Matilija Dam

Ecosystem Restoration Project

Robles Diversion Dam Modification
90% Design Documentation Report

Ventura County, California

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SYLLABUS

This Design Documentation Report presents the design modifications for the Robles Diversion Dam, part of the Matilija Dam Ecosystem Restoration Project. The design presented herein follows what is presented in the *Matilija Dam Ecosystem Restoration Feasibility Study* and the *Final Environmental Impact Statement/Environmental Impact Report*, dated July 2004.

This project is being developed under the authority of the Resolution of the U.S. House of Representatives Committee on Transportation and Infrastructure (Docket 2593), adopted 15 April 1999. The project local sponsor is the Ventura County Watershed Protection District.

The existing Robles Diversion Dam is located on the Ventura River, approximately 14 miles from the mouth of the river and 2 miles downstream of the Matilija Dam, in an unincorporated portion of Ventura County, California. It is owned by the U.S. Bureau of Reclamation and operated by the Casitas Municipal Water District under a highly regulated diversion schedule, affected by the highly variable river flows, large sediment loads, downstream water rights, and minimum flows to maintain fish passage. The Ventura River is designated as critical habitat for the endangered steelhead trout (*Eucyclogobius newberryi*).

When the Matilija Dam is removed, a significant increase in the sediment load is anticipated, which would adversely affect the operation of the Robles Diversion Dam. The proposed dam modifications would reduce the adverse impact on dam operations. The modifications are based on the selected alternative in the DPR (Alternative 4b) and consist of a high-flow bypass (HFB) spillway with four 30-foot Tainter gates, a stilling basin, and a high-flow fish bypass. Additionally, the existing dam embankments will be raised and a rock-armored embankment will be provided. The construction of the HFB and appurtenances is a mitigation component of the overall Matilija Dam Ecosystem Restoration project. The only deviations from the selected alternative are the addition of the fish bypass, as required on the basis of coordination with the National Marine Fisheries Service, and the rock ramp channel.

REPORTS PREVIOUSLY ISSUED

Project-related reports previously issued by the U.S. Army Corps of Engineers and others are the following:

- *Final Environmental Impact Statement/Environmental Impact Report for the Matilija Dam Ecosystem, Ventura, California*, U.S. Army Corps of Engineers, December 2004.

- *Matilija Dam Ecosystem Restoration Feasibility Study, Ventura, California*, U.S. Army Corps of Engineers, December 2004.

- *Matilija Dam Ecosystem Restoration Project Management Plan—Design Phase, Ventura, California*, U.S. Army Corps of Engineers, June 2005.

- *Hydrology, Hydraulics and Sediment Studies of Alternatives for the Matilija Dam Ecosystem Restoration Project, Ventura, California—Final Report,* Technical Service Center, U.S. Bureau of Reclamation, Denver, CO 80225, Greimann, B.P., 2004.

- *Hydrology, Hydraulics, and Sediment Studies for the Matilija Dam Ecosystem Restoration Project, Ventura, California—Draft Report*, Technical Service Center, U.S. Bureau of Reclamation, Denver, CO 80225, Greimann, B.P., 2004.

- *Robles Diversion Dam High Flow and Sediment Bypass Structure, Ventura, California, Physical Model Study*, Hydraulic Laboratory Report HL-2008-7, Technical Service Center, U.S. Bureau of Reclamation, Denver, CO, Mefford, B., Stowell, H., and Heinje, C., 2008.

- *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc., Oakland, CA, 12 November 2008.

- *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc., Oakland, CA, 19 January 2009.

- *Foundation Report for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc., Oakland, CA, 8 August 2008.

- *Foundation Report for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc., Oakland, CA, 2 August 2012.

- *Robles Diversion Dam Left Bank Fishway, Ventura, California*, Hydraulic Laboratory Report HL-2011-04, Technical Service Center, U.S. Bureau of Reclamation, Denver, CO, Mefford, B., 22 June 2010.

- *Memorandum for Matilija Dam Ecosystem Restoration Study, Project Deliver Team, U.S. Army Corps of Engineers, Los Angeles District, 3 April 2009.*

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Partial set of USBR Construction Specifications for the Robles Diversion (provided by CMWD staff).

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PERTINENT DATA

The purpose of the Robles Diversion Dam is water diversion. Pertinent data related to the dam are provided below.

Item	Description
Drainage area	74 square miles
100-year peak discharge at Robles Diversion Dam	27,100 cfs
20-year peak discharge at Robles Diversion Dam	19,000 cfs
10-year peak discharge at Robles Diversion Dam	15,000 cfs
Existing gate/spillway structure capacity (one 10-by-9.5-foot and three 16-by-9.5-foot radial gates)	6,000 cfs
Proposed gate/spillway structure capacity (four 30-by-12-foot radial gates)	11,000 cfs
Rock Ramp design flow rate	11,000 cfs
Robles Diversion Dam design capacity[1]	19,000 cfs
Existing diversion canal capacity (three 11-by-10.5-foot radial gates)	500 cfs
Existing crest elevation	767.00 feet[2] +/-
Existing crest length	350 feet
Proposed crest elevation	769.00 feet[2]
Proposed crest length	150 feet

cfs = cubic feet per second

[1] Design capacity is based on U.S. Army Corps of Engineers *Memorandum for Matilija Dam Ecosystem Restoration Study, Project Delivery Team*, 3 April 2009.

[2] Elevation is relative to NAVD 88

ABBREVIATIONS AND ACRONYMS

Amp	ampere
CMWD	Casitas Municipal Water District
DBE	design basis earthquake
DDR	Design Documentation Report
DL	dead load
EIS	Environmental Impact Statement
EM	Engineer Manual
ER	Engineer Regulation
ETL	Engineer Technical Letter
Fpm	feet per minute
ft/s	feet per second
HFB	high-flow bypass
HP	horsepower
Kip	1,000 pounds
Ksi	kips per square inch
kVA	kilovoltampere
kW	kilowatt
lbs/ft^3	pounds per cubic foot
LL	live load
MDE	maximum design earthquake
MDF	maximum design flood
OBE	operating basis earthquake
PGA	peak ground acceleration
Psf	pounds per square foot
Psi	pounds per square inch
UPS	uninterruptible power supply
USBR	U.S. Bureau of Reclamation
USACE	U.S. Army Corps of Engineers
V	volt
WSE	water surface elevation

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1. INTRODUCTION

The U.S. Army Corps of Engineers (USACE), Los Angeles District, in conjunction with the USBR, CMWD, and the Ventura County Water Protection District, completed the Feasibility Study and Environmental Impact Statement (EIS) for the Matilija Dam Ecosystem Restoration Project in December 2004. The recommended plan addressed the increased sediment supply and impacts on the existing Robles Diversion Dam that will result from the removal of the Matilija Dam upstream of the diversion dam. It proposed the construction of a high-flow bypass (HFB) spillway consisting of four 30-foot-wide-by-12-foot-high Tainter gates, a stilling basin, and a high-flow fish bypass. Additionally, the existing dam embankments will be raised to elevation 769 feet and a rock ramp and rock armored embankment will be provided. The plan provides a 20-year level of protection for the diversion structure and is a sediment mitigation component of the overall Matilija Dam Ecosystem Restoration project.

Numerical and physical model studies were conducted by the USBR to verify the proposed HFB layout, sizes, and location (see Appendix B). The physical model included the addition of a fish bypass structure, resulting in the final design recommendations.

PROJECT AUTHORIZATION

The Robles Diversion Dam Modification project is being implemented in response to the Resolution of the U.S. House of Representatives Committee on Transportation and Infrastructure (Docket 2593), adopted 15 April 1999, which reads as follows:

> *Resolved by the Committee on Transportation and Infrastructure of the United States House of Representatives, that the Secretary of the Army is requested to review the report of the Chief of Engineers on the Ventura River, Ventura County, California, published as House Document 323, 77th Congress, 1st Session, and other pertinent reports, with a view to determining whether any modifications of the recommendations contained therein are advisable at this time, in the interest of environmental restoration and protection, and related purposes, with particular attention to restoring anadromous fish populations on Matilija Creek and returning natural sand replenishment to Ventura and other Southern California beaches.*

PURPOSE

The purpose of this DDR is to provide the basis for the design of the Robles Diversion Dam Modification flood control project along the Ventura River. The project purpose is to mitigate the increased sediment loading and flood flows resulting from the removal of the Matilija Dam approximately 2 miles upstream. The project will provide protection from floods up to the 20-year flood.

SCOPE OF STUDIES

This DDR presents the design for the recommended plan, the estimated construction cost, and the schedule for the Robles Diversion Dam Modification project. The Robles Diversion Dam, which was originally built in 1958, diverts water from the Ventura River into Casitas Reservoir. A fish ladder was completed in the fall of 2005 to maintain fish passage (the Ventura River is designated as critical habitat for the endangered steelhead trout [*Eucyclogobius newberryi*]) upstream of the Robles Diversion Dam.

The existing Robles Diversion Dam consists of an approximately 10-foot-high-by-300-foot-wide in-channel embankment, a gate-controlled bypass structure for the Ventura River (one 10-by-9.5-foot radial gate and three 16-by-9.5-foot radial gates), a gate-controlled canal diversion structure with a debris barrier (three 11.5-by-10.5-foot radial gates), and a fish ladder (Figure 1.1).

Image from MSN Live Search

Figure 1.1 Existing Robles Diversion Dam

The recommended plan, to mitigate the large increases in sediment resulting from the removal of the Matilija Dam, includes the design and construction of an HFB spillway consisting of four 30-foot-wide-by-12-foot-high Tainter gates, a USBR stilling basin, and an additional high-flow fish bypass. To accommodate the additional fish ladder and improve operations, the existing dam embankments will also be raised to elevation 769 feet. The embankment will be armored to protect it from overtopping, and a rock ramp channel bed will be provided to protect the diversion dam from scour damage. The rock-armored embankment will increase the storm capacity of the diversion dam to a 20-year level of protection.

Surveying and Mapping

The mapping is based on aerial topography using the light detection and ranging (lidar) method in February 2005, with a scale of 1 inch equal to 100 feet and 2-foot contours. In March 2009, a detail field survey of the existing diversion structure and existing embankment was performed to supplement the 2005 topography and as-built drawings for the existing features. Horizontal control is based on the North American Datum (NAD) of 1983, 1986 adjustment, California transverse Mercator projection, east zone. Vertical control is based on the North American Vertical Datum (NAVD) of 1988.

Site Explorations

Subsurface investigations were performed by separate consultants under contract to the USACE for the design of the Robles Diversion Dam Modification project. They are described in Appendix C.

Coordination with Others

Extensive coordination of the design of the project was conducted. Items discussed included mapping, as-built plans, rights-of-way, easements, utility relocations, quantities of treated waste and excess water currently being discharged into Matilija Creek, dam safety considerations, and potential sources of water, disposal sites, and maintenance features.

USACE coordinated with the local sponsor for the project (Ventura County Water Protection District) whose contact information is the following:

Ms. Norma Camacho
Ventura County Watershed Protection District
800 S. Victoria Avenue
Ventura, CA 93009-1600

Rights-of-way. The boundaries of the project are fairly well defined along existing county rights-of-way and easements. The plans are based on topographic mapping created with the use of aerial photography performed in 2005. Rights-of-way requirements will be established in detail before the plans and specifications are completed.

Utility relocations. Utility relocations required for the project were determined by the project team. Interfering utilities include electrical lines and telephone lines. Where possible, relocations will be accomplished in advance of the construction.

Other relocations and modifications. A number of structures will be removed as a result of this project, including the existing concrete V-notched low-flow roadway crossing at the downstream end of the rock-armored ramp spillway. A new concrete low-flow crossing is proposed to replace the existing structure to be removed. Additionally, minor modifications to the existing fish ladder will be performed to accommodate the higher elevation in the stilling basin invert.

Maintenance items. Required maintenance features have been coordinated with the local sponsor and the project team. A 20-foot-wide roadway will be provided for maintenance access into each end of the existing diversion dam. The roadway will connect with the access roads provided for the Meiners Oaks Levee improvements. The existing seasonal low-flow crossing will be removed and replaced with a 20-foot-wide concrete structure that will also be used as a grade control structure for the rock ramp channel. Existing all-weather maintenance and access roads will remain in place without modification.

USACE also coordinated with the USBR, the U.S. Fish and Wildlife Service, the National Marine Fisheries Service, and the CMWD.

PROJECT LOCATION AND DESCRIPTION OF DRAINAGE AREA

The Robles Diversion Dam is located on the Ventura River, approximately 14 miles from the mouth of the river and 2 miles downstream of the Matilija Dam, in an unincorporated portion of Ventura County, California. The diversion dam is owned by the USBR and operated by the CMWD under a highly regulated diversion schedule, affected by the highly variable river flows, large sediment loads, downstream water rights, and minimum flows to maintain fish passage. The project area is located along the Ventura River and Matilija Creek in Ventura County (Figure 1.2).

Figure 1.2 Project Location Map

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2. SELECTED PLAN

The selected plan for the Robles Diversion Dam Modification project consists of adding an HFB structure with four 30-by-12-foot radial gates adjacent to the existing spillway structure (consisting of one 10-by-9.5-foot and three 16-by-9.5-foot radial gates). An additional fish passage will be constructed between the rock armored embankment and the HFB structure. The fish passage is proposed to allow the migration of the endangered steelhead trout (*Eucyclogobius newberryi*) during large flow events and will be designed as a streaming flow fishway. To increase the operating efficiency of the diversion structure and fishway, the existing embankment will be raised by approximately 2 feet. A concrete sill will be placed across the crest of the raised embankment to control the weir elevation and the forebay depth. Since the existing gates are only 9.5 feet high, a 2-foot extension will be connected to the existing gates to increase their depth capacity. A rock-armored ramp will be placed to approximately 400 feet downstream of the existing spillway structure and the proposed HFB structure. It will be designed to protect the downstream channel and focus the outlet flows to one stream, which will help to prevent stranding of fish as they migrate upstream. Figures 2.1 and 2.2 show the proposed upstream elevation for the pre-project and post-project layouts.

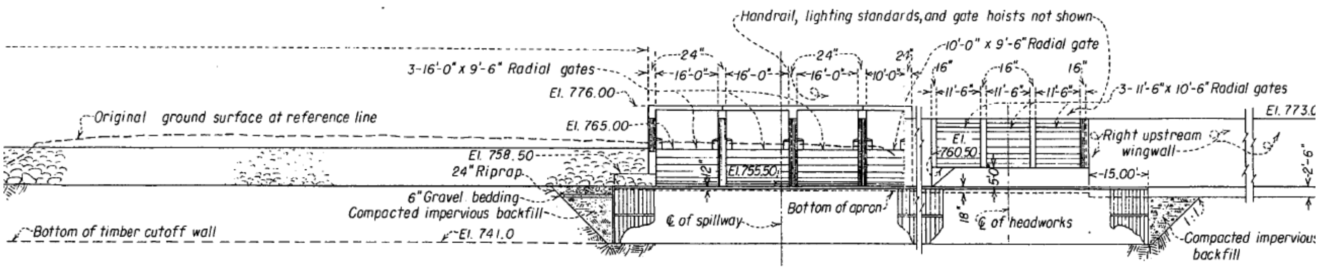

Figure 2.1 Pre-Project Elevation

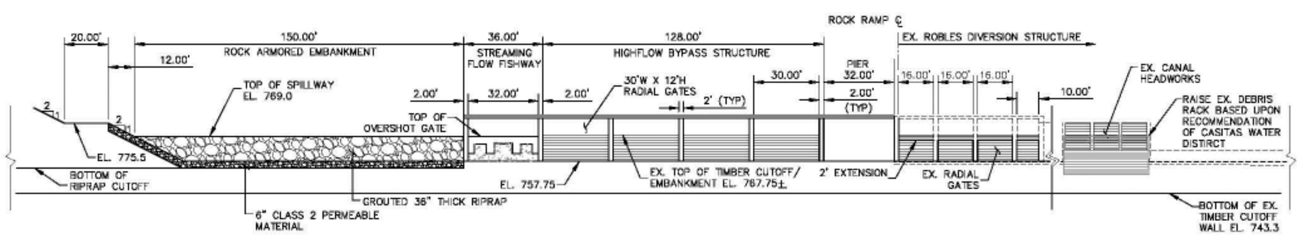

Figure 2.2 Post-Project Elevation

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3. HYDROLOGIC AND HYDRAULIC BASIS FOR DESIGN

The hydrologic and hydraulic analyses conducted in support of the design of the Robles Diversion Dam modifications and the overall Matilija Dam Ecosystem Restoration Project included rainfall-runoff modeling for the with-project conditions, a numerical sedimentation analysis, and a physical hydraulic and sediment model of the baseline and with-project conditions. Detailed descriptions of the assumptions, inputs, methodologies, and results of these studies for the Robles Diversion Dam Modification are provided in the hydrology and hydraulic analyses appendices contained in the Matilija Dam Ecosystem Restoration DDR and subsequent reports by the USBR included in Appendix B.

As described in the Matilija Dam Ecosystem Restoration DDR, the 100-year design discharge for the Ventura River at the Robles Diversion Dam is 27,100 cubic feet per second (cfs). For the Robles Diversion Dam, the USBR performed a numerical model and physical model for the baseline and with-project conditions. In the analysis, various locations and modifications were considered to optimize the design of the HFB structure. A description of the detailed analysis is provided in Appendix B.

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4. GEOTECHNICAL BASIS FOR DESIGN

The geotechnical basis of design is based on the findings of the Foundation Report prepared for the USACE by AMEC Geomatrix (draft dated August 2, 2012), the Ground Motion Hazard Evaluation report prepared for the USACE by AMEC Geomatrix (dated January 19, 2009), and the recommendations presented in the memorandum prepared by the USACE (dated July 13, 2012). The findings presented in the aforementioned documents are based on a review of previous geotechnical investigations performed in the proximity of the project, as well as a field reconnaissance performed by AMEC Geomatrix.

GEOLOGIC SETTING

The Robles Diversion Dam is located on the Ventura River, approximately 2 miles downstream (south) of its confluence with the Matilija River and the North Fork of the Matilija River, and approximately 2 miles downstream of the 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 in Dibblee, 1987; Tan and Jones, 2006).

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 dam 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.

SUBSURFACE CONDITIONS

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 before construction. 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 of the geotechnical appendix). Boulders greater than 12 inches in size, reportedly constituting about 3 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 spillway of the existing diversion canal, reportedly encountered about 6 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.

Based on the available drawings, the existing diversion dam is a zoned earthfill and rockfill embankment. To help mitigate seepage, a 15- to 20-foot-deep trench of “compacted impervious backfill” was constructed upstream and downstream of a timber cutoff wall. The dam embankment was originally approximately 530 feet long but currently extends only about 350 feet across the river bottom.

SEISMIC CONDITIONS

For the Ground Motion Hazard Evaluation report (dated January 19, 2009), AMEC Geomatrix performed both probabilistic and deterministic evaluations of potential peak ground acceleration (PGA) and seismic response at the dam site. The probabilistic assessment evaluated PGA for a range of annual frequencies of exceedance as summarized on Figure 4.1. In addition, acceleration response spectra were developed for various recurrence intervals. These spectra are plotted on Figure 4.2. The 50 percent probability of exceedance in 100 years was established as the operating basis earthquake (OBE). The 10 percent probability of exceedance in 100 years was established as the design basis earthquake (DBE). The acceleration response spectra for the strongest motion developed from the deterministic study are also plotted on Figure 4.2. Both median and 84th percentile spectra developed from a M_w 7.4 earthquake on the Santa Ynez Fault are presented.

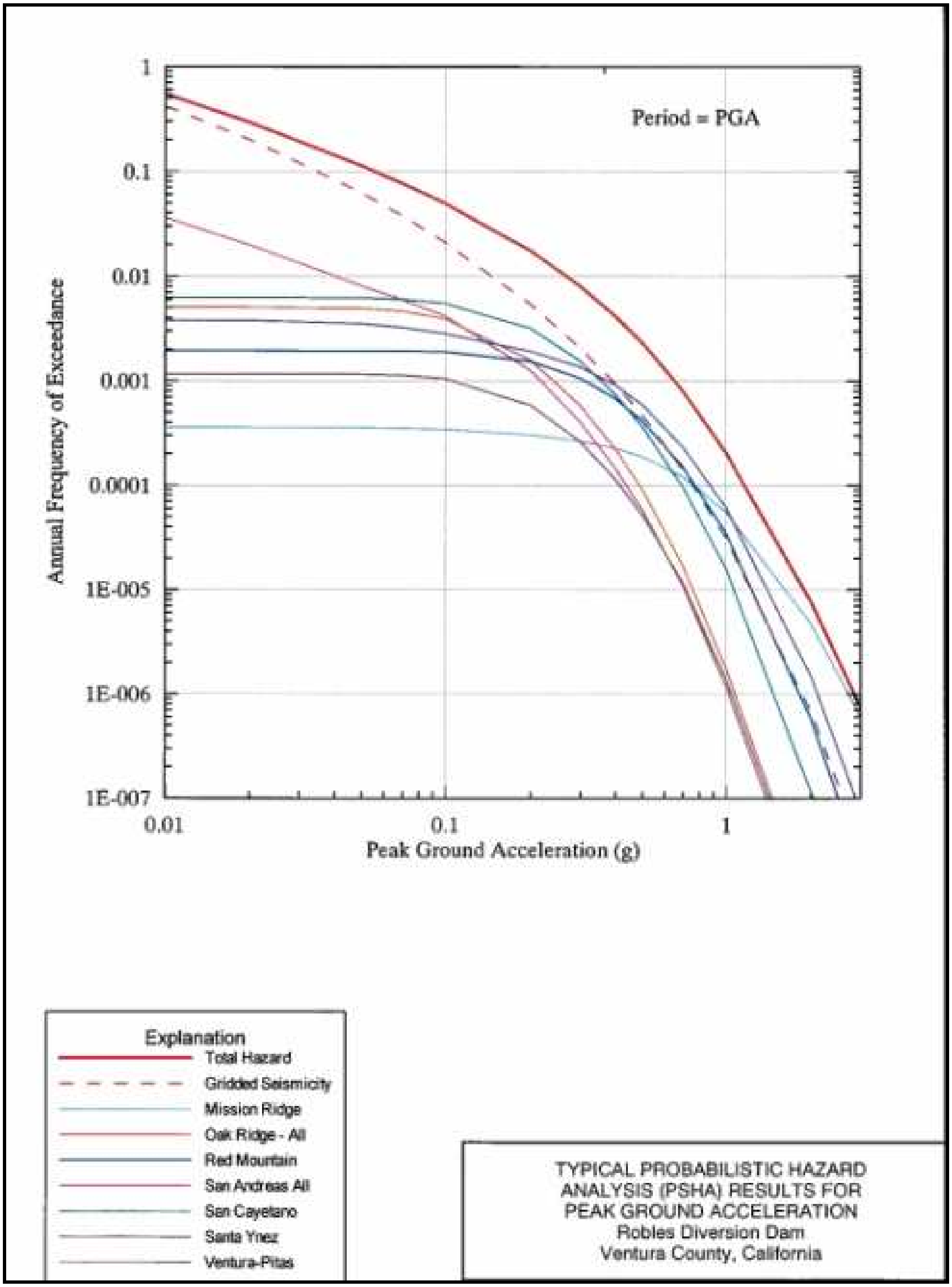

Figure 4.1 Peak Ground Acceleration versus Annual Frequency of Exceedance

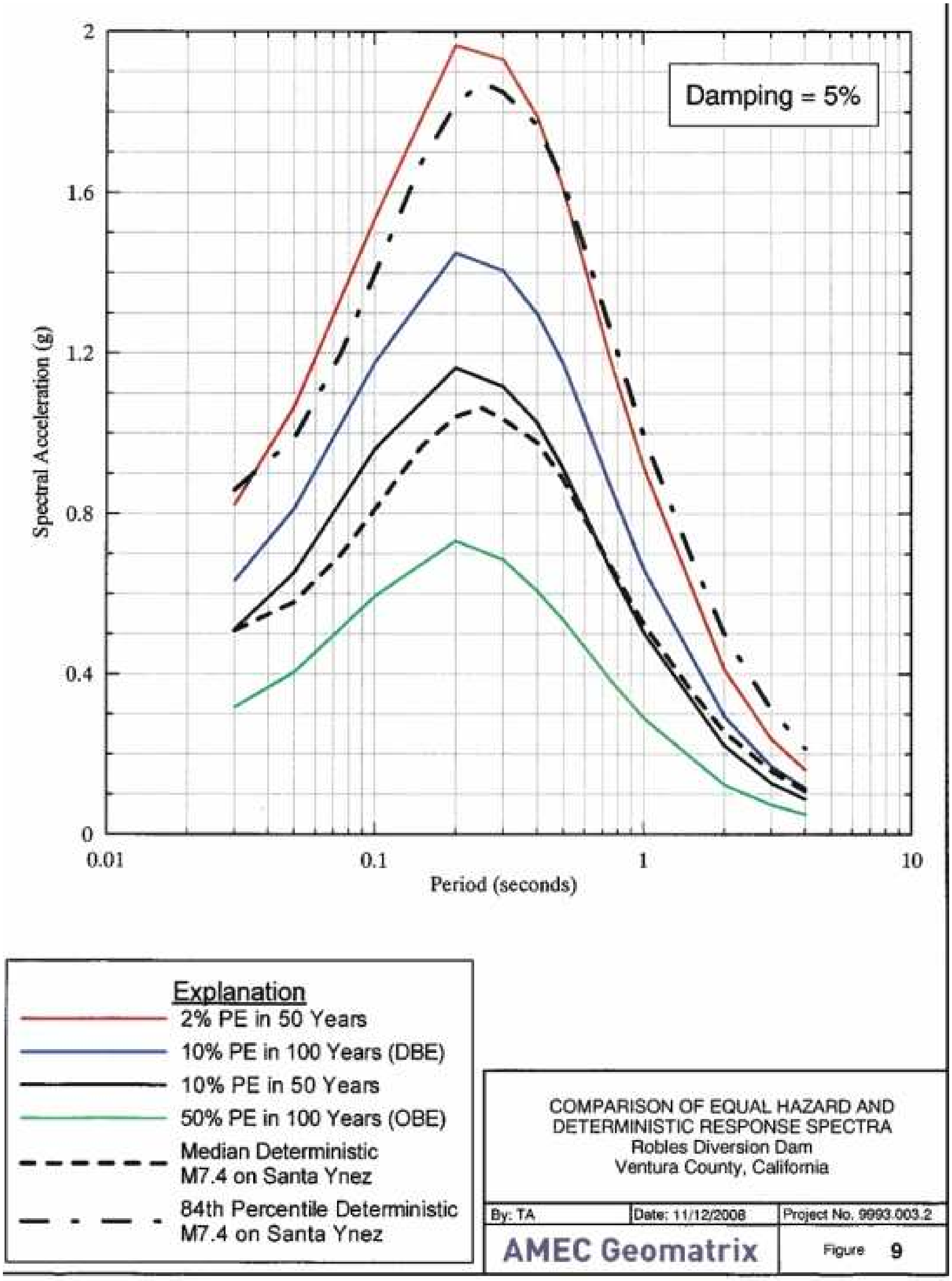

Figure 4.2 Response Spectra from Probabilistic and Deterministic Evaluation

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 saturated 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 site vicinity, AMEC Geomatrix judged the hazard posed by densification or lateral spreading of the site soils caused by earthquake shaking to be low.

The shear wave velocity (V_S) is based on the wave velocity of the materials in the uppermost 100 feet (30 meters, V_{S30}). The V_{S30} characterizes the site conditions in developing an estimate of ground shaking from ground motion. According to AMEC Geomatrix, the V_{S30} resulted at 450 meters per second (m/s) (1,476 ft/s). This corresponds to a very dense soil and soft rock for a Site Class ‘C’ with V_S in a range of 1,200 to 2,500 ft/s.

SEEPAGE EVALUATION

The design plans for the original diversion dam construction indicate that a seepage barrier, consisting of a timber cutoff wall embedded in an impervious backfill trench, is located under the current dam alignment. 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 unavailable. Therefore, the extent of the existing cutoff trench and the condition of the timber cutoff wall are unknown. The USACE intends to use the existing cutoff as much as possible. The USACE has recommended that a limited investigation of the existing cutoff be conducted before the project is put out for bids. Material for use as impervious backfill will have a soil classification of CL or CL-ML per ASTM International (ASTM D2487).

Seepage analyses performed for the current design included the seepage cutoff as shown on the original construction drawings. A steady-state seepage analysis was performed for three scenarios (Table 4.1).

Table 4.1 Results of Steady-State Seepage Analysis

Scenario	Headwater Elevation (feet)	Tailwater Elevation (feet)
Normal reservoir	757.75	--
Infrequent flood	769.00	753.50
Design flood	775.75	760.00

For each scenario, pressure head contours below the proposed structure were estimated by AMEC Geomatrix and provided to the structural engineer to evaluate uplift conditions. The uplift pressures were problematic to the design of the spillway and fishladder; therefore, a subdrainage relief system was added to the design. The subdrainage system consists of a layer of permeable drainage material (California Department of Transportation Class 2 permeable material). Per the USACE memorandum of August 24, 2012, the layer was designed to be 20 inches in thickness with 4-inch-diameter polyvinyl chloride (PVC) collector pipes. The design includes three collector pipes that run parallel to the dam axis: one near the dam centerline and two farther downstream. Outlets to the collector drains are situated within the spillway and fishladder wall sections, 12 inches above the slab invert. The USACE analysis indicated that the subdrain system would mitigate the high uplift pressures. The USACE memorandum is provided in Appendix C.

SLOPE STABILITY

The diversion dam embankment adjacent to the new spillway will be improved with new slope revetment (grouted rock). As part of the improvement, the upstream slope will be graded to an angle of 3 horizontal to 1 vertical (3H:1V). The downstream slope will be graded to a slope of approximately 11.4 percent. Other slopes constructed for ramps will be constructed at an angle no steeper than 2H:1V. The AMEC Geomatrix report recommended slope angles no steeper than 2:1V. The USACE memorandum recommended that weep holes not be constructed within the grouted rock protection, citing that pressures will dissipate through naturally occurring cracks in the grouted rock.

FOUNDATION DESIGN

Based on the findings in the AMEC Geomatrix report, the spillway structures can be supported on shallow foundations. Shallow foundations (e.g., mat, spread-type, 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 load (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 the adjacent grade. It was recommended that the foundations be supported on a building pad of compacted granular material such as aggregate base. Below the spillway and fishladder, a drainage layer will be placed in lieu of the aggregate base layer.

5. CIVIL BASIS OF DESIGN

Certain project features require the use of civil designs and criteria in their design. The project features include the raising of the existing diversion embankment and the construction of a rock ramp channel and rock armored embankment. Additionally, the existing embankment will be extended to join the Meiners Oaks Levee improvements currently being constructed by the USACE.

REFERENCE DOCUMENTS

The design of the embankment modifications and the channel and spillway of the rock-armored ramp were based on the following government and civilian publications:

- Bureau of Reclamation, U.S. Department of the Interior, 1987. *Design of Small Dams*, Third Edition.
- ER 1110-2-1150, *Engineering and Design for Civil Works Projects*, U.S. Army Corps of Engineers, 31 August 1999.
- U.S. Army Corps of Engineers, Los Angeles District, *Memorandum for Matilija Dam Ecosystem Restoration Study, Project Deliver Team*, 3 April 2009.
- Partial set of USBR record drawings of spillway dam, gates, and appurtenances (provided by CMWD staff).
- Partial set of USBR Construction Specifications for the Robles Diversion (provided by CMWD staff).
- AMEC Geomatrix, Inc., 2008. *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, Oakland, CA, 12 November.
- Mefford, B., H. Stowell, and C. Heinje, 2008. *Robles Diversion Dam High Flow and Sediment Bypass Structure, Ventura, California, Physical Model Study*, Hydraulic Laboratory Report HL-2008-7, Technical Service Center, U.S. Bureau of Reclamation, Denver, CO.
- Mefford, B., 2010. *Robles Diversion Dam Left Bank Fishway, Ventura, California*, Hydraulic Laboratory Report HL-2011-04, Technical Service Center, U.S. Bureau of Reclamation, Denver, CO, 22 June.

DAM EMBANKMENT MODIFICATION

To improve the operation of the diversion dam and fish passage, the existing embankment will be raised approximately 2 feet to elevation 769.00 feet. A concrete sill will be provided to control the weir elevation of the raised embankment. The existing embankment will be raised and armored with rock riprap to prevent scour. The existing timber cutoff wall and 15- to 20-foot-deep trench of "compacted impervious backfill" upstream and downstream of the timber cutoff wall will remain (based upon adequacy from field inspection at time of construction), with the proposed concrete sill cutoff wall extended into the impervious backfill to limit seepage. The embankment will also be extended to connect with the upstream limits of the Meiners Oaks Levee improvements.

ROCK RAMP CHANNEL AND ROCK ARMORED EMBANKMENT

Downstream of the HFB structure and USBR stilling basin, a rock ramp will provide additional dissipation of flow velocity and protect the river invert from scour. The rock ramp was designed in accordance with the USACE *Memorandum for Matilija Dam Ecosystem Restoration Study, Project Delivery Team* (3 April 2009) and the subsequent USBR design memoranda.

The rock-armored ramp will join the existing river channel approximately 400 feet downstream of the HFB and stilling basin. The slope of the rock ramp will vary because of the difference between the sill elevation of the existing stilling basin and that of the proposed basin, elevations 751.0 and 753.25 feet, respectively. To account for this elevation difference, the rock ramp directly downstream of the existing structure (within the low flow channel) will have a channel gradient of 1.5 percent from the existing structure. The gradient of the rock ramp downstream of the HFB structure will be 2.0 percent. The gradient of the rock ramp was designed to maintain sediment passage downstream of the Robles Diversion structure.

The maximum capacity of the existing and proposed bypass structures is approximately 16,000 cfs. To increase the high-flow diversion capacity of the Roles Diversion Dam, a rock armored embankment will be provided adjacent to the proposed HFB structure. Due to the steep gradient (11.4 percent), the rock-armored embankment will consist of grouted riprap and have an embankment height of 6 feet. The rock-armored ramp is specifically design to safely convey the excess flow between the 20-year design flow (19,000 cfs) and the maximum spillway capacity (16,000 cfs). This difference of approximately 3,000 cfs is expected to be conveyed by the rock armored embankment and the rock ramp without damage. Additionally, the rock ramp has been designed to convey flow even if the spillway gates are damaged and maintained in the lowered position. In this case, the rock-armored ramp is designed to prevent catastrophic damages to the diversion dam due to the storm flows.

STREAMING FLOW FISHWAY

A minimally baffled fishway was developed for Robles Diversion Dam to provide passage for adult steelhead trout. The fishway is designed to operate as an auxiliary fishway during flood events in excess of a 2-year event. An engineered roughened-channel–fishway design is recommended for the project. This type of fishway was selected because large flows needed for fish attraction could be passed directly through the fishway, thereby eliminating the need for auxiliary attraction flow facilities. The fishway is designed to convey a 300-cfs flow at normal diversion pool (elevation 768.0 feet) with flow increasing to about 400 cfs at maximum pool (elevation768.5 feet). The fishway will function as a step-pool-type fishway with resting areas located on the periphery of the main flow. The 8.0 percent slope of the fishway was chosen based on fitting the fishway into the proposed HFB spillway while meeting the flow requirements for fish passage.

6. STRUCTURAL BASIS OF DESIGN

Stability and seismic analyses were performed for the various project features that require structural design. The project features are grouped into five structural components: existing spillway, new spillway, fish ladder, baffle walls, and equipment supports. The designs for these structural components are provided in Appendix D.

The existing spillway will be checked for stability under the loading conditions set forth in the following sections according to Engineer Manual (EM) 1110-2-2100. Loads from the radial gate will be taken directly from the design and analysis of the Tainter gates and placed at the location of the existing corbel. Seepage below the structure will be considered and calculated by the geotechnical engineer using the flow net analysis method.

The analysis of the new spillway will consist of a stability check and a reinforced-concrete strength design of the structure and its components. The stability of the new spillway will follow the requirements set forth in the following sections. Seepage below the structure will be considered and calculated by the geotechnical engineer using the flow net analysis method.

For the concrete strength design, the existing spillway will be broken into components to include the baffle walls, foundations, and corbels. Each reinforced-concrete component will be designed to meet the hydraulic load factor design requirements described in EM 1110-2-2104. Loads from the radial gate will be taken directly from the design and analysis of the Tainter gates and placed at the location of the corbel. The strength design of each component will be in accordance with EM 1110-2-2104 and ACI 318-05.

The analysis of the fish ladder will consist of stability and reinforced-concrete strength design calculations. The stability of the fish ladder will follow the requirements set forth in the following sections. If the fish ladder is soil supported, the seepage below the structure will be considered and calculated by the geotechnical engineer using the flow net analysis method.

Once the information is received on the location, types, and construction material of equipment supports required for the various mechanical and electrical equipment, a design analysis will commence, and further design basis information will be provided.

REFERENCE DOCUMENTS

Analysis of the existing and new spillway structures and their components was based on the following government and civilian publications:

- IBC 2006. International Building Code, International Code Council.
- EM 1110-2-2100, *Stability Analysis of Concrete Structures.* U.S. Army Corps of Engineers, 1 December 2005.
- EM 1110-2-2104, *Strength Design of Reinforced Concrete Hydraulic Structures,* U.S. Army Corps of Engineers, Change 1, August 2003.

- EM 1110-2-2200, *Gravity Dam Design*, U.S. Army Corps of Engineers, 29 September 1989.
- EM 1110-2-2502, *Retaining and Flood Walls*, U.S. Army Corps of Engineers, 30 June 1995.
- EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures, U.S. Army Corps of Engineers, 1 May 2007.
- ER 1110-2-1806, *Earthquake Design and Evaluation for Civil Works Projects*, U.S. Army Corps of Engineers, 31 July 1995.
- ACI 318-05, *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute.
- ACI 350-06, *Code Requirements for Environmental Engineering Concrete Structures and Commentary*, American Concrete Institute.
- *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, 2005.
- FEMA 450, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Federal Emergency Management Agency.
- Partial set of USBR record drawings of spillway dam, gates, and appurtenances (provided by CMWD staff).
- Partial set of USBR 1957 Construction Specifications for the Robles Diversion (provided by CMWD staff).
- AMEC Geomatrix, Inc., 2009. *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, Oakland, CA, 19 January.
- AMEC Geomatrix, Inc., 2012. *Foundation Report for Robles Diversion Dam Modification Project*, Oakland, CA, 2 August.

DESIGN CRITERIA

Stability Analysis Method

The stability analysis was performed using the methods, stability criteria loads, and load combinations outlined in EM 1110-2-2100 and EM 1110-2-2502.

The stability analysis was performed using the gravity method, which assumes that the dam structure is a rigid two-dimensional block with a linear foundation pressure distribution. This method applies to dams that have a regular shape, with no curves or other irregularities. The stability model analyzed each gate (existing and new) by assuming that each gate bay has similar loading and resistance properties and that a single gate bay is representative of the dam as a whole. Based on the dam's geometry, foundation properties, and surrounding soils, this assumption is appropriate for both the existing and new spillways. The mathematical model used to determine both spillways' stability was developed using Mathcad and is provided in Appendix D.

The dam stability acceptance criterion is that the force and moment equilibrium are maintained without exceeding the allowable unit stress for the concrete and foundation materials. The allowable unit stresses are obtained by dividing the ultimate stress by the minimum safety factors outlined in EM 1110-2-2100. Because the dam is founded on soils with much lower allowable stress values than concrete, it is not necessary to check the concrete stresses.

The Robles Diversion Dam is classified as a normal structure and, therefore, the minimum safety factors shown in Table 6.1 are applicable. Table 6.1 is a reproduction of Table 3-3 in EM 1110-2-2100. Per the guidelines, the minimum safety factors are checked for sliding and foundation bearing capacity to ensure force and moment equilibrium. It does not require a calculation of the overturning stability safety factor (Mr/Mo).

Table 6.1 Minimum Stability Criteria

Inlet and Outlet Structure		Usual	Unusual	Extreme
Sliding	FS	1.5	1.3	1.1
Overturning	Percent base in compression	100%	75%	Within base
Bearing capacity	FS	3.0	2.0	>1.0
Flotation	FS	1.3	1.2	1.1

FS = factor of safety

The sliding stability safety factor was determined per equation 5-3 in the EM 1110-2-2100. The sliding factor of safety is defined in equation 6-1.

$$FS_{sliding} = \frac{(N \tan \phi + cL)}{T} \tag{6-1}$$

where:

N = resultant forces normal to the assumed sliding plane
ø = angle of internal friction
c = cohesion intercept
L = length of base in compression for a unit strip of dam

The foundation bearing stability is determined by summing the moments of the applied loads about the centerline of the dam foundation to determine the foundation bearing pressure required to achieve moment equilibrium. Because the applied loads do not produce uplift, the bearing pressure is determined assuming a linear bearing pressure distribution using equation 6-2.

$$Bearing\ \mathrm{Pr}essure = \frac{V}{A} \pm \frac{M}{S} \tag{6-2}$$

where:

V = sum of vertical loads
A = foundation area
M = sum of moments about the foundation centerline due to applied loads

The bearing stress factor of safety is determined from equation 6-3.

$$FS_{bearing} = \frac{Ultimate.Bearing.Capacity}{Maximum.Service.Load.Bearing.\Pr essure} \tag{6-3}$$

Loads

The required loads and loading combinations required for analysis are outlined in EM 1110-2-2100 and EM 1110-2-2502.

Two levels of earthquakes and associated performance objectives are defined for the project: OBE and MDE.

- OBE = 0.318 g (provided by AMEC)
- MDE = 0.633 g (provided by AMEC)]

Operational Basis Earthquake

The OBE is the design earthquake that represents ground motions for which the essential structures and critical components of the system are expected to sustain no permanent damage and the normal structures and noncritical components are expected to sustain either minor or no permanent damage. "Critical" components and equipment are defined as those whose malfunction could interfere with the safe and continuous operation of the dam. Under OBE loading, the structural response of the spillway will remain essentially elastic.

Maximum Design Earthquake

The MDE is the design earthquake in which normal structures may suffer permanent offsets, although no collapse may occur. Damage consisting of cracking, reinforcement yield, and major spalling of concrete is possible. These conditions may require closure of the spillways to repair the damage. The foundations must have sufficient capacity to withstand the earthquake loading without any damage. The peak response in the structure may be inelastic but will not exceed the prescribed residual deformations. Walls will remain stable for the normal loading condition under the permanently deformed state. Essential structures may exhibit some visible damage, but they will be limited to narrow flexural cracking of concrete and the onset of yielding in steel.

Hydrostatic Uplift Pressures

The hydrostatic uplift pressures were determined by the geotechnical engineer using the flow net analysis to account for the seepage barrier in front of the dam (see Appendix D).

Earthquake Earth Pressure

The lateral earthquake earth pressure forces were determined per EM 1110-2-2100. The lateral earthquake earth pressure forces were determined using the general wedge method to account for the inertia force of the water inside the backfill material.

Earthquake Inertia Force

The earthquake inertia force was determined per EM 1110-2-2200. This force is determined by equation 6-4 and acts at the center of gravity.

$$Pe_x = Ma_x = \frac{W}{g}\alpha k_h = W\alpha \tag{6-4}$$

where:

Pe$_x$ = horizontal inertia force
M = mass of element (dam)
a$_x$ = horizontal earthquake acceleration
W = weight of element (dam)
k_h = acceleration of gravity
α = seismic coefficient

Hydrodynamic Force

The hydrodynamic force was determined per EM 1110-2-2100. This force is considered to be parabolic, is determined using Westergaard's equation (equation 6-5), and acts at a height 0.4 times the height of the reservoir.

$$Pew = \frac{2}{3}Ce\alpha h^2 \tag{6-5}$$

where:

Pew = total additional water load due to inertia (kips)
Ce = factor equal to 0.051 for most usual conditions
α = seismic coefficient
h = total height of reservoir (feet)

Dynamic Soil Pressures

The dynamic soil pressures were determined using the Mononobe-Okabe theory as shown in equations 6-6 through 6-9.

Coefficient of Active Earth Pressure in Earthquake:

$$Kae := \frac{\cos(\phi b - \psi - \theta)^2}{\cos(\psi)\cdot\cos(\theta)^2\cdot\cos(\psi+\theta+\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi b+\delta)\cdot\sin(\phi b-\psi-\beta)}{\cos(\beta-\theta)\cdot\cos(\psi+\theta+\delta)}}\right)^2} \tag{6-6}$$

Dynamic Increment of Active Earth Pressure:

$$K_e = K_a - K_{ae} \tag{6-7}$$

Coefficient of Passive Earth Pressure

$$Kpe := \frac{\cos(\phi b - \psi + \theta)^2}{\cos(\psi)\cdot\cos(\theta)^2\cdot\cos(\psi - \theta + \delta)\cdot\left(1 - \sqrt{\frac{\sin(\phi b + \delta)\cdot\sin(\phi b - \psi + \beta)}{\cos(\beta - \theta)\cdot\cos(\psi - \theta + \delta)}}\right)^2} \tag{6-8}$$

where:

ϕ_b = internal friction angle of soil
k_h = horizontal seismic coefficient (acceleration in g)
k_v = vertical seismic coefficient (acceleration in g)
θ = angle between back face of wall and vertical
β = slope of backfill
δ = wall friction angle

$$\psi = \operatorname{atan}\left(\frac{kh}{1 - kv}\right)$$

<u>Seismic Coefficient Method</u>

Earthquake forces are treated as sustained forces and are combined with the hydrostatic pressures, uplift, backfill soil pressures, and gravity loads. The inertial forces acting on the structure are computed as the product of the structural mass, added-mass of water, and the effects of dynamic soil pressures, multiplied by a horizontal seismic coefficient. A seismic coefficient, equal to 2/3 the PGA divided by the acceleration of gravity (g), is defined by USACE in EM 1110-2-2100 to evaluate the potential for sliding.

The seismic coefficient method is used for checking the stability of the structure.

<u>Response Spectrum—Modal Analysis Procedure (Linear Dynamics Analysis)</u>

The response spectrum analysis is a linear dynamic analysis procedure. In the response spectrum analysis, the maximum response of the structure to earthquake excitation is evaluated by combining the maximum responses from individual modes and multicomponent input. This procedure is especially applicable to the majority of USACE hydraulic structures that are designed to remain essentially elastic when subjected to medium-intensity ground motions, such as the OBE. The modal analysis is also used for the MDE excitation, except that the computed linear elastic response is permitted to exceed the concrete cracking and yield stress levels for a limited amount in order to account for the energy absorption of the structure.

Seismic Design and Evaluation Using Demand to Capacity Ratio Approach

Table 6.2 DCR Allowable Values for Reinforced-Concrete Hydraulic Structures

Action in Terms of Forces	Performance Objective for Damage Control (MDE)	Performance Objectives for Serviceability (OBE)
Flexure	2.0	1.0
Shear	1.0	0.8
Sliding shear	1.0	0.8

Note: Load cases for reinforced concrete are based on EM 1110-2-6053, Table 6-1a.
DCR = demand to capacity ratio
MDE = maximum design earthquake
OBE = operating basis earthquake

Ice Loading

No ice loads were applied to the structure.

Silt Loading

No silt loads were applied to the structure.

Load Cases

The load cases used for the stability analysis were divided into three categories, which were obtained from Table B-1 in EM 1110-2-2100: usual (U), unusual (UN), and extreme (E).

Table 6.3 Basic Loading-Conditions Classification

Load Case	Loading Description	Classification
1	Construction condition	UN
2	Normal reservoir	U
3	Infrequent flood	UN
4	Construction with OBE[1]	E
5	Coincident pool with OBE	UN
6	Coincident pool with MDE[1]	E
7	Maximum design flood	U/UN/E

MDE = maximum design earthquake
OBE = operating basis earthquake
[1] Seismic ground accelerations are site specific based on a geotechnical report by AMEC.

Loading Case 1 is the construction condition, which includes the completed dam structure with no headwater or tailwater. This is considered an unusual load case.

Loading Case 2 is the normal operating condition, which includes headwater at the normal reservoir elevation (with a water surface elevation [WSE] of 769.00 feet) with no tailwater downstream (WSE of 750.25 feet).

Loading Case 3 is the infrequent flood condition, which includes the pool at an elevation representing a flood event (WSE of 775.75 feet), with minimum corresponding tailwater (WSE of 760.00 feet).

Loading Case 4 is the construction condition with the OBE, the horizontal acceleration in the upstream direction, and no headwater or tailwater loads.

Loading Case 5 considers the OBE occurring during the coincident pool, the horizontal acceleration in the downstream condition created by the OBE, corresponding tailwater, the uplift at the pre-earthquake level, silt pressure if applicable, but no ice pressure.

Loading Case 6 considers the MDE occurring during the coincident pool, the horizontal acceleration in the downstream condition created by the MDE, corresponding tailwater, the uplift at the pre-earthquake level, silt pressure if applicable, but no ice pressure.

Loading Case 7 consists of the loads created by the maximum design flood (MDF) including the combination of pool (WSE of 775.75 feet) and tailwater (WSE of 760.10 feet), which produces the worst structural loading condition.

Table 6.4 Loading Description (Strength Design)

Load Case	Loading Description	Classification
1a	$U = H_f*[1.4D+1.6(W_{wt}+P_w+P_e+U)+1.7*W_i]$	UN
2a	$U = 0.75*[H_f[1.6(W_e+P_w+U+W_w+P_e)+1.7(W_i)+1.4(F_{sr}+P_E+F_h)]]$	UN
3	$U = 0.75*[H_f[1.6(W_e+P_w+U+W_w+P_e)+1.7(W_i)+1.0(F_{sr}+P_E+F_h)]]$	E

Note: Load cases for reinforced concrete are based on EM 1110-2-2104.

D = dead load
L = live load
H_f = hydraulic load factor = 1.3 or 1.65 (direct tension members)
W_e = vertical soil load
W_{wt} = weight of water above structure
P_w = hydrostatic pressure due to saturated soil
P_e = lateral earth pressure
F_{sr} = dynamic soil pressure (Mononobe-Okabe)
F_h = seismic inertial load
P_E = hydrodynamic load
W_i = hydrostatic loading
U = uplift pressure

CONSTRUCTION MATERIALS

Concrete

All structural concrete will meet the following minimum requirements:

- The concrete will have a 28-day compressive strength of 5000 pounds per square inch (psi).
- The maximum water content of the structural concrete will be 0.40.
- The unit weight for concrete to be used in design is 150 pounds per cubic foot (lbs/ft^3).

Reinforcing Steel

All reinforcing steel will meet the following minimum requirements:

- Reinforcing steel will conform to ASTM A615M, Grade 60.
- Reinforcing development lengths and splices will be in accordance with EM 1110-2-2104.

UNIT WEIGHTS

The appropriate unit weights and soil properties to be used in the structural design are provided in the geotechnical reports in Appendix C. The unit weight of water to be used in the design is 62.4 lbs/ft^3.

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7. MECHANICAL AND ELECTRICAL BASIS OF DESIGN

REFERENCE DOCUMENTS

The analysis of the spillway Tainter gates and mechanical systems was based on the following government and civilian publications:

- EM 1110-2-2702, *Design of Spillway Tainter Gates*, U.S. Army Corps of Engineers, 1 January 2000.
- EM 1110-2-2105, *Design of Hydraulic Steel Structures*, U.S. Army Corps of Engineers, 31 March 1993.
- ER 1110-2-2610, *Lock and Dam Gate Operating and Control Systems*, U.S. Army Corps of Engineers.
- ER 1110-2-1806, *Earthquake Design and Evaluation for Civil Works Projects*, U.S. Army Corps of Engineers, 31 July 1995.
- *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, 2005.
- *Steel Construction Manual*, 13th Edition, AISC 360-05, American Institute of Steel Construction, 2005.
- *Federal Specification for Steel, Structural (Including Welding) and Rivet; for Bridges and Buildings*, QQ-S-741, Federal Standard Stock Catalog, December 1942.
- *Specifications for Top Running and Gantry Type Multiple Girder Electric Overhead Traveling Cranes*, CMAA 70, Crane Manufacturers Association of America, Inc., 2004.
- Partial set of USBR record drawings of spillway dam, gates, and appurtenances (provided by CMWD staff).
- Partial set of USBR 1957 Construction Specifications for the Robles Diversion (provided by CMWD staff).
- AMEC Geomatrix, Inc., 2008. *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, Oakland, CA, 12 November.

LOADS

The following loads are applicable to spillway Tainter gates per EM 1110-2-2702, Section 3-4(b).

- Hydrostatic (Hs)
- Gravity (D, C, and M),

  where:

  D = structure self-weight
  C = ice load
  M = mud and debris
- Gate-lifting system loads (Q)
- Impact (I)
- Side-seal friction loads (F_s)

- Trunnion pin friction loads (F_t)
- Earthquake (E)
- Wave (W_A)
- Wind (W)

The hydrostatic loads (Hs) were calculated based the gate sill elevation and the pool depths of the diversion dam's forebay (Table 7.1). The maximum hydrostatic load (H_1) is defined as the maximum net hydrostatic load that will ever occur. The design hydrostatic load (H_2) is the maximum net hydrostatic load considering any flood up to a 10-year event. The normal hydrostatic load (H_3) is the temporal average net load from upper and lower pools, i.e., the load that exists from pool levels that are exceeded up to 50 percent of the time during the year. A new crest elevation of 769.00 feet and an existing gate sill elevation of 757.75 feet (Record Drawings 767-D-232) were used to determine the case loading.

Table 7.1 Hydrostatic Loads

Return Period	Design Water Surface Elevation (feet)	Load Case	Water Depth (feet)
PMF event	769.75	H_1	12.00
10-year event	769.75	H_2	12.00
Annual event	769.75	H_3	12.00

PMF = probable maximum flood

The gravity loads (D, C, and M) include structure self-weight (D), ice load (C), and mud and debris (M). The gate self-weight was calculated from the finite element models for the existing gate structure and the new gate structure. The vertical ice load was calculated based on an iced surface on one side of the skin plate, top of girders, and downstream face of girders. An ice thickness of ¼ inch was used in the load determination. Mud load was computed based on future silt loading due to the removal of the Matilija Dam (top of girders filled with silt).

The gate-lifting system load (Q) consists of loads Q_1 (maximum downward), Q_2 (at-rest downward), and Q_3 (maximum upward). Loads Q_1 and Q_2 do not exist for wire rope hoist systems. The maximum upward operating machinery load (Q_3) is the maximum upward load that can be applied by the wire rope hoist system when a gate is jammed or fully opened. This load is the load due to wire rope contact pressure on the skin plate. The contact force (125 pounds per inch), is equal to the rope tension force divided by the gate radius.

The inflow hydrographs showed that the reservoir does not sustain a WSE long enough to allow icing, which was corroborated by CMWD staff, the impact load (I) was assumed to be zero.

The side-seal friction loads (F_s) are loads along the radius of the skin plate due to friction between the side seals and the side seal plate when the gate is opening or closing. The coefficient of friction (μ_s) is assumed to be 0.5 for the rubber seals.

Trunnion pin friction loads (F_t) are loads due to friction around the surface of the trunnion pin between the bushing and the pin. For this analysis, the coefficient of friction is assumed to be 0.30.

The earthquake load was determined using the OBE as defined in Engineer Regulation (ER) 1110-2-1806. This load includes the inertial hydrodynamic effects of the water moving with the structure. EM 1110-2-2702, Section 3.4.b(1)(g), states that "when a tainter gate is submerged, the inertial forces due to structural weight, ice and mud are insignificant when compared with hydrodynamic loads and can be ignored." In this analysis, inertial forces due to self-weight, mud, and ice were considered under gate fully opened conditions. The Westergaard pressure distribution was calculated using the following input values:

- Unit weight of water = 62.50 pounds per cubic foot
- OBE = 0.318 g (geotechnical report)
- Pool depth = 12.00 feet (new crest elevation minus existing sill elevation)

Wave (W_A) loads are site specific. For this analysis, the wave height is assumed to be 0 foot. The probability of wind on a full reservoir is sufficiently low to rule out wave generation.

The Wind (W) load calculation is based on the site-specific conditions and in accordance with ASCE/SEI 7-05. The wind force input variables are shown below (figures, tables and sections apply to ASCE/SEI 7-05):

- Basic wind speed = 85 mph (Figure 6-1)
- Occupancy Category III (Table 1-1)
- Importance factor = 1.15 (Table 6-1)
- Exposure C (6.5.6.3)
- Gust-effect factor = 0.85 (6.5.8)
- Net force coefficient = 1.40 (Figure 6-20)
- Velocity pressure = 17 psf (6.5.10)

Note that the wind load was applied to the projected surface of the gate; this area was calculated for a gate fully opened condition.

LOAD COMBINATIONS

The load combinations used in the design are those established by EM 1110-2-2702, Section 3-4 b (2), which are summarized in Table 7.2 by number and a brief description for use in the design.

Note that under the gate closed condition, load combinations U2, U3, and U4 are similar because loads Q_1, Q_2, Q_3, W_A, and I are not applicable. Similarly, under gate operating conditions, load combinations U7 and U8 are similar because loads I and W_A are not applicable. Under the gate jammed condition, only load combination U10 is applicable.

Table 7.2 Load Combinations

Load Condition	Load Combination	**EM** 1110-2-**2702** Equation
Gate closed	$U1 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_1 + 1.2\ Q_2$	3-5
	$U2 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.2\ Q_1$	3-6A
	$U3 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.2\ Q_2 + 1.2\ W_A$	3-6B
	$U4 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.2\ Q_3 + k_1 I$	3-6C
	$U5 = 1.2\ D + 1.6\ M + 1.6\ C + 1.2\ H_3 + 1.0\ E$	3-7
Gate operating with two hoists	$U6 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_1 + 1.4\ F_s + 1.0\ F_t$	3-8
	$U7 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.4\ F_s + 1.0\ F_t + 1.2\ W_A$	3-9A
	$U8 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + k_1\ I + 1.4 F_s + 1.0\ F_t$	3-9B
Gate operating with one hoist	$U9 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.4 F_s + 1.0\ F_t$	3-10
Gate jammed	$U10 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.2\ Q_3$	3-11A
	$U11 = 1.2\ D + 1.6\ M + 1.6\ C + 1.4\ H_2 + 1.2\ Q_1$	3-11B
Gate fully opened	$U12 = K_d\ D + 1.6\ M + 1.6\ C + 1.3\ W$	3-12A
	$U13 = K_d\ D + 1.6\ M + 1.6\ C + 1.0\ E$	3-12B
	$U14 = K_d\ D + 1.6\ M + 1.6\ C + 1.2\ Q_3$	3-12C
Overtopping	Unfactored hydrostatic load for 18-foot head (12-foot gate height + 6-foot overtopping). Check is performed for the 30-by-12-foot Tainter gates only.	NA

NA = not applicable

D = self-weight
C = ice load
M = mud load
W = wind load
W_A = wave load
H_1 = hydrostatic load (maximum)
H_2 = hydrostatic load (10-year event)
H_3 = hydrostatic load (1-year event)

Q_1 = equipment load (maximum downward)
Q_2 = equipment load (at-rest downward)
Q_3 = equipment load (maximum upward)
I = ice impact load
E = seismic load
F_s = side-seal friction load
F_t = trunnion friction load
$k_d = 1.2$

MATERIALS

For the analysis of the existing gate structure, the material specifications for the gates are provided in the original construction specifications, Section 79 (i), which states, "Tainter gate structural steel shall conform to Federal Specification QQ-S-741, type II, or ASTM Designation A7." Per this specification, the minimum yield point strength for a welded structure, with sections no more than 5/8 inch thick, is 33,000 psi. These values were used in analyzing the existing gate structure.

For the proposed 30-by-12-foot gate, the structural members will consist of structural steel. Embedded metals, including the side and bottom seal plates, should be corrosion-resistant steel. Table 7.3 indicates the selected materials for the various Tainter gate components, including ASTM standards, given normal conditions.

Table 7.3 Selected Materials for Tainter Gate Components

Component	Material Selection
Horizontal girders	ASTM A709, Grade 50
End girders and built-up sections	ASTM A709, Grade 50
Downstream vertical ribs	ASTM A709, Grade 50
Strut arms	ASTM A709, Grade 50
Strut arm bracing	ASTM A709, Grade 50
Skin plate	ASTM A709, Grade 50
Stiffener plates	ASTM A709, Grade 50
Lifting bracket	ASTM A514 steel, Grade F
Seal plates and bolts	ASTM A240, Type 304 stainless steel
Trunnion anchorage	ASTM F1554, Grade D
Trunnion bushing	Orkot C378 or equal stiffness and friction coefficient
Trunnion hub	ASTM A668, Class D, Grade X1
Trunnion pin	ASTM A705, Type 630, Condition H1150, steel forging
Guide wheel	ASTM A248, Grade 80-50

GATE ANALYSIS

Gate Analysis and Structural Modeling

SAP2000, Version 12 (Plus), was used for the structural modeling and analysis of the existing and new Tainter gates. For the analysis of the existing gates, an additional 2.5-foot extension was provided for the increased embankment elevation. The analysis was performed to confirm that the existing structure can accommodate the increased WSE and associated loading.

A three-dimensional model of the existing and new Tainter gates was created using frame and shell elements of SAP2000. Figures 7.1 and 7.2 show the general layout of the Tainter gates. The existing Tainter gate width is 16 feet and the height is 12 feet (with a 2.5-foot extension). The new Tainter gate width is 30 feet and the height is 12 feet.

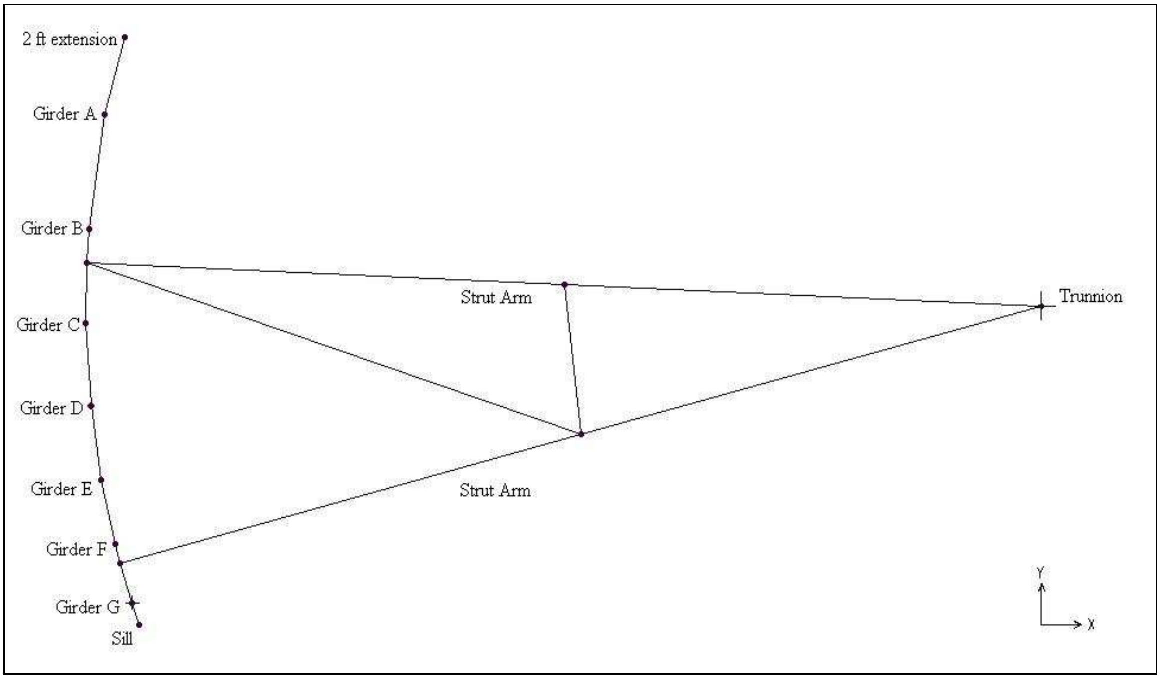

Figure 7.1 General Layout of Existing Tainter Gate with 2.5-Foot Extension

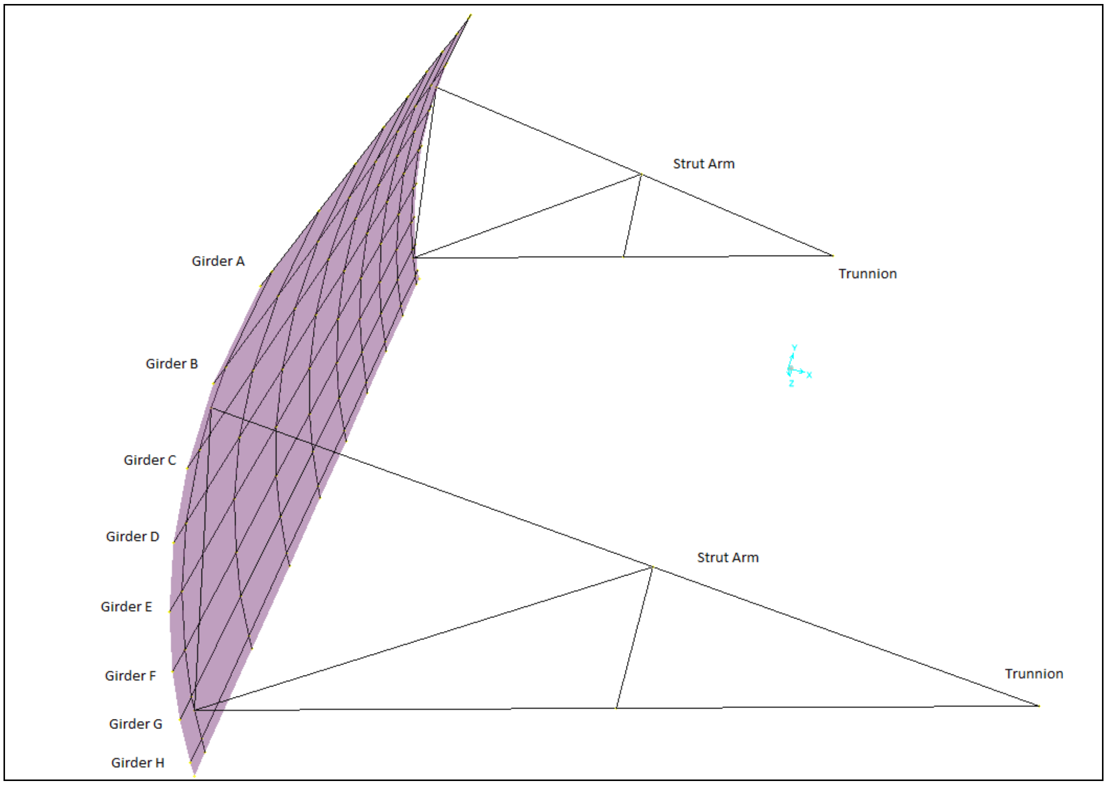

Figure 7.2 General Layout of New 30-by-12-Foot Tainter Gate

Existing Gate Analysis Results

The analysis results for the trunnion reactions, frame members, and skin plate of the existing Tainter gates are presented in this section. The plots resulting from the SAP2000 analysis are provided in Appendix E. The trunnion reactions for the load combinations in Table 7.2 are presented in Table 7.4.

Table 7.4 Trunnion Reactions for Existing Tainter Gates

Load Combinations	Fx at Each Trunnion (kips)	Fy at Each Trunnion (kips)
U1, U2, U3, U4	-48.87	-6.01
U5	-55.64	-6.06
U6, U7, U8, U9	-48.81	-6.11
U10	-63.15	-6.75
U12	-2.53	2.24
U13	-1.70	0.13
U14	-0.08	0.25

kip = 1,000 pounds

The DCRs for the frame members were determined from the SAP2000 finite element analysis, it was determined that Load Combination 5 ($U = 1.2\ D + 1.6\ M + 1.6\ C + 1.2\ H_3 + 1.0\ E$) is the controlling load combination. This load combination includes the factored dead, mud, ice, hydrostatic, and seismic loads. Table 7.5 shows DCRs for this load combination. Note that the code check module of SAP2000 does not consider the reliability factor α of 0.90 (EM 1110-2-2105, Section 3-4) for computing DCRs. See Appendix E for detailed hand calculations and plots from the finite element analysis.

Table 7.5 Demand to Capacity Ratios for Load Combination 5

Location	Maximum DCR
Girder A	0.31
Girder B	0.21
Girder C	0.23
Girder D	0.34
Girder E	0.50
Girder F	0.61
Girder G	0.91
End girder	0.33
Upper strut arm	0.65
Lower strut arm	0.83
Cross member	0.12

DCR = demand to capacity ratio

These DCRs are for the combined effects of axial force, minor axis flexure, and major axis flexure.

The stresses on the skin plate shell for Load Combination 5 are shown in Figure 7.3. The maximum factored stress is 33 ksi; this is a small area of stress concentration over three nodes where the lower strut arm connects to the skin plate. This is an anomaly of finite element modeling; discounting the stress concentration results in a maximum factored tensile stress of 24 kips per square inch (ksi). The maximum factored compressive stress is 19 ksi. The factored skin plate capacity is 29.70 ksi.

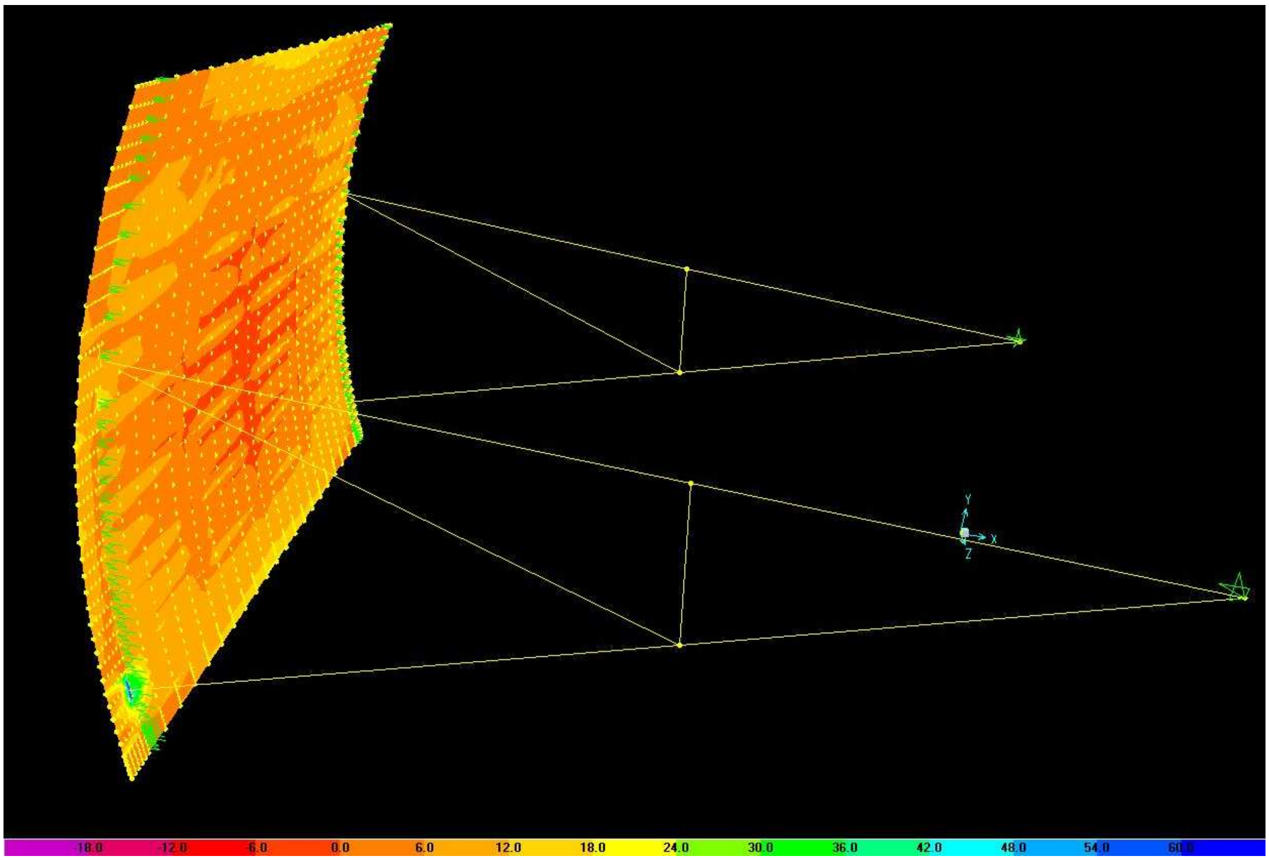

Figure 7.3 Stresses on Skin Plate Shell of Existing Tainter Gates

<u>New 30-by-12-Foot Tainter Gate Analysis Results</u>

The analysis results for the trunnion reactions, frame members, and skin plate of the new Tainter gates are presented in this section. See Appendix E for the plots resulting from the SAP2000 analysis.

The trunnion reactions for the load combinations in Table 7.2 are presented in Table 7.6. These reactions were used in the structural design of the proposed spillway structure, as described in the discussion of the gate analysis and structural modeling.

The DCRs of the frame members were determined from the SAP2000 finite element analysis, it was determined that Load Combination 5 (U = 1.2 D + 1.6 M + 1.6 C + 1.2 H_3 + 1.0 E) is the controlling load combination. This load combination includes the factored dead, mud, ice hydrostatic, and seismic loads. Table 7.7 shows DCRs for this load combination. Note that the code check module of SAP 2000 does not consider the reliability factor α of 0.90 (EM 1110-2-2105, Section 3-4) for computing DCRs. See Appendix E for detailed hand calculations and plots from the finite element analysis.

Table 7.6 Trunnion Reactions for New Tainter Gates

Load Combinations	Fx at Each Trunnion (kips)	Fy at Each Trunnion (kips)	Mz at Each Trunnion (kip-inches)
U1	-98.40	-13.70	0.00
U2, U3, U4	-98.40	-13.70	0.00
U5	-112.86	-14.31	0.00
U6	-98.44	-14.87	240.00
U7, U8	-98.44	-14.87	240.00
U9	-98.44	-14.87	0.00
U10	-110.98	-14.46	0.00
U11	-98.40	-13.70	0.00
U12	-4.92	-2.20	0.00
U13	-4.38	-2.32	0.00
U14	-0.07	-2.15	0.00

The DCRs are for the combined effects of axial force, minor axis flexure, and major axis flexure. The DCRs in Table 7.7 are low because deflections are controlling the design of the Tainter gates.

The frame member deflections in Table 7.8 show the deflection of each girder for an unfactored hydrostatic load from SAP2000. For the design of 30-foot Tainter gates, a limit for girder deflection is kept at span/360.

The skin plate shell stresses for Load Combination 5 are shown in Figure 7.4. The maximum factored stress is 33 ksi; this is a small area of stress concentration. Discounting the stress concentration results in a maximum factored tensile stress of approximately 18 ksi. The maximum factored compressive stress is 22.50 ksi. The factored skin plate capacity is 38.25 ksi.

Table 7.7 Demand to Capacity Ratios for Load Combination U5

Location	Maximum DCR
Girder A	0.18
Girder B	0.17
Girder C	0.25
Girder D	0.31
Girder E	0.35
Girder F	0.36
Girder G	0.38
Girder H	0.44
End girder	0.38
Upper strut arm	0.50
Lower strut arm	0.74
Cross member	0.13

DCR = demand to capacity ratio

Table 7.8 Deflections

Location	Ux, Deflection (inches)
Girder A	0.06
Girder B	0.21
Girder C	0.36
Girder D	0.52
Girder E	0.69
Girder F	0.82
Girder G	0.89
Girder H	0.92

New 30-by-12-Foot Tainter Gate Analysis Results for Overtopping Load Case

As agreed per the telephone conference call on January 8, 2010, regarding the Robles Diversion Dam Modification project and the USBR's comments, an overtopping load case was performed for the 30-by-12-foot Tainter gates. The following assumptions were used in the check:

- Head (H) = 18 feet (12-foot head plus 6-foot overtopping)
- Load factor = 1
- Seismic load = none
- Reliability factor (α) = 1
- Resistance factor (Ø) = 1

The reactions at each trunnion for the overtopping load case are indicated in Table 7.9.

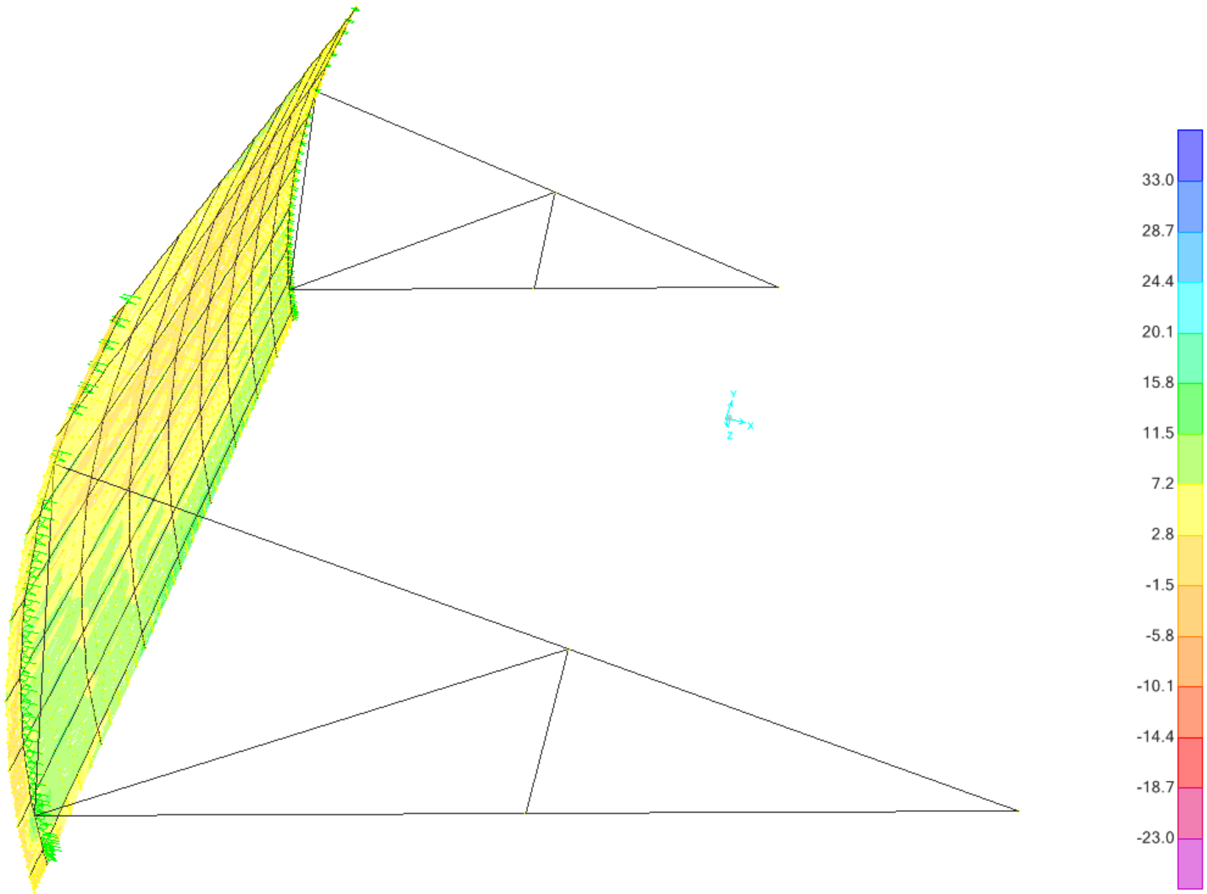

Figure 7.4 Stresses on Skin Plate Shell of New 30-by-12-Foot Tainter Gates, Load Combination 5

Table 7.9 Trunnion Reactions for Overtopping Load Case

Load Case	Fx at Each Trunnion (kips)	Fy at Each Trunnion (kips)	Mz at Each Trunnion (kip-inches)
Overtopping	-136.10	-8.66	0.00

Table 7.10 provides the DCRs for the overtopping load case. As the loads and capacities are kept unfactored, there is no significant difference in DCRs for the overtopping load case when compared to Load Combination U5.

Table 7.10 Demand to Capacity Ratios for Overtopping Load Case

Location	Maximum DCR
Girder A	0.60
Girder B	0.31
Girder C	0.30
Girder D	0.32
Girder E	0.34
Girder F	0.37
Girder G	0.40
Girder H	0.43
End girder	0.53
Upper strut arm	0.40
Lower strut arm	0.41
Cross member	0.13

DCR = demand to capacity ratio

Figure 7.5 shows the stress plot for the skin plate. The maximum skin plate stress for the overtopping load case is 25.2 ksi.

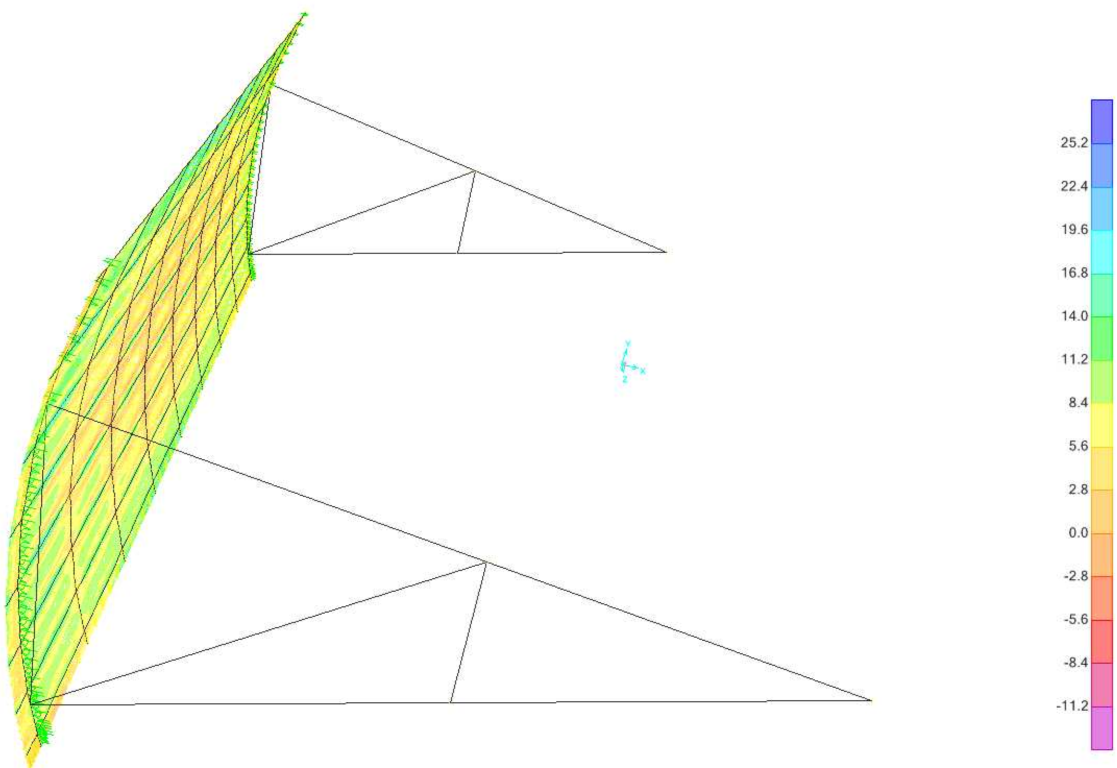

Figure 7.5 Stresses on the Skin Plate Shell of New 30-by-12-Foot Tainter Gate, Overtopping Load Case

In addition to SAP 2000 analysis, additional structural solid modeling and analysis was performed using SolidWorks at 90 percent design phase in order to improve the detail design. It should be noted that the unfactored loads were used for any stress analysis that was run using SolidWorks. It was determined from the analysis that the guide wheel load during a jammed condition is too large for the wheel to respond elastically. Therefore, the guide wheels were replaced by guide shoes located in approximately the same location. See Appendix E for the guide shoe computations.

The arm framing has also been improved so that the truss members intersect at a point. The flanges are not welded to the main arms because the welds are not required for load transfer and the weld would create high stresses in the flanges of the main arms. See Figure 7.6 for the finite element analysis stress plot of the arm to frame connection for the gate jammed condition.

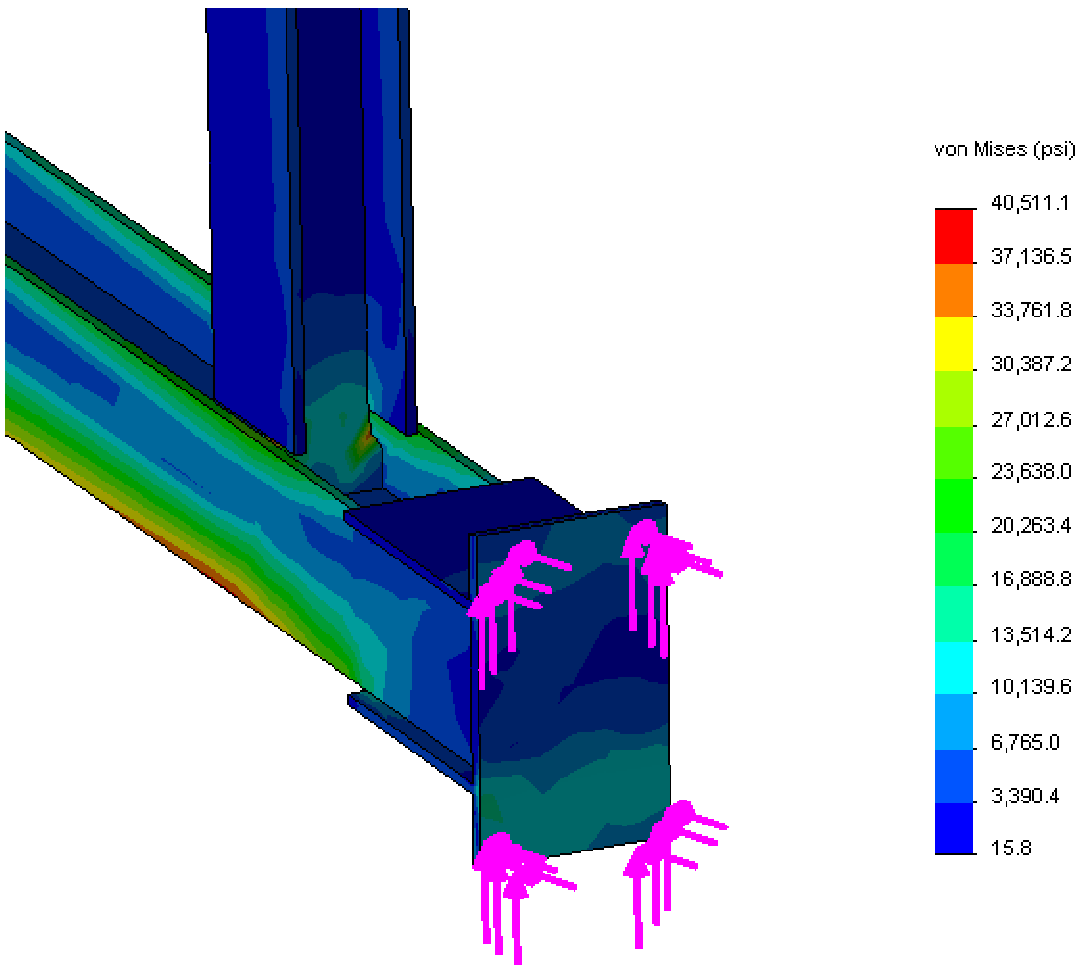

Figure 7.6 Frame Connection Stresses, Gate Jammed Condition

The hub connection was modified to allow machining of the high-strength material after welding on the flanges. The hub to arm connection was checked for stresses. The local finite element model for this connection was developed. The stress plot shown in Figure 7.7 shows that the partial connection provided by welding between the strut arm and the hub flange is adequate.

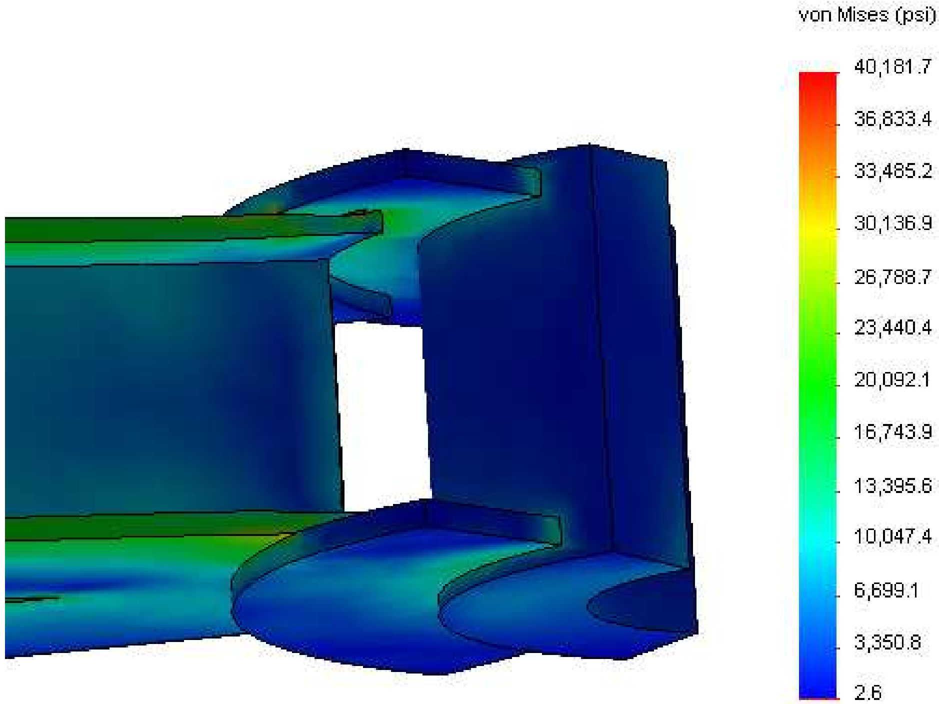

Figure 7.7 Maximum Trunnion Friction Plus Hydrostatic Load, Hub-Arm Connection

The ribs and vertical stiffeners were examined for their contribution to the hydrostatic load (Figure 7.8).

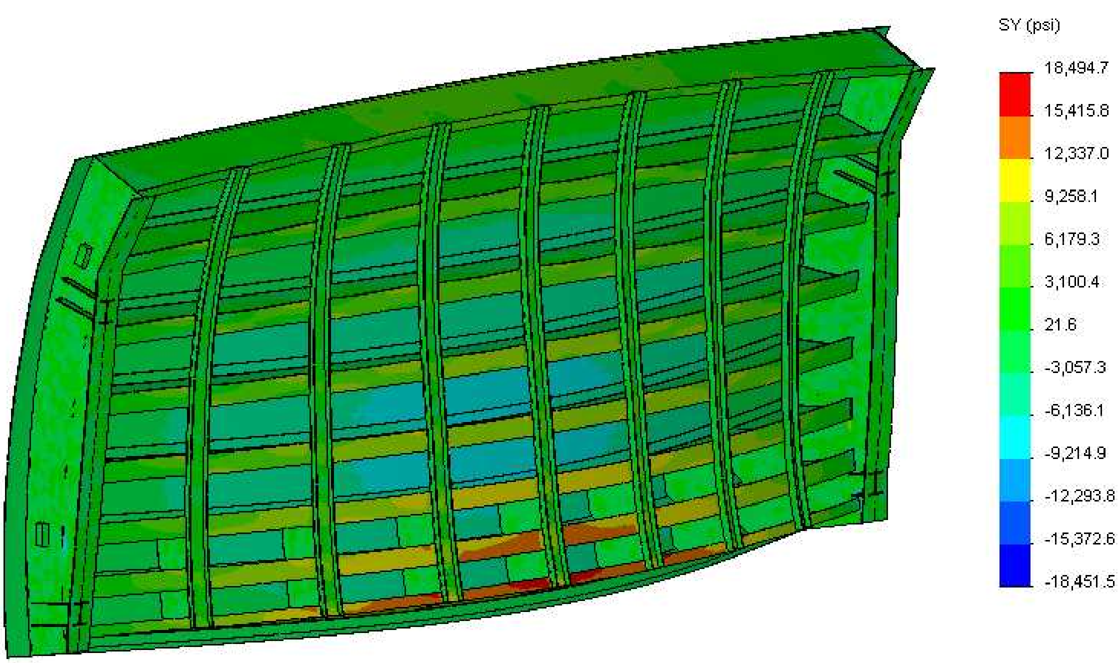

Figure 7.8 Bending Stresses, Maximum Unfactored Hydrostatic Load

Seals

The seals on the original gates have a J-bulb seal at the bottom of the gate. This is known to cause vibration when sluicing flow, when the bottom of the gate is in contact with flowing water. There are several alternative designs. The easiest to implement is to provide a compression, wedge seal at the bottom edge. This is not as flexible as a J-bulb seal and may allow a little leakage. However it is known to be less likely to vibrate during sluicing.

Corrosion Protection

To provide corrosion protection, some assumptions about the environment were made:

- The gate is normally closed, and the reservoir is normally dry.
- Rain occurs, but is infrequent. Solar exposure is daily, with minimum cloud cover.

Because the reservoir is normally dry, galvanic protection such as zinc or magnesium anodes are not be proposed. The primary corrosion protection system is paint.

Paint

The byproducts of some paints (volatile organic compounds) are known to harm the environment. Other paint components, such as coal tar, have toxic effects on the individuals applying the coatings. The proposed paint system has an excellent performance record and no volatile organic compounds or coal tar.

Bearing System

The original bearing system was a bronze alloy with counter-bored holes distributed around the cylinder. The holes are filled with a lubricant. The theory for this bearing is that the bronze provides strength/rigidity and the lubricant reduces friction. The criticism is that the many holes reduce the bearing area, increasing the contact stress, which can lead to galling, higher friction.
An alternative product used successfully in similar applications is a synthetic bushing. Low friction, Teflon material is captured in a fabric-stiffened polymer matrix. There are no points of high stress, rather the bearing stress is uniform. Furthermore, no additional lubrication is required. Therefore, the bronze alloy trunnion bushing is replaced with the synthetic bushing at the 90 percent design stage.

A synthetic bushing manufacturer with successful experience is Orkot, which has a range of products. This bearing application is rarely if ever submerged in lake water, although it is occasionally exposed to rain. Therefore, a low-friction dry-running bushing, Grade C378, was selected.

MECHANICAL SYSTEMS

This section describes the analysis of the mechanical systems of the existing Tainter gates and the new 30-by-12-foot Tainter gates.. The existing hoists and Tainter gate trunnions were checked against current criteria as a part of the mechanical systems.

Analysis of Existing Tainter Gate Hoist System

The hoist motor, wire rope, and drive shaft were checked against the CMAA 70 criteria. Table 7.11 summarizes the engineering computations for the mechanical components of the hoists. See Appendix E for detailed calculations.

Table 7.11 Analysis of Existing Hoist System

Component	Analysis Summary
Motor	For the rated capacity, required motor horse power (HP) is 0.44. Available HP of the existing motor is 1.5. Motor meets the criteria set by CMAA 70.
Wire rope	Wire rope meets the CMAA 70, Section 4.4.1, criteria. Wire rope has a safety factor of 7.73.
Drive shaft	Driveshaft meets the CMAA 70 deflection and strength criteria of Sections 4.11.3 and 4.11.4.

Analysis of Existing Trunnion Bearings

The existing Tainter gate trunnions were checked for bearing pressure. They were checked against the allowable bearing pressure of 5000 psi (EM 1110-2-2702, Section 4-3-b) for the unfactored load combination that produces maximum reaction at the trunnion (a combination of dead, mud, ice, hydrostatic, and earthquake loads). The calculations indicated that the maximum induced bearing pressure at the trunnion is 2,409 psi, which is well within the allowable limit of 5,000 psi. See Appendix E for the detailed computations.

Analysis of 30-by-12-Foot Tainter Gate Hoist System

The hoist system for 30-by-12-foot Tainter gates was determined by analyzing the required rated capacity, lifting speed, total lift, and wire rope pick point distance, with the following results:

- The required rated capacity of the hoist is 15 tons, which was computed by adding Tainter gate dead load, mud load, ice load, side-seal friction load, trunnion friction load, and 25 percent overload.

- The lifting speed of the Tainter gate is 2 feet per minute (fpm). Therefore, the hoists were sized for the required output speed of 2 fpm.

- Tainter gate hoist system should have minimum lift of 25 feet.

- The required wire rope pick point distance is 343 inches.

ELECTRICAL SYSTEMS

This section describes the existing electrical distribution system and the modifications to the electrical distribution and control system selected to support the new Tainter gates.

Existing Electrical Distribution System

The existing electrical distribution system consists of a 200-ampere (amp), 240/120-volt (V), three-phase, four-wire electrical service. The service conductors terminate in a service entrance rated disconnect switch. The electrical service is backed up by a 60-kilowatt (kW)/75-kilovoltampere (kVA) standby generator by means of a 200-amp automatic transfer switch. From the transfer switch, the conductors are routed to a 200-amp distribution panel "A." Panel A feeds three subdistribution panels and a control starter panel. Two of the subdistribution panels are 240/120-V, one-phase, three-wire service; the third panel and the starter control panel are fed by a 240/120-V, three-phase, four-wire service.

The service entrance disconnect and utility meter are located on the north face of the generator building.

A preliminary load estimate of the existing service indicates that the service is at or near capacity. A new service is expected to be required. A 12-month peak demand load of the facility will be obtained from the serving utility to confirm the existing service demand load.

Proposed Electrical Distribution System

Proposed is a new 200-amp, 480Y/277-V, three-phase, four-wire electrical service, including a new 200-amp service entrance rated panel, a new 200-amp automatic transfer switch, a new 200-amp distribution panel board, and a new 75-kVA, 480:240/120-V step-down transformer.

The new gate motors have been selected to function at 480-V, in three phases.

The gate motors will be fed by individual power feeders to each motor from a new starter panel. The new starter panel will be located in the existing control building with means to lock out the motor at the starter panel.

The existing control building appears to have sufficient space to house the new electrical distribution equipment. Where feasible, it is anticipated that the new electrical equipment will be installed in existing available space before the existing equipment is removed, thereby minimizing the system downtime.

Existing Controls System

The existing control system is to be retained and reused. The existing control panel does not have sufficient space on the enclosure front for new control and indicating devices; therefore, a new control panel will be provided and integrated with the existing control system. The existing control system hardware appears to have sufficient capacity for controlling and monitoring the new Tainter gates. There is an uninterruptable power supply (UPS) in the existing control panel.

Proposed Controls System

While there appears to be sufficient space in the existing control rack to accommodate the new gate control hardware, the activation buttons, annunciators, and display readouts will not fit on the exterior of the existing control cabinet; therefore, a new control panel will be provided. New digital input units are required in the existing control rack to control the new gates.

The new gate motors will be operated by means of new across-the-line starters, which will be located in a new starter panel located in the control building. The existing UPS in the existing control cabinet will help to minimize voltage dip effects on the PLC control system during motor starting.

On/off control will be located on the new control panel, as well as on the new starter panel. Control wiring will be routed from the starter panel back to the new control panel in the existing control building via surface-mounted conduit.

Existing Standby Generator System

The existing standby generator is a 60-kW/75-kVA, 240/120-V, three-phase, four-wire generator system. The generator is housed in a separate building adjacent to the control building. The automatic transfer switch is located in the generator building.

Proposed Modifications to the Existing Generator System

The USACE was presented with three options for providing a standby generator system:

- Option 1 is to leave the existing generator system as it is presently configured, providing standby power for the existing electrical distribution system. The four new gates would not have standby generator power back-up.
- Option 2 is to reconfigure the existing generator for 480Y/277-V, three-phase, four-wire service. The existing generator would still provide only 60-kW of standby power; therefore, interlocks would be required to prevent all systems from operating on the generator concurrently, but with selectability (such as a manual transfer switch between new gate motors and the existing system) such that at any given time, a predetermined block of equipment could be operated on the generator.

- Option 3 is to replace the existing generator with a new standby generator sized for the entire facility load. It is not known whether the existing generator building is of sufficient size to accommodate a larger generator.

Option 2 was selected as the preferred design for the standby generator power at the facility. A manual transfer switch and step-down transformer were added to allow selection of the preferred gates to operate on standby power.

Site Lighting

The new lighting poles will comply with lighting standards as required to meet the use requirements at the gates while minimizing light pollution and light spillage to the surrounding areas. The control of new lighting will be designed to provide light levels required for security purposes as well as maintenance/repair work while minimizing energy consumption. The lights will have photoelectric control, allowing the lights to function in low light, and a motion sensor will be provided in the control circuit to allow the lights to function only when motion in the area of the lights is detected (i.e., when someone is present on the operating machinery platform).

CONCLUSIONS

Analysis of Existing Tainter Gates

The existing leaves and operating equipment for the existing gates were analyzed to determine if they would be adequate for a higher reservoir elevation. The structural portions were evaluated in terms of the requirements of EM 1110-2-2702; the mechanical system was evaluated in terms of the requirements of CMAA 70. The structural members, skin plate, and hoisting equipment met the requirements. The existing gates will be provided with a 2.5-foot skin plate extension along the top of the Tainter gate. The extension will be welded onto the outside of the existing skin plate.

Analysis of New 30-by-12-Foot Tainter Gates

The leaves and operating equipment for the new gates have been designed in accordance with the requirements of EM 1110-2-2702; the mechanical system was evaluated in terms of the requirements of CMAA 70. The structural members, skin plate, and hoisting equipment met the requirements.

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8. CARE OF HABITAT DURING CONSTRUCTION

The area to be disturbed during construction is within an environmentally sensitive area. To limit the impact on this sensitive habitat, the work area footprint will be limited to avoid unnecessary destruction of native plants and species. The specific work area has been identified in the plans and specifications and also in the *Engineering Considerations and Instructions for Field Personnel (ECIFP).*

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9. CARE AND DIVERSION OF WATER DURING CONSTRUCTION

Surface flows within the construction area will be controlled by dikes, diversion pipes, and pumps. Excavation for the spillway, embankment, and rock-armored ramp can be conducted throughout the year, but during the rainy season, the weather should be monitored for storm activity and the contractor should protect the work site from storm flows. Groundwater should not be encountered at the proposed construction depths outside of the rainy season.

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10. DISPOSAL OF MATERIALS

There are at least two optional disposal sites for the excess soils to be excavated for the project. One is within the project limits, either directly north of the existing operation office and/or along the east overbank of the rock armored ramp. Additionally, at the time of construction, the commercial potential for selling the excess soil will be reviewed. The project cost estimate assumes that the material will be disposed of on site. It is expected that most of the excess material will be suitable for commercial use and will possibly be disposed of in locations outside the project limits.

The rubble and the asphalt on the site will be disposed of at a suitable location. A local pavement recycling center will take the rubble and the asphalt at a cost of $XX per ton.

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11. ENVIRONMENTAL ASSESSMENT

A Supplemental Environmental Assessment for the proposed improvements is currently being prepared by the Los Angeles District. The findings will be included in the next version of the DDR as an appendix and incorporated into the contract drawings.

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12. COST ESTIMATES

The estimated cost of the project will be determined at a later date. Actual unit prices will be established by the bid of the winning contractor, and all quantity amounts should be reviewed and verified by the contractor before bid placement.

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13. RECOMMENDATIONS

This report describes in detail the general design, including departures from the previously approved plan, of a portion of the Matilija Dam Ecosystem Restoration Project. It is recommended that this report serve as the basis for the development of plans and specifications for the construction of the Robles Diversion Dam Modification portion of the Matilija Dam Ecosystem Restoration Project.

The rounded combined federal and non-federal first costs of the recommended Robles Diversion Dam Modification are estimated at $xx,xxx,xxx based on October 2001 price levels.