Matilija Dam
Ecosystem Restoration Project

Robles Diversion Dam Modification
Design Documentation Report

Ventura County, California

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SYLLABUS

This Design Documentation Report (DDR) presents the results of the design of the Robles Diversion Dam design modifications, 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* (EIS/EIR), 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 (VCWPD).

The existing Robles Diversion Dam (Robles) is located on the Ventura River, approximately 14 miles from the mouth of the river and two miles downstream of the Matilija Dam. Robles, located in an unincorporated portion of Ventura County, California, is owned by the U.S. Bureau of Reclamation (USBR), and operated by the Casitas Municipal Water District (CMWD). Robles operates 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 critical habitat for the endangered Steelhead Trout (*Eucyclogobius newberryi*).

When Matilija Dam is removed, a significant increase in the sediment load is anticipated, which would negatively affect the operation of Robles Diversion Dam. The proposed improvements would alleviate the negative impact affecting the operation of the Robles Diversion Dam. The design modifications to Robles are based upon the selected alternative in the DPR, Alternative 4b, and will consist of a high flow bypass (HFB) spillway with four 30-foot tainter gates, stilling basin, and high flow fishway/ladder. Additionally, the existing dam embankments will be raised and an armored rock ramp spillway provided for the embankment. The construction of the HFB and appurtenances is a mitigation component of the overall Matilija Dam removal project. The only deviations from the selected alternative are the addition of the fish bypass, as required from the resource agency coordination with the National Marine Fisheries Service, and the Rock Ramp spillway. Documentation of the selected alternative design details was provided in a Memorandum, dated June 13, 2007, from XXX XXXX to the Corps of Engineers and is included as Appendix X.

REPORTS PREVIOUSLY ISSUED

Reports previously issued by the U. S. Army Corps of Engineers and others are:

a. "*Final Environmental Impact Statement/Environmental Impact Report for the Matilija Dam Ecosystem, Ventura, California*", Corps of Engineers, 2004.

b. "*Matilija Dam Ecosystem Restoration Feasibility Study, Ventura, California*", Corps of Engineers, 2004.

c. "*Matilija Dam Ecosystem Restoration Project Management Plan –Design Phase, Ventura, California*", Corps of Engineers, June 2005.

d. *"Hydrology, Hydraulics and Sediment Studies of Alternatives for the Matilija Dam Ecosystem Restoration Project, Ventura, California – Final Report"*, Technical Service Center, Bureau of Reclamation, Denver, CO 80225. Greimann, B.P., (2004).

e. " *Hydrology, Hydraulics, and Sediment Studies for the Matilija Dam Ecosystem Restoration Project, Ventura, California – DRAFT Report"*, Technical Service Center, Bureau of Reclamation, Denver, CO 80225, Greimann, B.P., (2004).

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

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

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

REFERENCES

1. Bureau of Reclamation, United States Department of the Interior, 1987. *"Design of Small Dams",* Third Edition.

2. Manuela Escarameia, 1998. *"River and Channel Revetments – A Design Manual",* Thomas Telford Publications.

3. EM 1110-1-1807, *"Standards Manual for USACE Computer-Aided Design and Drafting (CADD) Systems"*, 30 July 1990.

4. EM 1110-2-1302, *Civil Works Cost Engineering*, U.S. Army Corps of Engineers, 31 March 1994.

5. EM 1110-2-1601, *"Hydraulic Design of Flood Control Channels"*, U.S. Army Corps of Engineers, 30 June, 1994.

6. EM 1110-2-2000, *"Standard Practice for concrete for Civil Works Structures"*, U.S. Army Corps of Engineers, Change 2, 31 March 2001.

7. EM 1110-2-2007, *"Structural Design of Concrete Lined Flood Control Channels"*, U.S. Army Corps of Engineers, 30 April 1995.

8. EM 1110-2-2102, *"Waterstops and Other Preformed Joint Materials for Civil Works Structures"*, 30 September 1995.

9. EM 1110-2-2104, *"Strength Design for Reinforced Concrete Hydraulic Structures"*, June 1992.

10. EM 1110-2-2502, *"Retaining and Flood Walls"*, 29 September 1989.

11. ER 1110-2-1150, *"Engineering and Design for Civil Works Projects"*, 31 August 1999.

12. ER 1110-2-1806, *"Earthquake Design and Evaluation for Civil Works Projects"*, 31 July 1995.

13. ETL 1110-2-256, *"Sliding Stability for Concrete Structures"*, 24 June 1981.

14. ETL 1110-2-322, *"Retaining and Flood Walls"*, October 1990.

15. ETL 1110-2-307, *"Flotation Stability for Concrete Hydraulic Structures"*, 20 August 1987.

16. Design Guide, *"Structural Design of Flood Control Channels", U.S. Army Corps of Engineers, Los Angeles Districts, October 29, 1998.*

17. American Concrete Institute, *"Building Code Requirements for Reinforced Concrete, ACI 318M-95 and Commentary, ACI 318RM-95.*

18. American Institute of Steel Construction (AISC), *"Manual of Steel Construction"*, ASD, Ninth Edition, 1989.

19. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Concrete Strength Investigation and Design (CASTR)"*, May 1987.

20. Waterways Experimental Station (WES), Corps of Engineers Computer Program, *"Analysis of Retaining and Flood Walls (CTWALL)"*, 30 October 1993.

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

PERTINENT DATA

Purpose: Water Diversion

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	15,000 cfs
10-year peak discharge at Robles Diversion Dam	15,000 cfs
Existing Gate/Spillway Structure Capacity (1-10'x9.5' & 3-16'x9.5' Radial Gates)	6,000 cfs
Proposed Gate/Spillway Structure Capacity (4-30'x12' Radial Gates)	11,000 cfs
Rock Ramp Design Flow Rate	11,000 cfs
[1] Robles Diversion Dam Design Capacity	19,000 cfs
Existing Diversion Canal Capacity (3- 11' x 10.5 Radial Gates)	500 cfs
Existing Crest Elevation	767.00 +/-
Existing Crest Length	350 feet
Proposed Crest Elevation	769.00
Proposed Crest Length	150 feet

[1] Design flows based upon Army Corps of Engineers memorandum, *"Memorandum for Matilija Dam Ecosystem Restoration Study, Project Deliver Team"*, dated April 3, 2009

ABBREVIATIONS AND ACRONYMS

AE	Architect Engineer
AR	Army Regulation
CECWAR	Policy Review Branch, Policy Division, Civil Works Directorate
CECWE	Engineering and Construction Division, Civil Works Directorate
CECWEP	General Engineering Branch, Engineering and Construction Division, Civil Works Directorate
CEGS	Corps of Engineers Guide Specification
CEMPEV	Value Engineer Office, Military Programs Directorate
CFR	Code of Federal Regulations
DDR	Design Documentation Report
DM	Design Memorandum (Obsolete)
EC	Engineer Circular
EDR	Engineering Decision Report
EM	Engineer Manual
ENG Form	Corps of Engineers Form
EP	Engineer Pamphlet
ER	Engineer Regulation
FCSA	Feasibility Cost Sharing Agreement
FRC	Feasibility Review Conference
GDM	General Design Memorandum (Obsolete)
GFR	Government Furnished Property
GRR	General Reevaluation Report
HQUSACE	Headquarters, U.S. Army Corps of Engineers
HTRW	Hazardous, Toxic, and Radioactive Waste
IDC	Initial Design Conference
IPR	In Progress Review
IRC	Issue Resolution Conference
ITR	Independent Technical Review

LRR	Limited Reevaluation Report
MCACES	Microcomputer Aided Cost Engineering System
MFR	Memorandum for the Record
MP	Management Plan
MSC	Major Subordinate Command
NEPA	National Environmental Policy Act
NED	National Economic Development
O&M	Operation and Maintenance (generally used in reference to PreWRDA 1986 projects with Federal operations and maintenance)
OMRR&R	Operation, Maintenance, Repair, Replacement and Rehabilitation (generally used in reference to PostWRDA 1986 projects with sponsor operations and maintenance)
P.L.	Public Law
P&S	Plans and Specifications
PCA	Project Cooperation Agreement
PDT	Product Delivery Team or Project Delivery Team
PED	Preconstruction Engineering and Design
PES	Project Executive Summary
PM	Project Manager
PMP	Project Management Plan (obsolete, see Management Plan)
PSP	Project Study Plan (obsolete, see Management Plan)
PRB	Project Review Board
RRC	Reconnaissance Review Conference
SACCR	Schedule and Cost Change Request
TM	Technical Manual
TRC	Technical Review Conference
USACE	U.S. Army Corps of Engineers
VE	Value Engineering
WBS	Work Breakdown Structure
WRDA86	Water Resources Development Act, 1986

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1. INTRODUCTION

GENERAL

1.1 The U.S. Army Corps of Engineers Los Angeles District, in conjunction with the United States Bureau of Reclamation (USBR), Casitas Municipal Water District (CMWD), and Ventura County Water Protection District, completed the Feasibility Report and EIS for the Matilija Dam Removal Project in December 2004. The recommended plan addressed the increased sediment supply and impacts to the existing Robles Diversion Dam 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 x 12 foot high tainter gates, stilling basin, and high flow fishway/ladder. Additionally, the existing dam embankments will be raised to elevation 769 and an armored rock ramp spillway provided for the embankment. The plan provides a 20-year level of protection for the diversion structure and is a sediment mitigation component of the overall Matilija Dam removal project.

1.1.1 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

1.2 The Robles Diversion Dam Modification Project is prepared 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

1.3 The purpose of this Design Documentation Report (DDR) is to provide the basis for design of the Robles Diversion Dam Modification flood control project along the Ventura River. The project purpose is to provide mitigation for the increased sediment loading and flood flows 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

General

1.4 This Design Documentation Report presents the design for the recommended plan, the estimated construction cost, and the schedule for the Robles Diversion Dam Modification project. Robles Diversion Dam was originally built in 1958 and it 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 critical habitat for the endangered Steelhead Trout (*Eucyclogobius newberryi*)) upstream of the Robles Diversion Dam.

1.4.1 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 (1-10'x 9.5' Radial gate and 3-16'x 9.5' Radial gates), a gate-controlled canal diversion structure with debris barrier (3-11.5'x 10.5' Radial gates), and a fish ladder (Figure 1-2).

1.4.2 The recommended plan, to mitigate the large increases in sediment from the removal of the Matilija Dam, includes the design and construction of a high flow bypass (HFB) spillway consisting of four 30-foot wide x 12-foot high tainter gates, USBR stilling basin, and an additional high flow fishway/ladder. To accommodate the additional fish ladder and provide better operational ability, the existing dam embankments will also be raised to elevation 769. An armored rock ramp spillway is provided for the embankment and downstream channel bed to protect the diversion dam from scour damage. The rock ramp will also increase the diversion dam's storm capacity to a 20-year level of protection. See Plates for additional design details.
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Surveying and Mapping

1.5 T he mapping is based on Lidar method aerial topography flown in February 2005 at a scale of 1 inch = 100 feet, with 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

1.6 Subsurface investigations were performed by separate consultants under contract to the Corps of Engineers for the design of the Robles Diversion Dam Project and are presented in the Geotechnical Appendix.

Coordination with Others

1.7 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 the creek, dam safety considerations, and potential sources of water, disposal sites, and maintenance features.

a. Coordination occurred with the Local Sponsor. The local sponsor for the project is Ventura County Water Protection District (VCWPD).

Contact Person:
Ms. Norma Camacho
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800 S. Victoria Avenue
Ventura, Ca 93009-1600

i. Rights-of-Way. The boundaries of the project are fairly well defined along existing county rights-of-way and easements. The plans developed in this memorandum are based on topographic mapping obtained in 2005. Rights-of-way requirements will be established in detail prior to completion of plans and specifications.

ii. 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.

iii. 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 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.

iv. Maintenance Items. Required maintenance features have been coordinated with the local sponsor and the project team.

Maintenance Access. A 20-foot-wide roadway for maintenance access into each end of the existing diversion dam will be provided and connect with the access roads provided with 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 utilized as a grade control structure for the rock ramp channel. Existing all weather maintenance and access roads will remain in place without modification.

b. Coordination with Other Agencies included:

United States Bureau of Reclamation (USBR)

United States Fish and Wildlife Service (USFWS)

National Marine Fisheries Service (NMFS)

Ventura County Water Protection District (VCWPD)

Casitas Municipal Water Districts (CMWD)

PROJECT LOCATION AND DESCRIPTION OF DRAINAGE AREA

1.8 The Robles Diversion Dam (Robles) is located on the Ventura River, approximately 14 miles from the mouth of the river and two miles downstream of the Matilija Dam. Robles, located in an unincorporated portion of Ventura County, California, is owned by the U.S. Bureau of Reclamation (USBR), and operated by the Casitas Municipal Water District (CMWD). Robles operates 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 along the Ventura River and Matilija Creek in Ventura County California (See Figure 1-1).

Figure 1.1 Project Location Map

Image from MSN Live Search

Figure 1.2 Existing Robles Diversion Dam

2. SELECTED PLAN

GENERAL

2.1 The selected plan for the Robles Diversion Dam Modification Project consists of the addition of four 30' x 12' radial gates high flow bypass structure (HFB) adjacent to the existing spillway structure (consisting of one 10'x 9.5' and three 16'x 9.5' Radial Gates). An additional fish passage will be constructed between the proposed rock ramp spillway and the HFB structure. The fish passage is proposed to allow for migration of the endangered Steelhead Trout (*Eucyclogobius newberryi*) during large flow events and will be designed as a Streaming Flow Fishway. To increase operating efficiency of the diversion structure and fishway, the existing embankment will be raised by approximately 2 ft. A concrete sill will be placed across the crest of the raised embankment to control the weir elevation and the forbay depth. Since the existing gates are only 9.5 ft in height, a 2 ft extension will be connected to the existing gates to increase their depth capacity. A rock 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. This will assist in preventing any 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.

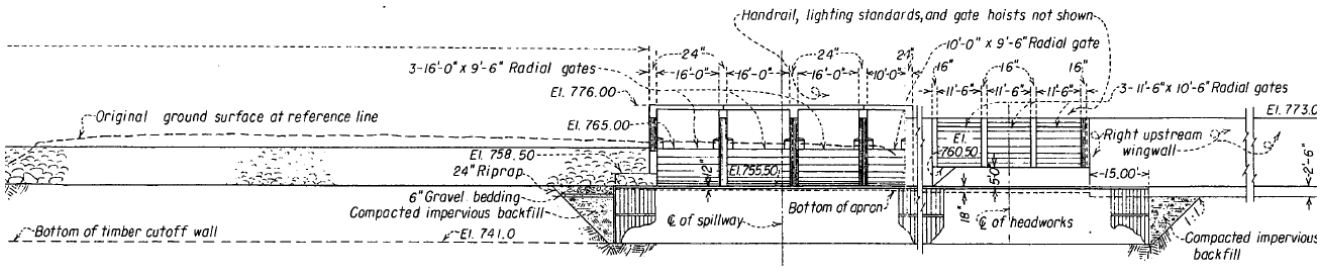

Figure 2.1 Pre-Project Elevation

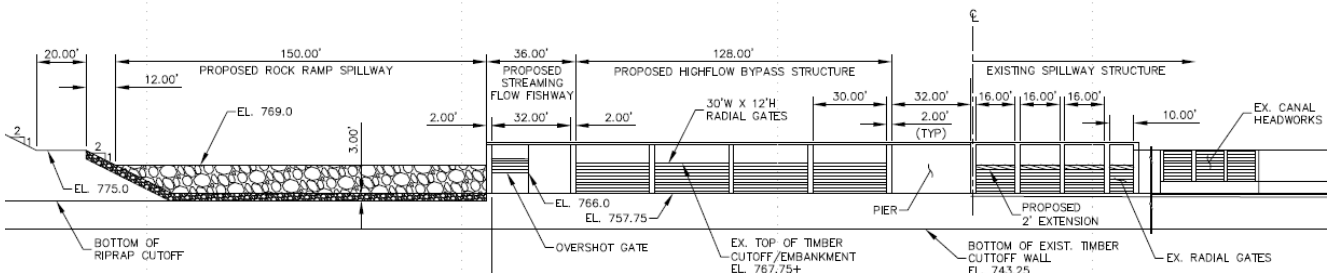

Figure 2.2 Post-Project Elevation

3. HYDROLOGIC AND HYDRAULIC BASIS FOR DESIGN

ROBLES DIVERSION DAM HYDROLOGY AND HYDRAULICS

3.1 This section describes the hydrologic and hydraulic analyses that were conducted to support the design of the Robles Diversion Dam modifications and the overall Matilija Dam Ecosystem Restoration Project. The hydrologic and hydraulic analyses for the project include rainfall-runoff modeling for the with-project conditions, numerical sedimentation analysis, and a physical hydraulic and sediment model of the baseline and with-project condition. 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 United States Bureau of Reclamation included in Appendix B.

3.1.1 As described in the project DDR, the 100-year design discharge for the Ventura River at the Robles Diversion Dam is 27,100 cu.ft./sec. For the Robles Diversion Dam, USBR performed a numerical model and physical model for the baseline and with project condition. In the analysis, various locations and modifications were considered to optimize the design of the high flow bypass structures. A copy of the detailed analysis is provided in Appendix B.

4. GEOTECHNICAL BASIS FOR DESIGN

Excerpts from the Geotechnical Appendix are provided herein. For more detailed information, please refer to Appendix C.

GENERAL

4.1 The project area is located within the Ventura River approximately 14 miles from the mouth of the river and two miles downstream of the Matilija Dam. The site generally consists of bars of course-grained material (gravel, cobbles, and boulders) which has formed near the mid-channel both upstream and downstream of the diversion structure. The river channel is about 10 to 15 feet below the eastern and western banks. The site does have a high groundwater table, which is susceptible to seasonal variations and flows. Additionally, due to the high ground water and presence of loose soils in the upper xx feet, the site is susceptible to liquefaction during a large earthquake event. Although site sight conditions allow for the possibility of liquefaction the probability of liquefaction is low, this is further discussed in section 4.x

4.2 Based upon the available drawings the diversion dam is a zoned earthfill and rockfill embankment. To help mitigate seepage a 15 to 20 ft deep trench of "compacted impervious backfill" was constructed upstream and downstream of a timber cutoff wall. The dam embankment was originally approximately 530 ft, but is currently only about 350 ft across the river bottom.

Selected Design Values

4.3 The design values are selected based on review of geotechnical investigations and reports previously performed by others. The values provided are based upon, properties of the in-situ soils, comparison of engineering properties of soil with similar materials from previous investigations, and engineering judgment. These values can be used for calculation of the earth pressure on the structures and retaining walls and slope stability of the embankment fills. Selected design values are presented in Table-4.1.

Table-4.1 Design Values

Unit Weights - Backfill and Embankment:	
Dry unit weight	118 pcf
Moist unit weight	130 pcf
Saturated unit weight	142 pcf
Drained Strength:	
Internal angle of friction, ϕ	33 deg
Cohesion, c	0
Undrained Strength:	

Internal angle of friction, ϕ	27 deg
Cohesion, c	800 psf
Lateral earth pressure coefficient:	
Active earth pressure coefficient, K_A	0.30
At-rest earth pressure coefficient, K_0	0.45
Passive earth pressure coefficient, K_P	2.50
Friction angle between wall and backfill material:	24 deg

COMPACTED FILL AND BACKFILL

4.4 Sufficient quantity of satisfactory material for compacted fills and backfills can be obtained from required basins excavation. Satisfactory materials include materials classified in accordance with ASTM D 2487 as GW, GP, GM, GC, SW, SP, SM, and SC and will be free of trash, debris, and organic matter, or material larger than 3/4 of the lift thickness in any dimension. Fill and backfill material will be placed in horizontal layers, which after compaction will not exceed 12 inches in depth for rubber-tired or vibratory rollers, 8 inches in depth for tamping and sheep-foot rollers, or 4 inches in depth when a mechanical tamper is used. Generally, excavated materials from basins would be considered a major source of the fill and backfill material.

LIQUIFACTION

4.5

CONSTRUCTION MATERIALS

Embankment Fills and Backfills

4.6 Alluvial materials available from the required excavations will be suitable for the construction of the embankment fills and backfills provided that trash debris and other discarded construction materials are not included.

Stone

4.7 Several commercial sources of suitable quarry stone for slope protection and grade control stabilizers are located within an xx-mile radius of the project site. Graded stones that would meet the requirements for stone work could be obtained from rock processing plants such as xxx Quarry. The xxx quarries have been used for the xxxx project, producing good quality limestone/marble.

Concrete

4.8 Sufficient quantities of concrete will be available from ready-mix suppliers in the project vicinity. Currently, xxx major ready-mix concrete producers are operating in the Ventura area. These sources are the xxx and xxx Company, and xxx Company. These sources have been used

extensively for concrete construction in Ventura County, in local commercial structures, for the California Department of Transportation (Caltrans), and in flood control projects constructed by Ventura County Watershed Protection District.

5. CIVIL BASIS OF DESIGN

GENERAL

5.1 This section presents the description of project features requiring civil designs and criteria to be used in their design. The project features the raising of the existing diversion embankment and the construction of a rock ramp channel and spillway. Additionally, the existing embankment will be extended to join the Meiners Oaks Levee improvements currently being performed by the Corps of Engineers.

Reference Documents

5.2 Design of the embankment modifications and rock ramp channel and spillway were based on the following Government and civilian publications:

- Bureau of Reclamation, United States Department of the Interior, 1987. *"Design of Small Dams",* Third Edition.
- ER 1110-2-1150, *"Engineering and Design for Civil Works Projects"*, 31 August 1999.
- 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 1957 Construction Specifications for the Robles Diversion [Provided by CMWD Staff.]
- *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc. Oakland, CA, November 12, 2008.
- Army Corps of Engineers, Los Angeles District, *"Memorandum for Matilija Dam Ecosystem Restoration Study, Project Deliver Team"*, 3 April 2009.

DAM EMBANKMENT MODIFICATION

5.3 To assist in the operation of the diversion dam and fish passage, the existing embankment will be raised approximately 2 feet to elevation 769.00. A concrete sill will be provided to control the weir elevation of the raised embankment. The existing embankment will be raised and lined with rip rap rock to prevent scour. The existing timber cutoff wall and 15 to 20 ft deep trench of "compacted impervious backfill" upstream and downstream of the timber cutoff wall will remain, with the proposed concrete sill cutoff wall extended into the impervious backfill to limit seepage. The embankment will also extend to connect with the upstream limits of the Meiner Oaks Levee improvements.

ROCK RAMP CHANNEL AND SPILLWAY

5.4 Downstream of the High Flow Bypass structure (HFB) and USBR stilling basin, a rock ramp will be provided to provide additional dissipation of flow velocity and protection of the river invert from scour. The rock ramp was designed with the in accordance with the

Memorandum for Matilija Dam Ecosystem Restoration Study, Project Deliver Team, Army Corps of Engineers, 3 April 2009 and the subsequent USBR design memorandums.

5.4.1 The rock ramp will join the existing river channel approximately 400 feet downstream. The slope of the rock ramp will vary due to the difference in sill elevations of the existing stilling basin and the proposed basin, elevation 751.0 and 753.25 respectively. To account for this elevation difference the rock ramp directly downstream of the existing structure will have a gradient of 1.5%. From the existing structure, the rock ramp will have a transverse cross gradient of 0.6%, additionally the rock ramp gradient downstream of the HFB structure will be 2.0%. The gradient of the rock ramp was designed to maintain sediment passage downstream of the Robles Diversion structure.

5.4.2 The storm capacity of the existing and proposed bypass structures is approximately 16,000 cu.ft./sec. To increase the high flow diversion capacity of the Roles Diversion Dam, a rock ramp spillway was provided adjacent to the proposed HFB structure. Due to the steep gradient (11.4%), the rock ramp is will be a grouted rip rap and will have an embankment height of 6 feet. The design of the rock ramp spillway is to increase the design capacity of the system to 19,000 cu.ft./sec, but also to protect the structures from larger flood events above 19,000 cu.ft./sec. The rock ramp spillway should flood flows up to 3,000 cu.ft./sec without damage and flows up to the 100 year return period without catastrophic damage to the Robles Diversion Dam.

STREAMING FLOW FISHWAY

5.5 To be analyzed and designed with 60% Plans, Specifications, and DDR.

6. STRUCTURAL BASIS OF DESIGN

GENERAL

6.1 T his section presents the stability and seismic analyses performed for the different features of the project requiring structural design. The project features are grouped into the following structural components: existing spillway, new spillway, fish ladder, baffle walls, and equipment supports. The design for these structural components can be found in Appendix D.

6.1.1 The existing spillway will be checked for stability under the loading conditions set forth in the following sections. 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.

6.1.2 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.

6.1.3 For the concrete strength design, the existing spillway will be broken into components to include the baffle walls, foundations, and corbels. Each component will be designed to meet the environmental factor (Sd) described in ACI 350. 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 ACI 350 and ACI 318.

6.1.4 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 using by the geotechnical engineer using the Flow Net Analysis Method.

6.1.5 For the concrete strength design, the fish ladder will be designed to meet the environmental factor (Sd) described in ACI 350. The strength design will be in accordance with ACI 350-06 and ACI 318-05.

6.1.6 Once 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

6.2 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
- EM 1110-2-2100, *"Stability Analysis of Concrete Structures"*
- EM 1110-2-2104, *"Strength Design of Reinforced Concrete Hydraulic Structures, Change 1, August 2003"*
- EM 1110-2-2200, *"Gravity Dam Design"*
- EM 1110-2-2502, *"Retaining and Flood Walls"*
- ER 1110-2-1806, *"Earthquake Design and Evaluation for Civil Works Projects"*
- ACI 318-05: *Building Code Requirements for Structural Concrete and Commentary*
- ACI 350-06: *Code Requirements for Environmental Engineering Concrete Structures and Commentary*
- *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
- 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.]
- *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc. Oakland, CA, November 12, 2008.

Engineer Manuals are referred to in abbreviated form, i.e. EM 2100, in this report.

DESIGN CRITERIA

Stability Analysis Method

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

6.3.1 The stability analysis was performed using the gravity method. The gravity method assumes that the dam structure is a rigid two dimensional block with a linear foundation pressure distribution. This method is applicable to dams that are regular in shape and are not curved or have 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 dam spillways. The mathematical model used to determine both dams' stability was developed using Microsoft Excel and can be found in Appendix D.

6.3.2 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 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.

6.3.3 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 from EM 2100 guidelines. 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 the overturning stability safety factor (Mr/Mo) to be calculated.

Table 6-1 Minimum Stability Criteria

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

6.3.4 The sliding stability safety factor was determined per Equation 5-3 in the EM 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 of 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

6.3.5 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 below.

$$Bearing\,Pressure = \frac{V}{A} \pm \frac{M}{S} \tag{6-2}$$

where

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

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

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

Loads

6.4 The required loads and loading combinations required for analysis are outlined in EM 2100 and EM 2502.

6.4.1 Two levels of earthquakes and associated performance objectives are defined for the project: Operational Basis Earthquake and Maximum Design Earthquake.

- OBE = 0.318g [Geotechnical engineer]
- MDE = 0.633g [Geotechnical engineer]

6.4.2 Operational Basis Earthquake
The Operational Basis Earthquake (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 non-critical components 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 the OBE earthquake loading, the structural response of the spillway shall remain essentially elastic under this earthquake loading.

6.4.3 Maximum Design Earthquake
The Maximum Design Earthquake (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 shall not exceed the prescribed residual deformations. Walls shall remain stable for the normal loading condition under the permanently deformed state. Essential structures may exhibit some visible damage, but shall be limited to narrow flexural cracking of concrete and the onset of yielding in steel.

6.4.4 Equivalent Earth Fluid Pressure
The equivalent earth fluid pressure was provided by the geotechnical engineer, see Section #.#. The analysis assumed active earth pressure was applied to the upstream face of the spillway dam from the top of the seepage barrier to the bottom of the upstream foundation shear key. It was also assumed that the soil downstream of the spillway may not be present and was not included in the analysis.

6.4.5 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 XX.

6.4.6 Earthquake Earth Pressure

The lateral earthquake earth pressure forces were determined by EM 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.

6.4.7 Earthquake Inertia Force

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

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

where

Pe_x = horizontal inertia force
M = mass of element (dam)
a_x = horizontal earthquake acceleration
W = weight of element (dam)
g = acceleration of gravity
α = seismic coefficient

6.4.8 Hydrodynamic Force

The hydrodynamic force was determined per EM 2200. This force is considered to be parabolic and determined using Westergaard's equation, equation 6-5 below, 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 (ft)

6.4.9 Dynamic Soil Pressures

The dynamic soil pressures were determined using the Mononobe-Okabe theory as follows.

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

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

where

ϕ_b: Internal friction angle of soil.
k_h: Horizontal seismic coefficient [acceleration in g's]
k_v: Vertical seismic coefficient [acceleration in g's]
θ: Angle between back face of wall and vertical.
β: Slope of backfill.
δ: Wall friction angle.

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

6.4.10 Seismic Coefficient Method

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, times a horizontal seismic coefficient. A seismic coefficient, equal to 2/3 the peak ground acceleration divided by the acceleration of gravity (g), is defined by USACE in EM 2100 to evaluate the potential for sliding.

Seismic coefficient method is used for general sizing of the structures. Reinforcement design and optimization are to be carried out using finite element modeling.

6.5.11 Ice Loading

For the purpose of analysis, an assumed pressure of 5,000 pounds per square foot applied to the contact surface of the dam.

Load Cases

6.6 The load cases used for the stability analysis were broken into three categories: Usual (U), Unusual (UN), and Extreme (E) and were taken from Table B-1 in EM 2200.

Table 6-2 Loading-Conditions Classification

Load Case	Loading Description	Classifications
1	Construction Condition	UN
2	Normal Operating	U
3	Infrequent Flood	UN
4	Construction with Operational Basis Earthquake (OBE*)	E
5	Coincident Pool with OBE	UN
6	Coincident Pool with Maximum Design Earthquake (MDE*)	E
7	Maximum Design Flood (MDF)	U/UN/E

* refer to Section 6.3.1

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

6.6.2 Loading Condition 2 is the normal operating condition which includes headwater at the normal pool elevation, the minimum tailwater corresponding with the above headwater, the uplift created by seepage, and the ice and silt pressure if applicable.

6.6.3 Loading Condition 3 is the infrequent flood condition which includes the pool at an elevation representing a flood event with a 300-year return period, minimum corresponding tailwater, the uplift created by seepage, and the ice and silt pressure if applicable.

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

6.6.5 Loading Condition 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.

6.6.6 Loading Condition 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.

6.6.7 Loading Condition 7 consists of the loads created by the MDF including the combination of pool and tailwater which produces the worst structural loading condition, with an unlimited return period, the uplift created by the seepage, silt pressure if applicable, but no ice pressure.

6.6.8 Load Cases Used for Design Analysis

Analysis Results

6.7

Conclusions

6.8

CONSTRUCTION MATERIAL

Concrete

6.9 All structural concrete shall meet the minimum requirements set below.

6.9.1 The concrete will have a 28-day compressive strength of 4000 psi.

6.9.2 The structural concrete max water content is 0.50.

6.9.3 The unit weight for concrete to be used in design is 150 lbs/ft^3.

Reinforcing Steel

6.10 All reinforcing steel shall meet the minimum requirements set below.

6.10.1 Reinforcing steel shall conform to ASTM A 615M, Grade 60.

6.10.2 Reinforcing development lengths and splices will be in accordance with EM 2104.

UNIT WEIGHTS

6.11 The appropriate unit weights and soil properties to be used in the structural design are given in Table 4.1. The unit weight water to be used in design is 62.4 lbs/ft^3.

7. MECHANICAL AND ELECTRICAL BASIS OF DESIGN

GENERAL

Reference Documents

7.1 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*
- EM 1110-2-2105, *Design of Hydraulic Steel Structures*
- ER 1110-2-1806, *Earthquake Design and Evaluation for Civil Works Projects*
- *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.]
- *Ground Motion Hazard Evaluation for Robles Diversion Dam Modification Project*, AMEC Geomatrix, Inc. Oakland, CA, November 12, 2008.

Engineer Manuals and Engineer Regulations are referred to in abbreviated form, i.e. EM 2702 and ER 1806, respectively in this report.

Loads

7.2 Following loads are applicable to spillway tainter gates *[*EM 2702, §3-4(b)*]*.

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

7.2.1 The hydrostatic loads (Hs) are calculated based the gate sill and the pool depths of the diversion dam's forebay. 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 – 757.75 feet [*Record Drawings: 767-D-232*] are utilized to determine the case loading.

Table 7.1 Hydrostatic Loads

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

7.2.2 The gravity loads (D, C, 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 skin plate, top of girders, and downstream face of girders. Ice thickness of ¼ inch was used in the load determination. Mud load was computed based on future silt loading from removal of the Matilija Dam (top of girders filled with silt).

7.2.3 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 lb/in, is equal to the rope tension force divided by the gate radius.

7.2.4 Since the inflow hydrographs showed that the reservoir does not sustain a WSEL sufficiently long to establish icing; collaborated by Casitas Municipal Water District (CMWD) staff, an Impact load (I) was assumed to be zero.

7.2.5 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. Coefficient of friction (μ_s) is taken as 0.5 for the rubber seals.

7.2.6 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, coefficient of friction is taken as 0.30.

7.2.7 The earthquake load was determined using the Operating Basis Earthquake (OBE) as defined in ER 1110-2-1806. This load includes the inertial hydrodynamic effects of the water

moving with the structure. EM 2702 §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 pcf
- OBE = 0.318g [geotechnical engineer]
- Pool depth = 12.00 feet [New crest elevation minus existing sill elevation]

7.2.8 Wave (W_A) loads are site specific. For this analysis wave height is taken as 0 ft. The probability of wind on a full reservoir is sufficiently low to rule out wave generation.

7.2.9 The Wind (W) load calculation is based on the site-specific conditions and in accordance with ASCE 7. The wind force input variables are shown below; all citations are to ASCE 7.

- 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

7.3 Load Combinations used in the design are as established by EM 2702 §3-4 b (2) and are tabulated below. The load combinations are numbered and provided with a brief description for use in the design.

Table 7.2 Load Combinations

Load Condition	Load Combination	EM 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 2 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 1 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

Where:

D = Selfweight
C = Ice load
M = Mud load
W = Wind load
W_A = Wave load
H_1 = Hydrostatic load (Maximum)
H_2 = Hydrostatic load (10-yr event)
H_3 = Hydrostatic load (1-yr 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

Note that in Table 7.2, 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 gate jammed condition, only load combination U10 is applicable.

Materials

7.4 For the existing gate structure analysis, the material specifications for the gates are provided in the original construction specifications, Section 79 (i). It states, "Tainter gate structural steel shall conform to Federal Specification QQ-S-741, type II, or ASTM Designation A7." Per this specification, minimum yield point strength for a welded structure, sections not

over 5/8 inch thick, is 33,000 psi. These values were utilized in the existing gate structure analysis.

7.4.1 For the proposed 30 ft x 12 ft gate, structural members shall consist of structural steel. Embedded metals, including the side and bottom seal plates should be corrosion-resistant steel. Table 7.3 provides material selection of various tainter gate components, including American Society for Testing and Materials (ASTM) standards, given normal conditions.

Table 7.3 Selected Materials

Component	Material Selection
Horizontal Girders	ASTM A992 Steel, Grade 50
End Girders & Built up Sections	ASTM A572 Steel, Grade 50
Downstream Vertical Ribs	ASTM A36 Steel
Strut Arms	ASTM A992 Steel, Grade 50
Strut Arm Bracing	ASTM A992 Steel, Grade 50
Skin Plate	ASTM A36 Steel
Stiffener Plates	ASTM A36 Steel
Lifting Bracket	ASTM A572 Steel, Grade 50
Seal Plates and Bolts	304 Stainless Steel
J-Seal Keeper Plates	410 Stainless Steel
Anchorage Steel	ASTM A772 Steel
Trunnion Bushing	ASTM B148 Aluminum Bronze
Trunnion Hub	ASTM A668 Steel Forging
Trunnion Pin	ASTM A705, Type 630, Condition H1150 Steel Forging

GATE ANALYSIS

Gate Analysis and Structural Modeling

7.5 SAP2000, Version 12 (Plus), was used for the structural modeling and analysis of the existing and new tainter gates. For the existing gate analysis an additional 2 foot extension was provided for the increased embankment elevation. The analysis was provided to confirm that the existing structure could accommodate the increased water surface elevation and associated loading.

7.5.1 A 3-D model of existing and new tainter gates was created using frame and shell elements of SAP2000. Figure 7.1 and 7.2 shows general layout of the tainter gates. The existing tainter gate width is 16 feet and the height is 11.5 feet (with 2 feet extension). The new tainter gate width is 30 feet and the height is 12 feet.

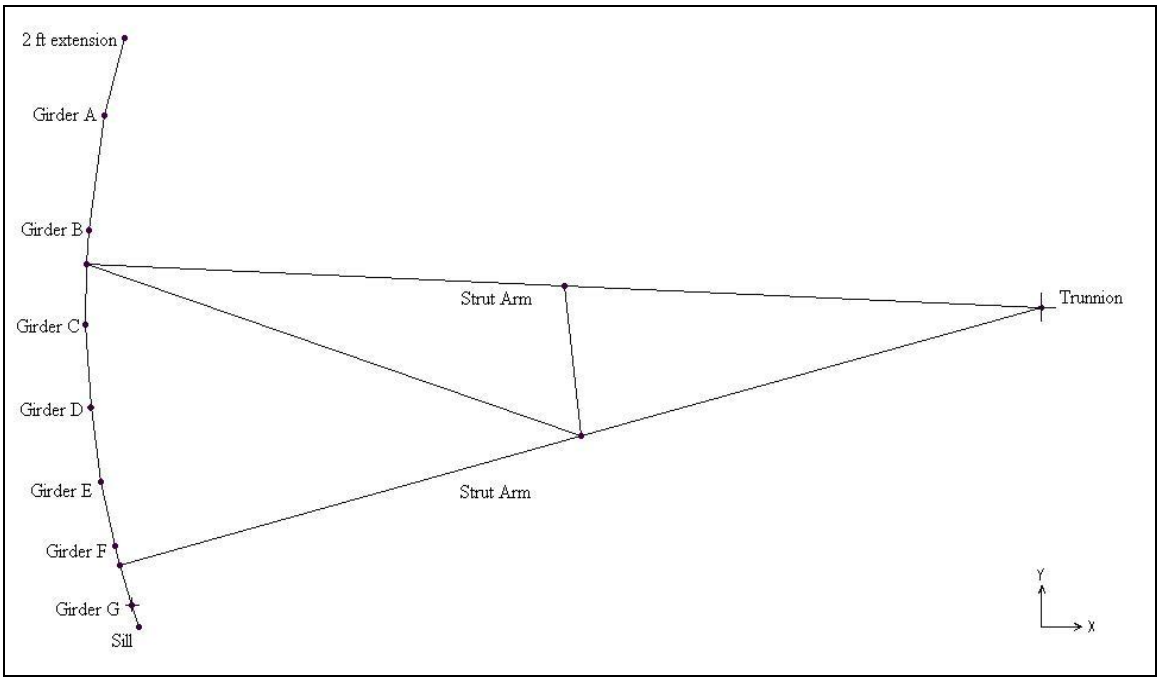

Figure 7.1 General Layout of Existing Tainter Gates (16ft x 11.5 ft)

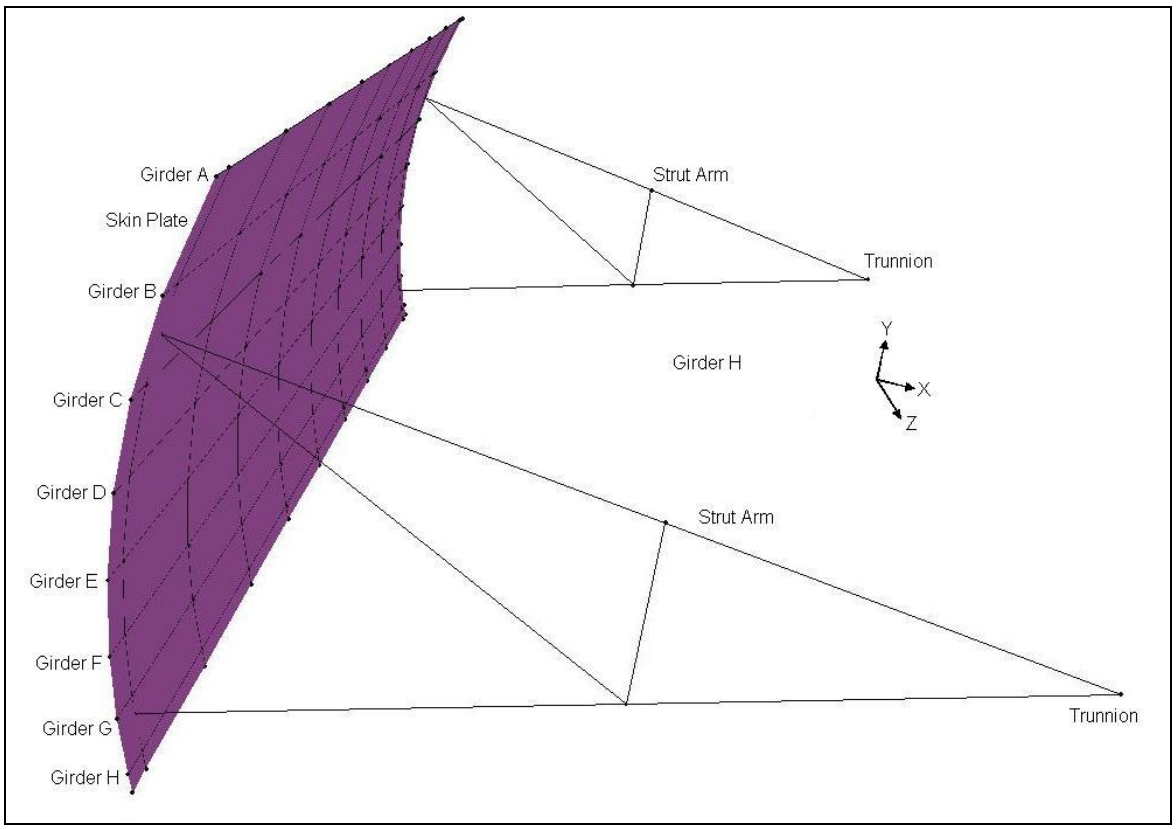

Figure 7.2 General Layout of new 30ft x 12ft Tainter Gates

Existing Gate Analysis Results

7.6 The analysis results for the trunnion reactions, frame members, and skin plate are presented in the following sections. See Appendix E for the SAP2000 analysis result plots.

7.6.1 The trunnion reactions for the load combinations described in Section 7.3 are presented in Table 7.4 below.

Table 7.4 Trunnion Reactions

Load Combinations No.	Fx at Each Trunnion (kip)	Fy at Each Trunnion (kip)
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

7.6.2 The frame members demand to capacity ratios (DCRs) were determined from SAP2000 finite element analysis, it was determined that the 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 DCR ratios for this load combination. Note that the code check module of SAP 2000 does not consider reliability factor α of 0.90 (Section 3-4, EM 2105) for computing DCRs. See Appendix E for detailed hand calculations and FEA plots.

Table 7.5 DCR Ratios

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

These Demand to Capacity Ratios are for the combined effects of axial force, minor axis flexure, and major axis flexure.

7.6.3 The skin plate shell stresses, for Load Combination 5, are shown in Figure 7.3 below. The maximum factored stress is 61 ksi; this is a small area of stress concentration over 3 nodes where lower strut arm connects to skin plate. This is an anomaly of finite element modeling;

discounting the stress concentration results in a maximum factored tensile stress of 24 ksi. The maximum factored compressive stress is 19 ksi. The factored skin plate capacity is 29.70 ksi.

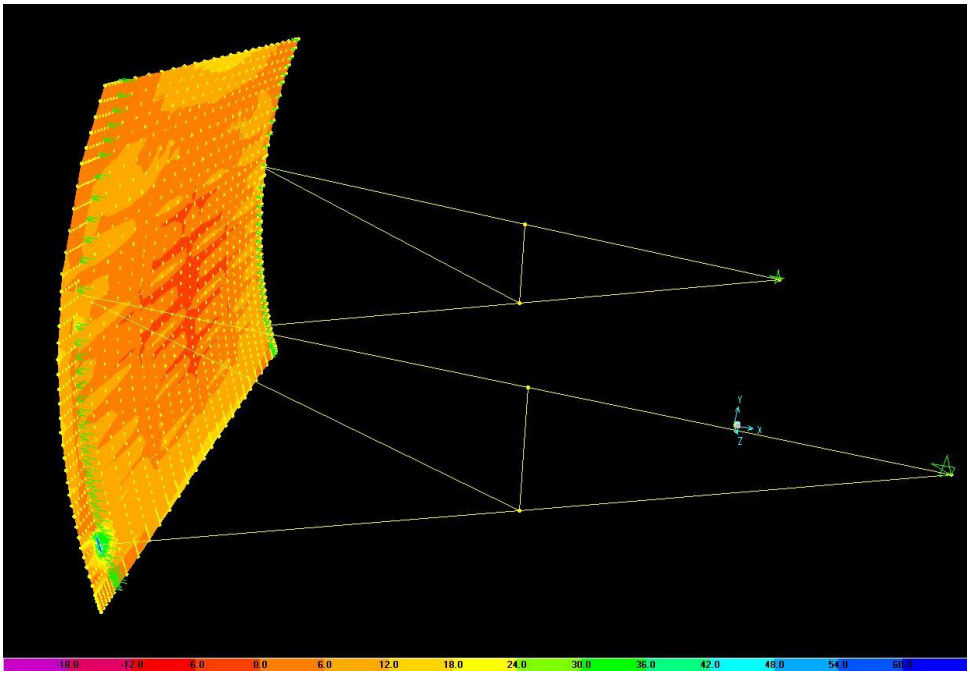

Figure 7.3 Existing Tainter Gate Skin Plate Shell Stresses

New 30ft x 12ft Tainter Gate Analysis Results

7.7 The analysis results for the trunnion reactions, frame members, and skin plate are presented in the following sections. See Appendix E for the SAP2000 analysis result plots.

7.7.1 The trunnion reactions for the load combinations described in Section 7.3 are presented in Table 7.6 below. These reactions are utilized in the structural design of the proposed spillway structure as described in section 7.5.

7.7.2 The frame members demand to capacity ratios (DCRs) were determined from the SAP2000 finite element analysis, it was determined that the 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 DCR ratios for this load combination. Note that the code check module of SAP 2000 does not consider reliability factor α of 0.90 (Section 3-4, EM 2105) for computing DCRs. See Appendix E for detailed hand calculations and FEA plots.

7.7.3 The Demand to Capacity Ratios are for the combined effects of axial force, minor axis flexure, and major axis flexure. DCR ratios mentioned in Table 7.7 are low because deflections are controlling the design of tainter gates.

Table 7.6 Trunnion Reactions

Load Combinations No.	Fx at Each Trunnion (kip)	Fy at Each Trunnion (kip)	Mz at Each Trunnion (kip-in)
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

Table 7.7 DCR Ratios

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	0.39
Lower Strut Arm	0.68

7.7.4 The frame member deflections in Table 7.8 show deflection of each girder for unfactored hydrostatic load from SAP model. For the design of 30 ft tainter gates, a limit for girder deflection is kept at span/360.

Table 7.8 Deflections

Location	Ux, Deflection (in)
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

7.7.5 The skin plate shell stresses, for Load Combination 5, are shown in Figure 7.4, below. The maximum factored stress is 36 ksi; this is a small area of stress concentration. Discounting the stress concentration results in a maximum factored tensile stress of 18 ksi. The maximum factored compressive stress is 22.50 ksi. The factored skin plate capacity is 32.40 ksi.

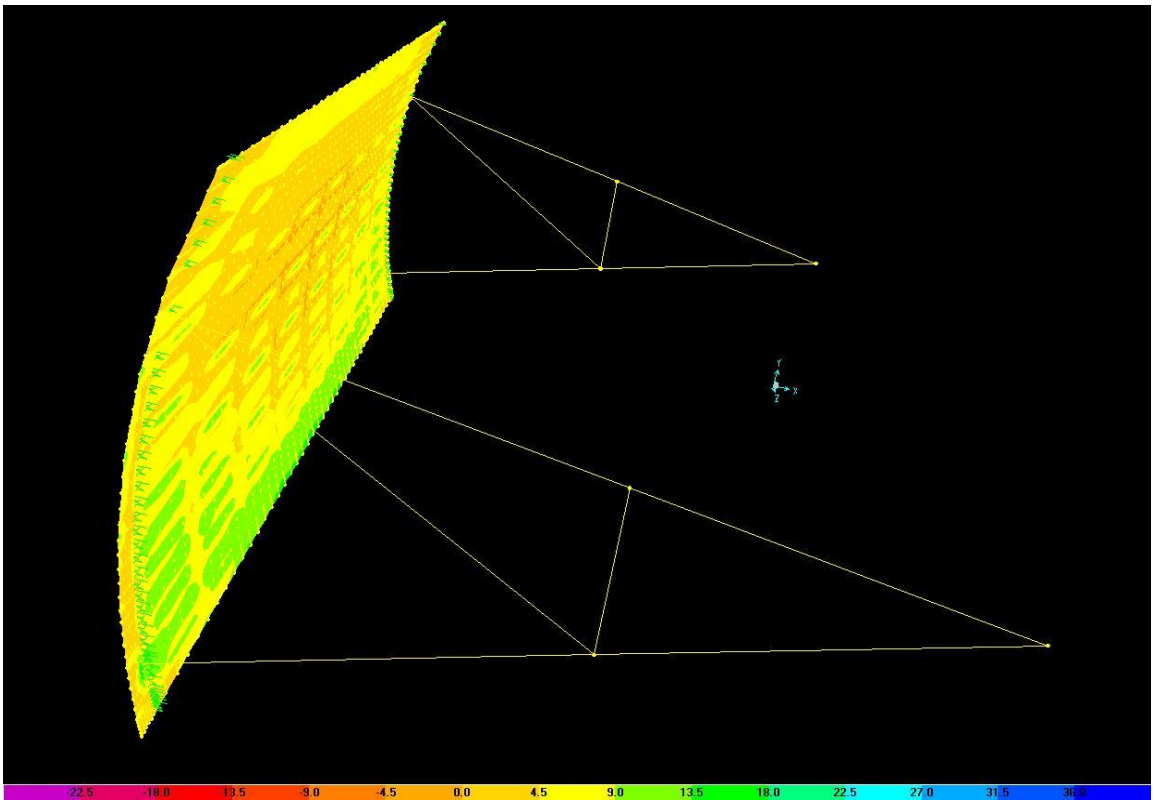

Figure 7.4 New 30 ft x 12 ft Tainter Gate Skin Plate Shell Stresses

MECHANICAL SYSTEMS

General

7.8 This section reports the analysis of existing tainter gate mechanical systems. Existing hoists and tainter gate trunnions were checked against current criteria as a part of mechanical systems.

Existing Hoist Analysis

7.9 Hoist motor, wire rope and drive shaft were checked against the CMAA 70 criteria. The following table represents the summary of the engineering computations for the mechanical components of the hoists. See Appendix E for detailed calculations.

Table 7.9 Hoist Analysis Summary

Component	Analysis Summary
Motor	For the rated capacity, required motor HP is 0.44. Available HP of existing motor is 1.5. Motor meets the criteria set by CMAA 70.
Wire Rope	Wire rope meets the CMAA 70 §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 §4.11.3 and 4.11.4.

Existing Trunnion Bearing

7.10 Existing tainter gate trunnions were checked for bearing pressure. Trunnions were checked against the allowable bearing pressure of 5000 psi [EM 2702, §4-3-b] for the unfactored load combination that produces maximum reaction at the trunnion (a combination of dead, mud, ice, hydrostatic and earthquake load). The calculations showed that the maximum induced bearing pressure at the trunnion is 2409 psi, which is well within the allowable limit of 5000 psi. See Appendix E for the detailed computations.

30 ft x 12 ft Tainter Gates Hoist System

7.11 The hoist system for 30 ft x 12 ft tainter gates was determined by analyzing the required rated capacity, lifting speed, total lift, and the wire rope pick point distance.

7.11.1 The required rated capacity of the hoist is 15 ton. It is computed by adding tainter gate dead load, mud load, ice load, side seal friction load, trunnion friction load and 25 percent overload.

7.11.2 The lifting speed of the tainter gate is 2 fpm. Therefore, hoist should be sized for the required output speed of 2 fpm.

7.11.3 Tainter gate hoist system should have minimum lift of 25 ft.

7.11.4 The required wire rope pick point distance is 343 in.

ELECTRICAL SYSTEMS

General

7.12 This section reports the selection of the modifications to the electrical distribution and control system in order to support the new tainter gates.

Existing Electrical Distribution System

7.13 The existing electrical distribution system consists of a 200 amp, 240/120 Volt, 3-phase, 4 wire electrical service. The service conductors terminate in a service entrance rated disconnect

switch. The electrical service is backed up by a 60kW standby generator via 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 sub distribution panels and a control starter panel. Two sub-distribution panels are 240/120V 1-phase, 3 wire, and the third panel and the starter control panel are fed via 240/120V 3-phase, 4 wire.

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

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

Proposed Electrical Distribution System

7.14 Proposed is a new 200 amp, 480Y/277V, 3-phase, 4 wire electrical service, including a new 200 amp service entrance rated disconnect switch, a new 200 amp automatic transfer switch, a new 200 amp distribution panel board, and a new 75kVA 480:240/120V step down transformer.

7.14.1 New gate motors are assumed to be selected in order to function at 480V, 3-phase.

7.14.2 Gate motors shall be fed via individual power feeders to VFD control panel for each motor. VFD control panel shall be located in the existing control building with a local disconnect at the motor location.

7.14.3 It is assumed there is sufficient space in the existing Control Building to house the new electrical distribution equipment. Where feasible, it is anticipated that the new electrical equipment shall be installed in existing available space prior to removal of the existing, thereby minimizing system downtime.

Existing Controls System

7.15 The existing control system is anticipated to be retained and reused. New controls and control panel shall integrate with the existing control system.

Proposed Controls System

7.16 While there appears to be sufficient space in the existing controls rack to accommodate the new gate control hardware, the activation buttons, annunciators, and display readouts may not fit on the existing control cabinet exterior, therefore it is anticipated that a new control panel shall be provided. New digital input units are required in the existing control rack to control the new gates.

7.16.1 New gate motors shall be operated via new across-the-line starters or VFD controllers, located in the control building. Controller selection would be based on how the standby generator system is configured, and to what extent the motor starting would affect the other facility systems. There is an existing uninterruptible power supply in the existing control cabinet which would aid in minimizing voltage dip effects on the PLC control system during motor starting.

7.16.2 Local on/off control shall be located adjacent to the gate motor. Control wiring shall be routed back to new control panel in existing control building via surface mounted conduit.

Existing Standby Generator System

7.17 The existing generator is a 60kW / 75kVA, 240/120V, 3-phase, 4 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

7.18 There are three options with respect to 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/277V, 3 phase, 4 wire. The existing generator would still only provide 60kW of standby power, therefore interlocks would be required that would prevent all systems from operating on the generator concurrently, but with selectability (such as a manual transfer switch between new gate motors and existing system) such that at any given time, a predetermined block of equipment can 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.

7.18.1 The least expensive option is Option 1, where the existing generator is left in place, but the new gates would not be on generator power and would not be operable upon loss of utility power. If that is operationally unacceptable, Options 2 or Option 3 would be the alternates to consider.

Site Lighting

7.19 New lighting pole standards shall be provided as required to meet use requirements at the gates while minimizing light pollution and light spillage to the surrounding areas. Control of new lighting shall be designed to provide light levels required for security purposes as well as maintenance/repair work while minimizing energy consumption.

CONCLUSION

Existing Gate Analysis

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

New 30ft x 12ft Tainter Gate Analysis

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

8. CARE OF HABITAT DURING CONSTRUCTION

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

9. CARE AND DIVERSION OF WATER DURING CONSTRUCTION

9.1 Surface flows within the construction area will be controlled by dikes, diversion pipes and pumps. The spillway, embankment, and rock ramp excavation could be conducted throughout the year, but during the rainy seasons weather should be monitored for storm activity and protection of the work site from storm flows provided by the contractor. Groundwater should not be encountered at the proposed construction depths outside of the rainy season.

10. DISPOSAL OF MATERIALS

10.1 There are at least two optional disposal sites for the excess soils to be excavated from 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 ramp. Additionally, at the time of construction the commercial ability for the excess soil to be sold will be reviewed. This project cost estimate assumes that the material will be disposed of onsite. It is expected that a majority of the excess material will be suitable for commercial use and will possibly be disposed in other locations other than onsite.

10.2 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.

11. ENVIRONMENTAL ASSESSMENT

11.1 A Supplemental Environmental Assessment for the proposed improvements is currently being prepared by the District. A copy of the findings will be included in Appendix X and incorporated in the contract drawings.

12. COST ESTIMATES

15.1 The cost estimate of the project will be determined at a later date. Actual unit prices will be set by the bid of the winning contractor and all quantity amounts should be reviewed and verified by the contractor prior to placing a bid.

13. RECOMMENDATIONS

20.1 This report describes in detail the general design, including departures from the previously approved plan, of the portion of the Matilija Dam Ecosystem Restoration Project. It is recommended that this report provide 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.

20.1.1 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.