground motion characteristics (e.g., response spectra ordinates, peak ground velocity) tend to be systematically stronger in the direction normal (perpendicular) to the fault strike than in the direction parallel to the fault strike (Somerville et al., 1997). At short periods of vibration (i.e., <0.5 second and including peak ground acceleration), there is no systematic tendency for one horizontal component to be stronger than the other component. These near-field effects described by Somerville et al. (1997) are considered to be significant only for sites located within approximately 15 km of an active fault.

Several recent studies have evaluated near-field effects of fault directivity and average versus maximum spectral demand based on expanded databases of earthquake ground motions (e.g, Spudich and Chiou, 2008; Huang et al., 2008). The results of these evaluations provide updated approaches to modify spectra for near-field effects. However, these studies also note that near-field effects are significant only at periods longer than about 0.5 to 0.6 seconds.

It is our understanding that the period range of interest for the existing Robles Diversion Dam structure is not longer than about 0.5 seconds. Therefore, the site-specific probabilistic and deterministic response spectra shown in Tables 2 and 3 need not be adjusted for near-field effects. If any facilities/construction related to the RDD modification project may be sensitive to

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[1] The deterministic spectra for the San Andreas fault rupture are significantly lower than for the other rupture scenarios; therefore, these spectra are not shown on Figure 8.

longer period motions, it would be appropriate to modify the spectra to account for these effects.

5.0 BASIS FOR RECOMMENDATIONS

This report was prepared for the exclusive use of USACE, the designers of the Robles Diversion Dam Project. The recommendations and other considerations presented in this report are intended for the planning and design of the new high flow bypass spillway and stilling basin included in the overall Robles Diversion Dam Modification Project described in Section 1.0. The recommendations were developed using subsurface information available for the site and our understanding of the site's geologic conditions.

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TABLES

TABLE 1

FAULT SOURCE CHARACTERISTICS

Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

Fault (Slip type, dip, dip direction)	Distance to Site (km)	Rupture Length (km)	Slip Rate (mm/yr)	M_W	Down-dip Width (km)	Rupture Top (km)	Rupture bottom (km)	Most Recent Rupture
Mission Ridge - Arroyo Parida - Santa Ana (r, 60 N)	3.8	69±7	0.4±0.2	7.2	15±2	0	13	Late Pleistocene and Holocene?
Santa Ynez (East) (ll-ss)	5.2	68±7	2±1	7.1	13±2	0	13	Late Pleistocene and Holocene?
San Cayetano (r, 60 N)	12.1	42±4	6±3	7.0	15±2	0	13	Holocene
Red Mountain (r, 60 N)	14	39±4	2±1	7.0	15±2	0	13	Holocene
Oak Ridge Mid Channel structure (r, 28 N)	18	37±4	1±1	6.6	11±2	5	10	Late Pleistocene
Ventura-Pitas Point (r-ll-o, 75 N)	19	40±4	1±0.5	6.9	13±2	1	14	Holocene
Big Pine (ll-ss)	23	41±4	0.8±0.8	6.9	13±2	0	13	Late Pleistocene
Oak Ridge (onshore) (r, 65 S)	24	49±5	4±2	7.0	14±2	1	14	Holocene
Santa Ynez (West) (ll-ss)	32	65±7	2±1	7.1	13±2	0	13	Late Pleistocene
Oak Ridge (blind thrust offshore) (r, 30 S)	33	39±4	3±3	7.1	20±2	5	15	Late Pleistocene
Simi-Santa Rosa (ll-r-o, 60 N)	33	40±4	1±0.5	7.0	15±2	1	14	Holocene
North Channel Slope (r, 26 N)	36	68±7	2±2	7.4	23±2	10	20	Late Pleistocene
San Andreas (Carrizo) (rl-ss)	45	146±15	34±3	7.7	12±2	0	12	Holocene
San Andreas (1857)	45	307	32.7±4.7	8.1	12±2	0	12	
San Andreas (All southern segments)	45	509	28.8±5.1	8.4	13.3±2	0	12	
San Andreas (Mojave) (rl-ss)	76	103±10	30±7	7.65	12±2	0	12	

TABLE 1

FAULT SOURCE CHARACTERISTICS
Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

Fault (Slip type, dip, dip direction)	Distance to Site (km)	Rupture Length (km)	Slip Rate (mm/yr)	M_W	Down-dip Width (km)	Rupture Top (km)	Rupture bottom (km)	Most Recent Rupture
San Gabriel (rl-ss)	47	72±7	1±0.5	7.2	13±2	0	13	Late Pleistocene and Holocene
Channel Islands thrust (r, 17N)	49	63±6	1.5±1	7.5	34±2	5	15	Late Pleistocene
Holser (r, 65 S)	49	20±2	0.4±0.4	6.5	14±2	0	13	Late Pleistocene
Santa Susana (r, 55 N)	50	27±3	5±2	6.7	16±2	0	13	Late Pleistocene
Garlock (West) (ll-ss)	52	98±10	6±3	7.3	12±2	0	12	Holocene
Pleito (r, 45 S)	53	44±4	2±1	7.0	15±2	0	11	Holocene
Northridge (r, 42 S)	53	31±3	1.5±1	7.0	22±2	5	20	Holocene
Santa Cruz Island (ll-r-o)	54	50±5	1±0.5	7.0	13±2	0	13	Holocene
Malibu Coast (ll-r-o, 75 N)	55	37±4	0.3±0.2	6.7	13±2	0	13	Late Pleistocene and Holocene
Anacapa-Dume (r,ll-o 45 N)	56	75±8	3±2	7.5	28±2	0	20	Late Pleistocene

Notes:
1. Fault rupture parameters from 2002 California Geological Survey Seismic Hazard Model (http://www.consrv.ca.gov/cgs/rghm/psha/Pages/index.aspx) and 2008 Working Group on California Earthquake Probabilities (WGCEP, 2008).
2. Recency of fault activity based on Jennings (1994), and SCEC (http://www.data.scec.org/faults/faultmap.html#MAP).

TABLE 2

COMPARISON OF HORIZONTAL SITE-SPECIFIC EQUAL HAZARD RESPONSE SPECTRA

Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

Period (seconds)	Spectral Acceleration (g)			
	50% P_E in 100 Years (OBE)	10% P_E in 50 Years	10% P_E in 100 Years (MDE)	2% P_E in 50 Years
0.03 (PGA)	0.318	0.511	0.633	0.822
0.05	0.405	0.653	0.814	1.067
0.1	0.594	0.960	1.176	1.535
0.2	0.732	1.164	1.450	1.965
0.3	0.686	1.118	1.406	1.931
0.4	0.607	1.029	1.298	1.789
0.5	0.533	0.914	1.172	1.611
0.75	0.384	0.664	0.867	1.187
1	0.290	0.506	0.662	0.920
2	0.124	0.222	0.295	0.413
3	0.073	0.127	0.169	0.238
4	0.050	0.089	0.116	0.162

Notes:
1. Spectra are five-percent damped, except for PGA.
2. P_E – Probability of exceedance.

TABLE 3

COMPARISON OF HORIZONTAL SITE-SPECIFIC DETERMINISTIC RESPONSE SPECTRA

Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

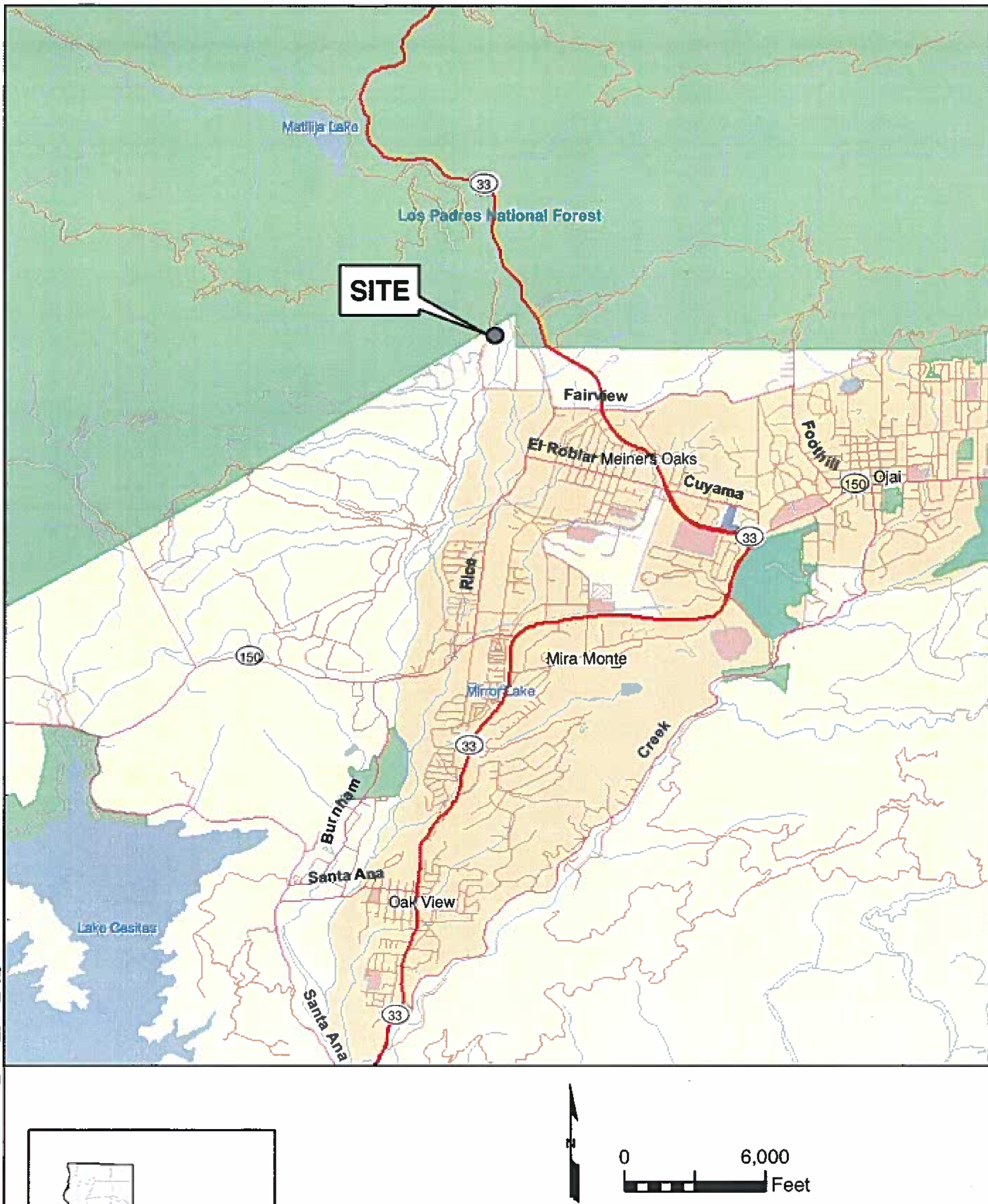
Period (seconds)[1]	Median Deterministic Spectral Acceleration (g)			
	M 7.4 Santa Ynez flt at 5.2 km	M6.8 MR-AR-SA flt at 3.8 km	M7.2 San Cayetano flt at 12.1 km	M8.1 San Andreas flt at 45 km
0.03 (PGA)	0.509	0.463	0.289	0.166
0.05	0.579	0.531	0.330	0.184
0.1	0.810	0.746	0.474	0.247
0.2	1.042	0.966	0.611	0.318
0.3	1.036	0.945	0.589	0.323
0.4	0.978	0.884	0.536	0.300
0.5	0.887	0.790	0.477	0.278
0.75	0.672	0.570	0.354	0.227
1	0.527	0.429	0.272	0.188
2	0.257	0.177	0.121	0.105
3	0.158	0.102	0.073	0.072
4	0.108	0.069	0.051	0.052

Period (seconds)[1]	84th Percentile Deterministic Spectral Acceleration (g)			
	M 7.4 Santa Ynez flt at 5.2 km	M6.8 MR-AR-SA flt at 3.8 km	M7.2 San Cayetano flt at 12.1 km	M8.1 San Andreas flt at 45 km
0.03 (PGA)	0.859	0.789	0.493	0.287
0.05	0.987	0.913	0.569	0.321
0.1	1.397	1.299	0.830	0.439
0.2	1.819	1.700	1.079	0.567
0.3	1.850	1.699	1.059	0.584
0.4	1.769	1.608	0.973	0.548
0.5	1.630	1.458	0.878	0.513
0.75	1.261	1.075	0.665	0.427
1	0.998	0.816	0.516	0.357
2	0.501	0.346	0.236	0.204
3	0.310	0.199	0.143	0.141
4	0.215	0.136	0.100	0.102

Notes:

1. Spectra are five-percent damped, except for PGA.

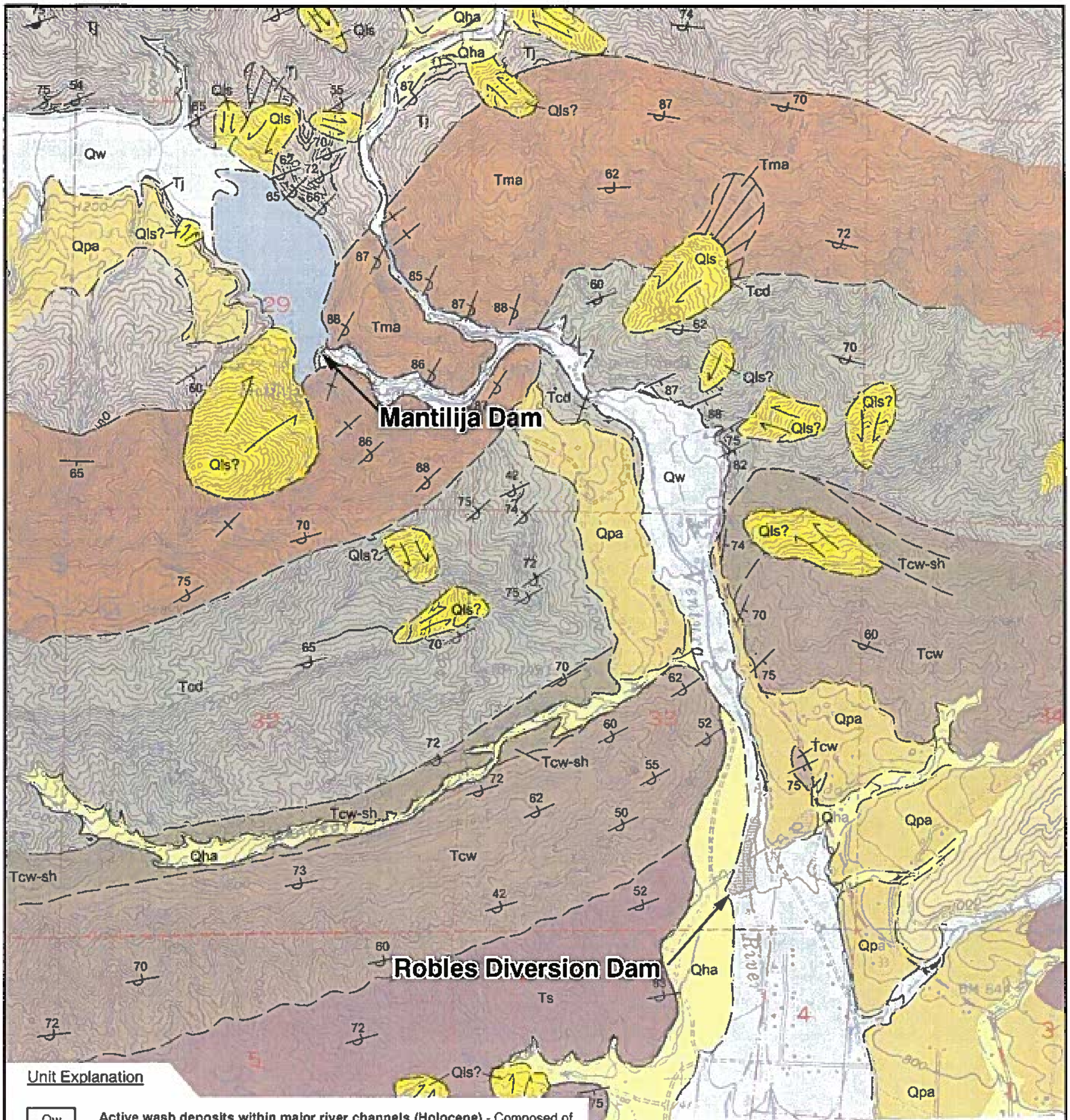
FIGURES



SITE LOCATION MAP
Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

By:	Date: 11/10/2008	Project No. 9993.003
AMEC Geomatrix		Figure 1

File path: S:\9900\9993\9993.003\9993.003.2208_1107_ghe_fig_01.mxd



Unit Explanation

Qw	**Active wash deposits within major river channels (Holocene)** - Composed of unconsolidated silt, sand and gravel.
Qha	**Alluvial and colluvial deposits, undivided (Holocene)** - Located on the floors of valleys; includes active stream deposits in hill slope areas; composed of unconsolidated sandy clay with some gravel.
Qls	**Landslide deposits (Holocene to late Pleistocene)** - Includes numerous active landslides, composed of weathered, broken up rocks; extremely susceptible to renewed landsliding, including their head scarp areas.
Qpa	**Alluvial deposits, undivided (late Pleistocene)** - Consists of semi-consolidated silt, sand, clay, and gravel.
Ts	**Sespe Formation (Oligocene)** - Composed of sandstone; locally pebbly, siltstone and claystone; rocks are generally reddish in color.
Tcw / Tcw-sh	**Coldwater Sandstone (late Eocene)** - Composed of hard arkosic sandstone with siltstone and shale interbeds; locally reddish in color, similar to appearance of Sespe Formation. Tcw-sh consists predominantly of shale.
Tma	**Matilija Sandstone (middle to late Eocene)** - Composed of hard arkosic sandstone with micaceous shale interbeds.
Tj	**Juncal Formation (early to middle Eocene)** - Consists of micaceous shale with arkosic sandstone interbeds; generally susceptible to landsliding.

0 1000 2000 Feet

Contour Interval 40 Feet
Dotted Lines Represent Half-Interval Contours
From Tan and Jones (2006)

GEOLOGIC MAP
Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

By: –	Date: 11/2008	Project No. 9993.003
AMEC Geomatrix		Figure 2

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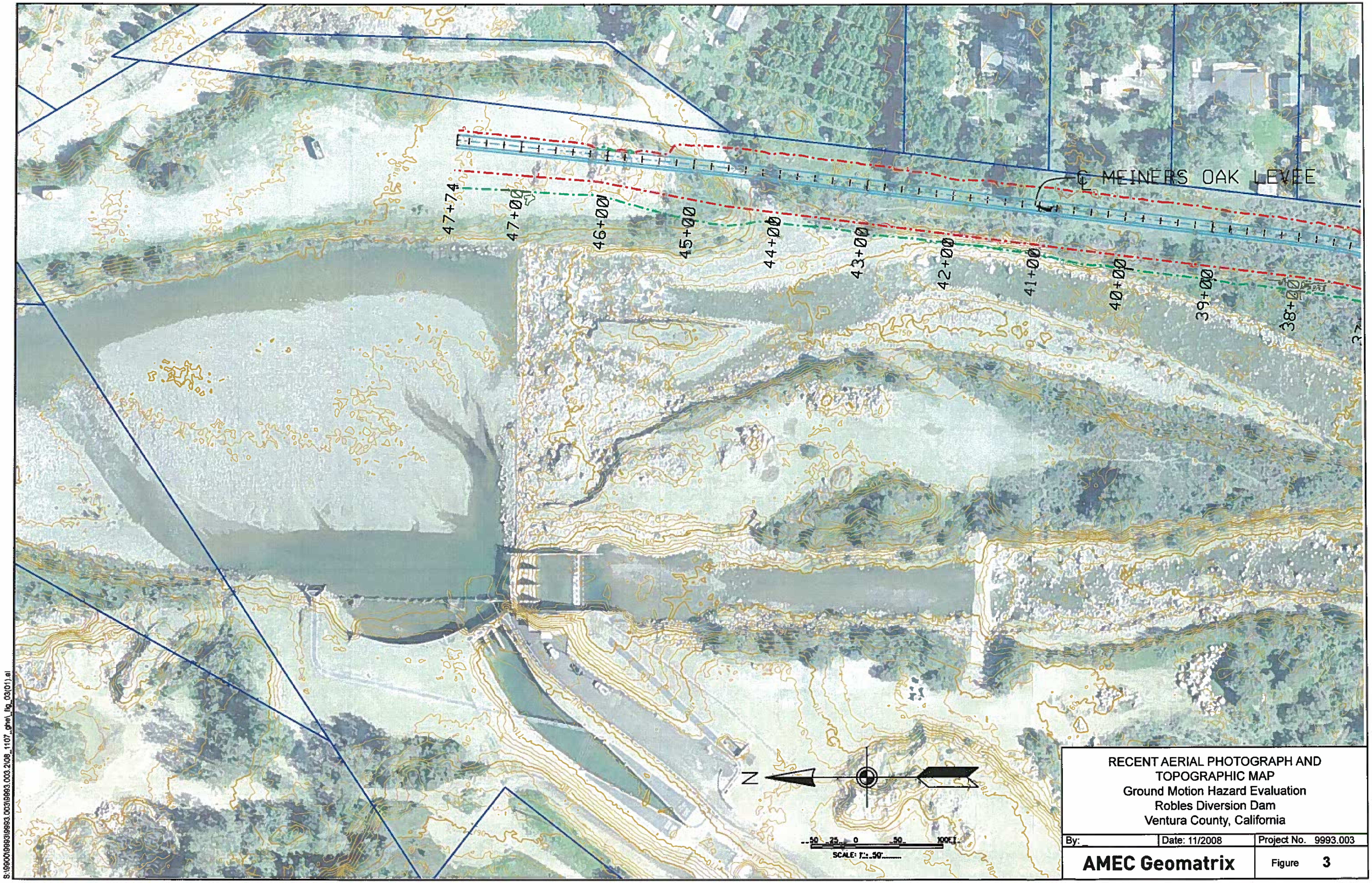

RECENT AERIAL PHOTOGRAPH AND TOPOGRAPHIC MAP
Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

By:	Date: 11/2008	Project No. 9993.003
AMEC Geomatrix		Figure 3

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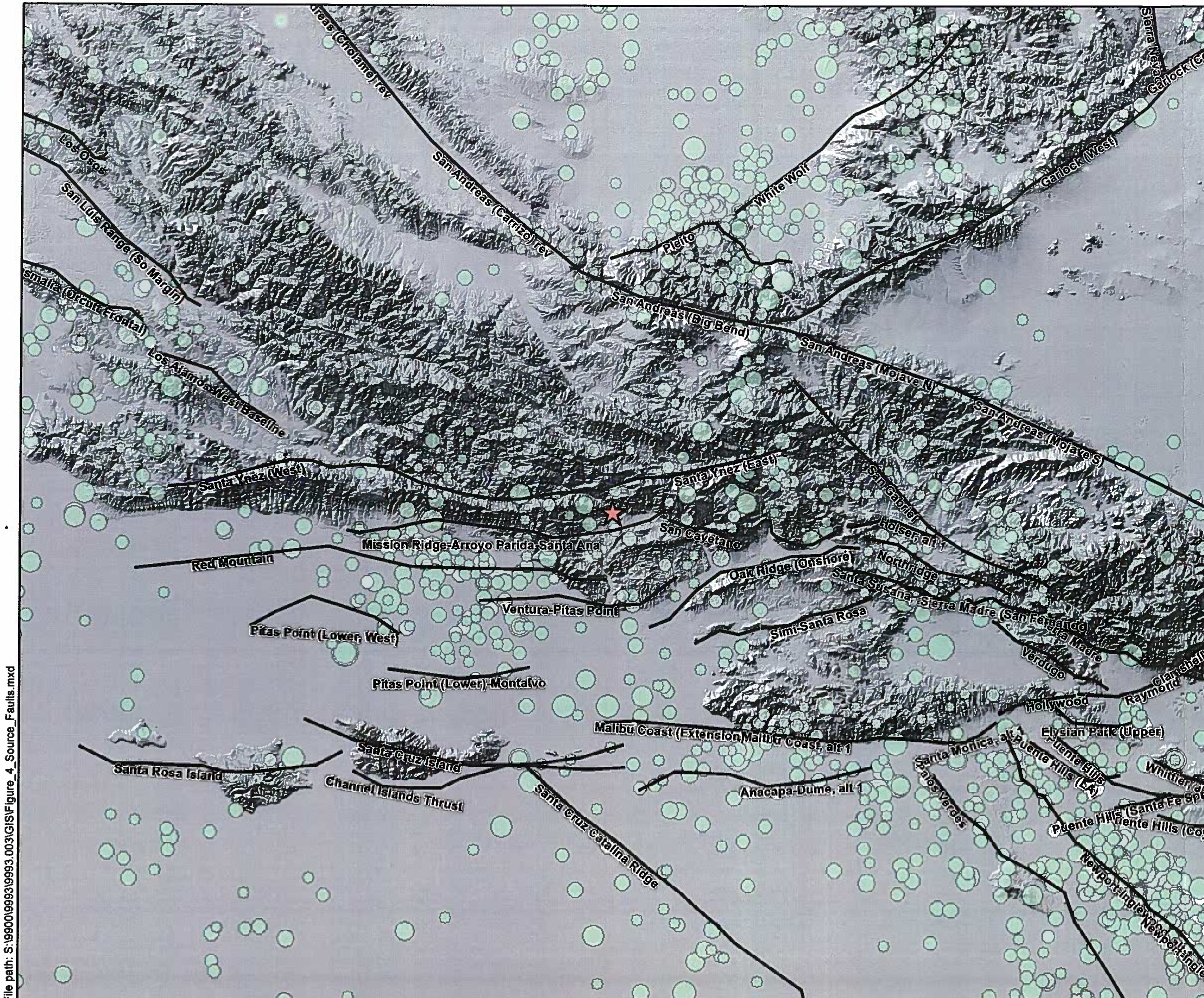


Legend

★ Robles Diversion Dam

Independant Seismicity
(1769-2008)

- 3.0
- 4.0
- 5.0
- 6.0
- 7.0
- 8.0

Fault Sources from CGS (2002)
and WGCEP (2008)

N

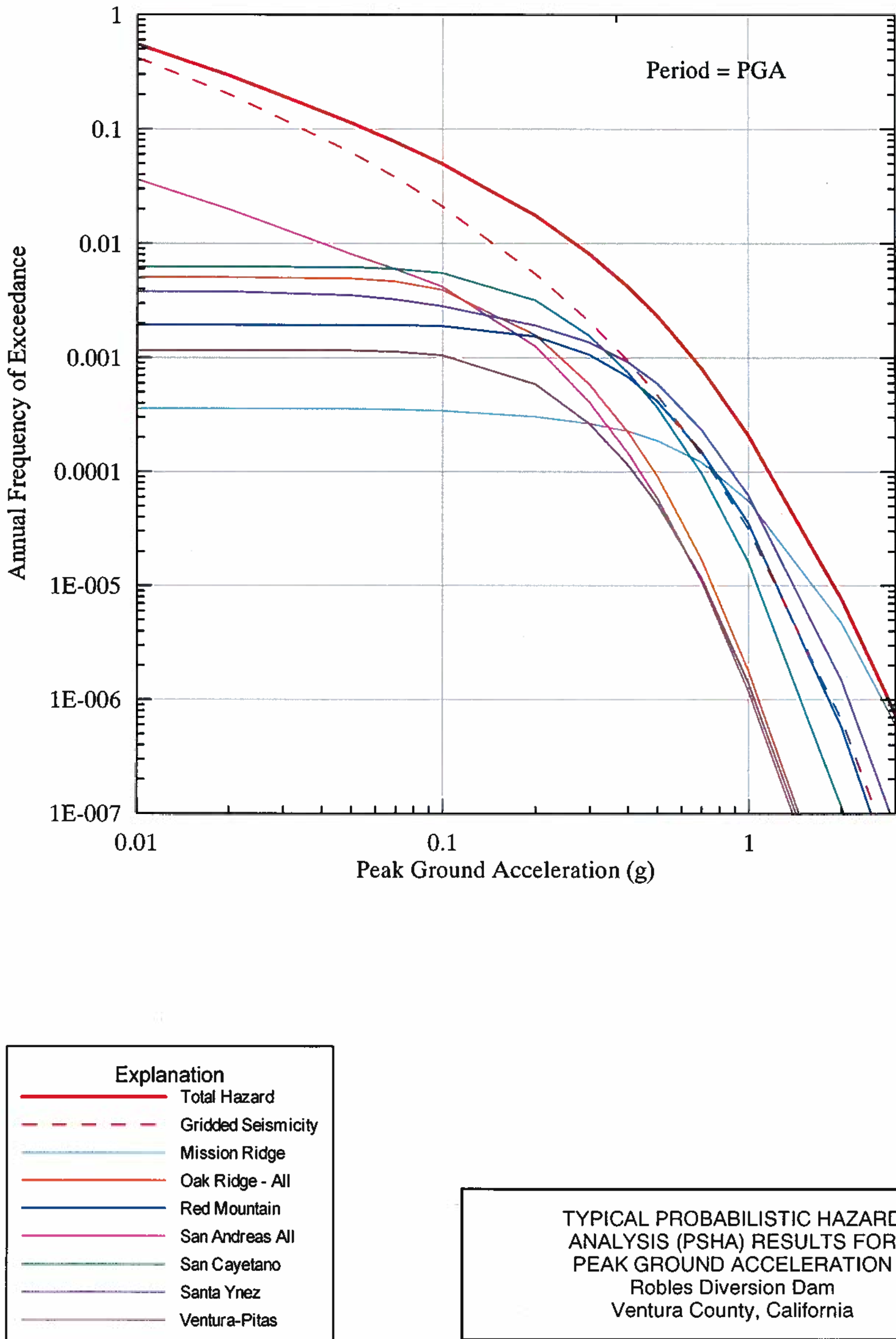
0 50 Kilometers

Scale 1:1,000,000

REGIONAL FAULT AND HISTORIC SEISMICITY MAP
Ground Motion Hazard Evaluation
Robles Diversion Dam
Ventura County, California

By: AL	Date: 11/9/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 4

File path: S:\9900\9993\9993.003\GIS\Figure_4_Source_Faults.mxd



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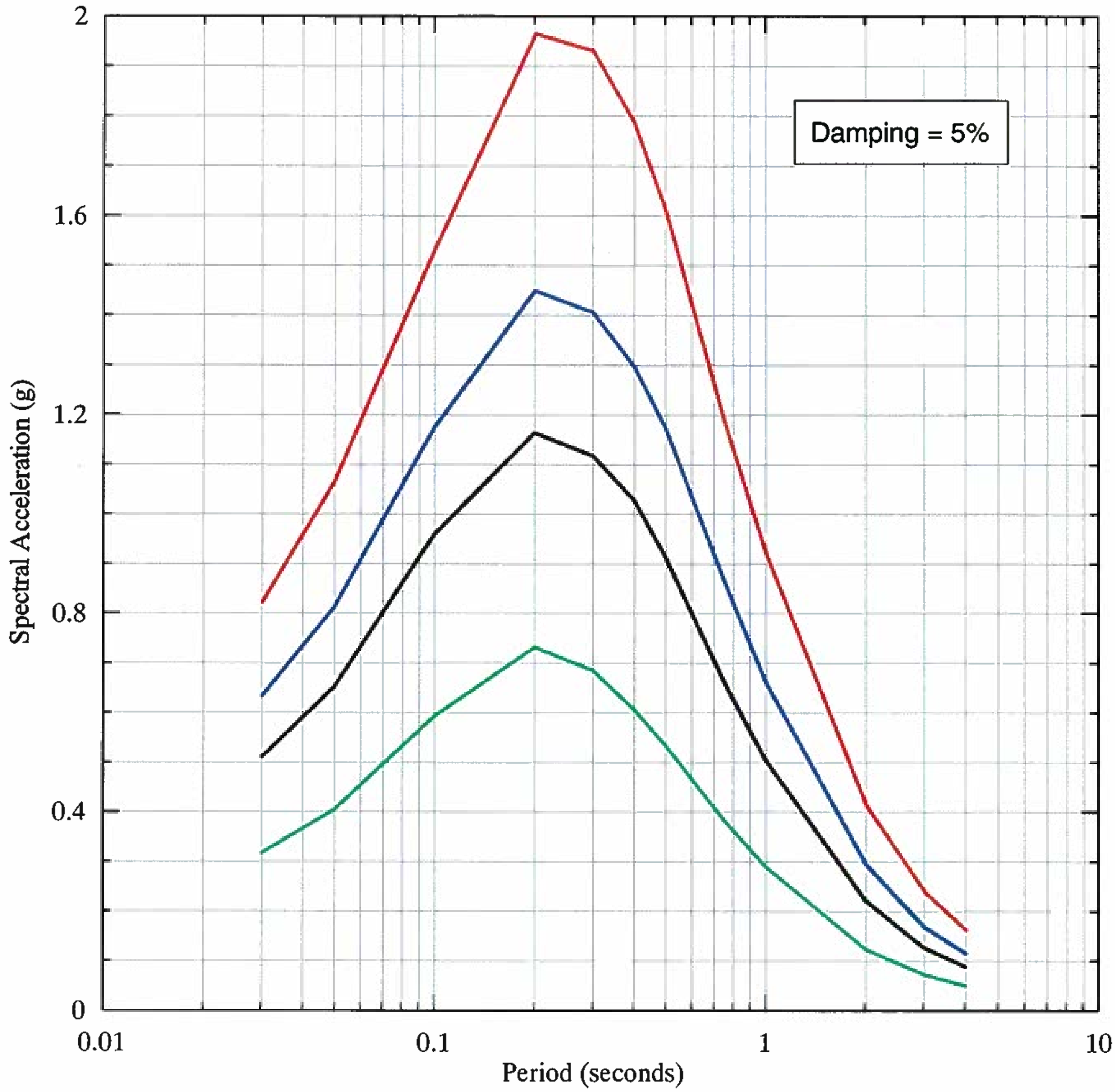
TYPICAL PROBABILISTIC HAZARD ANALYSIS (PSHA) RESULTS FOR PEAK GROUND ACCELERATION
Robles Diversion Dam
Ventura County, California

By: TA	Date: 11/12/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 5



Explanation

- 2% PE in 50 Years
- 10% PE in 100 Years (MDE)
- 10% PE in 50 Years
- 50% PE in 100 Years (OBE)

COMPARISON OF HORIZONTAL
FIVE PERCENT DAMPED MEAN
EQUAL HAZARD RESPONSE SPECTRA
Robles Diversion Dam
Ventura County, California

By: TA	Date: 11/12/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 6

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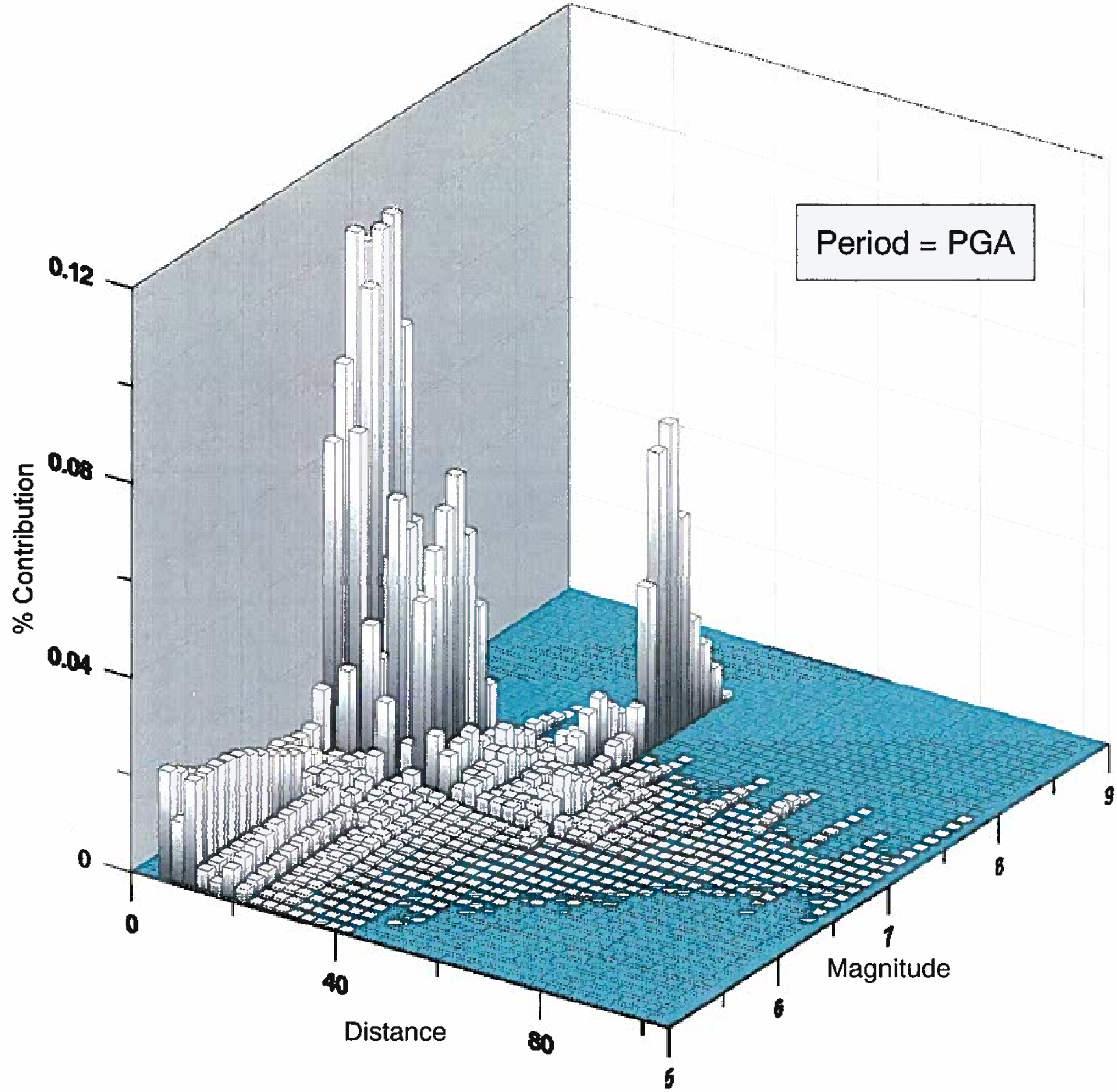

RELATIVE CONTRIBUTIONS FROM EARTHQUAKES IN DIFFERENT MAGNITUDE AND DISTANCE INTERVALS TO HAZARD FOR PEAK GROUND ACCELERATION Robles Diversion Dam Ventura County, California		
By: TA	Date: 11/12/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 7

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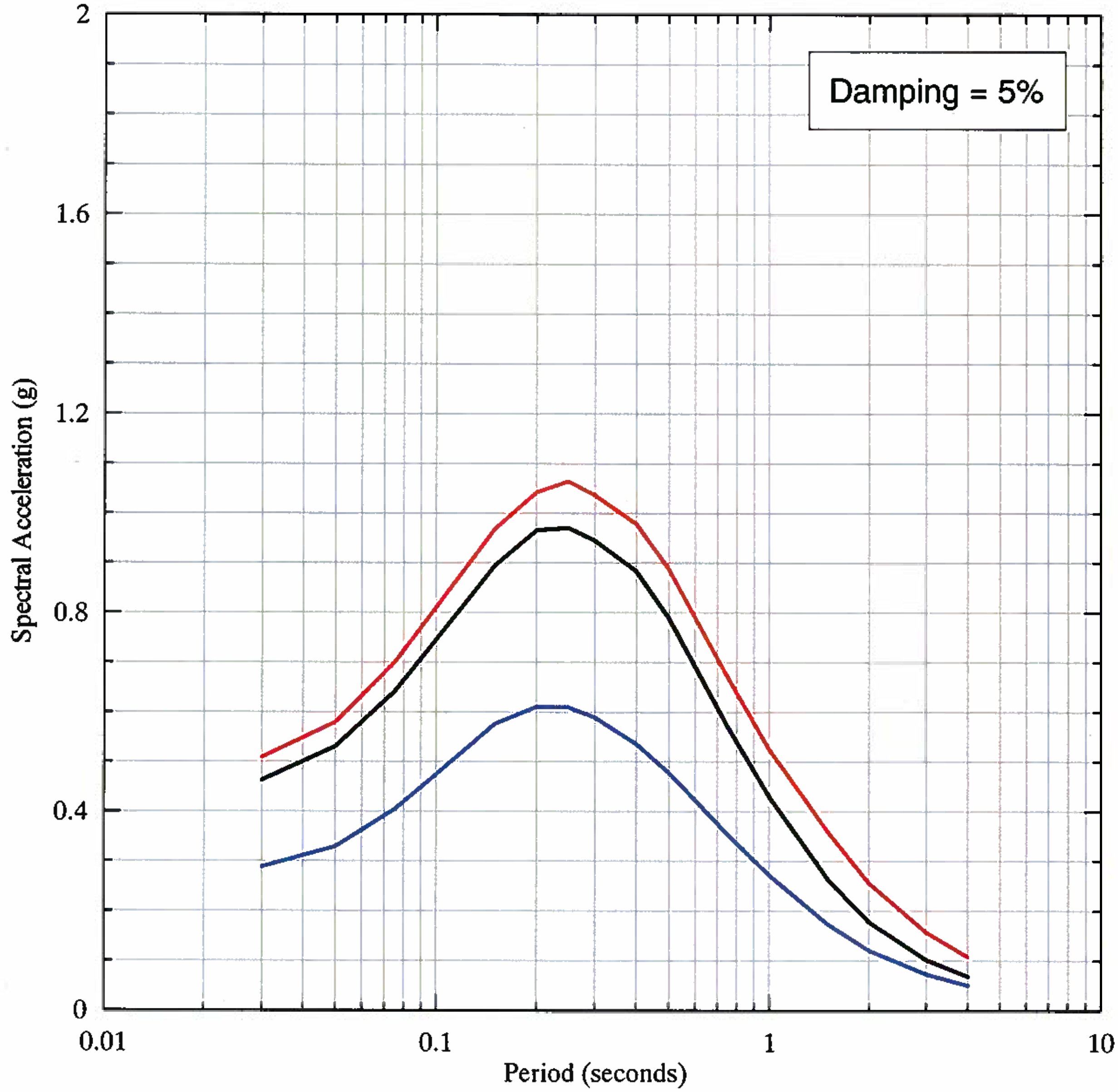

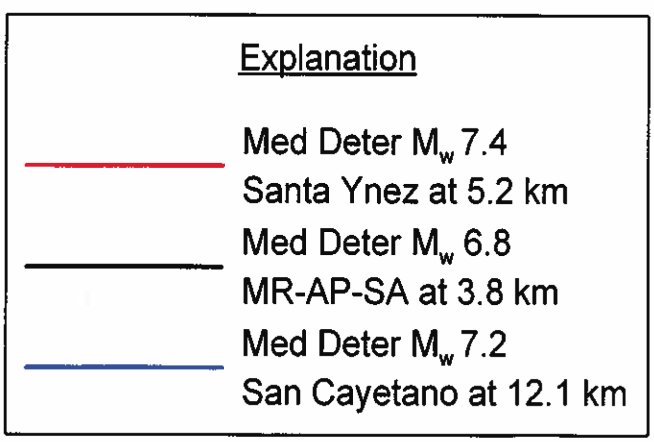

COMPARISON OF HORIZONTAL MEDIAN DETERMINISTIC RESPONSE SPECTRA FOR ACTIVE AND POTENTIALLY ACTIVE FAULTS
Robles Diversion Dam
Ventura County, California

By: TA	Date: 11/12/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 8

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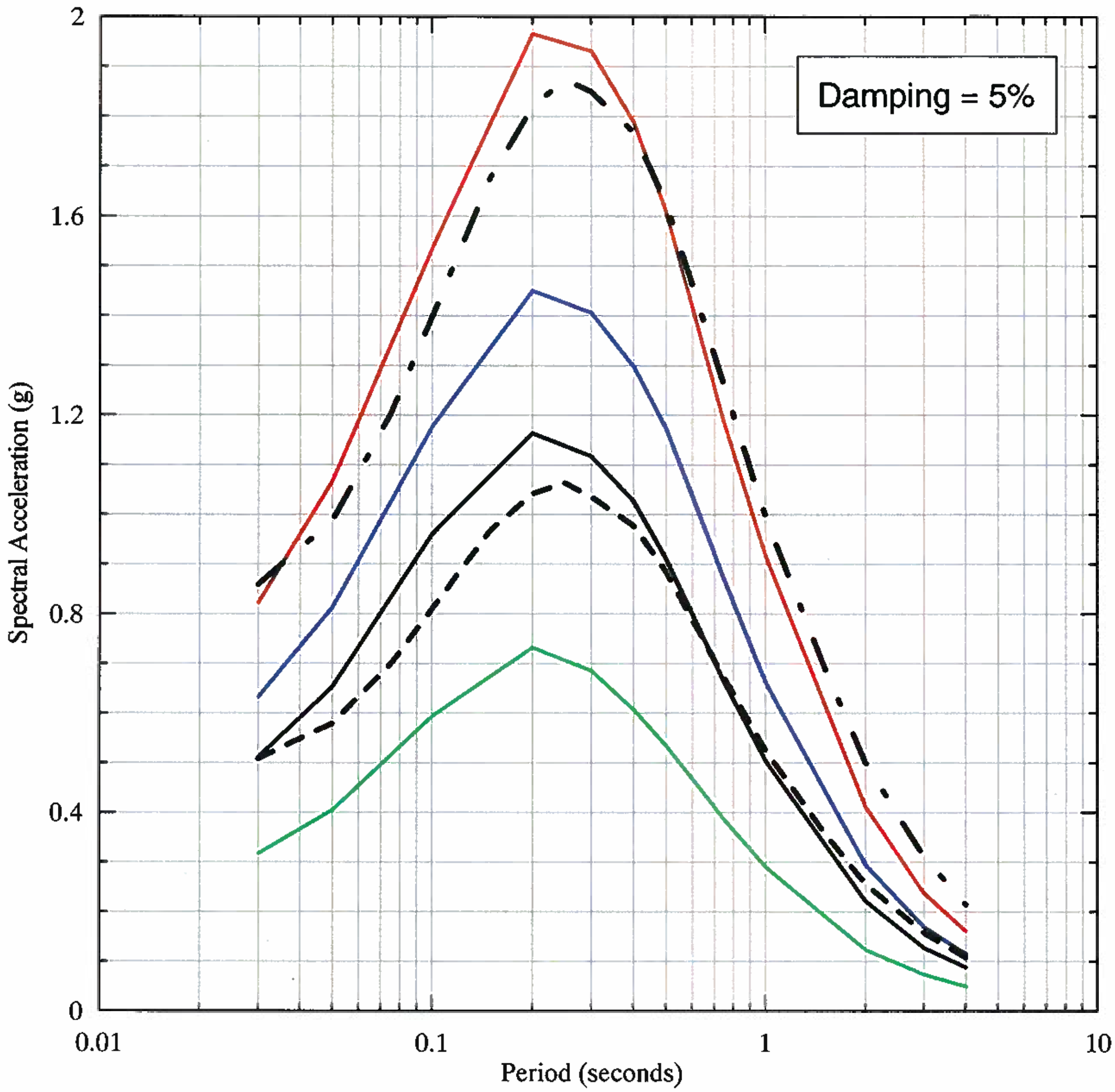

Explanation

- 2% PE in 50 Years
- 10% PE in 100 Years (DBE)
- 10% PE in 50 Years
- 50% PE in 100 Years (OBE)
- Median Deterministic M7.4 on Santa Ynez
- 84th Percentile Deterministic M7.4 on Santa Ynez

COMPARISON OF EQUAL HAZARD AND DETERMINISTIC RESPONSE SPECTRA
Robles Diversion Dam
Ventura County, California

By: TA	Date: 11/12/2008	Project No. 9993.003.2
AMEC Geomatrix		Figure 9

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